

ДГКС

Друштво грађевинских
конструктора Србије



ASES

Association of Structural
Engineers of Serbia

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PRVO SAVETOVANJE
JUGOSLOVENSKIH INŽENJERA KONSTRUKTERA
ZAGREB, decembar 1953.

DRUGI KONGRES
JUGOSLOVENSKOG DRUŠTVA GRAĐEVINSKIH KONSTRUKTERA
OPATIJA, maj 1958.

Predsednik: akademik, prof. dr h.c. Đorđe LAZAREVIĆ, dipl.inž.građ., Beograd
Sekretar: prof. Zvonimir PAVLOVIĆ, dipl.inž.građ., Beograd

TREĆI KONGRES
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SARAJEVO, septembar 1964.

Predsednik: prof. dr Kruno TONKOVIĆ, dipl.inž.građ., Zagreb
Potpredsednik: prof. Ljubomir JEVIĆ, dipl.inž.građ., Beograd
Sekretar: prof. Zvonimir PAVLOVIĆ, dipl.inž.građ., Beograd

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JUGOSLOVENSKOG DRUŠTVA GRAĐEVINSKIH KONSTRUKTERA
PORTOROŽ, jun 1969.

Predsednik: akademik, prof. dr Milan ĐURIĆ, dipl.inž.građ., Beograd
Potpredsednik: prof. Miloš MARINČEK, dipl.inž.građ., Ljubljana
Sekretar: Stojan DINIĆ, dipl.inž.građ., Beograd

PETI KONGRES
JUGOSLOVENSKOG DRUŠTVA GRAĐEVINSKIH KONSTRUKTERA
BUDVA, septembar 1974.

Predsednik: akademik, prof. dr Milan ĐURIĆ, dipl.inž.građ., Beograd
Potpredsednik: prof. Miloš MARINČEK, dipl.inž.građ., Ljubljana
Sekretar: Stojan DINIĆ, dipl.inž.građ., Beograd

SIMPOZIJUM

JUGOSLOVENSKOG DRUŠTVA GRAĐEVINSKIH KONSTRUKTERA
"GREŠKE, SANACIJE I ISKUSTVA U INVESTIRANJU, PROJEKTOVANJU,
GRAĐENJU I EKSPLOATACIJI GRAĐEVINSKIH OBJEKATA"
CAVTAT, novembar 1976.

ŠESTI KONGRES

JUGOSLOVENSKOG DRUŠTVA GRAĐEVINSKIH KONSTRUKTERA
BLED, septembar 1978.

Predsednik: akademik, prof. dr Nikola HAJDIN, dipl.inž.građ., Beograd
Potpredsednik: prof. Milenko PRŽULJ, dipl.inž.građ., Sarajevo
Sekretar: Stojan DINIĆ, dipl.inž.građ., Beograd
Sekretar za
međunarodne veze: prof. dr Života PERIŠIĆ, dipl.inž.građ., Beograd

SIMPOZIJUM

JUGOSLOVENSKOG DRUŠTVA GRAĐEVINSKIH KONSTRUKTERA I
INSTITUTA ZA MATERIJALE I KONSTRUKCIJE GRAĐEVINSKOG FAKULTETA
UNIVERZITETA U BEOGRADU
"INOVIACIJE JUGOSLOVENSKIH PROPISA ZA BETONSKE, METALNE I
SPREGNUTE KONSTRUKCIJE - ISKUSTVA I NOVA SAZNANJA"
TROGIR, maj 1980.

Na Skupštini održanoj maja 1980. u Trogiru, Jugoslovensko društvo građevinskih konstruktora je promenilo naziv u SAVEZ DRUŠTAVA GRAĐEVINSKIH KONSTRUKTERA JUGOSLAVIJE - SDGKJ. Na Skupštini je odlučeno da izvršni odbor, izabran na VI Kongresu na Bledu, u istom sastavu obavlja dužnosti PREDSEDNIŠTVA SDGKJ do sledećeg kongresa.

Za predsednika i potpredsednika SDGKJ sa jednogodišnjim mandatom izabrani su:

Predsednik: akademik, prof. dr Nikola HAJDIN, dipl.inž.građ., Beograd
Potpredsednik: prof. Milenko PRŽULJ, dipl.inž.građ., Sarajevo

a za predsednika i potpredsednika SDGKJ za naredni jednogodišnji mandat:

Predsednik: prof. Milenko PRŽULJ, dipl.inž.građ., Sarajevo
Potpredsednik: Marijan KOŠČAK, dipl.inž.građ., Zagreb

**SEDMI KONGRES
SAVEZA DRUŠTAVA GRAĐEVINSKIH KONSTRUKTERA JUGOSLAVIJE
CAVTAT, april 1983.**

Predsednik: prof. Milenko PRŽULJ, dipl.inž.građ., Sarajevo
Potpredsednik: prof. dr Stanko ŠRAM, dipl.inž.građ., Zagreb
Sekretar: Stojan DINIĆ, dipl.inž.građ., Beograd
Sekretar za
međunarodne veze: prof. dr Života PERIŠIĆ, dipl.inž.građ., Beograd

**SIMPOZIJUM '85
SAVEZA DRUŠTAVA GRAĐEVINSKIH KONSTRUKTERA JUGOSLAVIJE
DUBROVNIK, april 1985.**

Na sednici Predsedništva SDGKJ, održanoj aprila 1985. u Dubrovniku odlučeno je da se predsedniku, potpredsedniku, sekretaru i sekretaru za međunarodne veze, izabranim na VII Kongresu SDGKJ u Cavtatu, produži mandat do sledećeg kongresa.

**OSMI KONGRES
SAVEZA DRUŠTAVA GRAĐEVINSKIH KONSTRUKTERA JUGOSLAVIJE
CAVTAT, april 1987.**

Predsednik: prof. dr Života PERIŠIĆ, dipl.inž.građ., Beograd
Potpredsednik: prof. dr Franci KRŽIĆ, dipl.inž.građ., Ljubljana
Sekretar: Stojan DINIĆ, dipl.inž.građ., Beograd
Sekretar za
međunarodne veze: docent dr Jure RADIĆ, dipl.inž.građ., Zagreb

**JUGOSLOVENSKO SAVETOVANJE O PRIMENI NOVOG PRAVILNIKA ZA
BETON I ARMIRANI BETON BAB 87 ZAJEDNO SA JUDIMK, GRAĐEVINSKIM
INSTITUTOM ZAGREB I GRAĐEVINSKIM FAKULTETOM BEOGRAD
DUBROVNIK, april 1989.**

Na sednici Predsedništva SDGKJ održanoj aprila 1989. u Dubrovniku, ponovo su izabrani za sledeći dvogodišnji period

Predsednik: prof. dr Života PERIŠIĆ, dipl.inž.građ., Beograd
Potpredsednik: prof. dr Franci KRŽIĆ, dipl.inž.građ., Ljubljana
Sekretar: Stojan DINIĆ, dipl.inž.građ., Beograd
Sekretar za
međunarodne veze: docent dr Jure RADIĆ, dipl.inž.građ., Zagreb

**DEVETI KONGRES
SAVEZA DRUŠTAVA GRAĐEVINSKIH KONSTRUKTERA JUGOSLAVIJE
CAVTAT, april 1991.**

Predsednik: prof. dr Života PERIŠIĆ, dipl.inž.građ., Beograd
Potpredsednik: prof. dr Franci KRŽIĆ, dipl.inž.građ., Ljubljana
Sekretar: Stojan DINIĆ, dipl.inž.građ., Beograd
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*Posle raspada SFRJ, Savez društava građevinskih konstruktera Jugoslavije - SDGKJ transformisao se u **Jugoslovensko društvo građevinskih konstruktera - JDGK**, koje je nastavilo delatnost SDGKJ. Na Osnivačkoj skupštini, održanoj 1.12.1994. godine u Beogradu, usvojen je novi statut i imenovano predsedništvo u sastavu:*

Predsednik: prof. dr Života PERIŠIĆ, dipl.inž.građ., Beograd
Potpredsednik: prof. dr Radimir FOLIĆ, dipl.inž.građ., Novi Sad
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Vidoje ZELENOVIĆ, dipl.inž.građ., Priština
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prof. dr Aleksandar PAKVOR, dipl.inž.građ., Beograd
prof. dr Mihailo MURAVLJOV, dipl.inž.građ., Beograd

Jugoslovensko društvo građevinskih konstruktora, zajedno sa Građevinskim fakultetom u Beogradu i Saveznim zavodom za standardizaciju, organizovalo je dva jugoslovenska savetovanja:

PRVO JUGOSLOVENSKO SAVETOVANJE
EVROKODOVI I JUGOSLOVENSKO GRAĐEVINSKO KONSTRUKTERSTVO
BEOGRAD, jun 1995.

DRUGO JUGOSLOVENSKO SAVETOVANJE
EVROKODOVI I JUGOSLOVENSKO GRAĐEVINSKO KONSTRUKTERSTVO
BEOGRAD, april 1997.

DESETI KONGRES
JUGOSLOVENSKOG DRUŠTVA GRAĐEVINSKIH KONSTRUKTERA
VRNJAČKA BANJA, jun 1998.

Na Skupštini JDGK, održanoj juna 1998. u Vrnjačkoj Banji, izabrano je novo predsedništvo i nadzorni odbor JDGK, u sledećem sastavu:

Predsednik:	prof. dr Mirko AČIĆ, dipl.inž.građ., Beograd
Potpredsednik:	prof. dr Radenko PEJOVIĆ, dipl.inž.građ., Podgorica
Generalni sekretar:	Stojan DINIĆ, dipl.inž.građ., Beograd
Sekretar:	Svetislav SIMOVIĆ, dipl.inž.građ., Beograd
Članovi predsedništva:	akademik prof. dr Nikola HAJDIN, dipl.inž.građ., Beograd prof. dr Milorad IVKOVIĆ, dipl.inž.građ., Beograd prof. dr Života PERIŠIĆ, dipl.inž.građ., Beograd prof. dr Aleksandar PAKVOR, dipl.inž.građ., Beograd prof. dr Miodrag SEKULOVIĆ, dipl.inž.građ., Beograd prof. dr Arsenije VUJOVIĆ, dipl.inž.građ., Podgorica prof. dr Radimir FOLIĆ, dipl.inž.građ., Novi Sad prof. dr Živojin PRAŠČEVIĆ, dipl.inž.građ., Beograd prof. dr Dragan BUĐEVAC, dipl.inž.građ., Beograd prof. dr Milić MILIĆEVIĆ, dipl.inž.građ., Niš mr Vidoje ZELENOVIĆ, dipl.inž.građ., Gacko Veljko BELADA, dipl.inž.građ., Podgorica Ivan MAMUŽIĆ, dipl.inž.građ., Novi Sad Dragan VUKADINOVIĆ, dipl.inž.građ., Beograd Jožef KERMECI, dipl.inž.građ., Kanjiža
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Na istoj skupštini izabrani su za počasne članove JDGK:

akademik prof. dr Nikola HAJDIN, dipl.inž.građ., Beograd
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prof. dr Tihomir NIKOLOVSKI, dipl.inž.građ., Skoplje

takođe je izabrano i 14 zaslužnih članova JDGK:

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dr Vladimir STANKOVIĆ, dipl.inž.građ., Podgorica
Veljko BELADA, dipl.inž.građ., Podgorica

SIMPOZIJUM 2000.

JUGOSLOVENSKOG DRUŠTVA GRAĐEVINSKIH KONSTRUKTERA
VRNJAČKA BANJA, 1-3. novembar 2000.

Na Skupštini JDGK, održanoj 2. novembra 2000. u Vrnjačkoj Banji, predsedništvo JDGK je prošireno sa dva člana i njegov sastav čine:

Predsednik:	prof. dr Mirko AČIĆ, dipl.inž.građ., Beograd
Potpredsednik:	prof. dr Radenko PEJOVIĆ, dipl.inž.građ., Podgorica
Generalni sekretar:	Stojan DINIĆ, dipl.inž.građ., Beograd
Sekretar:	Svetislav SIMOVIĆ, dipl.inž.građ., Beograd
Članovi predsedništva:	akademik, prof. dr Nikola HAJDIN, dipl.inž.građ., Beograd prof. dr Milorad IVKOVIĆ, dipl.inž.građ., Beograd prof. dr Života PERIŠIĆ, dipl.inž.građ., Beograd prof. dr Aleksandar PAKVOR, dipl.inž.građ., Beograd prof. dr Miodrag SEKULOVIĆ, dipl.inž.građ., Beograd prof. dr Arsenije VUJOVIĆ, dipl.inž.građ., Podgorica prof. dr Radomir FOLIĆ, dipl.inž.građ., Novi Sad prof. dr Živojin PRAŠČEVIĆ, dipl.inž.građ., Beograd prof. dr Dragan BUĐEVAC, dipl.inž.građ., Beograd

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mr Vidoje ZELENOVIĆ, dipl.inž.građ., Banja Luka
Veljko BELADA, dipl.inž.građ., Podgorica
Ivan MAMUŽIĆ, dipl.inž.građ., Novi Sad
Dragan VUKADINOVIĆ, dipl.inž.građ., Beograd
Jožef KERMECI, dipl.inž.građ., Kanjiža

Članovi
Nadzornog
odбора:

prof. dr Dušan NAJDANOVIĆ, dipl.inž.građ., Beograd - predsednik
prof. dr Mladen ULIĆEVIĆ, dipl.inž.građ., Podgorica
Miroslav BAJIĆ, dipl.inž.građ., Novi Sad

Na istoj skupštini izabrani su za počasne članove JDGK:

prof. dr Sande ATANASOVSKI, dipl.inž.građ., Skoplje
prof. dr Dragan IVANOV, dipl.inž.građ., Skoplje
prof. dr Milenko PRŽULJ, dipl.inž.građ., Ljubljana
prof. dr Vukašin AČANSKI, dipl.inž.građ., Maribor
prof. dr Života PERIŠIĆ, dipl.inž.građ., Beograd
Stojan DINIĆ, dipl.inž.građ., Beograd

takođe je izabrano i 10 zaslužnih članova JDGK:

prof. Dimitrije ĆERTIĆ, dipl.inž.građ., Beograd
Predrag ŽELALIĆ, dipl.inž.građ., Beograd
prof. Gojko NENADIĆ, dipl.inž.građ., Beograd
Anđelko KOVAČEVIĆ, dipl.inž.građ., Beograd
prof. dr Živojin PRAŠČEVIĆ, dipl.inž.građ., Beograd
Miroslav BAJIĆ, dipl.inž.građ., Beograd
prof. dr Mihailo MURAVLJOV, dipl.inž.građ., Beograd
prof. dr Arsenije VUJOVIĆ, dipl.inž.građ., Podgorica
Lazar PRODANOVIĆ, dipl.inž.građ., Beograd
Johan SKLENA, dipl.inž.građ., Novi Sad

**JEDANAESTI KONGRES
JUGOSLOVENSKOG DRUŠTVA GRAĐEVINSKIH KONSTRUKTERA
VRNJAČKA BANJA, 25-27. septembar 2002.**

*Na Skupštini JDGK, održanoj 24. septembra 2002. u Vrnjačkoj Banji, izabrano je novo
predsedništvo i nadzorni odbor JDGK, u sledećem sastavu:*

Predsednik: prof. dr Mirko AĆIĆ, dipl.inž.građ., Beograd
Potpredsednik: prof. dr Radenko PEJOVIĆ, dipl.inž.građ., Podgorica

Generalni sekretar: Stojan DINIĆ, dipl.inž.građ., Beograd
Sekretar: Svetislav SIMOVIĆ, dipl.inž.građ., Beograd
Članovi
predsedništva: prof. dr Života PERIŠIĆ, dipl.inž.građ., Beograd
prof. dr Radomir FOLIĆ, dipl.inž.građ., Novi Sad
prof. dr Arsenije VUJOVIĆ, dipl.inž.građ., Podgorica
prof. dr Živojin PRAŠČEVIĆ, dipl.inž.građ., Beograd
prof. dr Dragan BUĐEVAC, dipl.inž.građ., Beograd
prof. dr Dušan NAJDANOVIĆ, dipl.inž.građ., Beograd
prof. dr Đorđe VUKSANOVIĆ, dipl.inž.građ., Beograd
prof. dr Mladen ULIĆEVIĆ, dipl.inž.građ., Podgorica
prof. dr Dragoslav STOJIC, dipl.inž.građ., Niš
mr Vidoje ZELENOVIĆ, dipl.inž.građ., Gacko
Dragan VUKADINOVIĆ, dipl.inž.građ., Beograd
Ivan MAMUŽIĆ, dipl.inž.građ., Novi Sad
Jožef KERMECI, dipl.inž.građ., Kanjiža

Članovi
Nadzornog
odбора: mr Dragana ČUKIĆ, dipl.inž.građ., Beograd
mr Đorđe LAĐINOVIĆ, dipl.inž.građ., Novi Sad
Igor ĐURANOVIĆ, dipl.inž.građ., Podgorica

Na istoj skupštini izabrani su za počasne članove JDGK:

prof. dr Stanko ŠRAM, dipl.inž.građ. - Hrvatska
prof. dr Miodrag VELKOV, dipl.inž.građ. - Makedonija
prof. dr Aleksandar PAKVOR, dipl.inž.građ., Beograd
prof. dr Miodrag SEKULOVIĆ, dipl.inž.građ., Beograd
Predrag ŽELALIĆ, dipl.inž.građ., Beograd
Veljko BELADA, dipl.inž.građ., Podgorica
mr Vladimir STANKOVIĆ, dipl.inž.građ., Podgorica

takođe je izabrano i 12 zaslužnih članova JDGK:

prof. dr Dušan NAJDANOVIĆ, dipl.inž.građ., Beograd
prof. dr Radenko PEJOVIĆ, dipl.inž.građ., Podgorica
prof. dr Mladen ULIĆEVIĆ, dipl.inž.građ., Podgorica
prof. dr Petar ČOLIĆ, dipl.inž.građ., Beograd
Dušan SIMOVIĆ, dipl.inž.građ., Beograd
Svetislav SIMOVIĆ, dipl.inž.građ., Beograd
Dragan VUKADINOVIĆ, dipl.inž.građ., Beograd
Boško TRIVIĆ, dipl.inž.građ., Beograd
Jožef KERMECI, dipl.inž.građ., Kanjiža
Vukan NJAGULJ, dipl.inž.građ., Beograd
Dragomir LUKIĆ, dipl.inž.građ., Beograd
Miroslav SUBOTIĆ, dipl.inž.građ., Beograd

SIMPOZIJUM 2004.

JUGOSLOVENSKOG DRUŠTVA GRAĐEVINSKIH KONSTRUKTERA
VRNJAČKA BANJA, 29. septembar - 1. oktobar 2004.

Predsednik: prof. dr Mirko AĆIĆ, dipl.inž.građ., Beograd
Potpredsednik: prof. dr Radenko PEJOVIĆ, dipl.inž.građ., Podgorica
Generalni sekretar: Stojan DINIĆ, dipl.inž.građ., Beograd
Sekretar: Svetislav SIMOVIĆ, dipl.inž.građ., Beograd

Članovi

predsedništva: prof. dr Života PERIŠIĆ, dipl.inž.građ., Beograd
prof. dr Radomir FOLIĆ, dipl.inž.građ., Novi Sad
prof. dr Arsenije VUJOVIĆ, dipl.inž.građ., Podgorica
prof. dr Živojin PRAŠČEVIĆ, dipl.inž.građ., Beograd
prof. dr Dragan BUĐEVAC, dipl.inž.građ., Beograd
prof. dr Dušan NAJDANOVIĆ, dipl.inž.građ., Beograd
prof. dr Đorđe VUKSANOVIĆ, dipl.inž.građ., Beograd
prof. dr Mladen ULIĆEVIĆ, dipl.inž.građ., Podgorica
prof. dr Dragoslav STOJIĆ, dipl.inž.građ., Niš
prof. dr Petar ČOLIĆ, dipl.inž.građ., Beograd
mr Vidoje ZELENOVIĆ, dipl.inž.građ., Gacko
Dragan VUKADINOVIĆ, dipl.inž.građ., Beograd
Ivan MAMUŽIĆ, dipl.inž.građ., Novi Sad
Jožef KERMECI, dipl.inž.građ., Kanjiža

Članovi

Nadzornog odbora: mr Dragana ČUKIĆ, dipl.inž.građ., Beograd
mr Đorđe LAĐINOVIĆ, dipl.inž.građ., Novi Sad
Igor ĐURANOVIĆ, dipl.inž.građ., Podgorica

Na istoj skupštini izabrani su za počasne članove JDGK:

prof. dr Predrag GAVRILOVIĆ, dipl.inž.građ.
Saško STREZOVSKI, dipl.inž.građ.
prof. Gojko NENADIĆ, dipl.inž.građ.
prof. dr Mihajlo TRIFUNAC, dipl.inž.građ.
prof. dr Milija PAVLOVIĆ, dipl.inž.građ.
prof. dr David LLOJD SMITH, dipl.inž.građ.
prof. dr. Arsenije VUJOVIĆ, dipl.inž.građ.
prof. dr Vlado GOCEVSKI, dipl.inž.građ.
prof. dr Tomislav RADOJIĆIĆ, dipl.inž.građ.
Ivan MAMUŽIĆ, dipl.inž.građ.

takođe je izabrano i 12 zaslužnih članova JDGK:

Jovo SMILJANIĆ, dipl.inž.građ.
Dragan MAJKIĆ, dipl.inž.građ.
Lajčo STIPIĆ, dipl.inž.građ.
prof. dr Milivoje STANKOVIĆ, dipl.inž.građ.
prof. dr Dragoljub DRENIĆ, dipl.inž.građ.
prof. dr Dušan PETKOVIĆ, dipl.inž.građ.
prof. dr Dragan BUDEVAC, dipl.inž.građ.
mr Dragana ČUKIĆ, dipl.inž.građ.
mr Milan ĐOKOVIĆ, dipl.inž.građ.
prof. dr Božidar S. PAVIĆEVIĆ, dipl.inž.građ.
mr Radivoje MRDAK, dipl.inž.građ.
prof. dr Vojislav MIHAILOVIĆ, dipl.inž.građ.

**DVANAESTI KONGRES
JUGOSLOVENSKOG DRUŠTVA GRAĐEVINSKIH KONSTRUKTERA
VRNJAČKA BANJA, 27-29. septembar 2006.**

Na Skupštini JDGK, održanoj 27. septembra 2006. u Vrnjačkoj Banji, izabrano je novo predsedništvo i nadzorni odbor JDGK, u sledećem sastavu:

Predsednik:	prof. dr Dejan BAJIĆ, dipl.inž.građ., Beograd
Potpredsednik:	Svetislav SIMOVIĆ, dipl.inž.građ., Podgorica
Sekretar:	prof. dr Snežana MARINKOVIĆ, dipl.inž.građ., Beograd
Članovi predsedništva:	prof. dr Dušan NAJDANOVIĆ, dipl.inž.građ., Beograd prof. dr Đorđe VUKSANOVIĆ, dipl.inž.građ., Beograd prof. dr Dragoslav STOJIĆ, dipl.inž.građ., Niš prof. dr Radomir FOLIĆ, dipl.inž.građ., Novi Sad doc. dr Đorđe LAĐINOVIĆ, dipl.inž.građ., Novi Sad doc. dr Bratislav STIPANIĆ, dipl.inž.građ., Beograd doc. dr Aleksandar RISTOVSKI, dipl.inž.građ., Kosovska Mitrovica dr Zoran FLORIĆ, dipl.inž.građ., Novi Sad Ivan MAMUŽIĆ, dipl.inž.građ., Novi Sad Zoran FILIPOVIĆ, dipl.inž.građ., Beograd Branko KNEŽEVIĆ, dipl.inž.građ., Beograd Slobodan CVETKOVIĆ, dipl.inž.građ., Beograd Slobodan MITROVIĆ, dipl.inž.građ., Beograd
Članovi Nadzornog odbora:	mr Slobodan GRKOVIĆ, dipl.inž.građ., Subotica Gojko GRBIĆ, dipl.inž.građ., Loznica Miroslav MIHAJLOVIĆ, dipl.inž.građ., Paraćin

Na istoj skupštini izabrani su za počasne članove JDGK:

:

Prof. dr Mirka AČIĆA, dipl.građ.inž.
Prof. dr Radomira FOLIĆA, dipl.građ.inž.
Prof. dr Radenka PEJOVIĆA, dipl.građ.inž.
Prof. dr Trifuna PASKALOVA, dipl.građ.inž.
Mr Vidoja ZELENOVIĆA, dipl.građ.inž.

takođe je izabrano i 5 zaslužnih članova JDGK:

Prof. dr Đorđe VUKSANOVIĆ, dipl.građ.inž.
Prof. dr Dejan BAJIĆ, dipl.građ.inž.
Prof. dr Dragoslav STOJIC, dipl.građ.inž.
Jožef BARNA, dipl.građ.inž.
Mr Predrag PAVLOVIĆ, dipl.građ.inž.

SIMPOZIJUM 2008.

DRUŠTVA GRAĐEVINSKIH KONSTRUKTERA SRBIJE
ZLATIBOR "ČIGOTA", 24-26. septembar 2008.

Predsednik: prof. dr Dejan BAJIĆ, dipl.inž.građ., Beograd
Potpredsednik: Svetislav SIMOVIĆ, dipl.inž.građ., Podgorica
Sekretar: prof. dr Snežana MARINKOVIĆ, dipl.inž.građ., Beograd

Nije bilo izbora za zaslužne i počasne članove društva.

TRINAESTI KONGRES

DRUŠTVA GRAĐEVINSKIH KONSTRUKTERA SRBIJE
ZLATIBOR "ČIGOTA", 22-24. septembar 2010.

Na Skupštini DGKS, održanoj 22. septembra 2010. na Zlatiboru izabrano je novo predsedništvo i nadzorni odbor DGKS, u sledećem sastavu:

Predsednik: prof. dr Miloš LAZOVIĆ, dipl.inž.građ., Beograd
Potpredsednik: Aleksandar BOJOVIĆ, dipl.inž.građ., Beograd
Sekretar: prof. dr Boško STEVANOVIĆ, dipl.inž.građ., Beograd
Članovi
predsedništva: prof. dr Snežana MARINKOVIĆ, dipl.inž.građ., Beograd
prof. dr Đorđe VUKSANOVIĆ, dipl.inž.građ., Beograd
prof. dr Mihajlo ĐURĐEVIĆ, dipl.inž.građ., Beograd
prof. dr Đorđe LAĐINOVIĆ, dipl.inž.građ., Novi Sad
doc. dr Bratislav STIPANIĆ, dipl.inž.građ., Beograd
prof. dr Dragoslav STOJIC, dipl.inž.građ., Niš
mr Slobodan GRKOVIĆ, dipl.inž.građ., Subotica

prof. dr Aleksandar RISTOVSKI, dipl.inž.građ., Kos.Mitrovica
Gojko GRBIĆ, dipl.inž.građ., Loznica
dr Zoran FLORIĆ, dipl.inž.građ., Novi Sad
Đorđe PAVKOV, dipl.inž.građ., Novi Sad
Svetislav SIMOVIĆ, dipl.inž.građ., Beograd
Goran VUKOBRATOVIĆ, dipl.inž.građ., Novi Sad
Branko KNEŽEVIĆ, dipl.inž.građ., Beograd

Članovi
Nadzornog
odбора:

Miroslav MIHAJLOVIĆ, dipl.inž.građ., Paraćin
prof. dr Zlatko MARKOVIĆ, dipl.inž.građ., Beograd
Aleksandar TRAJKOVIĆ, dipl.inž.građ., Beograd

Na istoj skupštini izabran je 1 počasni član DGKS:

Prof. dr Mirko AČIĆ, dipl.inž.građ.

SIMPOZIJUM 2012.

DRUŠTVA GRAĐEVINSKIH KONSTRUKTERA SRBIJE
VRNJAČKA BANJA, 19-21. septembar 2012.

Predsednik: prof. dr Miloš LAZOVIĆ, dipl.inž.građ., Beograd
Potpredsednik: Aleksandar BOJOVIĆ, dipl.inž.građ., Beograd
Sekretar: prof. dr Boško STEVANOVIĆ, dipl.inž.građ., Beograd

Nije bilo izbora za zaslužne i počasne članove društva.

ČETRNAESTI KONGRES

DRUŠTVA GRAĐEVINSKIH KONSTRUKTERA SRBIJE
NOVI SAD, 24-26. septembar 2014.

Na Skupštini DGKS, održanoj 24. septembra 2014. u Novom Sadu izabrano je novo predsedništvo i nadzorni odbor DGKS, u sledećem sastavu:

Predsednik: prof. dr Đorđe LADINOVIĆ, dipl.inž.građ., Novi Sad
Potpredsednik: Aleksandar BOJOVIĆ, dipl.inž.građ., Beograd
Generalni sekretar: prof. dr Zlatko MARKOVIĆ, dipl.inž.građ., Beograd
Sekretar: prof. dr Boško STEVANOVIĆ, dipl.inž.građ., Beograd
Članovi
predsedništva: prof. dr Snežana MARINKOVIĆ, dipl.inž.građ., Beograd
prof. dr Mira PETRONIJEVIĆ, dipl.inž.građ., Beograd
doc. dr Selimir LELOVIĆ, dipl.inž.građ., Beograd
mr Branko MILOSAVLJEVIĆ, dipl.inž.građ., Beograd
prof. dr Dragoslav STOJIĆ, dipl.inž.građ., Niš

prof. dr Danijel KUKARAS, dipl.inž.građ., Subotica
Branko KNEŽEVIĆ, dipl.inž.građ., Beograd
Đorđe PAVKOV, dipl.inž.građ., Novi Sad
Goran VUKOBRATOVIĆ, dipl.inž.građ., Novi Sad
Gojko GRBIĆ, dipl.inž.građ., Loznica
Lazar MARKOVIĆ, dipl.inž.građ., Subotica
Aleksandar ŽIVANOVIĆ, dipl.inž.građ., Beograd
Goran TADIĆ, dipl.inž.građ., Beograd

Članovi
Nadzornog
odбора:

Ivan IGNJATOVIĆ, dipl.inž.građ., predsednik, Beograd
prof. dr Dejan DIVAC, dipl.inž.građ., Beograd
Zoran KOVRLIJA, dipl.inž.građ., Beograd

Nije bilo izbora za zaslužne i počasne članove društva.

PRIZNANJA
SAVEZA DRUŠTAVA GRAĐEVINSKIH KONSTRUKTERA
JUGOSLAVIJE

ZA ŽIVOTNO DELO
U GRAĐEVINSKOM KONSTRUKTERSTVU

„Priznanje Saveza društava građevinskih konstruktera Jugoslavije za životno delo u građevinskom konstrukterstvu je najviše priznanje koje Predsedništvo SDGKJ dodeljuje svojim istaknutim članovima za izuzetan doprinos koji su dogodišnjim radom i rezultatima dali razvoju našeg građevinskog konstrukterstva i napretku Socijalističke Federativne Republike Jugoslavije.“

Priznanje je ustanovljeno 1989. godine.

Dobitnici priznanja SDGKJ za životno delo u građevinskom konstrukterstvu su:

1989.

DORĐE LAZAREVIĆ

akademik SANU, profesor dr h.c.,
diplomirani građevinski inženjer,
prvi predsednik Jugoslovenskog društva građevinskih konstruktera, od 1958. do 1964.

KRUNISLAV TONKOVIĆ

profesor dr,
diplomirani građevinski inženjer,
drugi predsednik Jugoslovenskog društva građevinskih konstruktera, od 1964. do 1969.

BRANKO ŽEŽELJ

akademik SANU, profesor,
diplomirani građevinski inženjer

1990.

NIKOLA HAJDIN

akademik SANU, profesor dr,
diplomirani građevinski inženjer,
predsednik Jugoslovenskog društva građevinskih konstruktera, od 1978. do 1981.

MILORAD IVKOVIĆ

profesor dr,
diplomirani građevinski inženjer

STANKO ŠRAM

profesor dr,
diplomirani građevinski inženjer

1991.

STOJAN DINIĆ

diplomirani građevinski inženjer,
sekretar i generalni sekretar SDGKJ od 1969.

ZLATKO KOSTREŃIĆ

profesor dr,
diplomirani građevinski inženjer,

FRANCI KRŽIĆ

profesor dr,
diplomirani građevinski inženjer,
potpredsednik SDGKJ od 1987.

PETAR SERAFIMOV

akademik MANU, profesor dr,
diplomirani građevinski inženjer

Od 1991. do 1998. godine nisu dodeljivana priznanja za životno delo.

Jugoslovensko društvo građevinskih konstruktora je 1998. godine donelo odluku o nastavljanju dodeljivanja priznanja JDGK za životno delo u građevinskom konstrukterstvu.

PRIZNANJA
JUGOSLOVENSKOG DRUŠTVA GRAĐEVINSKIH
KONSTRUKTERA

ZA ŽIVOTNO DELO
U GRAĐEVINSKOM KONSTRUKTERSTVU

„Priznanje Jugoslovenskog društva građevinskih konstruktera za životno delo u građevinskom konstrukterstvu je najviše priznanje koje Predsedništvo JDGK dodeljuje svojim istaknutim članovima za izuzetan doprinos koji su dogodišnjim radom i rezultatima dali razvoju našeg građevinskog konstrukterstva.“

Dobitnici priznanja JDGK za životno delo u građevinskom konstrukterstvu su:

1998.

GOJKO NENADIĆ

profesor,
diplomirani građevinski inženjer

ŽIVOTA PERIŠIĆ

profesor dr,
diplomirani građevinski inženjer,
predsednik SDGKJ/JDGK od 1987. do 1998.

2000.

BOŠKO PETROVIĆ

akademik SANU, profesor dr,
diplomirani građevinski inženjer

MILAN GOJKOVIĆ

profesor dr,
diplomirani građevinski inženjer

BOŽIDAR S. PAVIĆEVIĆ

diplomirani građevinski inženjer

2002.

MIRKO AĆIĆ

profesor dr,
diplomirani građevinski inženjer
predsednik JDGK od 1998.

RADOMIR FOLIĆ

profesor dr,
diplomirani građevinski inženjer

ARSENJE VUJOVIĆ

profesor dr,
diplomirani građevinski inženjer

DIMITRIJE ČERTIĆ

profesor,
diplomirani građevinski inženjer

2004.

MIROSLAV DEBELJKOVIĆ

profesor,
diplomirani građevinski inženjer

ALEKSANDAR PAKVOR

profesor dr,
diplomirani građevinski inženjer

PREDRAG ŽELALIĆ

diplomirani građevinski inženjer

2006.

MOMIR KRSTAVČEVIĆ

profesor,
diplomirani građevinski inženjer

PRIZNANJA
DRUŠTVA GRAĐEVINSKIH KONSTRUKTERA SRBIJE

ZA ŽIVOTNO DELO
U GRAĐEVINSKOM KONSTRUKTERSTVU

„Priznanje Društva građevinskih konstruktora Srbije za životno delo u građevinskom konstrukterstvu je najviše priznanje koje Predsedništvo DGKS dodeljuje svojim istaknutim članovima za izuzetan doprinos koji su dogodišnjim radom i rezultatima dali razvoju našeg građevinskog konstrukterstva.“

Dobitnici priznanja DGKS za životno delo u građevinskom konstrukterstvu su:

2008.

MIHAILO MURAVLJOV

profesor,
diplomirani građevinski inženjer

2010.

DANILO DRAGOJEVIĆ

diplomirani građevinski inženjer

VUKAN NJAGULJ

diplomirani građevinski inženjer

2012.

LJUBOMIR VLAJIĆ

profesor,
diplomirani građevinski inženjer

2014.

ŠERIF DUNICA

profesor,
diplomirani građevinski inženjer

2016.

VANJA ALENDAR

diplomirani građevinski inženjer

SLOBODAN CVETKOVIĆ

diplomirani građevinski inženjer

PRIZNANJA
SAVEZA DRUŠTAVA GRAĐEVINSKIH KONSTRUKTERA
JUGOSLAVIJE

ZA NAJBOLJA OSTVARENJA
U GRAĐEVINSKOM KONSTRUKTERSTVU

„U cilju podsticanja stvaralaštva, boljeg vrednovanja postignutih rezultata i najznačajnijih dostignuća našeg građevinskog konstrukterstva, Predsedništvo SDGKJ je početkom 1986. godine ustanovilo posebna godišnja stručna i društvena priznanja za najbolja ostvarenja u građevinskom konstrukterstvu u republikama i pokrajinama i u Jugoslaviji. Republička i pokrajinska društva građevinskih konstruktera dodeljuju svoja priznanja za najbolja ostvarenja u republikama i pokrajinama a Predsedništvo SDGKJ dodeljuje Jugoslovensko priznanje SDGKJ najboljem ostvarenju izabranom u konkurenciji ostvarenja koja su u toj godini dobila republička i pokrajinska priznanja.“

Priznanja su prvi put dodeljena za konstrukterska ostvarenja u 1986. godini.

Jugoslovenska priznanja SDGKJ za najbolja ostvarenja u građevinskom konstrukterstvu dobila su sledeća ostvarenja:

1986.

**KONSTRUKCIJA NOVOG HANGARA JUGOSLOVENSKOG AEROTRANSPORTA
NA AERODROMU BEOGRAD**

Odgovorni projektanti konstrukcije:
profesor dr Milorad Ivković, dipl.inž.građ.
profesor dr Života Perišić, dipl.inž.građ.
profesor dr Mirko Ačić, dipl.inž.građ.
profesor dr Aleksandar Pakvor, dipl.inž.građ.

Graditelji konstrukcije:
Živadin Mijailović, dipl.inž.građ.
Višeslav Milosavljević, dipl.inž.građ.

1987.

PROJEKAT KONSTRUKCIJE KOMPLEKSA "CIBONA" U ZAGREBU

profesor dr Milutin Anđelić, dipl.inž.građ.

1988.

Naučnoistraživački rad:

**POPREČNA RASPODELA OPTEREĆENJA KOD MOSTOVA SA VEĆIM BROJEM
GLAVNIH NOSAČA U POPREČNOM PRESEKCU**

asistent dr Zoran Desovski, dipl.inž.građ.

1989.

**TEHNIČKO-TEHNOLOŠKO UNAPREĐENJE GRAĐEVINSKOG
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**DOBITNICI PRIZNANJA DGKS
ZA ŽIVOTNO DELO
U GRAĐEVINSKOM KONSTRUKTERSTVU
U 2016. GODINI**

PRIZNANJE ZA ŽIVOTNO DELO U GRAĐEVINSKOM KONSTRUKTERSTVU VANJI ALENDARU, DIPL.INŽ.

Rođen 1947. u Zagrebu. Osnovnu i srednju školu završio u Beogradu. Građevinski fakultet Univerziteta u Beogradu upisao 1966. godine. Na prvoj godini studija dobitnik priznanja Fakulteta za uspeh na studijama, da bi konačno diplomirao na odseku za konstrukcije 1976. godine, nakon odsluženja vojnog roka. Od septembra 1976. godine, saradnik u Zavodu za betonske konstrukcije Građevinskog fakulteta Univerziteta u Beogradu. Za asistenta pripravnika za grupu predmeta Betonske konstrukcije na Građevinskom fakultetu u Beogradu izabran septembra 1977. godine. Karijeru na Građevinskom fakultetu završava 2007. godine, u zvanju višeg stručnog saradnika. Iste godine, kao tehnički direktor i partner pridružuje se D.Popoviću i N.Jovanoviću u osnivanju firme DNEC, koja danas ima biro u Abu Dhabiu-UAE, Beogradu i Bangaloru-Indija. Razveden, otac dva sina Andreja i Vladimira, i 'Deda Deko' dvojici unuka, Jovanu i Filipu.



Rad na Građevinskom fakultetu Univerziteta u Beogradu

Učestvuje u nastavi na grupi predmeta Betonske konstrukcije, Betonski mostovi i Građevinski materijali. U toku školske 1978/79. godine drži vežbanja iz predmeta Betonske konstrukcije na Građevinskom fakultetu u Subotici, a školske 1979/80. godine vežbanja iz predmeta Teorija betonskih konstrukcija na Fakultetu tehničkih nauka u Novom Sadu. Saraduje na izradi preko 150 diplomskih radova u Beogradu i Novom Sadu, od kojih su neki dali doprinos i razvoju prakse. Poslednjih godina rada na Fakultetu, drži nastavu na predmetu Betonske konstrukcije 2, sa težištem na aseizmičkom projektovanju i prethodno napregnutim konstrukcijama.

Sklon akcijama koje doprinose razvoju Katedre za betonske konstrukcije, uz podršku Ž.Perišića inicijator je nabavke računarskog sistema MicroVAX, čime je otvoren put formiranju šire obrazovnih inženjera. U okviru saradnje na izradi diplomskih radova, V.Alendar učinio je prve korake na uvođenju računarskih metoda u nastavu na predmetima betonskih konstrukcija. Pod njegovim rukovodstvom kandidat Z.Martinovski je 1987. godine izradio softver za kompletnu analizu prethodno napregnutih preseka i greda; kandidat D.Ješić 1989. u razrešava problem granične nosivosti proizvoljnog preseka napregnutog na složeno koso savijanje, da bi kandidat M.Mladenović 1990. uradio softver za graničnu analizu vitkih pomerljivih armiranobetonskih ramova približnim P-delta postupkom.

Kada se student B.Tepavčević zainteresovao za MicroVAX-ov Fortran, započinje nova era primene računara na Katedri za betonske konstrukcije. Rezultat je, između ostalog, moćan programski paket SAN za analizu konstrukcija. Praktično svi budući projekti rađeni na Katedri urađeni su uz pomoć SAN-a, uz obilato korišćenje prednosti sopstvenog otvorenog softvera, koji je prilagođavan specifičnim problemima. Polazeći od biblioteke rutina koje je razvio D.Ješić, V.Alendar razvija paket SLOMS za dimenzionisanje/kontrolu preseka i elemenata betonskih konstrukcija. Zajedno sa B.Tepavčevićem, SAN i SLOMS su integrisani u pouzdan i fleksibilan paket za projektovanje betonskih konstrukcija, sa kompletnom grafičkom

podrškom, razvijenom i uz pomoć i entuzijazam B.Milosavljevića. 1991. godine, tada diplomac, a danas menadžer u DNEC-u, N.Jovanović dodaje paketu SAN+SLOMS alternativni pred i post-processor u okruženju AutoCAD-a. Rad objavljen na CAD forumu u Novom Sadu 1995. osvojio je prvu nagradu. Koncept sopstvenog paketa za projektovanje omogućio je, na primer, da se složeni projekat konstrukcije 'Beogradske Arene' analizira i realizuje u kratkom roku, uz modeliranje faza građenja, procedura utezanja kablova, liftovanja, dodavanja kablova, finalnih korekcija geometrije dodatnim utezanjima, obrade rezultata testova i merenja itd.

U toku izrade projekta palate 'Beobanke' na Slaviji u Beogradu (odgovorni projektant S.Vukelić), V.Alendar radi analizu stubova i zidova, sa efektima zemljotresa. Za razliku od dotadašnje prakse da ozbiljne seizmičke analize rade drugi specijalisti, V.Alendar predlaže da se projektantski tim osposobi i preuzme i taj deo posla, jer je seizmika stvar koncepta i detalja konstrukcije, posao projektanata. Uz podršku Ž.Perišića, V.Alendar odlazi u posetu inštiutu IZIIS u Skolju, i donosi tada kulturni otvoreni softver TABS77 – bio je to početak ozbiljnog bavljenja seizmikom na Katedri za betonske konstrukcije. Entuzijazam za učenjem, napredovanjem nikada nije nedostajao, tako da je uz trud i B.Tepavčevića i B.Milosavljevića, jedan akademski softver nadograđen sa bazom podataka za integraciju sa ostalim 'kućnim' rutinama, kao i grafikom.

Sledeći neminovan korak bio je ulazak u nelinearne analize. Okidač za akciju bio je doktorat S.Marinković - proboj ivičnih stubova prethodno napregnutih ploča. Na inicijativu V.Alendara i S.Marinković, uz podršku Ž.Perišića, Institut nabavlja istraživačku verziju paketa DIANA (TNO-Delft). Slede meseci mukotrpnog učenja, testiranja. Rezultat su, osim numeričkog dela doktorata, i članci objavljeni u inostranim časopisima i simpozijumima (S.Marinković, V.Alendar), rad sa M.Sekulovićem i D.Šumarcem objavljen u časopisu *Theoretical and applied mechanics* 2000. kao i rad 'O seizmičkom odgovoru zgrada sa AB zidovima' (N.Stojanović, V.Alendar, M.Ačić). Kasnije, V.Alendar i S.Marinković za potrebe 'Energoprojekta-Entel' rade i studiju efekata udarnog talasa usled eksplozije naftnog postrojenja na objekat i opremu kontrolnog centra, ozbiljna i uspešna nelinearna dinamička naliza, publikovana na 'iNDiS 2003', Novi Sad. Bio je to začetak nelinearnih analiza metodom konačnih elemenata u Institutu za materijale i konstrukcije.

V.Alendar učestvuje u uvođenju u domaću praksu i nastavu Evrokodova iz oblasti betonskih konstrukcija, seizmike, prethodnog naprezanja i silosa, prvo kao prevodilac dela Evrokoda 8 1997., potom kroz radove objavljene na savetovanju 'Evrokodovi i Jugoslovensko građevinsko konstrukterstvo', kao i radove objavljene u domaćim časopisima.

Nekada student, danas dragi prijatelj S.Brzev, profesor u Kanadi i saradnik Američkog instituta EERI (Earthquake Engineering Research Institute), 2005. godine predlaže nam da učestvujemo na konkursu EERI-a za 'Male projekte u zemljama u razvoju'. V.Alendar animira i okuplja grupu entuzijasta (N.Stojanović, V.Koković, I.Ignjatović i V.Majstorović), konkuriramo i dobijamo ugovor (Grant) od 'čak' 3500 američkih dolara za temu 'Seizmičke pouzdanosti nadograđenih objekata u Beogradu'. Dok su nas kolege zavitalavale zbog sume, mi smo proveli sjajno vreme radeći zajedno. Rad je poslat EERI-u, kao i publikovan na Kongresu *fib*-a u Napulju 2006. Od dobijenih sredstava, mlađi učesnici su prisustvovali kongresu, videli i Pompeju, a od ostatka para kupljene su merne trake za buduće eksperimente. Koliko god simboličan, bio je to prvi međunarodni Grant (Ugovor) za istraživanje uopšte dobijen na Katedri za betonske konstrukcije, koliko je poznato.

Nakon ovoga rada, V.Alendar je prihvatio ponudu S.Brzev u ime EERI-a da bude stalni Reviewer (Recenzent) na EERI-evom projektu 'World Housing Encyclopaedia', u kome se

objavljaju radovi inženjera i istraživača iz celog sveta, koji prikazuju raznovrsna lokalna iskustva i praksu u zaštiti objekata od zemljotresa. Uzbudljiv volonterski posao, sa pisanom zahvalnicom EERI-a za uloženi entuzijazam i trud.

V.Alendar je u domaćim i stranim časopisima, kongresima i simpozijumima objavio više od trideset radova u kojima su obrađene teme iz oblasti graničnih i stanja upotrebljivosti klasičnih i prethodno napregnutih elemenata i konstrukcija, spoljašnje prethodno napregnutih konstrukcija velikih raspona, vitkih konstrukcija, zemljotresnog inženjerstva, kao i mogućnostima primene računara pri analizi granične nosivosti armirano betonskih i prethodno napregnutih preseka. Početkom 1989 godine, sa D.Najdanovićem i D.Ješićem, objavio je knjigu 'Dijagrami za dimenzionisanje armirano betonskih preseka prema graničnoj nosivosti'. Zahvaljujući uvođenju računarskih algoritama, V.Alendar je izvršio obimnu numeričku analizu uticaja pojedinih faktora na duktilitet armiranobetonskih preseka. Svojim radovima V.Alendar dao je doprinos rešenju problema uticaja tečenja i skupljanja betona u statički neodređenim linijskim armiranobetonskim nosačima, polazeći od algebarskih veza napona i dilatacija betona odnosno pojma 'korigovanog efektivnog modula elastičnosti', kako u formi metoda sila, tako i matičnom obliku tačne metode deformacija. Analiza glavnih nosača 'Hangara JAT-a' urađena je 'kućnim' softverom čiji se algoritam zasniva na navedenim rešenjima. Zaokružujući oblast primene računarskih algoritama, V.Alendar je dao pregled savremenih problema granične nosivosti vitkih armiranobetonskih konstrukcija, kao i tendencije njihovog rešavanja, uz kritičku analizu stanja u našoj tehničkoj regulativi i praksi. Za potrebe prakse, formulisan je hibridni postupak 'moguće ravnoteže model stuba', za čiju su primenu urađene i odgovarajuće tabele.

Kada radite na fakultetu, od vas se očekuje i da pišete nešto korisno studentima, a i praksi. Zajedno sa V.Korolijom, A.Pakvorom, M.Đurđevićem i D.Najdanovićem, V.Alendar je 1979. godine objavio 'Prednapregnuti beton - materijal za vežbanja', skripta korišćena za deo vežbanja na predmetu Masivne konstrukcije.

Početkom 1981. godine, sa Ž.Perišićem, objavio je zapaženu publikaciju 'Prethodno napregnuti beton-primeri za vežbe', koja je doživela četiri izdanja. U tom su materijalu prvi put u okviru redovne nastave prikazana rešenja pojedinih problema od praktičnog značaja. Pomenuta publikacija korišćena je u izvođenju nastave i na fakultetima u Novom Sadu i Titogradu.

Preuzevši organizaciju vežbanja na predmetu Betonske konstrukcije III, V.Alendar je, u saradnji sa predmetnim nastavnicima, uveo niz inovacija u redovnu nastavu, posebno u oblasti granične nosivosti vitkih armiranobetonskih konstrukcija. Rezultat toga rada je i samostalni prilog iz ove oblasti u okviru knjige 'Beton i armirani beton prema BAB 87' u izdanju Građevinske knjige, 1991. godine.

V.Alendar je 2003. publikovao elektronsko izdanje skripti 'Prethodno napregnuti beton'. Pet poglavlja je urađeno u toku pet nedelja nastave, teško ponovljiv entuzijazam i inspiracija.

Za potrebe kursa Projektovanje i građenje betonskih konstrukcija 2, V.Alendar je publikovao skripta 'Projektovanje seizmički otpornih armirano betonskih konstrukcija kroz primere', u kojima je po prvi put prikazana folozofija i praktična primena Evrokoda 8. Prva verzija je bila deljena studentima u rukopisu, da bi 2004, na uporno insistiranje dragog saradnika i prijatelja I.Ignjatovića, koji je preuzeo na sebe da otkuca ceo tekst, konačno bilo objavljeno i elektronsko izdanje, stavljeno na slobodnu upotrebu preko interneta. Ispostaviće se da je tako nešto u tom trenutku bilo potrebno i praksi, sudeći po pozivima ljudi iz regiona.

U okviru Programa permanentnog usavršavanja, V.Alendar je 2005. u Inženjerskoj komori Srbije održao predavanje pod naslovom ‘Gde smo i šta nas očekuje u oblasti aseizmičkog projektovanja’.

Stručni rad na Građevinskom fakultetu 1997-2007

V.Alendar je učestvovao u izradi preko osamdeset idejnih i glavnih projekata, studija, sanacija i rekonstrukcija, revizija i ekspertiza: stambenih i poslovnih zgrada, industrijskih i hidrotehničkih postrojenja, silosa, sportskih dvorana, komunikacionih tornjeva, vodotornjeva, hangara za avione, mostova itd. Stručni rad prikazan je u više od trideset radova u zemlji i inostranstvu. Kao član tima na projektu ‘Beogradske Arene’ dobitnik priznanja JDGK za najbolje konstruktorsko ostvarenje 1998-1999. Na *fib*-simpozijumu u Amsterdamu 2008., projekat konstrukcije ‘Delta City’ osvaja drugu nagradu u kategoriji poster prezentacija. Kao projektant ili konsultant radio je u Rusiji, Izraelu, Čehoslovačkoj, Iraku i Uzbekistanu. Od značajnijih/interesantnijih projekata, navode se:

- Studija ponašanja sa sanacijom prslina prethodno napregnutih prilaznih konstrukcija Pančevačkog mosta preko Dunava sa Beogradske strane (sa V.Korolijom, A.Božanovićem, M.Đurđevićem i M.Žikićem);
- Idejni i glavni projekat sanacije konstrukcije hotela ‘Mimoza’ i ‘Kamelija’ u Tivtu, posle zemljotresa od 15.04.1979. godine, (sa M.Ivkovićem, A.Pakvorom, M.Manjlovićem);
- Glavni projekat sanacionih mera na mostu kod Kule, preko kanala Bezdana-Vrba, (sa Ž.Darijevićem, A.Pakvorom);
- Idejni i glavni projekat konstrukcije filtarskog postrojenja u sklopu kompleksa za prečišćavanje vode sistema vodosnabdevanja baze 404 u Iraku, (Sa M.Ivkovićem, Ž.Perišićem, A.Pakvorom, V.Misitom);
- Glavni projekat poslovne zgrade ‘Aeroinženjeringa’, (Sa M.Ivkovićem, R.Stojadinovićem, D.Bajićem);
- Glavni projekat ‘Hangara 2 JAT-a u Surčinu’, (Sa M.Ivkovićem, Ž.Perišićem, M.Ačićem, A.Pakvorom, M.Đurđevićem, D.Ostojićem, J.Taranom, D.Bajićem);
- Idejni i glavni projekat vodotornja ‘Progar’ u okviru sistema sirove vode na levoj obali Save (sa D.Najdanovićem, D.Bajićem i J.Pregerom);
- Glavni projekat sistema montažnih armiranobetonskih proizvodnih hala i skladišta sa lakim krovnim dvopojasnim nosačima, za GRO ‘Jablanica’, Valjevo, (sa M.Ačićem i D.Ostojićem);
- Glavni projekat konstrukcije hrama ‘Svetog Arhangela Mihajla’ u Jabučju. Arhitekta Z.Bundalo;
- Glavni projekat sanacije konstrukcije objekta restorana društvene ishrane fabrike ‘Elektroporcelan’ u Arandjelovcu, (sa D.Najdanovićem);
- Glavni projekat konstrukcije objekta poslovne zgrade ‘Beobanka’ na Slaviji, (sa S.Vukelićem, Ž.Perišićem, D.Najdanovićem, N.Pecićem, S.Marinković);
- Glavni projekat konstrukcije stambenih zgrada u montažnom sistemu ‘Dom’ u Haifi – Israel (sa M.Ivkovićem, Ž.Perišićem, M.Đurđević, M.Stojanovićem, A.Pavićem, B.Milivojević);
- Glavni projekat konstrukcije objekta hotela ‘Kidmat-Eden’ u Haderi–Israel, GP ‘Dom’ (sa M.Ivkovićem, Ž.Perišićem, M.Đurđevićem, B.Milivojević);

- Idejni i glavni projekat stola i temelja nove rotacije 'Politika' u Krnjači, (sa Ž.Perišićem, D.Najdanovićem);
- Glavni projekat konstrukcije sportskog kompleksa 'Perm' u Rusiji (sa D.Ostojićem);
- Glavni projekat konstrukcije poslovnog centra 'Zepter' u ulici Kralja Petra u Beogradu, arhitekta B.Mitrović (sa D.Najdanovićem);
- Glavni projekat konstrukcije krova i glavnih stubova sportske dvorane 'Beogradska Arena' u bloku 25 na Novom Beogradu, arhitekta V.Slavica (sa M.Ivkovićem, Ž.Perišićem, M.Ačićem, A.Pakvorom, S.Marinković, B.Tepavčevićem);
- Projekat dva rashladna tornja za termolektranu Kolubara-B (paralelni proračun i nostrifikacija sa Ž.Perišićem, M.Sekulovićem, S.Marinković);
- Studija seizmičke otpornosti objekta Generalštaba na uglu ulica Kneza Miloša i Nemanjine u Beogradu, za IMS (sa S.Marinković i B.Milosavljevićem);
- Glavni projekat konstrukcije 'Poslovno-fnansijskog centra u Bloku 20' na Novom Beogradu (sa M.Ačićem i D.Ostojićem);
- Idejni projekat konstrukcije dvorane sportova u Herceg Novom - Igalo, arhitekta V.Slavica(sa M.Ivkovićem, Ž.Perišićem, S.Marinković);
- Tehnička kontrola glavnog projekta konstrukcije do kote 105 Železničke stanice Beograd - centar (Prokop) (sa A. Pakvorom, B.Milosavljevićem i drugima);
- Poslovna zgrada 'MCFU' u Taškentu - Uzbekistan, konsultant Energoprojekta-Visokogradnja za konstrukciju i seizmiku (sa Ž.Perišićem);
- Glavni projekat konstrukcije sportske dvorane 'Golubovci' u Crnoj Gori (sa M.Ivkovićem, D.Ostojićem, B.Milosavljevićem);
- Glavni projekat konstrukcije poslovne zgrade 'Delta Sport Centra' u Novom Beogradu (sa S.Marinković i Z.Markovićem);
- Glavni projekat konstrukcije hipermarketa 'Cora' u Beogradu (sa S.Marinković, D.Najdanovićem, Z.Markovićem i D.Buđevcem);
- Idejno rešenje konstrukcije poslovnog tornja 'Dubai Lighthouse' u Dubaiu visine 600m, arhitekta Z.Bojović (sa M.Ivkovićem, A.Slijepčevićem, S.Sindelićem);
- Idejno rešenje konstrukcije tornja visine 650m sa vetro-turbinama 'Burj Al Abraj' u Bahreinu, arhitekta Z.Bojović (sa M.Ivkovićem, A.Slijepčevićem, S.Sindelićem);
- Nelinearna dinamička analiza ponašanja konstrukcije kontrolnog objekta u Kataru pri dejstvu eksplozivnog talasa (sa S.Marinković);
- Nelinearna dinamička analiza ponašanja pri zemljotresu konstrukcije 'Domovinskog mosta' u Zagrebu, sa ciljem optimizacije dampera (sa B.Milosavljevićem);
- Projekat mosta na Adi, konkursno rešenje (sa Z.Markovićem, D.Buđevcem);
- Glavni projekat konstrukcije tržnog centra 'Delta City' (sa S.Marinković, V.Kokovićem, I.Ignjatovićem, B.Milosavljevićem, Z.Markovićem);
- Revizija konstrukcije novog mosta preko Dunava kod Beške (sa D.Najdanovićem);
- Tehnička kontrola glavnog projekta sanacije mosta 'Gazela' i prilaznih konstrukcija (sa D.Najdanovićem);

Rad u DNEC-u, 2007-

Privatna firma DNEC, biro u Beogradu osnovana je 2007. uz podršku, i pod okriljem Engleske kompanije 'Hyder Consulting', sa ciljem ojačanja njihove konstruktorske grupe za visoke objekte. Uspešna saradnja na žalost prebrzo se završava, zbog svetske ekonomske krize 2009. Na predlog DNEC-a, ugovor je zamrznut, nakon čega DNEC nastavlja svojim putem.

Kao tehnički direktor i partner, V.Alendar radi na razvoju firme, edukaciji mlađih inženjera i kao konsultant odnosno projektant. Od značajnijih/interesantnijih projekata, navode se:

- Projekat za izvođenje (for construction) rezidencijalnog tornja 'Pentominium Tower' u Dubaiu, UAE, 120 spratova, visina 518m;
- Koncept i idejni projekat tornja 'Burj Al Fatan Tower' u Dubaiu, UAE, 97 spratova, visina 463m;
- Koncept konstrukcije tornja 'Abu Dhabi Tower' u Dohi, Katar, 97 spratova, visina 520m;
- Projekat za izvođenje prethodno napregnutih tavanica tornja 'Doha Convention Center&Tower' u Dohi, Katar;
- Projekat za izvođenje za 'VSL', 100.000m² prethodno napregnutih tavanica kompleksa 'Lakeside Towers' u Dubaiu, UAE;
- Idejni projekat konstrukcije kompleksa 'Expo', Budva, Crna Gora;
- Projekat za izvođenje 360.000m² konstrukcije objekta 'Sheikh Khalifa Medical City', Abu Dhabi, UAE;
- 'CFPOS – Centar za promociju nauke', Beograd - Konkursno rešenje sa arhitektonskim biroom 'Sadar+Vuga', Ljubljana/Beograd;
- 'CFPOS – Centar za promociju nauke', Beograd, - Idejni i Glavni projekat prvonačadenog rešenja arh. Wolfganga Tschappelera, Austria, sa 'BWK Engineers', Beograd;
- Idejni projekat konstrukcije kompleksa 'Hawai Towers' u Kambodzi, za 'Houser AG';
- Glavni projekat konstrukcije tržnog centra 'Delta Bau Haus', Beograd;
- Konsultant Đ.Antelju i 'Gemaxu' na izgradnji/obnovi 'Saborne Crkve Svete Trojice' u Mostaru;
- Koncept i projekat za izvođenje konstrukcije hotela 'Plaza Hilton Garden Inn' u Dubaiu, UAE;
- 'CBA Tower' (Central Bank of Azerbaijan), Baku, za 'Besix'- Belgija. Seizmičke analize, projekat za izvođenje čeličnih konstrukcija, definitivna potvrda koncepta i svih detalja pomoću PBD – Performance Based Design nelinearnih procedura;
- Idejni i Glavni projekat konstrukcije objekta 'Kula Beograd' u kompleksu 'Beograd na vodi'. Vodeći projektant 'SOM', Čikago, USA. Lokalni partner konzorcijum Energoprojekt (EPUiA i Entel) i DNEC, Beograd.

V.Alendar nastavlja rad na uvođenju Evrokodova u Srbiji, član je Komisije Instituta za Standardizaciju Srbije koja priprema delove koji se odnose na dejstva, betonske konstrukcije i seizmiku.

PRIZNANJE ZA ŽIVOTNO DELO U GRAĐEVINSKOM KONSTRUKTERSTVU SLOBODANU CVETKOVIĆU, DIPL.INŽ.

Rođen 30. avgusta 1943. godine u Zaječaru. Osnovnu i srednju školu (V beogradska gimnazija) pohađao je u Beogradu. Na Građevinski fakultet Univerziteta u Beogradu upisao se 1962. godine, gde je i diplomirao 1967. godine sa prosečnom ocenom 8,29. Odmah nakon diplomiranja zaposlio se u Direkciji za izgradnju mostova grada Beograda (sada Mostprojekt). U toku svoje profesionalne karijere, pored Mostprojekta, radio je i na Građevinskom fakultetu u Beogradu (2 godine), Trudbeniku (1 godina), Progresu (2,5 godine) i kratko vreme u Mostogradnji. Pored redovnog posla bavio se puno i projektovanjem van matične radne organizacije. 1996. godine osniva firmu C&N Consult, odakle odlazi u penziju 2012. godine. Zvanično penzionisanje nije značilo kraj njegove profesionalne karijere - i danas je aktivan kao projektant, konsultant i saradnik Instituta za standardizaciju Srbije. Govori engleski jezik, a služi se ruskim. Oženjen je, ima jednu ćerku i unuku.



Osnovni moto profesionalne karijere

Uvek je smatrao da fakultet daje samo bazično znanje – stalan rad na stručnom usavršavanju je osnov za građenje uspešne profesionalne karijere. Inženjer početnik treba da prođe sve faze rada kako u birou, tako i na gradilištu. Imao je sreću da mu je u Direkciji za izgradnju mostova bio učitelj pokojni Borivoje Todorović.

Bio je timski igrač, najčešće je saradivao sa Dr. Branislavom Kolundžijom i Dr. Šerifom Dunicom, kao i sa pokojnim Vladislavom Matićem. Voleo je da radi sa mladim ljudima – jedan broj danas istaknutih inženjera bili su na početku svojih karijera njegovi saradnici.

On je veliki kolekcionar profesionalne literature - bio je poznat po svojoj stručnoj biblioteci; danas umesto knjiga i časopisa sakuplja članke iz oblasti koja ga interesuje sa Interneta.

Pored stručnog angažovanja u birou ili na gradilištu, uvek je nalazio vremena za aktivnosti koje treba da doprinose napretku profesije - više od 30 godina je saradnik Zavoda za standardizaciju odnosno Instituta za standardizaciju Srbije gde je u jednom četvorogodišnjem mandatu bio i predsednik Skupštine Instituta.

Odmah po formiranju Inženjerske komore Srbije, aktivno se uključio u njen rad gde je inicirao nekoliko projekata iz kojih su se “rodili” nacionalni aneksi za dva Evrokoda. 2014. godine, dobio je nagradu za životno delo Inženjerske komore Srbije.

Bio je jedan od ispitivača za polaganje stručnog ispita pri Savezu inženjera i tehničara, a zatim i predsednik Komisije za polaganje stručnog ispita iz građevinske struke pri Inženjerskoj komori Srbije.

Jedno kratko vreme bio je član Republičke revizione komisije.

Jedan je od osnivača ACES-a - Udruženja inženjera konsultanata Srbije.

Stručni rad

Svoj profesionalni rad nije vezao samo za teritoriju ex Jugoslavije, odnosno Srbije. Radio je širom sveta – preko Afrike, Srednjeg istoka, Dalekog istoka, Kanade i Kariba, do Rusije, Belorusije i Slovačke. To je podrazumevalo da je morao da primenjuje razne standarde, da dobro poznaje praksu međunarodnih ugovora i tehničkih specifikacija i da bude dovoljno fleksibilan u kontaktu sa ljudima raznih kultura.

Kada je bio u Srbiji, voleo je da učestvuje u konkursima - cilj mu je uvek bio da prezentuje nove ideje za racionalne konstrukcije. Nažalost, njegov pristup nije uvek nailazio na razumevanje ocenjivačkih komisija. Za njegovu satisfakciju, vreme je pokazalo na nakoliko objekata da je bio u pravu. Dobitnik je treće nagrade za idejno rešenje mosta preko Cetine u Omišu.

Svoj profesionalni rad nije vezao samo za projektovanje, za određen tip konstrukcija odnosno material. Voli da kaže da radi ono što drugi neće.

Projektovao je niz antenskih stubova različitih namena, velikih hala i hangara, sportskih stadiona, raznih zgrada, radio razne projekte za termoelektrane. Projektovao je i hangarska vrata velikih raspona i visina, vrata i nekoliko tipova specijalnih skela za kaponire, te opremu za televizijske i filmske studije.

Posebna oblast njegovog profesionalnog angažmana su mostovi gde je bio kao projektant, nadzor ili u timu izvođača odnosno u timu za pregled i ocenu stanja konstrukcija. Njegov prvi posao bila je opravka mosta preko Save kod Ekonomskog fakulteta u Beogradu(1967-1968.) , a njegov poslednji zadatak je prevođenje tramvaja preko mosta Ada (2015-2016.) u Beogradu.

Dvostruki je dobitnik priznanja za DGKS za najbolje konstruktorsko ostvarenje:

1. Drumski most preko Save kod Ostružnice;
2. Rehabilitacija visećeg mosta Puente Duarte u Santo Domingu.

Najznačajniji projekti

- | | |
|------------------|---|
| 2015 | Glavni projekt popravke visećeg mosta Puente Hermanos Putino u Dominikanskoj republici - Projektovano rešenje kombinacija lančanice i kosih vešaljki. |
| 2013 | Glavni projekt popravke visećeg mosta u Santo Domingu posle presecanja dela nosećih kablova. |
| 2012 | Glavni projekt Tržnog centra u Minsku, Belorusija. |
| 2008-2009 | Glavni projektant svih konstrukcija u krugu elektrane u okviru Projekta odlaganja pepela u TE Kostolac. |
| 2005–2006 | Glavni i izvođački projekt rehabilitacije visećeg mosta u Santo Domingu, projektant rekonstrukcije nosećeg kablovskog sistema, konsultant tokom radova. |
| 2003 | Institutional Capacity Building in Transport Sector in Serbia – rad na projektu u sklopu firme KAMPSAX. Naručilac: Evropska agencija za rekonstrukciju. |
| 2003 | Projekt pešačkog visećeg mosta preko reke Lim. Projekt urađen u kooperaciji MOSTOGRADNJA – MOSTPROJEKT – C&N CONSULT. Naručilac: Evropska agencija za rekonstrukciju. |

- 2002** Projekt sanacije i obnove porušenog mosta preko Save kod Ostružnice. Konsultant u MOSTPROJEKTU. Pregled porušene konstrukcije, tehnologija obnove, tehničke specifikacije za radove na obnovi.
- 2000-2001** Projekt čišćenja reke Dunav od ruševina mostova u Novom Sadu. Ekspert u timu COWI-UTIBER-MOSTPROJEKT-VMS. Učešće u istražnim radovima i izradi projekta čišćenja ruševina Petrovaradinskog i Žeželjevog mosta.
- 1996-2000** Odgovorni nadzorni inženjer za mostove u Sekciji 3 na gradnji Brzih pruga Koreje, u sklopu tima DE CONSULT. Vrednost radova u Sekciji 3 za dužinu trase od 20 kilometara iznosila je oko 250.000.000 US\$. U okviru sekcije, više od 12 kilometara trase ležalo je na mostovima. Svi mostovi bili su od prednapregnutog betona, građeni su FSM, MSS i PSM metodama. Prvi put je PSM metod primenjen na izgradnji železničkih mostova – svakih 36 sati po jedan most širine 14 metara i raspona 25 metara, težine 620 tona, je bio proizveden, utegnut i montiran.
Pored mostova, odgovoran za izgradnju dvokolosečnog tunela dužine 400 metara građenog po »cut&cover« metodi.
- 1995-1996** Rad u Rusiji na projektima zgradarstva. Rad za firmu FARMAN Engineering.
- 1994-1995** Pomoćnik generalnog direktora firme PROGRES za sektor investicione gradnje. Project management za više projekata u Rusiji i Kazahstanu vrednih oko 300 miliona US\$.
- 1991-1992** Glavni projektant mosta za toplodalekovod (4 Φ 1000 mm) preko reke Save kod Obrenovca. Konstrukcija mosta izvedena sa rešetkastim glavnim nosačima formiranih od tipiziranih elemenata na osnovu zahteva Investitora da želi "jeftin" most i brzu gradnju. Glavni otvori mosta imaju raspone 140 i 100 metara.
- 1988-1991** Glavni projektant mosta (Cvetković – Dragojević) preko reke Save u Beogradu kod Ostružnice. Most je projektovan u sklopu obilaznice Dobanovci – Bujanj potok. Donji stroj mosta izgrađen je za puni profil saobraćajnice, dok je gornji stroj (čelična konstrukcija) izveden samo za I fazu. Ukupna dužina mosta je 583 m, dok je centralni otvor raspona 198 m. Neposredno pred puštanje u saobraćaj, most je delimično porušen tokom NATO bombardovanja.
- 1987-1989** Glavni projektant čeličnog lučnog mosta preko akumulacije Gazivode. Ukupna dužina mosta je 323,65 metara, sa rasponom luka od 195 m.
- 1983-1994** Glavni projektant železničkog mosta preko reke Tise. Centralni otvor mosta ima raspon od 80 metara.
Pregled i rehabilitacija železničko-drumskog mosta preko Velike Morave kod Ljubičeva. Most je izgrađen krajem XIX veka.
Pregled i rehabilitacija mosta GAZELA u Beogradu.
Pregled železničko-drumskog mosta preko Dunava u Beogradu.
Pregled više stotina mostova na putnoj mreži u Srbiji.
Autor idejnog projekta konstrukcije (Hajdin – Cvetković) nove železničke stanice u Beogradu (Prokop).
Idejni projekat (Hajdin – Cvetković) prelaska železnice preko velikih reka (Dunav i Sava) preko mosta formiranog od tanker-barži.
Glavni projektant konstrukcije (Cvetković – Dunica – Kolundžija) nove zgrade RTS u Aberdarevoj ulici u Beogradu.

- Glavni projektant konstrukcije zgrada u okviru MZ Janko Lisjak u Beogradu (blok zgrada duž ulice Vojislava Ilića).
- Magistralni cevovod vodosnabdevanja 2 Φ 420 preko Pančevačkog mosta u Beogradu.
- Konsalting i usaglašavanje projekta objekta kotlarnice i gerista za termoelektranu TE TO Kolubara B. Tim (Hajdin – Dunica – Kolundžija – Cvetković) radio je u okviru ABB COMBUSTION.
- Projekt 1100 u Iraku. Rukovodilac tima zaduženog za projektovanje i kontakte sa Investitorom u domenu čeličnih konstrukcija.
- 1974–1979** Glavni projekt (Hajdin – Jevtović – Cvetković – Matić) dvokolosečnog železničkog mosta preko reke Save u Beogradu. Most je sistema kose vešaljke, sa centralnim otvorom raspona 254 m.
- Ostalo** Veći broj hangara za opravke aviona, raspona do 72 m. Pored noseće konstrukcije hangara, projektovane su posebne kranske staze za viseće kranove koji su pokrivali kompletnu površinu hangara. Hangari su izvedeni u Iraku, Libiji i Alžiru.
- Specijalna sigurnosna vrata i obloge na izduvnim cevima u kaponirima.
- Veći broj pasivnih reflektora površine 10 - 60 m² za potrebe pošta.
- Veći broj industrijskih hala različitih raspona i namena “Naša škola” u Obrenovcu, hala FOP u Smederevskoj Palanci, nekoliko hala na Kosovu.
- Zgrada Komandno-operativnog centra u Beogradu (Matić – Cvetković). Kompletna konstrukcija zgrade i heliodroma izgrađena je u MERO sistemu.
- Baterija silosa za cement kapaciteta 3000 tona (Cvetković – Dunica – Kolundžija) izgrađenog u luci Misurata – Libija.
- Višekomorni silosi za agregate kod fabrika betona (Cvetković – Matić).
- Veći broj antenskih stubova različitih konstrukcija projektovanih za potrebe televizije, pošte, milicije i vojske: Deli Jovan, Cviljen, Tupižnica, Ovčar
- Oprema u TV i filmskim studijima: TV Zagreb, TV Sarajevo, TV Titograd, Jadran film, svečana sala INA u Zagrebu.
- Pokriveni stadion za hokej u Trenčinu, Slovačka. (Cvetković – Đudurović).

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P1 – P5

**DOBITNICI PRIZNANJA DGKS
ZA NAJBOLJA OSTVARENJA
U GRAĐEVINSKOM KONSTRUKTERSTVU
U 2014. I 2015. GODINI**



Siniša Mihajlović¹, Biljana Čolić², Dejan Srejić³

IZGRADNJA ŽELEZNIČKOG MOSTA NA VELIKOJ MORAVI KOD ČUPRIJE

Rezime

P-1

Železnički most preko Velike Morave se nalazi na magistralnoj pruzi Beograd-Niš, deonica Gilje-Ćuprija-Paraćin. Most je dvokolosečni sa kolosekom položenim u tucaničkom zastoru. Most preko Velike Morave premošćuje reku sa pet otvora. Konstrukcija mosta je čelična rešetkasta kontinualna greda raspona $L = 52,5 + 67,5 + 82,5 + 67,5 + 52,5 = 322,5\text{m}$. Rešetkasta ispuna je sa dijagonalama, bez vertikalala. Konstrukcija je projektovana i izvedena u zavarenoj izradi. Konstrukcija je montirana na nasipu ispred mosta i sukcesivno podužno nagurivana preko stubova bez pomoćnih jarmova. Most je završen i pušten u saobraćaj 2015 godine.

Ključne reči: Železnički most, čelična rešetka, šavovi, podužno nagurivanje

CONSTRUCTION OF RAILWAY BRIDGE OVER THE VELIKA MORAVA NEAR CUPRIJA

Summary

The railway bridge over the Velika Morava River is located on the Belgrade – Niš main railway line, Gilje – Ćuprija – Paraćin section. It is a double-track bridge with track laid on crushed-stone ballast. The bridge over the Velika Morava River has five spans. The bridge construction is a steel truss continuous beam, span $L = 52.5 + 67.5 + 82.5 + 67.5 + 52.5 = 322.5\text{ m}$. Truss consists of diagonal members, without vertical members. The bridge construction is completely welded. The superstructure is assembled on embankment in front of the bridge and successively longitudinally launching over piers without the auxiliary trestles. The bridge was completed and put into service in 2015.

Key words: railway bridge, steel truss, welds, longitudinal launching

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1 UVOD

U radu je prikazana izgradnja čelične konstrukcije novog železničkog mosta preko Velike Morave u Čupriji, na magistralnoj pruzi Beograd-Niš, deonica Gilje-Čuprija-Paraćin. Investitor modernizacije pruge i izgradnje novog mosta je bilo preduzeće " JP Železnice Srbije " a finansiran je iz međunarodnog kredita EIB br.4. Projekat mosta (Generalni, Idejni i Glavni projekat) je izradio " Saobraćajni institut CIP d.o.o. " koji je sve vreme na izradi i montaži čelične konstrukcije vršio i projektantski nadzor. Na drugom ponovljenom tenderu posao na izgradnji mosta je dobio J.V. Ogranak ALPINE Bau Gmbh Beograd -METEORIT d.o.o. - Maribor, koji je trebalo da izradi i namontira čeličnu konstrukciju. Radovi na izgradnji su započeli 01.03.2011 god. Stručni nadzor je vršio Sektor za Investicije J.P. Železnica Srbije. Izgradnju su pratili brojni problemi sa više prekida radova i nesuglasticama među članicama konzorcijuma, da bi se ta faza okončala napuštanjem METEORITA d.o.o. na samom početku montaže čelične konstrukcije a potom i stečajem nosioca posla ALPINE Bau Gmbh. Nastavak radova na montaži i izradi nedostajućih elemenata je poveren domaćem konzorcijumu GOŠA-FOM A.D. i MOSTOGRADNJA A.D. U nastavku radova su se dogodile katastrofalne poplave, koje su iznova usporile radove ali je novi izvođač radova na montaži "Mostogradnja AD " uspešno završila montažu čelične konstrukcije mosta i time napokon okončala ovaj veliki posao.

2 OPIS MOSTA

Kako je most već detaljnije opisan u ranijem radu ovde će biti date samo osnovne karakteristike mosta a detaljnije će se opisati i na fotografijama prikazati montaža čelične konstrukcije mosta.

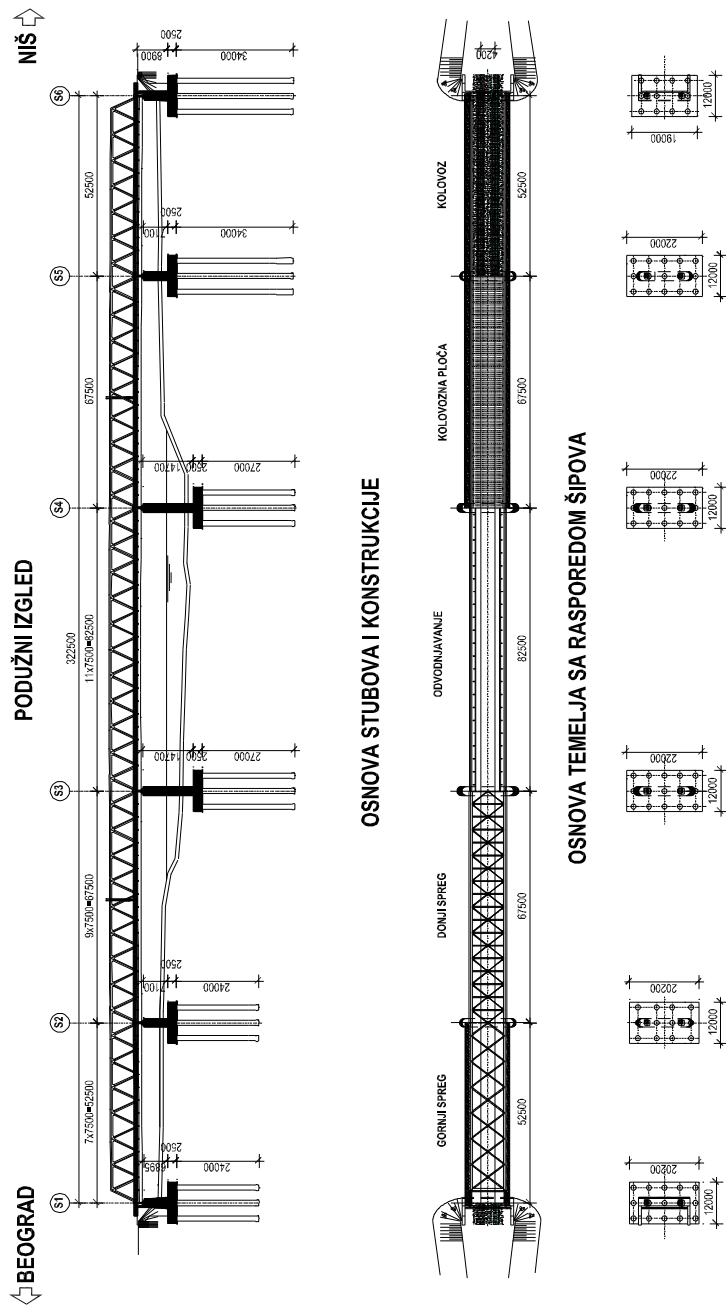
Železnički most je dvokolosečni sa kolosekom položenim u tucaničkom zastoru. Osovinski razmak koloseka na mostu je 4,20 m. Osovina koloseka na mostu je u pravcu a niveleta ima vertikalnu krivinu prečnika 30000 m.

Most premošćuje reku Veliku Moravu sa pet otvora. Konstrukcija mosta je čelična rešetkasta kontinualna greda raspona $L = 52,5 + 67,5 + 82,5 + 67,5 + 52,5 = 322,5$ m. Konstrukcija je konstantne visine $H=8,0$ m i osovinske širine $B=10,0$ m a dužina jednog polja je 7,5m. Rešetkasta ispuna je sa dijagonalama, bez vertikala. U ravni gornjeg i donjeg pojasa postoji rešetkasti spreg. Tucanički zastor se nalazi u čeličnom koritu sa dvostranim padom 2,5 % od sredine ka krajevima.

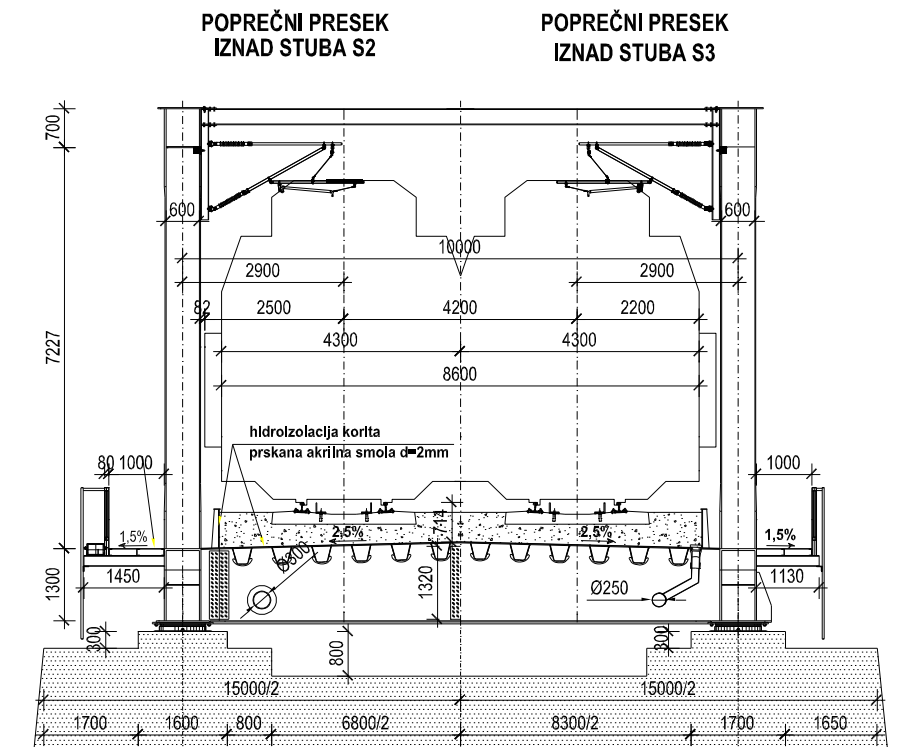
Stubovi su fundirani na HW šipovima $\varnothing 1500$ dužine do 34 m.

Ležišta na mostu su neoprenska (lončana).

Na(sl. 1) je data dispozicija mosta a na(sl. 2) poprečni presek čelične konstrukcije mosta.



Slika1 - Dispozicija mosta



Slika 2 – Karakteristični poprečni presek konstrukcije

3 OPIS KONSTRUKCIJE

Rešetkasti kontinualni glavni nosač je urađen kompletno u zavarenoj izradi. Gornji, donji pojas i dijagonale su sandučastog poprečnog preseka. Zbog stabilnosti pritisnutih štapova u eksploataciji i montaži, u sredini sanduka celom dužinom je zavareno podužno ukrućenje širine 600mm i debljine 12 i 16mm. Na svakih 3,75m, (na mestima gde se priključuje poprečni nosač) u donjem pojasu postoji dijafragma.

Zbog načina montaže koji je nešto promenjen u odnosu na glavni projekat (ukinuti su pomoćni jarmovi u sredini polja, sem prvog) u pojedinim poljima su neznatno podebljane debljine limova gornjeg i donjeg pojava i dijagonala.

Čvorni limovi su većinom debljine 26mm, a najdeblji su 36mm. Visina svih dijagonala (upravno na ravan glavnog nosača) je 600mm, a širina (u ravni glavnog nosača) je promenljiva. U ravni gornjeg pojava glavnog nosača se nalazi spreg sa rombičnom ispunom bez vertikalama, čije su dijagonale napravljene od valjanih nosača IPB1 300, visine 290 mm. Na početku i kraju sprega se nalazi portalna rigla, sandučastog preseka, dimenzija 700x600mm.

Donji spreg je takođe sa rombičnom ispunom, ali sa poprečnim nosačima kao vertikalama. Dijagonale sprega su cevi Ø168 x 7,1 mm.

Kolovoznu konstrukciju, u vidu korita koje nosi tucanički zastor, formiraju: kolovozni lim debljine 14mm sa podužnim trapeznim ukrućenjima – korubama i poprečni nosači na

međusobnom razmaku od 3,75m. Korube se nalaze na osovinskom razmaku od 715mm, visine su 285mm, širine 350mm i debljine 10mm, formirane abkantovanjem i podužno zavarene za kolovozni lim.

Kako je razmak čvorova glavnog nosača 7,5m to se poprečni nosači nalaze ne samo u čvorovima već i u sredinama polja donjeg pojasa, odnosno na svakih 3,75m. Visina poprečnog nosača se linearno menja od 1300mm uz donji pojas, do 1403mm u sredini raspona. Debljina vertikalnog lima je 16mm. Donja lamela je dimenzija 300x20mm.

Glavni noseći elementi na konstrukciji su urađeni od čelika S 235 J2G3(slabije napregnuti) i S 355 J2G3 (jače napregnuti). Svi ostali elementi (spregovi, dijafragme, ukrućenja, poklopci, pešačka staza, kanal za kablove ,penjalice i sl.) su urađeni od čelika S 235 JR G1 i S 235 JRG2.

4 TEHNOLOGIJA IZRADE

Formiranje svih konstruktivnih elemenata u radionici je izvedeno zavarivanjem. Kod sandučastih preseka spajanje horizontalnog i vertikalnog lima je obavljeno zavarivanjem sa punim provarom, ½ V šavom B kvaliteta. Kontrola 100% ultrazvukom. Svi sučeoni (poprečni) šavovi su kvaliteta B. Radiografska kontrola 100%.

Svi poprečni sučeoni šavovi su "B" kvaliteta (SRPS ISO 5817 i SRPS.C.T3.010) odnosno prema ranijoj oznaci "S" kvaliteta. Svi šavovi su izbrušeni sa lica i naličja - gde je to moguće. Svi radionički sučeoni šavovi u preseccima gde preovlađuje zatezanje su radiografski 100% kontrolisani, a gde je preovlađujući pritisak radiografska kontrola je 50%, ostatak ultrazvuk. Svi poprečni sučeoni šavovi na montaži su 100% radiografski kontrolisani. Svi ostali šavovi (K, ugaoni ½ V) su 100 % ultrazvučno kontrolisani.

5 ANTIKOROZIONA ZAŠTITA

Antikoroziorna zaštita čelične konstrukcije izvedena u svemu prema odredbama SRPS ISO 12944/2002 za kategoriju korozivnosti C4 sa vekom trajanja > 15 god. Izabran je sistem epoksid/poliuretana sa 4 premaza ukupne debljine 280 µm. Unutrašnje strane sandučastih poprečnih preseka glavnog nosača i koruba, koji su hermetički zatvoreni, zaštićene su samo osnovnim epoksidnim premazom bogatim cinkom ,debljine 40 µm. Hidroizolacija kolovozne konstrukcije je izvedena sa MMA (na bazi akrilnih smola) prskanjem pod pritiskom. Dvoslojna hidroizolaciona membrana je najmanje debljine 2mm. Na ovako izvedenu hidroizolaciju je u zoni koloseka postavljena elastomerna prostirka od reciklirane gume u cilju smanjenja buke i dodatne zaštite.

Hidroizolacija pešačke staze je izvedena nanošenjem prskanog protivkliznog premaza MMA (na bazi akrilnih smola).

Tarne površine u spojevima sa visokovrednim vijcima su zaštićene megalizacijom AlMg5 debljine 150 µm prema SRPS.U.E7.140.

6 MONTAŽA

Konstrukcija mosta je namontirana podužnim nagurivanjem konstrukcije.

Izrada montažnih komada čelične konstrukcije u radionici limitirana je gabaritom u toku transporta. Dužine montažnih komada su bile do 30 m. Kolovozna konstrukcija je izrađena iz četiri montažna dela približne širine 2,5 m. U sklopu sa kolovoznim limom su bile i korube.

Na desnoj obali ka Beogradu, na železničkom nasipu, neposredno ispred obalnog stuba S1 formirano je gradilište, plato dužine 80 m sa dve betonske staze, na koje su montirane čelične vodice, po kojima su se kretala kolica koja su se nalazila ispod svakog čvora rešetke. Staza je imala radijus vertikalne krivine isti kao čelična konstrukcija (30000 m) i na njoj su se prispeli montažni komadi dalje ukрупnjavali i vršila njihova ugradnja u konstrukciju.

Na stubovima su se nalazili privremeni oslonci, sa gornjom površinom od prohrom lima i specijalnim teflonskim klizačima preko kojih se podužno pomerala konstrukcija (sl.3). Guranje konstrukcije je vršeno sa dva mobilna hidraulična klipa (2x60 tona) koji su se ankerovali u betonsku stazu na montažnom platou i koji su se pomerali duž staze (sl.4).



Slika 3 – Privremeni oslonci sa teflonskim klizačima

Po formiranju jedne sekcije glavnog nosača pristupalo se spajanju kolovozne table sa donjim pojasom i poprečnim nosačem i montaži donjeg i gornjeg sprega (sl.5). Kompletirao se deo konstrukcije dužine 22,5 - 50m, sa isturenim "kljunom" dužine 15m, koji se sastoji samo od donjeg pojasa glavnog nosača ukrućenog donjim spregom. Prvih 45 m konstrukcije nije imalo kolovoznu tablu kako bi konzolni prepust koji se tokom nagurivanja pojavljivao bio što lakši (sl.6).

Tako sklopljena konstrukcija se podužno navlačila preko kolica na montažnoj stazi na nasipu i teflonskih klizača na privremenim čeličnim oslancima iznad stubova. Podužno pomeranje konstrukcije duž osovine mosta otpočelo je ka Nišu, dok se nije dovelo u položaj da se osloni na pomoćni jaram u prvom polju.



Slika 4 – Oprema za nagurivanje konstrukcije

Potom se na obali na već formiranu rešetku priključivao novi montažni deo za koliko je izvršeno dalje podužno nagurivanje konstrukcije. Proces se ponavljao dalje bez pomoćnih jarmova, sve dok se nije namontiralo svih 322,5m konstrukcije, odnosno dok se kljun nije oslonio na obalni stub ka Nišu. Tada je kljun nadograđen nedostajućim gornjim pojasom, dijagonalama i kolovoznom pločom u šest polja, i na taj način se formirala kompletna čelična konstrukcija mosta. Po završetku svih montažnih radova demontirani su privremeni oslonci na stubovima, montirana su neoprenska ležišta i konstrukcija spuštена na projektovanu kotu. U toku nagurivanja konzolni prepust mostovske konstrukcije iznad reke je iznosio 82 m (sl.7).

7 ISPITIVANJE MOSTA

Po završetku svih radova na mostu izvršeno je statičko i dinamičko ispitivanje mosta pod probnim opterećenjem (sl. 8). Opterećenje su sačinjavale dve železničke kompozicije. U sastavu jedne kompozicije je bila električna lokomotiva serije 461 i 8 vagona napunjenih tucanikom (FAD). Težina kompozicije je bila oko 750 tona, odnosno ukupno opterećenje koje je u pojedinim fazama merenja bilo na mostu je težilo oko 1500 tona. Statičko ispitivanje je izvršeno u 19 faza a dinamičko u 8 faza. Naponska i deformacijska slika dobijena merenjem je pravilna i usaglašena sa intezitetom i rasporedom opterećenja koje ga je izazvalo. Dinamički dijagrami su pravilni, imaju miran i ustaljen tok i male amplitude oscilovanja.



Slika 5 – Uključivanje konstrukcije na stazi



Slika 6 – Izgled "kljuna" prilikom nagurivanja konstrukcije



Slika 7 – Maksimalni konzolni prepust iznad reke



Slika 8 – Probno opterećenje na mostu

8 ZAKLJUČAK

Most, ukupne težine 2500 tona je posle nepune 4 godine kompletno završen, uspešno ispitan pod železničkim probnim opterećenjem ukupne težine 1500 tona i pušten u redovan saobraćaj krajem 2015 godine. Investiciona vrednost mosta je 11 miliona evra.

Izgradjen je ekonomičan i funkcionalan most sa veoma dopadljivom i prozračnom siluetom koja se odlično uklopila u predivan Moravski ambijent, što je bio jedan od osnovnih parametara prilikom projektovanja (sl. 9).



Slika 9 – Železnički most preko Velike Morave

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Zlatko Marković¹, Milomir Živanović², Jelena Dobrić³, Dragan Buđevac⁴

KONSTRUKCIJA ATLETSKE DVORANE U BEOGRADU

Rezime:

P-2

U ovom radu je dat prikaz projekta atletske dvorane „General Jovan Mišković“ u Beogradu koja je prva zatvorena atletska dvorana u Srbiji i okruženju. Kapacitet dvorane je 1000 gledalaca i ispunjava sve uslove za domaća i međunarodna atletska takmičenja. Glavni noseći sistem čine plitki čelični okvirni nosači raspona 54,8 m koji su zglobno oslonjeni na AB temelje. Značajne horizontalne reakcije prihvaćene su pomoću zatega koje su ankerisane u armirano-betonsku podnu ploču. Tribine, administrativni i tehnički blok su realizovani kao armirano-betonska konstrukcija. Atletska dvorana je projektovana i izvedena u periodu od 2011. do 2016. godine.

Ključne reči: atletska dvorana, čelična konstrukcija, okvirni nosač, veliki raspon.

ATHLETIC HALL STRUCTURE IN BELGRADE

Summary:

This paper presents the design project of athletic hall "General Jovan Mišković" in Belgrade which is the first closed athletic hall in Serbia and the region. The capacity of the hall is 1000 spectators and it meets all requirements for national and international athletic competitions. The main structural system consists of shallow steel frames with span of 54.8 m which are hinged supported on reinforced concrete foundations. Significant horizontal reaction forces are transmitted by tensioned reinforcement elements which are anchored in the concrete ground slab. The tribune, administrative and technical block were designed as a reinforced concrete structure. Athletic hall was designed and assembled during 2011-2016.

Key words: athletic hall, steel structure, frame structure, long span.

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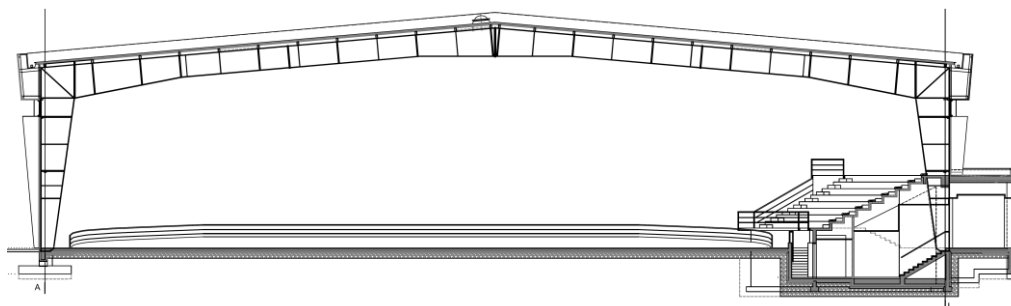
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1 UVOD

Nova atletska dvorana u Beogradu predstavlja prvu i jedinu zatvorenu atletsku dvoranu, ne samo u Republici Srbiji već i čitavom regionu. Kompleks Atletske dvorane nalazi se na Banjici, u okviru kasarne "General Jovan Mišković", neposredno uz jedan od tri ulaza u kasarnu, a ima direktan prilaz iz saobraćajnice Bulevar oslobođenja. Za potrebe izgradnje objekta izvedeno je izmeštanje dela Banjičkog potoka u osovину požarnog puta i zasecanje škarpe terena duž granice parcele. Atletska dvorana se nalazi sa leve južne strane pristupnog puta. Prostor oko objekta je sa tri strane popločan tako da omogućava pristup požarnog vozila, a kolsko-pešačka saobraćajnica se završava asfaltiranom saobraćajnicom širine 3,5 m, sa okretnicom uz zadnju fasadu objekta. Ispred objekta je manji parking za zaposlene u administraciji, kapaciteta 14 mesta, a sa desne, severne strane pristupnog puta nalazi se parking prostor za posetioce, kapaciteta 143 parking mesta. Neposredno uz kompleks Atletske dvorane nalazi se otvoren sportski teren za atletiku čime je ostvarena ideja o povezivanju ova dva sadržaja koji su u funkciji atletske sportske discipline.

Atletska dvorana sa pripadajućim administrativno-sportskim sadržajima je ukupne bruto površine 7202,7 m² sa gabaritom 114,25 x 62,0 m. Dvorana je kapaciteta 1000 gledalaca a funkcionalno je prilagođena potrebama budućih korisnika Vojne akademije i Atletskog saveza Srbije. Objekat se sastoji iz nekoliko funkcionalnih celina: atletske dvorane, administrativnog dela i tehničkog bloka.



Slika 1 – Karakterističan poprečni presek kroz tribine

Atletska dvorana poseduje fiksnu atletsku stazu sa četiri ovalne trake, projektovanu po standardima za dvoranske staze dužine 200 m, u čijem središnjem delu su staze za trčanje na 50 i 60 m sa osam traka i četiri borilišta: za skok s motkom, skok u dalj-troskok, skok u vis i bacanje kugle. Objekat je u potpunosti usklađen sa standardima Svetske atletske organizacije i svojim sadržajima zadovoljava uslove za organizovanje nacionalnih i međunarodnih takmičenja. U okviru objekta takođe su predviđena dva magacinska prostora: za opremu i rekvizite i za elektronsku opremu, kao i ekonomat. Glavni ulaz za posetioce je lociran sa severne glavne fasade objekta i nezavisan je u odnosu na službene ulaze za sportiste i administraciju. Uz ulaz su predviđene prostorija za prodaju karata, garderoba i bar. Tribinama se pristupa kroz dugački hol preko dva stepeništa. Naspram stepeništa su dva evakuaciona izlaza za publiku.

Glavni Projekat objekta je realizovan u prvoj polovini 2011. godine u okviru projektne organizacije „Centroprojekt d.o.o. Beograd“. Naručilac Glavnog Projekta je Ministarstvo

odbrane Republike Srbije, a korisnik, Ministarstvo omladine i sporta Republike Srbije i Atletski savez Srbije.

2 IZBOR KONSTRUKCIJNOG REŠENJA

U početnoj fazi realizacije ovog projekta sprovedena je komparativna analiza više varijantnih rešenja glavne noseće konstrukcije objekta, kako bi se dobilo optimalno rešenje. Osnovni zahtevi koji su postavljeni pred projektante su bili da se dobije jednostavan i racionalan objekat koji ispunjava sve funkcionalne zahteve predviđene projektnim zadatkom, a koji su prvenstveno vezani za uslove svetske atletske organizacije. Polazeći od toga, glavna noseća konstrukcija je takođe trebalo da zadovoljava pomenute uslove kako u pogledu smanjenja investicionih troškova, tako i u pogledu redukcije troškova održavanja. Raspon od oko 55 m je diktiran zahtevanim prostorom za održavanje atletske takmičenja i smeštanjem tribina za 1000 gledalaca sa jedne strane atletske borilišta. Imajući u vidu sve pomenute uslove analizirano je nekoliko prikladnih varijantnih rešenja: glavni nosač sa kosim zategama izvan gabarita objekta, tropojasni rešetkasti okvirni nosač, tropojasni rešetkasti krovni nosač sistema proste grede na uklještenim stubovima i obostrano zglobno oslonjen okvirni nosač.

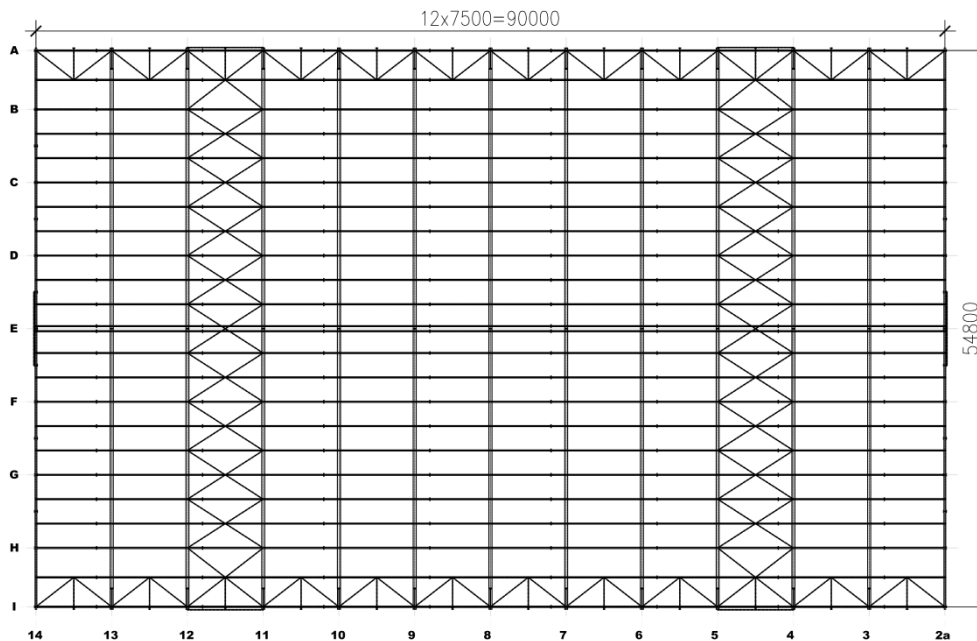
Iako možda atraktivnija, prva dva rešenja su odbačena kao ekonomski nepovoljnija sa stanovišta osnovnih zahteva Investitora, odnosno investicionih troškova, ali i sa stanovišta održavanja, odnosno trajnosti. Naime, preliminarnim analizama dobijeni su veći utrošci čelika u odnosu na rešenje sa ramovskom konstrukcijom. Rešenje sa ramovskom konstrukcijom je takođe povoljnije sa stanovišta održavanja i trajnosti. Kod nosača sa kosim zategama, pored veće cene čelične konstrukcije, nedostatak su i prodori zatega kroz krovnu ravan koji uvek predstavljaju potencijalno opasna mesta za procurivanje, što može bitno da ugrozi funkcionalnost i trajnost objekta. Varijante sa rešetkastim nosačima, pogotovo u slučaju proste grede, zahtevaju znatno veću građevinsku visinu, što pored povećanja troškova fasada bitno povećava zapreminu objekta, a samim tim i troškove održavanja. Osim pomenutih rešenja, analiziran je i nesimetričan okvirni nosač sa jednom kraćim stubom koji se oslanja na AB tribine, ali je on imao znatno nepovoljniji statički odgovor, praćen pojavom nesimetričnih pomeranja, većih horizontalnih reakcija u vrhu AB tribina, kao i momenata savijanja na mestu veze krovnog nosača i dužeg stuba, u poređenju sa simetričnim okvirnim nosačem. Imajući u vidu prethodno navedene prednosti, simetričan dvozglojni okvirni nosač se nametnuo kao najpovoljnije rešenje. Oblikovanjem unutrašnje siluete (intradosa) ramovskog nosača, takođe je postignut i prihvatljiv estetski utisak, što je takođe bitno, jer je konstrukcija vidna u enterijeru.

3 ČELIČNA KONSTRUKCIJA

Osnova čelične konstrukcije atletske dvorane je pravougaonog oblika, osovinskih dimenzija: 54,8 x 90,0 m. Osnovni raster u podužnom pravcu je 7,5 m ($12 \times 7,5 = 90,0$ m), dok je u poprečnom pravcu raster promenljiv u skladu sa arhitektonskim zahtevima i iznosi: $5,8 + 6 \times 7,2 + 5,8$ m (54,8 m). Objekat je prizeman sa krovom na dve vode. Nagib krova je 5° , a krovni pokrivač i fasadna obloga su od sendvič panela. Osnova krovne konstrukcije objekta prikazana je na slici 2.

Glavni noseći sistem čine dvozglojni okvirni nosači raspona 54,8 m (slika 3). Za poprečne preseke konstrukcije su usvojeni zavareni monosimetrični I nosači promenljive geometrije. Promena geometrije stubova i krovnih nosača ostvarena je promenom visine preseka i

promenom dimenzija njegovih delova (nožica i rebra), a sve u skladu sa ekstremnim vredostima unutrašnjih sila u odabranim karakterističnim poprečnim presecima. Velike vrednosti uticaja, uslovile su usvajanje visokih i vitkih poprečnih preseka koji su osetljivi na efekte izbočavanja usled dejstva normalnih i smičućih napona.



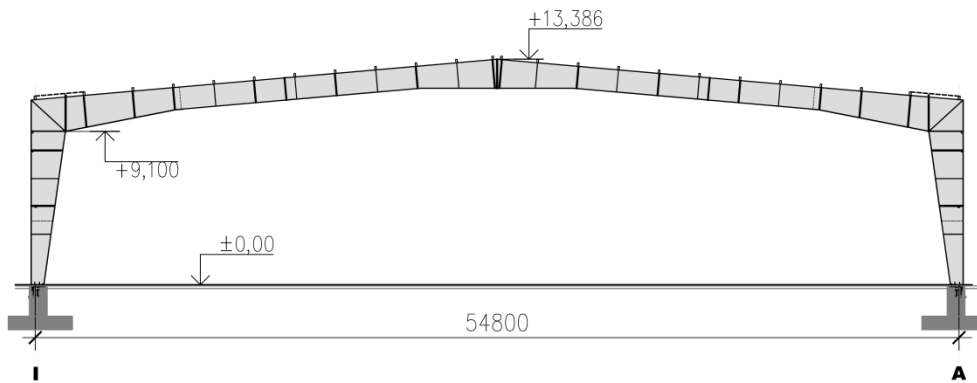
Slika 2 – Osnova krovne konstrukcije objekta

Visina rebra krovnih rigli se kreće od 2000 mm na mestu veze sa stubovima, preko 1300 mm u prelaznim zonama, do 1700 mm u sredini raspona. Debljina rebra je 10 mm, odnosno 12 mm na mestu vute kod veze sa stubom. Visina rebra stubova je 2000 mm u gornjem delu uz vezu sa riglom, dok je na mestu zglobnog oslonca 500 mm. Debljina rebra u gornjem delu je 14 mm, a u donjem 10 mm.

Sa druge strane, prateći promenu dijagrama momenata savijanja u nosaču, koji je za najveći broj kombinacija opterećenja davao zatezanje u spoljašnjoj zoni preseka, u cilju povećanja efikasnosti poprečnog preseka u zoni pritiska usvojena je veća debljina unutrašnjih nožica preseka. Nožice krovnih rigli su 300 mm širine, spoljašnja nožica je 15 mm, a unutrašnja 25 mm na mestu vute, odnosno 20 mm na preostalom delu. Kod stubova je spoljašnja nožica 300x15 mm, a unutrašnja 300x25 mm u zoni maksimalnih uticaja, odnosno 300x20 mm na preostalom delu. Rebra krovnih rigli i stubova su ukrućena poprečnim ukrućenjima u vidu obostranog ravnog lima na mestima rožnjača, odnosno horizontala u podužnim zidovima.

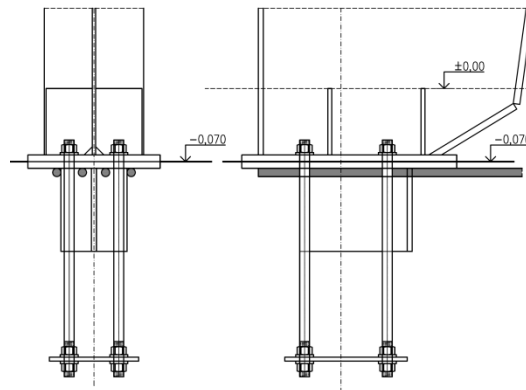
Kontrola stabilnosti na izbočavanje poprečnih preseka urađena je primenjujući koncept efektivne širine. U slučaju spoljašnje pritisnute zone preseka, krovni nosači su bočno pridržani rožnjačama, a stubovi horizontalnim nosačima koji su povezani sa čvorovima vertikalnih spregova u podužnim zidovima objekta. Unutrašnji pritisnuti pojasevi su bočno pridržani kosnicima postavljenim u nizu proračunom određenih tačaka. U zoni tribina, detalj veze glavnih stubova i AB tribinskog nosača projektovan je i konstruktivno oblikovan tako da

obezbedi dilataciju, odnosno nesmetana pomernja stuba u ravni okvirnog nosača, a da spreči pomeranja i obrtanja izvan ravni, odnosno da omogući bočno pridržavanje stuba AB konstrukcijom tribina. Rigla glavnog nosača je podeljena na četiri montažna segmenta. Montažni nastavci su izvedeni pomoću čeonih ploča i prednapregnutih visokovrednih zavrtneva klase čvrstoće 10.9. U cilju provere kompaktnosti i homogenosti materijala debelih čeonih ploča, odnosno sprečavanja moguće pojave lamelnog cepanja materijala, urađena je ultrazvučna kontrola. U jednom slučaju zabeležena je pojava grešaka nalepljivanja koje su nastale u postupku proizvodnje limova, pa je taj lim zamenjen.



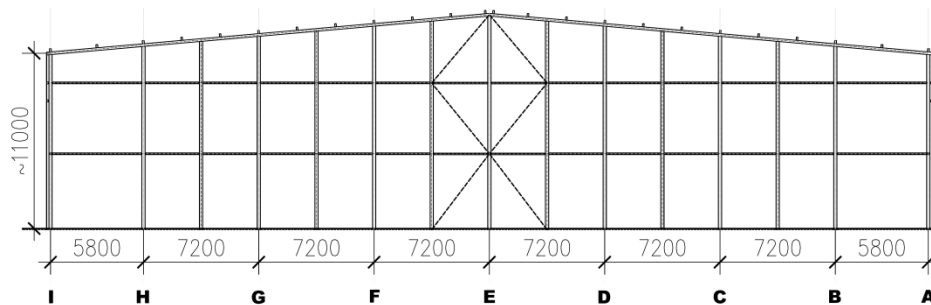
Slika 3 – Glavni nosač

Veza stubova sa AB temeljima je ostvarena ankerima klase čvrstoće 8.8 i čeličnim trnovima. Detalj veze stuba sa AB temeljom oblikovan je tako da se pomoću rebraste armature prihvati značajna horizontalna reakcija okvirnog nosača (slika 4). To je jedan od vitalnih delova konstrukcije koji veoma značajan za globalno ponašanje konstrukcije. Zbog specifičnosti veze armature sa stubom kontrola naprezanja u ovom detalju sprovedena je primenom 3D konačnih elemenata.



Slika 4 – Detalj veze stuba sa temeljom – prijem horizontalne reakcije

Za rožnjače su usvojeni kontinualni nosači izvedeni od šupljih hladno-oblikovanih profila pravougaonog poprečnog preseka (RHS200/100/4 mm). Na delu krova uz venac rožnjače su na nešto većem međusobnom rastojanju. Zbog veće pripadajuće širine, ali i učešća u radu podužnih krovnih spregova za ove rožnjače su usvojeni veći poprečni preseci (RHS200/120/6,3 mm).



Slika 5 – Konstrukcija kalkanskog zida

Kalkansku konstrukciju objekta čine fasadni stubovi, poklapača, horizontale i vertikalni spreg (slika 5). Fasadni stubovi su sistema proste grede, maksimalne visine 13,5 m. Kako je arhitektonskim rešenjem predviđena horizontalna orijentacija fasadnih panela, horizontale nemaju ulogu fasadnih rigli, već samo povezuju kalkanske stubove sa vertikalnim spregom i smanjuju njihovu dužinu izvijanja u ravni zida.

Budući da je razmak između glavnih stubova veći od 6,0 m i da je predviđena horizontalna orijentacija fasadnih panela, u sredinama glavnih rastera postavljeni su fasadni stubovi. Statičkog su sistema proste grede visine 11,0 m, odnosno 6,6 m u zavisnosti od toga da li se oslanjaju na AB temelje u jednom, odnosno na AB nosače tribina u drugom podužnom zidu.

Prostorna stabilnost objekta ostvarena je pomoću podužnih i poprečnih krovnih spregova, kao i vertikalnih spregova u podužnim i kalkanskim zidovima. Spregovi su sa ukrštenim dijagonalama od hladno-oblikovanih šupljih profila.

Za osnovni materijal rožnjača i glavnih okvirnih nosača usvojen je čelik S355JR, dok su svi ostali konstruktivni elementi izvedeni od čelika S235JRG2.

4 ARIMARNO-BETONSKE KONSTRUKCIJE I FUNDIRANJE

Kao što je već pomenuto u opisu čelične konstrukcije, glavni noseći sistem je pun limeni nosač okvirnog sistema sa zategom u armirano-betonskoj podnoj ploči borilišta koja je u osi A direktno vezana za temelj glavnog rama. Veza temelja glavnog rama u osi I i zatege ostvarena je posredno preko ramova tribine, koje čine potporni zid sa kontraforom, stub u osi H, udvojeni stubovi u osi I i grede koje ih povezuju. Sekundarni fasadni stubovi za nošenje fasade se u osi I oslanjaju na gredu na vrhu tribine, dok se u osi A oslanjaju na veznu gredu temelja. Armirano-betonska konstrukcija tribina i kosih greda gledališta izvedena je između osa 13-3/I-H. Tribine počinju od kote galerije +1,20 m i dosežu visinsku kotu +4,40 m.

Klupice tribina izvedene su kao montažni prefabrikovani armirano-betonski L-nosači, debljine rebra 12 cm, raspona od 7,50 do 8,00 m, dok su u dnu i vrhu tribina izvedene armirano-betonske ploče livene na licu mesta, debljine 15 cm. Između borilišta i tribina nalazi

se armirano-betonski zid sa kontraforima dimenzija 50/40 cm. Poprečne kose ili kolenaste grede, predviđene za oslanjanje L-nosača, livene su na licu mesta. One su udvojene, dimenzija 2x30/50 cm na delu od ose H do ose I, dok su na delu od ose I izvedene grede širine 50 cm promenljive visine 45 i 75 cm. Grede se oslanjaju na stubove u osi H i kontrafore u zidu na strani prema borilištu, a na drugom, gornjem kraju na stubove u osi I sa njima čine poprečne ramove. Osovinski razmak ovih ramova iznosi 7,50 m.

Stubovi u osi I su udvojeni, poprečnog preseka 30/80 cm, dok su svi ostali stubovi poprečnog preseka 50/50cm. Svi armirano-betonski zidovi su debljine 20 cm. Poprečni ramovi su u podužnom pravcu međusobno povezani armirano-betonskim gredama poprečnog preseka 50/60 cm u osi I. Delovi tribina postavljeni pod uglom od 30° oslanjaju se na 2 kosa rama i armirano-betonski zid. Iza tribina se nalazi hol za publiku, čija je tavanica monolitna armirano-betonska ploča debljine 15 cm. Tavanica hola za publiku se preko greda u osi J oslanja na stubove dimenzija 50/50 cm. Stepenice između suterena i prizemlja, kao i stepenice ka galeriji su armirano-betonske, na kolenastoj ploči.

Administrativni blok projektovan je kao skeletna armirano-betonska konstrukcija livena na licu mesta. Tavanica iznad administrativnog dela projektovana je kao livena armirano-betonska ploča debljine 20 cm sa kapitelima 300/300/40 cm i ivičnim gredama 20/80 cm. Grede i kapiteli se oslanjaju na stubove dimenzija 40/40 cm.

Tehnički blok projektovan je takođe kao skeletna armirano-betonska konstrukcija livena na licu mesta. Tavanica iznad tehničkog dela projektovana je kao livena armirano-betonska ploča debljine 22 cm koja se oslanja na armirano-betonske grede dimenzija 30/50 cm, koje se oslanjaju na stubove 30/30 cm.

Objekat je plitko fundiran na armirano-betonskim temeljima samcima debljine 30 i 50 cm povezanim veznim gredama dimenzija 30/50 cm u oba ortogonalna pravca i temeljnim trakama. Ispod svih temelja izveden je tampon sloj od betona MB 10 debljine 5 cm. Podna ploča je laka armirano-betonska debljine 15 cm. Dubina fundiranja je na koti -1,60 m, osim tehničkog i administrativnog aneksa gde je dubina fundiranja -1,00 m. Podtribinski deo koji je ukopan ima temelje na dubini -2,67 m. Temelji su dimenzionisani u skladu sa "Geotehničkim elaboratom za izradu glavnog projekta atletske dvorane u kasarni "General Jovan Mišković" u Beogradu", koji je izradio Geoput d.o.o, Beograd u januaru 2011. godine.

5 REALIZACIJA PROJEKTA

Izgradnja objekta je počela u januaru 2013. godine. Sama izgradnja, prema planiranim budžetskim sredstvima, je bila podeljena u dve faze. U prvoj fazi je izvedena gotovo kompletna konstrukcija, a u drugoj fazi završni radovi i instalacije. Izabrani izvođač radova prve faze bila je GP „Morava in“ d.o.o Kruševac, a druge faze GPD „KEJ“ d.o.o Valjevo. Radovi prve faze otpočeli su početkom 2013. godine, u periodu nepovoljnom za zemljane radove, posebno u zoni Banjičkog potoka zbog povišenog nivoa podzemnih voda i mogućeg prodiranja vode u temeljnu jamu. Zbog nemogućnosti prilaza zoni Banjičkog potoka u vreme izrade geoistražnih radova, projektanti i izvođači radova bili su uskraćeni za preciznije podatke o karakteristikama tla u zoni potoka. Iako je projektom predviđena veća dubina fundiranja u zoni potoka ispostavilo se da je nosivo tlo znatno ispod projektovane kote. Projektant konstrukcije, izvođač radova, nadzor i obrađivač geomehaničkog elaborata predložili su zamenu tla u toj zoni, što je prihvaćeno od strane investitora i ubrzo realizovano. Ovo su bile praktično jedine značajnije izmene glavnog projekta.

Posle završetka prve faze izgradnje nastao je zastoj u realizaciji objekta. Postojala je bojazan da montirana čelična konstrukcija na otvorenom može biti oštećena od atmosferskih uticaja. Na sreću zastoj u izgradnji nije trajao predugo, pa je ceo objekat pokriven na samom početku druge faze izgradnje. Druga faza izgradnje završena je početkom 2016. godine, kada je objekat i svečano otvoren.

Takođe treba napomenuti da je dvorana građena po preporukama i uz verifikaciju predstavnika evropske atletske federacije, koji su u nekoliko navrata obišli gradilište i verifikovali bitne elemente atletskih borilišta.



Slika 6 – Tribinski nosači u izgradnji



Slika 7 – Montaža kalkanske čelične konstrukcije



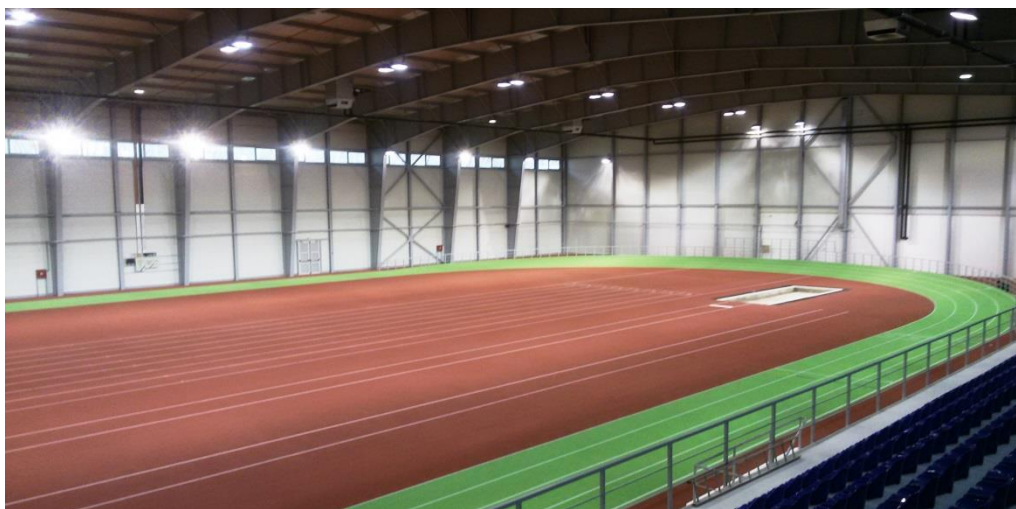
Slika 8 – Montaža glavnih stubova ramovskih nosača na strani tribina



Slika 9 – Montaža glavnih nosača i rožnjača



Slika 10 – Izgled konstrukcije neposredno pred završetak radova



Slika 11 – Izgled konstrukcije i borilišta



Slika 12 – Pogled sa tribina



Slika 13 – Spoljašnji izgled

6 ZAKLJUČAK

Pravilnim izborom glavnog konstruktivnog sistema kroz komparativnu analizu nekoliko varijantinih rešenja, kao i pravilnim oblikovanjem poprečnih preseka svih nosećih elemenata i adekvatnim izborom kvaliteta čeličnog materijala, postignuta je izuzetno mala težina čelične

konstrukcije za ovakve rasponne (62 kg/m^2), a istovremeno je postignut i veoma povoljan vizuelni efekat noseće čelične konstrukcije koja je vidna u enterijeru. Primer ove atletske dvorane pokazuje kako se ispravnim analitičkim inženjerskim pristupom i bez nepotrebnog rasipanja sredstava može doći do veoma uspehlikih rešenja, jer ne retko lepota i sklad leže upravo u jednostavnosti.

OSNOVNI PODACI O PROJEKTU

INVESTITOR: Ministarstvo odbrane Republike Srbije

PROJEKTNA ORGANIZACIJA: „Centroprojekt“ d.o.o. za proizvodnju, promet i usluge, Beograd

PROJEKTANTSKI TIM: Prof dr Zlatko Marković, dipl. inž. građ.
odgovorni projektant za čeličnu konstrukciju
Milomir Živanović, dipl. inž. građ.
odgovorni projektant za armirano-betonsku konstrukciju
Doc. dr Jelena Dobrić, dipl. inž. građ.
projektant čelične konstrukcije
Prof. dr Dragan Buđevac, dipl. inž. građ.
projektant čelične konstrukcije

IZVOĐAČI

FAZA 1: GP "MORAVA in" d.o.o. Kruševac, podizvođač za čeličnu konstrukciju „Čelik“ a.d. Bački Jarak

FAZA 2: GPD "KEJ", Valjevo

INVESTICIONA

VREDNOST OBJEKTA: 440.000.000 RSD



Jelena Dobrić¹

PONAŠANJE PRITISNUTIH ELEMENATA SLOŽENOG POPREČNOG PRESEKA OD NERĐAJUĆEG ČELIKA

Rezime:

P-3

Ovaj rad ukratko prikazuje važne segmente opsežne eksperimentalne, numeričke i analitičke analize ponašanja pritisnutih elemenata višedelnog poprečnog preseka od nerđajućeg čelika, sa fokusom na njihovu nosivost usled fleksionog izvijanja oko nematerijalne ose. Istraživanje je sprovedeno na Univerzitetu u Beogradu na Građevinskom fakultetu i publikovano u doktorskoj disertaciji „Ponašanja centrično pritisnutih elemenata složenog poprečnog preseka od nerđajućih čelika“. Primarni cilj ovog istraživanja je definisanje proračunskih izraza i preporuka za dimenzionisanje hladno-oblikovanih višedelnih elemenata sa blisko postavljenim pojasevima opterećenih na pritisak, obzirom na njihovo odsustvo u aktuelnom evropskom propisu za proračun konstruktivnih elemenata od nerđajućeg čelika EN 1993-1-4.

Ključne reči: nerđajući čelik, pritisak, višedelan presek, eksperiment, nosivost.

BEHAVIOUR OF BUILT-UP STAINLESS STEEL MEMBERS SUBJECTED TO AXIAL COMPRESSION

Summary:

This paper briefly presents important segment of the extensive experimental, numerical and analytical analysis of the behaviour of compressed stainless steel built-up member, emphasizing their flexural buckling capacity with respect to the minor cross-sectional axis. The research was carried out at the University of Belgrade, Faculty of Civil Engineering and was published in PhD Thesis “Behaviour of built-up stainless steel members subjected to axial compression”. The primary aim of this research is to define the predictive analytical expressions and recommendations for design of closely spaced cold-formed built-up stainless steel members subjected to compression load, taking into account their absence in the current European standard for stainless steel structural design EN 1993-1-4.

Key words: stainless steel, compression, built-up section, experiment, resistance.

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1 INTRODUCTION

Stainless steel belongs to the group of contemporary, sustainable and renewable structural materials whose recognizable visible characteristics are high corrosion resistance and superior aesthetic expression. The wider use of stainless steel in conventional structures is not limited only by its high cost but also by lack of design guidance which fully exploits all structural performance of this material and makes it economically acceptable. Developments over the last decade, both in available alloys and attitudes to durability, offer a new opportunity for stainless steels to be considered as important structural materials.

The structural response of stainless steel differs from that of carbon steel because stainless steel is highly non-linear material with an early deviation from linear elastic behaviour and a large strain-hardening domain. This can have different implications on the buckling behaviour of compressed members in compare to equivalent carbon steel members and to lead to overly conservative results.

Built-up members have a traditional application in civil engineering for compressed structural elements, mostly as columns and members of lattice structures. They are formed with mutual discontinuous connecting of two or more parallel chords, using bracing members: battening or lacings. The built-up members with closely spaced chords connected through packing plates, or those with chords in direct contact, represent the simplest form for this type of elements. The interconnection should be designed to provide a transmission of the longitudinal shear forces and to resist slipping between the chords. The overall shear flexibility of the built-up member depends on the properties of interconnections and their numbers along the chord members. Eurocode for stainless steel structures EN 1993-1-4 [1] does not provide explicit recommendations for design compressed built-up members and suggests implementation of design rules which are given in Eurocode for carbon steel EN 1993-1-1 [2] and EN 1993-1-3 [3]. Besides, there are no available results of experimental investigations of such elements. In this context, an extensive research addressing the load carrying capacity of cold-formed built-up members with chords in direct contact was conducted at the Faculty of Civil Engineering, University of Belgrade during 2012-2013. The investigation included experimental tests of material coupon, stub column specimens and structural built-up specimens in order to generate data on the key aspects of the structural design. The tested material was austenitic stainless steel grade EN 1.4301. For the flexural-buckling tests, the built-up specimens were formed by two discontinuously and directly connected C-section chords oriented back-to-back, forming an I-shaped cross-section. In parallel with the experimental study a numerical analysis was carried out to investigate further their structural behaviour. The main aim of this research is to define design recommendations which will provide the basis for complementing and advancement of existing stainless steel design guidelines. All results of this investigation are fully reported in [4].

2 EXPERIMENTAL STUDY

2.1 MATERIAL PROPERTIES

Distinctive properties of stainless steels are reflected in the nonlinear stress-strain relationship, prominent ductility, asymmetry and anisotropy of material and strain-hardening due to cold forming. In order to study the asymmetry and anisotropy of stainless steel and the

influence of cold working to the material response, two tests series with total of 24 coupons were performed. The first series included flat coupons taken from the steel sheet material. Both tensile and compressive material properties were obtained by means of coupon tests in longitudinal and transverse to the rolling sheet direction. The second series included the flat coupons used from the faces and corner regions of the final press-braked C-section. The key material parameters: 0,2% proof stress $f_{0.2}$, 1,0% proof stress $\sigma_{1.0}$, ultimate tensile stress σ_u , strain corresponding to the ultimate tensile stress ε_u , strain at fracture ε_f , and Young's modulus E , were specified. Values of the strain hardening exponents n , m , and $n_{0.2,1.0}$ were determined using predictive material models [5],[6],[7] and regression analysis method, providing that the experimental curve closely matches equivalent analytical material model. By way of example, Figure 1 shows nominal stress-strain curves obtained by testing longitudinally oriented flat coupons under compression (LC) and their average curve. Figure 1 is complemented with average values of measured material properties and the strain hardening exponents.

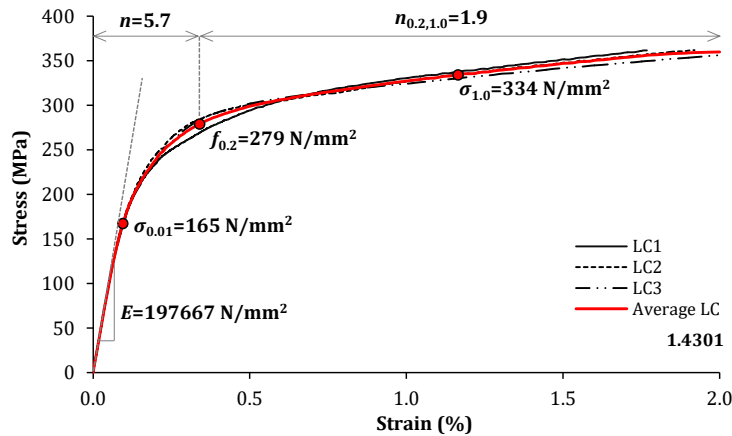


Figure 1 - Longitudinal compressive stress–strain curves

2.2 STUB COLUMN TEST

A total of 4 press-braked C-section specimens were tested in pure axial compression to assess local behaviour of the cross-section and to confirm Class 3 limit for the cross-section in compression (Figure 2). To prevent influence of the overall buckling mode, the length of the specimens was chosen to be less than three times the width of the widest cross-section dimension and the relative slenderness of specimens to be lower than the limiting slenderness according to EN 1993-1-4 [1]. They were tested between end plates which were prevented from rotating during the test. All specimens failed by local buckling. Buckling was localised in the middle part of the specimen height with concave web buckling and convex flange buckling. Obtained results confirmed the cross-section capability of reaching conventional yield stress prior to onset of local buckling and also were utilized to assess appropriateness of slenderness limit for class 3. In addition, the test values of the C-section ultimate resistance were used to verify the predictive values according to stainless steel European [1] and American design standards [8] and Continuous Strength Method [9].



Figure 2 – The nominal dimension of the specimen cross-section and test setup

2.3 FLEXURAL BUCKLING TEST

Flexural buckling tests were the most significant and the most extensive part in experimental research. Two series with a total of 36 specimens were experimentally tested in pure axial compression in order to obtain ultimate resistance data for a variety of built-up columns. The specimens were formed from two individual press-braked C chords having nominal dimensions 100x40x4 mm and internal corner radius of 8 mm. The individual members were oriented back-to-back and directly and discontinuously connected by groove welds or by bolts.

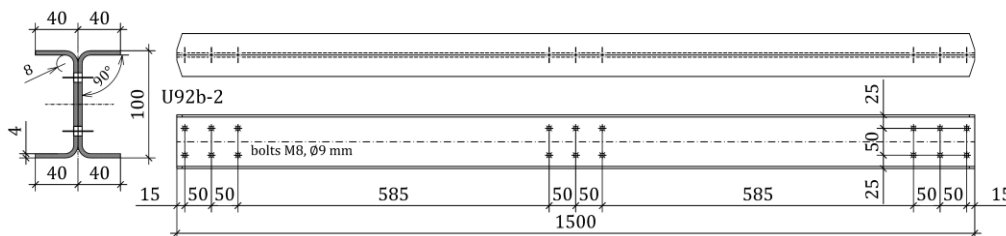


Figure 3 - View and cross section of built-up columns U92b-2

Three parameters with the most significant influence on flexural buckling resistance were varied: member length, type of connection between chords and spacing between interconnections. In case of the welded interconnections, the nominal length of welds was 100 mm. A bolted connection, which has been designed with 6 bolts M8 class 8.8. The nominal lengths of the specimens were 1500 mm and 3000 mm. The number of modules between interconnections was two, three and five. It should be noted that the specimens with interconnections at the ends and at one-half of the member length deviate from the recommendations given in EN 1993-1-1 [2] and AISI S100-2007 [10], regarding minimum number of modules between interconnections. Figure 3 shows nominal dimensions of built-up specimens U92b-2 with bolted interconnections at the ends and in the middle height of specimen.

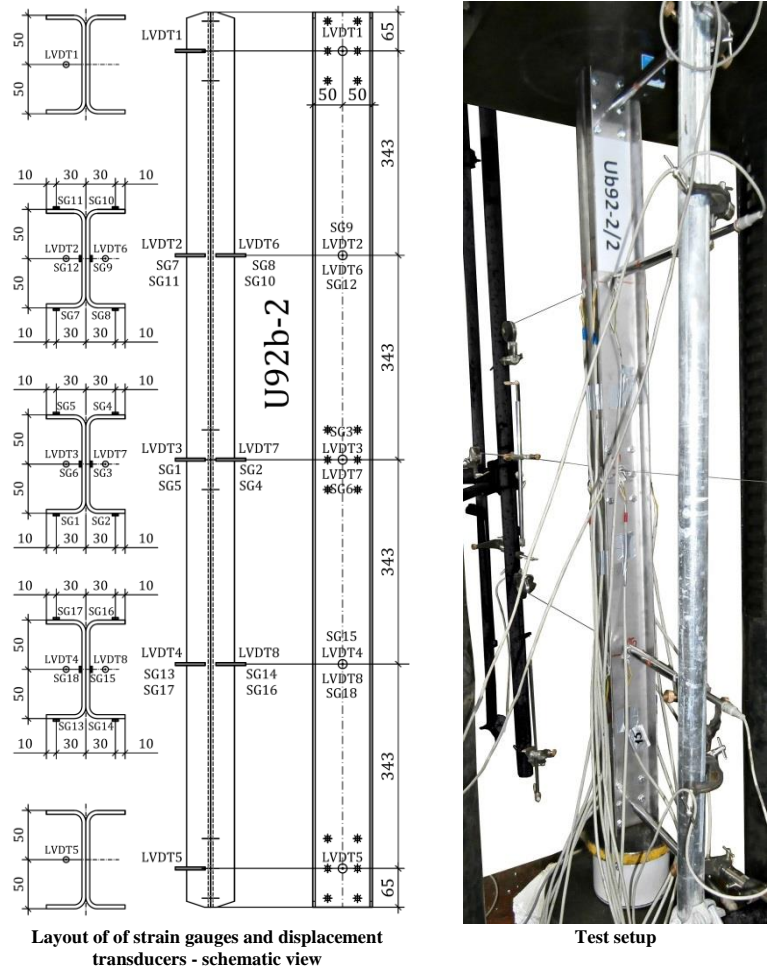


Figure 4 - Test setup for flexural buckling tests

Geometrical imperfections of the specimens were measured prior to testing their buckling resistance at the Metrological laboratory for angle and length calibrations at the Faculty of Civil Engineering, University of Belgrade. Deviations of the centroid axis from the ideally straight longitudinal axis connecting the centroid of the both specimen ends were found by using a mechanical levelling instrument and the scale. Observation of the results revealed that the geometric imperfections of the specimens were considerably lower than the fabrication tolerance $L/750$ proscribed by the EN 1090-2 [11].

The flexural buckling tests of specimens were carried out at the Institute for testing of materials in Belgrade. The specimens were set up vertically in the hydraulic testing machine. During the test, the lateral displacements of the specimens in the expected buckling plane were monitored using the displacement transducers. The longitudinal strain was measured using the

strain gauges. In case of specimens with two modules between interconnections, strain gauges were also attached at the mid-distance between interconnections, for the purpose of monitoring of the individual chord member behaviour. Figure 4 shows the detailed instrumentation layout for flexural buckling testing of specimens U92b-2.

The dominant mode of failure in all specimens was the overall flexural buckling about minor axis. The separate lateral displacement or the failure of the chords within built-up cross-section was not recorded. The interaction between elastic buckling of cross-section and overall stability of built-up specimen was not observed. Typical failure mode for the bolted specimens of the series U92 is shown in Figure 5.



Figure 5 - Typical failure mode of bolted specimens U92

Nominal curves of load-strain and load-lateral deflection were obtained at measuring positions across the specimens by direct reading of data from strain gauges and LVDTs. Figure 6 shows the increase of strains and lateral deflection of specimen U92b-2-4 in the function of the applied load during testing. It can be noticed that, in the initial part of stress range, a uniform application of the compression load into all elements of the cross-section was achieved to the greatest extent.

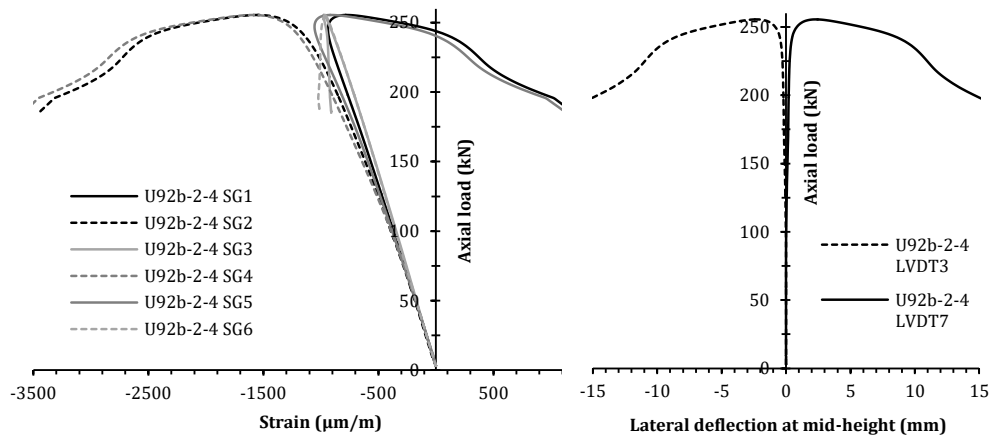


Figure 6 - Curves of flexural buckling tests of specimen U92b-2-4

Table 1 summarizes average values of test results for specimen group U92b-2: $N_{b,u}$ is the ultimate flexural buckling load about the minor axis; E is the modulus of elasticity, E_t is the

tangent modulus, $\sigma_{b,u}$ is the ultimate buckling stress and A is the nominal cross-sectional area of the specimens.

Table 1: Results of flexural buckling tests of specimens U92b-2

Specimens of group U92b-2	$N_{b,u}$ (kN)	E (N/mm ²)	$\sigma_{b,u} = N_{b,u}/A$ (N/mm ²)	E_t (N/mm ²)
Average value	241.0	206950	184.3	92319

Average value of ultimate flexural buckling stress $\sigma_{b,u}$ is between average value of proportionality limit $\sigma_{0.01}$ of 165 N/mm² and average value of 0.2% proof stress $f_{0.2}$ of 279 N/mm², which obtained by longitudinal compressive coupon test [4]. The ratio of tangent modulus and modulus of elasticity is 0.45, which indicates that the ultimate flexural buckling resistance of specimen U92b-2-4 was achieved in the non-linear stress domain.

2.4 COMPARISONS WITH DESIGN RESISTANCE

When considering the buckling resistance of compressed stainless steel members, it is necessary to take into account the effect of the low proportional limit, residual stresses and the gradual yielding behaviour of stainless steel. Average, experimental values of the ultimate flexural buckling resistance of specimens were compared with the design, ultimate values, according to the recommendations given in the existing Eurocode [1], [2] and American Specification [8], [10] for stainless and carbon steel. It is demonstrated that the experimental values are noticeably higher in comparison with the predictive design values. Deviations in the design and test values of ultimate resistance were caused by the low values of geometrical imperfections of the tested specimens. The design approach of flexural buckling resistance according to the American Specification [8], [10] does not explicitly include the influence of structural imperfections in the calculation, making the deviations smaller according to this standard. If buckling curve D is used in calculation of the flexural buckling resistance of built-up members with welded interconnections, according to EN 1993-1-4 [1], the deviations in respect to the experimental values ranged between 1.39 and 2.11. This result points to the need of modification buckling curve for flexural buckling about minor axis of the built-up members with welded interconnections.

3 NUMERICAL ANALYSIS

In order to obtain a relevant data base for defining of design recommendations, an extensive numerical analysis in the Abaqus software was performed. A quasi-static, dynamic, non-linear analysis with an explicit modulus was applied. Non-uniform mass scaling to reduce the calculation time was used. The first phase of analysis included simulation of experimental testing of built-up members to flexural buckling. The analysis included all the important parameters: non-linear stress-strain relationship, improvement of mechanical properties in the cross section corners, geometrical imperfections, residual stresses, boundary conditions at the ends of the elements and contact conditions between the individual elements of the model. The results of the numerical analysis indicated that the failure of FE models was defined by the flexural buckling about the minor axis. The failure pattern observed in the experiment is accurately fitted by deformed shape of the FE model. A high coincidence of the ultimate load, longitudinal strain and lateral deflection in relation to the experiment was achieved, see Figure 7.

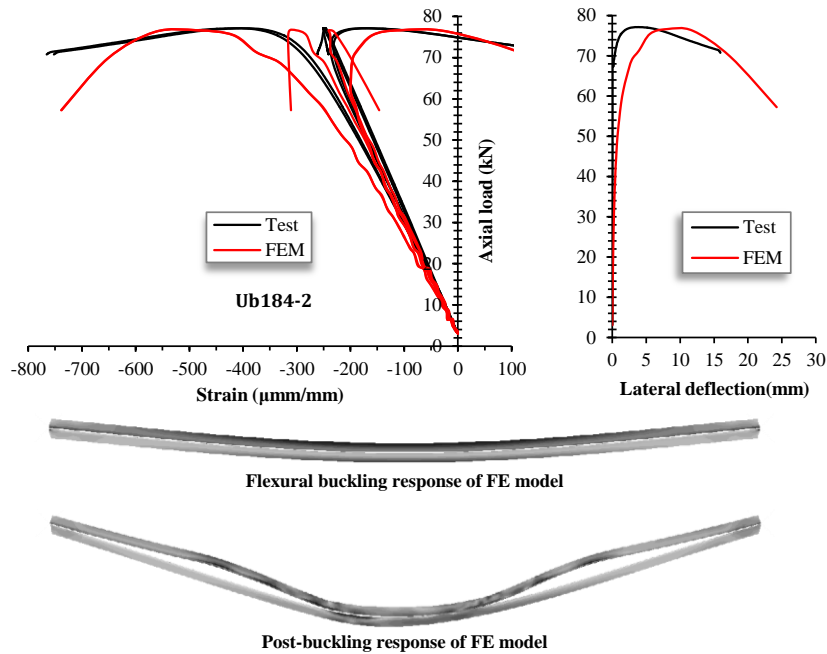


Figure 7 - Validation of numerical results with experimental results

In order to extend the range of structural performance data and investigate the effects of systematic variation of key individual parameters, multiple parametric series have been created with total of 40 FE models that were calibrated and verified by the results of the experimental testing. The boundary conditions at the ends of the elements were simulated as ideal pin supports. The initial imperfections were distributed according to the first overall buckling shape produced by the linear eigenvalue analysis with amplitude of $L/750$ that corresponds to the fabrication tolerances in EN 1090-2 [11]. The following parameters were varied: the length of the built up member L and distance between the interconnections a . Two series of models with bolted and welded interconnections were analysed. Finally, the FE values of ultimate flexural buckling resistance about minor axis were used for the developing of a new design approach for closely spaced built-up stainless steel members subject to axial compressive load.

4 DESIGN RECOMMENDATIONS

Reliable experimental and numerical data base was provided the definition of the predictive expressions for calculation flexural buckling resistance about minor axis of compressed stainless steel built-up members with chords in contact. The analysis is based on the Bleich's analytical criterion for equivalent slenderness of compressed built-up battened members [12]. For the purpose of harmonization and general acceptability, the rules given in the Eurocode for stainless steel [1] and carbon steel [2] were unified. The calculation recommendations are defined separately for the members with bolted and welded interconnections (Figure 8).

Developed predictive method is based on utilization of buckling curve C and modified equations for shear stiffness S_v depending on the type of interconnections between the chords [4].

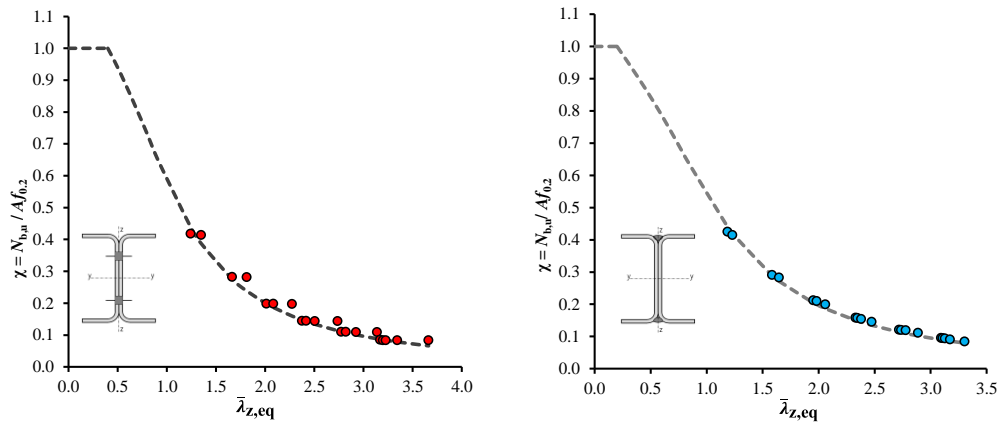


Figure 8 - Normalised FEM results and buckling curve C

5 CONCLUSIONS

In this paper, the experimental and numerical investigation of the compressed built-up stainless steel member behaviour has been briefly presented. Following conclusions are drawn:

- Significant negative influence of membrane residual stresses on the resistance of specimen with welded interconnections is observed in the intermediate slenderness domain. By reducing the number of modules between interconnections, the resistance of the specimens increases. Contrary to this observation, the resistance of specimens with bolted interconnections increases with the increase of the module numbers between interconnections. This kind of response can be caused by slip capacity of the bolted interconnection.
- Smaller influence of thermal residual stresses on the flexural stability of the welded specimens is observed in the high slenderness domain. Influence of the bolted connection flexibility is more prominent than in the intermediate slenderness field. The welded connection provides a higher flexural buckling resistance than bolted connections, referring to the structural solutions of built-up specimens with three and two modules between interconnections.
- The proposed predictive approach for determination of design buckling resistance of the compressed closely spaced stainless steel built-up member offers improved accuracy over the current Eurocode approach for equivalent carbon steel member [1].

ACKNOWLEDGMENTS

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Inox" Ltd. in Novi Sad, the Institute for Testing of Materials in Belgrade and the Institute for Materials and Structures, University of Belgrade for their financial and technical support.

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Mladen Ćosić¹

NELINEARNA STATIČKA I DINAMIČKA SEIZMIČKA ANALIZA OKVIRNIH ZGRADA PREMA PERFORMANSAMA

Rezime:

P-4

U radu je prikazan jedan deo razvijenih teorijskih formulacija i rezultata numeričkih istraživanja sprovedenih u doktorskoj disertaciji koja je odbranjena 2015. godine na Građevinskom fakultetu Univerziteta u Beogradu. U skraćenom obimu su prikazana istraživanja vezana za akceleroگرامe i spektre odgovora, kao i istraživanja na polju nelinearnog odgovora armiranobetonskih okvirnih sistema koji reprezentuju konstruktivni sistem skeletnih zgrada. U proučavanju nelinearnog odgovora korišćeni su i 3D modeli okvirnih zgrada, a istraživан je i odnos kapaciteta konstrukcije/seizmički zahtev za ove modele.

Ključne reči: zemljotres, zgrade, nelinearne analize, performanse, Nonlin Quake

PERFORMANCE-BASED NONLINEAR STATIC AND DYNAMIC SEISMIC ANALYSIS OF FRAMEWORK BUILDINGS

Summary:

The paper shows a part of the developed theoretical formulations and results of numerical analyses conducted in doctoral dissertation, which is defended at the Faculty of Civil Engineering University of Belgrade in 2015. In summary volume presents the research in the field of ground motion records and response spectras, research in the field of non-linear responses of frame system that participate in the structural system of frame buildings, research in the field of non-linear response of the 3D model of frame buildings and research in the field of relations of the 3D model of frame buildings response/seismic demand.

Key words: earthquake, buildings, nonlinear analyses, performances, Nonlin Quake

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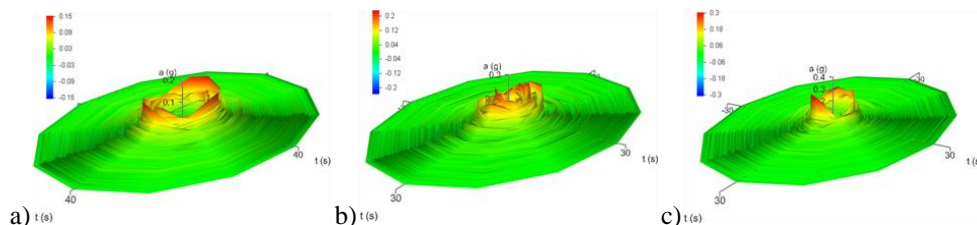
1 UVOD

Metodologija analize zgrada prema performansama za uslove dejstva zemljotresa (PBEE - *Performance-Based Earthquake Engineering*) je inicirana u poslednjih petnaestak godina prvo na determinističkom, a zatim i na probablističkom pristupu. PBEE metodologija je multidisciplinarna oblast nauke o zemljotresnom inženjerstvu razvijena u cilju kompletne i kompleksne analize i procene stanja postojećih i novih zgrada za uslove dejstva zemljotresa. PBEE metodologija se zasniva na nelinearnim seizmičkim analizama, pri čemu se problem razmatra kroz analize: hazarda, konstrukcije, oštećenja i štete.

Metodološki postupak analize okvirnih zgrada prema performansama, predložen/razvijen u doktorskoj disertaciji autora ovog rada, zasniva se na konceptu PBEE metodologije i implementiran je u originalno softversko rešenje *Nonlin Quake* koji se sastoji iz programa: *Nonlin Quake DB* (baza podataka), *Nonlin Quake GMP* (procesiranje akceleroograma), *Nonlin Quake AA* (generisanje veštačkih akceleroograma), *Nonlin Quake SHA* (analiza seizmičkog hazarda), *Nonlin Quake RSP* (procesiranje spektra odgovora), *Nonlin Quake PP* (projektni parametri), *Nonlin Quake PCS* (*pushover* krive-površ), *Nonlin Quake HINSDA* (hibridna inkrementalna nelinearna statička-dinamička analiza), *Nonlin Quake TD* (ciljno pomeranje), *Nonlin Quake SP* (procedura skaliranja) i *Nonlin Quake TDE* (anvelopa ciljnog pomeranja). U zavisnosti od toga da li se analiza performansi sistema sprovodi primenom NSPA (nelinearna statička *pushover* analiza), INDA (inkrementalna nelinearna dinamička analiza), IDA (inkrementalna dinamička analiza) ili HINSDA, razvijena su četiri bitno različita toka proračuna *Nonlin Quake* ekspertskog sistema. Za potrebe istraživanja na 3D modelima zgrada definisani su i uvedeni u *Nonlin Quake* novi termini, od kojih se izdvajaju: površ zapisa ubrzanja tla, površ spektra odgovora, anvelopa mere intenziteta, *pushover* površ, koeficijent površi duktilnosti, anvelopa ciljnog pomeranja, površ globalnog i međuspratnog drifta.

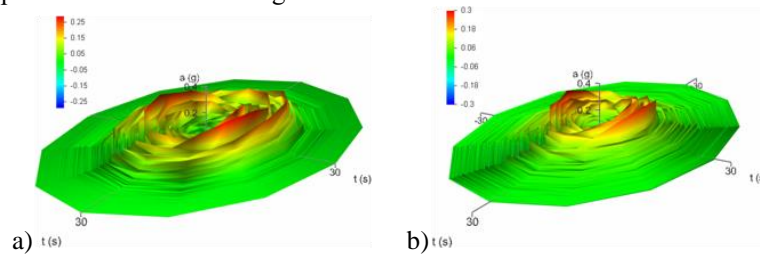
2 POVRŠ AKCELEROGRAMA I POVRŠ SPEKTRA ODGOVORA

Postupak procesiranja akceleroograma i spektara odgovora obuhvata analizu, interpretaciju i prezentaciju kroz faze: selekcija, formatiranje, konvertovanje, skaliranje, proračunavanje, kompatibilizacija, filtriranje, generisanje i transformacija. Ove procedure se izvršavaju u vremenskom, frekventnom, frekventno-vremenskom i kapacitativnom domenu. Metodološki postupak za analizu okvirnih zgrada prema performansama, a koji je prezentovan u ovom istraživanju, zasniva se na akceleroogramima prirodnih zemljotresa i akceleroograma veštački generisanih zemljotresa. Oba tipa akceleroograma su dvokomponentalni, s tim što akceleroogrami prirodnih zemljotresa imaju komponentu paralelno FP i upravno FN na pravac pružanja raseda. Procedura razvoja površi akceleroograma zasnovana je na primeni matrice rotacije FP i FN komponentata i dodatnom procesiranju. Na slici 1 su prikazane površi nekih akceleroograma.



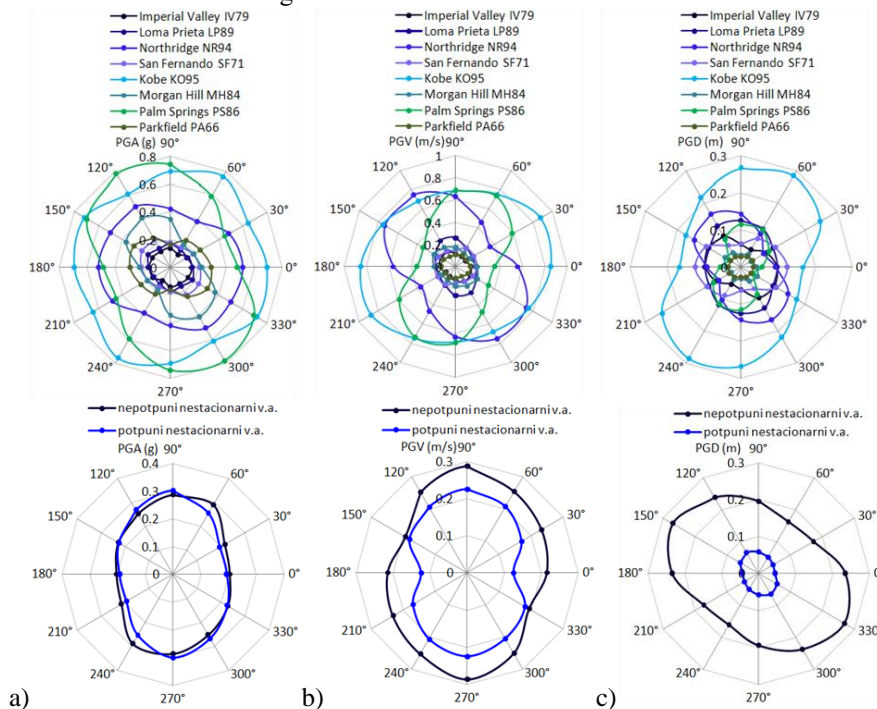
Slika 1. Površ akceleroograma: a) Imperial Valley, b) San Fernando, c) Parkfield

Alternativa korišćenju prirodnih akcelorograma, koji se preuzimaju iz PEER GMDB baze zemljotresa, je primena veštačkih akcelorograma, koji su generisani kao nepotpuni i potpuni nestacionarni signal u vremenskom domenu. Ovi akcelorogrami, između ostalog, razvijeni su u funkciji promene ubrzanja u toku vremena, vrednosti amplituda, anvelope amplituda akceleracija, ugaone frekvencije, faznog ugla i -te sinusoide koji se generiše funkcijom slučajnog broja i funkcije spektralne gustine snage kretanja tla. Na slici 2 su prikazane generisane površi veštačkih akcelorograma.



Slika 2. Površni akcelorograma: a) nepotpuni nestacionarni veštački akcelorogram, b) potpuni nestacionarni veštački akcelorogram

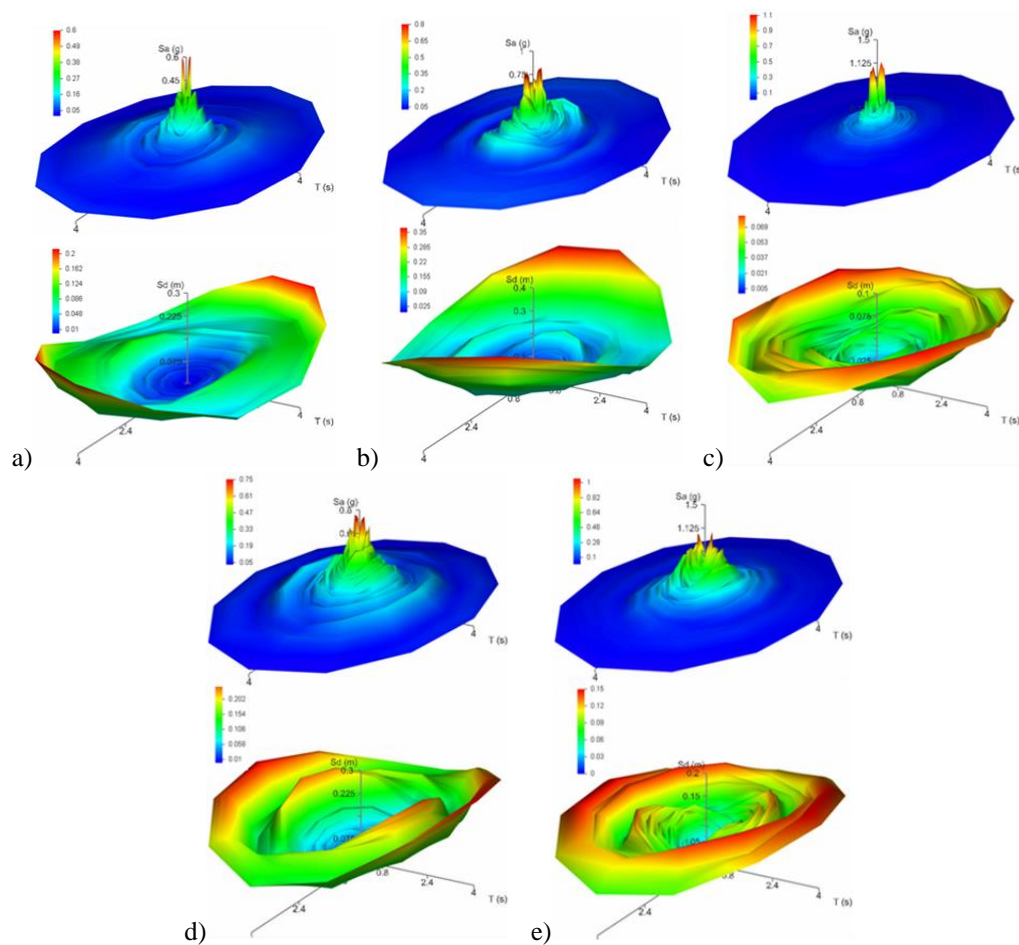
S obzirom da se pri procesiranju akcelorograma po uglovima određuje po jedan diskretan parametar mere intenziteta IM , to je uspostavljena veza između njih primenom anvelope mere intenziteta. Na slici 3 su prikazane anvelope mera intenziteta akcelorograma prirodnih zemljotresa i veštačkih akcelorograma.



Slika 3. Anvelope mere intenziteta akcelorograma prirodnih zemljotresa i veštačkih akcelorograma: a) PGA, b) PGV, c) PGD

Izveden je koeficijent raspoložive površi C_{IM} mere intenziteta, kojim se može brzo i efikasno analizirati uticaj dve ortogonalne komponente akcelerograma. Dovoljno je samo da se odrede mere intenziteta IM_{max} i IM_{max-90} za ortogonalne komponente akcelerograma i preko koeficijenta C_{IM} se može uspostaviti uticaj IM_{max-90} mere na IM_{max} meru.

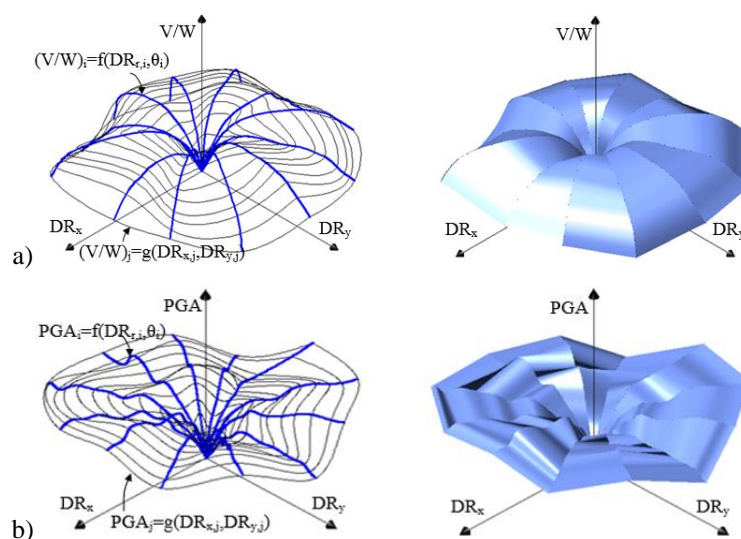
Konstrukcija površi spektra odgovora sprovedena je slično konstrukciji površi akcelerograma, pri čemu se prvo generišu spektri odgovora u ravanskom polarnom koordinatnom sistemu, a zatim se svakom spektru odgovora dodeljuje vertikalna koordinata (S_a ili S_d) u prostornom cilindričnom koordinatnom sistemu, a nakon toga se sprovodi konverzija u prostorni pravougli koordinatni sistem. Na slici 4 su prikazane površi spektara odgovora ubrzanja i pomeranja prirodnih zemljotresa i veštačkih simuliranih akcelerograma.



Slika 4. Površni spektara odgovora ubrzanja i pomeranja: a) Imperial Valley, b) San Fernando, c) Parkfield, d) nepotpuni nestacionarni veštački akcelerogram, e) potpuni nestacionarni veštački akcelerogram

3 ANALIZA NELINEARNOG PONAŠANJA OKVIRNIH ZGRADA

Odgovor 3D modela zgrada u kapacitativnom domenu prema NSPA i INDA analizi prezentuje se primenom *pushover* krivih. Međutim, u istraživanju prikazanom u disertaciji odgovor konstrukcije se predstavlja primenom seta *pushover* krivih za različite uglove dejstva zemljotresa, tako da se za 3D model zgrade generiše *pushover* površ. Na slici 5 su prikazani 3D mrežni modeli i renderovane 3D NSPA i INDA *pushover* površi. Prezentovane *pushover* površi predstavljaju opšti slučaj *pushover* površi za 3D model asimetrične neregularne zgrade i za analizirane uglove dejstva zemljotresa.

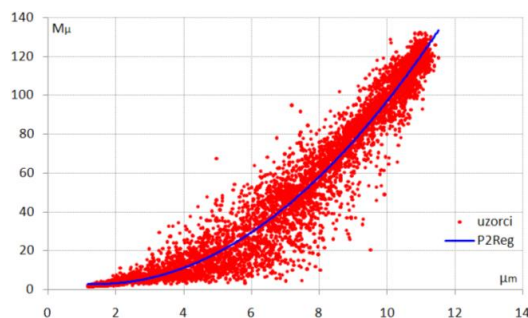


Slika 5. 3D mrežni modeli i renderovane 3D *pushover* površi: a) NSPA, b) INDA

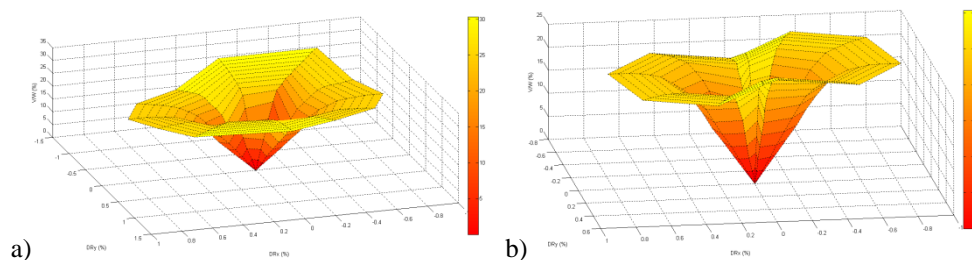
U odnosu na klasičan pristup gde se koristi koeficijent duktilnosti μ za procenu nelinearnog odgovora zgrade za jedan razmatrani pravac, uveden je novi koeficijent površi duktilnosti M_μ za procenu nelinearnog odgovora 3D modela zgrada. U opštem slučaju koeficijent površi duktilnosti M_μ predstavlja odnos površina oblasti ograničene maksimalno realizovanim pomeranjima i oblasti ograničene pomeranjima na granici tečenja. Korelacija koeficijenta duktilnosti μ i koeficijenta površi duktilnosti M_μ je uspostavljena primenom funkcije slučajnog izbora (*random* funkcije) maksimalno realizovanih pomeranja i koeficijenata duktilnosti po uglovima dejstva zemljotresa, uz dodatna postavljena ograničenja. Na slici 6 su prikazane diskretne vrednosti dobijene iteracijama i regresiona analiza za polinom drugog stepena. Klase koeficijenta površi duktilnosti M_μ su određene analogno klasama koeficijenta duktilnosti μ prema FEMA 273, tako da je $M_\mu < 3$ (za $\mu < 2$), $3 \leq M_\mu \leq 11$ (za $2 \leq \mu \leq 4$) i $M_\mu > 11$ (za $\mu > 4$).

Numerička istraživanja su sprovedena na 3D modelima okvirnih zgrada. S obzirom na prostor koji je dostupan za prezentovanje prikazani su samo proračuni izdvojenih modela zgrada. Na slikama 7 i 8, za 4x6x3 i 9x6x5-12 3D modele zgrada, prikazane su generisane NSPA *pushover* površi u funkciji globalnih driftova DR_x i DR_y i relativne vrednosti ukupne smičuće sile V/W za za bidirekciona seizmička dejstva $E_\theta + 0.3E_{\theta+90^\circ}$ i $E_x + 0.3E_y$. Kod oba 3D modela zgrada krutost u nelinearnom domenu je gotovo horizontalna $K_n \approx 0$ ili blago pozitivna, osim u pojedinim slučajevima, gde sa povećanjem nelinearnih deformacija krutost postaje

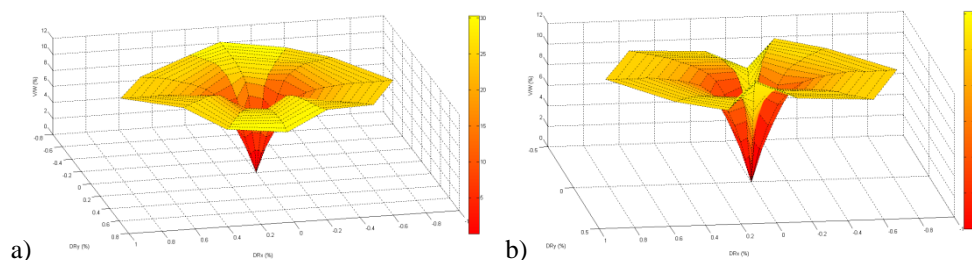
negativna $K_n < 0$. Kod 4x6x3 3D modela zgrade su realizovane maksimalne relativne vrednosti ukupne smičuće sile u intervalu od $V/W=(25\div 30)\%$, dok su kod 9x6x5-12 3D modela zgrade realizovane maksimalne relativne vrednosti ukupne smičuće sile u intervalu od $V/W=(10\div 12)\%$. Maksimalne relativne vrednosti ukupne smičuće sile kod 9x6x5-12 3D modela zgrade su znatno manje od rezultata dobijenih kod svih ostalih modela zgrada (šire u doktorskoj disertaciji), što je direktna posledica proračuna seizmičkog dejstva 9x6x5-12 3D modela zgrade prema SRP propisima.



Slika 6. Diskretne vrednosti dobijene iteracijama i regresiona analiza za polinom drugog stepena u cilju određivanja koeficijenta površi duktilnosti M_μ



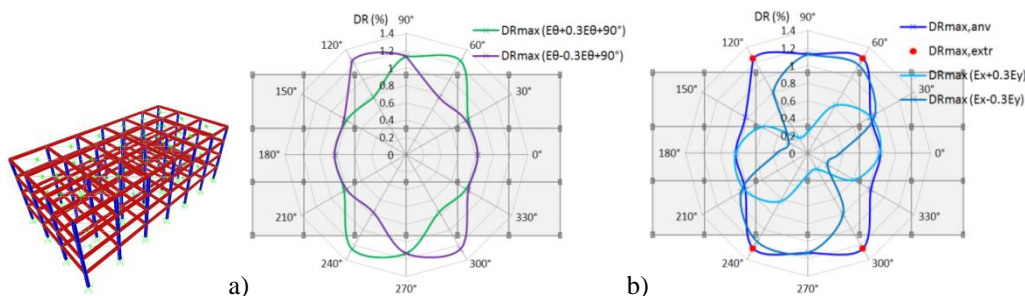
Slika 7. 4x6x3 3D model zgrade; generisana NSPA pushover površ u funkciji globalnih driftova DR_x i DR_y i relativne vrednosti ukupne smičuće sile V/W : a) $E_\theta+0.3E_{\theta+90^\circ}$, b) $E_x+0.3E_y$



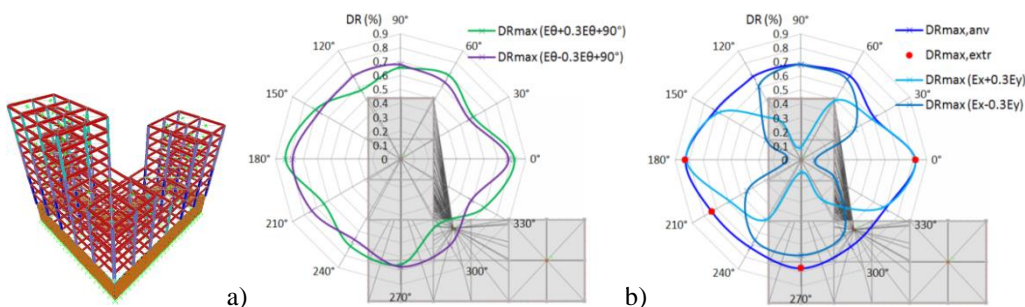
Slika 8. 9x6x5-12 3D model zgrade; generisana NSPA pushover površ u funkciji globalnih driftova DR_x i DR_y i relativne vrednosti ukupne smičuće sile V/W : a) $E_\theta+0.3E_{\theta+90^\circ}$, b) $E_x+0.3E_y$

Na slikama 9 i 10, za 4x6x3 i 9x6x5-12 3D modele zgrada, prikazani su maksimalni globalni driftovi DR_{max} za dejstva $E_\theta+0.3E_{\theta+90^\circ}$ i $E_\theta-0.3E_{\theta+90^\circ}$, anvelopa maksimalnih globalnih driftova $DR_{max,anv}$, ekstremne vrednosti maksimalnih globalnih driftova $DR_{max,extr}$ i maksimalni

globalni driftovi DR_{max} za $E_x+0.3E_y$ i $E_x-0.3E_y$. Globalni driftovi i anvelope maksimalnih globalnih driftova su dobijeni za pomeranja centra mase pri bidirekcionom seizmičkom dejstvu. Kod 4x6x3 3D modela zgrade, za bidirekciono seizmičko dejstvo, dobijene su ekstremne vrednosti maksimalnih globalnih driftova $DR_{max,extr}$ za uglove $\theta=(60^\circ, 120^\circ, 240^\circ, 300^\circ)$, dok bi se u praktičnim inženjerskim linearnim seizmičkim analizama ovakve zgrade razmatrali driftovi za uglove $\theta=0^\circ$ i $\theta=90^\circ$. Kod 9x6x5-12 3D modela zgrade, za bidirekciono seizmičko dejstvo, dobijene su ekstremne vrednosti maksimalnih globalnih driftova $DR_{max,extr}$ za uglove $\theta=(0^\circ, 180^\circ, 210^\circ, 270^\circ)$, pri čemu je zgrada asimetrična u osnovi, neregularna po visini i torziona osetljiva. U ovom slučaju je kod provere globalnih driftova potrebno uzeti u obzir ne samo uobičajene pravce (X i Y), već i ugao $\theta=210^\circ$. 9x6x5-12 3D model zgrade, projektovan prema SRP propisima, realizuje viši nivo duktilnosti, u odnosu na 4x6x3 3D model zgrade projektovan prema EC propisima. Generalno se može tvrditi da kod zgrada veće spratnosti lakše se realizuje viša klasa duktilnosti, pri čemu se povećanjem neregularnosti u osnovi ova duktilnost smanjuje.



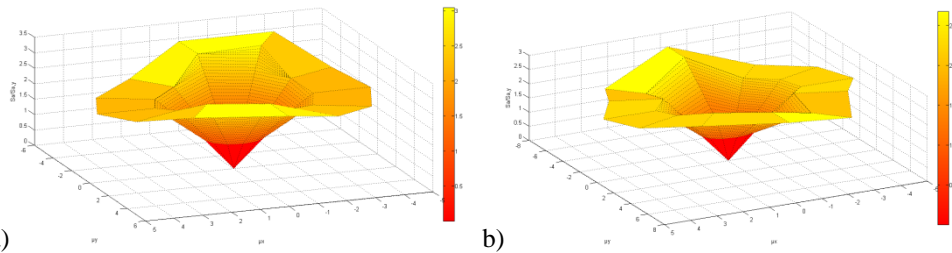
Slika 9. 4x6x3 3D model zgrade: a) maksimalni globalni driftovi DR_{max} za $E_{\theta}+0.3E_{\theta+90}$ i $E_{\theta}-0.3E_{\theta+90}$, b) anvelopa maksimalnih globalnih driftova $DR_{max,anv}$, ekstremne vrednosti maksimalnih globalnih dr. $DR_{max,extr}$, maksimalni globalni dr. DR_{max} za $E_x+0.3E_y$ i $E_x-0.3E_y$



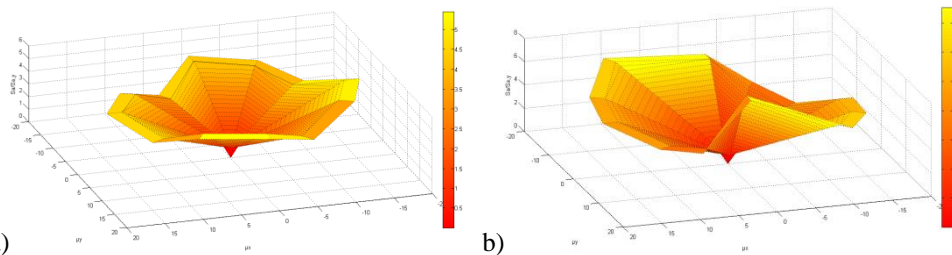
Slika 10. 9x6x5-12 3D model zgrade: a) maksimalni globalni driftovi DR_{max} za $E_{\theta}+0.3E_{\theta+90}$ i $E_{\theta}-0.3E_{\theta+90}$, b) anvelopa maksimalnih globalnih driftova $DR_{max,anv}$, ekstremne vrednosti maksimalnih globalnih dr. $DR_{max,extr}$, maksimalni globalni dr. DR_{max} za $E_x+0.3E_y$ i $E_x-0.3E_y$

Na slikama 11 i 12, za 4x6x3 i 9x6x5-12 3D modele zgrada, prikazane su generisane INDA pushover površi u funkciji duktilnosti μ_x i μ_y i relativnog spektralnog ubrzanja $S_d/S_{a,y}$ za bidirekciono seizmičko dejstvo $E_{\theta}+0.3E_{\theta+90}$ i $E_x+0.3E_y$. Razmatrane su INDA pushover krive za 16%, 50% i 84% fraktilne vrednosti. Krive relativnog spektralnog ubrzanja $S_d/S_{a,y}$ su u osnovi gotovo geometrijski identične sa krivama duktilnosti μ_{coll} i μ_{extr} za $E_{\theta}+0.3E_{\theta+90}$ i E_{θ}

$0.3E_{\theta+90^\circ}$ seizmička dejstava kod velikog broja razmatranih modela zgrade. U odnosu na krive relativne vrednosti ukupne smičuće sile u osnovi zgrade $(V/W)_{adeq}$ za maksimalni globalni drift DR_{max} određene NSPA analizama koje su kružnog ili približno kružnog oblika, krive relativnog spektralnog ubrzanja $S_d/S_{a,y}$ određene IDA analizama su elipsoidnog oblika ili složenije forme.



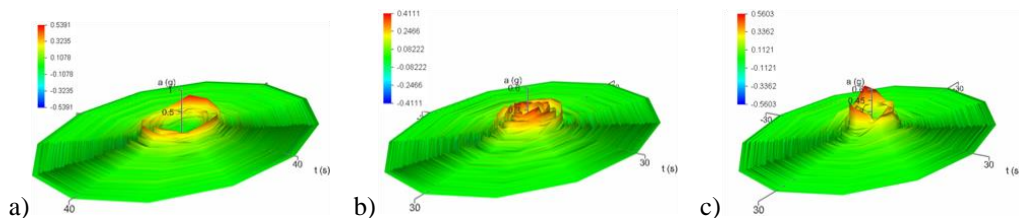
a) b)
Slika 11. 4x6x3 3D model zgrade; generisana INDA pushover površ u funkciji duktilnosti μ_x i μ_y i rel. spek. ubrzanja $S_d/S_{a,y}$ za bidirekciona seizmička dejstva: a) $E_{\theta}+0.3E_{\theta+90^\circ}$, b) $E_x+0.3E_y$



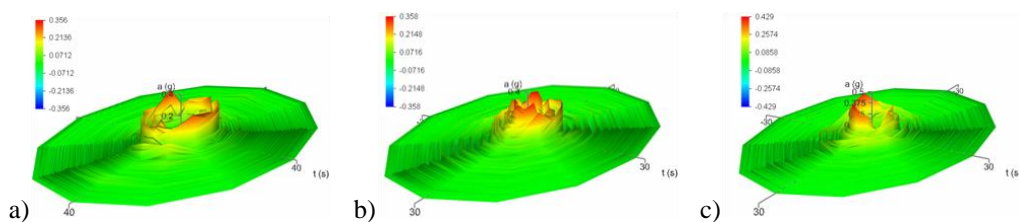
a) b)
Slika 12. 9x6x5-12 3D model zgrade; generisana INDA pushover površ u funkciji duktilnosti μ_x i μ_y i rel. spek. ubrzanja $S_d/S_{a,y}$ za bidirekciona seizmička dejstva: a) $E_{\theta}+0.3E_{\theta+90^\circ}$, b) $E_x+0.3E_y$

4 ANALIZA ODNOSA KAPACITET KONSTRUKCIJE/SEIZMIČKI ZAHTEV

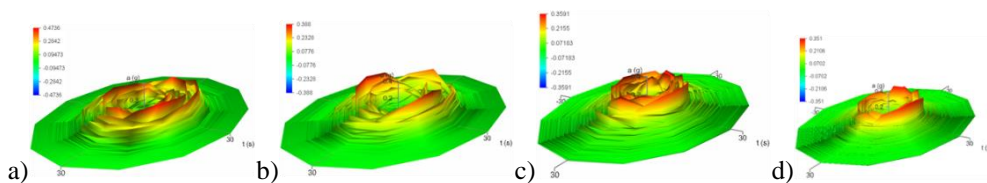
Nakon generisanja *pushover* krivih i površi iz seizmičkih analiza, sprovedene su analize ciljnih pomeranja za svaki ugao dejstva zemljotresa. Procedura određivanja nivoa ciljnog pomeranja sprovedena je primenom unapređene iterativne metode koeficijenata pomeranja (IDCM). Ova metoda ima razvijen dvostruki iterativni algoritam, pri čemu se u prvom koraku simultano po silama i pomeranju, kroz iteracije, pretražuje nivo ciljnog pomeranja, dok se u drugom koraku iterativno koriguje nivo ciljnog pomeranja po *pushover* krivi. Sa druge strane, seizmički zahtev se, između ostalog, određuje i korekcijom spektara odgovora. Za skaliranje akceleroograma i određivanje odgovarajućih spektara odgovora primenjena je metoda najmanjih kvadrata. Primenjena je i procedura kompatibilizacije (*spectral matching*) kojom se kreira kompatibilan akceleroogram na osnovu realnog akceleroograma i ciljnog (projektnog) spektra odgovora. Skaliranje i kompatibilizacija akceleroograma i spektara odgovora su sprovedeni prema FEMA 750P projektnim spektrima. Na slikama 13 i 14, prikazane su skalirane i kompatibilizovane površi akceleroograma prirodnih zemljotresa, dok su na slici 15 prikazane skalirane i kompatibilizovane površi akceleroograma veštački generisanih zemljotresa.



Slika 13. Skalirane površi akceleroograma: a) Imperial Valley, b) San Fernando, c) Parkfield

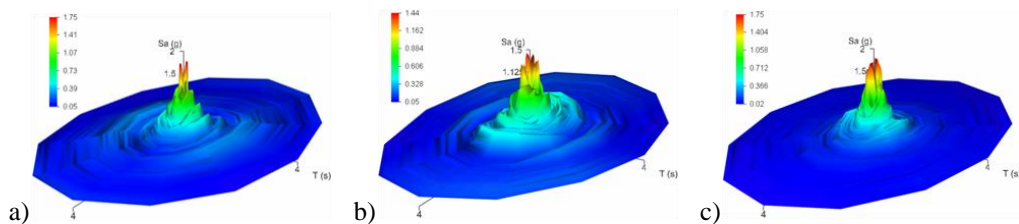


Slika 14. Kompatibilizovane površi akceleroograma: a) Imperial Valley, b) San Fernando, c) Parkfield

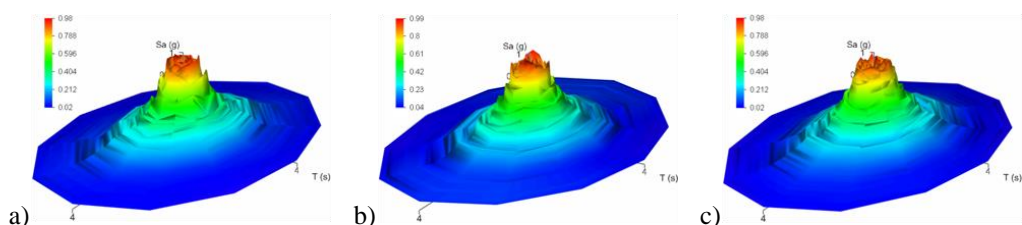


Slika 15. Skalirane i kompatibilizovane površi akceleroograma: a) i b) nepotpuni nestacionarni veštački akceleroogram, c) i d) potpuni nestacionarni veštački akceleroogram

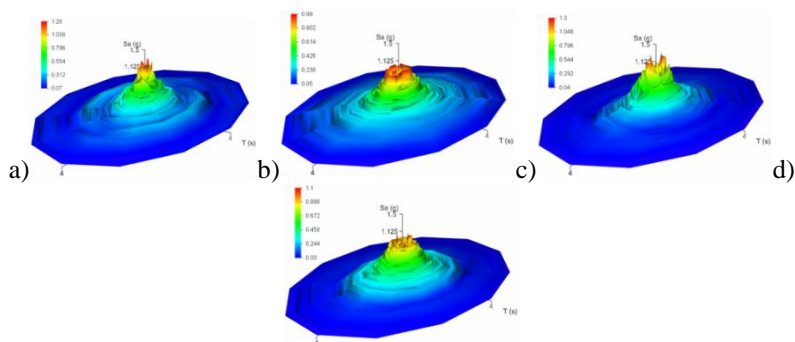
Na slikama 16 i 17 su prikazane skalirane i kompatibilizovane površi spektara odgovora ubrzanja prirodnih zemljotresa, dok su na slici 18 prikazane skalirane i kompatibilizovane površi spektara odgovora ubrzanja veštački generisanih zemljotresa. Površi spektara odgovora, generisane primenom postupka kompatibilizacije, imaju domen konstantnih ubrzanja veći, u odnosu na ovaj domen kod površi spektara odgovora skalirane metodom najmanjih kvadrata.



Slika 16. Skalirane površi spektara odgovora ubrzanja: a) Imperial Valley, b) San Fernando, c) Parkfield

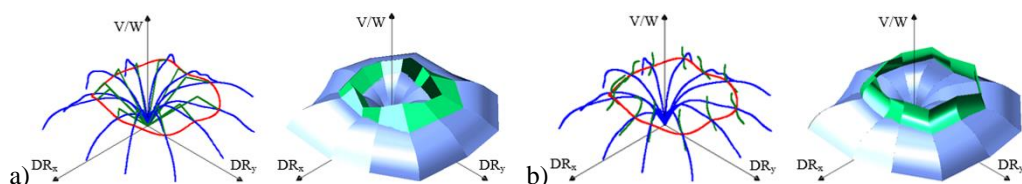


Slika 17. Kompatibilizovane površi spektara odgovora ubrzanja: a) Imperial Valley, b) San Fernando, c) Parkfield



Slika 18. Skalirane i kompatibilizovane površi spektara odgovora ubrzanja: a) i b) nepotpuni nestacionarni veštački akcelerogram, c) i d) potpuni nestacionarni veštački akcelerogram

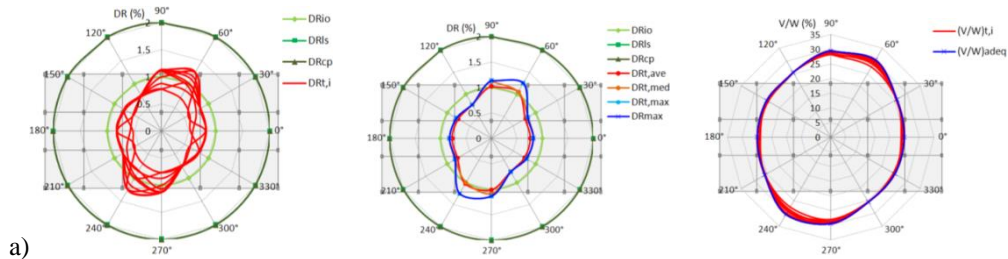
S obzirom da se proračuni nivoa ciljnog pomeranja 3D modela okvirnih zgrada sprovode za svaki ugao dejstva zemljotresa posebno, to se ovako određene diskretne vrednosti ciljnog pomeranja međusobno povezuju interpolacijom, tako da se konačno generiše zatvorena kriva u prostoru za koju je uveden termin anelopa ciljnog pomeranja. Na slici 19 je prikazan postupak dobijanja anelope ciljnog pomeranja iz preseka *pushover* krivih (*pushover* površi) i krivih bilinearizacije ili seizmičkog zahteva.



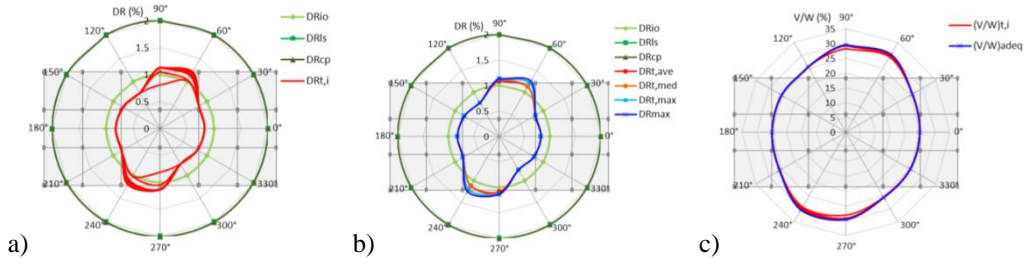
Slika 19. Postupak dobijanja anelope ciljnog pomeranja iz preseka *pushover* krivih (*pushover* površi) i krivih: a) bilinearizacije, b) seizmičkog zahteva

U odnosu na klasičan pristup gde se koriste jednodimenzionalni koeficijenti za procenu nelinearnog odgovora, kao što je duktilnost μ za razmatrani pravac, uvedeni su novi koeficijenti dvodimenzionalnog karaktera kojima se može razmatrati nelinearan odgovor 3D modela zgrada: M_μ koeficijent (raspoložive) površi duktilnosti, $M_{d,v/d,Y}$ koeficijent realizovane površi duktilnosti, $M_{d,x/d,Y}$ koeficijent odgovarajuće površi duktilnosti za x osu, $M_{d,y/d,Y}$ koeficijent odgovarajuće površi duktilnosti za y osu, $M_{d,v/IO}$ koeficijent odnosa površina oblasti ograničene anvelopom ciljnog pomeranja i oblasti ograničene anvelopom IO performansnog

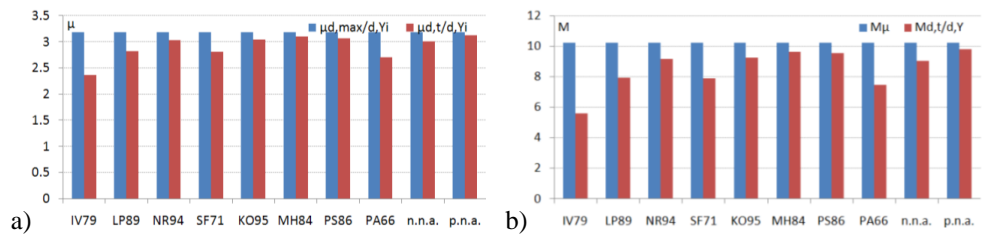
nivoa, $M_{d,t/LS}$ koeficijent odnosa površina oblasti ograničene anvelopom ciljnog pomeranja i oblasti ograničene anvelopom LS performansnog nivoa, $M_{d,t/CP}$ koeficijent odnosa površina oblasti ograničene anvelopom ciljnog pomeranja i oblasti ograničene anvelopom CP performansnog nivoa, $M_{d,t,max/d,t}$ koeficijent odnosa površina oblasti ograničene anvelopom maksimalnog ciljnog pomeranja i oblasti ograničene anvelopom minimalnog ciljnog pomeranja, $M_{d,t,min/d,t}$ koeficijent odnosa površina oblasti ograničene anvelopom minimalnog ciljnog pomeranja i oblasti ograničene anvelopom ciljnog pomeranja, $C_{d,t,max/code}$ koeficijent koji je u funkciji minimalne potrebne površi duktilnosti za nivo ciljnog pomeranja prema propisima, $\mu_{d,max/d,Y}^m$ koeficijent prosečne raspoložive duktilnosti po uglovima θ_i za sve zemljotrese i $\mu_{d,t/d,Y}^m$ koeficijent prosečne realizovane duktilnosti po uglovima θ_i za sve zemljotrese. Detaljnije izvođenje svih ovih koeficijenata može se videti u doktorskoj disertaciji. Na slikama 20÷29 su prikazane anvelope, proračunati koeficijenti (izdvojeno) i površi driftova 4x6x3 i 9x6x5-12 3D modela zgrade.



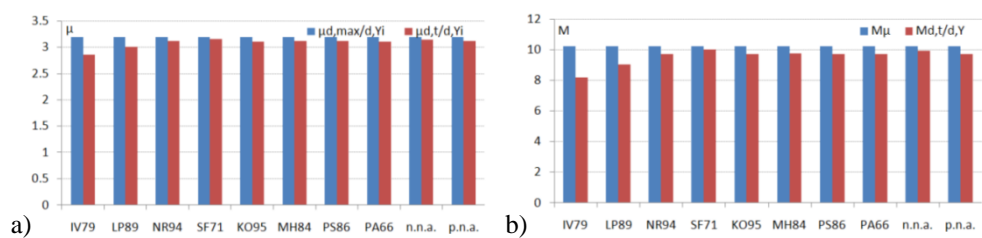
Slika 20. 4x6x3 3D model zgrade, $E_{\theta}+0.3E_{\theta+90^{\circ}}$ (skalirani akcelerogrami): a) globalni driftovi DR_{IO} , DR_{LS} , DR_{CP} i DR_t za ciljna pomeranja, b) globalni driftovi DR_{IO} , DR_{LS} , DR_{CP} , $DR_{t,ave}$, $DR_{t,med}$, $DR_{t,max}$ i DR_{max} , c) relativne vrednosti ukupnih smičućih sila u osnovi zgrade za ciljna pomeranja $(V/W)_i$ i $(V/W)_{adeq}$ za maksimalni globalni drift DR_{max}



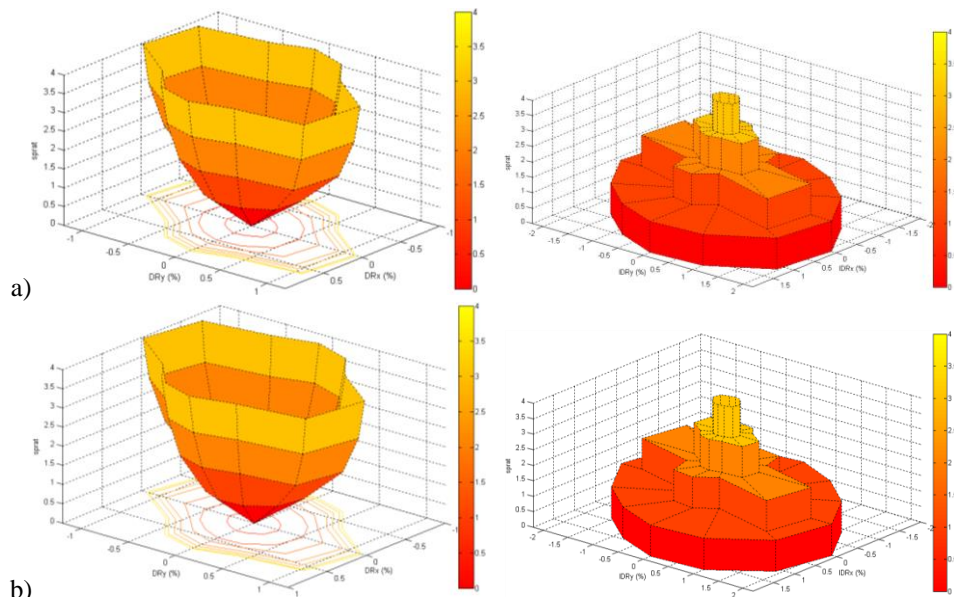
Slika 21. 4x6x3 3D model zgrade, $E_{\theta}+0.3E_{\theta+90^{\circ}}$ (kompatibilizovani akcelerogrami): a) globalni driftovi DR_{IO} , DR_{LS} , DR_{CP} i DR_t za ciljna pomeranja, b) globalni driftovi DR_{IO} , DR_{LS} , DR_{CP} , $DR_{t,ave}$, $DR_{t,med}$, $DR_{t,max}$ i DR_{max} , c) relativne vrednosti ukupnih smičućih sila u osnovi zgrade za ciljna pomeranja $(V/W)_i$ i $(V/W)_{adeq}$ za maksimalni globalni drift DR_{max}



Slika 22. 4x6x3 3D model zgrade, $E_\theta+0.3E_{\theta+90^\circ}$ (skalirani akcelerogrami), koeficijenti: a) $\mu_{d,max/d,Y}^i$ i $\mu_{d,t/d,Y}^i$, b) M_μ i $M_{d,t/d,Y}$



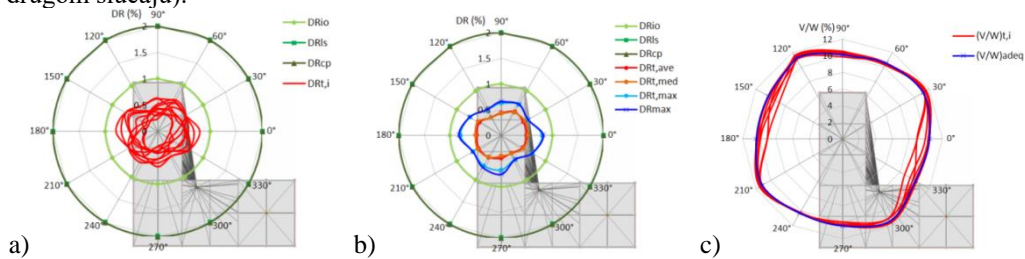
Slika 23. 4x6x3 3D model zgrade, $E_\theta+0.3E_{\theta+90^\circ}$ (kompatibilizovani akcelerogrami), koeficijenti: a) $\mu_{d,max/d,Y}^i$ i $\mu_{d,t/d,Y}^i$, b) M_μ i $M_{d,t/d,Y}$



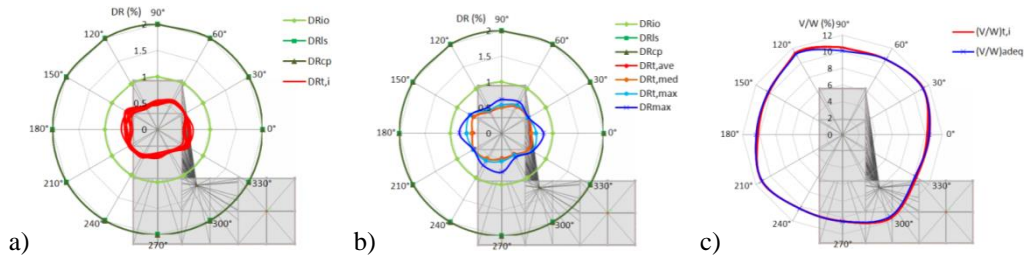
Slika 24. 4x6x3 3D model zgrade, $E_\theta+0.3E_{\theta+90^\circ}$, površ globalnih driftova za nivo ciljnog pomeranja DR_p , površ međuspratnih driftova za nivo ciljnog pomeranja IDR_i ; a) skalirani akcelerogrami, b) kompatibilizovani akcelerogrami

Kod 4x6x3 3D modela zgrade veliki broj globalnih driftova, za nivo ciljnog pomeranja DR_p , po uglovima θ_i , jednak je maksimalnim globalnim driftovima DR_{max} . Ovo se događa skoro kod svih zemljotresa (i za skaliranje i za kompatibilizaciju), tako da se maksimalne vrednosti

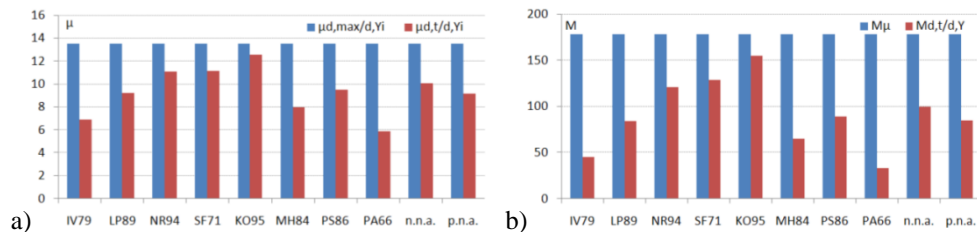
globalnih driftova za nivo ciljnog pomeranja $DR_{i,max}$ izjednačavaju sa maksimalnim globalnim driftovima DR_{max} . Srednje vrednosti i medijane globalnih driftova za nivo ciljnog pomeranja $DR_{i,ave}$ i $DR_{i,med}$ su međusobno skoro identične, ali su manje od maksimalnih vrednosti globalnih driftova za nivo ciljnog pomeranja $DR_{i,max}$ u intervalu $\theta=(60^\circ\div 120^\circ)$, odnosno u intervalu $\theta=(240^\circ\div 300^\circ)$. Relativne vrednosti ukupnih smičućih sila u osnovi zgrade, za ciljna pomeranja $(V/W)_i$, su nešto niže od relativnih vrednosti ukupnih smičućih sila u osnovi zgrade $(V/W)_{adeq}$ za maksimalni globalni drift DR_{max} u intervalu $\theta=(60^\circ\div 120^\circ)$, odnosno u intervalu $\theta=(240^\circ\div 300^\circ)$. U ostalim intervalima ugla θ_i ove vrednosti su međusobno jednake, što ukazuje na činjenicu da je nelinearna krutost pozitivna (u prvom slučaju) ili približno jednaka nuli (u drugom slučaju).



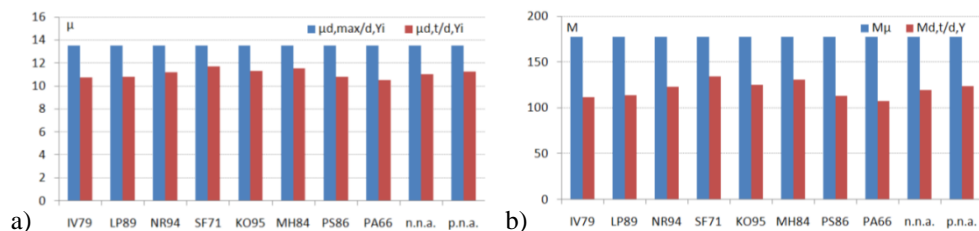
Slika 25. 9x6x5-12 3D model zgrade, $E_\theta+0.3E_{\theta+90^\circ}$ (skalirani akcelorogrami): a) globalni driftovi DR_{iO} , DR_{iS} , DR_{CP} i DR_i za ciljna pomeranja, b) globalni driftovi DR_{iO} , DR_{iS} , DR_{CP} , $DR_{i,ave}$, $DR_{i,med}$, $DR_{i,max}$ i DR_{max} , c) relativne vrednosti ukupnih smičućih sila u osnovi zgrade za ciljna pomeranja $(V/W)_i$ i $(V/W)_{adeq}$ za maksimalni globalni drift DR_{max}



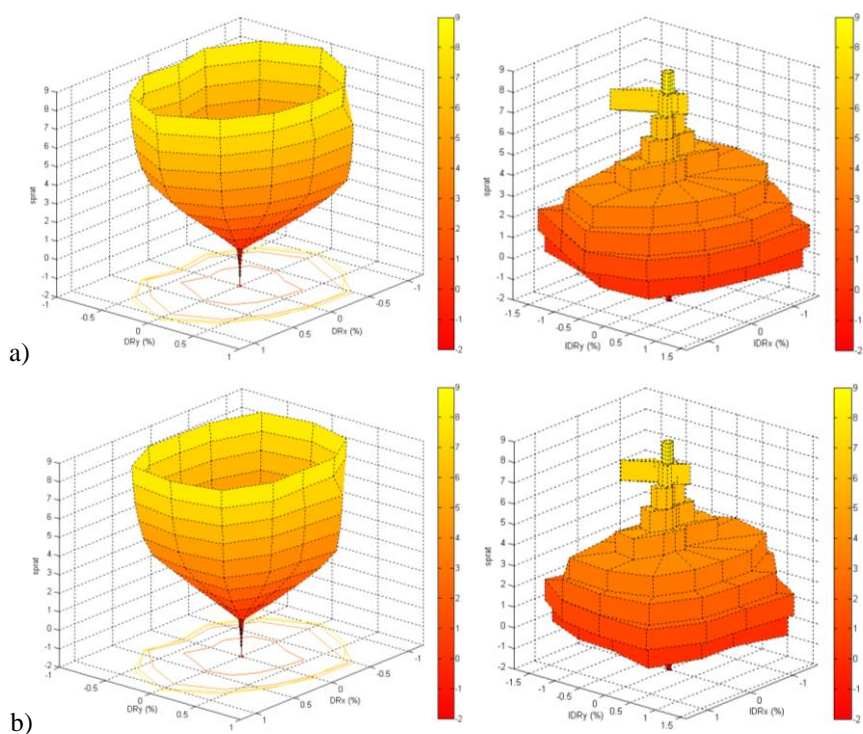
Slika 26. 9x6x5-12 3D model zgrade, $E_\theta+0.3E_{\theta+90^\circ}$ (kompatibilizovani akcelorogrami): a) globalni driftovi DR_{iO} , DR_{iS} , DR_{CP} i DR_i za ciljna pomeranja, b) globalni driftovi DR_{iO} , DR_{iS} , DR_{CP} , $DR_{i,ave}$, $DR_{i,med}$, $DR_{i,max}$ i DR_{max} , c) relativne vrednosti ukupnih smičućih sila u osnovi zgrade za ciljna pomeranja $(V/W)_i$ i $(V/W)_{adeq}$ za maksimalni globalni drift DR_{max}



Slika 27. 9x6x5-12 3D model zgrade, $E_\theta+0.3E_{\theta+90^\circ}$ (skalirani akcelorogrami), koeficijenti: a) $\mu_{d,max/d,Y}^i$ i $\mu_{d,t/d,Y}^i$, b) M_μ i $M_{d,t/d,Y}$



Slika 28. 9x6x5-12 3D model zgrade, $E_\theta+0.3E_{\theta+90^\circ}$ (kompatibilizovani akcelerogrami), koeficijenti: a) $\mu_{d,max/d,Y}^i$ i $\mu_{d,t/d,Y}^i$, b) M_μ i $M_{d,t/d,Y}$



Slika 29. 9x6x5-12 3D model zgrade, $E_\theta+0.3E_{\theta+90^\circ}$, površ globalnih driftova za nivo ciljnog pomeranja DR_p , površ međuspratnih driftova za nivo ciljnog pomeranja IDR_i ; a) skalirani akcelerogrami, b) kompatibilizovani akcelerogrami

Kod 9x6x5-12 3D modela zgrade, pri skaliranju akcelerograma metodom najmanjih kvadrata, veliki broj globalnih driftova za nivo ciljnog pomeranja DR_i po uglovima θ_i jednak je maksimalnim globalnim driftovima DR_{max} . Sa druge strane, u slučaju *Imperial Valley* IV79, *Loma Prieta* LP89, *Parkfield* PA66 zemljotresa i potpunog nestacionarnog veštačkog akcelerograma svi globalni driftovi za nivo ciljnog pomeranja DR_i su manji od maksimalnih globalnih driftova DR_{max} . Nešto je blaža varijanta u slučaju kompatibilizacije akcelerograma, tako da su samo kod četiri vrednosti ugla θ_i globalni driftovi za nivo ciljnog pomeranja DR_i jednaki maksimalnim globalnim driftovima DR_{max} . Srednje vrednosti i medijane globalnih

driftova za nivo ciljnog pomeranja $DR_{t,ave}$ i $DR_{t,med}$ su znatno manje od maksimalnih globalnih driftova DR_{max} pri skaliranju akceleroograma, dok su pri kompatibilizaciji akceleroograma ove vrednosti bliže maksimalnim globalnim driftovima DR_{max} . Relativne vrednosti ukupnih smičućih sila u osnovi zgrade za ciljna pomeranja $(V/W)_t$ su bliže relativnim vrednostima ukupnih smičućih sila u osnovi zgrade $(V/W)_{adeq}$ za maksimalni globalni drift DR_{max} pri kompatibilizaciji akceleroograma, dok su za slučaj skaliranja akceleroograma ova odstupanja veća. U određenim slučajevima relativne vrednosti ukupnih smičućih sila u osnovi zgrade za ciljna pomeranja $(V/W)_t$ su i veće od relativnih vrednosti ukupnih smičućih sila u osnovi zgrade $(V/W)_{adeq}$ za maksimalni globalni drift DR_{max} , što ukazuje na činjenicu da nelinearna krutost postaje negativna nakon dostizanja nivoa ciljnog pomeranja.

Koeficijenti prosečne raspoložive duktilnosti $\mu_{d,max/d,y}$ kod 4x6x3 3D modela zgrade su u intervalu (2÷4), što ukazuje na srednje duktilno ponašanje. U slučaju 9x6x5-12 3D modela zgrade ovaj koeficijent je veći od 4, što ukazuje na visoko duktilno ponašanje. Koeficijenti realizovane (zahtevane) površi duktilnosti $M_{d,v/d,y}$ kod 4x6x3 3D modela zgrade su u intervalu (3÷11), što odgovara srednjem duktilnom ponašanju. Kod svih 3D modela zgrada, i za skaliranje i za kompatibilizaciju akceleroograma, dobijeni su najveće vrednosti globalnih driftova, i za nivo ciljnog pomeranja DR_t i za nivo maksimalnog pomeranja DR_{max} , u najvišim spratovima. Kod 4x6x3 3D modela zgrade dobijeni su najveći međuspratni driftovi za nivo ciljnog pomeranja IDR_t u nižim spratovima, dok se ka višim spratovima njihove vrednosti smanjuju. Kod 9x6x5-12 3D modela zgrade dobijeni su najveći međuspratni driftovi za nivo ciljnog pomeranja IDR_t od prizemlja do četvrtog sprata, dok se kod viših spratova ove vrednosti smanjuju. Kod dve podzemene etaže ovog objekta međuspratni driftovi su značajno redukovani. S obzirom da međuspratni driftovi ukazuju na povredljivost ispune po spratovima, to se vizuelnom prezentacijom njihovih površi može ukazati na potencijalna mesta oštećenja.

5 ZAKLJUČAK

Na osnovu istraživanja u doktorskoj disertaciji se mogu formulisati sledeći zaključci:

- Originalno razvijena površ akceleroograma omogućava njegovu prezentaciju akceleroograma kroz komponente koje se mogu koristiti za analizu 3D modela zgrada. Sa druge strane, prezentacija površi akceleroograma daje uvid u trodimenzionalni prikaz u cilindričnom ili ortogonalnom koordinatnom sistemu. Jednostavnost u generisanju komponentata akceleroograma i transformacija istih u površ akceleroograma potvrđuje na dodatu pragmatičnost razvijene metodologije.

- Analizom površi akceleroograma, odnosno njihovih komponentata akceleroograma dokazano je da je akceleroogram $a_{\theta}(t)$ za ugao θ identičan akceleroogramu sa negativnim predznakom $-a_{\theta+180}(t)$ za ugao $\theta+180^{\circ}$. Posledica ovako izvedenog stava je da za mere intenziteta IM važi ista analogija (radijalna simetrija). S obzirom da se određene IM mere proračunavaju za apsolutne diskretne vrednosti, to one postaju identične za uglove θ i $\theta+180^{\circ}$.

- Razvijen je i analiziran koeficijent raspoložive površi IM mera C_{IM} . S obzirom da je maksimalna moguća vrednost koeficijenta raspoložive površi IM mera $C_{IM}=1$, to vrednosti bliže jedinici ukazuju na veću jednakost među IM merama (kružni oblik anvelope IM mere). Sa druge strane, manje vrednosti ukazuju na postojanje jednog izraženog glavnog pravca IM mere i jednog ortogonalnog sporednog pravca IM mere (elipsoidni oblik anvelope IM mere).

- Originalno razvijena površ spektra odgovora ubrzanja i površ spektra odgovora pomeranja omogućuje prezentaciju spektara odgovora kroz komponente koje se mogu koristiti za analizu 3D modela zgrada.

- Istraživanjem su razmatrani aspekti skaliranja i kompatibilizacije akceleroograma i njihov uticaj na formu površi spektra odgovora. Utvrđeno je da površi spektara odgovora generisane kompatibilizacijom imaju domen konstantnih ubrzanja veći, u odnosu na površi spektara odgovora generisane skaliranjem po metodi najmanjih kvadrata. Sa druge strane, s obzirom da je kompatibilizacija izvršena u odnosu na elastičan spektar dogovora prema propisima, to generisane površi spektara odgovora imaju određeni stepen rotacione simetrije oko vertikalne ose spektralnog ubrzanja S_a .

- Originalno razvijena *pushover* površ omogućuje prezentaciju nelinearnog odgovora 3D modela zgrada. S obzirom da se dosadašnja istraživanja zasnivaju na *pushover* krivama, to je ovakav koncept prezentacije prvi put predstavljen u predmetnoj doktorskoj disertaciji. Prednosti u prezentaciji nelinearnog odgovora 3D modela zgrada primenom *pushover* površi su: jasna geometrijska prezentacija kroz detaljan prikaz kompletnog nelinearnog odgovora po svim uglovima i mogućnost da se primene dalja matematička razmatranja.

- Na osnovu uvedenog novog koeficijenta površi duktilnosti M_μ moguće je razmatrati nelinearan odgovor 3D modela zgrada uzimajući u obzir duktilnost za sve pravce, odnosno uglove dejstva zemljotresa. Regresionom analizom i koeficijentom korelacije utvrđeno je da se veza između koeficijenta duktilnosti i koeficijenta površi duktilnosti uspostavlja primenom stepene funkcije.

- U odnosu na NSPA *pushover* površ, istraživanjem je pokazano da INDA (IDA) *pushover* površ ima drugačiji oblik, s obzirom da je primenjena druga mera intenziteta IM i inženjerski parametar zahteva EDP . U domenu nelinearnog ponašanja, odnosno u domenu kolapsa INDA *pushover* krive se asimptotski približavaju horizontali, tako da celokupna INDA *pushover* površ postaje gotovo horizontalna. Ovo je posledica toga što u kolapsnom domenu razlika u vrednostima driftova za dve uzastopne NDA analize postaje izuzetno velika.

- Razmatrajući realizovane NSPA *pushover* površi u prostornim koordinatama utvrđeno je da je kod svih 3D modela zgrada krutost u nelinearnom domenu gotovo horizontalna $K_r \approx 0$ ili blago pozitivna, osim u pojedinim slučajevima, gde sa povećanjem nelinearnih deformacija krutost postaje negativna $K_r < 0$. Kada se razmatraju samo $E_x + 0.3E_y$ i $E_x - 0.3E_y$ seizmička dejstva, što bi bilo kod standardnih seizmičkih analiza, tada se i dobijaju gotovo najveće vrednosti nelinearnih deformacija za pravac gde se uzima 100% seizmičko dejstvo, a najmanje vrednosti nelinearnih deformacija za pravac gde se uzima 30% seizmičko dejstvo. Za ova seizmička dejstva ortogonalna projekcija ($DR_x - DR_y$ ravan) NSPA *pushover* površi ima elipsoidan oblik sa uklještenjem na mestu manjeg poluprečnika elipse. Kod 3D modela zgrade, projektovane prema SRP propisima, realizovane su znatno manje maksimalne relativne vrednosti ukupne smičuće sile u odnosu na relativne vrednosti ukupne smičuće sile kod 3D modela zgrada projektovanih prema EN (EC) tehničkim propisima.

- Pri kompatibilizaciji akceleroograma se dobijaju ujednačenija rešenja u pogledu globalnih driftova za nivo ciljnog pomeranja DR_r , nego što je to slučaj kod skaliranja akceleroograma metodom najmanjih kvadrata. Međutim, preporuka je da se, s obzirom na kompleksnu problematiku kompatibilizacije akceleroograma, dovoljno pouzdano u praktične svrhe može koristiti skaliranje akceleroograma primenom metode najmanjih kvadrata.

- Originalno razvijena anvelopa ciljnog pomeranja omogućava prezentaciju maksimalnog projektnog nivoa pomeranja 3D modela zgrade po uglovima θ_i i proračun odgovarajućih koeficijenta za ocenu performansi zgrada.

- Kod svih 3D modela zgrada, i za skaliranje i za kompatibilizaciju akceleroograma, dobijene su najveće vrednosti globalnih driftova i za nivo ciljnog pomeranja DR , i za nivo maksimalnog pomeranja DR_{max} u najvišim spratovima. Istraživanjem je utvrđeno da su kod svih 3D modela zgrada koeficijenti $M_{d,LS}$ i $M_{d,CP}$ znatno manji od 1 (do 0.25). Povoljna rešenja se mogu smatrati kada je $M_{d,LS} < 1$, što bi značilo da pri zahtevanom duktilnom ponašanju zgrade nije prekoračen LS performansni nivo.

LITERATURA

Zbog ograničenja prostora ovde nije dat detaljan prikaz i širi popis literature koji se nalazi u doktorskoj disertaciji.

Marko Pavlović¹, Zlatko Marković², Milan Veljković³

NOSIVOST ZAVRTNJEVA KAO SREDSTVA ZA SPREZANJE U PREFABRIKOVANIM SPREGNUTIM KONSTRUKCIJAMA

Rezime:

P-5

Prefabrikacijom betonske ploče skraćuje se vreme izgradnje i ukupni troškovi tokom životnog veka spregnutih konstrukcija zgrada i mostova. U ovom članku prikazano je istraživanje mogućnost upotrebe zavrtnjeva kao sredstva za sprezanje. Rezultati statičkih „push-out“ eksperimenata sa zavrtnjevima M16 i M24, klase čvrstoće 8.8, i parametarske studije ukazuju da se zavrtnjevi sa navrtkama sa obe strane nožice čeličnog profila mogu uspešno koristiti kao sredstva za sprezanje. Za ovaj tip sprezanja betonske ploče i čeličnih nosača razvijeni su proračunski modeli za nosivost i klizanje (deformacija) u trenutku loma, t.j. duktilnost.

Ključne reči: prefabrikovani spregnuti nosači, sredstva za sprezanje, visokovredni zavrtnjevi, nosivost na smicanje, duktilnost

RESISTANCE OF BOLTED SHEAR CONNECTORS IN PREFABRICATED STEEL-CONCRETE COMPOSITE DECKS

Summary:

Prefabrication of concrete slabs reduces construction time and the total life-cycle cost of composite steel-concrete buildings and bridge decks. Possibility to use bolted shear connectors embedded in prefabricated concrete slabs is investigated in the research presented in this paper. Results of static push-out experiments on M16 and M24 bolts, (grade 8.8) and parameter study based on validated FEA indicate that the bolts with nuts on both sides of the steel flange can be successfully used as the demountable shear connectors. Calculation models that can be used in the design are provided for the strength and ductility of the shear connectors.

Key words: prefabricated steel-concrete composite beams, shear connectors, high-strength bolts, shear resistance, ductility

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1 INTRODUCTION

Steel-concrete composite decks is competitive structural system in buildings and bridges.. In-situ casted concrete often requires temporary supports and formwork, which leads to a longer construction time compared to prefabricated concrete slabs. Composite action between a steel profile and a concrete slab is most commonly established by grouting grouped headed studs welded to the top flange of the steel section in envisaged openings (pockets) of prefabricated concrete slabs. The time needed for grout hardening and difficulties at dismantling stage reduces competitiveness of the composite slab.

By using bolted shear connectors, faster erection methods can be developed, as illustrated in Figure 1(a). Bolts can be casted in prefabricated concrete slabs and on site assembled to the top flange of the steel section part of composite member. Long-term behaviour and durability issues may require replacement of concrete slabs or their parts during maintenance of composite bridge decks. With the use of bolted shear connectors easier dismantling for the purpose of replacement, removal or reuse of concrete slabs can be achieved.

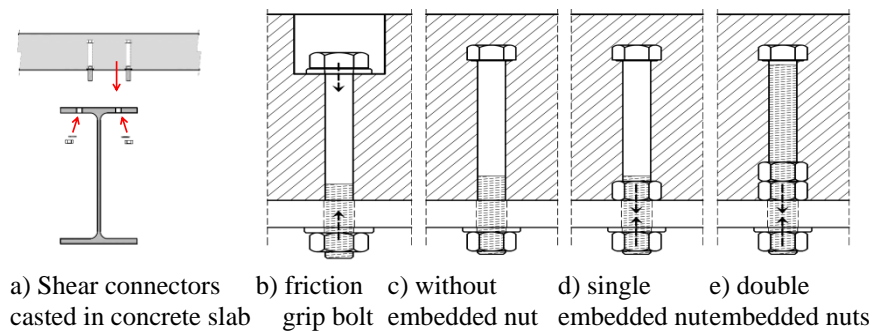


Figure 1 - Prefabrication with use of bolted shear connectors

Various types of bolted shear connectors shown in Figure 1(b) to (e) were analysed in previous research mostly by push-out experiments. Friction grip bolts shown in Figure 1(b) have been investigated by Marshall et al. [1], Kwon [2] and Lee and Bradford [3]. They have high initial stiffness because the shear force is transferred by friction but the problem is creep of concrete through the thickness of the concrete slab which influences large long term loss of the preloading force in the bolt. Embedded bolted shear connectors shown in Figure 1(c), were investigated by Dedic and Klaiber [4], Hawkins [5], Sedlacek et al. [6], Lam et al. [7], Schaap [8] and Kwon [2], Pavlović [9]. Transfer of the interface shear force is achieved by bearing in concrete and hole in the steel flange and shear across the threaded part of the bolt. Nearly double shear resistance can be achieved compared to friction grip bolts but the initial stiffness is lower because of the slip and rotation of the bolt in the hole. Initial stiffness is improved by embedding the nuts in the concrete, as shown in Figure 1 (d) and (e), thus preventing rotation of the bolt at the connection to the flange of the steel beam. The embedded also facilitates the prefabrication process by mounting the shear connectors to both sides of a template steel sheet during casting of the prefabricated concrete slabs. The aim of the research presented here is to provide calculation models for strength and ductility of bolted shear connectors with embedded nuts that can be used in the design, based on push-out experiments validated by Finite Element Analysis (FEA) and parameter study.

2 EXPERIMENTS AND NUMERICAL ANALYSIS

Eight push-out specimens of bolted shear connectors M16 and M24, grade 8.8, with single embedded nut were prepared and tested according to EN1994-1-1 [10] test set-up at University of Belgrade, Faculty of Civil Engineering, see Figure 2. Bolts are preloaded to half of the full preloading force. Two fundamentally different failure modes are observed in series with M16 and M24 bolts, shear failure of the bolts and pryout failure of concrete, respectively.

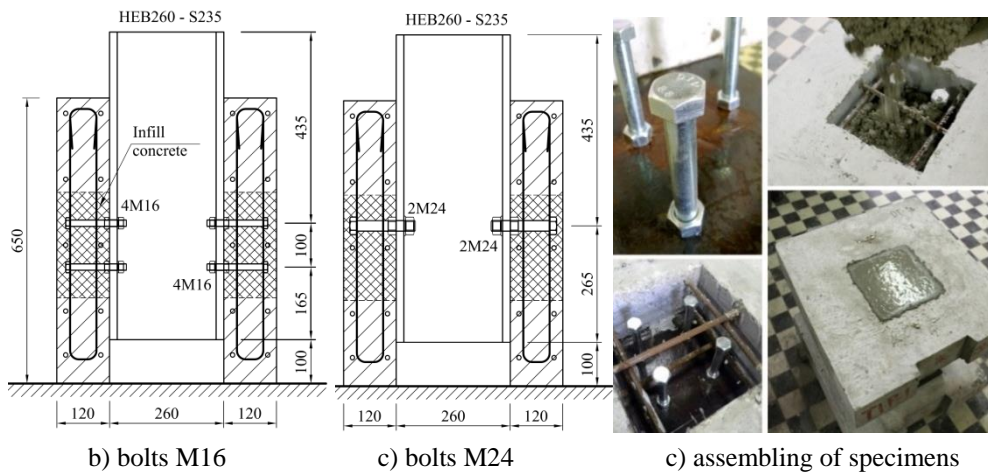


Figure 2 - Push-out tests.

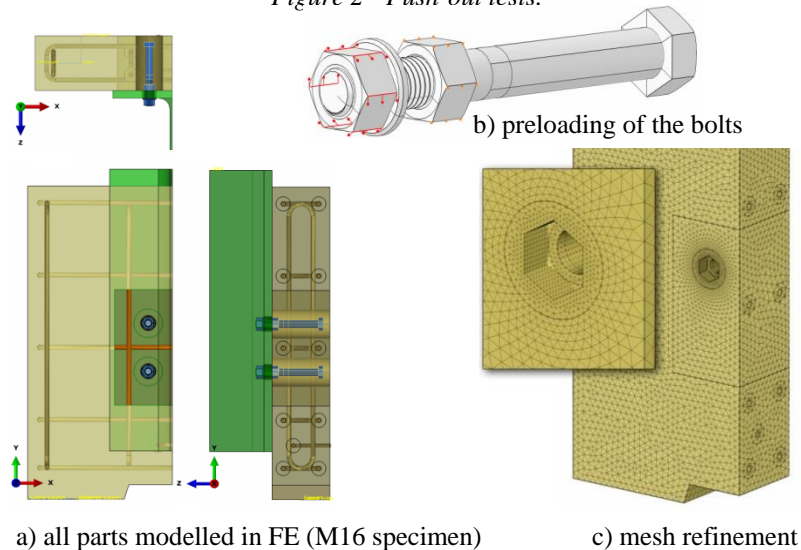


Figure 3 - Details of FE models of push-out tests

By using the same set of boundary conditions and material model parameters in both cases of the M16 and M24 bolts, very good agreement is achieved between the results of numerical analysis and experiments, see Figure 4. The difference in shear resistance obtained numerically

and in experiments is in range of 3 % while the slip capacity varies up to 10 %. Since the damage material models are used for the concrete and the bolt, both failure modes from experiments: shear failure of bolts and pryout failure of concrete in case of M16 and M24 bolts, respectively, are successfully represented in FEA.

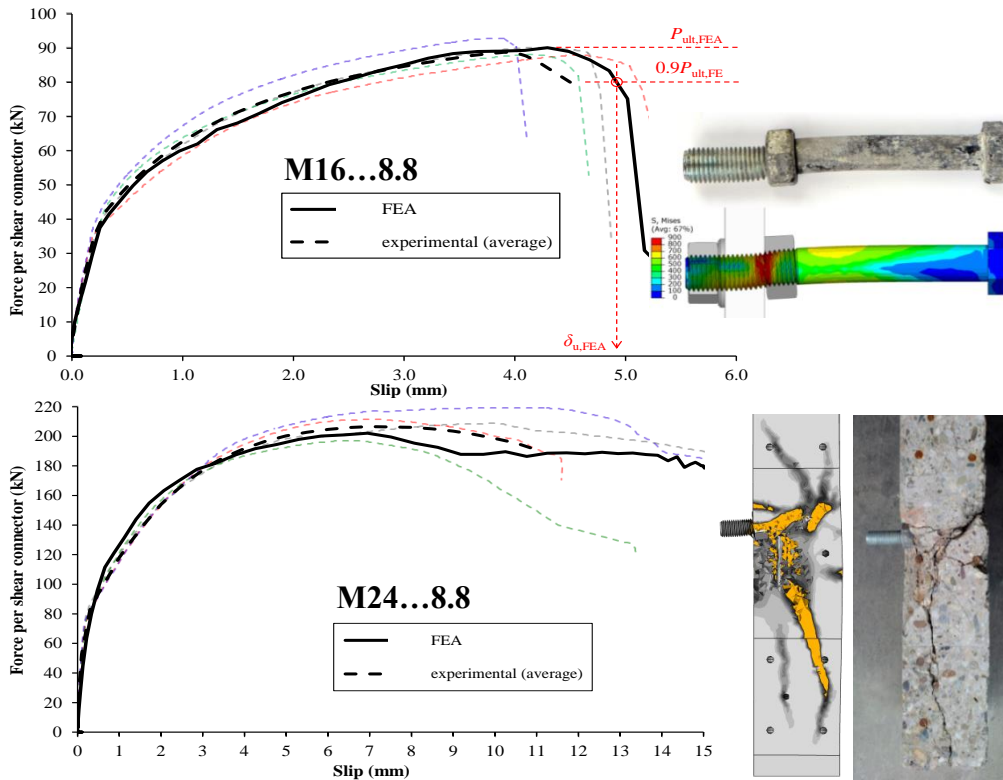


Figure 4 - Force-slip curves from experiments and FEA

3 PARAMETER STUDY

Parameter study, considering parameters shown in Table 1, is performed based on FE models that are validated by push-out experiments in order to obtain data for development of calculation models for shear resistance P_{Rd} and ductility δ_u (slip capacity).

The initial parameter study on bolts M16, including parameters P1 to P4 in Table 1, shows that the bolt preloading force F_p and number of embedded nuts have no influence on the shear resistance or the ductility. Longitudinal spacing $s=5d$ was identified as the minimum required, providing the independent (non-group) behaviour of bolted shear connectors. Shear connector height h_{sc} showed to have large influence on the failure mode.

The main parameter study included three parameters, P4 to P6 in Table 1, that are identified to have the most important influence on the shear resistance and the ductility of the shear connector. Example of force-slip curves for different bolt diameters and concrete strengths,

with height to diameter ratio $h_{sc} / d = 4$ are given in Figure 5. The failure modes can be identified directly from the force-slip curves: bolt failure and concrete failure.

Table 1 - Parameters and ranges considered in the parameter study

Parameter	Parameter label	Designation	Range
Bolt preloading force	P1	F_p (%)	0, 25, 50, 75, 100
Number of embedded nuts	P2	n (-)	1, 2
Longitudinal spacing ratio	P3	s / d (-)	2.5 – 6.25
Shear connector height ratio	P4	h_{sc} / d (-)	2.5 – 6.25
Blot diameter	P5	d (mm)	12, 16, 20, 24
Concrete class	P6	f_{cm} (MPa)	28, 38, 48, 58

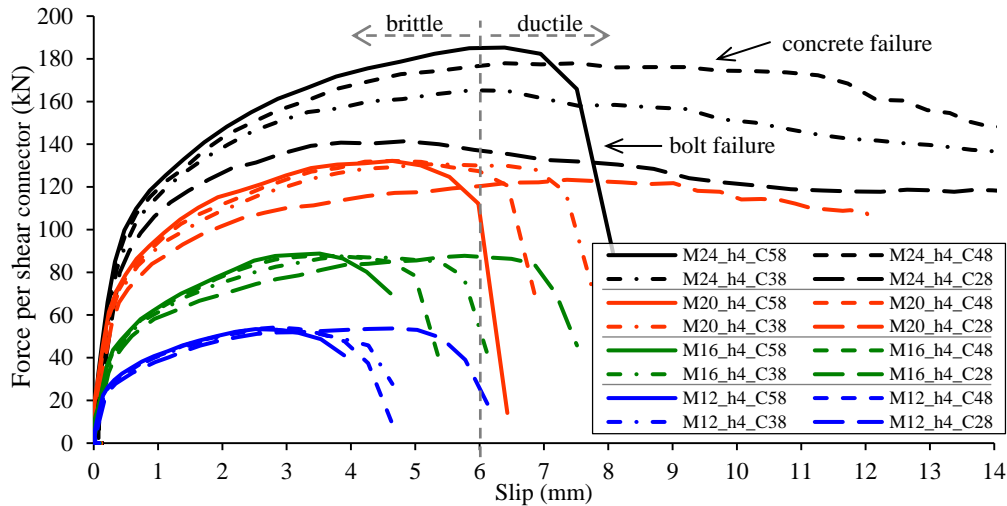


Figure 5 - Force-slip curves from the parameter study - results for $h_{sc}/d=4$

4 FAILURE MODES

Two failure modes of bolted shear connector are illustrated in Figure 6(a) showing models with different bolt diameters but with same height and concrete class at the stage of ultimate load. The case with bolts M12, $h_{sc} = 60$ mm ($h_{sc} / d = 5$) exhibited bolt failure at the threaded part, while the case with larger diameter and the same height, M20, $h_{sc} = 60$ mm ($h_{sc} / d = 3$), exhibited pryout failure of the concrete. Maximum principal strains (PE, Max. Principal) are shown, indicating both shearing strains in the bolts and tensile cracks in concrete. Additionally, variable of tension damage in concrete DAMAGET [11] is overlapped and shown with the red colour. In the case of bolt failure (M12) tensile crack in concrete behind the shear connector is relatively small, and does not reach the outer concrete surface at the ultimate load. In the case of concrete pryout failure (M20) the crack located behind the shear connector

which is characteristic for the concrete pryout failure, exhibits some large plastic strains indicating the crack opening up to the surface of concrete slab. This crack is illustrated in Figure 6(b), by isosurfaces of maximum principal strains between values 0 and 0.05, having a shape of half of the concrete cone surface typical for the headed anchor bolts loaded in tension.

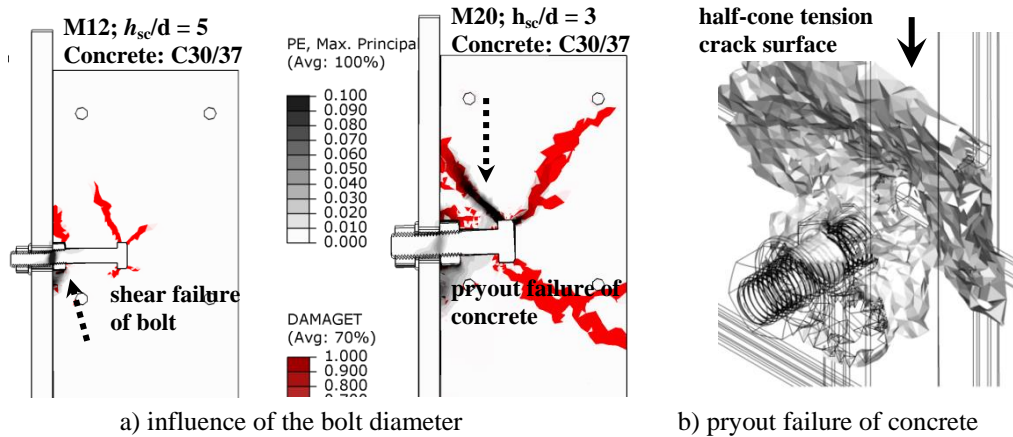


Figure 6 - Failure modes described with use of FEA results

4.1 BOLT FAILURE

M16 bolted shear connectors exhibited bolt failure both in experiments and FEA. Characteristic shearing shape of a failure area is shown in Figure 7(a). The obtained shear resistance in push-out test is approximately 20% higher when compared to the pure shear resistance of the bolt according to EN1993-1-8 [12].

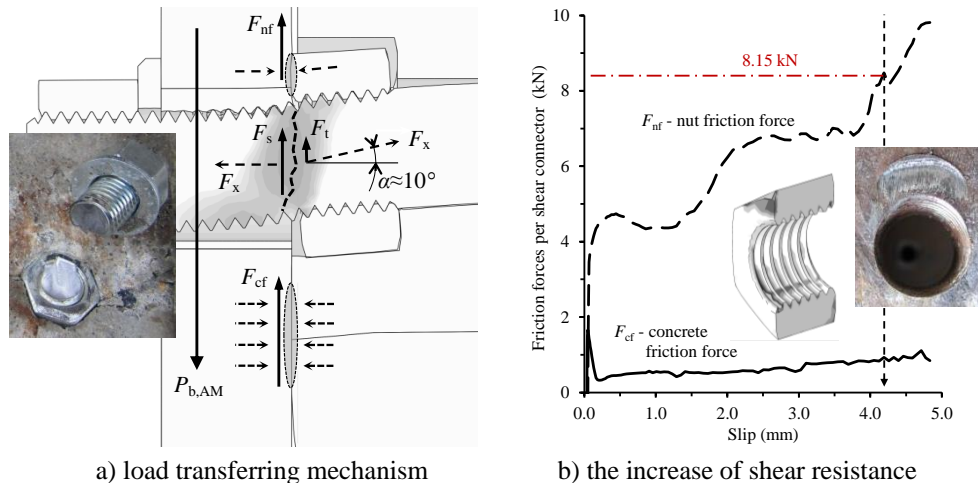


Figure 7 - Bolt failure mode

The increase in load-bearing capacity for the bolt failure mode comes from friction and contact forces acting on the embedded nut and concrete as well as the catenary effects in the

bolt, see Figure 7(a). Inclination of the embedded nut results in high contact stresses at the nut-flange interface, thus producing friction forces. Simple friction at the nut-flange interface is improved by the contact between the nut edge and the vault surface of the flange (groove) produced by the nut inclination, as shown in Figure 7(b). Shear resistances for the cases where bolt failure occurred are shown in Figure 8(a) in function of all the three main parameters that are analysed in the parameter study. It is concluded that shear resistance for the bolt failure is not dependent on parameters h_{sc}/d and the concrete strength f_{cm} .

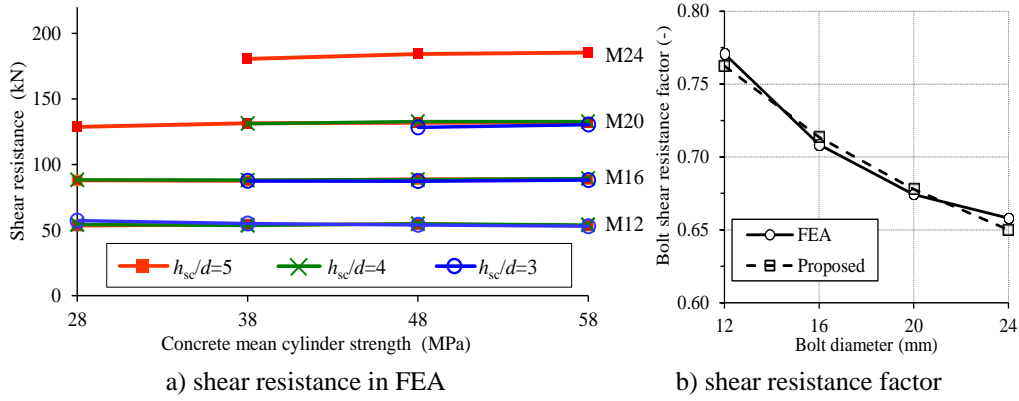


Figure 8 - Shear resistances of bolts

A bolt shear resistance factor $\alpha_{b,FEA}$, as introduced according to Eq. (1), is shown in Figure 8(b) in function of the bolt diameter. The bolt shear resistance factor $\alpha_{b,FEA}$ decreases with the increase of the bolt diameter because for the larger bolt diameters, higher bending stiffness of the bolt will induce lower embedded nut inclination and therefore lower nut friction force.

$$\alpha_{b,FEA} = P_{b,FEA} / f_{ub} A_s \quad (1)$$

Calculation model (CM) for the bolt shear resistance $P_{b,u}$ is proposed in Eq. (2) in similar form as the shear resistance of bolts according to EN1993-1-8 [12]. Instead of factor $\alpha_v=0.6$ used in [12], a bolt shear resistance factor α_b dependent on the bolt diameter, is introduced to take into account the increases described above.

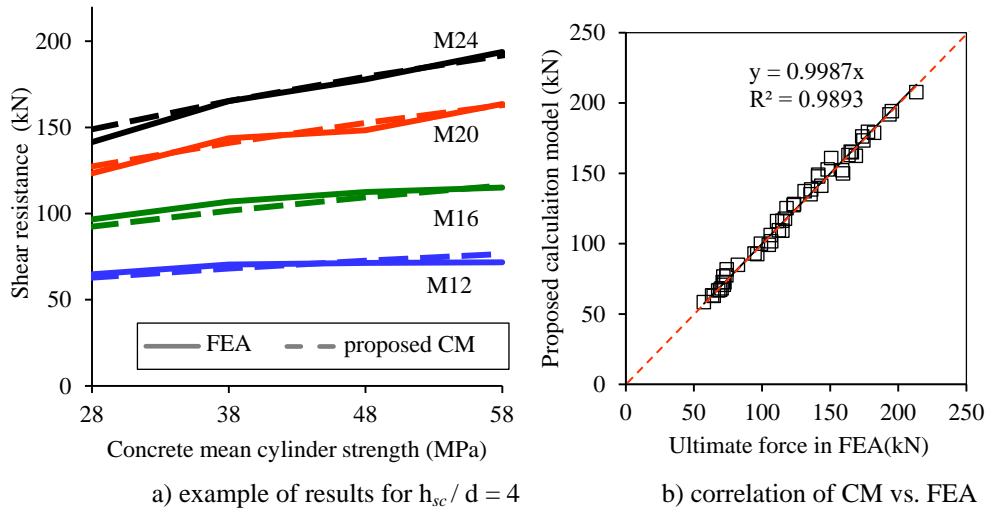
$$P_{b,u} = \alpha_b f_{ub} A_s, \text{ with } \alpha_b = 0.6(34 \text{ mm}/d)^{0.23}, \text{ for } d \geq 12 \text{ mm} \quad (2)$$

4.2 CONCRETE FAILURE

Concrete failure mode for bolted shear connector loaded in shear is governed by its pryout resistance, as shown in Figure 6, which relies on confinement effects in concrete in front of the shear connector. The calculation model for shear resistance of concrete is proposed in a form similar to equation given in Japanese Standard Specifications for Steel and Composite Structures [13] for welded headed studs, based on curve fitting of the parameter study results. The proposed criterion is given in Eq. (3), with concrete shear resistance reduction factor for large bolt diameters α_c .

$$P_{c,u} = 55 \alpha_c d^{1.9} (f_{cm} h_{sc}/d)^{0.4} + 22000 ; \text{ with } \alpha_c = 22.5/(d+3) \leq 1.0, \text{ } d \text{ in (mm)} \quad (3)$$

Comparison of proposed calculation model to the results of the parameter study is shown in Figure 9. Low variation coefficient $V_X=0.036$ and very high coefficient of determination $R^2 = 0.99$, are achieved.



a) example of results for $h_{sc} / d = 4$ b) correlation of CM vs. FEA
 Figure 9 - Calculation model for the concrete failure vs. FEA results.

5 SLIP CAPACITY - DUCTILITY

High slip capacity is obtained in all cases in the parameter study where concrete failure is the governing, or where the resistance of the bolts is close to the resistance of the concrete, see Figure 5. This means that the crushing of the concrete in front of the shear connector provides the ductile behaviour of the connection.

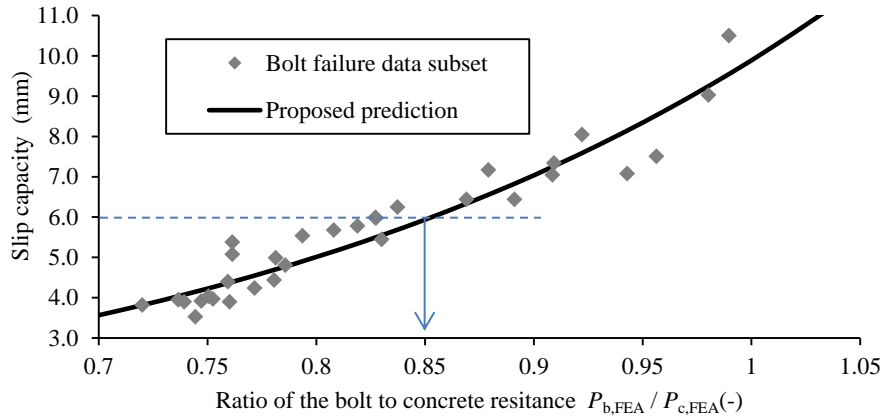


Figure 10 - Calculation model for the slip capacity

The calculation model for the slip capacity δ_u is proposed in Eq. (4), based on the ratio of the bolt resistance to the concrete resistance: $P_{b,FEA}$ and $P_{c,FEA}$, see Figure 10, The ductility

criterion $\delta_{uk} > 6$ mm, specified in EN1994-1-1 [10] is fulfilled providing that the ratio of the bolt and concrete strength is $P_{b,u} / P_{c,u} > 0.85$. Using the high strength bolts is favourable for the slip capacity because relatively high shear resistance of the bolts is achieved compared to resistance of the concrete due to small diameters needed. If lower bolt grades would be used, e.g. grade 5.6, the higher ductility of the bolt material is favourable for the slip capacity. However it would be hard to achieve that the resistance of the bolts is close to the resistance of the concrete for the usual concrete strengths and diameters of the bolts that are used.

$$\delta_u = 0.33e^{3.4(P_{b,FEA} / P_{c,FEA})}, \text{ (mm)} \quad (4)$$

6 VALIDATION OF CALCULATION MODELS

Calculation models for shear resistance and slip capacity are validated with regard to the results of own and experiment results from the literature. Ranges of the parameters covered by the experiments are: concrete strength: $f_{cm} = 22\text{--}48$ MPa; bolt tensile strength: $f_{ub} = 780\text{--}1160$ MPa; bolt diameter: $d = 16\text{--}24$ mm; height ratio $h_{sc} / d = 4.4\text{--}7.9$. Figure 11 show that the proposed calculation models predict the results of experiments very well.

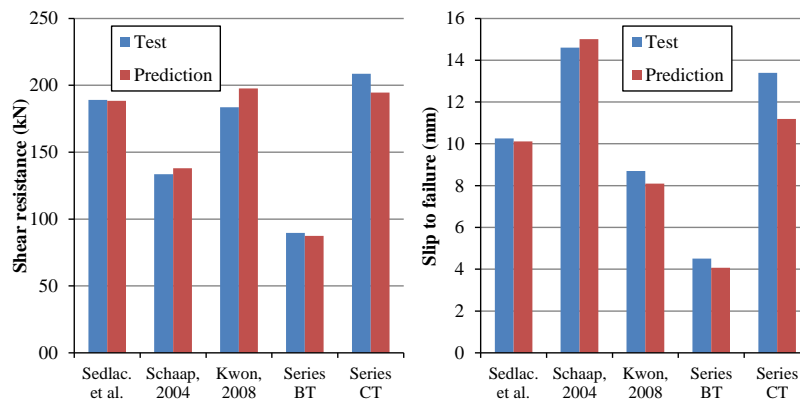


Figure 11 - Correlation of proposed calculation models to the results of experiments

7 CONCLUSIONS

Bolted shear connectors with single embedded nut are analysed with focus on their resistance and ductility in static push-out test. Following main conclusions are drawn:

- 1.) Shear resistance for the bolt failure mode is increased up to 30% (for M12) when compared to pure shear resistance of the bolt at the threaded part. This effect is achieved by the embedded nut, concrete friction interaction with the flange and catenary effects in the bolt.
- 2.) Calculation models for the bolt and concrete failure: $P_{b,u}$ and $P_{c,u}$, respectively, are proposed as functions of the bolt diameter d , bolt tensile strength f_{ub} , height to diameter ratio h_{sc} / d , and mean cylinder compressive strength of concrete f_{cm} . The proposed

calculation models are validated by own and experiment results from the literature and very good correlation is obtained.

- 3.) Calculation model for the slip capacity δ_u is given based on the ratio of the bolt to concrete resistance $P_{b,u} / P_{c,u}$ and very good correlation to result of relevant experiments are obtained. Ratio of bolt and concrete resistance higher than 0.85 is needed in order to consider the bolted shear connector to be ductile according to EN1994-1-1 [10].

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U1 – U5

UVODNA IZLAGANJA



Rüdiger Höffer

MODERNE ZGRADE I KONSTRUKCIJE – DOPRINOS PRORAČUNU NA DEJSTVO VETRA

Rezime:

U-1

U radu su opisani odabrani primeri nedavno izgrađenih, modernih konstrukcija koje su zahtevale nova istraživanja i proračune kako bi postale tehnički izvedivljive. Osnovne strategije modeliranja dejstva vetra i efekata vetra prema Eurocode 1991-1-4, kao primer moderne proračuna, su opisani. Prikazane su kreativne metode istraživanja i objašnjeni su novi eksperimenti izvršeni za analizu dejstva vetra na oscilovanje konstrukcija. Takođe, predstavljeni su primeri o proceni vibracija izazvanih vetrom za potrebe projektovanja konstrukcija, kao i uloga praćenje vetrom opterećenih konstrukcija radi detekcije oštećenja.

Keywords: proračun dejstva vetra, proračun, vibracije, praćenje stanja konstrukcije

MODERN BUILDINGS AND STRUCTURES – CONTRIBUTIONS FROM WIND ENGINEERING

Summary:

The paper presents selected examples of recently erected, modern structures requiring high demands for novel research and design tasks to become technically feasible. The basic strategies of modeling wind actions and wind effects for design purposes after Eurocode 1991-1-4 as an example of modern codification are outlined, creative investigation methods are presented, and novel experiments on wind structure interactions at oscillating structures are explained. Examples on the assessment of wind-induced vibrations for structural design, and on the role of health monitoring of wind loaded structures for damage detection are presented.

Key words: wind engineering, codification, structural vibrations, health monitoring

1 INTRODUCTION

Structural building aerodynamics is a core sector in the field of wind engineering which deals with the actions of wind and its effects at structures and building components. This sector relates to the field of turbulence theory and fluid mechanics, structural mechanics, dynamics of structures, probability theory, and design of civil engineering structures. For common cases the wind loads for the structural design are sufficiently exact when being determined after the respective codification. The framework for all loading specifications within the Structural Eurocodes is specified by the EN 1990, *Basis of Structural Design*, the wind actions are calculated with part 1-4 of the Eurocode 1991, *Actions on Structures*, [1].

Wind effects can play a dominant role in the design of ambitious building projects showing high architectural and/or structural complexity. Recent examples are the new telecommunication tower at Skopje on the mount Vodno (figure 1) and the new bridge crossing the river Sava at Belgrade (figure 2). The aerodynamic coefficients of the shown tower shaft, the platforms and the curved columns cannot be derived from code stipulations but are urgently required to predict the ensemble of wind loads acting at the tower. Wind tunnel tests became therefore necessary. This is also true for specific rope and girder oscillations of the Sava crossing as well as for many other bridges of this and similar types.



*Figure 1 –
Animation of the
Telecommunication Tower
at Skopje, [2]*



*Figure 2 –
Side view of Ada Bridge, the new Sava crossing at Belgrade with
pylon, cables and girder, [3]*

In such cases the Eurocode foresees design assistance by testing and measurements which often is realized as wind tunnel experiments, but also properly validated numerical methods or

technical information gained from appropriate full scale data are taken into account (see sections 1.5 in [1])

2 DETERMINATION OF WIND EFFECTS RELEVANT FOR DESIGN

2.1 APPLICATION OF CODES

The code “EN 1990: Basis of Structural Design” defines as a basic requirement that the verification of a structural design shall be performed in the serviceability state, i.e. comfort and appearance, and in the ultimate limit states, i.e. collapse or other forms of structural failure. The partial factor method is applied, in which the resistance and the action effects are represented in each of the limit states by special values, defined in terms of statistical parameters of the respective probability distributions. It can be associated with the serviceability limit state. The design value applied in the collapse situation is the characteristic value increased by the partial safety factor, 1.5 in the case of a wind load. The action effect on the structure is calculated at the increased load level, i.e. non-linear effects are included in the procedure in principle.

With Part 1-4 of the Eurocode ([1], s. also table 1) wind actions are calculated based on characteristic values of wind velocity or velocity pressure as well as using multiple types of coefficients such as aerodynamic and structural coefficients. The basic values describing the wind climate are characteristic values having annual probabilities of exceedance of 0.02, commonly referred to as having a mean return period of 50 years. The wind velocity for the design is represented as a *wind map velocity*. The fundamental value of the basic wind velocity, v_{b0} , shall be determined as the characteristic 10 minutes mean wind velocity, irrespective of wind direction and time of year, at 10 m above ground of terrain with low vegetation such as grass and isolated obstacles with separations of at least 50 obstacle heights. The *seasonal effects* and the *terrain roughness effect* can be and the *effect from orography* (hills, cliffs, escarpments) must be examined, *directional effect due to prevailing winds* may be investigated as well [2].

Table 1 - Overview over the Eurocode EN 1991-1-4

- *Design Situations*
- *Modelling of Wind Actions*
- *Modified Gust Response Factor Concept*
- *Wind Velocities for Design*
- *Accounting for Wind Direction*

In view of the stochastic and partially dynamic character, equivalent static loads are specified, representing (to the best of our knowledge) the load effect on the structure. In this manner, the structural behaviour comes into the picture. Therefore, when there is a non-linear load effect such as the resonant gust response, the load level at which the equivalent load is investigated, must be specified. The once in 50 yrs wind has been chosen. The alternative option could have been to identify the equivalent static loads at the ultimate limit state level, at which the return period is in the order of several thousands of years. However, this is a rather fictitious event beyond our experience and does not seem very real. Since building structures are normally not very prone to wind induced, resonant vibrations, the non-linearity is weak and can be included in the partial safety factor. The considered design situations in EN 1991-1-4

are design against collapse, design for serviceability, and design against fatigue, here due to along wind turbulence and due to vortex excitation in cross wind directions.

2.2 WIND TUNNEL INVESTIGATIONS

One of the most effective research tools in wind engineering is a boundary layer wind tunnel which can generate a downscaled physical model of the atmospheric flow during strong storms. Figure 3 shows the boundary layer wind tunnel (BLWT) at the Ruhr-University of Bochum. It was constructed and put into action in 1977. An important feature is the artificial increase of the boundary layer after Counihan's system through the castellated barrier, the spires at the inlet and the roughness field. Building models of ca. 1:100 down to 1:500 are placed in the test section at the turntable.

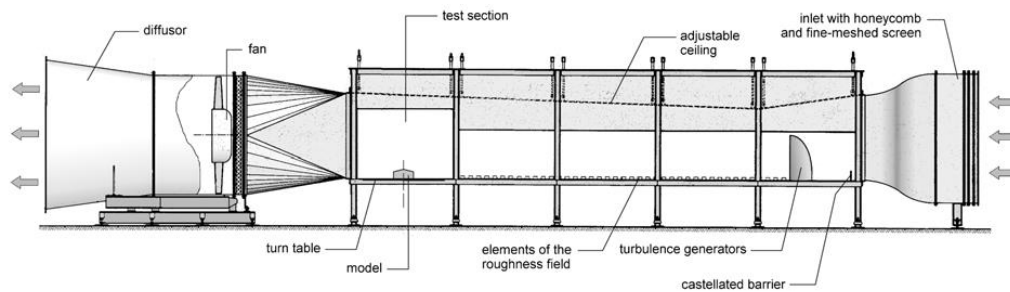


Figure 3 – Boundary layer wind tunnel of the Ruhr-Universität Bochum, Building Aerodynamics Laboratory



Figure 4 – Wind tunnel model of a cooling tower for the measurement of surface pressures and internal forces

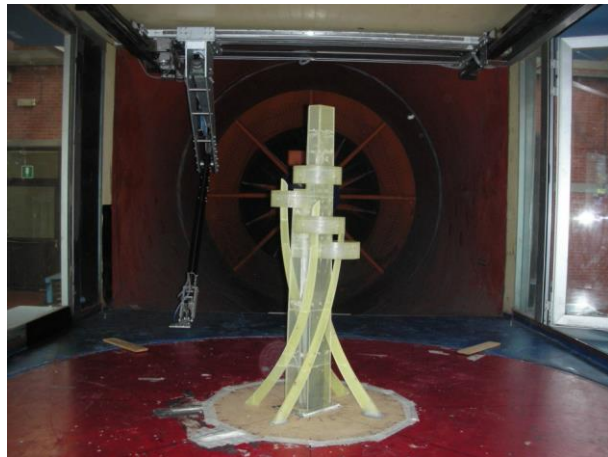


Figure 5 – Wind tunnel model of the telecommunication tower at Skopje for the measurement of surface pressures, [4]

Sensors at the model's surface or at the model's fixings and a data acquisition system are used to register time histories of wind speeds and wind responses, such as surface pressures, support forces, internal forces and deformations. Figure 4 shows a deformable model of a cooling tower where pressure taps and strain gauges are monitored simultaneously. The time histories are statistically evaluated and mean values, fluctuation measures and spectral quantities are determined. Figure 5 shows a detailed tower model of fluid mechanical similarity for pressure measurements.

2.3 FULL SCALE MEASUREMENTS

Measurements at executed structures are predominantly done for the purpose of performance supervision in the framework of structural health monitoring (SHM) campaigns. An advanced project is explained in chapter 3.1. Few examples exist on tests of wind impacts at buildings. These experiments are nearly exclusively undertaken for the purpose of research. An example is the investigation on the damage accumulations due to the excitation of cyclic stresses at a hanger of an arch bridge (see figure 6). An optical sensor was used, the experiment and the evaluation is presented in literature [5].

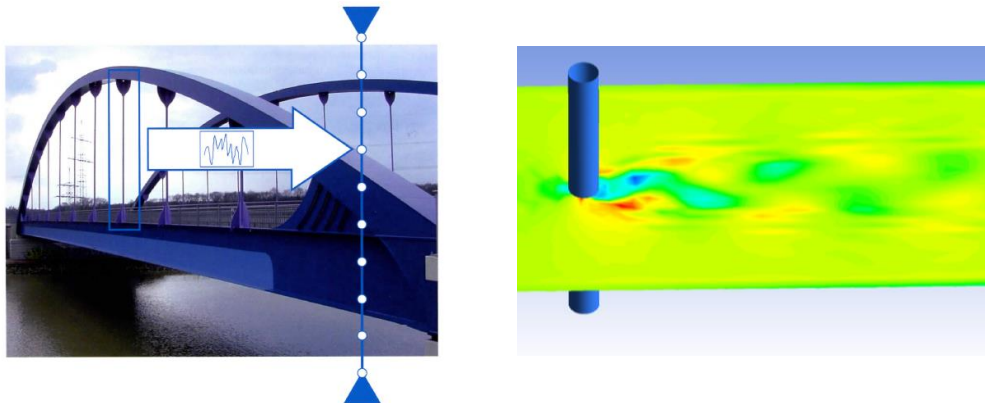


Figure 6 – Loaded hanger rod of a steel arch bridge (left) and CFD calculation of the load generations through regular vortex separations of von-Kármán type (right)

2.4 CALCULATIONS BASED ON COMPUTATIONAL FLUID DYNAMICS

For a certain family of tasks in structural wind engineering computational solutions are needed more and more. Computational Fluid Dynamics (CFD) is able to solve problems which deal with a turbulent fluid flow, transport, heat transfer and other physical properties by solving the equations of fluid motion, heat transfer and other physical processes in a region of interest with specified boundary conditions. CFD has gained a considerable importance also in the field of applications. It has been shown that e.g. the complex mechanism of bridge flutter can be investigated applying Large Eddy Simulations of the aeroelastic derivatives (aeroelastic coefficient functions); a finite volume grid is shown in figure 7. The solutions have been tested by performing specifically designed validation experiments. Chapter 3.2 reports on the numerical treatment of bridge girder aero-elasticity.

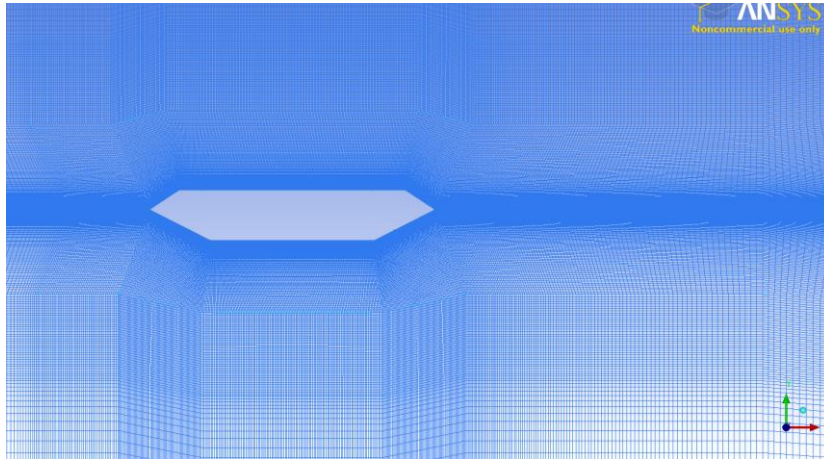


Figure 7 – Discretization of the fluid continuum around the bridge deck

3 SELECTED EXAMPLES

3.1 HEALTH MONITORING OF WIND ENERGY STRUCTURES

The supervision of the correct operation of wind energy converters with respect to a reliable prediction of structural lifetime is of high economical relevance. Modern structural lifetime predictions can be based on a hybrid combination of continuing health monitoring and extensive numerical modeling. The conceptual core of such advanced approach is a continuous adaption of a complete numerical model of the considered structure with respect to the transient system behavior of the actual facility including as accurately as possible the current state of existing or evolving damages and deteriorations. The required data for the model adaption can be advantageously completed through experiments in a boundary layer wind tunnel. The interaction between in-situ measurements, model experiments and numerical modeling is illustrated in figure 8.

A typical monitoring system is explained in the following using the example of a wind energy plant which is in service for already 10 years, see figure 8 as well. An overview about the presently installed sensors is presented in figure 9. Here, the measurements of the structural response are limited to the steel shaft of the WEC. The data of six three-dimensional accelerometers can be used for the identification of the bending and torsion mode. The accelerometers are installed at five different heights at the inner surface of the steel shaft, see figure 10. More information is presented in [3]. At the top level, two accelerometers are mounted in a sufficient distance from each other in order to measure torsion oscillations. One three-dimensional accelerometer is installed at each of the neighboring measurement levels. Furthermore there are two additional planes with three displacement transducers and three temperature measurement points. The distribution of stresses in each plane can be directly evaluated from the measured data if linear material behavior of the steel shaft is assumed.

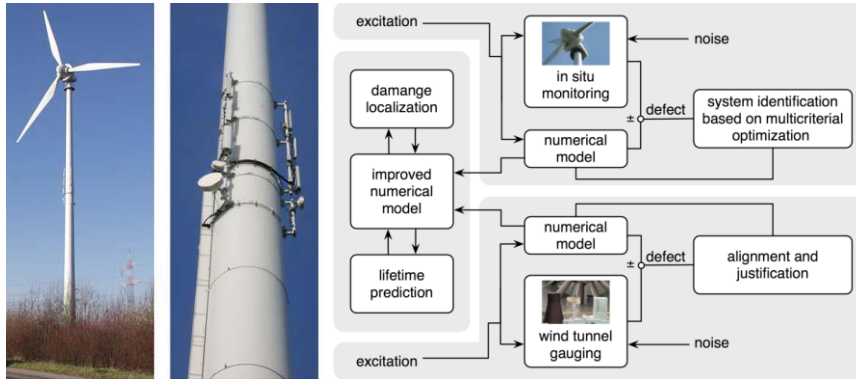


Figure 8 – Wind energy converter and general scheme of an advanced system identification

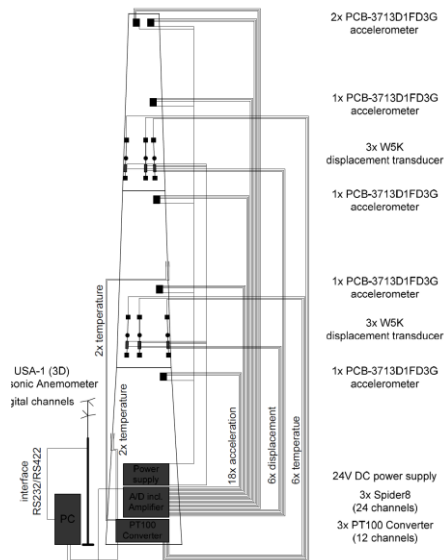
A three-dimensional ultrasonic anemometer for the measurement of turbulent wind (USA-1) is mounted on a telescopic mast and used for an additional, independent monitoring of wind speeds at the location of the WEC. On site, the data are captured by a data logger and a connected computer. The computer sends the measured data via DSL to a server system where software agents transfer the measurements into a relational database. The data in the database can easily be accessed using a web application which is equipped with the capability of exporting various data formats.

Reference measurements at a vibratory model in a boundary layer wind tunnel shall accompany the in-situ monitoring. A structural replica model of the WEC is required because it allows the physical simulation of the widely-complete response spectrum of the vibrating WEC. In particular it must reflect the elastic and dynamical characteristics of the WEC besides geometrical similarity which is realized in a model scale of approximately 1:100. The original stiffness and mass relation of the shaft is simulated by a hollow casting made from an epoxy matrix containing a mixture of plastic and metal-powder. In a boundary layer wind tunnel turbulent eddies in a suitable scale can be generated e.g. by thickening of the boundary layer according to Counihan's method. The response of the replica can be transferred to the original WEC for the purpose of calibration and in order to complete the in-situ measurements.

The realistic prediction of the remaining lifetime of the structure concerns in the first instance the damages accumulated during service. This includes most importantly the investigation of welded connections according to the notch stress concept. For parts of the structure which are mostly subjected to proportional loadings, the usual von-Mises equivalent stresses are utilized as effective stress values. For areas with non-proportional loading, a comparison to experiments should be carried out in order to determine which of the available critical-plane concepts is best suited.

Consequently, the fatigue- and deterioration process is modeled stochastically, nonlinear and time-variant. Based on a refined representation of the nonlinearities and the time variance, the load history can be used more efficiently for capturing damage evolution compared to current approaches. The classical S-N-concept of Wöhler (based on linear damage accumulation) does not satisfy the requirement. Based on a stochastic model of the loading process, the remaining lifetime of the structure is determined using modern subset simulation

techniques. The actual finite element model will be adapted either by modeling damages directly or by using special crack elements (XFEM).



- 2x PCB-3713D1FD3G accelerometer
- 1x PCB-3713D1FD3G accelerometer
- 3x W5K displacement transducer
- 1x PCB-3713D1FD3G accelerometer
- 1x PCB-3713D1FD3G accelerometer
- 3x W5K displacement transducer
- 1x PCB-3713D1FD3G accelerometer
- 24V DC power supply
- 3x Spider8 (24 channels)
- 3x PT100 Converter (12 channels)



Figure 9 – Sketch of the measurement setup Figure 10 – Installed sensors at the shaft wall

3.2 NUMERICAL SIMULATION OF BRIDGE GIRDER AEROELASTICITY

As an illustration, of one of the examples of numerical simulations around bridge decks, parts of the ongoing project are here shown. A box bridge deck section is considered as shown in the fig. 7. As the first step, simulations to obtain static coefficients have been done. Later simulations of the forced vibration set-up are carried out. The results of the numerical simulation should be compared with the results obtained from wind tunnel tests which have been carried out in the boundary layer wind tunnel of the Ruhr-University Bochum.

The numerical simulations have been carried out using open source program OpenFOAM, which is based on the Finite Volume Method. The flow has been modeled as two-dimensional. To model the turbulence, $k-\omega$ SST model has been used. In both, the numerical and experimental simulations, sectional model of the bridge deck is forced to move in three degrees of freedom in a sinusoidal motion with constant amplitude. For this purpose solver which applies mesh deformation algorithm has been created.

As in every numerical simulation modeling of boundary conditions plays an important roll. Since the numerical simulation needs to be verified with the wind tunnel tests, it is important to have the same input parameters. As inflow boundary conditions uniform velocity u_∞ and turbulence intensity need to match with the respective parameters from the wind tunnel experiments. Since top and bottom have to have no influence on the flow, for these boundaries slip boundary condition is treated.

It is clear that a smaller computation domain has less finite volumes elements and less numerical costs. Therefore, it would be preferable if small domain can be used. But there are

some limits restricting the minimum size of the computational domain. First of all in very small domain, flow circulations and big vortices around the object cannot be resolved. Furthermore, boundaries of a domain which is not large enough, affects the loading on the bridge deck. A qualitative analysis is needed to decide about the proper domain size. The analysis has been done by performing a series of simulations in different computation domain sizes. Usually three sides of the domain have been kept constant; while only one has been taking different positions. Analyzing the convergence in the static coefficients, the necessary domain is obtained as shown in table 2.

Table 2 – Experimental technical data

<i>Geometry of the domain</i>	<i>Windward</i>	$10H$
	<i>Leeward</i>	$15B$
	<i>top and bottom</i>	$15H$
<i>Parameters</i>	H – height of the bridge deck B – width of the bridge deck	

The finite volume grid has been created as a block structured mesh using the ANSYS tool ICEM. Near the deck surface, the grid was progressively refined. Near the edges the grid has been more strongly refined. This fine discretization extends through the boundary layer. The far field has been modeled with a coarse grid, because the velocity is assumed to be constant here.

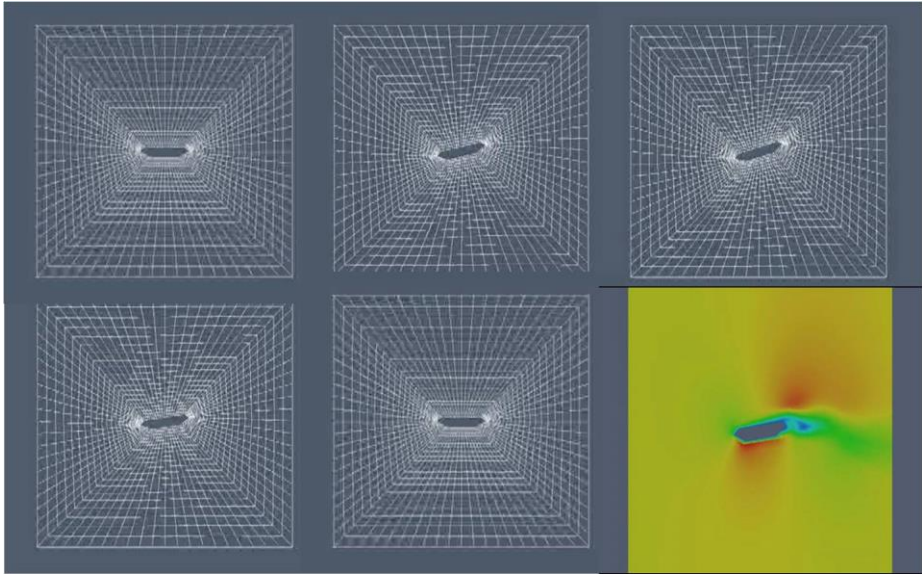


Figure 11 – Moving mesh sequences and velocity distribution of a coarse testing mesh applying forced rotation

To model the sinusoidal motion of the deck, the whole grid has to be moved separately in each of the three degrees of freedom (translational movements normal and in the direction to the stream and rotational movement causing periodic changes of the angle of attack). The

movement is controlled with solver created of the program system OpenFOAM. The amplitude of the motion and angle must match the respective ones used in wind tunnel experiments. For each simulation used time step must satisfy the Courant number (an indicator of accuracy and numerical dispersion) is less than unity. The used solver is per each time step performing the grid movement and the integration of the pressure distribution in order to obtain the forces which are acting on the moving bridge section, comp. figure 11. Complete information is given in literature [7].

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Predrag Popovic¹, Douglas Crampton²

ISPITIVANJE I SANACIJA PEŠAČKE NADSTREŠNICE NA O'HARE AERODROMU U ČIKAGU

Rezime:

U-2

U toku izvodjenja pešačke nadstrešnice na O'Hare aerodromu neki spojni šavovi nisu bili izvedeni po propisima i nekoliko čeličnih limova je instalirano sa sabstandardnim kvalitetom čelika. U ispitivanje koustrukcije su bili uključene vizuelna inspekcija, ispitivanje nerazarajućim metodama i statičke analize čeličnih elemenata. Efektivne metode sanacije su projektovane da uzmu u obzir potrebnu nosivost, otpornost na krti lom čelika i estetske zahteve projekta. Sanirani su stub da se izbegne krti lom i veza čeličnog stuba i sandučastog nosača upotrebom visoko vrednih zavrtnjeva nosivosti od 250 tona.

Ključne reči: šavovi, čelik, lom, sanacija

EVALUATION AND RETROFIT OF PEDESTRIAN CANOPY AT CHICAGO O'HARE INTERNATIONAL AIRPORT

Summary:

During the construction of pedestrian canopy at O'Hare airport it was discovered that certain welds were not properly constructed and that few steel plates had lower strength than specified. An investigation included visual inspection, non-destructive testing, structural and fracture critical analysis of steel members. Retrofits were designed and constructed to address strength, fracture, and aesthetic requirements of the project. Included are retrofit for a fracture critical column detail and retrofit for a deficient column-box girder junction using 250-ton high strength bolts.

Key words: welds, fracture, steel, retrofit

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1 DESCRIPTION OF STRUCTURE AND BACKGROUND

The pedestrian canopy at O'Hare International Airport was designed to expand the Terminals 1, 2, and 3 by about 16 ft. (4.80 m) and/or to provide a protection to pedestrians on the upper departure level. This project included the investigation and retrofit of the canopy for Terminals 2 and 3 (Figure 1). The total length of this canopy is about 2,000 ft. (610 m). The typical cross-section of the canopy structure consists of a 40 ft. (12.20 m) steel column which supports a 61.5 ft. (18.75 m) wide steel canopy. The canopy itself has two wings, a 41.3 ft. (12.6 m) wide roadside wing and a 20.2 ft. (6.15 m) wide building side wing, which is partially inside the terminal building (Figure 2). The architectural intent was to provide an impression of a flying wing supported by a series of single columns (Figure 3).



Figure 1. Pedestrian canopy at O'Hare International Airport

The structural support system of the canopy consists of 3.5 ft. (1.07 m) deep and 5.7 ft. (1.75 m) wide steel box girders spanning between the steel box columns spaced 50 ft. (12.25 m) or 70 ft. (21.25 m) along the length of the canopy. The canopy wings are supported by steel outriggers spaced at 10 ft. (3.05 m) which cantilever from the box girder. Because of uneven lengths of the wings, significant unbalanced forces had to be taken in the account for the box girder and columns.

Columns and 14 ft. (4.3 m) long sections of box girder above the columns were prefabricated as a "hammerhead" unit in the fabricating shop and were erected first. The box girder section between adjacent "hammerheads" were bolted in place in the field. Then the outriggers were connected to the sides of already erected box girders. Finally, steel skin plates with skylights were placed between the outriggers.

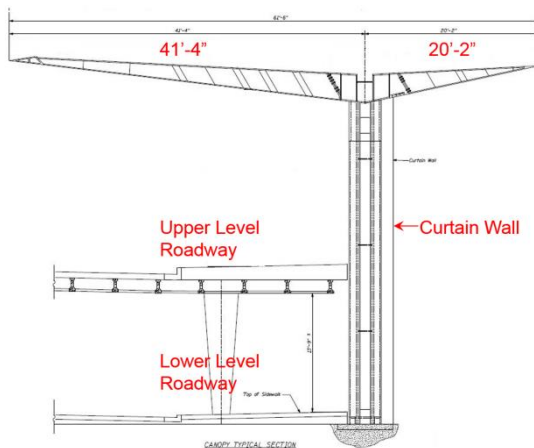


Figure 2. Pedestrian canopy geometry



Figure 3. View of the canopy from the ground level

Background

After most of the canopy was erected, in November 2005, a crack was discovered in one of 35 columns (Figure 4). Considering that the thickness of the cracked column steel plate was 2 in. (50 mm), there was a great concern about safety of the overall structure which spanned over an active roadway and walkways in use by the airport passengers. A number of weld and plate samples were taken and an investigation began about the possible causes of the column plate fracture. While this initial investigation was ongoing, Wiss, Janney, Elstner Associates, Inc. (WJE) was engaged by the general contractor Walsh Construction Company (Walsh) to design temporary shoring for the loads specified by the Engineer of Record for the canopy. The shoring for the canopy was designed by WJE in one day using the structural steel shapes that the steel fabricator had available in his shop. The erection of the shoring was completed in three weeks (Figure 5).

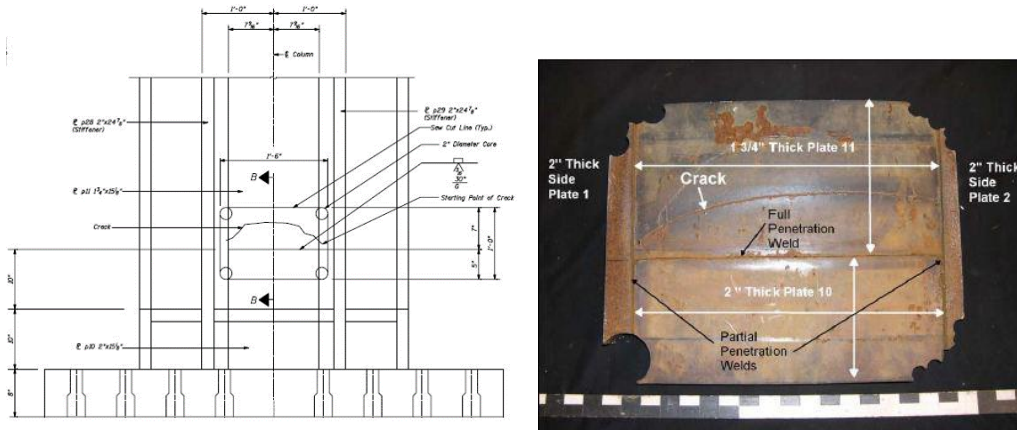


Figure 4. Crack discovered in column in November 2005



Figure 5. Canopy shoring

2 SCOPE OF THE INVESTIGATION

A team was assembled to oversee the assessment of the as-built condition of the canopy. The approach to address the as-built condition of the canopy was developed by WJE and presented to the members of the team. The Team included Chicago Department of Aviation (Owner), Architect and Engineer of Record for the original design, Walsh Construction Company (General Contractor), Construction Manager, Project Management Consultants and Structural and Material Consultants to the Owner and WJE (consultant to Walsh).

The so-called “WJE Approach” was presented to the Team and included the following tasks:

Task 1: Nondestructive Testing and Evaluation of Welds

The critical welds in the canopy structure were identified and evaluated using ultrasonic testing (UT) methods. The data gathered from this nondestructive testing were used to complement the information presented in the past testing reports. Based on the data gathered from the past reports and the UT testing, the weld defects/flaws were identified. The frequency of the weld testing was the greatest in the rotunda canopy. WJE also evaluated the effects of an identified potential delaminations in top flange plate of the box girder.

Task 2: Analyses

After the weld defects/flaws were defined, quantitative analyses were performed to evaluate the adequacy of the as-built welds. The load-carrying capacity of the weld was determined based on the as-built conditions determined from the nondestructive testing. Forces for the welded members were obtained from the design calculations submitted by the Engineer of Record (EOR). A detailed computer model of the canopy structure was used to determine the stresses acting on the structural members. Snow, wind, and thermal loads from design calculations were used in combination with rational engineering judgment to determine the most critical load cases.

Weld defects/flaws identified in Task 1 were evaluated using linear elastic/plastic principles of fracture mechanics in conjunction with stress analyses. Weld toughness tests were performed using weld material from existing cores. Core samples were also used for toughness testing of metal in heat affected zones and of base material. Typical testing also included Charpy V-notch tests.

Task 3: Design and Implement Repair for Deficient Welds

An engineered repair was developed by WJE for the welds that could not carry the intended loads or welds that could not perform satisfactorily under fracture critical scenarios. The proposed repairs were presented to the Owner and their Structural Consultants for review and approval. After the proposed repairs were approved, the implementation of the weld repairs were performed.

The proposed approach was accepted by all parties involved and was implemented by WJE.

3 EVALUATION AND RETROFIT OF CANOPY WELDS

Destructive and non-destructive testing methods were used to determine the as-built quality of the canopy welds and the test locations, sample size, and type of testing were selected by the Team. Non-destructive methods included Visual Testing (VT), Ultrasonic Testing (UT) and Magnetic Particle Testing (MT). UT was used to estimate depths of penetration of welds and to evaluate flaws in the welds. MT was used to evaluate and locate potential cracks or flaws at or near the surface of the welds, and VT was used to identify visually apparent defects. In some cases a boroscope was used to visually evaluate internal welds. This non-destructive testing was performed by other testing agencies. All testing and inspection was witnessed by other inspectors for quality assurance. In several cases special procedures were developed where access and/or weld type prevented testing in strict accordance with American Welding Society (AWS) procedures. An example is the testing of partial joint penetration (PJP) welds, for which AWS does not prescribe testing procedures or acceptance standards. Phased array was also

used in cases where conventional UT needed to be verified or supplemented. The evaluation of the canopy welds was based in large part by the non-destructive test results.

Destructive testing included removal of core and coupon samples from canopy welds. These samples were sectioned, polished, and etched. Weld geometry, fusion, inclusions, and cracks were documented and evaluated. These macro examinations were performed by other metallurgical testing agencies. Tensile and chemical tests were also performed on the base metal. Sample locations were repaired by approved weld repair procedures or, in cases where the long term performance of the structure was not impaired by sample removal, by filling openings with non-shrink grout.

In order to determine a lower bound fracture toughness for use in evaluation of the welds, standard Charpy V-Notch (CVN) specimens were machined from samples removed from the canopy base metal, weld metal, and heat affected zones (HAZ). These specimens were tested in accordance with the requirements of ASTM E23 over a temperature range of -40 degrees F to 70 degrees F. Two stage CVN- K_{Ic} - K_{Ic} and temperature shift correlations were used with the Master Curve approach to estimate the 5 percent lower bound fracture toughness levels for the canopy material. Lower bound fracture toughnesses of 70 $\text{ksi}\sqrt{\text{in}}$ for the base metal and 65 $\text{ksi}\sqrt{\text{in}}$ for the weld metal and HAZ were used in WJE's evaluation of the canopy (Figure 6). In addition, early on during the weld evaluation, the Engineer of Record indicated that the canopy was a statically loaded structure and was not subjected to fatigue loads.

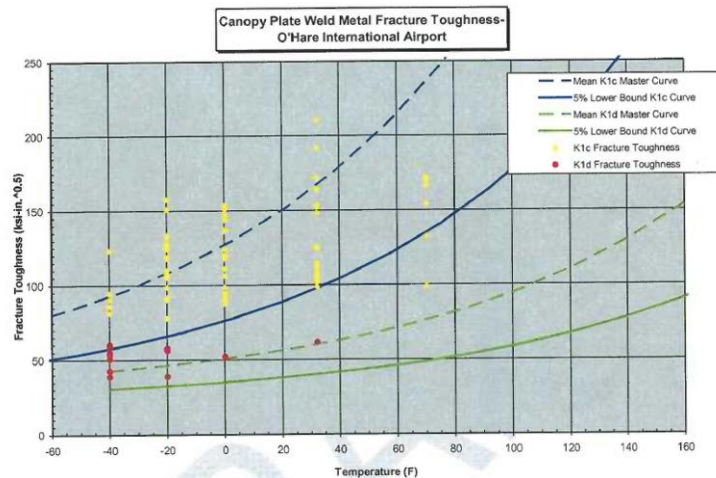


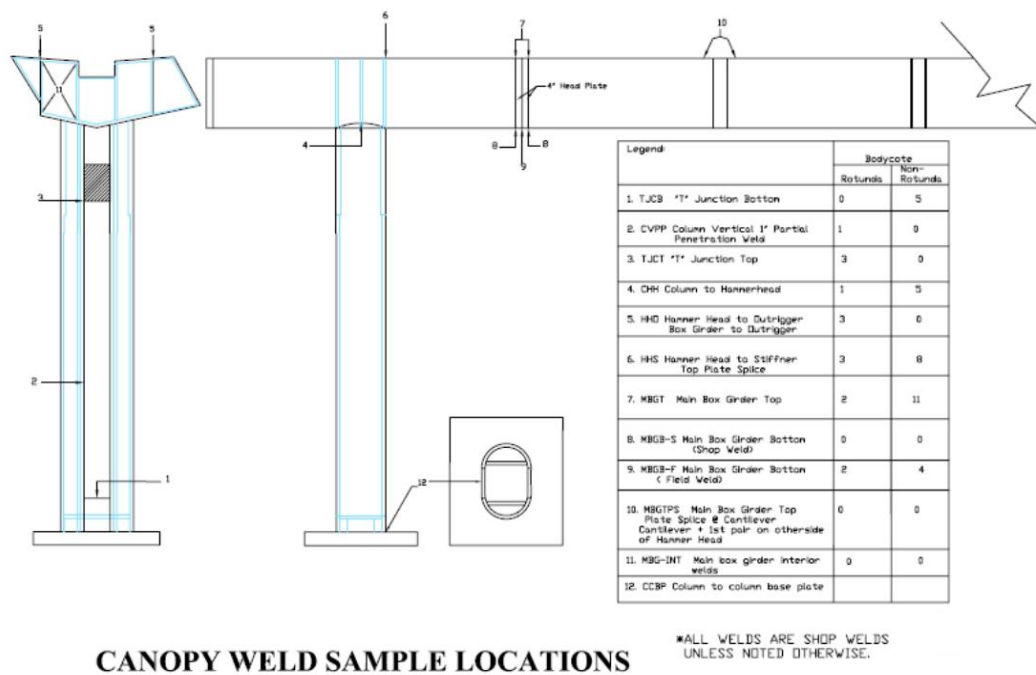
Figure 6. Material testing - fracture toughness

After determination of the as-built quality of the welds through the testing methods previously described, each weld type was evaluated for strength and susceptibility to fracture. Design forces provided by designer were used in the evaluations. Strength evaluations were based on the AISC Manual of Steel Construction, Load and Resistance Factor Design, Second Edition, the specification used in the design of the structure. Demand to capacity ratios less than 1.0, determined in these evaluations, indicated that welds met code-prescribed strength requirements. Ratios greater than 1.0 indicated that weld capacities were inadequate. Elastic-

plastic fracture mechanics analyses were based on a Failure Assessment Diagram (FAD) approach using the general methodology described in British Standard BS 7910:2005. In general, residual stresses were assumed to be uniform and equal to 0.67 Fy for complete joint penetration (CJP) welds and 0.5 Fy for PJP welds. In some cases an approximate residual stress distribution was used in the analysis. For a given flaw, points lying above the assessment line on the diagram indicated an unacceptable flaw, while points below the line indicated an acceptable flaw.

The Team identified the location of critical welds in the canopy to be evaluated, as shown in Figure 7. The welds were considered critical if the failure of any one of the welds can result in partial or complete failure of the main framing of the canopy.

Although all critical welds were evaluated during this project, only three critical welds are presented here because of the space limitations.



CANOPY WELD SAMPLE LOCATIONS

Figure 7. Weld types and locations

3.1 WELD 1

Weld 1 is a shop weld located at the lower splice of the column web plates, as shown on the diagram in Figures 4 and 7. Each column contains two of these splices, for a total of seventy Weld 1 locations. A partial elevation of the column, reproduced from the shop drawings, is shown in Figure 8. Weld 1 joins the lower, 2 in. (51 mm) thick column web plate (designated

p10) to the upper, 1 3/4 in. (44 mm) thick column web plate (designated p11). This weld is called out as a complete joint penetration weld on the shop drawings.

Extensive non-destructive and destructive testing was carried out at Weld 1. Photographs of the polished and etched cross sections show cracks and crack-like defects at the root of the welds and incomplete penetration (Figure 9). Ultrasonic testing was performed at all Weld 1 locations in the straight canopy sections by three separate testing agencies. According to the UT results, none of the welds exhibited complete penetration across the full width of the joint. Minimum depths of penetration ranged from 1.08 in. (27 mm) to 1.70 in. (43 mm).

The geometry, defects, and residual stress fields at the intersection of Weld 1 and Weld 2 are difficult to model accurately, even using complex finite element techniques. Using non-destructive testing methods, it is very difficult to accurately characterize the size and orientation of existing defects at this location. In addition, the highest applied tensile stresses (caused by column flexure) in Weld 1 occur at the end of the weld, where it intersects a vertical column weld (Weld 2). Residual stresses approaching the yield strength of the plate material are also present at this location. Due this weld intersection positioned at the end of Weld 1, where each welding pass was likely started or terminated, the quality of the weld is likely at its poorest at this intersection.

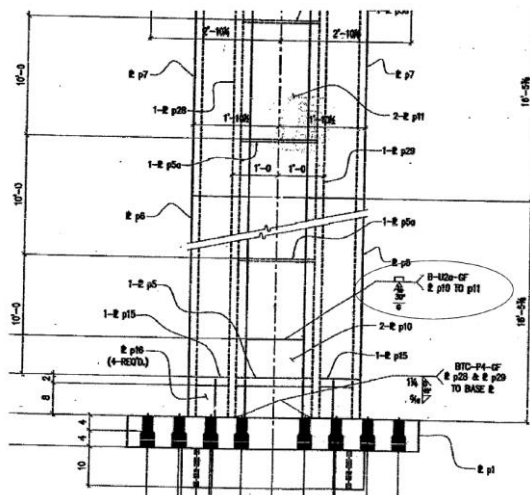


Figure 8. Partial elevation of column

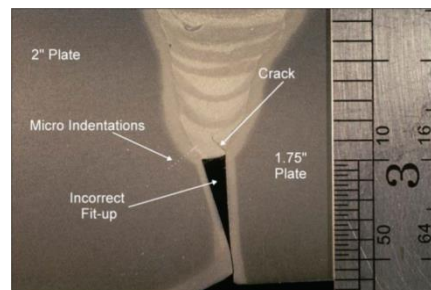


Figure 9. Weld 1 (CJP) defects

WJE's review of the as-built conditions at Weld 1 indicated that there was a potential for fracture at several column locations. As a result of our evaluation, WJE recommended retrofitting Weld 1 at all thirty-one columns in the straight sections of Terminals 2 and 3. It was determined that the most effective solution for addressing a potential fracture initiating at Weld 1 was installation of a drilled crack arrest hole at the intersection of Weld 1 and Weld 2, at each end of Weld 1. The 3 in. (76 mm) diameter retrofit holes and installation procedure are detailed in Figure 10. These retrofit holes completely remove the potential fracture initiation point. In addition, should a fracture occur in the remaining portion of Weld 1, the fracture would be attracted to the hole and arrested, due to the direction of the principal stresses. This is demonstrated by the finite element analysis of the retrofit that was performed by WJE. An example of a completed retrofit is shown in Figure 11.

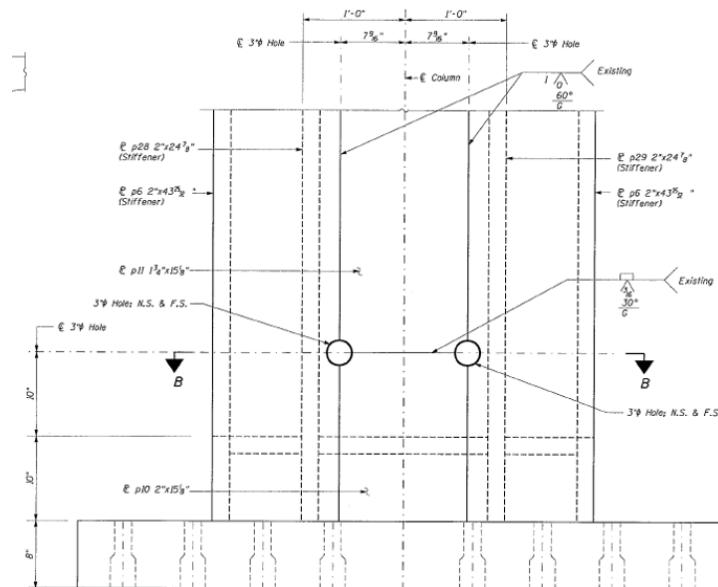


Figure 10. Weld 1 - drilled hole retrofit

Using design forces provided by the design engineer, WJE evaluated the strength of the as-built Weld 1 at each column, considering the reduced cross section due to installation of the crack arrest holes. The as-built welds were determined to possess adequate strength to resist design forces. In addition, WJE evaluated the unlikely scenario of both webs in a given column fracturing at the Weld 1 locations. Note that very low temperatures, large applied forces, significant defects at both Weld 1 locations, and propagation of fractures across the full width of both web plates would all have to occur simultaneously for this scenario to develop. Our analysis demonstrated that the column cross section would maintain adequate capacity to resist design forces in the unlikely event that these fractures occur.



Figure 11. Weld 1 - drilled hole retrofit

3.2 WELD 7

Weld 7 is a 1-1/2 in. (38 mm) partial joint penetration shop weld that joins the 1-5/8 in. (41 mm) thick hammerhead and box girder top flange plates to a 4-inch thick splice head plates. The Weld 7 location is shown schematically in Figure 7. A plan view of the box girder showing the weld location and type, reproduced from shop drawings, is shown in Figure 12. Each Weld 7 is approximately 23 in. (585 mm) long and in total there are 276 Weld 7 locations.

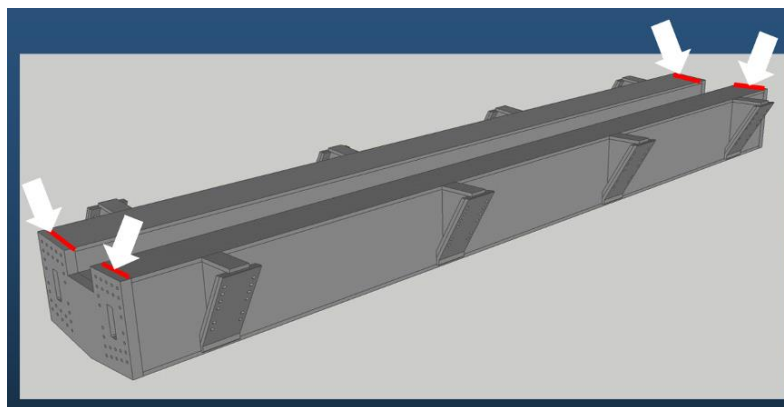


Figure 12. Weld locations

Eleven cores were removed from Weld 7 and were examined. In addition, ultrasonic testing was performed along the full length (approximately 23 in., 585 mm) of the welds at all of the Weld 7 locations. Ultrasonic test results and core samples identified weld deficiencies at several Weld 7 locations. Deficiencies included incomplete fusion, slag inclusions, cracks, and root depths were shallower than specified on the shop drawings (Figure 13).

A strength analysis was performed for the as-built conditions Weld 7. This analysis considered the defects detected by ultrasonic testing. The effective area of Weld 7 was determined by neglecting the area of defects identified in the weld. Box girder section properties were then calculated at a cross-section through Weld 7 and the stresses were determined using the design forces. The strength analysis revealed that Weld 7, as-built, possess adequate strength to resist factored design forces provided.

WJE performed a fracture mechanics evaluation of Weld 7 based on as-built conditions identified from the ultrasonic testing and core sample results. Evaluation was done for both cantilever and non-cantilever locations and was performed to determine the acceptability of the flaws detected in these welds. This evaluation was based on a Failure Assessment Diagram approach, using the general methodology described in British Standard BS 7910:2005. Forces provided by design engineer were used to determine the stresses in the welds due to the applied loads. The fracture mechanics evaluation identified weld defects that were acceptable and defects that require remediation. In total, thirty-one Weld 7 locations required remediation.

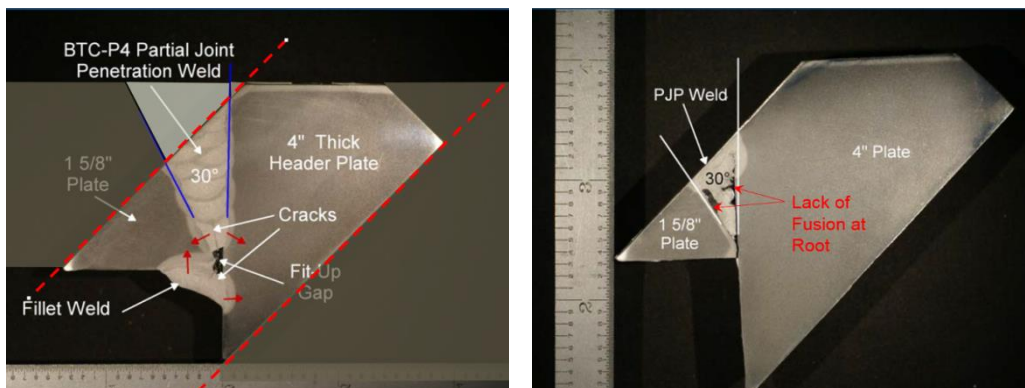


Figure 13. Weld 7 defects.

The retrofit for the various Weld 7 locations requiring remediation consisted of a top flange cover plate to bypass Weld 7. The cover plate was attached to the box girder and hammerhead top flange using fillet welds. Due to the design of the retrofit plates, which each cover two Weld 7 locations, and the desire to provide symmetric repairs at the cantilever locations, thirty-eight retrofit plates were installed. These retrofit plates cover a total of seventy-six Weld 7 locations. An example of a Weld 7 retrofit plate is shown in Figure 14.



Figure 14. Weld 7 - Retrofit plate

3.3 WELD 6.2

Weld 6.2 is a shop weld that connects the interior stiffeners of the hammerhead box girder to the hammerhead box girder bottom flange. Weld 6.2 works in conjunction with the hammerhead box girder bottom flange to column weld (Weld 4) to transfer forces from the hammerhead box girder to the top of the column (Figure 15). Weld 6.2 is shown on the shop drawings as a partial joint penetration weld executed from both sides of the 1-1/2 inch (38 mm) thick stiffener plate. The designated depth of the stiffener plate bevel is shown as 11/16 in. (17 mm) on each side of the plate. A typical plan view of the column hammerhead, from the shop drawings, showing the location of the stiffeners is shown in Figure 16. Due to the layout of the internal stiffeners, the center stiffeners on the road side and building side have the highest calculated stress. A section through the main box girder showing Weld 6.2 between the center stiffener and the box girder bottom flange is shown in Figure 17.

An analysis of Weld 6.2 was performed using forces at the top of the columns provided by design engineer. This analysis revealed relatively high demand to capacity interaction ratios for this weld. As a result, the welds were initially inspected using a borescope through openings made during removal of steel coupons. The borescope inspection of twenty welds revealed that the welds were present and appeared to be continuous and without any apparent surface flaws.

Because of the inaccessibility of these welds for ultrasonic testing and the calculated high interaction ratios, it was decided that five, three-inch diameter, cores be taken for laboratory evaluation. As a first step, three of the five cores were taken and evaluated. This evaluation revealed significant defects with Weld 6.2 including cracking, lack of fusion, slag inclusions, and severe underfill (Figure 18). Based on the defects and weld quality observed in these three cores, further sampling was discontinued and the Team decided that a retrofit was required at all columns due to the small sample length of the destructive cores, the variability of the observed weld quality, and the accessibility of this weld for additional testing.

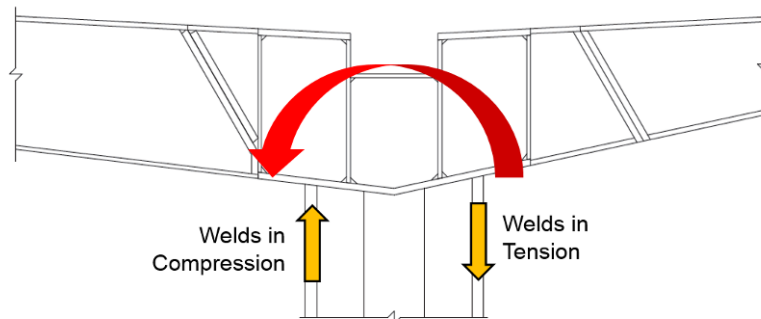
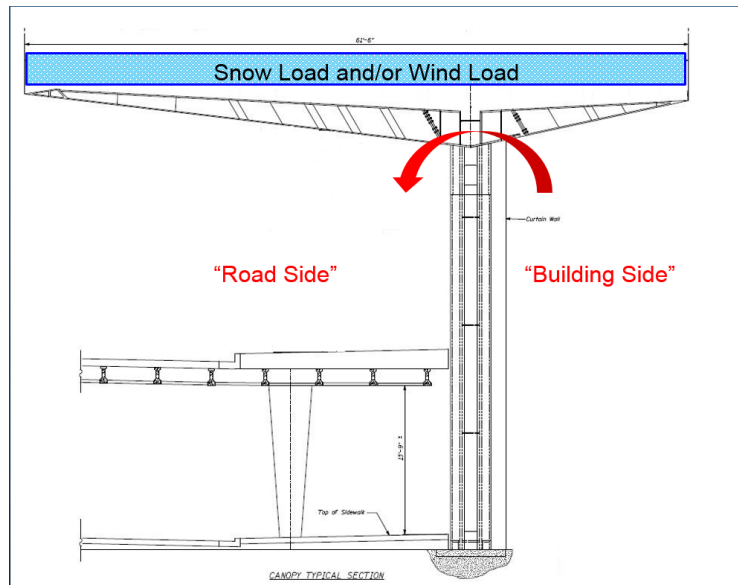


Figure 15. Column to box girder connection

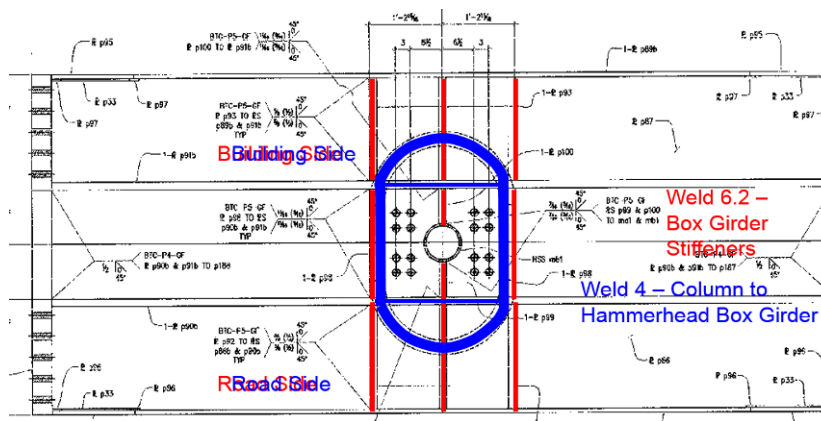


Figure 16. Column to box girder connection

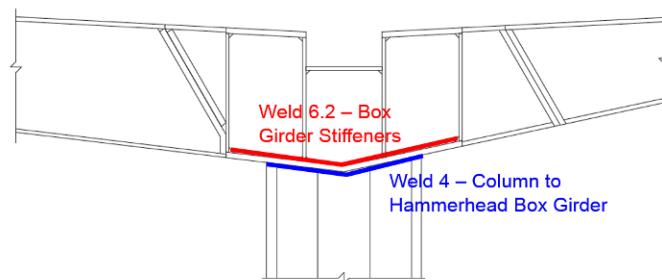


Figure 17. Column to box girder connection

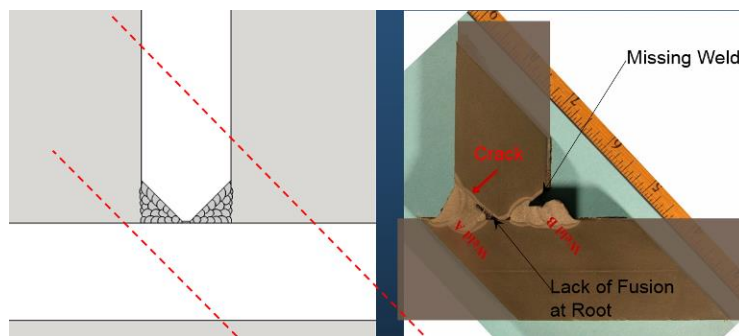


Figure 18. Weld 6.2 defects

WJE developed a retrofit to provide an alternate load path to transfer the tensile forces between the column and the hammerhead box girder. This retrofit utilized high-strength all-thread rods through the box girder to “clamp” the column to box girder connection and bypass the deficient welds (Figure 19). The 2.5 in. (64 mm) diameter rods have an ultimate capacity of over 350 tons each and were pretensioned to approximately 100 tons. A bracket was welded to the top of the column to provide an attachment for the all-thread rods to the column.

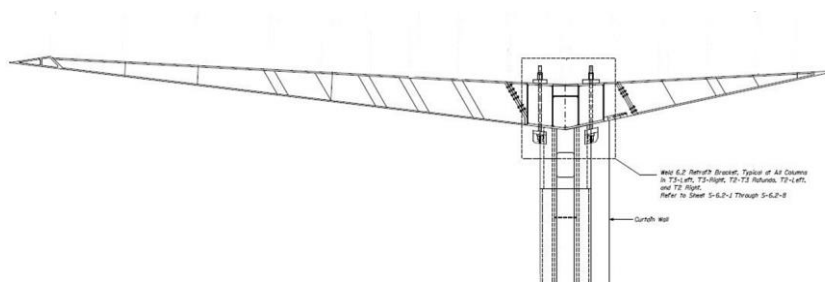


Figure 19. Column to box girder - Weld 6.2 connection retrofit

The final design of the Weld 6.2 retrofit is shown in Figure 20. An example of a Weld 6.2 retrofit bracket is shown in Figures 21 (bracket at top of column) and 22 (top of hammerhead box girder).

Strain gaging was also performed during tensioning of a Weld 6.2 retrofit bracket (Figure 23). Strains in the column and jacking bracket stiffeners were monitored during all-thread rod tensioning at one column. The maximum measured stress was approximately 20 ksi (138 MPa) in the bracket stiffener plate. Overall the maximum measured strain during the tensioning indicates that the bracket stiffener plate remained elastic and the corresponding stresses are within a range of what would be expected from a theoretical analysis. In addition, the measured plate and column stresses meet design code requirements. Upon completion of the repair, for aesthetic reasons the retrofit brackets were covered with metal plate shrouds (Figure 24).

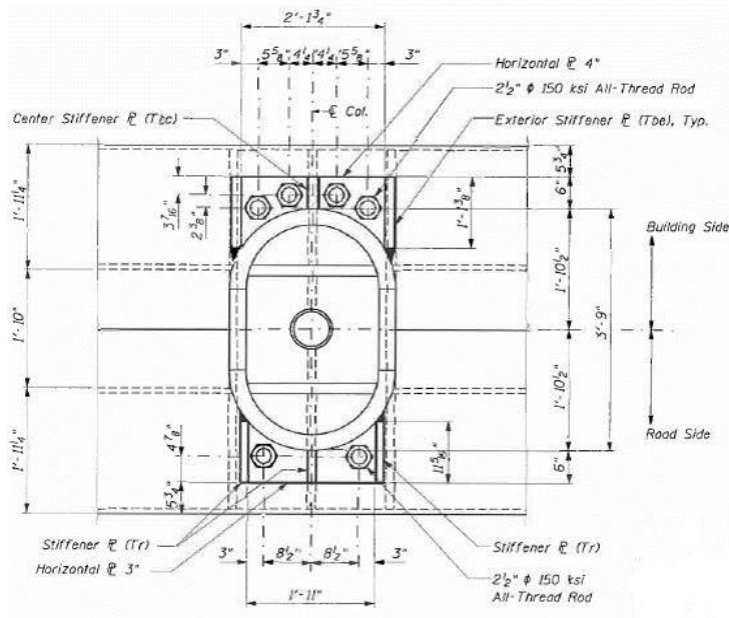
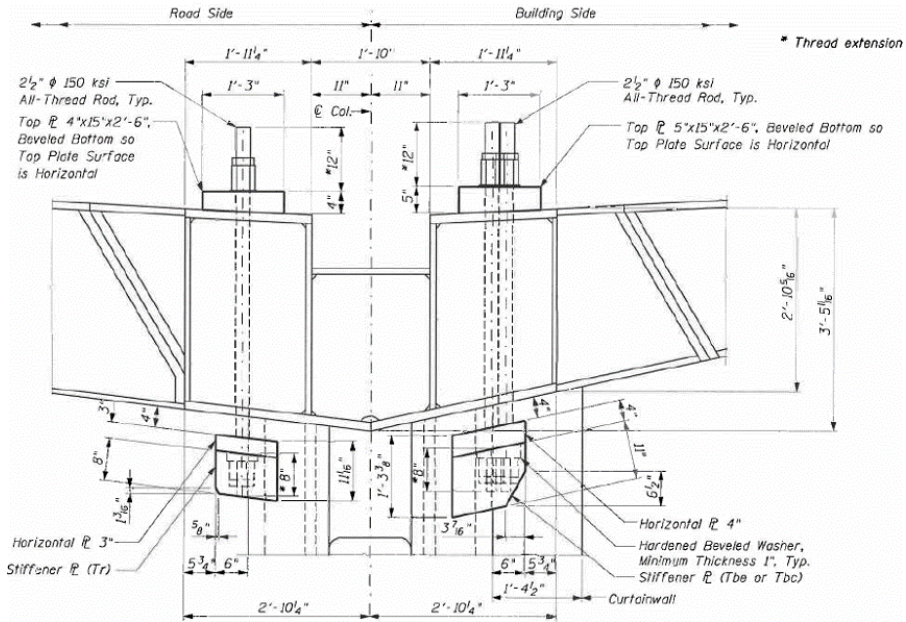


Figure 20. Column to box girder - Weld 6.2 connection retrofit



Figure 21. Weld 6.2 retrofit bracket

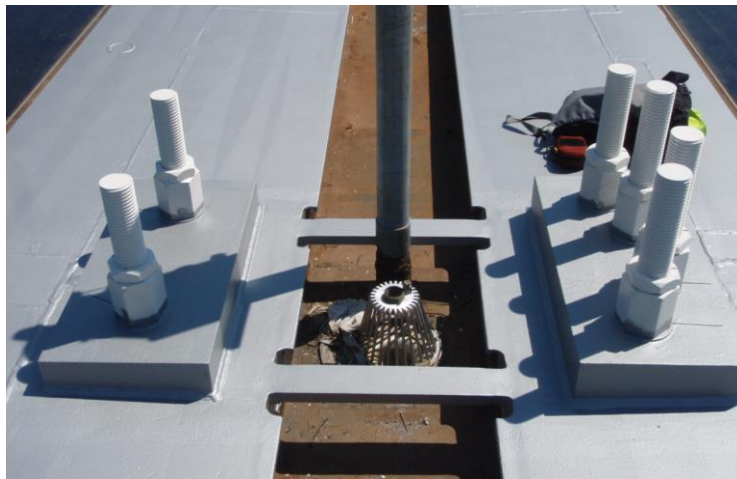


Figure 22. Weld 6.2 retrofit - top of canopy



Figure 23. Verification strain gages



Figure 24. Retrofit shroud

4 EVALUATION AND RETROFIT OF PLATES WITH YIELD STRENGTH LOWER THAN SPECIFIED

4.1 BACKGROUND

Early on in the canopy testing program, coupon samples were removed from various steel plate elements. The initial sample, Coupon 0, was removed from a box girder top flange plate to investigate what appeared to be plate delaminations noted during ultrasonic testing to verify plate thicknesses. Subsequent chemical and tensile testing on this coupon revealed a yield strength of approximately 42.5 ksi (293 MPa). This yield strength was less than the 50 ksi (345 MPa) minimum yield strength specified for ASTM A572 Grade 50 steel, which was specified for the project.

In total, thirty-five coupons were removed from various steel plate elements. The Testing Agency performed chemical and tensile testing on these samples to determine if the plates conform to ASTM A572, Grade 50. A total of three coupons were found to have yield strengths less than 50 ksi (345 MPa.). The lowest tested yield strength was approximately 32.5 ksi (224 MPa) and was found in a box girder top flange plate.

4.2 DEVELOPMENT OF THE HARDNESS/CHEMICAL/TENSILE TESTING PROGRAM

After the presence of understrength plates in the canopy was confirmed, WJE proposed hardness testing to further investigate the potential for understrength steel plates based on the relationship between hardness and the ultimate tensile strength of steel from ASTM A370. Hardness testing was a method to perform a large-scale investigation of plate elements without the destructive nature of removing and replacing coupons. While hardness testing alone was not proposed to definitively determine the yield strength of the tested steel, hardness testing was proposed as a non-destructive method to locate potentially deficient material in the Canopy structure. As a first step, WJE recommended performing hardness tests on the steel coupons already removed from the Canopy in order to establish a project-specific correlation between hardness values and tensile test results.

Another metallurgical consultant performed hardness testing on the steel coupons already removed from the canopy, where the tensile strength was known, and correlation between hardness and tensile strength was developed. It was recommended that metal filings be taken for chemical analysis if the Rockwell hardness value is less than 76 HRB. In addition, coupons for tensile testing were recommended if the plate hardness level was measured to be less than 72 HRB. After the hardness/chemical/tensile testing program had been implemented for a few months, the metallurgical consultant reviewed the test results and confirmed the validity of the original recommendations. Therefore the final recommendations remained in place throughout the testing program.

4.3 TESTING PROGRAM OVERVIEW

The hardness testing program was performed on a sampling of plate elements throughout the Terminal 2 and 3 canopy structures to identify and address plates with a yield strength less than 50 ksi (345 MPa). In general, plate elements with a hardness value equal to or less than 76 HRB were subjected to additional tensile strength testing and/or chemical analyses and a

coupon for tensile strength testing was removed from plates with a hardness value less than 72 HRB.

Based on the results of a chemical test, the approximate yield strength for that plate was calculated. The empirical correlation between yield strength and chemical composition was developed by the metallurgical consultant. The equation accounts for the intrinsic strength of steel and the strengthening alloys while also considers the weakening effects of other elements. The yield strength correlation equation was based on the results of several tensile/chemical tests performed at the onset of the testing program.

During the testing program, a review of the chemical test results revealed that the chemistry plates with a yield strength less than 36 ksi (248 MPa) was notably different than the chemistry of plates with a tested yield strength greater than 36 ksi (248 MPa). For example, plates with tested yield strengths of approximately 36 ksi (248 MPa) or less generally a) had carbon contents less than or equal to 0.10% and manganese contents less than or equal to 0.50% or b) had carbon contents less than 0.02%. In addition, plates with a tested or predicted yield strength greater than 50 ksi (345 MPa) typically had carbon contents greater than or equal to 0.10% and manganese contents greater than 1.00%.

Tensile strength tests on plate coupons were performed to determine the actual yield strength of a plate. Typically, coupons for tensile testing were performed on plate elements with a hardness value less than 72 HRB. For plates with a tensile yield strength of 50 ksi (245 MPa) or greater, no further structural evaluation was required. Plates identified with a tensile coupon yield strength less than 36 ksi (248 MPa) were retrofitted. An example of box girder flange retrofit is shown in Figure 25. If a tensile test on a plate coupon revealed a yield strength between 36 (248 MPa) and 48 ksi (330 MPa), the actual yield strength was used for the structural evaluation of that plate.

Hardness testing was performed on about 5,000 plates and if confirmed adequacy of 95% of the plates. 25 plates had yield strength between 36 ksi (248 MPa) and 50 ksi (345 MPa) and 37 plates had yield strength below 36 ksi (248 MPa).

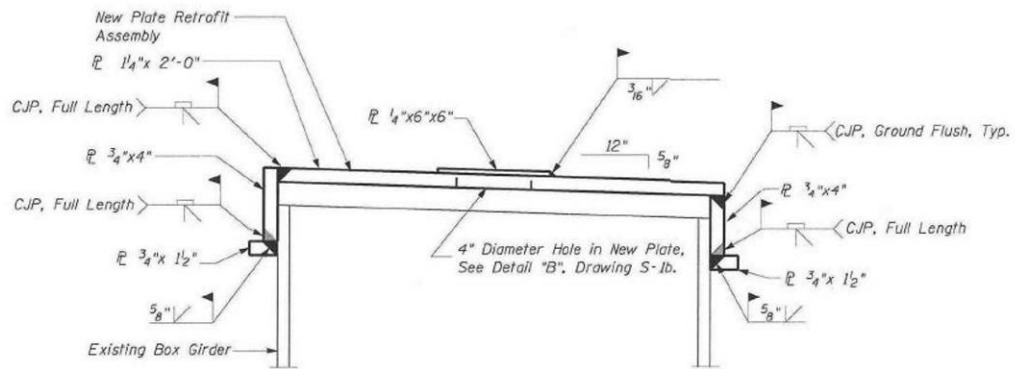
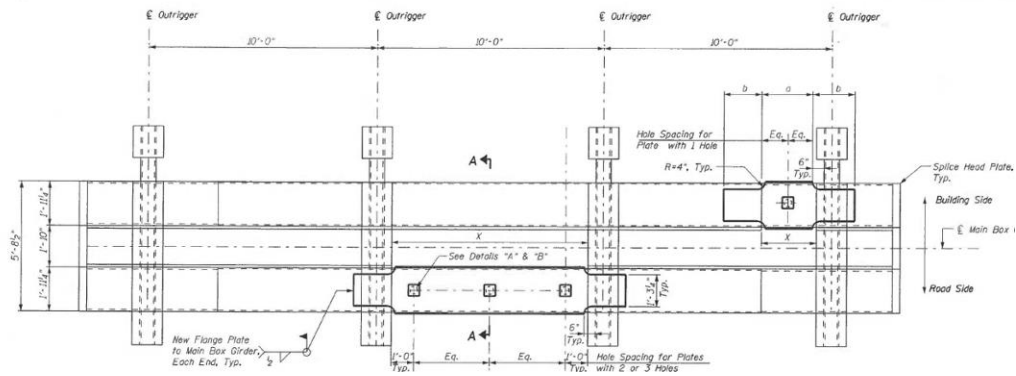


Figure 25. Box girder flange retrofits

5 FIVE YEAR WARRANTY INSPECTIONS

As part of the collaborative effort, the Owner has requested that Walsh provide periodic inspections of the Terminals 2 and 3 Canopy structure at O'Hare International Airport over the course of the five year warranty period. The primary objective of the periodic inspections is to verify that the remediated structure is performing as expected. WJE has proposed an inspection program to be implemented as part of Walsh's five year warranty, commencing upon final acceptance of the structure. The inspection program is to be performed on an annual basis over the course of the five year warranty period.

Items in the proposed inspection program included:

- Level survey of roadside outrigger tip elevations.
- Overall visual inspection of various components of the structural steel canopy.
- Arms-length visual inspection at selected retrofits and welds.
- In depth evaluation if any evidence of cracking is detected during the arm's length visual inspection.
- Report and Recommendations.

At this time, four annual warranty inspections were already performed and no major distress was observed in structural members.

6 SUMMARY AND CONCLUSIONS

WJE had completed an evaluation of certain welds and plates in the canopy structure. All the elements of the structure that were evaluated and did not have capacity to carry the design loads required by the City of Chicago Building Code were retrofitted by Walsh. WJE was responsible for the design of all structural retrofits to the canopy structure. All portions of the structural elements of the canopy that were evaluated by WJE now have capacity to safely carry the design loads required by the City of Chicago Building Code and the design loads provided by the designer of the structure.

This project was performed over a period of four years with over 20 WJE employees participating in various phases of the project. Authors wish to thank their colleagues for their contribution to the success of this project.

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OPTIMIZACIJA TOPOLOGIJE U GRAĐEVINARSTVU

Rezime:

U-3

U radu je prikazan način optimizacije topologije, tj. raspodele materijala i mogućnosti njene primene u građevinarstvu kod armirano betonskih ili hibridnih konstrukcija. Primena je pokazana na primeru industrijskog objekta kod optimizacije plana armature u zoni diskontinuiteta i kod rešetkastih krovnih nosača. Optimizacija topologije je predstavljena kao korisna dizajn alatka za podršku građevinskim inženjerima u različitim fazama procesa projektovanja.

Ključne reči: optimizacija topologije, plan armature, rešetke, robustni dizajn

TOPOLOGY OPTIMIZATION IN BUILDING CONSTRUCTION

Summary:

The paper presents the method of topology optimization and its application possibilities to reinforced concrete or hybrid structures in building construction. The practicability is demonstrated at the example of an industrial building by designing reinforcement layouts in regions of discontinuity and trusses for roof girders. Topology optimization is presented as a valuable design tool to support the structural engineer at various stages of the design process.

Key words: topology optimization, reinforcement layout, trusses, robust design

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1 INTRODUCTION

Virtually all engineering disciplines require the designing engineer to employ new scientific findings and state of the art methods to facilitate a practical, economically and technically reasonable solution for a given problem. The designer will try to ensure an optimal concept with respect to predefined constraints. To achieve such a solution an iterative process as well as a combination of intuition and experience is needed. Engineers can resort to a variety of different assisting tools to obtain such a solution. General methods of optimization – for technical applications especially methods of structural optimization – pose one of the most substantial resources [1; 2]. Structural optimization can be utilized to optimize the inner as well as the outer characteristics of construction components. Hence, the area of structural optimization can be subdivided into three major categories (see Fig.1). Topology optimization is employed to define the topology, the material distribution within a previously defined design space. By using shape optimization the boundaries of the design space can be modified to improve the element's mechanical performance and, finally, sizing is applied to define the section properties, e.g. thickness, to meet certain criteria like stress limits or deformation thresholds.

Each of these subsections can be employed separately to improve a structural element's performance. The most effective strategy, however, is to apply all of these methods consecutively. Due to the extensiveness of the area of structural optimization the authors want to restrict the scope of this publication to the method of topology optimization and in particular its applications to building construction.

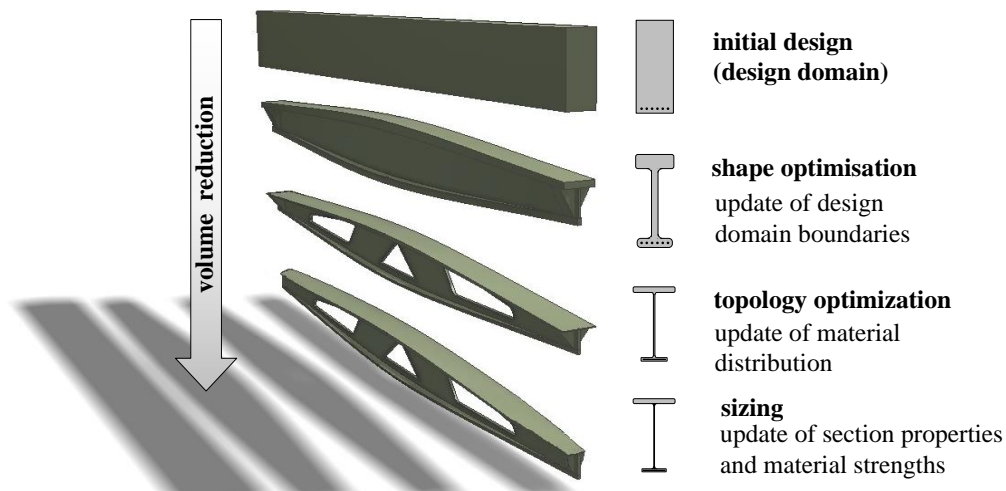


Figure 1: Subcategories of structural optimization

The advance of topology optimization with focus on theoretical settings and effective methods over the last years paved the way for practical application. It is emerging as a common digital design tool in various fields of engineering, such as the automotive or aerospace industry [1]. Unfortunately, the method is widely unknown in the field of structural engineering despite its great potential as a design aid, which can be versatily employed at

various stages of the design process. To fully use the advantages of this method it should be applied early on in the design process, preferably during the stages of conceptualization and preliminary design. Thus, structural systems can be identified with regards to a minimization of material cost and an improved mechanical performance. The efficient use of building materials does not only have the obvious advantages of using less material and the coherent minimization of costs, but can also positively affect the times for building execution and improve the overall sustainability of the project. Furthermore, current research substantiates a wide range of application possibilities in structural engineering such as the optimization of segmental tunnel lining [3; 4] or parabolic trough collectors [5].

In the article basics and typical applications to RC structures are presented. First, the authors will give a short introduction to the basic theoretical principles of topology optimization in section 2. Section 3 will be devoted to the practical application in building construction by using a fictional case study. The extension of the theoretical principles to multiple load cases facilitates a robust design, which will be the focus of section 4. Finally, the current results are discussed and an outlook on future works will be given in section 5.

2 BASICS OF TOPOLOGY OPTIMIZATION

Topology optimization is a mathematical approach based on structural calculations like the Finite Element Method (FEM) to distribute material within a given design domain Ω improving the overall performance of a structure. The main characteristic of these systems is then its load path dependent topology. Because the material can be distributed within the design domain freely, the method is employed early in the design process to fully activate its potential.

The first reference in the context of topology optimization is attributed to *Michel*, who investigated optimality criteria for the least-weight layout of truss structures in terms of ideal load transfer. His research yielded the so called *Michel*-structures, which are well known in the fields of optimization [6]. Following this initial findings, several researchers concentrated their efforts on analytical optimization of truss structures [7] or based upon discrete ground-structures [8; 9].

With the introduction of the homogenization approach by *Bendsoe* and *Kikuchi* in 1988 [10] an extension to continuous structures was achieved. This transition, as well as the continuously growing computational capacities, provided the foundation for a new popularity. Since, a wide variety of developments and extensions of the method were researched, which noticeably expanded the field. Hence, giving a copious overview would go beyond the scope of this publication. The reader is referred to the publications of *Bendsoe / Sigmund* [11], *Sigmund / Maute* [12] and *Rozvany* [13; 14] for further inquiry. The most popular FEM-based topology optimization method [13] is the SIMP algorithm. SIMP abbreviates “Solid Isotropic Material with Penalization”, which was proposed by *Bendsoe* [15]. One of the basic assumptions of this approach are the linear-elastic, isotropic material properties within the design domain Ω . To visualize the internal load path as good as possible, a minimization of the compliance c is chosen for the objective function. The minimization of the compliance corresponds to a maximization of the overall stiffness. In accordance with the principle of virtual work and energy the compliance reads [2]:

$$\delta W_{int} = \int_{\Omega} \boldsymbol{\sigma}^T \delta \boldsymbol{\varepsilon} d\Omega \quad \delta W_{ext} = \int_{\Gamma} \mathbf{t}^T \delta \mathbf{u} dA + \int_{\Omega} \mathbf{f}^T \delta \mathbf{u} d\Omega \quad (\delta \mathbf{u} = 0 \text{ on } \Gamma_u)$$

$$c = \int_{\Omega} \boldsymbol{\sigma}^T \delta \boldsymbol{\varepsilon} d\Omega = \sum_{i=1}^n \mathbf{u}_i^T \mathbf{K}_i \mathbf{u}_i = \mathbf{u}^T \mathbf{K} \mathbf{u}$$

To avoid contradicting aims, the guiding constraint of the optimization usually is chosen to the limitation of available material relative to its initial amount. This fraction of the initial design domain's volume is a priori determined by the user and can be assumed to approximately $\omega = 0.1 - 0.5$ for good results. Other values might be better suited depending on the design problem. The design variables of the optimization are the artificial densities ρ_i of each finite element. Thus, the problem has as many variables as elements defined within the design domain. Compared to other mathematical optimization problems this leads to a distinct increase in design variables and therefore to an increase of the optimization efforts.

With the start of the algorithm the material will be redistributed within the design domain at each iteration step in accordance with the constraints (available material, bounds of design variables) while minimizing the value of the objective function (minimum compliance). During this iterative procedure the material will be redistributed to preferentially generate a 0-1 design, which leads to elements with a remaining density value of 1 or 0. Intermediary values of graded densities are usually avoided as they lack physical interpretability. Elements with $\rho_i = 1$ are considered solid and therefore contain material, $\rho_i = 0$ equals void elements without material. Hence, a truss-like or arch-like structure arises, which illustrates the inner load paths of the design domain and therefore indicates ideal material distributions. However, results depend on formulations of the problems, constraints and posed design aims as well as accepted scatter of variables.

A typical formulation with linear material conditions yields [15]:

$$\min_{\boldsymbol{\rho}} : c(\boldsymbol{\rho}) = \mathbf{u}^T \mathbf{K} \mathbf{u} = \sum_{i=1}^n E_i(\rho_i) \mathbf{u}_i^T \mathbf{K}_{i,0} \left. \vphantom{\sum} \right\} \text{objective function}$$

$$s.t.: \left. \begin{array}{l} V(\boldsymbol{\rho}) / V_0 = \omega \\ \mathbf{K}(\boldsymbol{\rho}) \mathbf{u} = \mathbf{f} \\ 0 < \rho_i \leq 1 \end{array} \right\} \text{constraints}$$

where:

c	compliance	ρ_i	design variable („density“)
\mathbf{u}	global displacement vector	\mathbf{u}_i	local displacement vector
\mathbf{K}	global stiffness matrix	E_i	element Young's Modulus
$\mathbf{K}_{i,0}$	element stiffness matrix for $E_i=1$	n	number of Elements
$V(\boldsymbol{\rho}_i)$	volume at iteration step	V_0	initial volume
ω	prescribed volume fraction	\mathbf{f}	global force vector

The Young's modulus E_i of each element i is dependent on the element's density ρ_i :

$$E_i(\rho_i) = E_{\min} + \rho_i^\eta (E_0 - E_{\min})$$

E_0 describes the material's Young's modulus, E_{\min} a fictitious, small number (e.g. 10^{-6}) to avoid singularity of the global stiffness matrix for elements with $\rho_i \rightarrow 0$. η denotes a penalization factor to improve convergence of the element densities towards a 0-1 design and penalizes intermediate values. Good results are obtained with $\eta = 3$ [15]. With each iteration step the material is redistributed until a convergence criteria is met, e.g. a negligible change in design variables of each element $< 1\%$ between two iteration steps (Fig.2)

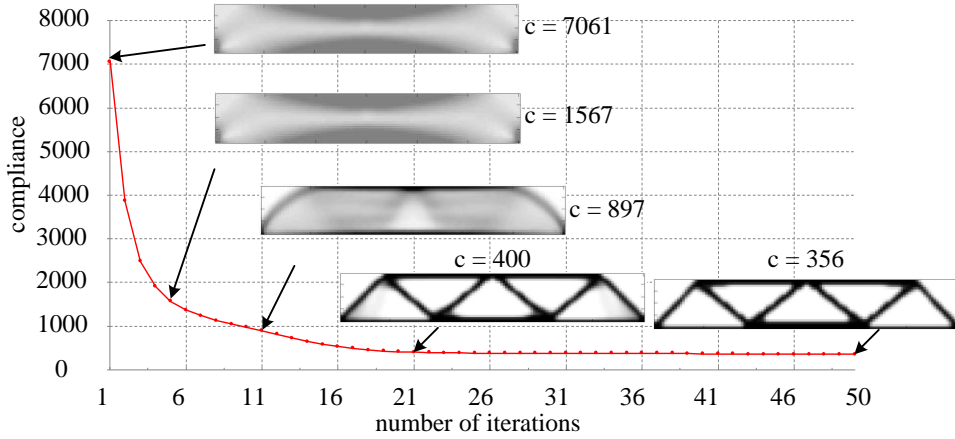


Figure 2: Convergence of the objective function

Mathematically, the optimization problem might be solved with several different approaches, such as the Optimality Criteria Method (OC) [15] or the Method of Moving Asymptotes (MMA) [16].

Both algorithms are first order methods, using the first derivative of the objective function and of the constraints to guide the update of the design variables. These derivatives comply to the sensitivities [15]:

$$\frac{\partial c}{\partial \rho_i} = -\eta \rho_i^{\eta-1} (E_0 - E_{\min}) u_i^T k_0 u_i \quad ; \quad \frac{\partial V}{\partial \rho_1} = 1$$

Increased values of an element's sensitivity indicate an increased impact of the local density on the global compliance. They are favored by the optimization approach and therefore material is launched. To improve the algorithm's numerical stability and avoid incorrect solutions, e.g. checker-board patterns, different kinds of filtering methods can be employed. Some well-known filtering techniques are the sensitivity filter, the density filter or the Heaviside projection method, which are omitted here for brevity. For a detailed description see [15; 17; 18]. It is worth to notice that the optimization problem is usually solved for one guiding load case. Multiple load cases or uncertainties can be incorporated by superimposing

single load cases or by stochastic approaches, which lead to different topologies with additional structural members (see section 4).

3 TOPOLOGY OPTIMIZATION IN BUILDING CONSTRUCTION

This section is dedicated to the demonstration of the method in the field of RC structures [19]. For this, a fictitious industrial building will serve as the object of study (see Fig. 3). Doing so, the highlighted structural elements will be designed with topology optimization methods applying the 88-line code for MATLAB by *Andreassen et al.* [20].

The structural elements analyzed read:

- (a) a girder with block-outs
- (b) a deep beam with block-outs
- (c) a column offset stabilized within a floor slab
- (d) a roof girder with an inclined top shape
- (e) stabilizing walls (deep beams)

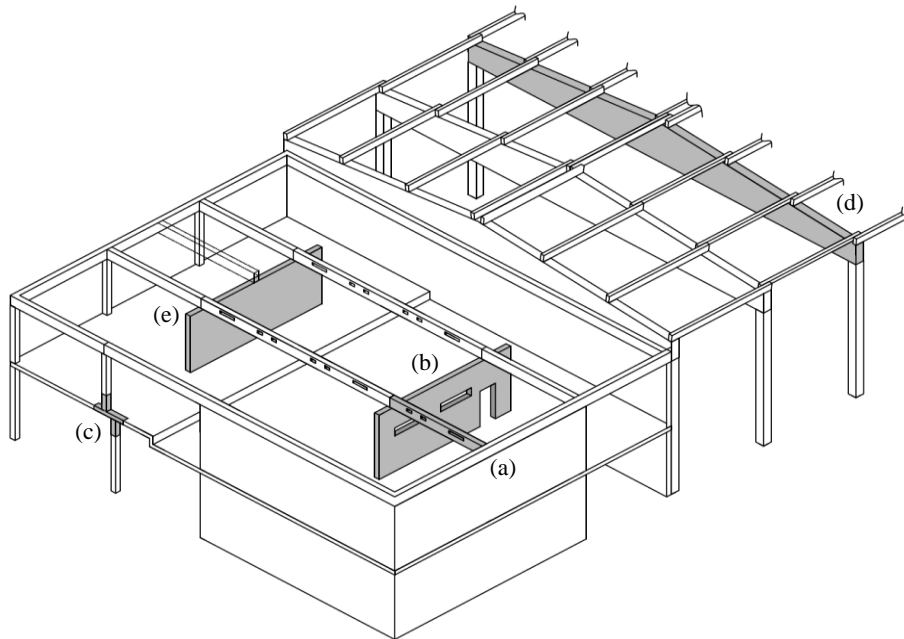


Figure 3: Industrial building

The aims of the investigations are first, to derive improved inner reinforcement layouts without changing the concrete volume [4] and second, to obtain ideal truss structures out of continuous initial designs.

3.1 OPTIMIZED REINFORCEMENT LAYOUT

For the column offset, the girder and the deep beam which can all be considered as structural elements including regions of discontinuity, topology optimization will be employed to derive suitable strut-and-tie models to improve the reinforcement layout with respect to the structural element's mechanical performance. When optimizing the reinforcement layout the compressive members of the optimization result are of minor importance due to the chosen design space filled with concrete. Thus, the compressive struts intentionally do not govern the design.

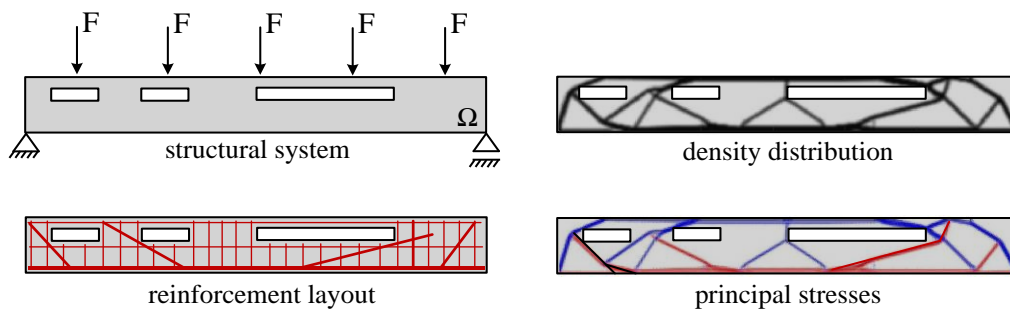


Figure 4: Girder with block-outs

In all three cases considered here the volume fraction constraint is set to $\rho = 0.3$ resulting in a reduction of 70% of the initial design domain Ω . For the sake of simplicity a uniform Poisson's ratio of $\nu=0.3$ is chosen for the reinforced concrete model. The ratio ν does have a minor impact on results, if changed in between 0,2 to 0,3. When employing topology optimization it has to be considered that the results will only provide truss structures with the essential compressive and tensile members necessary to satisfy the equilibrium. Secondary reinforcement required for practical reasons, for the limitation of crack widths or to ensure the elements ductile mechanical behavior is not included and has to be complemented by the designing engineer. Furthermore, the result does not provide the required amount of reinforcement but a principle concept for an updated strut-and-tie model.

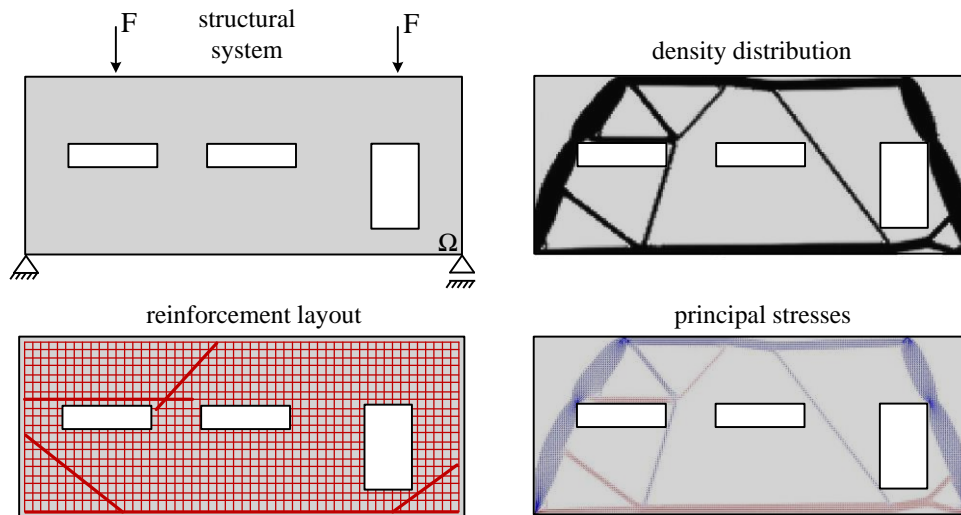


Figure 5: Deep beam with block-outs, loads, densities and a possible reinforcement layout

Figures 4 to 6 display the optimization results and give example how the reinforcement layout could be improved in comparison to traditional strut-and-tie models.

A typical application starts with the definition of structural systems, deriving density distributions as well as principle stresses σ_1 (red, tension) and σ_2 (blue, compression) and simplifying them to strut-and-tie models. In the final steps reinforcement amounts are determined and the bearing capacities of the compressive struts are checked, just as it is known for strut-and-tie modelling.

Figure 4 depicts the results of a single-span regular beam with block outs and Figure 5 a deep beam with cutouts. Both systems are loaded by point loads. The optimized trusses are characterized by a dominant tensile member at the bottom of the design spaces and inclined tensile struts next to the block-outs. The obtained densities show a certain resemblance to traditional models but are more amorphous overall, following the inner load paths. Admittedly, this seems disadvantageous in terms of practicability at a first glance. However, superimposed to an underlying grid of minimum reinforcement the overall amount decreases noticeably.

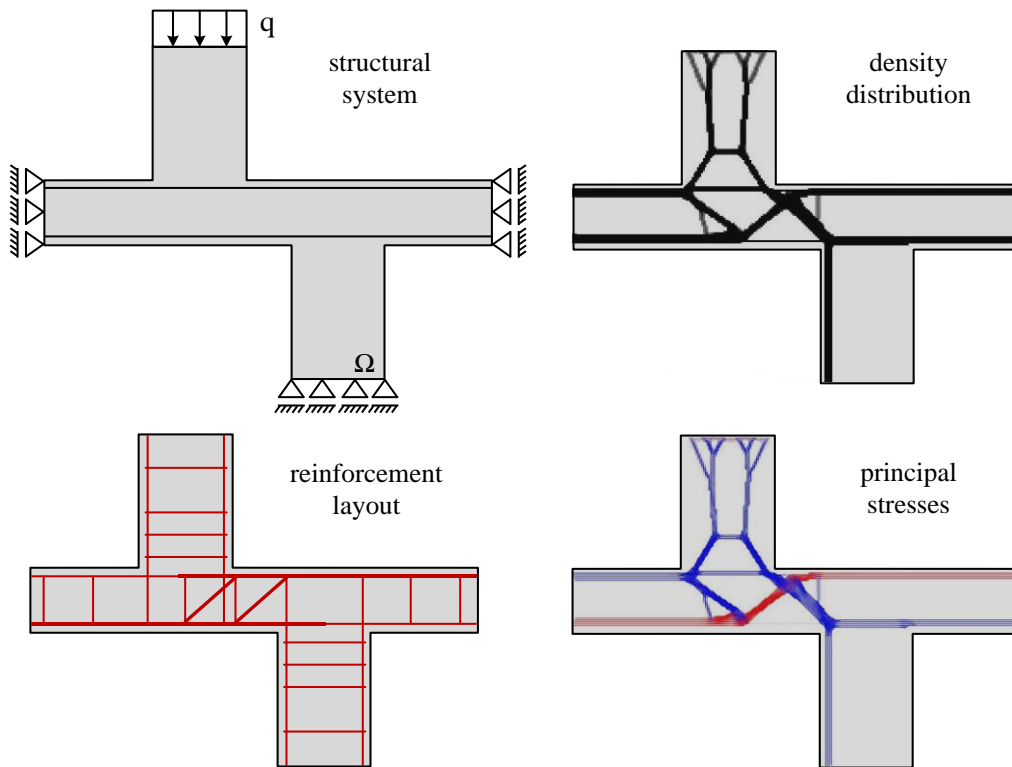


Figure 6: Column offset with loads, densities and reinforcement layout

Figure 6 shows another typical region of discontinuity in building construction, a column offset, where the vertical loads have to be deviated by the horizontal slab through bending. Therefore the intermediary slab is included into the load transfer mechanism with lateral boundaries of bending ability. These lateral elements are passive to the optimization procedure. By defining such elements the designer has the possibility to use already existing structural details and include them into the optimization formulation. The density distribution and the subsequent visualization of the principal stresses show two dominant, inclined struts as well as an inclined tie. Instead of the classical vertical shear reinforcement, inclined reinforcement arises, indicating the coupled bending and shear area between the columns.

3.2 OPTIMIZATION OF TRUSSES

Topology optimization on a system level can be applied to identify optimal truss structures, for example for roof girders (see Fig. 3). Instead of improving the structural performance with respect to its inner mechanical behavior, a suitable truss structure is aimed for within a given design space Ω . In contrast to the reinforcement layout the compressive and tensile members are of equal importance.

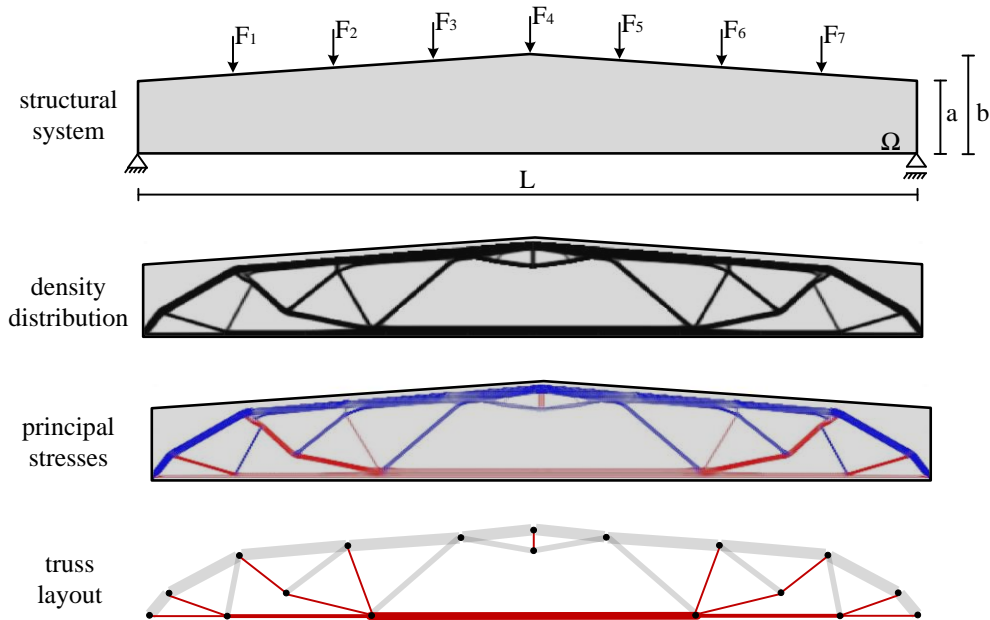


Figure 7: Consecutive development of a hybrid roof girder with topology optimization

In the contemplated case the truss structure consists of high performance concrete hollow sections for compressive and steel ties for tensile struts. This hybrid truss allows for a high degree of prefabrication. The single members are then assembled at the construction site, which promotes a time saving construction method as well as an enormous advantage in logistics. The hybrid truss concept combines the material specific advantages of concrete and steel for compressive and tensile members, but poses only one of many possibilities. Fig. 7 shows the design space for a typical roof girder. The dimensions are set to $L/b = 8$ and $a/b = 2/3$. For the optimization procedure the volume fraction constraint is set to $\omega = 0.3$, the Young's modulus to an artificial value of $E_0=1$ and the Poisson's ratio to $\nu=0.3$. The optimization results show a unique design for the truss, with struts and ties inclined at varying angles and of different lengths. The main challenge in realizing the presented truss lies in ensuring the connectivity of the different members. Following the idea of prefabrication, one solution would be the development of unitized nodal elements for various combinations of connected elements (see Fig. 8).

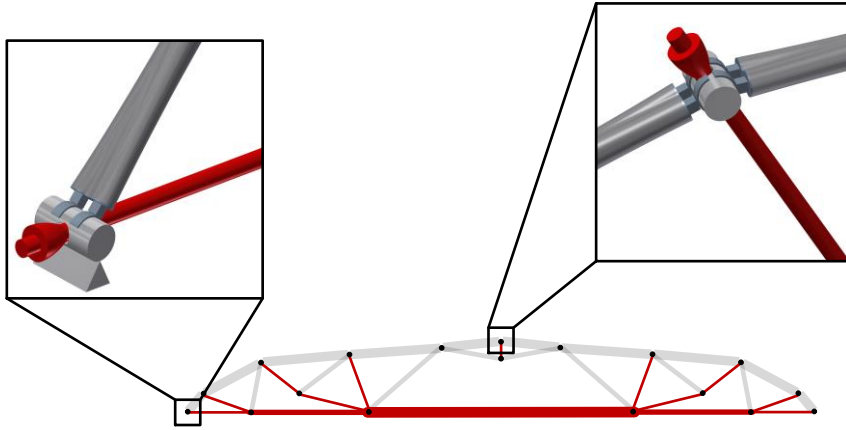


Figure 8: Unitized nodal elements

4 ROBUST DESIGN BY CONSIDERING VARIOUS LOAD CASES

Systems and structural elements in civil engineering are usually subjected to a number of load cases and uncertain parameters. Such are the scatter of material properties, geometric imperfections or the load conditions, which influence the mechanical performance. These uncertainties may have a noticeable effect and therefore may cause the actual structure to deviate from the theoretical model. To reduce the structure's sensitivity to these aforementioned uncertainties, the scatter of parameters has to be included in the design process. There are several ways to do so and to achieve a robust design. Since the extensiveness would go beyond the scope of this publication, the reader is referred to *Schueller/Jensen* [21] and *Beyer/Sendhoff* [22] for further inquiry.

In this section the influence of uncertain load conditions is demonstrated at the example of a deep beam (see Fig.3). The analyzed structural system is depicted in Figure 9 a). One way to include uncertainties of loads is to achieve density distributions from a weighted sum of the single objective functions corresponding to each single load case:

$$f(\mathbf{x}) = \sum_{k=1}^n w_k f_k(\mathbf{x}) \quad c = \sum_{k=1}^n w_k \mathbf{u}_k^T \mathbf{K} \mathbf{u}_k = \sum_{k=1}^n w_k c_k$$

In the presented case the weighting factor w_k is assumed to coincide for all three load cases ($k=3$). Figures 9 b), c), d) show the optimized strut-and-tie models for each of the load cases, which exhibit obvious differences. The weighted average optimization result is depicted in Figure 9 e). Each of the three loads could act separately or in any combination among each other. The design is able to transfer the loads in any case. A robust design is achieved. In contrast to that, Figure 9 f) shows the result of a classical optimization with all three loads simultaneously acting upon the structure. An arch-like structure develops that obviously will very sensitively react on load uncertainties.

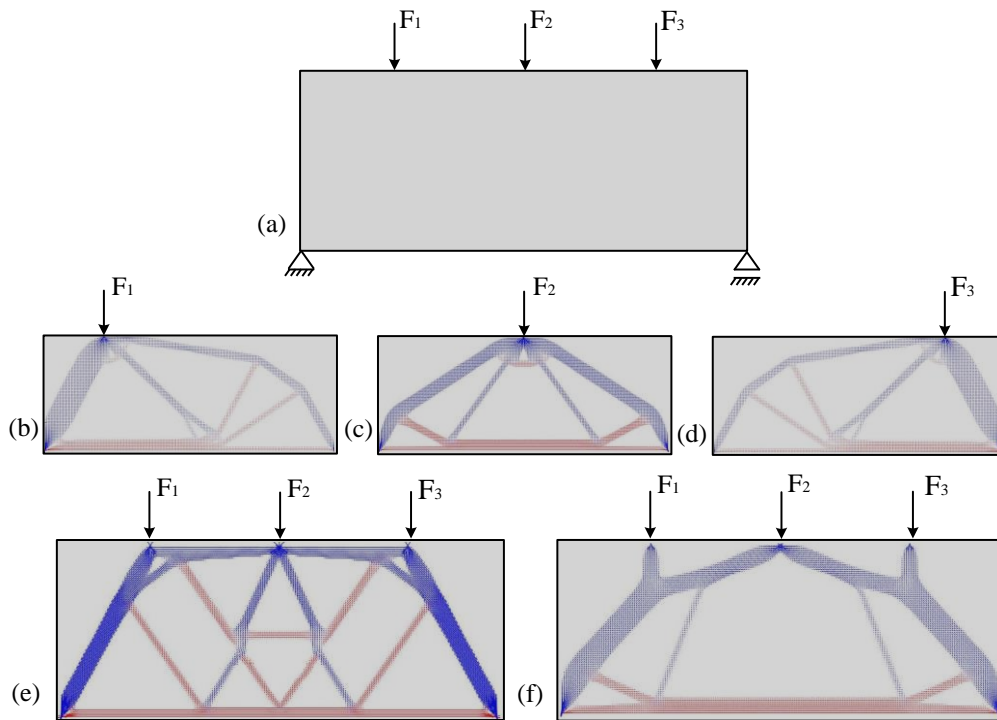


Figure 9: Deep beam and density distributions a) structural system b) optimization result to load F_1 c) to load F_2 d) to load F_3 e) weighted result f) summative result

5 DISCUSSION AND OUTLOOK

This paper presents the method of topology optimization and its application to RC structures in building construction. The practicability is demonstrated at the example of an industrial building by designing reinforcement layouts in regions of discontinuity and trusses for roof girders. It becomes evident that optimizations can be a valuable design tool to support engineers at various stages of the design process. However, to establish this asset as a common digital design tool it still has to be refined to meet the requirements of everyday engineering work. The implementation of multi-material models, using the specific advantages of different materials and the consideration of uncertainties in the optimization process will be in focus of current and future research.

ACKNOWLEDGMENTS

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Milan Veljković¹

VEZE U ČELIČNIM KONSTRUKCIJAMA - PAR PRIMJERA IZ POSLJEDNJIH PROJEKTA I STATUS EN 1993-1-8

Rezime:

U-4

Izabrani rezultati posljednjih istraživačkih projekta koji su dominantno finansirani sredstvima Naučnog Fonda za Ugalj i Čelik (RFCS) su predstavljani. Glavni kriterijum za izbor primjera je bio vrijednost za lakše projektovanje ovih veza i detalja koji u ovom trenutku nisu potpuno obuhvaćene aktuelnom verzijom Evrokoda za projektovanje veza EN 1993-1-8. U drugom dijelu članka, osnove za dalje unapređivanje trenutnog stanja Evrokoda iz sklopa koji je obuhvaćen aktivnostima komiteta CEN/TC250 je predstavljen. Glavna aktivnost i stanje napretka grupe za donošenje prijedloga za drugu verziju Evrokoda za veze EN 1993-1-8 su detaljnije razmatrane.

Ključne reči: veze sa zavrtnjevima, ovalne rupe za veze sa zavrtnjevima, tarni spojevi, injektirani zavrtnjevi, Evrokodovi, aktivnosti ekspertske grupe SC3PT2

CONNECTIONS IN STEEL STRUCTURES - RECENT RESULTS AND STATE OF ART OF EN 1993-1-8

Summary:

Selected results obtained in recently finished research projects partially supported by Research Fund for Coal and Steel (RFCS) are presented. The main focus of this part is on design recommendations of innovative connections which are not completely supported for design according to EN 1993-1-8. In the second part of the paper, basic background information and evaluation of current Eurocodes, included in the scope of CEN/TC250, are provided. Scope of the work, state of the progress and the activity plan of the SC3PT2 is given as a part of the creation of the 2nd generation of EN 1993-1-8.

Key words: bolted connection, open slotted holes, friction connection, injected bolted, Eurocodes, project team SC3PT2.

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1 INTRODUCTION

In recent years there have been a couple of research projects partially financed by RFCS that have dealt with connections and joints which cannot be designed by EN 1993-1-8 [1]. In a selected and short overview of results of the recently finished projects: HISTWIN [2] and HISTWIN2 [3], FRAMEUP [4] and on-going project SIROCO [5] the main characteristic of a specific connection will be addressed. These projects cover various applications, which is rather obvious from the title, such as: towers for wind turbine [2] and [3], modular buildings [4], or generic research related to connection of a special type, such as injected bolts [5]. However, a common characteristic of these research projects is that they are conceived to increase competitiveness of the steel structures by providing design models for connections that have not been utilized in engineering practice because there are no recommendations in [1] for their use.

2 BOLTED CONNECTION IN A LONG OPEN SLOTTED HOLES

Tubular steel towers supporting wind turbines dominate market because of their streamlined production, fast execution and maintenance efficiency. They are transported to a construction site in segments of about 20 m length. Costs of such towers and corresponding foundation are about 20% and 10%, of the total investment, respectively. The needs to increase in the hub height raises new questions concerning the design, construction and maintenance costs of the support structures posing significant transportation and execution challenges. Limits of transportation depends on the public road routes, which defines the max diameter often around 4 m.

Towers supporting wind turbines are often about 100 m height, having commonly 3 to 4 connections which consist of two ring flanges. Every ring flange is welded on each end of the cylindrical tower, see Figure 1.a/. However, recent research results has created a competitive alternative: friction connections with long open slotted holes shown in Figure 1.b/. Using the friction connection, the fatigue resistance of the flanges may be overcome as the limiting design criterion, because the fatigue class is about 100 compared to the ring flange connection which has class 40 to 71. Additionally, the assembly process on site will be accelerated.

For design purpose, it is assumed that the resistance of the three dimensional connection detail, which is loaded mostly in bending, can be described by the resistance of a segment with a single bolt and bolt row, respectively. The segment width c is equivalent to the arc length between two bolt holes or bolt rows in the tension zone of the shell, see Figure 1.a/.

Friction connections with normal clearance holes have been used in structural engineering for decades. Their behaviour has been extensively examined by various researchers and is comprehensively described by Kulak et al. [6]. Slip resistant joints rely on load transfer between the joined elements due to friction, which is ensured by a clamping force provided by preloaded high strength bolts, see Figure 1.

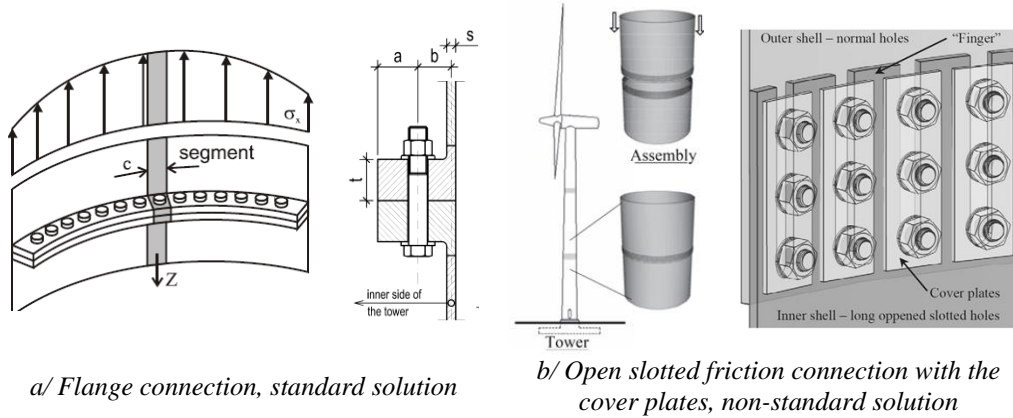


Figure 1 Connections between two parts of a tower for wind turbines

The ultimate resistance of the friction connection corresponds to its ultimate slip resistance, formula (1). The friction connection with long open slotted holes is a single lap joint, thus providing one friction surface $n = 1$. Recent findings in the HISTWIN2 project [3] have shown that the second friction surface is activated after the ultimate limit state is reached. This was possible because the bolts are fitted in the tower segment with circular holes and no slip exist between the outer cylindrical segment and the cover plates used inside the tower. The ultimate bearing resistance is easily determined according to the following hand-calculation procedure:

1. Determination of the ultimate slip design resistance $Z_{s,Rd}$ of one bolt row of the friction joint (segment model):

$$Z_{s,Rd} = \frac{n_r \cdot \mu \cdot k_s \cdot F_{p,C}}{\gamma_{M3}} \quad (1)$$

Where:

- n_r = number of bolts per slotted hole,
- μ = slip factor,
- k_s = reduction factor for long slotted holes,
- $F_{p,C}$ = characteristic preload force in bolt,
- $\gamma_{M3} = 1,25$ partial safety factor.

2. Determination of the corresponding maximum stress in shell $\sigma_{x,t}$ due to force Z_{Ed} in segment model, see Figure 1.b/:

$$Z_{Ed} = A_{shell} \cdot \sigma_{mean} = c \cdot s_{mean} \cdot \sigma_{mean} \cong c \cdot s_{mean} \cdot \sigma_{x,t} \quad (2)$$

where:

- A_{shell} = area of cross section of segment model,
- σ_{mean} = mean of tension/compression stress in segment model,
- $\sigma_{x,t}$ = maximum tension/compression stress in shells,

c = segment width = distance between two bolt rows,
 s_{mean} = mean shell thickness of the two connected tower segments.

Comparative cost analysis in [2] showed that up to 80 % reduction of the connection costs in a tower can be obtained by using the novel friction connection with long open slotted holes instead of the classical ring-flange connection.

3 BEAM TO COLUMN CONNECTION USING TUBULAR SECTIONS

The joint with “reverse channel” is an emergent typology of joint, which when properly characterized, can become a competitive solution for intersections where the use of standard bolts is blocked due the difficulties of accessing the bolts during the tightening process.

The reverse channel joint, see Figure 2 is characterized by the use of an auxiliary stub, commonly called reverse channel due its cross section geometry. The reverse channel is welded to a supporting member and the incoming member is then bolted in the reverse channel web enabling the tightening process with standard bolts. In the particular case of the beam to column joints of the FRAMEUP building, both supporting and incoming members are tubular. The adoption of a bolted solution, with standard bolts, to directly connect those members is obviously unfeasible. On the other hand, the requirement for assembly, disassembly and subsequent re-assembly of the structure implied the use of a bolted solution with standard bolts. Moreover, the joint had to be moment resistant and stiff enough to assure an adequate performance of the 6 storey building without vertical bracings. Thus, the joint with reverse channel is adopted for the FRAMEUP beam-to-column joints.

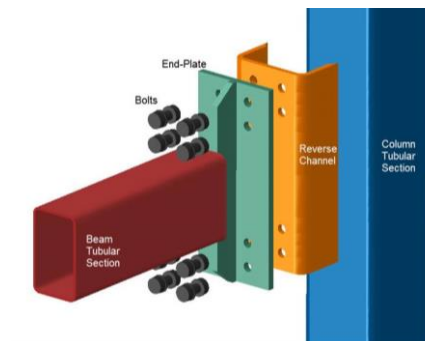


Figure 2 Joint with the reverse channel

The basic components of the FRAMEUP reverse channel joint are identified, characterized and assembled, enabling the evaluation of the structural response of the whole joint in chapters below. The component method procedure is then illustrated with five examples similar to specimens used in laboratory experiments.

The basic components of the reverse channel joint, under hogging bending moment, are grouped according to the nature of forces they are submitted to. The identified components are summarized below and identified in Figure 3.

Table 1 Identification of the basic FRAMEUP joint

Tension Zone	Shear zone	Compression zone
<ul style="list-style-type: none"> - End plate in bending - Beam in tension - Reverse channel “legs” in transverse tension - Column web in transverse tension - Reverse channel web in bending due tension - Bolts in tension - Welds in tension 	<ul style="list-style-type: none"> - Column web and reverse channel legs in transverse shear 	<ul style="list-style-type: none"> - Reverse channel legs in compression - Column webs in compression - Beam in compression - Reverse channel web in bending due compression - Welds in compression

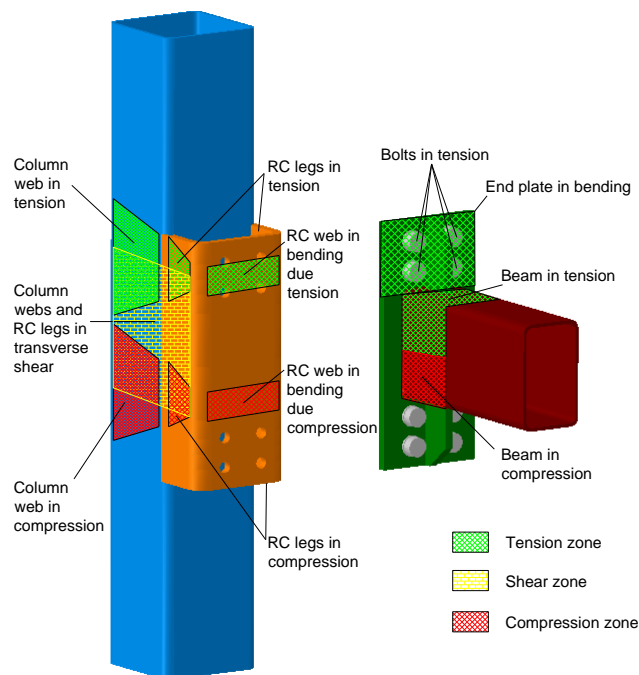


Figure 3 Components of the joint with the reverse channel

Characterization of the basic components is done in two steps. In a first step components which are already covered by formulae in [1] are identified. In the second step new components beyond the scope of [1] are defined. Then, the characterization of the components is made following the recommendations in [1]. For the new components, those related to the reverse channel, the characterization is made considering different approaches based on recent studies on the reverse channel joint provided in [4]. Those components are the reverse channel web in bending due tension and the reverse channel in bending due compression.

Components of reverse channel legs in tension and reverse channel legs in compression as well as the reverse channel legs in transverse shear, although being part of the reverse channel, are characterized by analogy with components covered in [1].

Summary of the resistance and stiffness parameters which characterize each basic component are provided in Table 2 .

Table 2: Identification of the resistance and stiffness parameters which characterize each basic component

	Basic component	Resistance	Stiffness
Tension zone	End plate in bending	$F_{t,Rd}$	k_5
	Bolts in tension	$F_{t,Rd}$	k_{10}
	Beam in tension	$F_{t,b,Rd}$	-
	Column webs in transverse tension	$F_{t,wc,Rd}$	k_{3c}
	Reverse channel legs in transverse tension	$F_{t,rcl,Rd}$	$k_{3,rcl}$
	Reverse channel web in bending due tension	$F_{t,rcw,Rd}$	$k_{rcw,t}$
Shear zone	Column web and reverse channel legs in transverse shear	$V_{wp,Rd}$	k_1
Compression zone	Column webs in compression	$F_{c,wc,Rd}$	$k_{2,c}$
	Reverse channel legs in compression	$F_{c,rcl,Rd}$	$k_{2,rcl}$
	Beam in compression	$F_{c,b,Rd}$	-
	Reverse channel web in bending due compression	$F_{c,rcw,Rd}$	$k_{rcw,c}$

According to [1], the design moment resistance $M_{j,Rd}$, of a beam to column joint with a bolted end-plate connection may be determined using following formulae:

$$M_{j,Rd} = \sum_r h_r \cdot F_{tr,Rd} \quad (3)$$

where:

$F_{tr,Rd}$ is the effective design tension resistance of bolt-row r ;

h_r is the distance from bolt-row r to the centre of compression;

r is the bolt-row number

For the FRAMEUP joint, with two bolt rows, a simplification similar to the one shown in is appropriate. Thus, in spite of two levels of tension forces, aligned with the bolt rows in tension, the resultant of those forces acting at a distance z_{eq} from the centre of compression it is only considered as depicted in Figure 4.

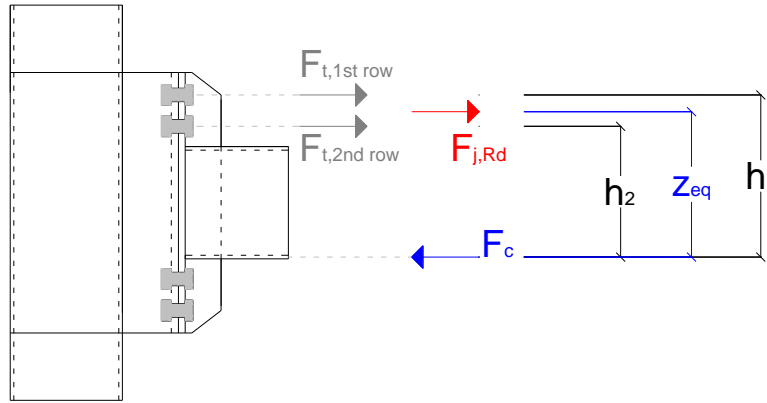


Figure 4. The pair of forces in tension and lever arms considered in the calculation of the $M_{j,Rd}$

The centre of compression is considered in line with the mid-thickness of the compression flange and the lever arm is conservatively taken as z_{eq} according to the equation presented in the next chapter. The z_{eq} is adopted as lever arm to take into account the bolt rows stiffness in the determination of the equivalent forces binary. Given this, the previous equation can be simplified according to the following equation.

$$M_{j,Rd} = \min[F_{j,Rd}] \cdot z_{eq} \quad (4)$$

where:

$F_{j,Rd}$ is the tensile resistance of the two bolt rows in tension

z_{eq} is the lever arm

It's important to notice that the beam capacity to bending, $M_{pl,Rd}$, is determined according to clause 6.2.5(2) from EN 1993-1-1, disregarding the partial factor, in spite of consider a binary of forces.

According to EN 1993-1-8 [1] the rotational stiffness, S_j , for an acting moment, $M_{j,Ed}$, less than the design moment resistance of the joint, $M_{j,Rd}$, may be obtained from:

$$S_j = \frac{E \cdot z^2}{\mu \cdot \sum_i \frac{1}{k_i}} \quad (5)$$

where:

E is the elastic modulus, assumed as the average value in between the elastic modulus values obtained from the tensile tests of each joint element

k_i is the stiffness coefficient for basic joint component i ;

z is the lever arm, the same one considered in the previous chapter;

μ is the stiffness ratio $S_{j,ini}/S_j$, according to clause 6.3.1(6) of EN 1993-1-8, taken as 1 for the calculation of the initial rotational stiffness $S_{j,ini}$.

In cases with two bolt rows, such as this case, the same standard recommend that the bolt rows should be represented by a single equivalent stiffness coefficient k_{eq} determined from

$$k_{eq} = \frac{\sum_r k_{eff,r} \cdot h_r}{z_{eq}} \quad (6)$$

$k_{eff,r}$ – effective stiffness coefficient for bolt row r , determined from: $k_{eff,r} = \frac{1}{\sum_i \frac{1}{k_{i,r}}}$

z_{eq} – lever arm, taken as $z_{eq} = \frac{\sum_r k_{eff,r} \cdot h_r^2}{\sum_r k_{eff,r} \cdot h_r}$, according to clause 6.3.3.1(3) EN 1993-1-8

h_r – distance between bolt row r and the centre of compression.

Thus, for the calculation of the joint rotational stiffness, the components assembly should follow the steps depicted in Figure 5.

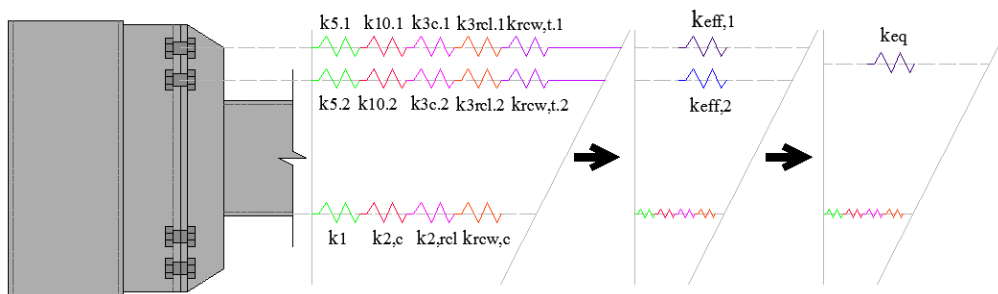
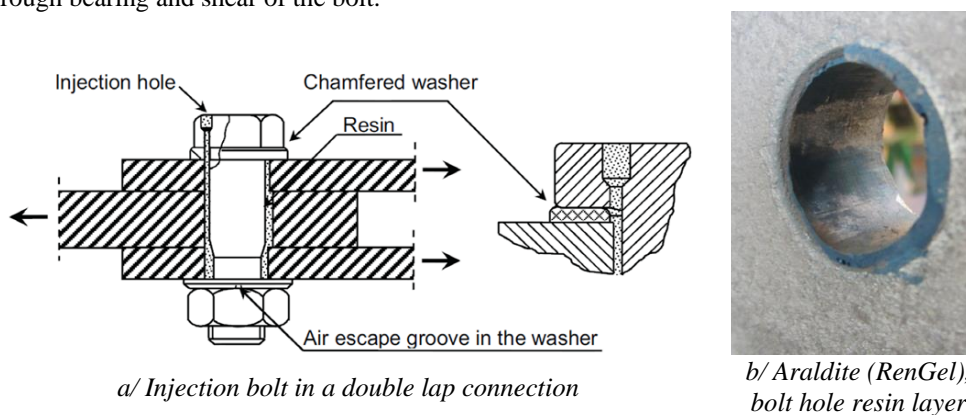


Figure 5. Springs model considered for the calculation of the rotational stiffness

4 INJECTED BOLTS

Injection bolts are bolts in which the cavity produced by the clearance between the bolt and the wall of the hole is completely filled up with a two component resin. Filling the clearance of an injection bolt is carried out through a small hole in the head of the bolt, see Figure 6. After injection and curing of the resin, the connection is slip resistant. Shear load is transferred through bearing and shear of the bolt.



a/ Injection bolt in a double lap connection

b/ Araldite (RenGel), bolt hole resin layer

Figure 6 Injection bolt

Injection bolt is obtained by drilling the hole through the head of the standard structural bolt. The standardized washer is adapted to enable the injection of the resin. Injection bolts may be applied in new structures and in existing structures for repair and strengthening. Injection bolts have the advantage that in case of overloading no sudden large displacements occur as may happen in high strength friction grip bolts when they slip. This fact is reflected in a higher bearing stress accepted for the ultimate limit state by a factor 1.2, compared to the serviceability stresses, which was defined in ECCS Recommendations (1994) [11]. These rules were the starting document for creation of the section 3.6.2 on injection bolts in EN 1993-1-8, where the design bearing resistance of an injection bolt $F_{b,Rd,resin}$ should be determined according to the following formulae:

$$F_{b,Rd,resin} = \frac{k_t k_s d t_{min} \beta f_{b,resin}}{\gamma_{M4}} \quad (7)$$

where

k_t is 1,0 for serviceability limit state (long duration),

is 1,2 for ultimate state,

k_s is taken as 1,0 for holes with normal clearances (1,0-0,1 m) for oversized hole,

m is difference (in mm) between the normal and oversized hole dimensions,

d is bolt diameter,

$f_{b,resin}$ is the bearing strength of the resin determined as it is shown below,

$\gamma_{M4} = 1,0$ partial safety factor for injection bolts

Validation of the design bearing resistance of an injection bolt is based on the equation below:

$$nF_{b,Rd,resin} \leq nF_{b,Rd} \leq \min(N_{pl,Rd}, N_{u,Rd}, nF_{v,Rd}) \quad (8)$$

where:

n is number of bolts

$F_{b,Rd}$ is the design bearing resistance of bolts;

$N_{pl,Rd}$ is the design plastic resistance to normal forces of the gross cross section

$N_{u,Rd}$ is design ultimate resistance to normal forces of the net cross-section at holes

for fasteners

$F_{v,Rd}$ is the design shear resistance per bolt

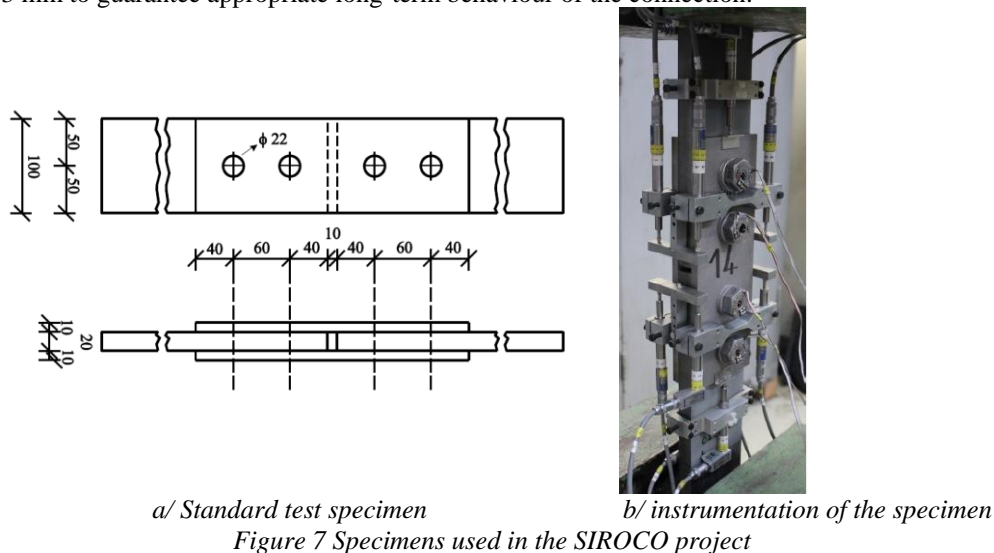
Rules for testing the resin for its bearing strength can be found in EN 1090-2 (2008) [12]. The geometry of the standard test specimen with M20 bolts and instrumented test specimens at the beginning of the test are shown in Figure 7.a/ and Figure 7.b/, respectively. For each connection type four short-duration tests have to be done (10 min – 15 min) and the bearing resistance is determined at 0.15 mm slip. The fifth test is long-duration creep test at constant load equal to 90% of the average resistances from the first four tests. Each specimen has two connections, there are actually eight results from 4 specimens. The average slip in each of the four specimens for the time period of 5 minutes to 3 hours after the full load is applied should not exceed 0.002 mm. If this is achieved, results of 4 specimens used to reach the slip of 0.15 mm are basis to define characteristic bearing resistance of tested resin is determined based on the obtained results (mean - 2.05 x standard deviation). If a creep test has shown slip that exceeds 0.002 mm, the long-duration test should be extended until safe extrapolation of slip at

50 years is obtained; this slip should be smaller than 0.3 mm to guarantee appropriate long-term behaviour of the connection.

It is worth mentioning that for connections with preloaded injection bolts EN 1993-1-8 allows summation of design slip resistance and design bearing resistance of the resin

Resins are susceptible to creep-deformation depending on the bearing stress level. In the Fourth International Workshop on Connections in Steel Structures (2000) [12], test results from long duration testing were presented on the two component epoxy resin Araldit SW 404 + HY2404 hardener, now available as RenGel SW 404 + Ren HY 2404. The most of the work on injected bolts have been conducted at TUD, RWTH and at Ljubljana university. Based on their results it was concluded that injection bolts are an excellent alternative for replacing faulty rivets in (old) riveted structures and for strengthening of existing structures.

The geometry of the standard test specimen with M20 bolts and instrumented test specimens at the beginning of the test are shown in Figure 2 and Figure 3, respectively. For each connection type four short-duration tests have to be done (10 min – 15 min) and the resistance is determined at 0.15 mm slip. The fifth test is long-duration creep test at constant load equal to 90% of the average resistances from the first four tests. Because each specimen has two connections, there are actually eight results. The average slip in each of the four connections for the time period of 5 minutes – 3 hours after the full load is applied should not exceed 0.002 mm. If this is achieved, the specimen is tested to reach the slip of 0.15 mm and the characteristic resistance is determined based on the obtained results (mean - 2.05 x standard deviation). If the creep test shows slip that exceeds 0.002 mm, the long-duration test should be extended until safe extrapolation of slip at 50 years is obtained; this slip should be smaller than 0.3 mm to guarantee appropriate long-term behaviour of the connection.



It is worth mentioning that for connections with preloaded injection bolts EN 1993-1-8 allows summation of design slip resistance and design bearing resistance of the resin, section 3.6.2.2 (4)P.

In ongoing research SIROCO project this rule as well as extension and improvement of other design rules in Eurocode 3 Part 1-8 (2005), taking into account various loading

conditions will be done. Furthermore, the systematic research will be performed for improvement of the test procedures for the design bearing stresses in EN 1090-2 (2008), taking into account various loading conditions, and development of guidance on quality assurance of installation and applied resin.

5 STATE OF ART OF THE EN 1993-1-8

Differences in design/calculation methods of the national building regulations had imposed obstacles to the free circulation of engineering and architectural services within the Community. Therefore the original objectives, which dates from 1975, for the first generation of Eurocodes, which remain current, is to establish a set of common technical rules for the design of building and civil engineering works and to replace the differing rules in force in the various Member States.

The Eurocodes, which comprise a set of 58 harmonized European standards on 5219 pages (without national Annexes), which are part of the suit from EN 1990 to EN 1999. They have been developed to enable the design of construction works (building and civil engineering works) in order to comply with mechanical resistance and stability, safety in case of fire and safety in use and to determine the performance of structural construction products.

Under the Public Procurement Directive [15], it is mandatory that Member States accept designs to the EN Eurocodes. Comprehensive overview of state of implementation of Eurocodes in different member states is provided in [16].

The first generation of Eurocodes have been published in 2007, which has become mandatory in member states. The revision of the Eurocodes is on-going in period from 2015 to 2020 when it is expected to published the second generation of Eurocodes.

Each Member State should be provided with the ongoing opportunity to make input to their National Standards Body and hence, via National Delegations, to CEN/TC250 (European Committee for Standardization, Technical Committee 250 “Structural Eurocodes”) and its SCs (Subcommittees) and WGs (Working Groups). It should be noted that there is a clear and vital distinction between design codes and national regulations. The harmonization of National Regulations is not within the scope of the National and European Standards Bodies.

There are 10 focus areas for further development of Eurocodes listed in [17] among which two most interesting, for the purpose of this overview, are:

- reduction of numbers of NDPs (Nationally Determined Parameters), and
- refinement to improve the ‘ease of use’ of Eurocodes by practical users.

Revision of EN 1993-1-8 has started, together with EN 1993-1-1, in the first phase of the evolution to 2nd generation which is planned for period 2015-2018. The work on the revision is carrying out through the transparent consultations between practitioners and academics gathered in Evolution Group, who creates valuable and necessary input to a small group of 6 experts of the project team, SC3PT2. Furthermore, the ECCS Technical Committee 11 provides additional knowledge support for the activity of WG8 (Working Group 8 on connections, former Evolution Group) which are creating input to CEN/TC250/SC3 amendments

SC3PT2 objectives:

Revision of EN 1993-1-8 to incorporate collated comments prepared by the sub-groups of WG 8 on EN 1993-1-8, based on the periodic review. The comments should be approved by WG 8 and SC3.

Integration of connection rules developed for high strength steels EN 1993-1-12, and harmonization with other parts such as EN 1993-1-3 (cold formed members and sheeting) and EN 1993-1-2 (Structural fire design). In order to enhance the ease of use the component method will be revisited and modification developed.

Due attention to accomplish simple routes throughout the document and to avoid additional and/or empirical rules.

All above objectives will be considered to extent that is reasonably practical and in terms of allowable time.

The main course of work will be recognition of users' needs and consensus among the NSBs.

The WG8 has received, analysed and systemized 395 comments on EN 1993-1-8.

The classification of the comments is done within four subgroups of WG8: bolting (Ch.3, Welding (Ch.4), Component method (Ch.5 and Ch6) and Tubular sections (Ch7). In the systematic review process all comments are classified as follows:

(1) Accepted (PT):

Editorial and obvious mistakes that can be directly dealt by PT2

(2) Accepted (WG):

Relevant comment that need to be dealt and further elaborated by WG8, and probably accepted by SC3 decision

(3) Rejected:

In this case a reason should be given, e.g. demand is against policy of "clarity" and "ease of use".

(4) Clarification (WG):

Comment that cannot easily be answered and needs further considerations and discussions in WG8.

(5) Decided (SC3):

Comment and issue that is already treated by SC3 and decided as "basket"-decision, no discussion again.

(6) Under discussion (WG8).

The biggest majority, 173 comments, has been received for Ch.7. In addition, a new development of component method for joints of tubular section has been proposed [18] for inclusion in the 2nd generation of the Part 1-8.

In addition to above mention tasks, PT2 is dealing with following items:

Table 3 Activity of SC3PT2

Item	Progress and milestones	Next steps and remaining work
Extension of design rules for modern joints and connections.	Preliminary state of art is prepared.	<ul style="list-style-type: none"> - Detailed rules for implementation. - Discussion at WG8 - Dialog with NSBs.
Stainless steel bolted connections	Dialog with the Convenor for WG4, which covers the design of structural stainless steel.	- The work on design of connections will be prepared by the WG4 of EN 1993-1-4 on stainless steel.
Extension of design rules for welded joints and for mixed steels connections	Modifications to comply with the Systematic Review addressing all "A" comments is done.	- Continue dialog with subgroup of WG8 on welded connection for remaining technical tasks.
Further development of the component models for joints in order to increase the robustness of joints.	Preliminary comments on modelling of joints and proposal for amendments including seismic design are provided.	<ul style="list-style-type: none"> - Simplification of design rules for standard joint solution. - Use of 3D modelling - Specification of necessary and available rotation capacity for the joints.
Integration of design rules for advanced material	Proposals for amendments for connection using high strength steel are collected.	- Harmonization of application rules for advanced materials: cast iron and steels for quenching and tempering, and alloy steels.
Hollow section joints	Preliminary discussion with members of ISO 14346 and CIDECT group.	- Implementation and integration of new research results.
Clarification and specification of the application of EN 1090-2	Preliminary report on bolted connection, product standard for bolts and specific problems related to base plate are discussed.	- Preparation of report on product standard for bolts and tightening method for bolts.

6 CONCLUSIONS

Innovations related to the connections strongly increase competitiveness of steel structures. There is a still lot improvement to be made to improve design and execution of existing connections and joints. In the paper just a couple of examples are highlighted as well as activity plan for improvements of the 2nd generation of EN1993-1-8.

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Dragan Majkić¹

SPECIFIČNA INŽENJERSKA REŠENJA U IZVOĐENJU MOSTOVA

Rezime:

U-5

U radu će biti prezentovana neka od specifičnih inženjerskih rešenja koja su bila diktirana raznoraznim okolnostima u toku izvođenja: Coal River British Columbia most izveden na ledu; Lions Gate u Vancouver–u, most sa zamenom kolovozne konstrukcije sa neprekidnim saobraćajem u toku dana; Bay Bridge u San Francisku sa dizanjem i navlačenjem 1500 t mostovskih segmenata; Kikoman most gde je istovremeno primnjena metoda izvlačenja i navlačenja mosta; Walterdale most u Edmontonu sa interesantnim prebacivanjem tereta sa obale na barže i kasnije podizanjem 144m centralnog luka teškog 2000 t; Slims River most Yukon, gde je primenjena veoma jednostavna metoda navlačenja. Navedeni mostovi u određenim segmentima izvođenja predstavljaju na neki način inovaciju i visoku tehnologiju, dok su na drugoj strani prikazani jednostavni i praktični metodi za izvođenje mostova.

SPECIFIC BRIDGE ERECTION ENGINEERING SOLUTIONS

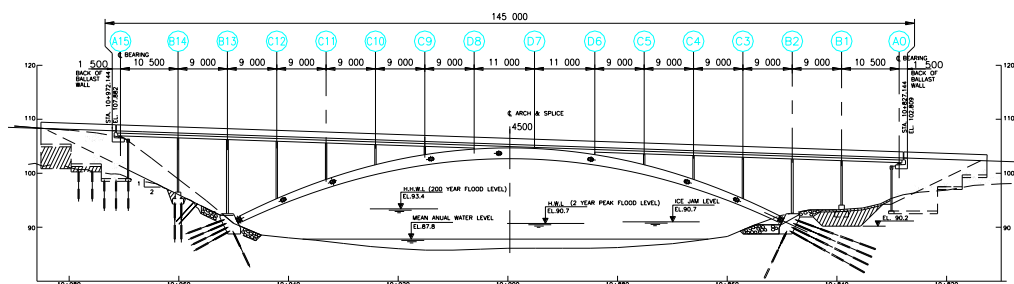
Summary:

This paper will present some specific engineering solutions which were dictated by various circumstances during execution: Coal River in British Columbia Bridge - Installation carried out on the ice; Lions Gate Bridge in Vancouver - Deck Replacement without traffic closure during reconstruction; Bay Bridge in San Francisco – Lift and Movement of a 1500 t bridge segment; Kikoman Bridge - Launch-Delaunch of the bridge; Walterdale Bridge in Edmonton - interesting load transfer scheme from shore on barges and later lifting a 2000t, 144m central arch segment; Slims River bridge Yukon - a very simple – practical bridge launching method. These examples of actual bridge construction show a high degree of innovation and technology development, while also exemplifying simple and practical methods of bridge installation.

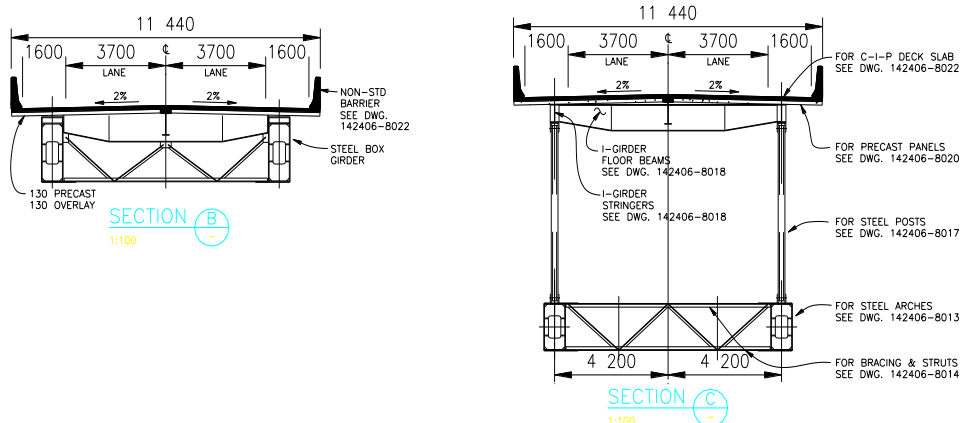
¹ P.Eng., Allnorth Bridge Engineer Specialist, Vancouver, BC, Canada

1 COAL RIVER MOST, ALASKA HIGHWAY, BC, KANADA

Ovaj projekat predstavlja izuzetan primer građenja u zimskom periodu sa ekstremnim temperaturama. Niska temperatura -15°C do -35°C je omogućila primenu neuobičajene tehnologije izvođenja mosta uzimajući u obzir da je najbliže naseljeno mesto na severu Whitehorse, i na jugu Ford Nelson na međusobnoj udaljenosti od 1000km.



Slika 1.1 Izgled



Slika 1.2 Karakteristični poprečni preseći mosta

1.1 OPIS KONSTRUKCIJE

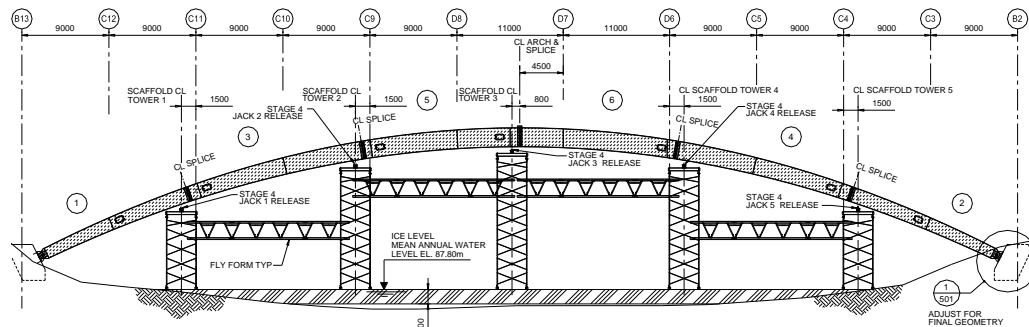
Ukupna dužina mosta je 145m sa glavnim lučnim rasponom od 100m i strelom 12m. Glavnu konstrukciju mosta čine dva čelična luka promenljivog sandučastog preseka, zglobno oslonjena na obalne stubove. Kolovozna konstrukcija, spregnuta betonska ploča, je oslonjena na lukove putem tri glavna podužna čelična "I" nosača preko stubova kutijastog preseka.

1.2 TEHNOLOGIJA IZVOĐENJA ČELIČNE KONSTRUKCIJE

Ovaj projekat je urađen od strane Kanadske izvođačke kompanije Surespan, West Vancouver, B.C. Izvođač je odabrao QR Engineering za razradu tehnologije izvođenja mosta. Činjenica da je izvođenje padalo u zimskom periodu, uslovljavala je da se usvoji tehnologija izvođenja sa leda, koristeći led za oslanjanje skele, kranova i transportnih vozila (vidi Sl 1.3 i 1.4). Zatečena debljina leda u trenutku početka radova na montaži čelične konstrukcije mosta

je bila 60 cm. Proračunom se zahtevala debljina leda minimum 1.5m. Ledena zona montaže mosta je zagrađena i kroz prethodno izbušene rupe u ledu tokom dana pumpana je voda iz reke na površinu u debljini od 5cm. Preko noći, zbog niskih temperatura, voda se zaleđivala. Ovaj postupak se ponavljao sve dok se nije postigla projektovana debljina leda.

Za izvođenje lučnih sistema je veoma važno održavanje geometrije sistema. Ovo je bio glavni izazov u projektovanju tehnologije izvođenja jer je projekat morao da obuhvati razradu problema velikih deformacija leda i veliku promenu dužine luka iz razloga niskih temperature tokom izvođenja. Uzimajući u obzir činjenicu da će elementi mostovske konstrukcije biti fabrikovani na prosečnoj temperaturi od +16°C, a izvođenje treba da se obavi na veoma niskim temperaturama proračun je pokazao da će luk biti kraći između 22-36 mm zavisno od trenutne temperature na gradilištu. Analizom luka kao proste grede, sa levim pokretnim osloncem, konstatovana je velika fleksibilnost luka.



Slika 1.3 Podužni izgled skele



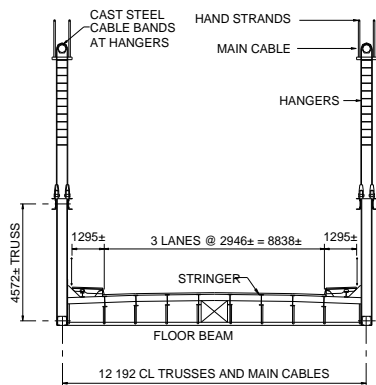
Sl. 1.4 Pogled na gradilište u toku izvođenja lukova mosta

2 REKONSTRUKCIJA LIONS GATE MOSTA, VANCOUVER, BC

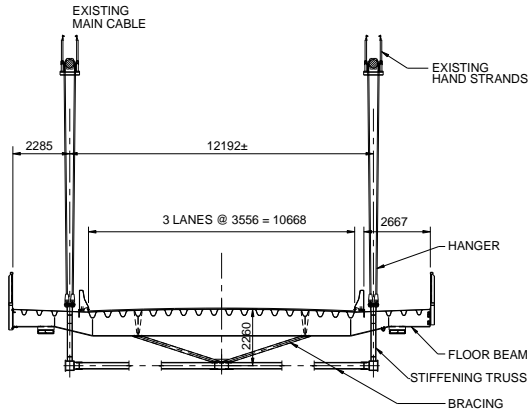
Rekonstrukcija Lions Gate mosta uključuje kompletnu zamenu zatega i rešetke za ukrucenje zajedno sa kolovoznom konstrukcijom, uključujući inženjerski rad na planiranju i razvoju procedure izvođenja, projektovanje uređaja za dizanje i izvođenje istih. Ovaj projekat je urađen od strane Kanadske ekspoziture “American Bridge” kompanije iz Pittsburgh-a PA (USA) u saradnji sa “Surespan General Contractor”, North Vancouver, B.C. Izvođač je odabrao Parsons (jednu od svetski najpoznatijih projektanskih kompanija za viseće mostove, poznata još pod imenom Steinman) da uradi analizu konstrukcije tokom izvođenja. Uređaji za dizanje i detaljne procedure za izvođenje su razrađivani od strane izvođačkog inženjerskog tima lociranog na samom gradilištu uz asistenciju inženjerskih kompanija iz Vankuvera “QR Engineering”, “Klohn-Crippen” i “Associated Engineering”. Parsons Brinckerhoff, Inc. (PB), najveća Severno – Americka projektantska kompanija za mostove velikog raspona) je uradio nezavisnu tehničku kontrolu, Parson-ovu inženjersku analizu izvođenja i glavne uređaje za dizanje. “West Wind Laboratory Inc”, Marina, CA, jedna od malobrojnih američkih aerodinamičkih inženjerskih kompanija je angažovana od strane izvođača kao specialista za vetar.



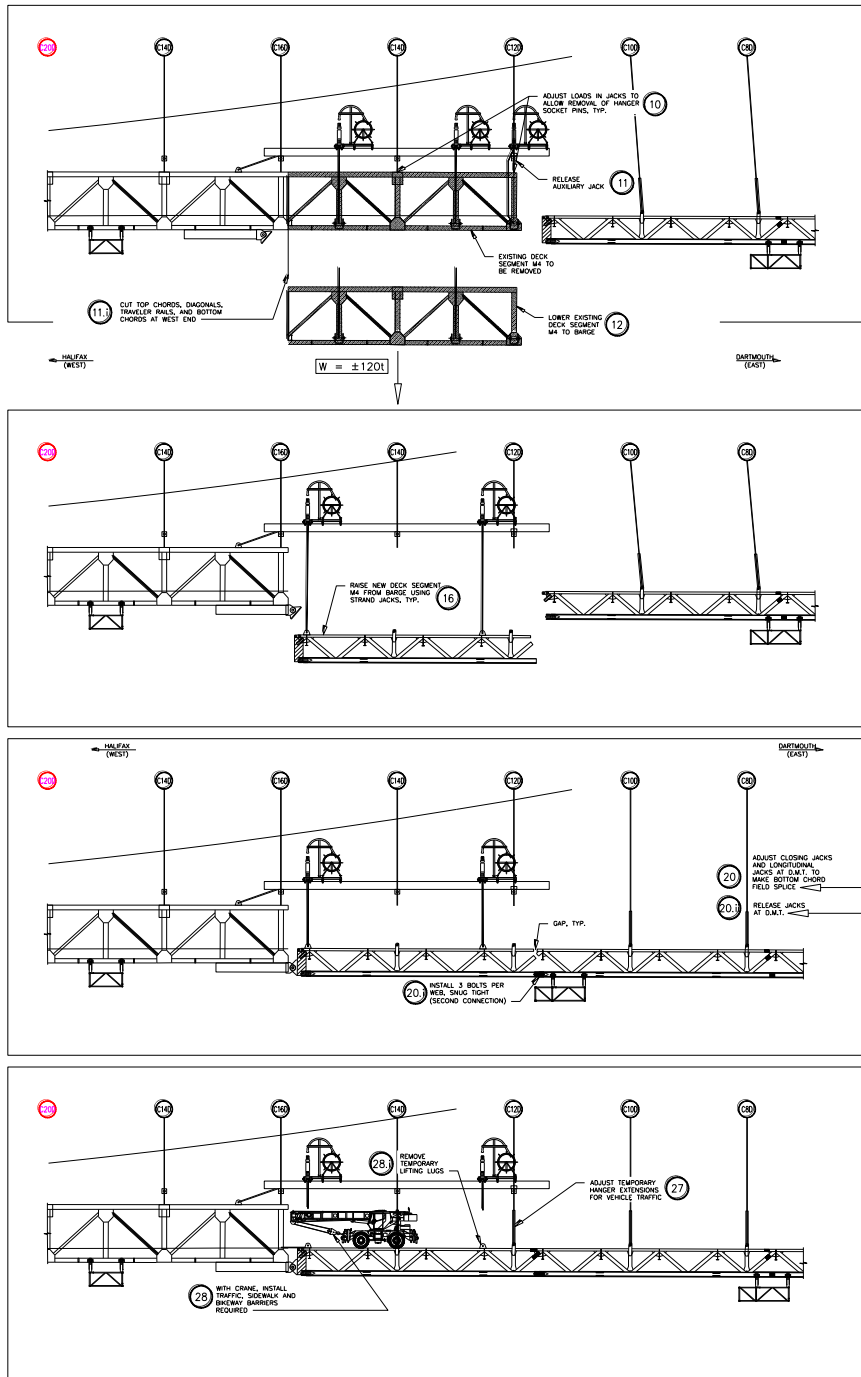
Sl. 2.1 Izgled



Sl. 2.2: Poprečni presek starog mosta



Sl. 2.3: Poprečni presek novog mosta



Sl. 2.4 Šema tehnologije zamene segmenta za severni i srednji raspon mosta

2.1 KONAČNA IZVOĐAČKA ŠEMA IZVOĐENJA

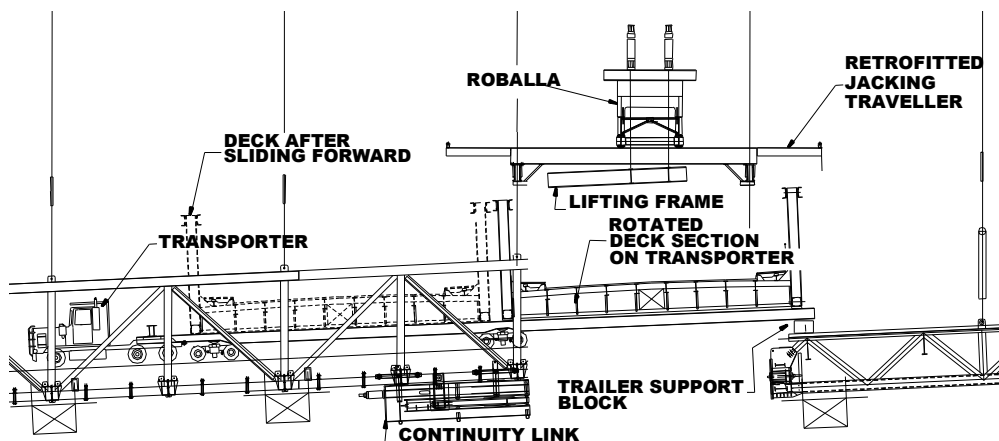
2.1.2 Severni i glavni raspon mosta

Severni raspon mosta, sadrži 10 panela i lociran je iznad zemlje. Glavni raspon sadrži 25 panela iznad vode. Panel sekcije su fabrikovane u dužini od 20 m izuzev kraćih sekcija na kraju svakog raspona. Postojeća konstrukcija je sečena, spštana kao što je prikazano na sl.3.1 i odvožena a nova sekcija je podizana i komponovana u novi most.

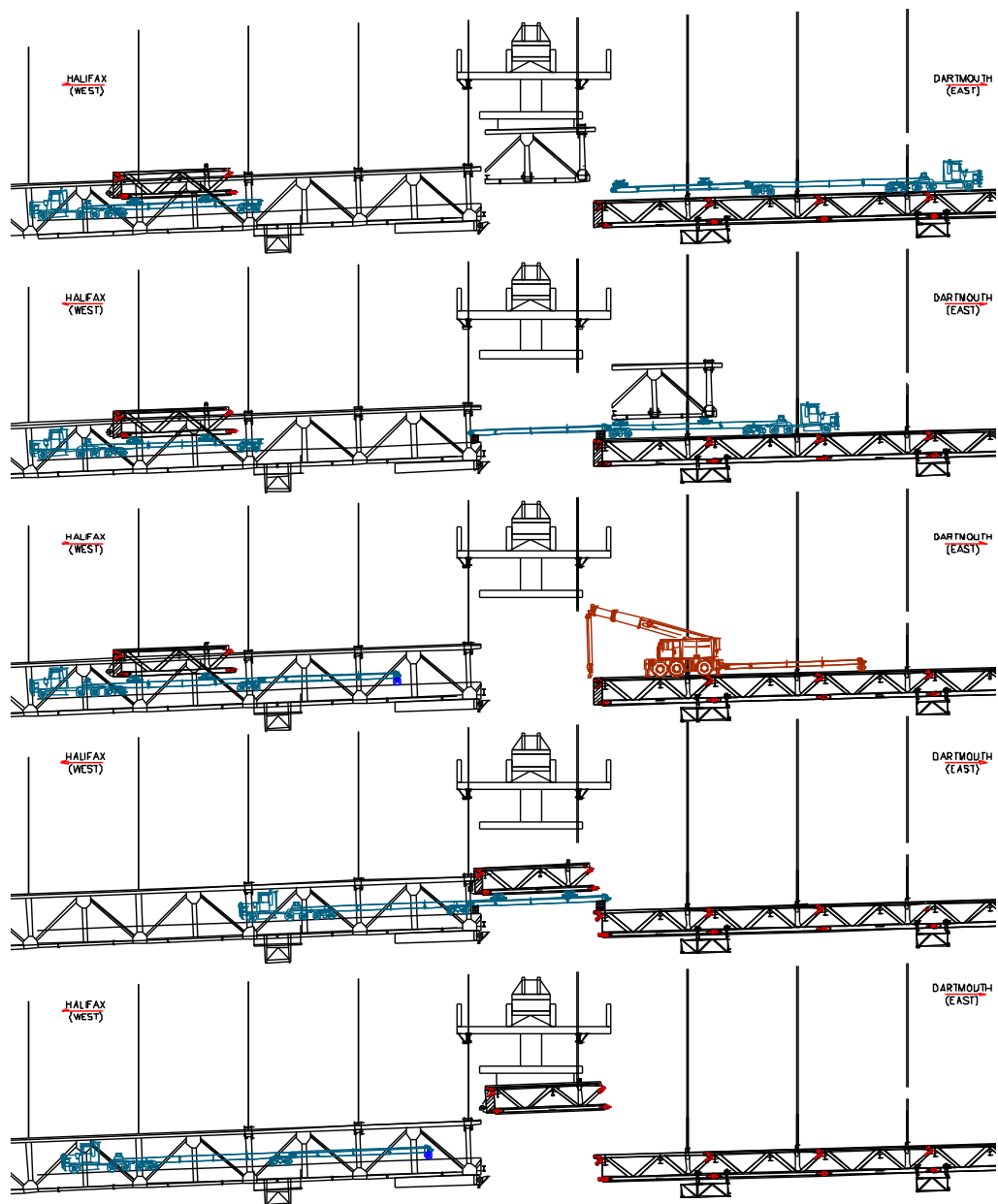
2.1.3 Južni raspon zahteva različitu šemu izvođenja

Južni raspon je loaciran preko vrlo strmog i ekološki senzitivnog zemljišta Stanley parka. Da bi se izbegao finansijski rizik u vezi sa ekološkim problemom, paneli južne strane mosta su fabrikovani u sekcijama po 10m dužine. Ovim se redukovala težina panela i povećala fleksibilnost za izvođenje. Izvođač je razmatrao više rešenja i konačno je odabrao metodu prikazanu na sl.2.5

Posle sečenja, podizanja i rotiranja postojeća sekcija mosta je utovarivana na transporter i odvožena u pravcu južnog kraja mosta. Sekcija je sečena sa hidrauličkim makazama a isečeni komadi su utovarivani u kamion za dalji transport čime se oslobađao prostor za jutarnji saobraćaj. Na severnoj strani mosta transporter natovaren novom sekcijom mosta čekao je spreman odvoženje postojeće sekcije. Nova sekcija je transportovana preko novog prethodno urađenog dela most do radnog fronta. Nova sekcija je tada podizana, rotirana i montirana u finalnu poziciju.



Sl. 2.5: Zona zamene segmenta na južnom rasponu – Transporter šema



Sl. 2.6: Zona zamene segmenta na južnom rasponu – Tehnologija izvođenja

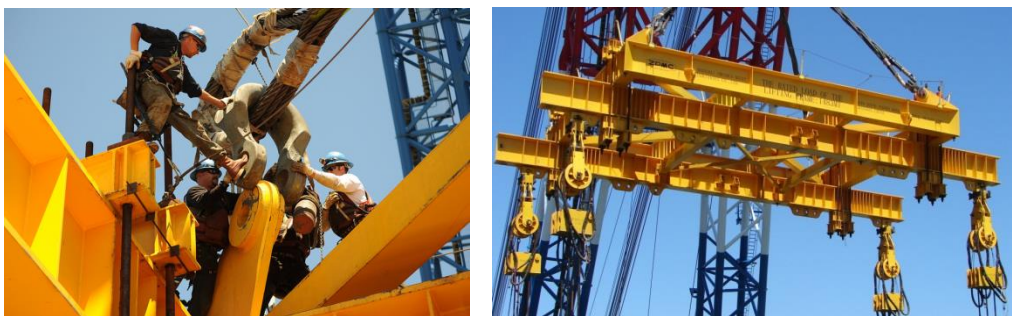
3 SAS BAY MOST, SAN FRANCISCO, UREĐAJI ZA MONTAŽU MOSTA



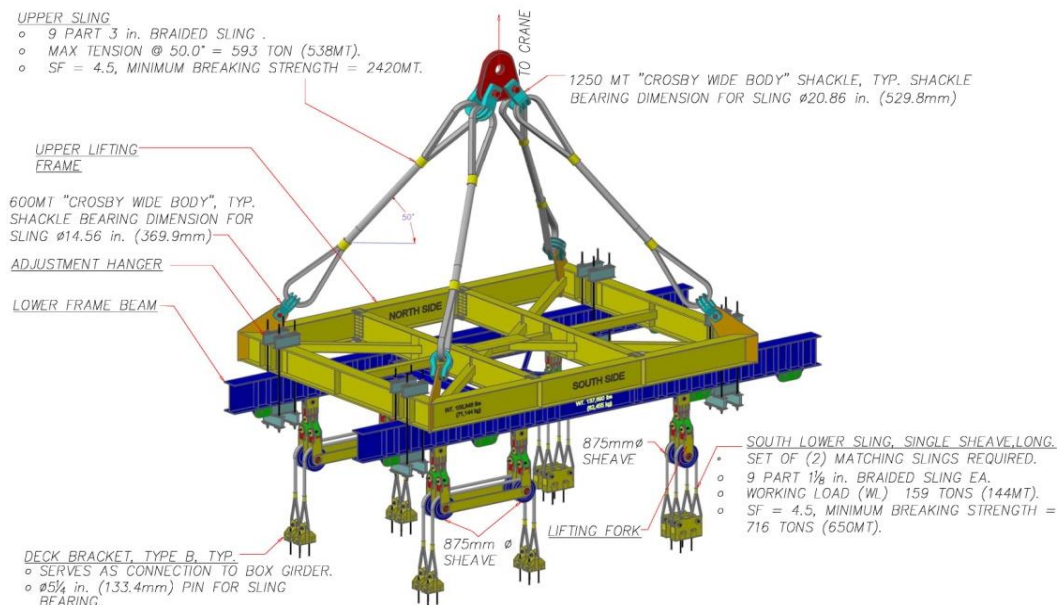
Slika 3.1 – Dispozicija SAS Bay Mosta

3.1 RAM ZA DIZANJE TERETA

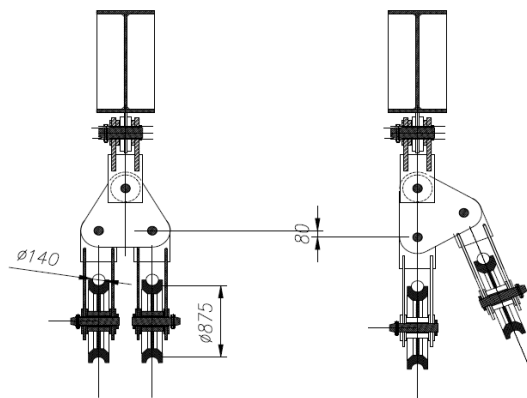
Po planu montaže nekoliko sandučastih mostovskih segmenata je trebalo da bude podignuto sa barže na privremenu rešetkastu skelu 230 m daleko od finalne pozicije. Čelični sandučasti montažni segmenti su podizani na prethodno pripremljene “saonice” (cradle). Svaki mostovski segment ima različitu poziciju težišta te je bilo potrebno različito prihvatanje elemenata u cilju sigurnosti podizanja kranom i da bi se izbeglo nepoželjno opterećenje segmenata u toku dizanja. U tom cilju je projektovan ram za dizanje koji je mogao da se prilagodi za svih 28 različitih segmenata mosta. Ram za dizanje (“Lifting Frame”) su u stvari dva rama, donji fiksni i gornji pokretni, koji svojim međusobnim položajem mogu da se prilagode bilo kojem mostovskom segmentu. Kada su se donji i gornji ram pozicionirali u odgovarajući projektovani položaj spajali su se u jednu celinu u više tačaka sa po četiri šipke za utezanje prečnika 64 mm.



Slika 3.2 - Aranžman i detalj rama za dizanje mostovskih segmenata



Slika 3.3 - Dispozicija rama za dizanje mostovskih segmenata

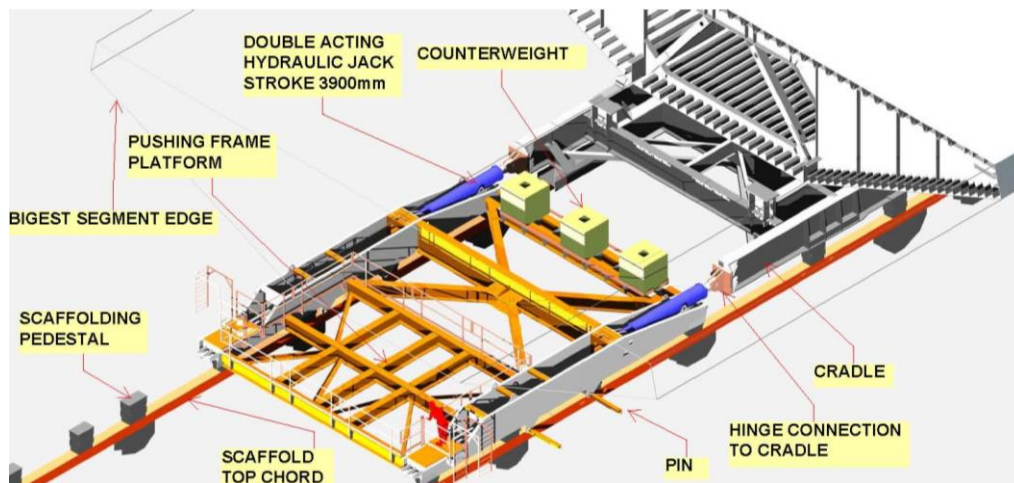


Slika 3.4 – Simulacija prekida jednog kabla za dizanje Slika 3.5 – Segment dužine 70m (1500t)

Četiri šipke za utezanje, na mestu svakog spojnog čvora, su se utezale istovremeno sa silom 126 tona po šipki. Donji ram je projektovan sa više pozicija za kačenje segmenata mosta. Segment mosta se kačio za donji ram preko čeličnog sklopa ("load equalizers sheave forks") koji je izjednačavao sile u sajlama za dizanje u podužnom i poprečnom pravcu. Za dizanje su korišćene sajle sa 9 upredenih čeličnih kablova od po 29 mm. Radi predostrožnosti kompletan sistem je analiziran za slučaj prekida jednog kabla za dizanje (Slika 4).

3.2 RAM ZA GURANJE SEGMENTATA (PUSHING FRAME)

Bay most je delimično lociran na vodi a delimično na zemlji. To je razlog da svi elementi ne mogu biti instalirani u svojoj konačnoj poziciji korišćenjem samo kрана na barži. Svaki element mosta je podignut na specijalno pripremljenu bazu ("saonice") sa kranom na barži, a onda su elementi zajedno sa saonicama gurani u njihovu konačnu poziciju. Prvi zadatak pri realizaciji projekta guranja segmenata na SAS Bay mostu u San Francisku je bio osmisлити što jednostavniji metod koji nije podložan devijacijama za vreme guranja segmenata. Radi bolje kontrole guranja segmenata inicijalna koncepcija korišćenja Hilman Roller-a je promenjena sa teflonskim klizačima. U tu svrhu projektovan je specijalni hidraulički ram za guranje segmenata mosta (pushing frame) koji je bio zakačen na saonice i gurao saonice zajedno sa mostovskim segmentom preko kratkih stubova privremene rešetke locirane u čvorovima gornjeg pojasa. Čelični hidraulički ram za guranje segmenata težine 100 tona sa dizel motorom od 100 konjskih snaga je upravljан sa jednim radnikom na bazi informacija od nekoliko radnika koji su kontrolisali klizanje u nekoliko kritičnih tačaka. Kompletan proces guranja projektovan je sa tolerancijom za guranje od 150 mm.



Slika 3.6 – Šematski prikaz uređaja za guranje

Klizanje je obavljeno preko specijalno projektovanih teflonskih ležišta lociranih na kratkim stubovima rešetke. U cilju smanjenja trenja teflonska ležišta su premazivana sintetičkim uljem, a saonice su imale zavaren ispolirani tanki lim od nerđajućeg čelika na donjoj flanši koja je bila u kontaktu sa ležištima. U toku guranja zabeleženo je maksimalno trenje od 5%.

Za klizanje su korišćene dve hidrauličke dizalice sa maksimalnim pomeranjem klipa od 3.9 m. Pre guranja hidraulički ram je osiguran sa dve bolcne 76x150x1220 mm, oslonjene u horizontalnom smislu na kratke stubove privremene rešetke. Projektovane su ukupno četiri bolcne ali za pomeranje su korišćene samo dve uparene bolcne, dve prednje ili dve zadnje, zavisno od pozicije kratkih stubova na privremenoj rešetki. Kada hidraulička dizalica gurne mostovski element sa postoljem za klizanje otprilike 3.9 m bolcne su se vadile da bi se hidrauličke dizalice skupile. Posle ovoga hidraulički ram za guranje bi se pripremio za novi ciklus guranja instaliranjem dve bolcne. Ovaj proces se ponavljao sve dok mostovski segment

ne dodje do otprilike 1 m od svoje konačne lokacije. U toj poziciji hidrauličke dizalice su se odvajale od rama za klizanje i kompletan ram je povlačen nazad korišćenjem dva vitla.

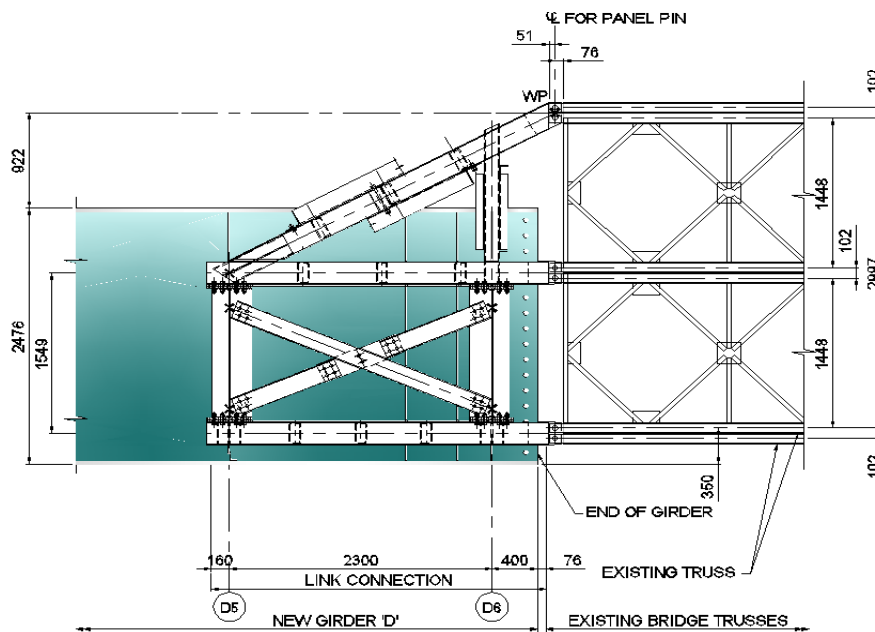
4 KIKOMAN MOST, BC, KANADA



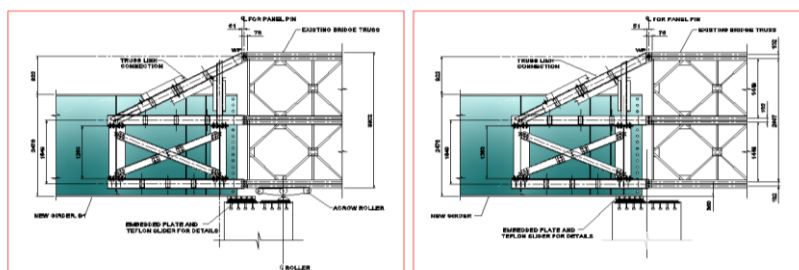
Slika 4.1 Proces demontaže/montaže Kikoman mosta

Postojeći “bailely” most ukupne dužine 200 m sa četiri raspona od po 50 m je trebalo zameniti sa novim sastavljenim od dve grede visine 2.4 m (vidi Sl. 4.2) Ovde je primenjena originalna metoda koja je omogućila istovremenu demontažu i montažu mosta. Na južnoj strani mosta projektovan je kruti (momentni) nastavak između postojećeg bailely mosta i novog grednog mosta (vidi Sl. 4.2 i 4.3). Primenom krute veze eliminisan je problem sa velikim deformacijama u toku navlačenja, te je most mogao da se montira inkrementalnim navlačenjem (incremental launching) bez upotrebe “nosa” za navlačenje. Idejnim projektom je bilo predviđeno da se postojeći most demontira zajedno sa drvenom kolovoznom konstrukcijom ali se na kraju odustalo jer je proračun pokazao da bi došlo do prekoračenja napona u donjem pojasu rešetke postojećeg mosta. Prilikom demontaže-montaže mosta korišćena su dva sistema oslanjanja na stubovima. Postojeći beily most se demontirao oslanjajući se na valjkaste oslonce “rocking rollers” a novi most se klizao preko teflonske ploče premazane tečnim sapunom.

Korišćenjem ove metode izvođenje je pojednostavljeno što je omogućilo da se glavna konstrukcija mosta demontira i montira u rekordnom roku od 17 dana sa grupom od 12 radnika.



Slika 4.2 Kontinualna veza između postojećeg i novog mosta



Slika 4.3 Šema oslonaca u toku izvođenja



Slika 4.4 Montaža kontinualne veze

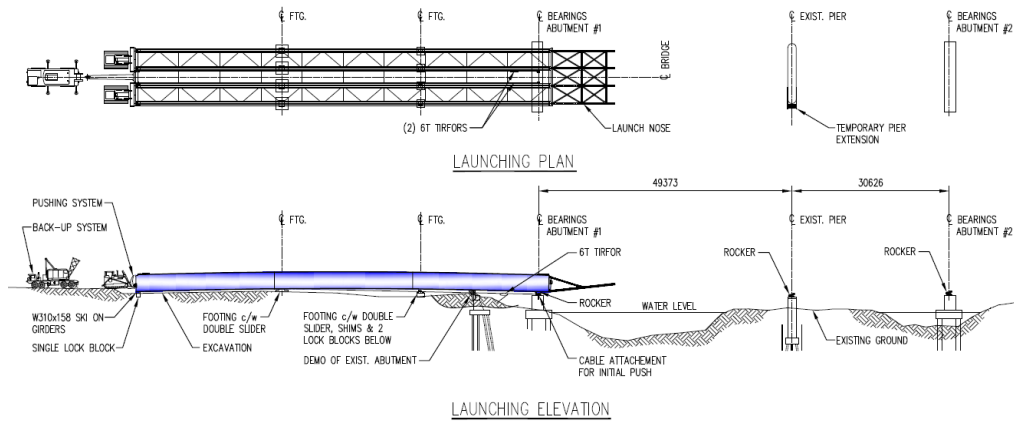
5 WALTERDALE MOST EDMONTON, KANADA

Kada bude završen, novi Walterdale most će sačinjavati dva čelična luka koja će premostiti Saskatchewan reku u centralnom delu Edmontona, AB, Kanada

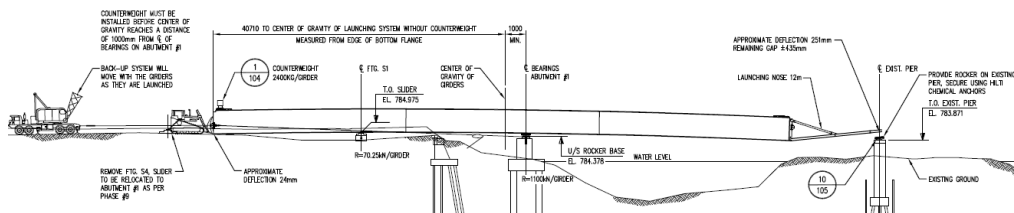
Za montažu lukova je izabrana veoma kompleksna operacija. Centralni deo lukova, približno 1/3 ukupne dužine lukova, je montiran na zemlji u samoj blizini mosta. Po montiranju, ovaj deo, težine 1000 t, klizan je do reke i posebnom operacijom teret je prebačen na dve barže. Segment je sa barža podignut putem hidrauličkih džekova u poziciju za spajanje sa prethodno pripremljenom konstrukcijom luka u dve faze. Allnorth je takođe razvio tehnologiju izvođenja kolovozne konstrukcije i konačnog podešavanja kablova zatega. I ovaj deo projekta kao i prethodni dio je interesantna tehnička rešenja. Mnogo više podataka o izvođenju ovog mosta se može naći u radu „Walterdale most, Edmonton, Kanada“



Sl.5.1 Montaža prvog 88.0 m lučnog segmenta (1000 t)



Slika 6.3- Šema sistema za navlačenje



Slika 6.4 – Faza navlačenja u momentu kada “ nos” pridje stubu u vodi



Slika 6.5 – Dva buldožera D8N su korišćena za guranje mosta



Ivan Ivanchev¹

**EKSPERIMENTALNO ODREĐIVANJE SREDNJE
DILATACIJE ZATEGNERUTE ARMATURE. SADEJSTVA
ZATEGNETOG BETONA I MOMENTA NASTANKA
PRSLINE ZA AB ELEMENTE**

Rezime:

S-1 U radu su predstavljeni eksperimentalni rezultati određivanja srednje dilatacije zatezanja na nivou težišta zategnute armature armiranobetonskih elemenata opterećenih na savijanje. Dvanaest armiranobetonskih greda je podvrgnuto testu savijanja sa opterećenjem u 4 tačke. Grede se razlikuju po prečniku podužne armature, procentu armiranja i veličini zaštitnog sloja betona. Veličina sadejstva zategnutog betona je određena za svaku gredu pri različitim nivoima savijanja. Za sve ispitivane grede je određen moment nastanka prsline.

Кljučне речи: eksperiment, AB, srednja dilatacija, sadejstvo, moment prsline

**EXPERIMENTAL DETERMINATION OF THE MEAN
TENSILE STRAIN OF THE REINFORCEMENT, TENSION
STIFFENING AND BENDING CRACKING MOMENT FOR RC
ELEMENTS**

Summary:

In this paper are described experimentally determined results for the mean tensile strain of the reinforcement at level of its centroid for reinforced concrete elements subjected to bending. Twelve reinforced concrete beams were subjected on 4-point bending test. They differ in the diameter of the longitudinal reinforcement, reinforcement ratio and concrete cover. The value of the tension stiffening was determined for each of the beams for different values of the bending moment. For each of the beams is determined the bending cracking moment.

Key words: experiment, RC, mean tensile strain of reinforcement, tension stiffening, cracking moment

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1 INTRODUCTION. MEAN TENSILE STRAIN OF THE REINFORCEMENT AND TENSION STIFFENING. CRACKING MOMENT

For reinforced concrete elements subjected to bending the tensile force in cracked section is taken only by the reinforcement. In the middle between two cracks the tensile force in concrete is greatest due to bond when bending moments in cracked sections are the same, while in cracked section the tensile force in concrete is zero. For this reason the tensile force in reinforcement varies between the cracks and is greatest in cracked section.

After formation of cracks there is slip of reinforcement relative to concrete. The strains in the concrete and in the reinforcement outside the anchor lengths are the same.

The mean strain in the reinforcement ε_{sm} with increasing the tensile force is shown on figure 1 [1], where "0" is the strain of the bare reinforcing bar, „I" is mean strain in the reinforcement in phase before the formation of the first crack, „II" is mean strain in the reinforcement in crack formation phase, „III" is mean strain in the reinforcement in stabilized cracking stage, „IV" is mean strain in the reinforcement at yield.

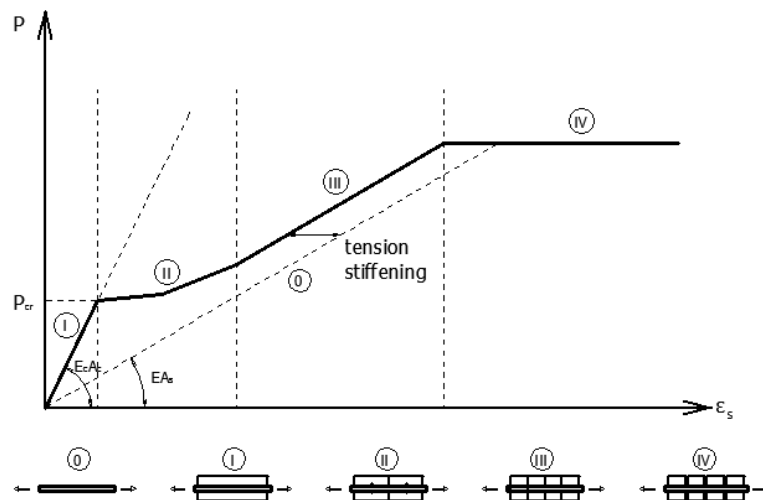


Figure 1 - Mean strain of the reinforcement with increasing the load.
Influence of tension stiffening

Tension stiffening $\Delta\varepsilon_{sm}$ can be described as the difference between the strain of the reinforcement in cracked section ε_{s2} and the mean strain in the tensile reinforcement ε_{sm} :

$$\square \varepsilon_{sm} = \varepsilon_{s2} - \varepsilon_{sm} \quad (1)$$

Bending cracking moments M_{cr} according to EC2 at the days of the tests of specimens of beams is determined by:

$$M_{cr} = f_{ct} \frac{I_I}{h - x_I} \text{ [kNm]} \quad (2)$$

where:

f_{ct} - experimentally determined axial tensile strength of the concrete at the day of testing;

I_I - moment of inertia of the adduced section calculated for its centre of gravity;

x_I - height of compression zone of the adduced section before forming the first crack.

2 EXPERIMENTAL SETUP

For experimental determination of mean strain of the tensile reinforcement at the level of its centre of gravity and tension stiffening were produced 12 reinforced concrete beams – 4 specimens with 3 beams in each one (Table 1). The specimens differ in provided longitudinal reinforcement, concrete cover and reinforcement ratio. The spans and width of cross sections are equal for all the beams [2], [3].

Table 1 – Parameters of reinforced concrete beams

Beam	-	A1	A2	A3	B1	B2	B3	C1	C2	C3	D1	D2	D3
Bottom reinforcement	-	2N12	2N12	2N12	2N18	2N18	2N18	2N12	2N12	2N12	2N18	2N18	2N18
Reinforcement ratio	-	0,0064	0,0064	0,0064	0,0145	0,0145	0,0145	0,0064	0,0064	0,0064	0,0145	0,0145	0,0145
Concrete cover [mm]		28	28	28	28	28	28	58	58	58	58	58	58

Testing scheme of the specimens is simply supported beam with two concentrated forces in the thirds (4 point bending test). The load is static and is performed by using a hydraulic compression press. The force of the hydraulic cylinder is divided into two equal parts which are applied symmetrically relative to the centre of the beam by using the traverse. The distance between the two applied forces is 100 cm. The supports of the beam are on axial distance 300cm. The load is carried out in stages, different for each of the specimens corresponding to their bearing capacity. For specimens A and C the force for each point at different loading stages is: 1 kN, 2 kN, 3 kN, 4 kN, 5 kN, 6 kN, 7 kN, 8 kN, 9 kN, 10,5 kN, 12 kN, 13,5 kN, 15 kN, 17 kN, 19 kN, 21 kN, 23 kN, 25 kN and 27 kN. For specimens B and D the force for each point at different loading stages is: 1 kN, 2 kN, 3 kN, 4 kN, 5 kN, 6 kN, 7 kN, 8 kN, 10,5 kN, 13,5 kN, 17 kN, 21 kN, 25 kN, 29 kN, 33 kN, 37 kN, 41 kN, 45 kN, 49 kN, 53 kN and 57 kN.

For measuring the pressure acting on the piston of the press, by which the load on the reinforced concrete beams is performed pressure sensor type P3MA of the company Hottinger Baldwin Messtechnik GmbH (HBM) with parameters 500 Bar and 2 mV/V was used.

For determining the cracking moment the following transducers located in the middle third of the beam were used (figure 2a):

- 3 inductive displacement transducers for each of the side surfaces in the tensile zone (figure 2b);
- 7 strain gauges for each of the side surfaces in the compression zone (figure 2c);
- 6 strain gauges on the top compressive surface of the beams (figure 2c);
- 3 strain gauges located on tensile bottom surface of the beams (figure 2d).

For determining the mean tensile strain of reinforcement the two inductive displacement transducers located on the level of center of gravity of tensile reinforcement were used.

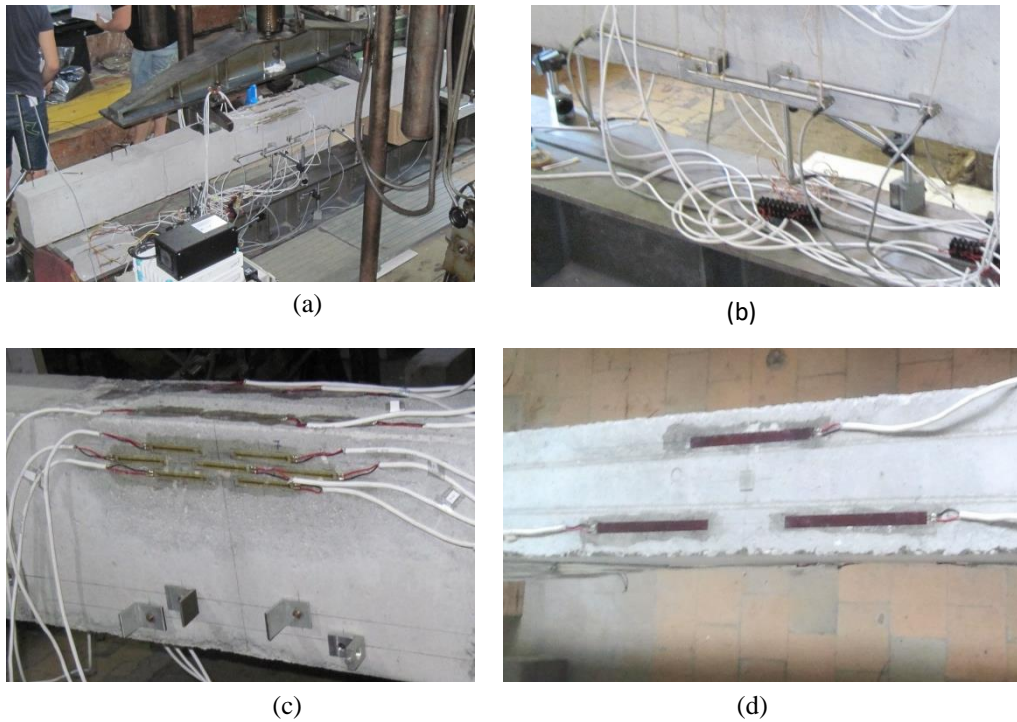


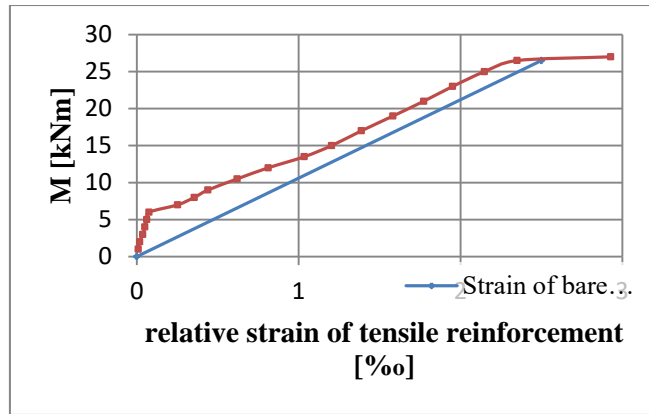
Figure 2 - Positioning of the transducers

3 EXPERIMENTAL DETERMINATION OF MEAN TENSILE STRAIN OF THE REINFORCEMENT, TENSION STIFFENING AND CRACKING MOMENT

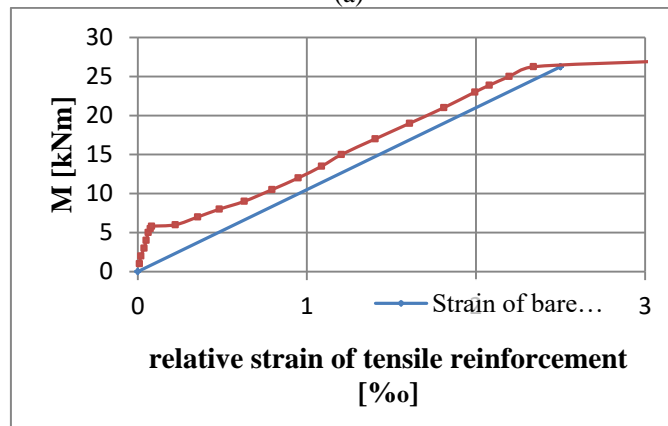
Experimentally obtained dependencies for the mean tensile strain of reinforcement at the level of its centroid in ‰ (mm/m) in function of the bending moment in kNm in the part with constant bending moment for specimens A, B, C and D are shown on figure 3, 4, 5 and 6 with red color. The relative tensile strain of bare reinforcement bar is shown with blue line.

Clearly defined are the parts of ε_{sm} in the phase before forming the first crack, crack formation phase, stabilized cracking phase and at yield of reinforcement.

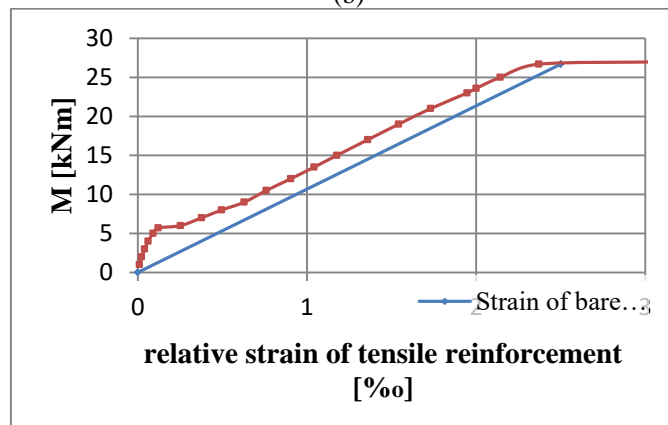
The values of tension stiffening before formation of the first crack for different beams are shown on Table 2. With increasing ϕ / ρ_{eff} ratio increases the value of the tension stiffening. For beams with similar values of ϕ / ρ_{eff} ratio bigger value of tension stiffening have those with bigger concrete cover.



(a)

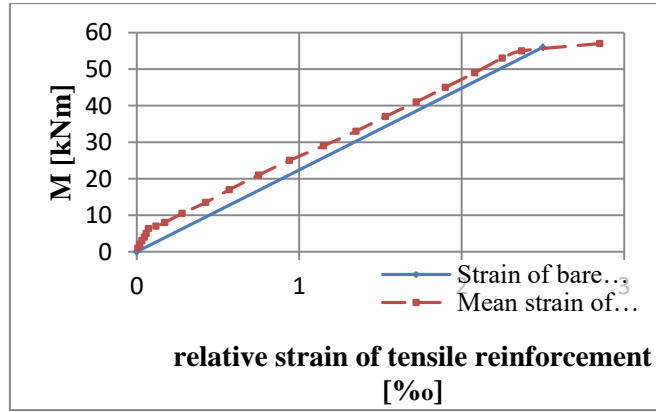


(b)

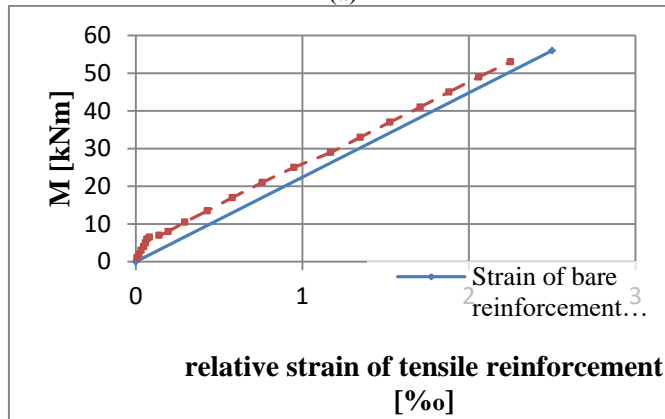


(c)

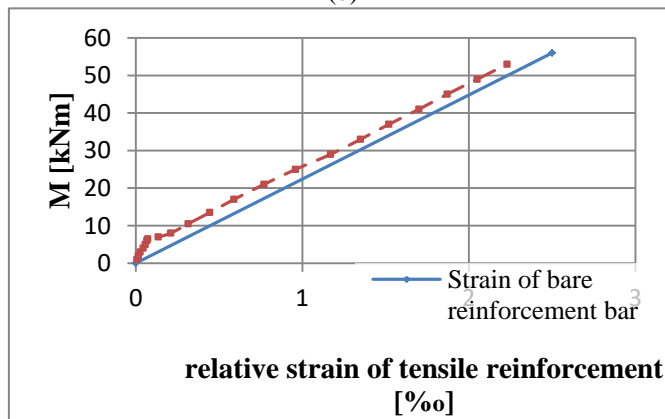
Figure 3 - Mean tensile strain of reinforcement in function of the bending moment for beam A1(a), A2(b) and A3 (c). Influence of tension stiffening



(a)

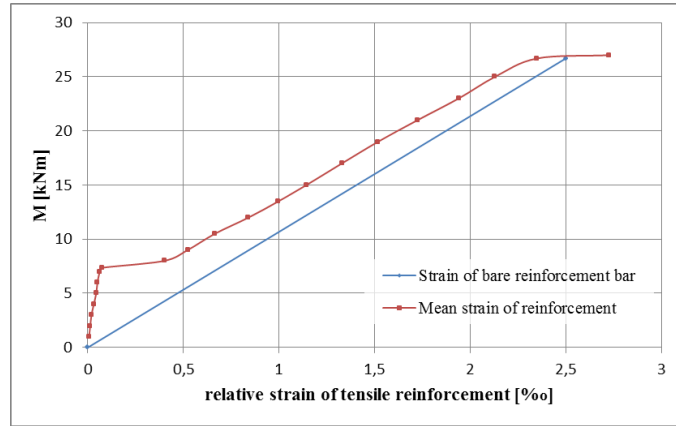


(b)

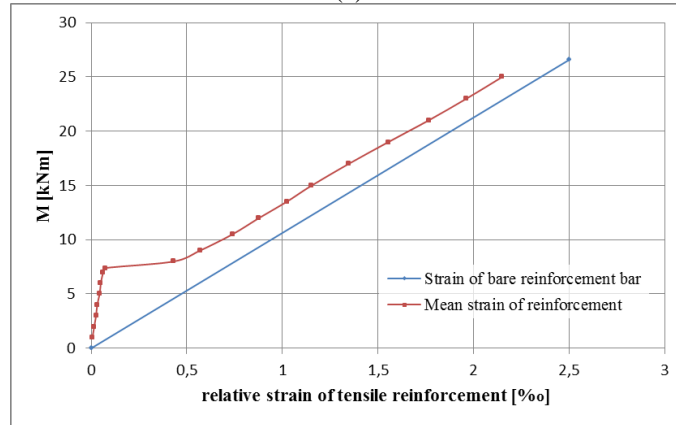


(c)

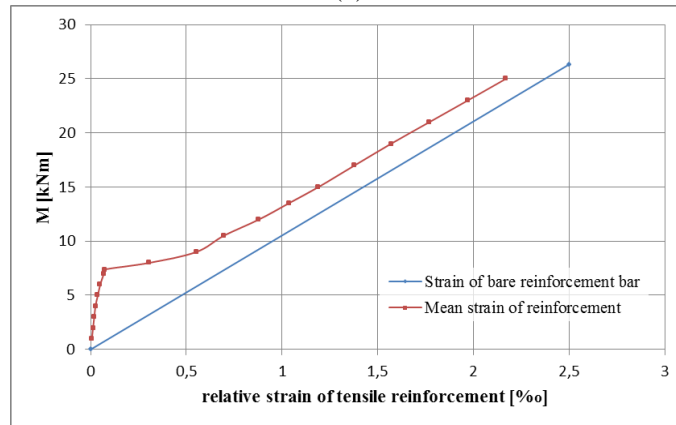
Figure 4 - Mean tensile strain of reinforcement in function of the bending moment for beam B1(a), B2(b) and B3 (c). Influence of tension stiffening



(a)

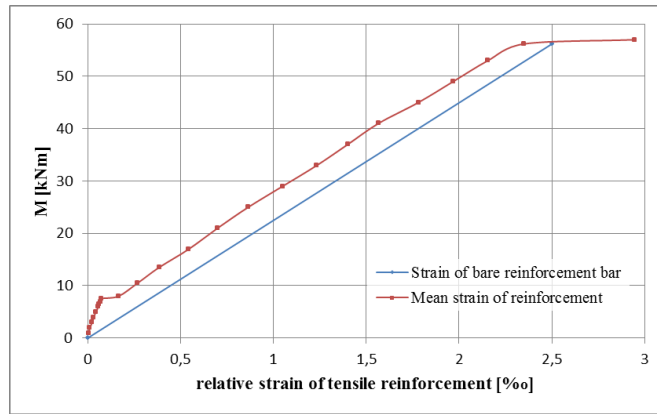


(b)

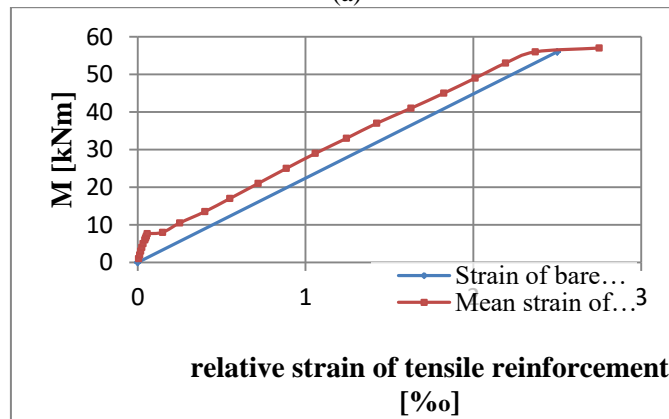


(c)

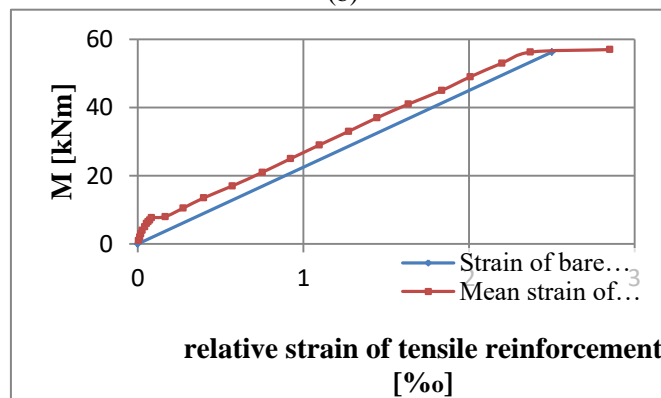
Figure 5 - Mean tensile strain of reinforcement in function of the bending moment for beam C1(a), C2(b) and C3 (c). Influence of tension stiffening



(a)



(b)



(c)

Figure 6 - Mean tensile strain of reinforcement in function of the bending moment for beam D1(a), D2(b) and D3 (c). Influence of tension stiffening

Table 2 - Experimentally determined values of tension stiffening before formatting the first crack

Beam	A1	A2	A3	B1	B2	B3	C1	C2	C3	D1	D2	D3
Tension stiffening [% _o]	0,52	0,51	0,46	0,23	0,23	0,23	0,63	0,63	0,63	0,28	0,29	0,27

From the record for the dependence of force in time (figure 7), the results of strain gauges and inductive displacement transducers experimentally were determined cracking moments of the beams.

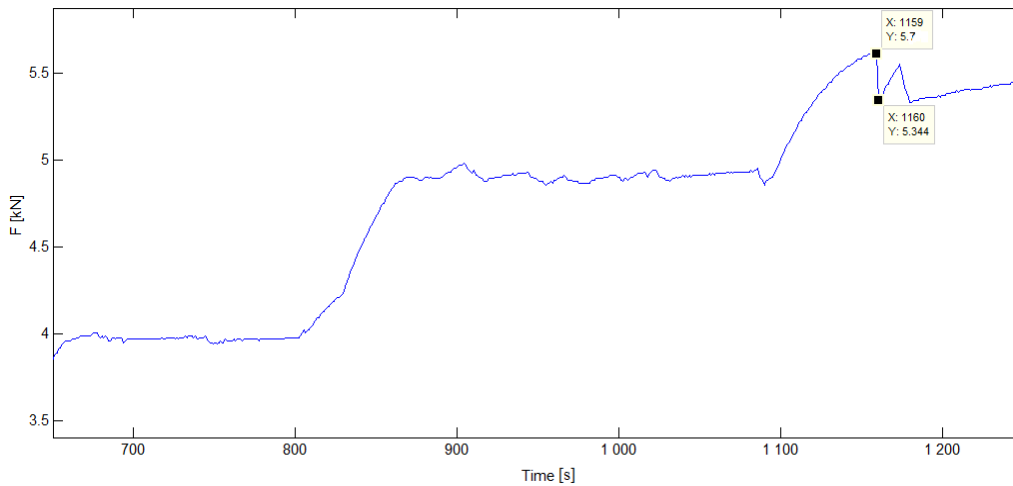


Figure 7 - Registering formation of the first crack

Calculated by (2) and experimentally obtained bending cracking moments and the difference between them in % are given in Table 3.

Table 3 - Bending cracking moment – experimentally determined and according to EC2

Beam	-	A1	A2	A3	B1	B2	B3	C1	C2	C3	D1	D2	D3
Experimental M_{cr}	[kNm]	6,10	5,80	5,70	6,35	6,70	6,60	7,36	7,39	7,38	7,55	7,63	7,70
EC2 M_{cr}	[kNm]	5,85	5,85	5,85	6,58	6,58	6,58	7,20	7,20	7,20	7,58	7,58	7,58
Difference	[%]	4,27	0,85	2,56	3,50	1,82	0,30	2,22	2,64	2,50	0,40	0,66	1,58

The differences between the calculated and experimentally obtained values of bending cracking moments are very small, ranging from 0.3% to 4.27%.

It is experimentally confirmed that:

- for beams with the same reinforcement ratio, similar values of ϕ / ρ_{eff} ratio and different concrete cover the value of cracking moment is bigger for bigger concrete cover;

- for beams with the same concrete cover, different reinforcement ratio and significantly different values of ϕ / ρ_{eff} ratio the value of cracking moment is bigger for bigger reinforcement ratio;

- biggest values of cracking moment were obtained for beams D1, D2 and D3, which have the bigger reinforcement ratio ($\rho = 0,0145$) and bigger concrete cover (58 mm);

- smallest values of cracking moment were obtained for beams A1, A2 and A3, which have the smaller reinforcement ratio ($\rho = 0,0064$) and smaller concrete cover (28 mm).

At the end of each of the experiments 8 of the 12 beams were loaded to yield of longitudinal reinforcement (A1, A2, A3, B1, C1, D1, D2 and D3) and the remaining 4 (B2, B3, C2 and C3) were left for further researches. Yield of reinforcement for different beams occurs at mean tensile strain varying from 2,34 ‰ to 2,37‰.

4 CONCLUSION

From the record for the dependence of force over time, the results of strain gauges and inductive displacement transducers were determined bending cracking moments in the middle part for each of the 12 beams. It is confirmed that with the increase of the reinforcement ratio and concrete cover increases the value of cracking moment and it is confirmed the validity of the proposed formula for its calculation in EC2.

For all tested specimens was observed typical breakage in the dependence bending moment-strain at appearance of cracks. Upon further increasing the load graphics become linear and this is direct evidence for completing the process of cracking.

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Ivan Ivanchev¹

ISTRAŽIVANJE UTICAJA VELIČINE ZAŠTITNOG SLOJA NA ŠIRINU I RAZMAK PRSLINA KOD AB ELEMENATA IZLOŽENIH SAVIJANJU

Rezime:

S-2

U radu je prikazano eksperimentalno istraživanje uticaja debljine zaštitnog sloja betona na maksimalnu i srednju vrednost širine i razmaka prslina kod armiranobetonskih elemenata izloženih savijanju.

Ključne reči: eksperiment, AB, širina prslina, razmak prslina, zaštitni sloj

INVESTIGATION OF THE INFLUENCE OF CONCRETE COVER ON THE CRACK WIDTH AND SPACING FOR REINFORCED CONCRETE ELEMENTS SUBJECTED TO BENDING

Summary:

In this paper the influence of concrete cover of the longitudinal reinforcement on maximum and mean crack width and maximum and mean crack spacing for reinforced concrete elements subjected to bending is investigated experimentally

Key words: experiment, RC, crack width, crack spacing, concrete cover

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1 INTRODUCTION

According to Broms [1], Ferry Borges [2], Welch [3], Leonhardt [4], Beeby [5] and other researchers, building codes - EC2 [6] and MC2010 [7] concrete cover of longitudinal reinforcement influences crack width and spacing in reinforced concrete elements subjected to bending.

Beeby [5] states that when the concrete reaches its tensile strength and hence the cracks are forming, it must be taken into account the stress lag near the cracks. Stress lag increases the crack spacing (figure 1).

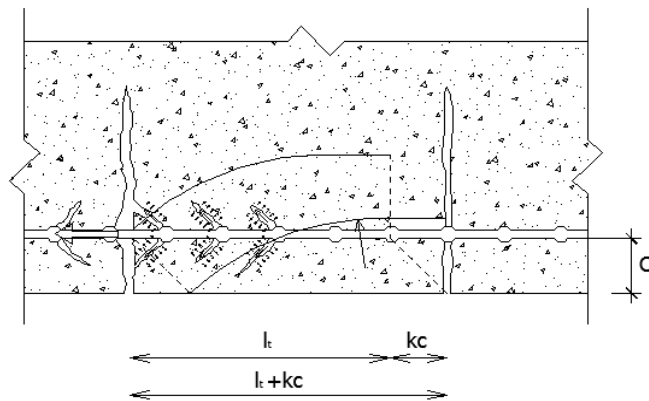


Figure 1 - Influence of concrete cover on crack spacing

Experimentally it is established the influence of concrete cover on crack width. This effect is known from the researches of Husein and Ferguson [8] – figure 2 and the researches of Borosnyoi and Snobli [9].

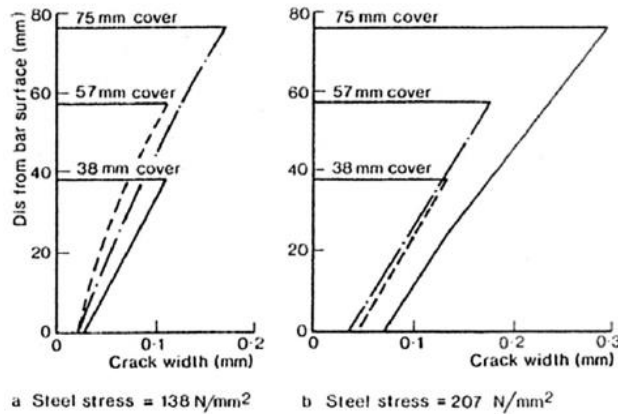


Figure 2 - Dependence of crack width of the distance from the center of gravity of the reinforcing bar in the concrete cover

For this reason, many building codes define the crack spacing as the sum of the part representing the two anchoring lengths and the other taking into account the influence of the concrete cover.

2 EXPERIMENTAL SETUP

For the experimental determination the influence of the concrete cover on crack width and spacing were produced 12 reinforced concrete beams – 4 specimens with 3 beams in each one (Table 1). The specimens differ in provided longitudinal reinforcement, concrete cover and reinforcement ratio. The spans and width of cross sections are equal for all the beams. The effective depths are equal to 23,6 cm for specimens A and C and equal to 23,3 cm for specimens B and D [10], [11].

Table 1 – Parameters of reinforced concrete beams

Beam	-	A1	A2	A3	B1	B2	B3	C1	C2	C3	D1	D2	D3
Bottom reinforcement	-	2N12	2N12	2N12	2N18	2N18	2N18	2N12	2N12	2N12	2N18	2N18	2N18
Reinforcement ratio	-	0,0064	0,0064	0,0064	0,0145	0,0145	0,0145	0,0064	0,0064	0,0064	0,0145	0,0145	0,0145
Concrete cover [mm]		28	28	28	28	28	28	58	58	58	58	58	58

Testing scheme of the specimens is simply supported beam with two concentrated forces in the thirds (4 point bending test). The load is static and is performed by using a hydraulic compression press. The force of the hydraulic cylinder is divided into two equal parts which are applied symmetrically relative to the centre of the beam by using the traverse. The distance between the two applied forces is 100 cm. The supports of the beam are on axial distance 300cm. The load is carried out in stages, different for each of the specimens corresponding to their bearing capacity. For specimens A and C the force for each point at different loading stages is: 1 kN, 2 kN, 3 kN, 4 kN, 5 kN, 6 kN, 7 kN, 8 kN, 9 kN, 10,5 kN, 12 kN, 13,5 kN, 15 kN, 17 kN, 19 kN, 21 kN, 23 kN, 25 kN and 27 kN. For specimens B and D the force for each point at different loading stages is: 1 kN, 2 kN, 3 kN, 4 kN, 5 kN, 6 kN, 7 kN, 8 kN, 10,5 kN, 13,5 kN, 17 kN, 21 kN, 25 kN, 29 kN, 33 kN, 37 kN, 41 kN, 45 kN, 49 kN, 53 kN and 57 kN.

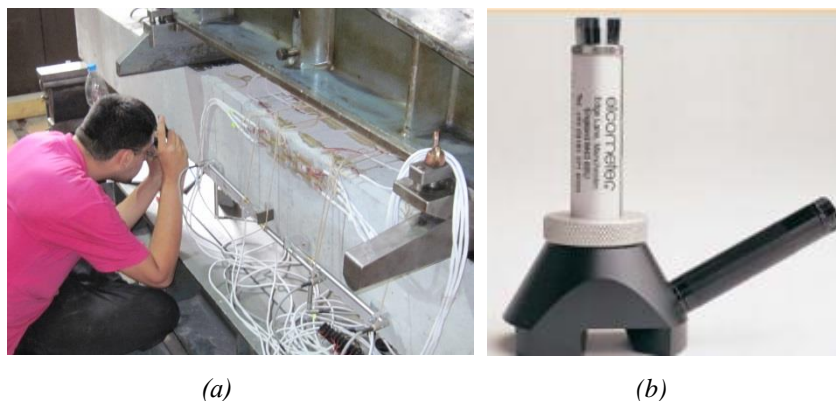


Figure 3 - (a) Observing and measuring the crack widths; (b) Microscope Elcometer 900

Cracks in concrete along the entire length of the beam were registered at their appearance and crack width and depth were monitored in the process of testing at each loading stage (figure 3 – a). This control is performed by using crack magnifier Proceq and microscope Elcometer 900 (figure 3 – b) with magnification respectively 8x and 100x, equipped with scales with accuracy respectively 0,1 mm and 0,01 mm.

During the experimental researches on the two side surfaces of the beams are outlined obtained cracks (figure 4 ÷ figure 7) in order of their appearance. Their depths are marked at each of loading stages to yield of the longitudinal tensile reinforcement (beams A1, A2, A3, B1, C1, D1, D2 and D3) or to the loading stage before yield of reinforcement (beams B2, B3, C2 and C3).

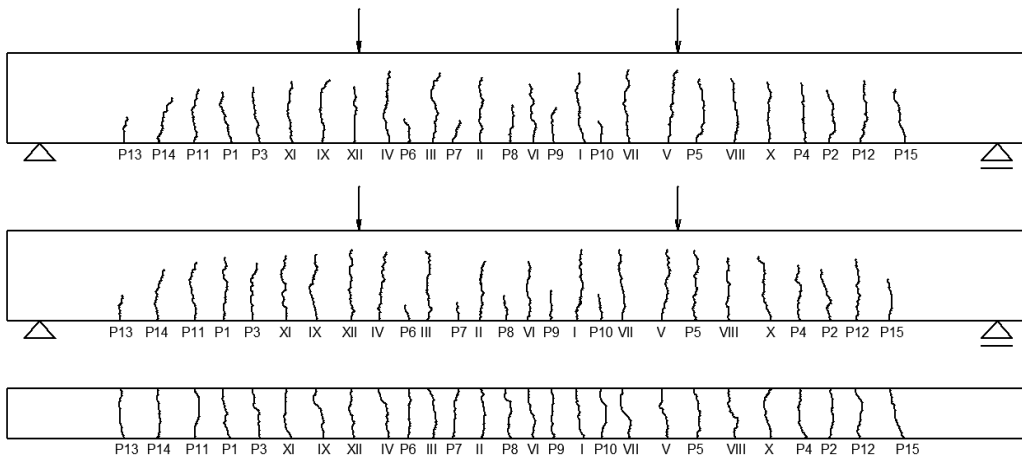


Figure 4 - Scheme of cracks location on the bottom and both side surfaces of beam A1

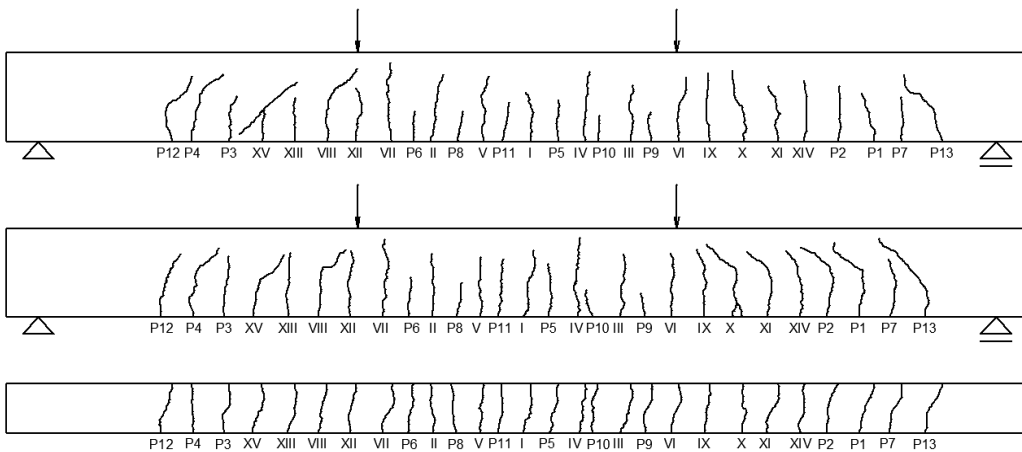


Figure 5 - Scheme of cracks location on the bottom and both side surfaces of beam B1

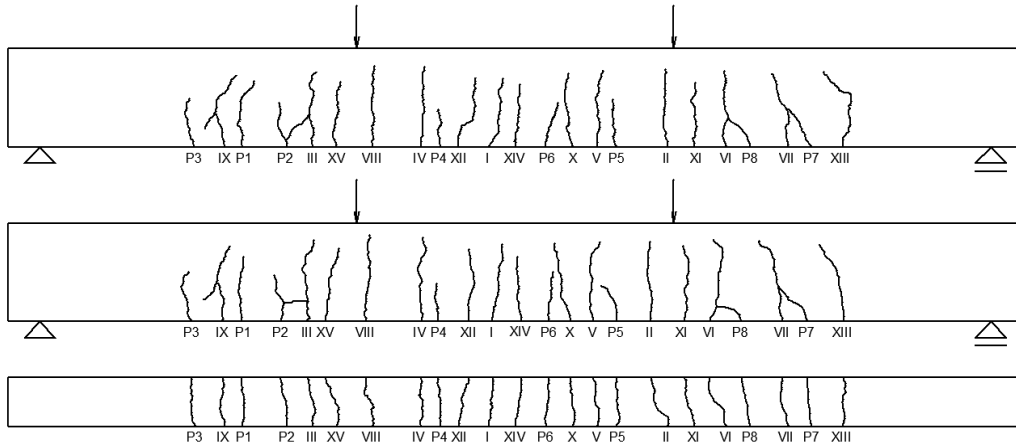


Figure 6 - Scheme of cracks location on the bottom and both side surfaces of beam C1

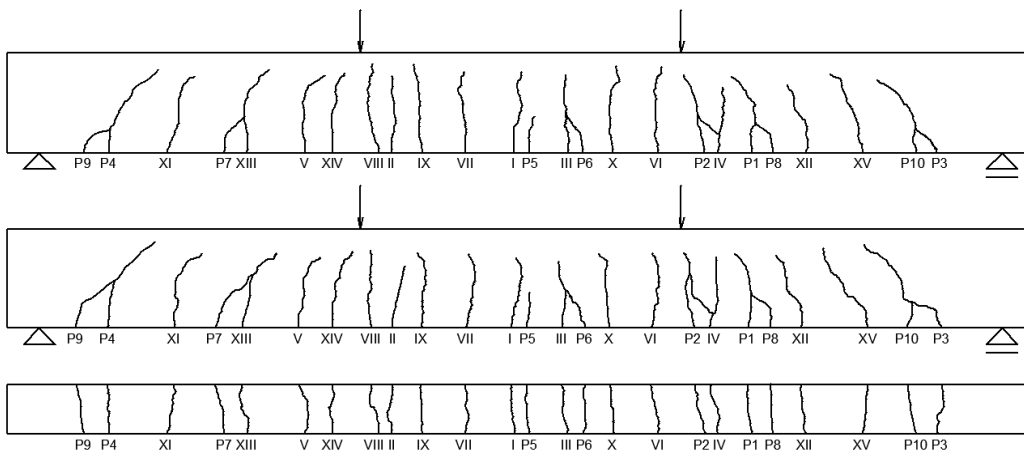


Figure 7 - Scheme of cracks location on the bottom and both side surfaces of beam D1

3 EXPERIMENTAL DETERMINATION THE INFLUENCE OF THE CONCRETE COVER ON CRACK WIDTH AND SPACING

Experimentally is determined the influence of the concrete cover on crack width and spacing for elements with different concrete cover, approximately equal values of ϕ / ρ_{eff} ratio and same values of the stress in tensile reinforcement at cracked section (σ_s).

The influence of the concrete cover on crack width in the part with constant bending moment is investigated separately for specimens A ($\phi / \rho_{eff} = 554,3mm$) and C ($\phi / \rho_{eff} = 634,6mm$) and specimens B ($\phi / \rho_{eff} = 329,5mm$) and D ($\phi / \rho_{eff} = 382,6mm$). This is due to close values of ϕ / ρ_{eff} ratio.

In this way the influence of the concrete cover on maximum and mean crack width can be estimated more accurately as the influence of ϕ / ρ_{eff} ratio can be neglected.

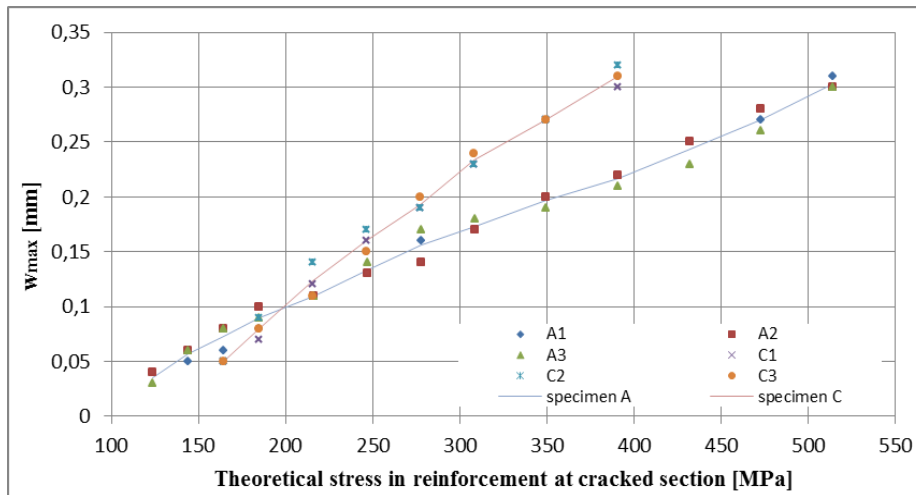


Figure 8 - Influence of the concrete cover on maximum crack width – specimens A and C

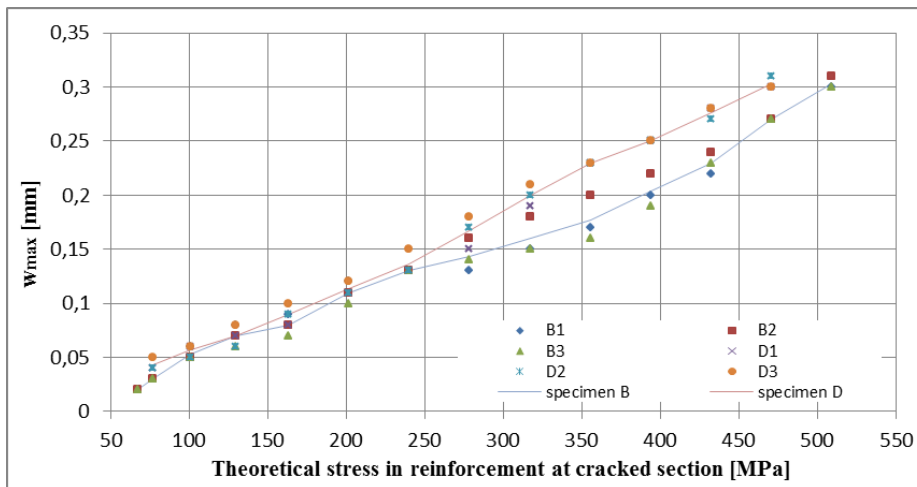


Figure 9 - Influence of the concrete cover on maximum crack width – specimens B and D

Experimentally determined crack width (figure 8 ÷ figure 11) is shown as a function of the theoretical stress in reinforcement at cracked section, which is calculated for each of the different loading stages. The stresses in the tensile reinforcement at cracked section of elements in the building structures usually exceed 200 MPa at quasi permanent and characteristic combination.

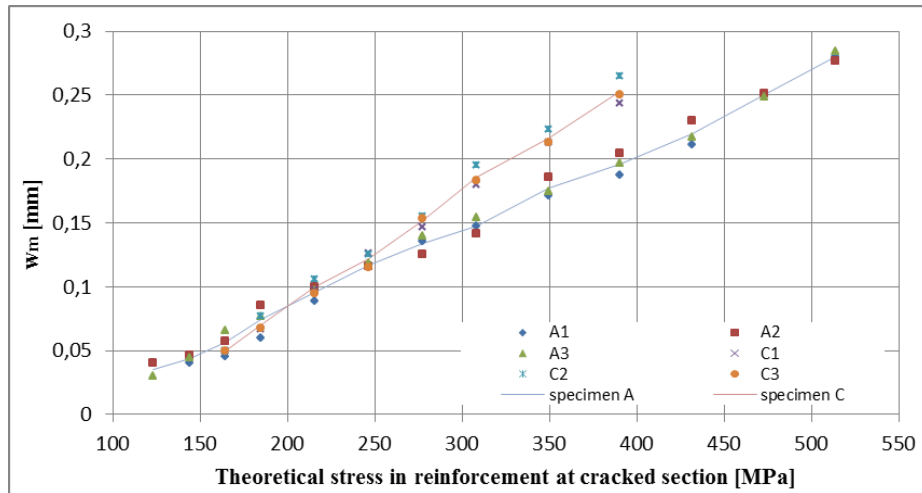


Figure 10 - Influence of the concrete cover on mean crack width – specimens A and C

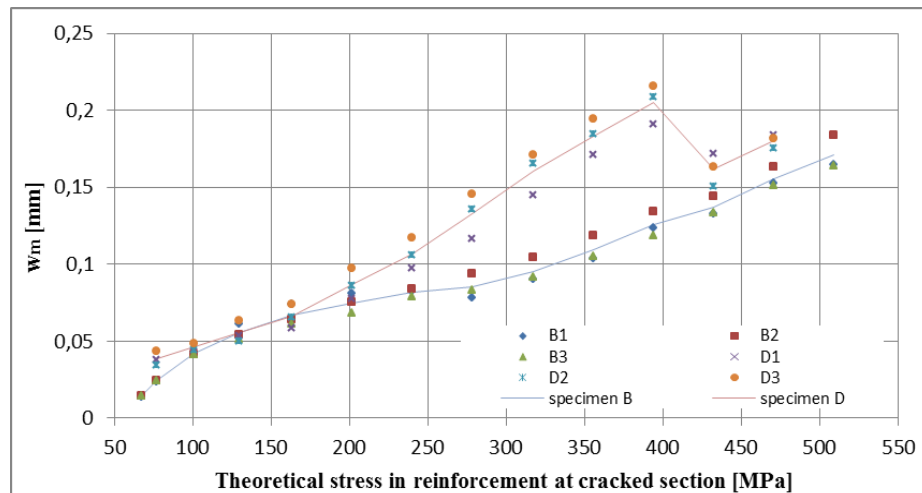


Figure 11 - Influence of the concrete cover on mean crack width – specimens B and D

For beams (specimens A and C) with different concrete cover at the same value of σ_s , when σ_s is smaller than 200 MPa for a bigger concrete cover were obtained smaller values of maximum (figure 8) and mean (figure 10) crack width. At values bigger than 200 MPa for a bigger concrete cover were obtained bigger values of maximum and mean crack width.

For beams (specimens B and D) with different concrete cover with increasing the concrete cover increase maximum (figure 9) and mean (figure 11) crack width at the same value of σ_s . For specimen D (figure 11) at values of σ_s bigger than 390 MPa there is decrease of the mean crack width, which is a result of the appearance of a large number of new cracks, which are forming at this stress stage.

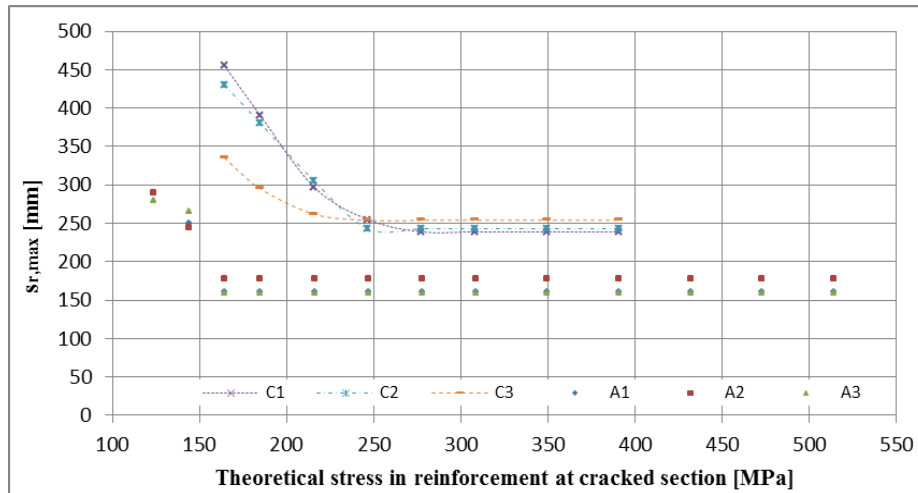


Figure 12 - Influence of the concrete cover on maximum crack spacing – specimens A and C

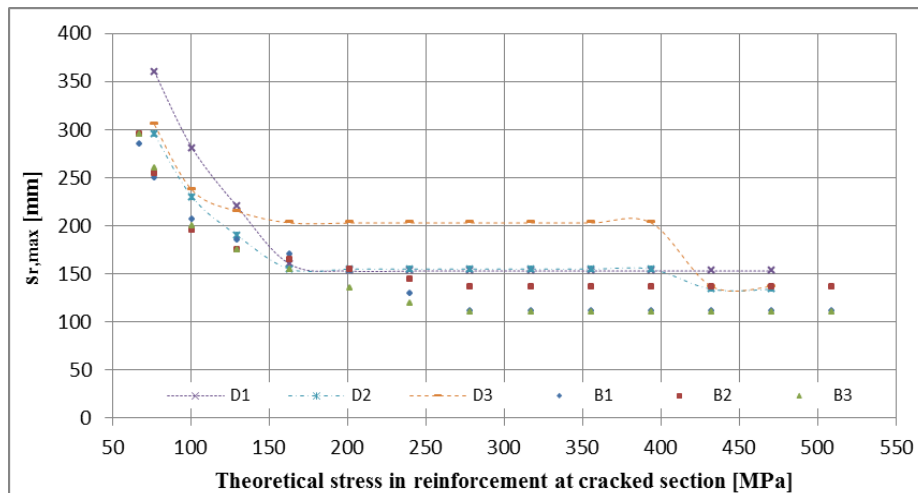


Figure 13 - Influence of the concrete cover on maximum crack spacing – specimens B and D

Experimentally determined crack spacing (figure 12 ÷ figure 15) is shown as a function of the theoretically determined σ_s , which is calculated for each of the different loading stages.

For specimens A and C (specimens B and D) with bottom longitudinal reinforcement 2N12 (2N18) and different concrete cover with increasing the concrete cover increase the maximum – figure 12 (figure 13) and mean – figure 14 (figure 15) crack spacing at the same value of the stress in reinforcement at cracked section. For beams of specimen D at values of σ_s bigger than 390 MPa there is reduction of the maximum (figure 13) and mean (figure 15) crack spacing. Their values become similar to those for beams of specimen B.

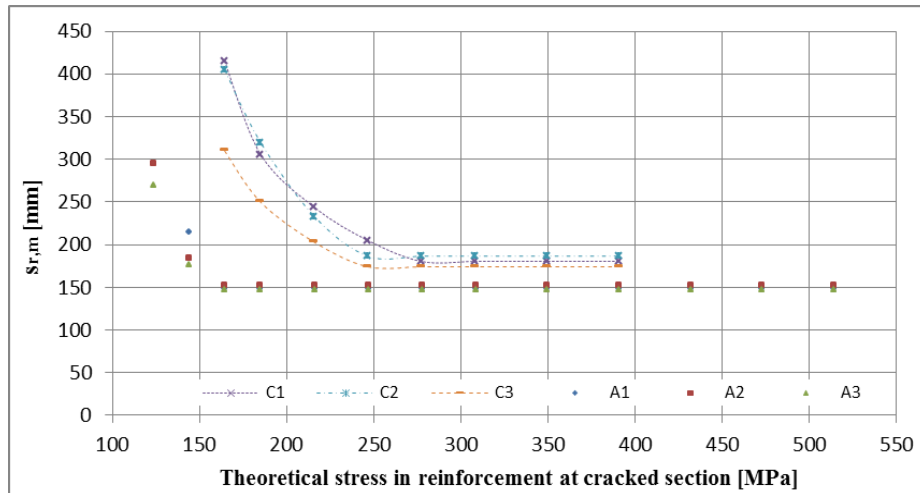


Figure 14 - Influence of the concrete cover on mean crack spacing – specimens A and C

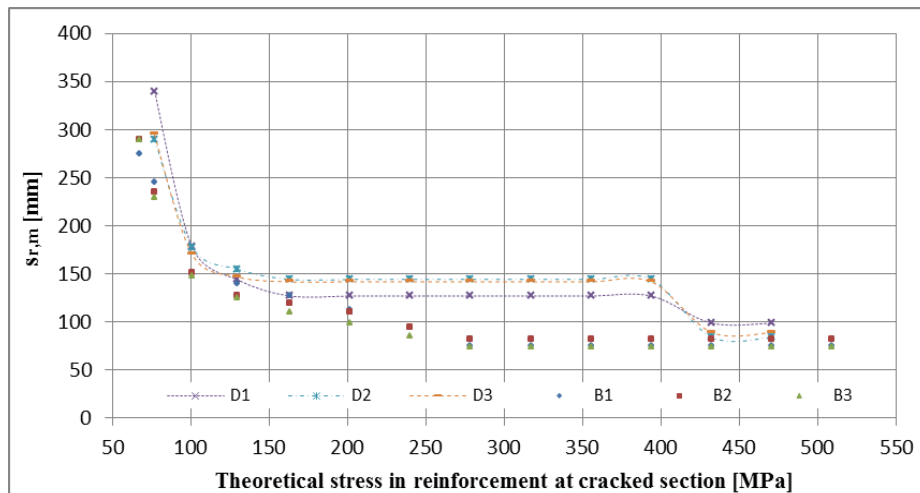


Figure 15 - Influence of the concrete cover on mean crack spacing – specimens B and D

4 CONCLUSION

With the performed experimental researches is confirmed the appropriateness of taking into account the influence of concrete cover on maximum and mean crack width and maximum and mean crack spacing.

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FIBRE MODELIRANJE AB ELEMENATA IZLOŽENIH MEHANIČKIM I TERMIČKIM DEJSTVIMA U VREMENU

Rezime:

S-3

Ponašanje betona tokom vremena utiče na upotrebljivost armiranobetonskih konstrukcija usled progresivnog povećanja deformacija i prslina. Kada je potrebno uzeti u obzir kompleksne istorije povremenog opterećenja, ili kompleksne fizičke procese kao što je varijacija temperature u betonu, neophodno je primeniti prefinjene proračunske metode. Za tu svrhu je učinjen napor da se upotreba lameliranih modela proširi na određivanje dugotrajnih deformacija. U radu je predstavljen opšti inkrementalno-iterativni pristup baziran na fukcijama moment-krivina.

Ključne reči: fibre model, zavisnost moment-krivina, dugotrajne deformacije, povišene temeperature

FIBRE MODELLING OF RC ELEMENTS SUBJECT TO TIME-VARIANT MECHANICAL AND THERMAL ACTIONS

Summary:

Time-dependent behaviours of concrete affect the serviceability of reinforced concrete (RC) structures due to progressively increasing deformations and cracks. When complex load histories like variable loads and complex physical processes like temperature variations in concrete have to be considered, practically it is necessary to employ refined calculation methods. For that purpose, an attempt has been developed to extend the application of lamellae models for the derivation of long-term deflections. The paper presents a general incremental-iterative approach based on moment-curvature relations.

Key words: fibre model, moment-curvature-relation, long-term deflections, elevated temperatures

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1 INTRODUCTION

In many practical cases the load conditions on a structure are alternating within the working life, either gradually or abruptly. A load history can be a result of a construction phase of a structure, gradual building up of live loads or of inherent nature of the live loads themselves (parking garage, bridges, storages etc.). Design of reinforced concrete structures has to be done for ultimate (ULS) and serviceability limit states (SLS) taking into account repeated and sustained loads as well as accidental actions (e.g. fire exposure). Thereby, stiffness and strength of both, concrete and rebar, progressively decrease sometimes yielding to enhanced deformations up to failure. To prevent damage, the time-dependent response of reinforced concrete structures has to be accurately determined. In the paper, an approach is proposed based on lamellae techniques to derive time-variant moment-curvature relations. Main advantage of the procedure is an easy incorporation of arbitrary stress-strain relations and cross-sections. Time-dependent responses of concrete due to creep, shrinkage and fire exposure are considered subsequently. Thereby, time-incrimination is done explicitly by a step-by-step procedure.

2 FIBRE MODELS

For numerical stress and strain evaluations under uniaxial bending with axial forces it is meaningful to discretise a cross-section into a reasonable number of fibres, also called lamellae, of equal thickness. In each lamella the stresses are assumed to be constant over its height. Thus, starting from a linear strain distribution over the cross-section's height, stresses directly arise, no matter of the material laws being linear or nonlinear. The strain level is then changed iteratively within permissible boundaries until equilibrium between forces and stress resultants can be ensured. The shapes of the cross-section as well as stress-strain relations can be of arbitrary, but still reasonable types. However, biaxial bending requires cross-section's discretisation by fibres, so subdivided into two principal orientations of the section, or finite elements [1]. Figure 1 illustrates the general procedure for a uniaxial bending situation.

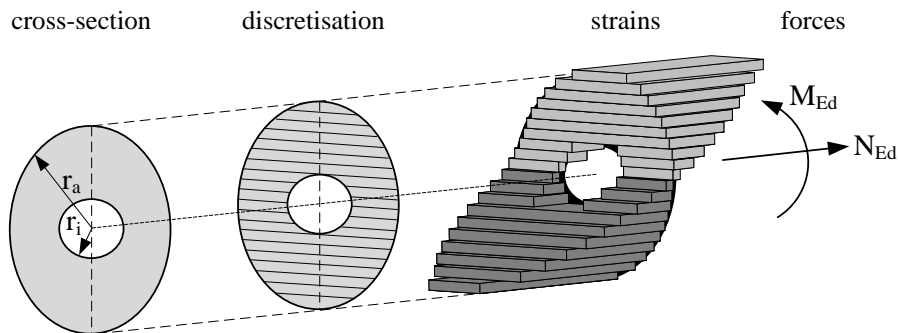


Figure 1 – Fibre model: discretization of a cross-section into lamellae

2.1 EQUILIBRIUM ON CROSS-SECTIONAL LEVEL

The governing equations for the design of RC sections can be derived from the familiar equilibrium conditions between sectional forces N , M and inner stress integrals:

$$\Sigma N = 0 = \iint_A \sigma_c(\theta, t) dA_c + A_{s1} \cdot \sigma_{s1}(\theta, t) + A_{s2} \cdot \sigma_{s2}(\theta, t) \quad (1)$$

$$\Sigma M = 0 = \iint_A \sigma_c(\theta, t) \cdot z_c \cdot dA_c + A_{s1} \cdot \sigma_{s1}(\theta, t) \cdot d_1 + A_{s2} \cdot \sigma_{s2}(\theta, t) \cdot (h - d_2) - M_{Ed}(\theta, t) \quad (2)$$

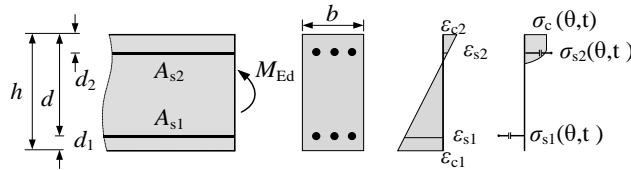


Figure 2 – Stress-strain distributions of RC beams

Subscripts c and s denote concrete and steel contributions, σ , A and z stresses, areas and inner lever arms, respectively. Thereby, temperature- (θ) and time-dependent (t) stress-strain relations may be assimilated. For valid states of the equilibrium the curvature of each cross-section can be derived from the ratio of the strains ε_{c1} , ε_{c2} to the cross-section's height h . Since for pure bending curvatures correspond to the ratio of the bending moment to the stiffness, it is suitable to model cracking and yielding by a decreasing curvature value. The integration of moment-curvature relations along a specimen's longitudinal axis finally yields the overall deflections.

2.2 MOMENT-CURVATURE RELATIONS

Typically, moment-curvature relations can be simplified to multilinear shapes, representing the initial uncracked state, crack-formation as well as yielding of the rebar up to failure in the subsequent sections. For simplification tension stiffening can be integrating in a smeared way on a cross-sectional level by modifying the moment-curvature relations as it is schematically shown in Figure 3.

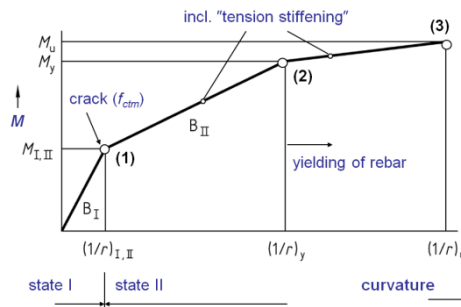


Figure 3 – Scheme of a moment-curvature relation

For the practical implementation the moment-curvature relations can be efficiently numerically integrated employing e.g. Simpson's rule:

$$\int_{x_0}^{x_n} f(x)dx = \frac{\Delta x}{3} (f(x_0) + 4f(x_1) + 2f(x_2) + 4f(x_3) + \dots + 2f(x_{n-2}) + 4f(x_{n-1}) + f(x_n)) \quad (3)$$

Of course, for each different cross-section considered a separate moment-curvature relation has to be derived.

3 APPLICATIONS TO STRUCTURAL ANALYSIS

The general procedure is applicable to a wide range of applications. Subsequently, time-variant deformations due to creep, shrinkage and fire exposure are treated.

3.1 TIME-DEPENDENT RESPONSE DUE TO CREEP

Predominantly, time dependent behaviours of concrete structures are related to the action of long-term loads coinciding with the permanent loads. Nevertheless, according to many experimental results reported in the literature [2], concrete can exhibit pronounced creep deformations under variable loads depending on their intensity and duration. According to Eurocodes, EN 1992-1-1, the effects of creep in concrete caused by variable repeated loads in SLS design can be considered assuming a quasi-permanent share of the live loads denoted $\psi_2 Q$ [3]. For a more precise deformational analysis instead of this coarse assumption, the real loading history has to be detailedly considered.

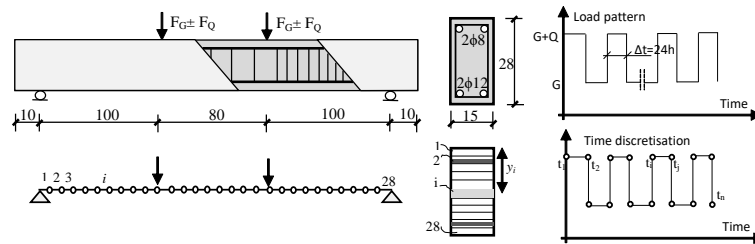


Figure 4 – Geometry, load scheme and discretisation of the analysed RC beam

In order to investigate the time-dependent behaviour of RC beams under variable loads, a numerical procedure based on a lamellae approach was applied on a reference beam made of ordinary concrete, class C30/37 subjected to variable loads in four point bending (Figure 4). The results are validated against experimental ones taken from the literature [2]. Also, experimentally obtained concrete strengths and modulus of elasticity at age of 28 days were used in the calculations. The load history consists of repeated loading and partially unloading cycles in a daily scale as displayed in Figure 4. The level of the load corresponds to a serviceability limit state.

The beam has been discretised into 28 elements along its length and 28 layers were employed over its cross-sectional height accounting for the nonlinear constitutive behaviours of concrete and reinforcement. The time is subdivided into steps Δt adopting the turning points of the cyclic load pattern (Figure 4) applying an explicit step-by-step calculation method (SSM). This method requires a separated storing of concrete stresses for each time step performed in a MATLAB environment.

3.1.1 Long-term analysis using a step-by-step method (SSM)

The applied procedure consists of multiple, aligned short-term and long-term analyses. In the initial moment when the total amount of loading ($G+Q=11.6\text{kN}$) is applied for the first time, short-term analysis was performed including the effect of shrinkage strains, since it was assumed that parts of the shrinkage have occurred prior to the first loading (the first load is applied at the concrete age of 28 days and shrinkage is assumed to start at 7 days). In each time step where the total amount of load acts on the element for 24 hours, a long term analyse was performed using corresponding creep functions for live loads. Moreover, in the steps where the variable load is removed and only permanent loads act on the element, creep recovery was taking into account in a simplified way considering a usual creep function but with a negative sign [4]. In each moment where the variable load is applied or removed, additionally the instantaneous elastic strains are determined using the corresponding concrete properties at that time (modulus of elasticity, concrete strengths and shrinkage strains). Doing so, creep and shrinkage effects are calculated as if they are acting only within the uncracked part of the cross-section.

The internal stresses carried by concrete (integrated to axial forces and bending moments) are calculated by summing up the contributions of each uncracked layers in each time step t_j using Eq. (4) and (5) [5]:

$$N_{c,j} = \sum_{i=1}^{m_c} A_{c(i)} \cdot \sigma_{c(i),j} = \sum_{i=1}^{m_c} (A_{c(i)} \cdot E_{c(i),j} (\varepsilon_{(i),j} - \varepsilon_{sh,j}) + \sum_n^{j-1} F_{e,j,n} \cdot \sigma_{c,(i),n} \cdot A_{c(i)}) \quad (4)$$

$$M_{c,j} = \sum_{i=1}^{m_c} y(i) \cdot A_{c(i)} \cdot \sigma_{c(i),j} = \sum_{i=1}^{m_c} (y(i) \cdot A_{c(i)} \cdot E_{c(i),j} (\varepsilon_{(i),j} - \varepsilon_{sh,j}) + \sum_n^{j-1} F_{e,j,n} \cdot \sigma_{c,(i),n} \cdot y(i) \cdot A_{c(i)}) \quad (5)$$

there m_c denotes the number of the layers, $A_{c(i)}$ the area of each concrete layer, $E_{c(i),j}$ the effective modulus of elasticity of concrete at time t_j (zero if the layer is cracked), $\varepsilon_{(i),j}$ the strain in the i -th layer at a time t_j , $\varepsilon_{sh,j}$ the shrinkage strain at t_j , $y(i)$ the distance between the centroid of the i -th layer and the cross-sectional centroid and $\sigma_{c,(i),n}$ the concrete stresses in i -th layer in a time step n .

$$r_{c,i} = \begin{bmatrix} N_{c,i} \\ M_{c,i} \end{bmatrix} = \begin{bmatrix} A_c & B_c \\ B_c & I_c \end{bmatrix} E_{c,i} \cdot \varepsilon_i + f_{cr,i} - f_{sh,i} \quad (6)$$

The effects of creep and shrinkage are considered through fictitious external forces and bending moments calculated through Eq. (7) and (8), assuming that the shrinkage strain is acting uniformly throughout the cross-section [6].

$$f_{cr,j} = \sum_{i=0}^{j-1} F_{e,j,i} \cdot r_{c,i} \quad (7) \quad f_{sh,j} = \begin{bmatrix} A_c \\ B_c \end{bmatrix} E_{c,j} \varepsilon_{sh,j} \quad (8)$$

$F_{e,j,i}$ describes a stress modification factor defined with Eq. (9) where J denotes a creep compliance function and $r_{c,i}$ a vector of concrete internal actions from the previous time step i . In absence of experimental results for creep functions, shrinkage strains and modulus of elasticity of concrete for each time step, they are predicted according to MC2010 resulting in strains ε_j over the section (10). A detailed description of the procedure can be found in [5].

$$F_{e,j,i} = \frac{J_{j,i+1} + J_{j,i}}{J_{j,j}} \quad (9) \quad \varepsilon_j = F_j(r_{e,j} - f_{cr,j} + f_{sh,j}) \quad (10)$$

For each discretised element the cracked and the uncracked curvatures are calculated. The tension stiffening effect is included interpolating between these two curvatures through a distribution coefficient ζ (11), as suggested in Eurocode 2 [3]. After that, the average curvature is calculated using Eq. (12).

$$\zeta = 1 - \beta_1 \beta_2 \left(\frac{M_{cr}}{M}\right)^2 \quad (11) \quad k_{ave} = \zeta \cdot k_{cr} + (1 - \zeta) \cdot k_{uncr} \quad (12)$$

Here M_{cr} corresponds to the cracking moment, M to the actual outer moment of the considered element, a coefficient β_1 models the bond between concrete and reinforcement bars (1.0 for deformed bars) and β_2 models the duration of the load (0.5 for sustained or repeated loads).

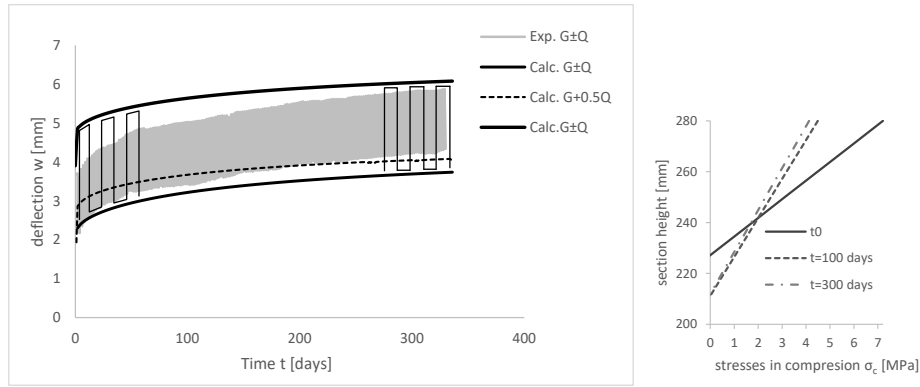


Figure 5– Time-dependent deflections under variable loading (left) ; distributions of compressive stresses in concrete over time (right)

Figure 5 (left) presents long term mid-span deflections calculated with the described method for the load pattern displayed in Fig.4 (solid black line) versus experimentally obtained deflections (grey line). The black solid lines present the deflections after removing (the lower one) and after applying the live load Q in a sustained manner for 24 hours (upper one). It should be noted that since no codes and recommendations account for a time dependence of the tension-stiffening reduction, β_2 was taken to a constant of 0.5 from the beginning, underestimating the tension-stiffening effect. According to results reported in the literature, tension stiffening reduces its value for about 50% after 30-40 days of first loadings [7]. The dashed line in Fig.5 (left) presents the long-term deflection calculated on the basis of a quasi-permanent combination of loads according to Eurocode 2 [3]. The value of the quasi-permanent coefficient ψ_2 was taken for 0.5, already established on the basis of the used experimental results [2] (quasi permanent load: $F = F_G + \psi_2 F_Q = 4 + 0.5 \cdot 7.6 = 7.8 \text{ kN}$). Even the results in both cases show a trend to underestimate deflections with time, in general they are in a good accordance with the experimental data.

Figure 5 (right) shows the decrease in stress and growing of compressive zone of concrete with time due to the effect of creeping under variable loadings. The applied model yields increasing deflections over time accompanied by significant contraction by the compressive zone [5].

3.2 TIME-DEPENDENT RESPONSE DUE TO ELEVATED TEMPERATURES

The time-dependent response of three single-span steel fibre reinforced concrete (SFRC) slabs subjected to elevated temperatures induced by fire and steady mechanical loading is investigated. The specimens have recently been tested in a cooperative research project between the two German Universities TU Kaiserslautern and Ruhr-University Bochum as reported in [8]. During testing a heat impact in the shape of the uniform-temperature-time curve (UTTC) according to DIN EN 1992-1-2 and Equation (13) models one-sided fire exposure for 120 minutes at maximum.

$$\theta(t) = 20 + 345 \cdot \log_{10}(8t + 1) \text{ [}^\circ\text{C]} \text{ with } t \text{ [min]} \quad (13)$$

The specimens of $l/b/h/d_1$ with 400/100/18/2 [cm] effectively span $l_{\text{eff}} = 3,84$ m one loaded by a live load of about $q = 4,5$ kN/m² and made of concrete C 35/45. The three samples of equal geometry differ in the amounts of rebar and steel fibres only. While specimen P1 is of plain concrete reinforced with rebar mats of $a_{s1} = 3,35$ cm²/m lengthwise and about 20% of a_{s1} across, SFRC is used for P3 and P4 with rebar mats of $a_{s1} = 1,88$ cm²/m lengthwise and about 20% of a_{s1} across, respectively. They contain hooked-end steel fibres of length $l = 60$ mm and aspect ratio of length to diameter of $l/d = 60$. The dosage of fibres is about 40 kg/m³ in case of P3 and increases to 80 kg/m³ for P4. This results in performance classes L1/L2 = 2,7/2,1 (P3) and L1/L2 = 3,0/3,0 (P4) according to [9], where the values L1/L2 denote average bending strength values after cracking in MPa. Analyses focus on two aspects, namely the influence of fibres on the heat transfer within the cross-section and the fibre's effects on the temperature-dependent bending stiffness. While test setup and results are elaborately depicted in [8] and illustrated in basic principles in Figure 6 a combined numerical recalculation procedure is presented here.

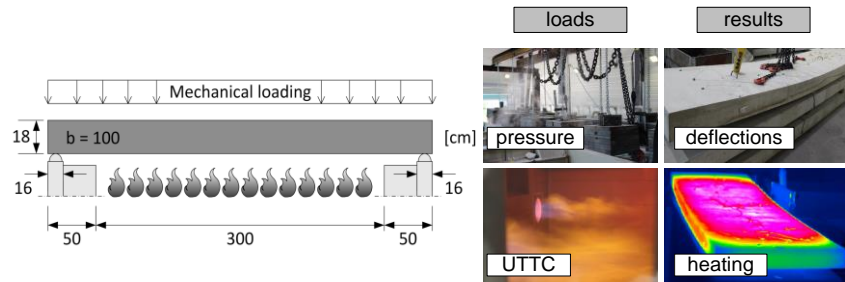


Figure 6– Test setup and qualitative results

In detail, an incremental-iterative approach is favoured subdivided into thermal and mechanical analyses. Since the current stress-state is assumed to not influence heat transmission due to fire exposure, heat flux can be derived a priori. But, mechanical analysis to compute time-dependent recalculations of stresses and deformations is linked to heat transmission via temperature-dependent stress-strain relations. According to the general

calculation procedure of fibre models (c.f. section 2) they have to be taken into consideration lamellae-wise for each time-increment of UTTC investigated. In a step-by-step time discretisation approach, stress-strain iterations on a cross-sectional level serve to derive time- or rather temperature-dependent moment-curvature-relations for valid states of equilibrium between stress resultants and forces. Thereby, the softening material behaviour in the course of fire exposure of both, rebar and (steel fibre reinforced) concrete has to be adequately considered. While DIN EN 1992-1-2 [11] recommends temperature-dependent stress-strain relations for plain concrete in compression and rebar only, temperature-dependent but strain-independent reduction factors $k_t^f(\theta)$ of concrete's tensile strength as well as steel fibres post-cracking tensile strength f_t^f are given in equation (14) derived from [10]. Here, $k_t^f(\theta)$ specifies the ratio of the material's strength at elevated temperatures to the initial one at normal temperatures.

$$k_t^f(\theta) = \begin{cases} 1.0 & 20 \leq \theta \leq 150 \text{ }^\circ\text{C} \\ 1.0 - 1.0 \cdot (\theta - 150^\circ\text{C}) / 550^\circ\text{C} & 150 \leq \theta \leq 700 \text{ }^\circ\text{C} \\ 0 & \theta > 700 \text{ }^\circ\text{C} \quad \text{with } \kappa^f(\theta) = f_t^f(\theta) / f_t^f(\theta = 20^\circ\text{C}) \end{cases} \quad (14)$$

Heat flux analysis can be done starting from a uniform initial temperature field of $\theta = 20^\circ\text{C}$. Thermal parameters in terms of temperature dependent density, heat capacity and conductivity are given in DIN EN 1992-1-2 [11] for concrete and DIN EN 1993-1-3 [12] for rebar, respectively. For simplification, those parameters of concrete can be adopted for steel fibre reinforced concrete up to dosages of about 80 kg/m³ as well [10]. It is proven that portions up to about 1.0 Vol.-% of fibre dosage do almost not affect heat transfer.

3.2.1 Heat transmission

Starting from the first law of thermodynamics, the differential equation of heat flux by Fourier can be deduced [13]. Neglecting internal heat sources the temperature distribution $\mathcal{G}(x,t)$ inside a solid body at time t and location x follows to:

$$\frac{\partial \mathcal{G}(x,t)}{\partial t} - a \nabla^2 \mathcal{G}(x,t) = 0 \quad (15)$$

Thereby, a denominates the temperature dependent material parameter of thermal diffusivity. It is defined to the quotient of the thermal conductivity λ and the product of material's density ρ and the specific heat capacity c_p . To efficiently calculate temperature fields, the heat transfer given in numerical form according to Lichte [14] serves:

$$\frac{\Delta Q_i}{\Delta t} = \sum_{k=1}^m q_{i,k} \cdot l_{i,k} + q_{i,0} \cdot l_{i,0} \quad (16)$$

Herein, $q_{i,k}$ denotes the internal heat flow density and $l_{i,k}$ the contact length between a rectangular element i and a number of m surrounding elements subscripted with k . Each directed heat flow density can be approximated according to Eq. (17) depending on the two lengths l_i and l_k of both adjacent elements, their thermal conductivities λ_i , λ_k and temperatures \mathcal{G}_i and \mathcal{G}_k at the current time instant.

$$q_{i,k} = \frac{2\lambda_i\lambda_k}{\lambda_k s_i + \lambda_i s_k} (g_k - g_i) \quad (17)$$

Boundary conditions, i.e. thermal loads according to Eq. (15) are treated similarly, but one adjacent element is replaced by the heat transfer at the edge

$$q_{i,0} = \frac{2\lambda_i \cdot \alpha_{K,i}}{\alpha_{K,i} \cdot s_i + 2\lambda_i} (g_{ID,i} - g_i) \quad (18)$$

To account for convective and radiative portions of heat flow, an equivalent temperature g_{ID} w.r.t. [15] is applied.

$$g_{ID} = g_i + \frac{1}{\alpha_{K,e}} \cdot q_0 = g_i + \frac{1}{\alpha_{K,e}} [\alpha_{K,e} \cdot (g_L - g_i) + q_r] \quad (19)$$

The temperature field at the end of a time increment Δt can finally be updated

$$\Delta g_i = \frac{1}{c_i(g) \cdot \rho_i(g) \cdot a_i \cdot b_i} \cdot \frac{\Delta Q_i}{\Delta t} \Delta t \quad (20)$$

to gain $g(t + \Delta t) = g(t) + \Delta g$. Figure 7 exemplarily shows numerical results for discrete time steps of $t_{UTTC} = 15, 60$ and 120 minutes with respect to Equation (13). Obviously, one-sided fire exposure causes significant heating of the whole specimen, non-linearly distributed over the height and length due to the specific location of the fire contact. The numerical implementation has been performed using spread sheet calculations in Microsoft Excel.

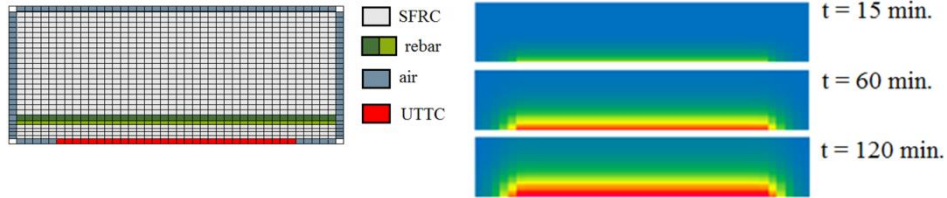


Figure 7– Fibre discretisation (left) and results of the numerical heat flux analysis for specimens subjected to one-sided fire at three time stages (right)

3.2.2 Temperature-dependent deformations

To assess total but time variant deflections thermal-induced deformations can additively be superimposed to stress-dependent ones. Moreover, thermal deformations δ^{th} depend on the linear vertical temperature gradient Δg_v . It is obtained from the holistic non-linear temperature field (Figure 7) from a function of the cross-section's height h , its moment of inertia I_y , its area A and the temperature θ at each point based on a coordinate system having its origin in the centre of gravity [16]. The temperature gradient for numerical applications results from Eq. (22).

$$\Delta g_v \cong \frac{h}{I_y} \sum_i \sum_j \theta_{i,j} z_{i,j} A_{i,j} \quad (22)$$

Considering the material dependent coefficient of thermal expansion $\alpha_T(\theta)$ according to DIN EN 1992-1-2 yields thermal strains ε^{th} . They can be used to derive the total curvature κ of a cross-section arising from the sum of stress- (subscript “ σ ”) and temperature-dependent (subscript “ th ”) curvatures:

$$\kappa = \kappa^\sigma + \kappa^{th} = \frac{M}{EI} = \frac{|\varepsilon_{c2}^{ges}| + \varepsilon_{s1}^{ges}}{d} = \frac{\left(|\varepsilon_{c2}^\sigma| + |\varepsilon_{c2}^{th}\right) + \left(\varepsilon_{s1}^\sigma + \varepsilon_{s1}^{th}\right)}{d} \quad (24)$$

Numerical integration employing e.g. Simpson’s rule (Eq. (3)) finally yields total deformations δ . Figure 8 contrasts calculated deformations (denoted “calc”) to measured ones (“test”) for specimens P1, P3 and P4. Obviously, the results are in good accordance on average exhibiting pronounced deformations up to about 1/30 of the span. Obviously, temperature-dependent behaviour is strongly non-linear with time even though steel fibres provide benefits in terms of fire resistance duration and temperature-dependent bending stiffness when combined with rebar.

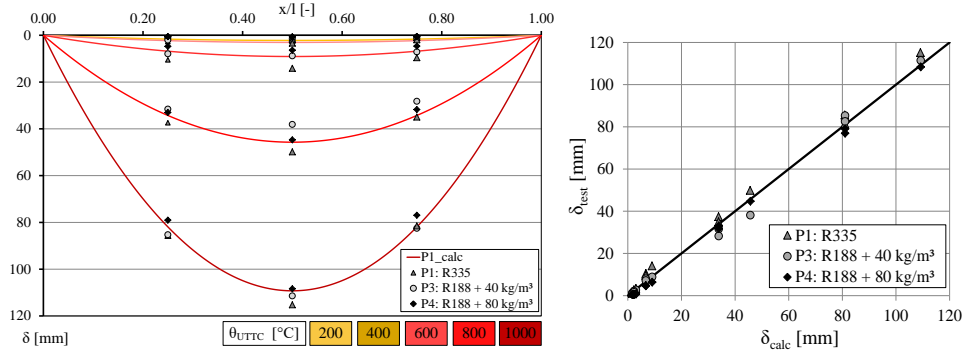


Figure 8– Experimental vs. numerically derived deformations

Since the derivation of moment-curvature relations requires stress-strain iterations on the cross-sectional level, temperature-dependent redistribution of stresses can be assessed [10]. Due to the softening material response of concrete and rebar when subjected to elevated temperatures the height of the compressive zone progressively decreases even though mechanical loading stays constant over time. Here, strong redistributions of stresses occur once temperature exceeds about 600°C at the surfaces.

4 CONCLUSIONS

In the paper generalised fibre models for transient stress and deformation analyses based on moment-curvature relations are presented. The approaches efficiently include complex concrete issues like time- and temperature-dependent stress-strain laws due to fire, creep and shrinkage. Experimental data serve for their verification. The results are in good accordance on average indicating decreasing stiffnesses and strengths due to long-term sustained loads and elevated temperatures, respectively.

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OPTIMIZACIJA AB SLOŽENOG RAVANSKOG NOSAČA STRUT-AND-TIE MODELIMA

Rezime:

S-4

Problem statičke analize i dimenzionisanja armiranobetonskih nosača sa diskontinuitetima, usled iznenadnih promena statičkih veličina ili geometrije elementa, može se rešiti primenom Strut-and-Tie metode. U radu se koristi diskretna topološka optimizacija koja podrazumeva zamenu realnog nosača ekvivalentnim rešetkastim, odnosno Strut-and-Tie modelom koji se koristi za dimenzionisanje. U ovom radu, na osnovu predloženog načina optimizacije, analiziran je i optimizovan armiranobetonski zidni nosač sa otvorom, prema količini potrebne armature koja je određena za različite favorizovane pravce postavljanja.

Ključne reči: topološka optimizacija, Strut-and-Tie model, ab zidni nosač

OPTIMIZATION OF THE RC COMPLEX PLANE MEMBER USING STRUT-AND-TIE MODELS

Summary:

The problem of static analysis and dimensioning of reinforced concrete members with discontinuities, due to sudden changes of static sizes or element geometry, can be solved using the Strut-and-Tie method. In this paper, the discrete topology optimization is used, which includes the replacement of the real member with the equivalent truss or Strut-and-Tie model that is used for dimensioning. Based on the proposed manner of optimization, the reinforced concrete wall with an opening is analyzed and optimized, according to the required reinforcement amount that is determined for different reinforcement layouts.

Key words: topology optimization, Strut-and-Tie model, rc wall member

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1 INTRODUCTION

In the places of discontinuity, as in the case of sudden changes of static sizes or member geometry, where classical dimensioning cannot be applied, the Strut-and-Tie method can be used. It provides, in a relatively simple and crude manner, a real insight into the behaviour of a reinforced concrete element [1], [2] and [3]. This is a method of compressed and tensioned elements and can be successfully applied in the design of member zones with complex internal forces or in the whole member design.

In the base of the Strut-and-Tie method, there is a truss member with axially loaded elements of finite dimensions and the placement of their crossing, the so-called nodal zone, also with the defined dimension. The principle of the method is that the compressed elements replace concrete parts (area or volume) of reinforced concrete member, while the tensioned reinforcement replaces the reinforcement. The selection of the Strut-and-Tie model is affected by the geometry of the member, the configuration of the load, and the additional conditions related to the preferred directions of the reinforcement, which causes the determination of the Strut-and-Tie model of a reinforced concrete member.

The paper uses the topology member optimization that includes a special arrangement of structural elements and their connections (nodes) according to [4], [5], [6] and [7]. Topological member optimization is applied for the formation of the design model, and later, for the dimensioning, i.e. the adoption of the necessary quantity of reinforcement in the reinforced concrete member.

2 OPTIMIZATION MANNER

The goal of optimization is to increase structural performances. It is necessary to know which design variables best describe the characteristics of the element. The best solution that is extracted from a large number of solutions is obtained by changing variables according to certain criteria. The choice of appropriate optimization solution has a decisive role in further design, in dimensioning of the optimized structural member.

Design variables can be member dimensions, geometric characteristics of the element, mechanical and physical properties of materials, price, various physical and structural constraints in the field of application of the structural member, etc.

During the design of the structural element, it is desirable to determine the transfer path of the external load, through the element to the supporting parts of the member. Thus, the unloaded zones can occur and can be eliminated, so the element can be modelled, i.e. optimized and used for dimensioning.

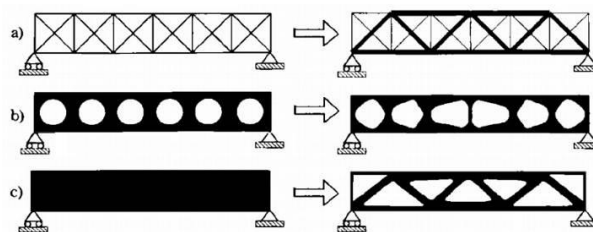


Figure 1. Optimization types (left – the initial model, right – optimized element) according to [4]

Optimization types of the structural system are determined by the member size, shape and topology. Size optimization means a change, for example, of element thickness, cross-sectional area of the element or the thickness distribution in the part of the element, Figure 1a. Shape optimization represents the shape changes of the member part contour, Figure 1b. Topology optimization involves the redistribution of the structural system material with the change of the element connection within the system, Figure 1c.

Topology optimization of the trusses and frames includes a special arrangement of structural elements and their connections (nodes). Topology is determined in the earliest stage of design, and it is the spatial distribution of the material that is the most loaded in the element. There are two types of topology optimization, discrete and continuum optimization of the structural system. In discrete optimization, the structure is generally modelled as a discrete truss system with linear elements. In continuum optimization, the structure is modelled as a continuum, [4] and [5].

Strut-and-Tie model can be presented as a kind of the discrete topology optimization of reinforced concrete elements. The member is discretized and reduced to a truss system which is obtained by connecting all points of the member with line elements, Figure 2.

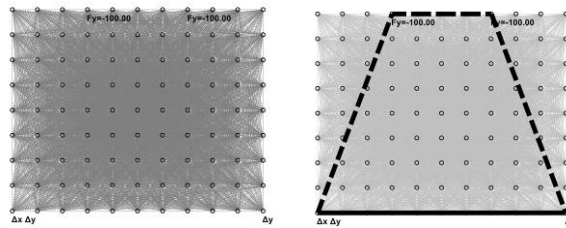


Figure 2. Discretization of a rc member - left; the optimal form of the member - right

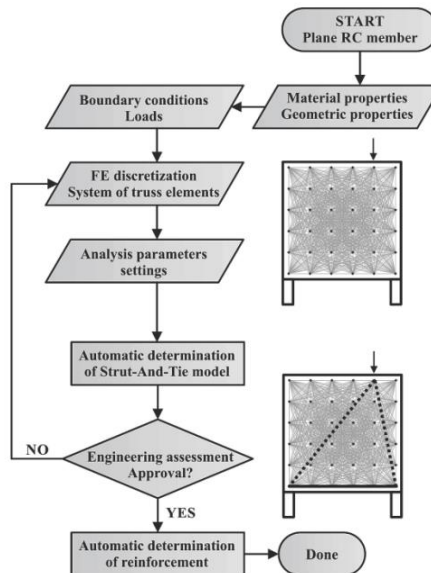


Figure 3. Flowchart of the “ST method” program according to [11]

A proposal for the discrete topology optimization manner is presented in [8], [9], [10] and [11]. For research purposes, the proposed method of the discrete optimization is implemented into the software program “ST method”, and it is used for the parametric analysis in this paper. Also, the program offers the possibility of automatic dimensioning of the Strut-and-Tie models. The flowchart of the “ST method” program is shown in Figure 3.

In this paper, according to the proposed manner of optimization, a reinforced concrete wall member is analyzed, Figure 4, and it is optimized according to the amount of the required reinforcement that is determined for different favoured layouts.

3 NUMERICAL EXAMPLE

Figure 4 shows the geometrical and mechanical characteristics of the reinforced concrete wall with an opening, where the optimization procedure was applied according to the reinforcement amount. The member is discretized with the finite element system, where nodes are connected to each other within the network whose mesh density is 1x1 m, Figure 5.

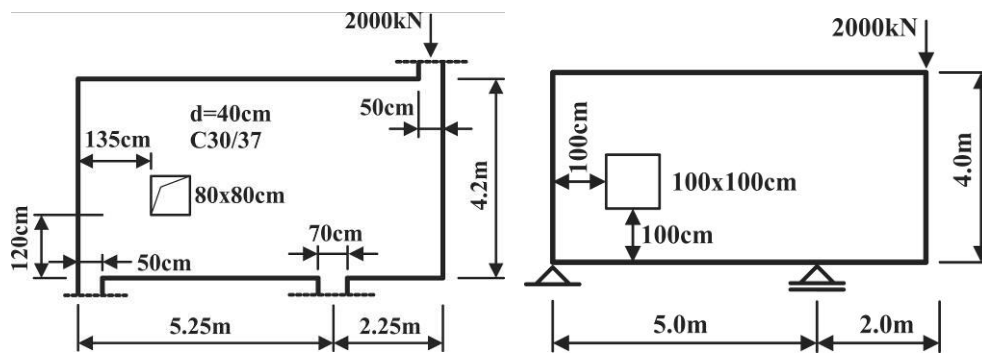


Figure 4. Reinforced concrete wall with an opening – left and static system - right

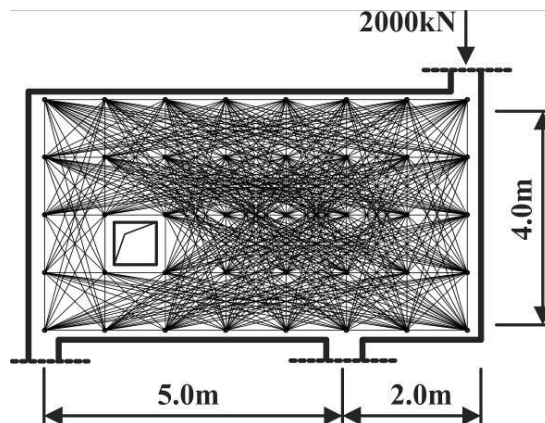


Figure 5. Finite element mesh for determining the Strut-and-Tie model

Table 1 – Strut-and-Tie model for the variant 1

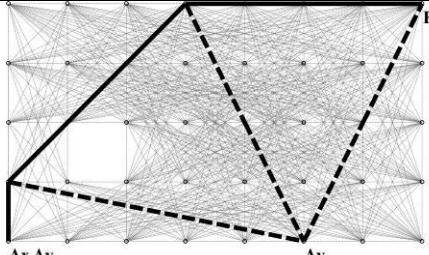
<p style="text-align: center;">Variant 1</p> 	<p>Analysis parameters</p> <p>$E_c = 33 \text{ GPa}$, $E_s = 200 \text{ GPa}$</p> <p>Compressed element: $\beta = 1$</p> <p>Tensioned element:</p> <p>(0°) $\beta = 1$;</p> <p>(45°) $\beta = 1$;</p> <p>(90°) $\beta = 1$;</p> <p>(other angles) $\beta = 0.01$</p>
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Table 2 – Strut-and-Tie model for the variant 2

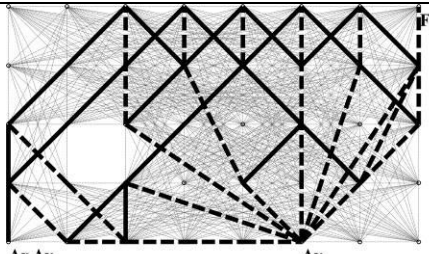
<p style="text-align: center;">Variant 2</p> 	<p>Analysis parameters</p> <p>$E_c = 33 \text{ GPa}$, $E_s = 200 \text{ GPa}$</p> <p>Compressed element: $\beta = 1$</p> <p>Tensioned element:</p> <p>(0°) $\beta = 0.01$;</p> <p>(45°) $\beta = 1$;</p> <p>(90°) $\beta = 1$;</p> <p>(other angles) $\beta = 0.01$</p>
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Table 3 – Strut-and-Tie model for the variant 3

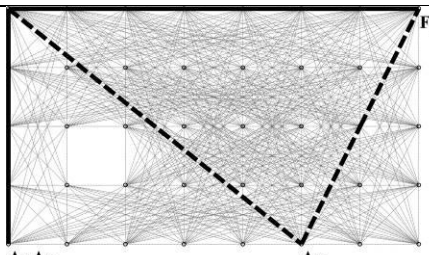
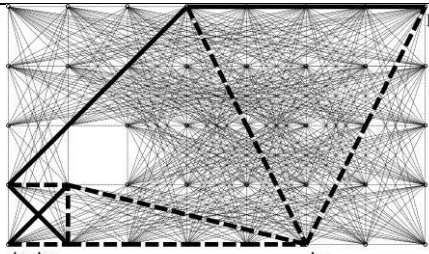
<p style="text-align: center;">Variant 3</p> 	<p>Analysis parameters</p> <p>$E_c = 33 \text{ GPa}$, $E_s = 200 \text{ GPa}$</p> <p>Compressed element: $\beta = 1$</p> <p>Tensioned element:</p> <p>(0°) $\beta = 1$;</p> <p>(45°) $\beta = 0.01$;</p> <p>(90°) $\beta = 1$;</p> <p>(other angles) $\beta = 0.01$</p>
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Table 4 – Strut-and-Tie model for the variant 4

<p style="text-align: center;">Variant 4</p> 	<p>Analysis parameters</p> <p>$E_c = 33 \text{ GPa}$, $E_s = 200 \text{ GPa}$</p> <p>Compressed element: $\beta = 1$</p> <p>Tensioned element:</p> <p>(0°) $\beta = 1$;</p> <p>(45°) $\beta = 1$;</p> <p>(90°) $\beta = 0.01$;</p> <p>(other angles) $\beta = 0.01$</p>
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Analysis and member dimensioning were conducted in four different combinations of β coefficients that were defined in order to determine the optimal solution by the amount of the required reinforcement [11]. Considering that, for the acceptance of the member self-weight, the same amount of reinforcement is required for each analyzed combination of β coefficient values, the optimization according to the amount of reinforcement was made only for the acceptance of the concentrated force on the member, Figure 5. Tables 1 to 4 show the Strut-and-Tie models and the combinations of β coefficients for the determined models.

Strut-and-Tie model for Variant 1 represents the equally desired reinforcement directions of 0° , 45° and 90° , as seen in Table 1. In Table 2, the model for Variant 2 is represented, where the reinforcement directions of 45° and 90° are favoured. The model for Variant 3 with the desired reinforcement directions of 0° and 90° is shown in Table 3. Variant 4 is represented by the model with the favoured reinforcement directions of 0° and 45° , observed in Table 4. Required reinforcement amounts for different combinations of β coefficient values are shown in Figure 6 and in Table 5.

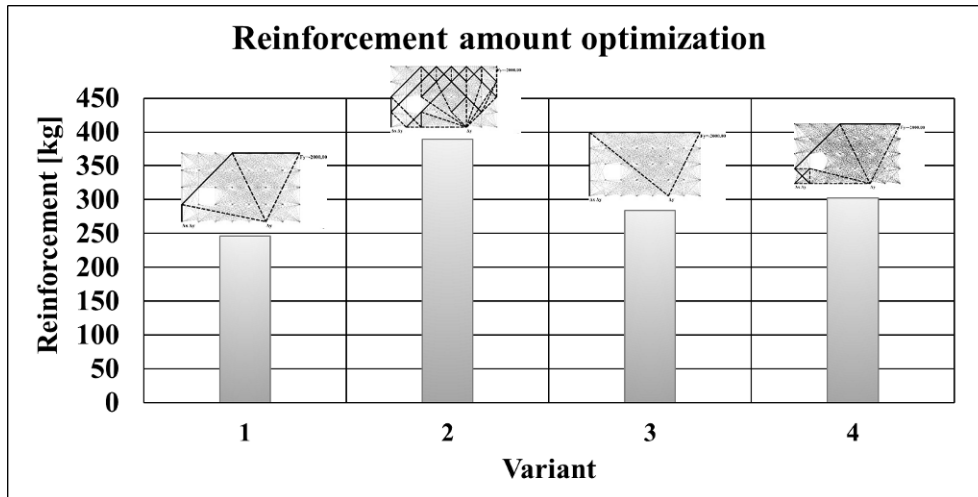


Figure 6. Required reinforcement amounts for various combinations of β coefficient values

Table 5 – Reinforcement amount optimization

Variant	Reinforcement [kg]	Percentage difference [%]
1	246	/
2	389	+ 58.1
3	284	+ 15.4
4	390	+ 58.5

Based on the results of the analysis, it can be concluded that the optimal solution is Variant 1, because it requires the minimum reinforcement amount compared to other variants. Also, Variant 1 and Variant 3 require the simplest schemes of reinforcement member layouts, and that favours them in comparison to other variants.

4 CONCLUSION

In the discontinuity zones, due to sudden changes of the static sizes or member geometry, the classical way of dimensioning stops to be valid, and the problem can be solved by using simple and rational models, such as the Strut-and-Tie model.

In the basis of the Strut-and-tie models, there is the discrete topology optimization, where the real member is modelled by the system of simple line elements. Strut-and-Tie model represents the real member with the equivalent truss system, where compressed elements replace the concrete parts of the reinforced concrete member and tensioned elements replace reinforcement.

The proposal manner for the discrete topology optimization is shown in [8], [9], [10] and [11]. For research purposes, the proposed method for the discrete optimization is implemented in the computer program "ST method", which is used for the parametric analysis in this paper. Likewise, the program offers the possibility of automatic dimensioning of the Strut-and-Tie models.

During the member analysis, the β coefficients can influence the reinforcement layout in the element. The usual directions of the reinforcement (horizontal, vertical and at an angle of 45°) are conditioned by the faster and easier building construction, and often they are not in line with the tensile stress trajectories obtained by the elasticity theory solutions.

The reinforced concrete wall member with an opening is analyzed and optimized using the proposed manner of optimization. Based on the analyzed reinforced concrete member, it can be concluded that, in terms of the required reinforcement amount, the optimal model is Variant 1. If the simplicity of the reinforcement layout is regarded, Variant 1 and Variant 3 can be considered as optimal.

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UTICAJ POLOŽAJA OTVORA NA PONAŠANJE ARMIRANO BETONSKE VISOKE GREDE

Rezime:

S-5

Na ponašanje deep beam bitno utiče položaj otvora u odnosu na smičuće zone nosača koje se javljaju na prirodnom putu prenosa opterećenja od mesta delovanja do oslonačkih zona. Kao posledica toga javljaju se naponski diskontinuiteti u nosaču, koji se može analizirati Strut-and-Tie metodom. U radu je analizirana deep beam sa različitim položajem otvora u nosaču, a osnovni parametri za usvajanje optimalnog rešenja su inženjerska procena složenosti oblika Strut-and-Tie modela i vrednost ugiba sredine donje ivice nosača. Jednostavniji oblici Strut-and-Tie modela podrazumevaju jednostavniju šemu armiranja nosača, a manje vrednosti ugiba su povoljnije sa aspekta upotrebljivosti.

Ključne reči: visoka greda sa otvorom, optimizacija, Strut-and-Tie model

IMPACT OF THE OPENING POSITION ONTO THE BEHAVIOUR OF A REINFORCED CONCRETE DEEP BEAM

Summary:

The behaviour of a deep beam is significantly affected by the position of the opening relative to the member shear zones occurring in the natural load path from the load to the support bearing blocks. As a result, stress discontinuities appear in the member, which can be analyzed using the Strut-and-Tie method. The paper analyzes the deep beam with different opening positions in the member; the basic parameters for the adoption of optimal solutions are the engineering assessment of the model shape complexity and the mid span bottom edge vertical displacement value. Simpler Strut-and-Tie model forms lead to the simpler member reinforcement, and smaller displacement values are more favourable in terms of serviceability.

Key words: deep beam with opening, optimization, Strut-and-Tie model

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1 INTRODUCTION

In the construction, for the future building purposes, the appearance of the opening is common in deep beams, Figure 1. The classic manner of dimensioning is not applicable in the design of the complex geometry member. Bernoulli's and Navier's assumptions are not valid because the stress discontinuities occur due to the changes in the geometry of the member or static values.

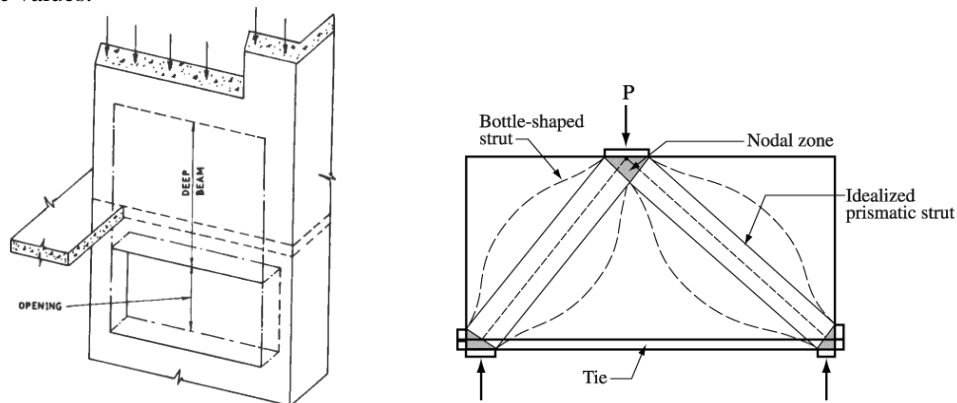


Figure 1. Deep beam with an opening (left) [15], Strut-and-Tie model (right) [12]

In the cases of discontinuity, the Strut-and-Tie method can be applied, providing an insight into the real behaviour of reinforced concrete elements on a relatively simple and crude way, [1], [13] and [3]. Basically, the Strut-and-Tie method is equivalent to the truss such that the compressed elements replace the concrete parts of the reinforced concrete member, while the tensioned elements represent the reinforcement, Figure 1 (right).

For the purpose of this research, the discrete topology optimization is applied and implemented in the computer program "ST method", which is used for the parametric analysis, [16], [7], [8] and [11]. Also, the program offers the possibility for the automatic dimensioning of the Strut-and-Tie models.

Since the failure of the reinforced concrete deep beams often occurs due to shear, special attention is to be paid to the loaded shear zone in the member.

In this paper, the reinforced concrete beam is analyzed with different positions of the opening. Adoption of the optimal model solution is done based on the engineering assessment of the Strut-and-Tie model simplicity.

2 SHEAR ZONES IN DEEP BEAMS

The main parameters that influence on the behaviour of the deep beams with openings are: span to depth ratio, the cross-sectional shape, amount and layout of the main longitudinal reinforcement, the amount, type and position of web reinforcement, properties of concrete and reinforcement, type and position of loading, size, shape and position of web opening, etc. [15].

Loaded shear zones are areas near the load bearing or the support bearing blocks, Figure 2. Openings in the members, if they are in loaded shear zones, unfavorably affect on the shear resistance of the member and, on that basis, it is necessary to avoid the holes in these zones.

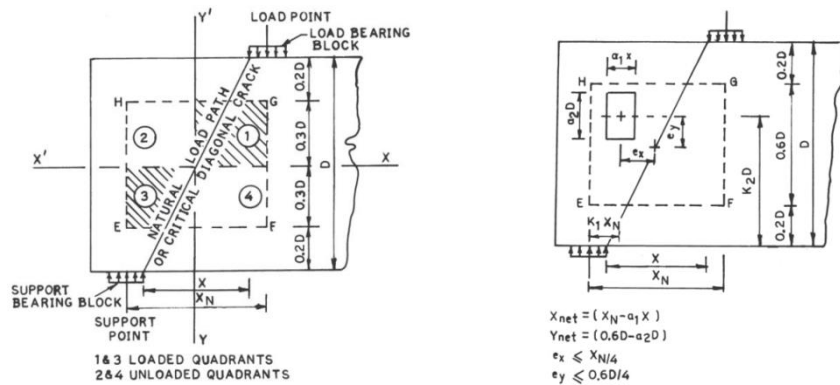


Figure 2. Shear zones (loaded and unloaded) in RC deep beam, [15]

Depending on the placement of the opening, the behaviour of the loaded deep beam is different. If the opening is found in the unloaded shear part of the member, the deep beam acts as a full member. Such placed holes in deep beams are desirable in terms of the member shear resistance.

In this paper, the reinforced concrete deep beam with an opening is analyzed and optimized, Table 1, according to different positions of the opening in the member for common directions of the reinforcement, horizontal, vertical, and at an angle of 45°.

3 NUMERICAL ANALYSIS

The geometrical and mechanical characteristics of the analyzed reinforced concrete deep beam with different positions of the opening are shown in Tables 1 to 5. Likewise, the schematic views of the shear zone (the hatched part of the member), i.e. a network of finite elements and forms of the Strut-and-Tie model is provided. The values of vertical displacements of the mid span bottom edges for all analyzed member variants are presented in Table 6.

Table 1 – Strut-and-Tie model for variant 1

Variant 1	
	C30/37, B500B Compressed element: $\beta = 1$ Tensioned element: (0°) $\beta = 1$; (45°) $\beta = 1$; (90°) $\beta = 1$; (other angles) $\beta = 0.01$

Table 2 – Strut-and-Tie model for variant 2

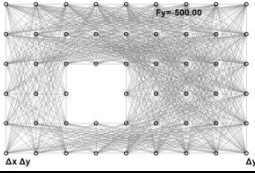
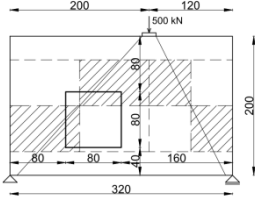
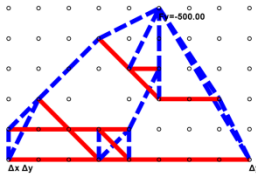
Variant 2	
	<p>C30/37, B500B Compressed element: $\beta = 1$ Tensioned element: (0°) $\beta = 1$; (45°) $\beta = 1$; (90°) $\beta = 1$; (other angles) $\beta = 0.01$</p>
	

Table 3 – Strut-and-Tie model for variant 3

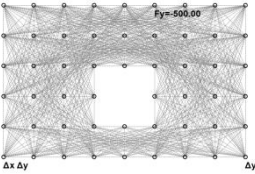
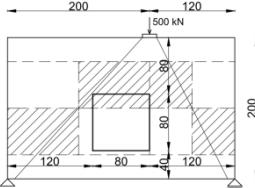
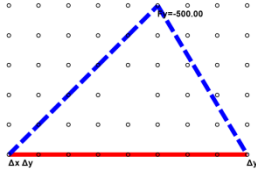
Variant 3	
	<p>C30/37, B500B Compressed element: $\beta = 1$ Tensioned element: (0°) $\beta = 1$; (45°) $\beta = 1$; (90°) $\beta = 1$; (other angles) $\beta = 0.01$</p>
	

Table 4 – Strut-and-Tie model for variant 4

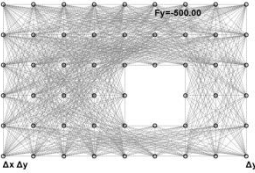
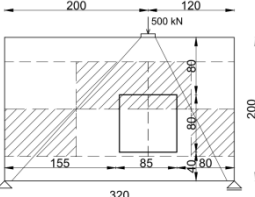
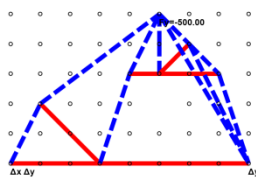
Variant 4	
	<p>C30/37, B500B Compressed element: $\beta = 1$ Tensioned element: (0°) $\beta = 1$; (45°) $\beta = 1$; (90°) $\beta = 1$; (other angles) $\beta = 0.01$</p>
	

Table 5 – Strut-and-Tie model for variant 5

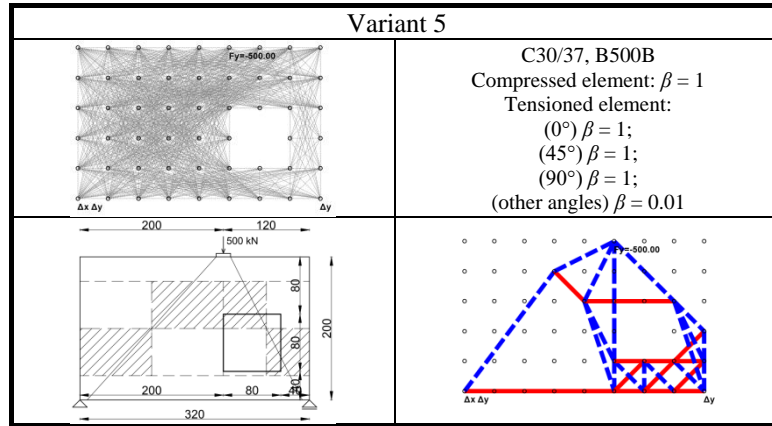


Table 6 – Mid span bottom edge vertical displacement



Based on the analysis, it can be concluded that, from an engineering standpoint, variant 3 is optimal, due to the fact that it requires the simplest form of the Strut-and-tie model in comparison to other variants, and that includes the simplest scheme of the member reinforcement. This behaviour is affected by the opening that is not placed in the direction of the natural load path from the load bearing to the support bearing blocks. In this case, the member can be analyzed as full. More complex forms of the Strut- and-Tie models in versions 1, 2, 4 and 5 are the result of opening positions in the loaded part of the member shear zones. The same conclusion can be reached on the basis of the vertical displacement analysis for the mid span bottom edge of the member. In the case of variant 3, in which the location of the opening is outside the member shear zone, the value of the vertical displacement is smaller, from 6.8 to 78.4%, as compared to other variants.

4 CONCLUSION

The usual way for dimensioning is not applicable, due to stress discontinuity, for the reinforced concrete wall members with complex geometry, with one or more openings. In such cases, the problem can be solved using the Strut-and-Tie method, which replaces the real member with the equivalent truss system where compressed elements replace the concrete parts of the member and tensioned elements replace reinforcement.

One of the parameters that significantly influences the behaviour of deep beams with openings is the location of the opening, whose influence is analyzed in this paper. When the openings are placed in the member shear zones, they can reduce the shear resistance of the element [15].

Based on the analysis of the results of the reinforced concrete deep beam, where the location of the opening is varied for the usual directions of the reinforcement, it can be concluded that variant 3 is optimal in terms of the simplicity of the Strut-and-Tie model. Its form and consequences present the simplest scheme of the reinforcement layout. More complex forms of the Strut- and-Tie models in variants 1, 2, 4 and 5 are the result of the position of the opening in the loaded member shear zones. Also, according to the analysis of the vertical displacement for the mid span bottom edge of the member, it can be concluded that, in the case of variant 3 where the opening location is outside the member shear zone, the lowest value of the vertical displacement is obtained.

The obtained results show that the openings should be preferably placed outside the natural load path, since in this case the member acts as full, has a greater capacity, and, in terms of serviceability, smaller vertical displacement values occur.

ACKNOWLEDGEMENTS

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Branko Milosavljević¹, Drago Ostojić²

PRIMER OJAČANJA ARMIRANOBETONSKOG STUBA DOBETONIRANJEM U ČELIČNOJ CEVI

Rezime:

S-6

U radu je prikazan primer proračuna ojačanja pretežno aksijalno napregnutog kvadratnog stuba dobetoniranjem u kružnoj čeličnoj cevi. Prikazan je proračun efekata utezanja na povećanje čvrstoće betona na pritisak. Razmatrani su uslovi vezani za nosivost i maksimalne napone pritiska, propisani u tehničkoj regulativi, sa aspekta primenjivosti na ovakvo rešenje ojačanja. Prikazan je proračun efekata utezanja betona za dva prečnika čelične cevi. Razmotrene su prednosti i nedostaci ovakvog rešenja sa aspekta kvalitetnog ugrađivanja betona.

Ključne reči: ojačanje, kružni stub, čelična cev, utezanje preseka

STRENGTHENING OF THE RC CONCRETE COLUMN BY CONCRETE SUPPLEMENT IN STEEL TUBE EXAMPLE

Summary:

An example of the design of strengthening of the predominantly axially loaded reinforced rectangular concrete column by concrete supplement in steel tube is presented in this paper. Calculation of the confining effects on the increase of concrete compressive strength is presented. Technical regulations related to bearing capacity and compressive stress limitation, in the scope of their applicability on presented strengthening solution, are considered. The confining effect was presented for two different steel tube diameters. Advantages and disadvantages of presented solution in the scope of the concreting quality are considered.

Key words: strengthening, circular column, steel tube, cross-section confining

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1 UVOD

Dogradnja postojećih objekata dodavanjem jednog ili više spratova može, kao što je poznato, dovesti do značajnog povećanja vertikalnih sila u nosećim elementima koji sprovedu opterećenje do temelja. Na to su naročito osetljivi stubovi, gde i pri manjim povećanjima opterećenja može doći do prevelikog napona pritiska u eksploataciji, kao i iscrpljenja nosivosti postojećeg stuba. S tim u vezi je često neophodno pristupiti projektovanju i izvođenju ojačanja postojećih stubova u objektu nad kojim se vrši dogradnja.

Poznate su razne metode ojačanja stubova, dodavanjem armiranobetonskih ili čeličnih delova preseka, uz različito njihovo međusobno povezivanje i sprezanje, ojačanje stuba čeličnim profilima na uglovima koji su međusobno povezani flahovima, sa dodavanjem sloja torkret ili ugrađenog betona, itd.

U ovom radu je prikazan jedan primer ojačanja kvadratnog stuba, dobetoniranjem kvadratnog stuba u čeličnoj cevi, tako da se dobije stub kružnog poprečnog preseka. Cev je formirana od relativno tankog lima, i nije projektovana da direktno doprinese nosivosti stuba na vertikalne sile, već, osim što služi kao oplata u fazi betoniranja, doprinosi značajnijem bočnom utezanju postojećeg i dobetoniranog dela preseka stuba. Ovakav način ojačanja stuba nije, naravno, nepoznat u građevinskoj praksi. Projektanta čelični plašt oko kružnog preseka asocira na "veoma dobro" utegnuto presečenje, što dovodi do "značajnog" poboljšanja karakteristika armiranobetonskog preseka ojačanog stuba. Može se postaviti pitanje kvantifikacije tog poboljšanja, kao i uslova koje tom prilikom treba zadovoljiti.

U ovom radu je izvršen numerički proračun efekata utezanja, a zatim ti efekti analizirani sa stanovišta ograničenja naprezanja i nosivosti postojećeg stuba u uslovima povećanog opterećenja prema još uvek važećem BAB87 i Evrokodu.

2 USVOJENO REŠENJE OJAČANJA STUBA

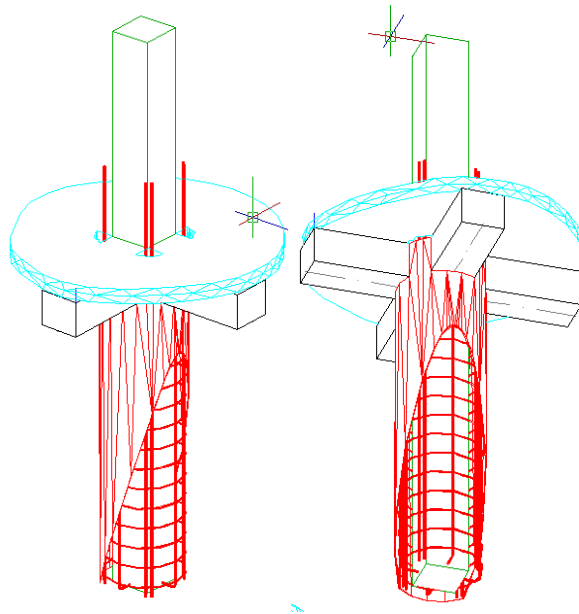
Rešenje ojačanja stuba koje je prikazano u ovom radu se odnosi na slučaj dogradnje postojećeg četvorospratnog objekta sa skeletnom ramovskom armiranobetonskom konstrukcijom. Stubovi si dimenzija 40x40cm, osim na najnižem spratu, gde je dimenzija 40x50cm. Dimenzije greda su 40x50cm. Međuspratne konstrukcije su krstaste armirano betonske ploče debljine 15cm osonjene na grede. Ramovi su dakle postavljeni u dva pravca, u rasteru 6.0x6.0m. Objekat se dograđuje za još četiri sprata, uz odgovarajuće ojačanje temelja i formiranje novog sistema konstrukcije za prihvat horizontalnih sila dodavanjem armiranobetonskih zidova u dva pravca, koji zajedno sa postojećim ramovima čine mešoviti sistem.

S obzirom na broj spratova koji se u konkretnom slučaju dodaju, i konsekventno povećanje vertikalne sile u stubovima, neophodno je ojačanje stubova postojeće konstrukcije. Ovde je prikazano ojačanje jednog srednjeg stuba, kod koga je povećanje sile usled dogradnje najviše izraženo.

2.1 DISPOZICIJA I DETALJI OJAČANJA

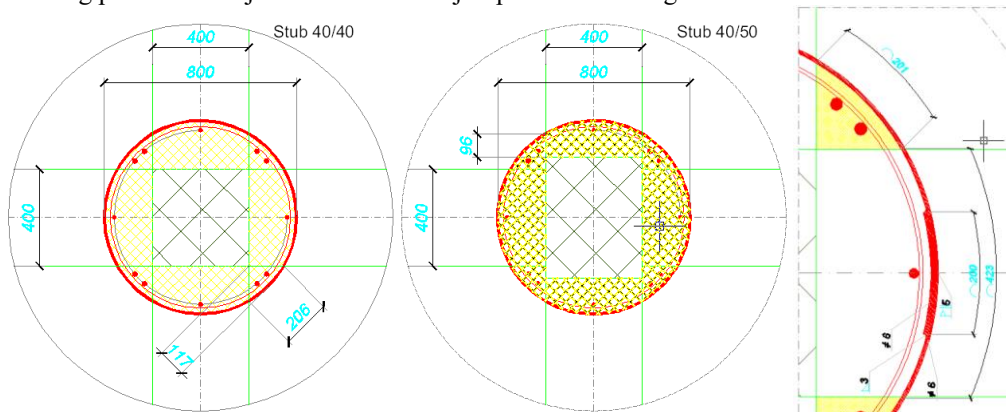
Predviđeno je da se ojačanje srednjeg stuba konstrukcije izvede nakon što se sa postojeće konstrukcije uklone pregradni zidovi, podovi i plafoni. U neposrednoj blizini stuba, u uglu između međuspratnih greda koje se susiće, ploča se štemovanjem probija. Oko postojećeg stuba se postavlja novoprojektovana podužna i poprečna armatura (*Slika 1*). Podužna armatura

je postavljena tako da je njen najveći deo postavljen na mestu gde je probijen otvor u ploči (Slika 2 levo). Time je omogućena kontinualizacija podužne armature kroz spratove preklapanjem.



Slika 1 - Dispozicija ojačanja stuba

Nakon postavljanja uzengija $\text{Ø}10/20\text{cm}$, postavlja se čelični cilindrični plašt, i to iz dva dela koji se spajaju u cev prečnika 80cm zavarivanjem prema detalju (Slika 2 desno). Elementi čeličnog plašta su ukrojeni tako da zatvaraju i prostor između greda.



Slika 2 - Detalji ojačanja stuba

Nakon zatvaranja čeličnog plašta, vrši se betoniranje prostora između postojećeg stuba i čeličnog plašta sitnozrnim betonom kroz otvore probijene u ploči, korišćenjem levka za

nadvišenje sveže betonske mešavine i vibriranjem čelične cevi, kako bi se obezbedilo potpuno popunjavanje oplata betonom. Potpuno ispunjavanje prostora koji se betonira je od posebnog značaja, kako bi se obezbedio prenos sile pritiska sa sprata na sprat kroz dobetonirani deo stuba.

2.2 PRORAČUN NAPREZANJA OJAČANOG STUBA

Na osnovu proračunskog 3D modela za nedograđenu konstrukciju (postojeće stanje) i dograđenu konstrukciju (novoprojektovano stanje) sračunata su vertikalna naprezanja u najopterećenijem srednjem stubu, za koji se sprovodi proračun ojačanja.

U cilju analize efekata dodatnog utezanja stuba koji se ojačava, ovde će biti razmotrena dva rešenja, jedno sa dobetoniranim stubom do prečnika od 80cm, i drugo do prečnika od 75cm. Tako će analiza biti sprovedena za dva nivoa konačnog opterećenja, uz razmatranje uslova vezanih za maksimalne napone i nosivost utegnuto i neutegnuto preseka postojećeg stuba.

Već je napomenuto da se u konkretnom slučaju, u okviru projekta dogradnje konstrukcije, projektuju nova armiranobetonska platna, tako da ona preuzimaju veći deo uticaja od horizontalnih opterećenja. Stubovi i grede, svakako prihvataju deo horizontalnih sila, ali s obzirom na krutost armiranobetonskih zidova, ti uticaji su relativno mali. U prikazanoj analizi akcenat se stavlja na vertikalne, aksijalne sile u stubu, uz odgovarajuće razmatranje istovremenog uticaja savijanja.

2.2.1 Ograničenja naprezanja i nosivosti stuba

Prema Evrokodu, osim zadovoljenja nosivosti poprečnog preseka stuba za dejstva normalne sile i momenta savijanja u uslovima graničnog stanja (ULS), za granično stanje upotrebljivosti (SLS) naponi pritiska u betonu su ograničeni na vrednost /3/:

$$0.6f_{ck} \quad (1)$$

kada je u pitanju ograničenje pojave podužnih prslina, i

$$0.45f_{ck} \quad (2)$$

kada može da se usvoji linearno tečenje betona, gde je:

f_{ck} karakteristična čvrstoća betona na pritisak (cilindra).

S druge strane, EC8 ograničava normalnu silu /2/, za srednju klasu duktilnosti (DCM), na nivo od:

$$\nu_{ed} = \frac{N_{ed}}{A_c f_{cd}} \leq 0.65 \quad (3)$$

gde je:

N_{ed} proračunska aksijalna sila u stubu,

A_c površina betonskog preseka,

f_{cd} računski čvrstoća betona na pritisak.

Prema još uvek važećem "Pravilniku o tehničkim normativima za izgradnju objekata visokogradnje u seizmičkim propisima", za stubove u ramovskim konstrukcijama, napon od vertikalnog opterećenja se ograničava na:

$$\sigma \leq 0.35 \times 0.7 \times \beta_k \quad (4)$$

gde je:

β_k čvrstoća kocke d=20cm (marka betona).

2.2.2 Efekat poprečnog utezanja na čvrstoću betona

Sva navedena ograničenja prikazana u prethodnoj tački, zavise od čvrstoće betona na pritisak. U literaturi postoji veliki broj objavljenih istraživanja karakterisitka i ponašanja utegnutoг betona. Tako se, na osnovu /1/, može prikazati zavisnost povećanja čvrstoće usled utezanja poprečnom armaturom:

$$f_{cck} = f_{ck} \cdot \left[-1.254 + 2.254 \sqrt{1 + \frac{7.94 \cdot f_p}{f_{ck}}} - 2 \frac{f_p}{f_{ck}} \right] \quad (5)$$

gde je:

f_p poprečni napon od utezanja,
 f_{ck} karakteristična čvrstoća utegnutoг betona.

Poprečni napon od utezanja dat je izrazom:

$$f_p = \frac{1}{2} \omega_{wd} \cdot f_y \quad (6)$$

gde je:

f_y napon tečenja u čeliku kojim se presek uteže,
 ω_{wd} zapreminski koeficijent armiranja poprečnom armaturom, koji je, za slučaj utezanja limom debljine t , definisan izrazom:

$$\omega_{wd} = \frac{4t}{D} \quad (7)$$

gde je:

D prečnik čelične cevi.

Za konkretni slučaj ojačanja stuba, marka ugrađenog betona kod postojećeg i novoprojektovanog dela stuba je MB30, odnosno C25/30, prema EC2. U Tabela 1 su prikazani rezultati proračuna čvrstoće na pritisak betona utegnutoг čeličnom cevi debljine zida 6mm, prema BAB, za marku MB30 i prema EC, za klasu betona C25/30, a prema izrazima (5) do (7). Proračun je sproveden za dva prečnika stuba, 80 i 75 cm.

Tabela 1 - Čvrstoće utegnutoг betona

			Prečnik stuba D	80	75	cm	
$MB=$	30	MPa	$D_{ef} =$	79.4	74.4	cm	
$f_B =$	20.5	MPa	$\omega =$	0.030	0.032		
$0.35f_B =$	7.35	MPa	$f_p =$	3.552	3.790	MPa	
$C25/30$			Čvrstoća prema BAB	$f_p/MB =$	0.118	0.126	
$f_{ck} =$	25	MPa		$f_{cc}/MB =$	1.649	1.683	
$f_{cd} =$	16.7	MPa		$f_{ccBAB} =$	49.460	50.504	MPa
$0.45f_{ck} =$	11.3	MPa	Čvrstoća prema EC	$f_p/f_{ck} =$	0.142	0.152	
$0.6f_{ck} =$	15.0	MPa		$f_{cc}/f_{ck} =$	1.750	1.789	
$f_y =$	235	MPa		$f_{cckEC} =$	43.748	44.722	MPa
$t_{lima} =$	0.6	cm		$f_{ccEC,d} =$	29.165	29.815	MPa

Povećanje čvrstoće betona pri troosnom stanju napreznja je definisano u EC2 (3.1.9 (2)), gde je za vrednost bočnog napona $\sigma_2 = \sigma_3 \geq 0,05f_{ck}$, uvećana karakteristična čvrstoća data je izrazom:

$$f_{ck,c} = f_{ck} (1.125 + 2.5\sigma_2 / f_{ck}) \quad (8)$$

što, za utezanje u cevi debljine 6mm, prečnika 80cm, sa $\sigma_2 = f_p = 3.552\text{MPa}$ (Tabela 1), iznosi:

$$f_{ck,c} = 1.6f_{ck} \quad (9)$$

dakle, nešto na strani sigurnosti u odnosu na vrednosti iz Tabela 1.

U Tabela 2 prikazane su veličine sila u predmetnom stubu po fazama (F1- postojeće stanje, F2 - dograđen objekat) i po opterećenjima g i p. Kolona 4 predstavlja razliku kolona 3 i 1, a kolona 6 zbir kolona 4 i 5.

Tabela 2 – Sile u stubu koji se ojačava

Sprat	F1: Postojeće		F2: Dogradnja			
	N_{F1g}	$\sigma_{un,F1}$	$N_{F2g,tot}$	N_{F2g}	N_{F2p}	$N_{max,F2}$
	kN	MPa	kN	kN	kN	kN
	1	2	3	4	5	6
Suteren	1190	6.0	3684	2494	820	3314
Prizemlje	954	6.0	3295	2341	738	3079
1	674	4.2	3294	2620	659	3279
2	448	2.8	2585	2137	559	2696
3	228	1.4	2196	1968	459	2427

2.2.3 Nosivost ojačanog stuba

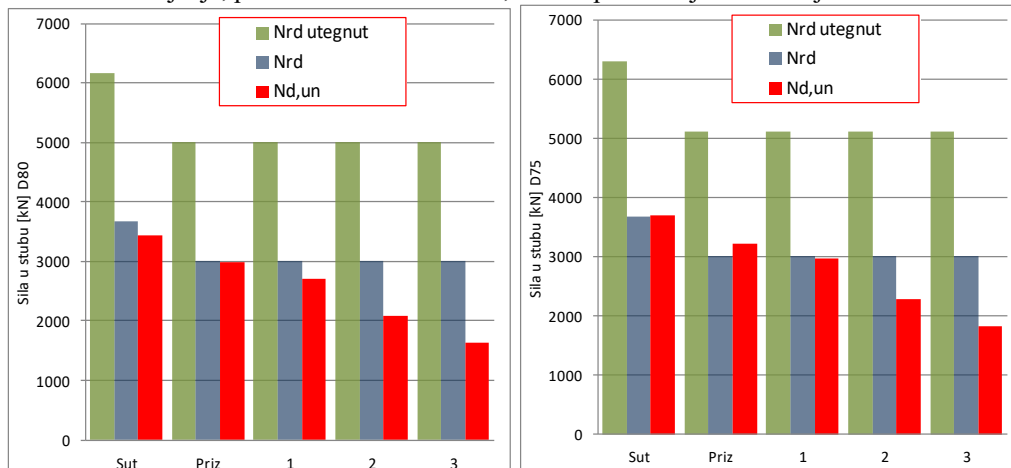
Prikazane su proračunske granične vrednosti sila u stubu N_d prema EC2, upoređene sa nosivošću stuba N_{rd} pri centričnom pritisku, za unutrašnji (postojeći) i spoljni (novoprojektovani) deo stuba (Tabela 3). Raspodela sile na dva dela preseka stuba izvršena je prema pripadajućim površinama. Proračunska sila u unutrašnjem delu $N_{d,un}$ (kol. 4 i 11) dobijena je sabiranjem $1,35 \times N_{F1g}$ (Tabela 2, kol. 1) i $N_{d,unF2}$ (kol. 3 i 10).

Tabela 3 – Granične sile i nosivost ojačanog stuba (kN)

Sprat	D=80cm									D=75cm						
	N_d	$N_{d,F2}$	Unutrašnji				Spoljni			Unutrašnji				Spoljni		
			$N_{d,un,F2}$	$N_{d,un}$	N_{rd}	$N_{rd,UT}$	$N_{d,sp,F2}$	N_{rd}	$N_{rd,UT}$	$N_{d,un,F2}$	$N_{d,un}$	N_{rd}	$N_{rd,UT}$	$N_{d,sp,F2}$	N_{rd}	$N_{rd,UT}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	
Sut.	6203	4597	1830	3436	3668	6168	2767	6132	9912	2082	3689	3668	6298	2515	5118	8294
Priz.	5555	4267	1699	2987	3001	5001	2569	6799	11078	1933	3221	3001	5105	2335	5785	9487
1	5435	4526	1802	2711	3001	5001	2724	6799	11078	2050	2960	3001	5105	2476	5785	9487
2	4328	3723	1482	2087	3001	5001	2241	6799	11078	1686	2291	3001	5105	2037	5785	9487
3	3653	3345	1332	1640	3001	5001	2014	6799	11078	1515	1823	3001	5105	1830	5785	9487

Kao što je već napomenuto, stub je deo ramovske konstrukcije koja, u sistemi sa AB zidovima, prihvata i deo momenata savijanja, pre svega od horizontalnih dejstava. Maksimalni moment u stubu iznosi 304kNm , što predstavlja relativno malu vrednost u odnosu na ukupnu graničnu aksijalnu silu (Tabela 3, kol. 1), i može se zanemeriti u ovoj analizi. Ukoliko se

posmatraju pojedini delovi preseka, unutrašnji deo stuba, posmatrano prema pripadajućoj krutosti na savijanje, prihvata 10.6% momenta, što tek predstavlja zanemarljivu vrednost.



Slika 3 - Nosivost unutrašnjeg dela stuba za $D=80\text{cm}$ (levo) i $D=75\text{cm}$ (desno)

Nosivost spoljnog dela peseka je značajno iznad odgovarajuće proračunske sile. Na dijagramima (Slika 3) je prikazana granična sila za unutrašnji deo preseka, kao i odgovarajuća nosivost za neutegnut i utegnut beton, za dva razmatrana prečnika stuba. Kod stuba prečnika 75cm u unutrašnjem delu preseka sila premašuje nosivosti u prva dva sprata, gde je neophodno uvesti efekte utezanja betona. U oba slučaja, s obzirom na iscrpljenu ili skoro iscrpljenu nosivost unutrašnjeg dela, uvođenje značajnijih momenata savijanja, na primer, u slučaju dominantno ramovske konstrukcije, bilo bi nemoguće bez pozitivnih efekata utezanja na nosivost ojačanog stuba, kao i duktilnost krivine preseka /4/.

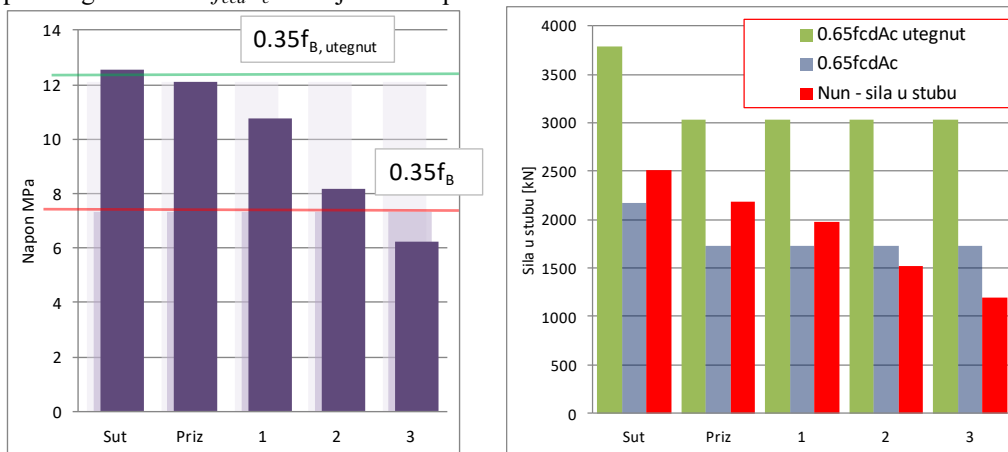
2.2.4 SLS sile i naponi u stubu

Eksploatacione sile i naponi u unutrašnjem i spoljnom delu stuba prečnika 80cm su prikazani u Tabeli 4. Treba naglasiti da ukupna sila u unutrašnjem stubu (kolona 4) je dobijena sabiranjem postojeće sile u stubu (pre početka dogradnje, Tabela 2) i njegove pripadajuće sile od dodatnog opterećenja usled dogradnje (Tabela 4, kolona 1).

Tabela 4 – SLS sile i naponi u stubu

Sprat	Unutrašnji deo stuba						Spoljni deo stuba			
	$N_{un,F2}$	$\sigma_{un,F2}$	N_{un}	σ_{un}	$0.65f_{cd}A_c$	$0.65f_{ccd}A_c$	N_{sp}	σ_{sp}	$0.65f_{cd}A_c$	$0.65f_{ccd}A_c$
	kN	MPa	kN	MPa	kN	kN	kN	MPa	kN	kN
	1	2	3	4	5	6	7	8	9	10
Sut.	1319	6.6	2509	12.5	2167	3792	1995	6.6	3276	5733
Priz.	1226	6.1	2180	12.1	1733	3033	1853	5.4	3709	6491
1	1305	6.5	1979	10.7	1733	3033	1974	5.8	3709	6491
2	1073	5.4	1521	8.2	1733	3033	1623	4.7	3709	6491
3	966	4.8	1194	6.3	1733	3033	1461	4.3	3709	6491

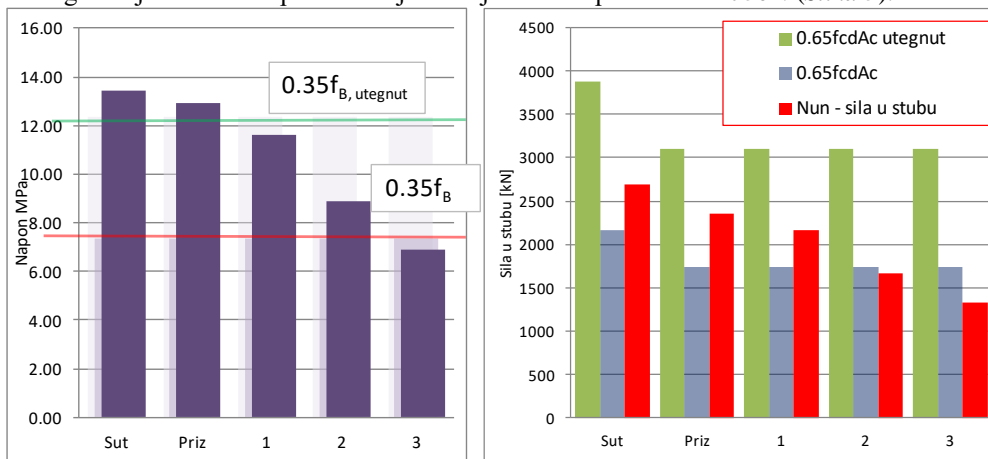
Naponi i odgovarajuće sile u unutrašnjem delu stuba su prikazani na dijagramima (Slika 4), gde se na dijagramu levo vidi da, za neutegnut beton, naponi prevazilaze granicu iz uslova (4) od $0.7f_B=7.35MPa$ (Tabela 1). Pri utezanju betona čeličnim plaštom ova granica se uvećava odnosom čvrstoća utegnuto i neutegnuto betona, koja u ovom slučaju u iznosu 1.649 (Tabela 1), pa napon u unutrašnjem utegnutom stubu zadovoljava uslov (4). Na desnom dijagramu prikazan je uslov (3), koji za prva tri sprata nije ispunjen za neutegnut beton. Efekat utezanja podiže granicu $0.65f_{cd}A_c$ značajno iznad potrebnih vrednosti.



Slika 4 - Normalni napon pritiska i sila u ojačanom stubu $D=80cm$

Uslovi ograničenja napona (1) i (2), propisani u EC2, se ne pokazuju kao merodavni. Uslov $0.6f_{ek}$ daje relativno visoku vrednost napona, a uslov (2) nije od značaja za unutrašnji deo stuba zbog njegove starosti, kada tečenje betona dostiže asimptotu.

Odgovarajuća analiza sprovedena je i za ojačan stub prečnika $D=75cm$ (Slika 5).



Slika 5 - Normalni napon pritiska i sila u ojačanom stubu $D=75cm$

U ovom slučaju zbog većeg udela unutrašnjeg dela stuba u preraspodeli sila u ojačanom preseku, za neutegnut beton uslov (4) nije ispunjen praktično po celoj visini, a uslov (3) na prva tri sprata. Uvođenje efekata utezanja nije, u ovom slučaju, dovoljno da se ispuni uslov (4), kao što se vidi na levom dijagramu (*Slika 5*). Uslov ograničenje sile pema EC8 je zadovoljen kada se uzme uticaj poprečnog utezanja u obzir.

2.2.5 Analiza rezultata proračuna

Prikazano ojačanje stuba obuhvata i proračun odnosa čvrstoće neutegnutog i utegnutog betona (Tabela 1), odnosno kvantifikaciju efikasnosti utezanja. Efekat utezanja je direktno proporcionalan, kao što je poznato, debljini čelične cevi, a obrnuto proporcionalan njenom prečniku. Treba napomenuti da se i kružne uzengije usvojene u preseku mogu uključiti površinu čelika koja uteže beton. U ovom primeru je to zanemareno, jer bi uključenje usvojenih uzengija Ø10/20, u ovom slučaju, efektu utezanja doprinelo manje od 10%.

U primeru je razmatrana nosivost ojačanog stuba u celini, kao i unutrašnjeg (postojećeg) dela preseka, samo za aksijalnu silu pritiska, što je, u ovom slučaju, opravdano zbog zanemarljivih momenata savijanja, s obzirom da u konstrukciji postoje i AB zidovi. Generalno, ukoliko savijanje nije zanemarljivo, efekat utezanja je još značajniji, jer osim nosivosti treba obezbediti i duktilnost krivine. Treba pritom imati u vidu, da zbog usvojene dispozicije preseka, gde se postojeći stub nalazi unutar novoprojektovanog preseka, udeo unutrašnjeg preseka u nošenju i većih momenata savijanja, je relativno mali.

Uslov (1) koji predstavlja ograničenje vertikalnih napona se svakako mora ispuniti. Ovaj uslov dobija na značaju kada je unutrašnji deo preseka više opterećen normalnom silom od spoljnog, što je može desiti u slučaju velikih "zarobljenih" sila, ili kada je relativno mala površina dobetoniranog dela preseka, što, s druge strane, može kompromitovati pravilno ugrađivanje betona. U slučaju ojačanja postojećih stubova uslov (2), koji se odnosi na tečenje, nije od značaja, kao što je već naglašeno.

U primeru je razmatrano ispunjenje uslova (3) i (4). Uslov ograničenja sile (3), koji propisuje EC8, merodavan je kod ramovskih konstrukcija, i pre svega je usmeren na obezbeđenje projektovane duktilnosti, za stubove sa značajnim momentima savijanja, ili kod konstrukcija sa velikim horizontalnim pomeranjima. Uslov (4), iz naših važećih propisa, ima praktično isti smisao kao i uslov (3). Dakle, ako stub koji se ojačava predstavlja sekundarni element u sistemu koji prihvata horizontalne uticaje, ne mora nužno da ispunjava ova dva uslova. Ovo naročito dobija na značaju kada se uzme u obzir, već opisano, relativno malo učešće postojećeg stuba u nosivosti ojačanog preseka na savijanja.

2.3 TEHNOLOŠKI ASPEKTI PRIKAZANOG REŠENJA OJAČANJA STUBA

Pri opisu usvojenog rešenja ojačanja stuba je naglašeno da čelična cev, formirana iz dva zakrivljena lima, pored funkcije utezanja preseka, predstavlja i oplatu za betoniranje, što je pogodnost sa tehnološkog aspekta. Međutim sa aspekta tehnologije betoniranja i kontrole kvalitete ugrađenog betona, predloženo rešenje zahteva posebnu pažnju.

Problem pri betoniranju i ugrađivanju betona predstavlja mali prostor između površine stuba i čelične cevi. Težnja projektanta da primeni najmanje moguće proširenje stuba, uz zadovoljenje nosivosti, u direktnoj je suprotnosti sa potrebom da se sprovede kvalitetno ugrađivanje betona. Svakako treba težiti ka balansu između ove dve težnje, uz korišćenje sitnozrog ili samougradivog (SCC) betona.

Prilikom betoniranja beton se u cev može unositi kroz otvore u ploči, kao što je već opisano, gde je kritično normalno rastojanje od ivice stuba do zida cevi. Za rešenje ojačanja stuba 40x40cm do cevi $D=80\text{cm}$ to rastojanje iznosi 10.6cm , za $D=75\text{cm}$ 9.2cm , a za $D=70\text{cm}$ 6.7cm . S obzirom na prisutnu podužnu armaturu u prostoru za betoniranje, u poslednja dva slučaja bi se morao, na primer, konično proširiti gornji deo stuba, ili vršiti betoniranje SCC betonom odozdo na gore.

Kada se oplata od lima zadržava kao deo preseka posle betoniranja, nemoguće je izvršiti vizuelnu kontrolu kvaliteta betoniranja. Već je ukazano na značaj dobro ugrađenog betona za transfer sila između spratova, što ponovo ističe neophodnost primene većih preseka cevi, SCC betona, kao i eventualnu kontrolu ugrađenog betona ultrazvukom.

3 ZAKLJUČAK

Prilikom dogradnje ramovskih konstrukcija koje podrazumevaju ojačanje stubova, prosto proširenje poprečnog preseka stuba nije dovoljno, jer su naprezanja u postojećem stubu, nastala usled "zarobljene" sile pre početka dogradnje i dodate sile od težine dograđenog dela objekta i dodatnog stalnog i povremenog opterećenja, često su, sa aspekta nosivosti i ograničenja napona pritiska, iznad odgovarajućih propisanih granica. Ukoliko su "zarobljene" sile u postojećem stubu velike, površina dodatog dela preseka, odnosno dimenzije ojačanog stuba postaju veoma velike. Poboljšanje karakteristika, kako postojećeg, tako i novougrađenog betona, efikasnim poprečnim utezanjem, predstavlja u takvim slučajevima dobro rešenje. Poznato je da je najefikasnije utezanje kod kružnog poprečnog preseka, a prisustvo čelične cevi ili plašta oko preseka obezbeđuje dovoljnu količinu čelika za utezanje.

Uslovi vezani za ograničenje sile i napona u postojećem stubu, kao unutrašnjem delu novoprojektovanog ojačanog elementa, zavise, pre svega, od uloge koju ojačani stub ima u konstrukciji. Što je veće njegovo učešće, u prihvatanju horizontalnih sila, odnosno što su veći uticaji savijanja, to raste potreba sa efikasnim utezanjem stuba, u cilju povećanja čvrstoće betona i duktilnosti elementa u celini.

Proračunom, na način koji je prikazan u radu, moguće je kvantifikovati efekte utezanja i, uz pravilnu analizu uslova koje po naponima i nosivosti ojačani stub treba da zadovolji, obezbediti sve proračunske dokaze u okviru projekta ojačanja konstrukcije.

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PRORAČUN PRETHODNO NAPREGNUTIH KONSTRUKCIЈA PREMA EVROKODOVIMA

Rezime:

S-7

U tekstu su prikazane preporuke Evrokoda 2 (EN1992-1-1) za proračun prethodno napregnutih konstrukcija. Proračunski zahtevi su upoređeni sa odredbama važećeg „Pravilnika o tehničkim merama i uslovima za prednapregnuti beton“ iz 1971. godine. Ukazano je na važnije razlike u instrukcijama ovih pravilnika u domenu proračuna granične nosivosti i kontrolama predviđenim za granično stanje upotrebljivosti.

Ključne reči: prethodno napregnuti beton, Evrokod 2, proračun

PRESTRESSED CONCRETE DESIGN TO EUROCODES

Summary:

Design recommendations of Eurocode 2 (EN1992-1-1) regarding prestressed concrete structures are reviewed in the paper. Requirements are compared with Serbian code for prestressed structures „Pravilnik o tehničkim merama i uslovima za prednapregnuti beton“ (1971.). Major differences between Eurocode 2 and Serbian code concerning ultimate limit state design and serviceability checks are commented.

Key words: prestressed concrete, Eurocode 2, design

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1 UVOD

U osnovi projektovanja i proračuna betonskih konstrukcija prema Evrokodovima nalazi se **koncept graničnih stanja**. Ona su podeljena, kao što je uobičajeno, u dve grupe: proračuni u oblasti granične nosivosti (*ULS*) i proračuni u stanju eksploatacije (*SLS*).

Granična nosivost (*ULS*) označava iscrpljivanje nosivosti konstrukcije pri određenom nivou graničnog opterećenja. Potrebna sigurnost izražena je odnosom veličina graničnog i maksimalnog eksploatacionog opterećenja. „Isrpljivanje nosivosti“ može da označava iscrpljivanje kapaciteta upotrebljenih konstrukcijskih materijala, što se uobičajeno kod nas povezuje sa ovom vrstom proračuna, ali i gubitak stabilnosti čitave konstrukcije ili njenog dela, prelazak u mehanizam, otkaz zbog zamora i slično. To su, u stvari, pojednostavljeni računski modeli stanja koja prethode kolapsu konstrukcije.

Proračuni pod eksploatacionim opterećenjem (*SLS*) imaju za cilj da obezbede raznovrsne aspekte funkcionalnosti i trajnosti konstrukcija u predviđenom eksploatacionom veku. To uključuje ograničavanje deformacija, vibracija, nastanka i širine prslina. Za ove vrste provera koristi se nekoliko „nivoa“ eksploatacionih opterećenja. Na primer, pod maksimalnim eksploatacionim opterećenjima sprovode se provere stanja čiji i jednokratni nastanak izaziva trajnu posledicu („štetu“) na konstrukciji.

Obe vrste proračuna (*ULS*, *SLS*) su **već prisutne** u domaćoj projektantskoj praksi. Koncept graničnih stanja zastupljen u Evrokodovima je, međutim, dosta širi i doslednije sproveden.

2 OSNOVNI PRISTUP

Aktuelno stanje domaće regulative su **zasebni pravilnici za armirani i prethodno napregnuti beton**, različite generacije i različite osnovne koncepcije. Pravilnik BAB 87 [1] je usmeren na koncept graničnih stanja, sa proračunom prema graničnoj nosivosti kao osnovnim alatom za određivanje dimenzija elemenata nosećih konstrukcija.

U osnovi, važeći „Pravilnik o tehničkim merama i uslovima za prednapregnuti beton“ [2] iz 1971. godine **polazi od** proračuna i **ograničavanja napona** u eksploatacionim uslovima, kao alata za projektovanje prethodno napregnutih konstrukcija od betona. Pritom su ograničenja napona postavljena tako da se maksimalno ili u najvećoj meri **izbegne nastajanja prslina**, koje su, pre svega, povezivane sa uvećanim rizikom po nosivost samih konstrukcija. Proračun prema graničnoj nosivosti ostao je, iako obavezna stavka, objektivno u drugom planu. Tome su doprinele dve činjenice. Prvo, u vreme donošenja Pravilnika, koncept proračuna prema graničnoj nosivosti na našim prostorima bio je tek na početku uvođenja u regulativu, tako da je broj samih instrukcija u Pravilniku mali. Drugo, iako je kasniji razvoj regulative izražen kroz Pravilnik BAB 87 [1] stvorio osnovne uslove, šira primena projektovanja prema graničnoj nosivosti u oblasti prethodno napregnutih elemenata nije se dogodila na našim prostorima. To je posledica iskustvene činjenice: elementi projektovani prema dopuštenim naponima redovno su iskazivali dovoljnu graničnu nosivost; obrnuti put – od granične nosivosti ka kontroli napona često nije davao zadovoljavajući rezultat u celini.

Evrokod 2 [3] u suštini **ne pravi razliku između armiranobetonskih i prethodno napregnutih** betonskih konstrukcija u pristupu projektovanju. Dosledno tome, pojava prslina usled zatezanja dozvoljena je i u slučaju prethodno napregnutih konstrukcija u širokom dijapazonu projektnih uslova, za razliku od našeg Pravilnika. Pritom, ograničenja u pogledu

širine prslina su ista ili umereno pooštrena u odnosu na ona koja se primenjuju na armiranobetonske konstrukcije.

Stoga, iako to nije eksplicitno deklarirano, kao **osnovni alat** za usvajanje potrebnih dimenzija elemenata nosećih konstrukcija nameće se **proračun prema graničnoj nosivosti**, a ne dimenzionisanje prema ograničavanju napona, čiji je osnovni zadatak kontrola zatezanja. **Prethodno naprezanje se posmatra kao** dugotrajno, u pogledu intenziteta veoma kontrolisano i, usled poznatih promena tokom vremena, umereno promenljivo **dejstvo**.

Postoji i drugi implicitni razlog koji u prvi plan postavlja *ULS* proračune: većina klauzula koja reguliše ove proračune daje manje-više detaljne instrukcije i ima obavezujući karakter. Kod *SLS* proračuna to uglavnom nije tako: klauzule uglavnom ne pokrivaju sve projektne situacije i često su **formulisane u obliku preporuke** ili **prepuštene nacionalnim aneksima** (*NA*) Evrokodova („nacionalnim priložima“, kako su kod nas službeno nazvani). Ova povećana „sloboda“ ne predstavlja uslugu projektantima jer ih stavlja u poziciju da moraju sami da kreiraju nedostajuće instrukcije. Pritom, projektant preuzima odgovornost za izbor dopunskih uslova i validnost projekta u smislu ispunjenja uslova iz projektnog zadatka, za razliku od aktuelne domaće prakse gde se podrazumeva da je projekat ispravan, a projektant „pokriven“ ako je urađeno u skladu sa regulativom.

2.1 PORAČUN PREMA GRANIČNOJ NOSIVOSTI (*ULS*)

Kombinacija dejstava koja uključuje prethodno naprezanje ima oblik

$$\sum \gamma_g G_k + \gamma_p P_k + \gamma_q Q_{k1} + \sum_{i \neq 1} \gamma_q (\psi_{0,i} Q_{ki}) \quad (1)$$

gde je:

γ_g koeficijent sigurnosti za stalna dejstva G_k , sa osnovnom vrednošću 1,35,

γ_p koeficijent sigurnosti za uticaje prethodnog naprezanja P_k , sa osnovnom vrednošću 1,0,

γ_q koeficijent sigurnosti za promenljiva dejstva Q_k , sa osnovnom vrednošću 1,50.

Formiranje kombinacija sledi veoma kompleksan model koji uključuje primenu načela **teorije verovatnoće, matematičke statistike i teorije pouzdanosti**. Ovaj pristup može proizvesti veoma veliki broj kombinacija za razmatranje. Najjednostavniji ishod pri proračunu bio bi $\gamma_g = 1,35$ ili 1,0 (ako deluje povoljno), $\gamma_q = 1,50$ ili 0 (izostavlja se ako deluje povoljno) i $\gamma_p = 1,0$. G_k , Q_k i P_k , predstavljaju takozvane „**karakteristične**“ vrednosti dejstava, sa preciznom statističkom definicijom, i mogu imati interval vrednosti, definisan koeficijentima r_{inf} i r_{sup} (na primer ($r_{inf} \times P_k$, $r_{sup} \times P_k$)). U najjednostavnijem slučaju, pri proračunu granične nosivosti, može se koristiti jedna karakteristična vrednost za svako dejstvo.

Za uobičajene *ULS* proračune koristi se samo „srednja“ vrednost prethodnog naprezanja P_k , ali koeficijent sigurnosti nije uvek 1,0 (na primer, za proračun ankerovanja $\gamma_p = 1,2$).

Pri kombinovanju više promenljivih dejstava (Q), drugo, i sva naredna, redukuju se statističkim faktorom $\psi_0 \leq 1,0$, koji uvodi verovatnoću istovremene pojave maksimalne vrednosti više promenljivih dejstava. Na ovaj način izbegava se istovremeno kombinovanje maksimalnih projektnih vrednosti više promenljivih dejstava Q_k , što je sada slučaj u našoj praksi. Time se mogu postići sistematske uštede, kada se utrošak materijala razmatra na nacionalnom nivou. ψ_0 i ostali ψ faktori za kombinovanje promenljivih dejstava (ψ_1 i ψ_2) dati su u Evrokodu 0 [4] i Evrokodu 1 [5].

Kod armiranobetonskih konstrukcija, primena principa iskazanog relacijom (1) za određivanje graničnih uticaja, zajedno sa dimenzionisanjem koje podrazumeva upotrebu i

parcijalnih koeficijenata sigurnosti za materijale ($\gamma_c = 1,50$, za beton, i $\gamma_s = 1,15$, za armaturu) uglavnom vodi sličnim rezultatima za elemente napregnute na savijanje, kao i kod primene BAB 87 [1]. Pri istim inputima opterećenja i odgovarajućim kvalitetima materijala, prikladne su iste dimenzije betonskih preseka i slične površine armature. Pri dimenzionisanju na smicanje, zbog razlike u pristupu, Evrokod 2 [3] je nešto „zahtevniji“ (osiguranje armaturom počinje pri nižem nivou naprezanja i potrebno je više armature, jer se ne uzima doprinos betona).

Kod prethodno napregnutih konstrukcija, međutim, situacija je drugačija. Nije teško pokazati da se, kod konstrukcije koja ima opterećenje balansirano prethodnim naprežanjem tako da pri delovanju maksimalnog opterećenja nema napona zatezanja (potpuno prethodno naprežanje u smislu Pravilnika [2]), *ULS* proračunom prema (1) iskazuje potreba za (dodatnom) armaturom za obezbeđenje potrebne nosivosti. Osim što to deluje nelogično sa aspekta dosadašnje prakse, postavljanje veće površine armature u zonu u kojoj se ostvaruje rezerva pritiska prethodnim naprežanjem, dovodi do gubitka dela rezerve kao posledica skupljanja i tečenja betona, što je dodatno kontraproduktivno.

Kao mogućnost za otklanjanje ove nelogičnosti ponuđena je opcija da se **efekti prethodnog naprežanja uvedu kroz nosivost preseka**, analizom unutrašnjih sila. To je postupak koji se kod nas sada uobičajeno primenjuje za određivanje granične nosivosti na savijanje prethodno napregnutih preseka sa kablovima sa spojem (*bonded*). Kod kablova sa spojem naprežanja se određuju iz uslova kompatibilnosti dilatacija, čime se praktično iskazuje nosivost koja ne zahteva dodatnu armaturu u opisanom slučaju. Kod kablova bez spoja predviđeno je paušalno uvećanje napona u kablovima od 100 MPa u graničnom stanju (dato u *NA* [6]). Ovaj pristup pomaže u slučaju provere granične nosivosti na savijanje, ali se ne može primeniti na proračun na smicanje koji ostaje sa istim problemom: iako je eksploataciono opterećenje balansirano, ostaje računaska sila za osiguravanje u graničnom stanju.

Za *ULS* i za *SLS* proračune potrebno je uračunati relevantne gubitke sile prethodnog naprežanja. U ovim proračunima nema većih novina u odnosu na pristup u domaćoj praksi. U slučaju da se ne raspolože kataloškim podacima, Evrokod 2 [3] daje orijentacione vrednosti za većinu parametara potrebnih za proračun gubitaka.

2.2 PORAČUNI U STANJU EKSPLOATACIJE (*SLS*)

Pri *SLS* proračunima predviđeno je korišćenje tri vrste kombinacija opterećenja. To su:

- karakteristična (retka) kombinacija, izraz (2),

$$\sum G_k + (r_{inf} \text{ ili } r_{sup}) \times P_k + Q_{k1} + \sum_{i \neq 1} (\psi_{0,i} Q_{ki}) \quad (2)$$

- česta kombinacija, izraz (3),

$$\sum G_k + (r_{inf} \text{ ili } r_{sup}) \times P_k + \psi_{1,1} Q_{k1} + \sum_{i \neq 1} (\psi_{2,i} Q_{ki}), i \quad (3)$$

- kvazi-stalna kombinacija, izraz (4)

$$\sum G_k + (r_{inf} \text{ ili } r_{sup}) \times P_k + \sum_i (\psi_{2,i} Q_{ki}). \quad (4)$$

Nije teško primetiti da, zbog međusobnog odnosa statističkih faktora verovatnoće pojave promenljivih dejstava, koji ispunjavaju odnos $1,0 \geq \psi_0 \geq \psi_1 \geq \psi_2 \geq 0$, izrazi za kombinacije (2) – (4) definišu u stvari **tri nivoa eksploatacionog opterećenja** opadajućim redosledom. Prva,

karakteristična kombinacija, ranije se nazivala „retka“, što je naziv koji je odlično opisuje: to je situacija u kojoj je jedno promenljivo dejstvo (Q_1) dostiglo svoju maksimalnu („karakterističnu“) vrednost Q_{k1} . Ta vrednost se određuje primenom metoda matematičke statistike tako da može da bude prekoračena samo u veoma malom broju slučajeva (uobičajeni fraktil je 5%), to jest „retko“. Ovo dejstvo se u toj kombinaciji naziva „dominantno“, što je nesrećno izabran termin. Pod tim se podrazumeva da je nastupilo njegovih „5 minuta“ (to jest 95%-ni maksimum), a ne da je ono najveće od svih dejstava. Drugim rečima, ukoliko ima više promenljivih dejstava Q , svako od njih ima pravo da bude Q_1 , a preostala su u sumi (Σ). Smenjivanje promenljivih dejstava na mestu broj 1, naravno, generiše više karakterističnih kombinacija, između kojih se bira merodavna, koja u stvari označava **najviši nivo eksploatacionog opterećenja**. Ovaj princip primenjuje se i kod *ULS* proračuna prema kombinaciji (1). Kvazi-stalna kombinacija definiše stalno prisutno opterećenje na konstrukciji i sastoji se od dugotrajnih tereta i manje-više stalno pristutnog dela povremenih opterećenja. To je reprezent **najnižeg** efektivnog **nivoa eksploatacionog opterećenja**. Česta kombinacija predstavlja nivo opterećenja između kvazi-stalnog i retkog. Kod betonskih konstrukcija, uglavnom je namenjena za pooštavanje kriterijuma u odnosu na kvazi-stalno opterećenje (zahteva se da se neki uslov ispuni za ovaj nivo, umesto pri kvazi-stalnom).

Karakteristična kombinacija se primenjuje za proveru situacija pri kojima nastaju **trajne posledice** (strukturne prsline u betonu, trajna izduženja čelika). Vršiti se proveru maksimalnog napona u betonu, armaturi i čeliku za prethodno naprezanje. **Kvazi-stalna** kombinacija se koristi za **kontrolu ugiba** i **prslina** armiranobetonskih elemenata i prethodno napregnutih elemenata sa kablovima bez spoja (*unbonded*). **Česta** kombinacija je namenjena za proveru **prslina prethodno napregnutih elemenata** sa injektiranim kablovima koji prijanjaju (*bonded*).

Uticiji koji potiču od prethodnog naprezanja (P_k) u svim kombinacijama (1) – (4) određuju se sa **reprezentativnom** vrednošću sile $P_k = P_k(t)$, koja uključuje uobičajene promene usled gubitaka, i prema trenutku t na koji se proračun odnosi. Pored toga, razmatra se i variranje veličine sile, kao posledica nesavršenosti proračunskog modela i tolerancija pri izvođenju. Vrednost sile može biti u opsegu ($r_{inf} \times P_k$, $r_{sup} \times P_k$). Definisane veličine r_{inf} i r_{sup} prepušteno je **nacionalnim aneksima**. Preporučene vrednosti u Evrokodu 2 [3] su: $r_{inf} = 0,90$, $r_{sup} = 1,10$ za naknadno utegnute injektirane kablove (*bonded*), i $r_{inf} = 0,95$, $r_{sup} = 1,05$ za kablove bez prijanjanja (*unbonded*) i prethodno naprezanje na stazi, pri čemu je ostavljena mogućnost da bude $r_{inf} = r_{sup} = 1,0$ u svim slučajevima „kada su preduzete odgovarajuće mere“ (pod tim se podrazumevaju merenja sa ciljem verifikacije unete sile). Naš NA [6] je u celini usvojio ove preporuke. To ostavlja mogućnost da se koristi jedinstvena vrednost, pod uslovom da se radovi izvode pedantno, što je u slučaju prethodnog naprezanja, bar dosad, bilo uobičajeno na našim prostorima. Pristup ovom pitanju umnogome **varira među državama**. Tako, na primer, britanski NA deklariše korišćenje $r_{inf} = r_{sup} = 1,0$ u svim slučajevima, dok nemački NA isključuje mogućnost da se navedene vrednosti r_{inf} i r_{sup} zamene jedinicom u bilo kom slučaju.

SLS proračuni prema Evrokodu 2 [3] uključuju **kontrole napona, širine prslina i ugiba**. Postupak za kontrola vibracija nije obuhvaćen tekстом standarda.

2.2.1 Kontrole napona

Evrokod 2 predviđa kontrolu napona u betonu i čelicima, za sve betonske konstrukcije ([3], poglavlje 7.2), a ne samo prethodno napregnute, kako je to sada kod nas. Ove instrukcije, međutim, formulisane su tako da je faktički ostavljena mogućnost da se kontrola napona u betonu u ne malom broju slučajeva zaobiđe, tako da u celini nije jasna namera autora. U

zasebnom poglavlju (5.10.2.2), ipak, naložena je obavezna kontrola napona pritiska u betonu pri prethodnom naprežanju, u svim slučajevima. S druge strane, kontrola napona za čelike deklarirana poglavlju 7.2 data je u formi za koju bi se moglo reći da je obavezujuća.

Ograničavanju u Evrokodu 2 [3] **podleže samo napon pritiska u betonu**. Napon pritiska nakon izvršenog prethodnog naprežanja ograničen je na veličinu $0,6f_{ck}(t)$, gde je $f_{ck}(t)$ čvrstoća pri prethodnom naprežanju (klauzula 5.10.2.2(5)), sa ciljem da se izbegne formiranje mikroprrslina i narušavanje strukture betona, sa negativnim uticajem na trajnost. Nakon toga, prelazi se u „nadležnost“ poglavlja 7.2. **Ograničenje pritiska** je ponovo **$0,6f_{ck}$ za karakterističnu kombinaciju**, a f_{ck} je čvrstoća koja deklarirše klasu ($t = 28$ dana). Međutim, ograničenje se primenjuje samo na sredine koje predstavljaju klase izloženosti XD (izloženost hloridima), XS (izloženost hloridima iz morske vode) i XF (izloženost agresiji kao posledica zamrzavanja/topljenja), a za X0 (bez agresije i korozione opasnosti) i XC (korozija karbonacijom) nije obavezno. Šta više, **ograničavanje pritiska se može izbeći** ukoliko se primeni neka druga mera obezbeđenja trajnosti (uvećanje zaštitnog sloja, utezanje poprečnom armaturom).

Druga „granica“ u pogledu pritiska, koja se pominje u Evrokodu 2, je **$0,45f_{ck}$ za kvazi-stalnu kombinaciju**. Ovo nije limit pritiska, već je naznačeno da je, ukoliko napon pri kvazi-stalnom opterećenju prelazi ovu granicu, potrebno u proračun uvesti nelinearno tečenje. Efekat nelinearnog tečenja može se obuhvatiti kroz adekvatnu vrednost koeficijenta tečenja.

Napon zatezanja u betonu **ne podleže** posebnom **ograničavanju**, ukoliko nije od posebnog interesa da se izbegne nastajanje prslina (kao na primer kod rezervoara klase vodonepropusnosti 3). Pri proračunima koji uključuju analizu preseka, na primer napona ili ugiba, može da se pretpostavi da nema prslina ukoliko nije prekoračena srednja vrednost čvrstoće na zatezanje f_{ctm} . U Tabeli 1 dat je pregled granica pritiska i zatezanja prema Evrokodu 2 [3] i Pravilniku za prednapregnuti beton iz 1971. godine [2], za dve adekvatne čvrstoće – MB30 \approx C25/30 i MB45 \approx C35/45.

Tabela 1 – Pregled granica za napone u MPa

Pravilnik PNB 1971.				Evrokod 2 (EN 1992-1-1)			
Beton	pritisak pri PN	pritisak u eksp.	zatez. pri PN	$0,6f_{ck}$	$0,45f_{ck}$	f_{ctm}	Beton
MB30	15,0	13,0	2,5	15,0	11,3	2,6	C25/30
MB45	21,3	17,3	3,3	21,0	15,8	3,2	C35/45

Vrednosti u Tabeli 1 ukazuju da ograničenja pritiska pri prethodnom naprežanju u Evrokodu 2 [3] **odgovaraju dopuštenim naponima** pri prethodnom naprežanju **prema našem Pravilniku** [2]. Evrokod 2, kao što je napred objašnjeno omogućava da se „izbegne“ ograničavanje napona u eksploataciji. Ukoliko se, u sredinama u kojima je preporučeno ograničenje, projektant ipak odluči za to, granica je ponovo $0,6f_{ck}$, za karakterističnu kombinaciju. To je 15 – 20 % više nego što dopušta domaći Pravilnik [2]. S druge strane, „granica“ nelinearnog tečenja $0,45f_{ck}$ veoma dobro odgovara domaćoj granici napona u eksploataciji jer se naša granica odnosi na ukupno eksploataciono opterećenje, a $0,45f_{ck}$ na kvazi-stalno koje, generalno, obuhvata samo deo promenljivog opterećenja. **Granice** u pogledu napona **zatezanja** su praktično iste, s tim da one u domaćem Pravilniku [2] predstavljaju ograničenja, a u Evrokodu 2 [3] **samo kriterijum za analizu preseka**.

Napon u čeliku za prethodno naprezanje u Evrokodu 2 [3] podleže uobičajenim granicama – 75 % prekidne čvrstoće f_{pk} i 85 % konvencionalne granice elastičnosti $f_{p0,1k}$. Granice se odnose na napone nakon izvršenog prethodnog naprezanja (nakon početnih gubitaka). Granice za određivanje sile na presi su $0,80 f_{pk}$ i $0,90 f_{p0,1k}$. **Napon u armaturnom čeliku** ograničava se, pre svega, sa ciljem ograničavanja širine prslina. Granica je 80 % granice razvlačenja f_{yk} ($\approx \sigma_v$) i praktično je uvek zadovoljena, ako je armatura zadovoljila *ULS* proračun.

2.2.2 Kontrola širine prslina

Među većim novinama u Evrokodu 2 [3], u odnosu na naš važeći Pravilnik [2], nalazi se mogućnost da se prethodno napregnute konstrukcije mogu projektovati tako da je dozvoljena pojava prslina u eksploataciji, odnosno, dozvoljena je primena „**parcijalnog prethodno naprezanja**“. Iako se ono najčešće primenjuje kod međuspratnih konstrukcija i to primarno kao kombinacija ubrzane tehnologije građenja i metode za kontrolu ugiba pri povećanim rasponima, dijapazon primene omogućen Evrokodom 2 [3] je znatno širi. Upotreba je moguća i u sredinama koje iskazuju agresiju tipa XC, XD i XS, a **dopuštene širine prslina su prilično liberalne**. Tako su, na primer, u slučaju prethodno napregnutih konstrukcija sa kablovima bez spoja granice iste kao i za armiranobetonske konstrukcije. Pritom su ove dopuštene širine prslina osetno veće u odnosu na one koje dopušta Pravilnik BAB 87 [1]. Pregled preporučenih vrednosti za maksimalnu širinu prslina, kako je usvojeno i u srpskom *NA* [6], dat je u tabeli 2.

Tabela 2 – Preporučene maksimalne širine prslina prema Evrokodu 2 [3]

Klasa izloženosti	AB i prethodno napregnute konstrukcije sa kablovima bez spoja	Prethodno napregnute konstrukcije sa kablovima sa spojem
	pri kvazi-stalnoj kombinaciji	pri čestoj kombinaciji
X0, XC1	0,4	0,2
XC2 - 4	0,3	0,2
XD1- 2, XS1- 3	0,3	uslov dekompresije

Ograničenja za kablove sa spojem su oštrija i to po dva osnova: granice su niže, a kontrola se sprovodi pri višem nivou opterećenja („česta“ kombinacija). Uslov dekompresije podrazumeva da kablovi ostaju u pritisnutoj zoni, to jest da prslina ne seže do njih.

Kao kriterijum da li nastaju prsline, u analizi napona u preseccima može se koristiti granica zatezanja jednaka srednjoj vrednosti čvrstoće na zatezanje $f_{cm}(t)$, prema starosti u kojoj se proverava nastanak prslina. Kriterijum bi trebalo proveriti pri najvišem nivou eksploatacionog opterećenja („retka“ kombinacija).

Ispunjenje kriterijuma u pogledu **širine prslina** moguće je ispitati **na dva načina**, kao i kod armiranobetonskih konstrukcija. **Prvi način je direktno sračunavanje širine**, a **drugi** se sprovodi **utvrđivanjem maksimalnog razmaka** ili prečnika **armature**. U suštini, i kod prethodno napregnutih elemenata, širina prslina se kontroliše uz pomoć **napona u armaturi** koja je smeštena uz ivicu na kojoj se otvara prslina. Čak i u slučajevima kada je njena površina mala, formalni postupak može da se sprovede tako da daje smislene rezultate. Za oba načina potrebno je sračunati napon u armaturi pri odgovarajućoj kombinaciji opterećenja.

Napon se određuje po modelu preseka sa prslinom, upotrebom uobičajenih postupaka (linearna slika napona i dilatacija, isključenje zategnutog betona). Ukoliko je učešće meke armature u ukupnoj površini čelika u zategnutoj zoni znatno (na primer kao rezultat obezbeđenja potrebne nosivosti - *ULS*), izračunati napon bi trebalo uvećati. Instrukcije za ovo,

međutim, **nisu date u samom Evrokodu 2** [3], već se mogu naći u literaturi koja objašnjava prateće dokumente [8]. Kablovi se u proračun uvode pomoću zamenjujućeg prečnika i zamenjujuće površine, čime se u proračun uvode njihove (slabije) karakteristike prijanjanja u odnosu na armaturu. Kablovi bez spoja (*unbonded*) ne učestvuju u površini.

Ako se širina prslina izračunava, potrebno je sračunati i razmak. Obrazac za izračunavanje je drugačiji od onog koji je prikazan u Priručniku [9], a koji vodi poreklo iz CEB priručnika [10]. „Stari“ i „novi“ obrazac nakon izračunavanja daju veoma slične vrednosti. Međutim, koeficijent koji uvodi sadejstvo zategnutog betona je izmenjen tako da, kada se proračun sprovede prema Evrokodu 2, sadejstvo biva veće. Pri istom inputu, to rezultuje manjom sračunatom širinom prslina, [7], i, zajedno sa liberalizovanim granicama, dodatno relaksira problem zadovoljenja ograničenja.

Kontrola širine prslina bez direktnog proračuna sprovodi se ograničavanjem prečnika ili razmaka armature. Granica se dobija iz tabela 7.2N (prečnik, Tabela 3) ili 7.3N (razmak, Tabela 4) Evrokoda 2 [3], očitavanjem vrednosti **prema** prethodno određenom **naponu u armaturi** pri relevantnoj kombinaciji opterećenja (kvazi-stalna ili česta). Takvi kriterijumi mogu se izvesti i za širine prslina određene prema obrascima koji se sada primenjuju kod nas [11]. Pritom je dovoljno **ispuniti samo jedan** od kriterijuma.

Tabela 3 – Maksimalni računski prečnik šipki \varnothing_s^* (mm) prema Evrokodu 2 [3] u skladu sa ograničenjem širine prslina w_k

Napon u čeliku [MPa]	$w_k = 0,4$ mm	$w_k = 0,3$ mm	$w_k = 0,2$ mm
160	40	32	25
200	32	25	16
240	20	16	12
280	16	12	8

Važno je napomenuti da prečnik koji se očitava iz Tabele 3 treba korigovati sa nekoliko faktora (mutiplikatora) da bi se dobila veličina granice za fizički prečnik šipke. Ukupni korekcionni faktor može znatno da izmeni polaznu vrednost uzetu iz tabele.

Tabela 4 – Maksimalno rastojanje šipki (mm) prema Evrokodu 2 [3] u skladu sa ograničenjem širine prslina w_k

Napon u čeliku [MPa]	$w_k = 0,4$ mm	$w_k = 0,3$ mm	$w_k = 0,2$ mm
160	300	300	200
200	300	250	150
240	250	200	100
280	200	150	50

U većini slučajeva lakše se zadovoljava **kriterijum po maksimalnom razmaku armature**, prikazan u Tabeli 4.

2.2.3 Kontrola ugiba

Ugib prethodno napregnutih elemenata se kod nas tradicionalno sračunava približnim postupkom koji je na strani sigurnosti. Zanemaruje se doprinos armature na sprečavanju slobodnih deformacija betona, a primenjiv je za elemente koji nemaju prslina. Kada je **dozvoljeno nastajanje prslina, proračun se izuzetno usložnjava**.

U pogledu kontrole ugiba, Evrokod 2 [3] uvodi, bar formalno, **dve mogućnosti: proveru ugiba proračunom i indirektnu kontrolu.**

Same instrukcije za **proračun ugiba** u Evrokodu 2 [3] su nerazrađene, i teško se primenjuju kada opterećenje nastaje u više faza. Način na koji se to prevazilazi kod armiranobetonskih konstrukcija polazi od toga se veličine napona, dilatacija i krivina u preseccima uglavnom samo uvećavaju pri dodavanju novih opterećenja, ali se oblik dijagrama ne menja. Ovaj pristup nije jednostavno primenjiv na prethodno napregnute elemente: slike napona i dilatacija po fazama su veoma različite, sa nastankom prslina koji se vezuje tek za naknadna opterećenja.

Postupak Evrokoda 2 [3] za **indirektnu kontrolu ugiba** ograničavanjem odnosa raspon/statička visina, koji se relativno lako sprovodi u slučaju armiranobetonskih konstrukcija, nije primenjiv na prethodno napregnute elemente.

Ovo ne znači da je problem nerešiv: teorijska i softverska rešenja ovih problema postoje već dugo, ali uglavnom nisu prilagođena za svakodnevnu upotrebu. Projektant koji se odlučio za prethodno napregnutu konstrukciju sa prslinama moraće da u proračun ugiba uloži mnogo veći rad nego dosad. Drugim rečima, ono je što je iskustveno poznato – da prethodno napregnute konstrukcije u većini slučajeva zadovoljavaju kriterijume ugiba, prilično je složeno za formalno dokazivanje kada se dopusti nastajanje prslina.

3 ZAKLJUČAK

Prelaskom na Evrokodove, dobićemo mogućnost da se prethodno napregnute betonske konstrukcije projektuju savremenijim pristupom. To, pre svega, znači proširivanje opcija pri projektovanju, što nas približava praksi u razvijenim zemljama.

Kriterijumi koje sada koristimo biće u priličnoj meri liberalizovani: ograničenja napona su ublažena, a u određenim situacijama i ne moraju da se sprovedu. Ukoliko izvođenje uključuje povećane mere kontrole, moguće je dodatno ublažavanje kriterijuma.

Ove novostečene „slobode“ treba veoma pažljivo koristiti. Odgovornost projektanta za ishod građenja, funkcionalnost i trajnost objekta je znatno uvećana. Za razliku od važećih domaćih pravilnika, gde se implicitno podrazumevalo da je poštovanjem njihovih odredbi projektant postupao ispravno u pogledu struke, Evrokodovi u mnogo slučajeva svoje kaluzule formulišu kao preporuke, bez garancije u pogledu ishoda. Odluka, a time i odgovornost, šta primeniti u cilju ispunjenja projektnog zadatka ostaje projektantu. Treba podvući da sam tekst Evrokoda 2 ne sledi logiku projektovanja svojom organizacijom, a, nažalost, često ni sadržajem: ne mali broj instrukcija nije izveden do kraja koji bi nedvosmisleno regulisao najčešće proračunske situacije. To dodatno čini da je projektant prepušten sebi i izlaže se riziku pogrešnog izbora.

Najvećoj novini sa aspekta domaće prakse, parcijalnom prethodnom naprezanju, treba prilaziti razumno. Dozvoljavanje nastanka prslina rezultat je više činjenica koje su se utvrdile u prethodnim decenijama. Svestrano izučavanje i široko i dovoljno dugotrajno iskustvo umanjilo je bojazni u pogledu korozione opasnosti koja se pojavljuje nastankom prslina. Osim toga, napredak je ostvaren i u tehnologiji proizvodnje i zaštite čelika koji se primenjuje za prethodno naprezanje. Nesumnjiv je i doprinos u tehnologiji betona, sa svim pozitivnim efektima na ukupnu trajnost konstrukcije. Parcijalno prethodno naprezanje se u građevinskoj praksi najčešće vezuje za međuspratne konstrukcije, gde mu je i mesto. Pritom je primena prethodnog naprezanja motivisana specifičnim razlozima tehnologije građenja. U pogledu postizanja

nosivosti nastaje hibridna konstrukcija, često sa znatnim „armiranobetonskim“ udelom. Rano opterećivanje, uslovljeno tehnologijom građenja, inicira nastajanje prslina, čije je postojanje irelevantno imajući u vidu sredinu i fabrički zaštićenu užad koja se najčešće koristi. S druge strane, u slučaju konstrukcija kod kojih je prethodno naprezanje primenjeno iz tradicionalnih razloga (veliki rasponi ili opterećenja, nastojanje da se smanji presek), po mišljenju autora ovog teksta, nije jednostavno opravdati parcijalno prethodno naprezanje. Ovakve konstrukcije, u većini slučajeva, već moraju da imaju solidan deo opterećenja balansiran prethodnim naprežanjem – dakle, i odgovarajuće kablove, i činjenica da zatezanje nije svedeno na mali nivo označava samo „deficit“ u broju kablova. Ekonomskim razlozima - da se konstrukcija realizuje sa manjim brojem kablova, suprotstavljaju se u ovakvim slučajevima drugi negativni efekti pojave prslina, kao što su smanjenje krutosti i otpornosti na zamor. Stoga bi trebalo pažljivo odvagati da li je „nedorađena“ (u smislu efekata prethodnog naprežanja) parcijalno prethodno napregnuta konstrukcija zaista optimalno rešenje, prevashodno kroz razliku u ceni u odnosu na, na primer, ograničeno prethodno naprežnutu.

Kratka poređenja prikazana u tekstu ukazuju da se konstrukcije projektovane u skladu sa važećim domaćim pravilnicima uklapaju u preporuke Evrokoda 2, kada su u pitanju prethodno napregnute konstrukcije. Rezultat se može označiti i kao blago konzervativan, ali, po mišljenju autora ovog teksta, ne u meri koja bi podrazumevala kritiku.

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ANALIZA I PRORAČUN ARMIRANOBETONSKE ZGRADE UZ PRIMENU BETONA RAZLIČITIH KLASA

Rezime:

8-5

Često, usled arhitektonskih zahteva, projektant konstrukcije mora da preduzme posebne mere pri projektovanju. Tipičan primere je, kao posledica zahtevanog velikog raspona između stubova, upotreba betona viših klasa, u cilju ispunjenja ULS i SLS kriterijuma, definisanih u propisima. Analizirana je devetospratna zgrada kako bi se ocenio značaj primene betona različitih klasa u konstrukciji. Razmatrane su tri različite klase betona na pet različitih modela, od kojih su dva proračunata prema propisima Republike Makedonije, a tri prema proračunskim kriterijumima definisanim u Evrokodu.

Ključne reči : duktilnost, klase betona, seizmička analiza, ekonomičnost

ANALYSIS AND DESIGN OF A REINFORCED CONCRETE BUILDING WITH DIFFERENT CLASSES OF CONCRETE

Summary:

Often, due to some architectural requisitions, a structural engineer has to take special constructive measures while designing a construction. A typical example is, because of a previously determined large span between columns, to use higher concrete class in the design of columns, while following the ULS and SLS criteria's, defined with the design codes in use. To evaluate the significance and effect of using different classes of concrete in a construction, a nine story building was subjected to various analyses. Three different classes of concrete were used in exactly five models, two of them designed by following the codes in use in Republic of Macedonia, and the other three following the EUROCODES design criteria's.

Key word : ductility, concrete classes, seismic analysis, cost - effective (economical)

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1 STRUCTURE AND MODELS DESCRIPTION

The reinforced concrete structure, that is subjected to these analyses, is a 9 story garage building, with a total area of 20 000 square meters, that provide 500 parking places. A total height of 28.8m and dimensions at the base $L_x= 48\text{m}$ and $L_y=36\text{m}$. It has been constructed and erected using class 30 concrete and reinforcement bars RA 400/500. Considering the height and the fact that the structure was designed as rather rigid (because of the presence of RC shear walls), it seems that the choice of this class of concrete is most reasonable and suitable. Even so, since the needed parking spaces are predefined, spans of about 8m are required. This type of solution would request relatively large cross section areas of columns, due to ductility issues. The needed cross section areas would reduce the open space, that is the parking space in a significant value. In a case like this, one of the possible solutions is to increase the concrete class which will result in smaller cross sections. This is the subject to this paper. Only solutions with rectangular cross sections will be analyzed. Foundations and soil properties won't be subject of this analysis. The structure in all models will be studied as fixed at the base.

2 MATHEMATICAL MODELS

The models are analyzed following two codes for seismic design and dimensioning of RC elements, MK codes - PIOVS/81 and PBAB/87 in comparison with EC8 and EC2.

In total five models were subjected to the purpose of the analysis:

- MODEL 1 (designed with MB 30);
- MODEL 2 (designed with MB 60).

[following PBAB/87 and PIOVS/81]

- MODEL 1a (designed with C25/30)
- MODEL 2a (designed with C50/60)
- MODEL 3a (designed with C60/75)

[following EC2 and EC8]

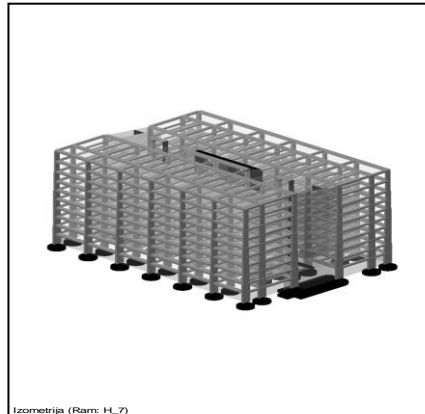


Fig.1 Isometric view of the structure

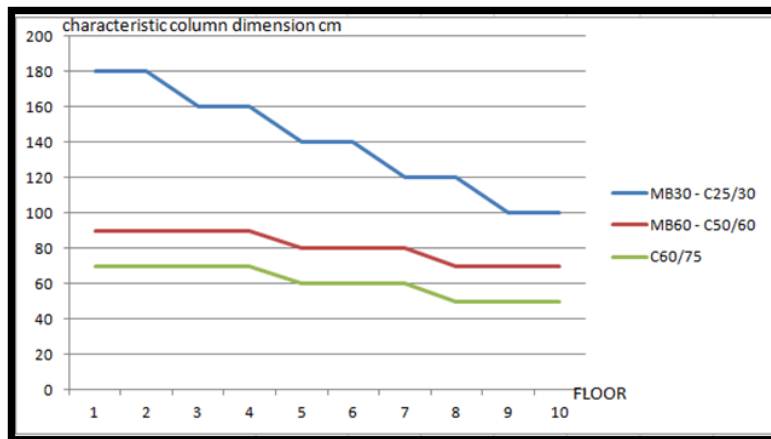
In the mathematical model , in Radimpex Tower v6 {1}, all the construction members (columns, beams, RCwalls, slabs) are modeled with a uniform class of concrete. The main accent is on the columns, on the changes in their dimensions (from the models with a smaller class to the ones with higher). As in for the beams, the increase of the concrete class, does not provide significant decrease of their height, but reduction of their width. The RC walls are with constant dimensions in all the models, so that it would not affect the horizontal stiffness, and are not subject of analysis. So the main interest is on the change in dimensions of the columns, by the ductility conditions for every class of concrete.

For each and every model, a seismic analysis is made. After comes dimensioning and design of cross sections for columns and beams. Maximum horizontal and vertical displacements are checked and evaluated. Finally the joints were checked, following the requisitions in EC8{3}, for a characteristic joint in the three models designed by EUROCODES. For the design of cross sections in the model C60/75, the program RC solver is used.

Table 1 - The decrease of column dimensions

Floor number	Column dimensions									
	1	2	3	4	5	6	7	8	9	10
MB30 - C25/30	40/180	40/180	40/160	40/160	40/140	40/140	40/120	40/120	40/100	40/100
MB60 - C50/60	40/90	40/90	40/90	40/90	40/80	40/80	40/80	40/70	40/70	40/70
C60/75	40/70	40/70	40/70	40/70	40/60	40/60	40/60	40/50	40/50	40/50
Rate of reduction [%], Highest vs Lowest concrete class	61.1	61.1	56.3	56.3	57.1	57.1	50.0	58.3	50.0	50.0

Graph 1. Graphical presentation of (a characteristic) column reduction for every deck, for the three classes of concrete



The seismic analysis, following PIOVS/81 is with the method of equivalent horizontal force (linear static analysis) and the reduction coefficients of the base shear force are given in table 2.

PIOVS/81

Table 2 - Coefficients according PIOVS/81

Seismic intensity coefficient (IX - MCS ,T=500 years.)	0.1
Category of object coefficient (II)	1.0
Dynamic coefficient (soil characteristics dependent)	0.7/T
Ductility and damping coefficient	1.0

The seismic analysis following EC8, is with the design spectra method, which is the reduction of the elastic spectra by the means of the behavior factor q . The structure is regular in height and base view.

Spectra T1 is used, for expected earthquake magnitude of $M_S > 5.5$. Soil type B, and relative horizontal acceleration $a_g = 0.3g$.

EUROCODE 8

Table 3 - Koefficients according EC8

S - фактор на почвата	1.2
T _B – долна граница на периодата од константен дел на спектар	0.15
T _c – горна граница на периодата од константен дел на спектарот	0.5

3 RESULTS

The results of the above mentioned analysis are presented in the following table 4, and commented below.

The different values of the seismic forces, evaluated by the two codes, should result with differences in the bending moment diagrams and requirements reinforcement in the cross sections of beams and columns. But table 4. shows no significant differences appear. This is due to the conception of the chosen building, moreover the placement and dimensions of the RC walls. In this specific case most of the horizontal forces are accepted by the RC walls, which were not subject of analysis, but have great influence on the horizontal stiffness, and regulates the maximum horizontal displacement.

The maximum displacements differ by the two used codes, but even with the 50% reduction of column cross sections it is far less than the highest levels of tolerances.

Table 4. Comparison of results

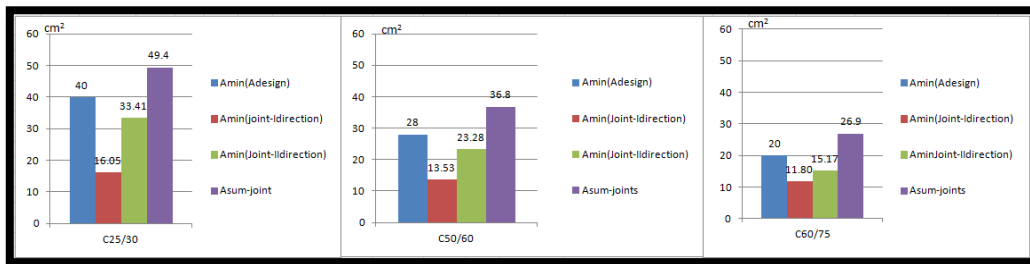
Comparison of results					
MB30 - C25/30	Column dimensions	Results		MODEL 1 PIOVS/81,PBAB /87	MODEL 1a EUROCODE 8 EUROCODE 2
	180/40 160/40 140/40 120/40 100/40	First vibration mode	X-direction	0,551s	0,539s
			Y-direction	0,396s	0,387s
		Seismic force	X-direction	18148,8[KN]	29446,7[KN]
			Y-direction	18148,8[KN]	8505,3[KN]
	Maximum horizontal displacement	X-direction	16,7mm	23,12mm	
	Reinforcement	Beams	Minimum / Reinforcement by design	Minimum / Reinforcemen t by design	
		Columns	Minimum reinforcement	Minimum reinforcement	
MB60 - C50/60	Column dimensions	Results		MODEL 2 PIOVS/81,PBAB /87	MODEL 2a EUROCODE 8 EUROCODE 2
	90/40 80/40 70/40	First vibration mode	X-direction	0,585s	0,572s
			Y-direction	0,440s	0,430s
		Seismic force	X-direction	17068[KN]	25863,4[KN]
			Y-direction	17068[KN]	7393,4[KN]
	Maximum horizontal displacement	X-direction	18,8[mm]	24,8[mm]	
	Reinforcement	Beams	Minimum / Reinforcement by design	Minimum / Reinforcemen t by design	
		Columns	Minimum reinforcement	Minimum reinforcement	
C60/75	Column dimensions	Results		PIOVS/81,PBAB /87	MODEL 3a EUROCODE 8 EUROCODE 2
	70/40 65/40 60/40 55/40 50/40	First vibration mode	X-direction	/	0,573s
			Y-direction	/	0,421s
		Seismic force	X-direction	/	22115,8[KN]
			Y-direction	/	5407,7[KN]
	Maximum horizontal displacement	X-direction	/	24,9[mm]	
	Reinforcement	Beams	/	Reinforcemen t by design	
		Columns	/	Minimum / Reinforcemen t by design	

As in for the main results of interest, which are dimensions and type of reinforcement of columns and beams, it has been concluded that: As the concrete class increases, dimensions of column members reduce - while sufficient ductility is provided. Dimensions of beams also reduce, especially the width which is governed more by (the now reduced) dimensions of columns, than the ULS or SLS requirements. Having this in mind, with the reduction of the beam cross section, the minimum reinforcement comes to be the same with the reinforcement by design. This results with a more economical solution in terms of reinforcement.

At the : **MODEL 1a**, **MODEL 2a** and **MODEL 3a** , a joint requirements check (by EC2 design rules) is made. The results are for a two sided characteristic joint for each model :

- Required vertical reinforcement in a joint for both directions, for the three classes

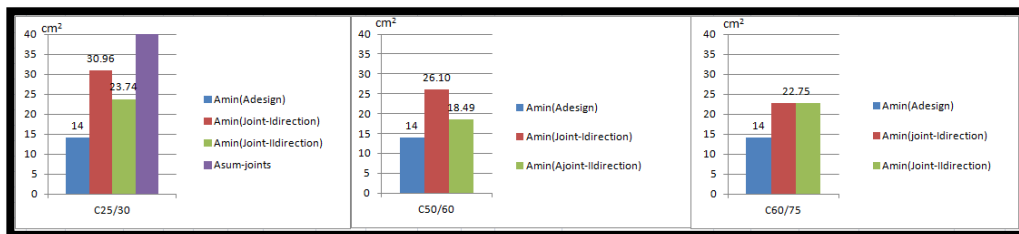
Graph 2. Required reinforcement from column analysis in comparison with joint requirements



For each of the three different classes the required reinforcement from the joint analysis is somewhat larger than the one from column analysis, at a whole cross section level. It is just important to highlight that it is possible to insert reinforcement bars, while the minimum required distances between them and other reinforcement rules are satisfied.

- Required horizontal reinforcement (stirrups) in a joint for both directions, for the three classes

Graph 3 . Required reinforcement from column analysis in comparison with joint requirements



The results for the horizontal reinforcement show that where the higher classes are used, because of the smaller cross sections, we have the same requirement for both directions. This means the same, in example stirrup would satisfy both directions. This is the result of more uniform dimensions of the column and beam in the joint. In comparison, the joints where lower concrete classes are used, the columns have rectangular cross sections, the beams are wider in one direction (on the side of the larger dimension of the column). This leads to non equal reinforcement, which would mean that the wider beam would govern the final reinforcement. This is less economical than with similar beams in both directions (as it is with higher concrete classes).

The results show that joint design request somewhat more reinforcement than the one for beam design. But differences are smaller in the cases of higher concrete classes.

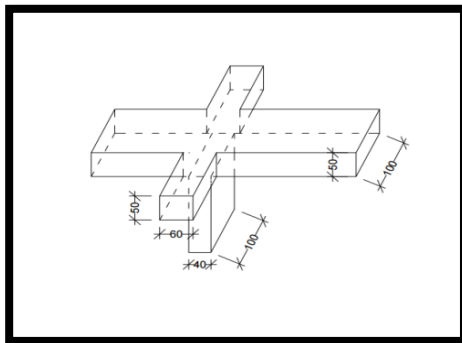


Fig.2 Characteristic MODEL 1a joint

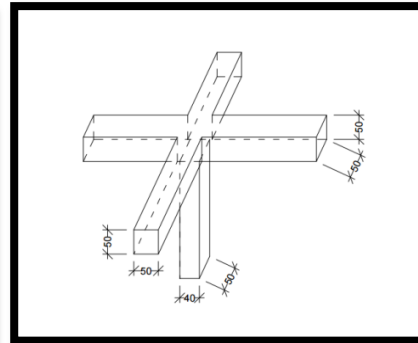


Fig.3 Characteristic MODEL 3a joint

4 CONCLUSION

The results of these numerous analytical researches are as it follows:

- No matter the code that was used, the final results, by means of dimensioning and reinforcement requirements in beams and columns, do not differ in significant matter. This is mostly because of the RC walls arrangement and placement in the building, which in this case were such that they would bear most of the horizontal forces, and were not subject of analysis.
- The use of higher concrete classes, for this type of building, has shown as a good and reasonable solution for the previously mentioned issue, for the following reasons
 - Even with the 50% reduction of column cross sections, horizontal displacements remained far lower than the reasonable level of maximum tolerance.
 - With reduction of cross sections, the dynamic mass did not change in a significant matter, so the seismic forces were approximately the same. This makes for a better use of the cross sections in terms of reinforcement, or the reinforcement by design becomes equal to the minimum one.

- From a design point of view, joints became geometrically far more regular, with uniform beam dimensions in both directions, which have shown to be better in joint design requirements.

As subject of further research, would be a moment resisting frame building, where columns would play the main role in horizontal stiffness, so the results probably would be different.

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ISPITIVANJE AB KONTINUALNIH GREDNIH NOSAČA OJAČANIH VLAKNIMA ARMIRANIM POLIMERIMA

Rezime:

6-9

U okviru ovog rada su prikazani rezultati eksperimentalnog ispitivanja AB kontinualnih grednih nosača ojačanih armaturom od vlaknima armiranim polimerima (Fiber Reinforced Polymer – FRP). Ojačavanje je izvršeno ugradnjom FRP šipki od staklenih i karbonskih vlakana u zaštitnom sloju betona. Ispitana su tri kontinualna nosača izloženih uticaju statičkog opterećenja do loma, pri čemu su merene vrednosti ugiba i dilatacija u čeličnoj armaturi. Na osnovu sprovedenog ispitivanja zaključeno je da se ojačavanjem kontinualnih nosača FRP armaturom može ostvariti značajno povećanje nosivosti bez obzira na vrstu armature.

Ključne reči: Kontinualni nosač, Vlaknima armirani polimeri, NSM metoda

INVESTIGATION OF RC CONTINUOUS BEAMS STRENGTHENED WITH FIBER REINFORCED POLYMER

Summary:

The paper presents the results of experimental investigations of the RC continuous beams strengthened with fiber reinforced polymer (FRP). Strengthening was performed by mounting glass and carbon FRP bars under the concrete surface. Three continuous beams were tested during which the deformations of beams and the strains in steel were recorded until failure under monotonically increased loading. Based on the carried out investigation it is concluded that a significant increase in capacity can be achieved by strengthening of continuous beams using reinforcement made of FRP materials, regardless of the type of FRP reinforcement.

Key words: Continuous beam, Fiber reinforced polymer, NSM method

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1 UVOD

Građevinskom industrijom su tokom istorije dominirala četiri tradicionalna materijala: kamen, drvo, beton i čelik. Do pre nekoliko stotina godina, kamen i drvo su bili osnovni materijali za izradnju građevinskih konstrukcija. U proteklih dvesta godina, konstrukcijski čelik i armirani beton su postali vodeći konstrukcijski materijali, tako da je većina modernih konstrukcija izrađena od ovih materijala. Upotreba čelika i betona omogućila je ubrzani razvoj građevinske industrije i izgradnju savremene infrastrukture koja je značajno doprinela ekonomskom prosperitetu razvijenog sveta. Međutim, višedecenijsko nesavesno i prekomerno korišćenje konstrukcija od čelika i betona rezultiralo je situacijom koja se približava globalnoj infrastrukturnoj krizi. Postojeća infrastruktura u Evropi, Sjedinjenim Američkim Državama, Kanadi i ostalim razvijenim zemljama je u nezadovoljavajućem stanju po mnogim aspektima, a bez javnih sredstava potrebnih za zamenu postojećih konstrukcija ili izgradnju novih [1]. U Srbiji je situacija takva da sa jedne strane postoji ogromna potreba za sanacijama i ojačavanjem postojećih objekata, naročito infrastrukturnih, dok, sa druge strane, nove metode i postupci teško nalaze put do normativnih dokumenata i građevinske prakse [2].

Ojačavanje betonskih konstrukcija moguće je postići na više načina: smanjenjem statičkih uticaja, povećanjem nosivosti preseka, promenom stanja naprezanja itd. Povećanje nosivosti preseka je najčešći vid ojačavanja betonskih konstrukcija i može se ostvariti povećanjem dimenzija betonskog preseka, dodavanjem čelične armature ili dodavanjem armature od kompozitnih materijala. Vlaknima armirani polimeri – FRP (Fiber Reinforced Polymer), relativno nova klasa nekorozivnih materijala visoke čvrstoće i male težine, pokazali su se u proteklih dvadesetak godina kao materijali pogodni za veliki broj primena u građevinskom inženjerstvu. Kao što i njihovo ime sugeriše, ovi materijali su sastavljeni od vlakana visoke čvrstoće utopljenih u polimernu matricu [3].

Pri ojačavanju armiranobetonskih (AB) konstrukcija armaturom od FRP materijala najčešće se koriste dve osnovne metode: ojačavanje lepljenjem laminata od FRP materijala na površini betona – EB (Externally Bonded) metoda i ojačavanje ugradnjom šipki ili traka od FRP materijala u zaštitnom sloju betona – NSM (Near Surface Mounting) metoda. Na osnovu brojnih, pre svega eksperimentalnih [4], [5], [6], a zatim teorijskih i numeričkih istraživanja uočeno je da se kapacitet nosivosti na savijanje AB greda značajno povećava njihovim ojačavanjem armaturom od FRP materijala. Većina ovih istraživanja bavi se ponašanjem nosača statičkog sistema prosta greda, opterećenih sa dve koncentrisane sile (four point load), a mnogo češći slučaj u realnim konstrukcijama je statički sistem kontinualnog nosača, koji je znatno manje istraživani. Pored toga, u domaćoj građevinskoj regulativi još uvek nema praktičnih preporuka, ni propisa i standarda za primenu bilo kog sistema ojačanja FRP armaturom i uglavnom se koriste preporuke i softveri svetski priznatih proizvođača ovih proizvoda (Sika, Mapei).

Predmet istraživanja u ovom radu je analiza moguće primene vlaknima armiranih polimernih (FRP) materijala u ojačavanju AB kontinualnih greda od samougrađujućeg betona. Istraživanje je obavljeno sa ciljem utvrđivanja mogućnosti primene NSM metode ojačavanja korišćenjem šipki od staklenih – GFRP (Glass Fiber Reinforced Polymer) i karbonskih – CFRP (Carbon Fiber Reinforced Polymer) vlakana.

2 OJAČAVANJE AB GREDA NSM METODOM

NSM metoda zasniva se na tehnici kojom se u podužne proreze (šliceve), u zaštitnom sloju betona, postavlja FRP armatura oblika šipke (bar) ili trake (strip), kao dodatna armatura, i zatapaju u epoksidne ili cementne lepkove, čime se ostvaruje adhezija sa betonom i obezbeđuje sidrenje [7].

NSM metoda ojačanja se sastoji iz sledećih koraka [8]:

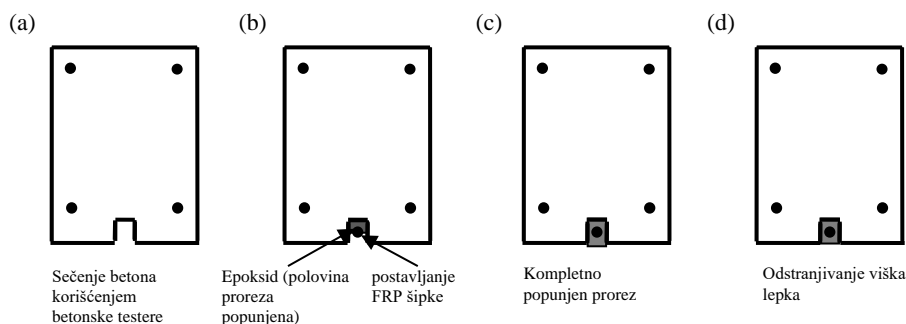
- Isecanje proreza u zaštitnom sloju betona, u zavisnosti od njegove debljine i dimenzija FRP armature koja se koristi. U konkretnom slučaju širina i dubina proreza bili su dva i po puta veći od prečnika ugrađenih šipki. Kod GFRP šipki to je bio prorez 25×25 mm, dok je kod CFRP šipki iznosio 20×20 mm (Slika 1(a));

- Pažljivo čišćenje proreza upotrebom vazduha ili vode pod pritiskom kako bi se uklonili svi ostaci od sečenja u prorezima. Ukoliko se koristi epoksidni lepak prorez mora biti сув pre ugradnje. Ukoliko se koristi cementni malter preporuka je da površina proreza bude vlažna u vreme njegove ugradnje;

- U postupku ugradnje FRP armature prvo se tri zasečene strane proreza premazuju dvokomponentalnim prajmerom;

- Sledeći korak je nanošenje dvokomponentalnog epoksidnog lepka do polovine dubine proreza. U do pola popunjeni prorez postavlja se FRP šipka zatapanjem u epoksidnu pastu (Slika 1(b));

- Posle toga, epoksidnom pastom zapunjuje se ostatak proreza, a površina poravnava, uz uklanjanje viška epoksida (Slika 1(c) i (d)).

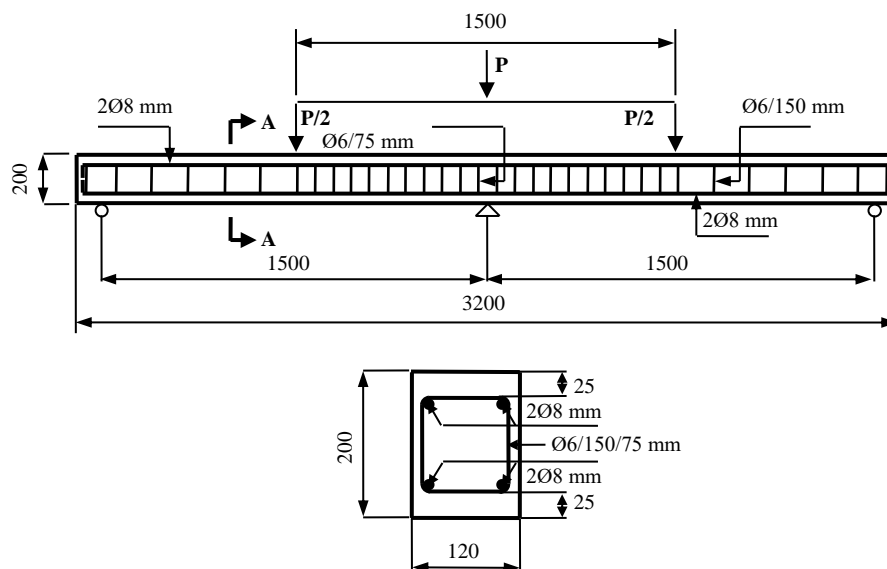


Slika 1 - Procedura ojačavanja grednih nosača opterećenih pretežno na savijanje primenom NSM metode

3 EKSPERIMENTALNA ISPITIVANJA

3.1 POSTAVKA EKSPERIMENTA I PROGRAM ISPITIVANJA

Program ispitivanja obuhvatio je tri gredna AB nosača pravougaonog poprečnog preseka dimenzija $120/200$ mm, ukupne dužine 3200 mm, statičkog sistema „kontinualac“ na dva polja. Osovinski raspon između oslonaca iznosio je 1500 mm. Armiranje greda obavljeno je rebrastom armaturom RA 400/500-2, prema prikazanim detaljima na Slici 2.



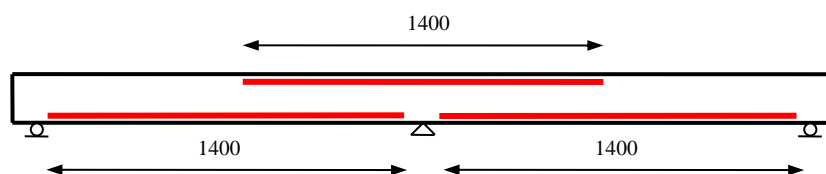
Slika 2 - Detalji armiranja ispitivanih grednih nosača

Ispitivani su sledeći gredni nosači:

Kontrolna B-con greda bez dodatne FRP armature,

B-G greda ojačana GFRP armaturnom šipkom dužine 1400 mm postavljenom u gornjoj zoni nad srednjim osloncem i GFRP armaturnim šipkama dužine 1400 mm postavljenim u donjoj zoni oba polja nosača (Slika 3) i

B-C greda ojačana CFRP armaturnom šipkom dužine 1400 mm postavljenom u gornjoj zoni nad srednjim osloncem i CFRP armaturnim šipkama dužine 1400 mm postavljenim u donjoj zoni oba polja nosača (Slika 3).



Slika 3 – Položaj NSM FRP šipki kod ojačanih grednih nosača

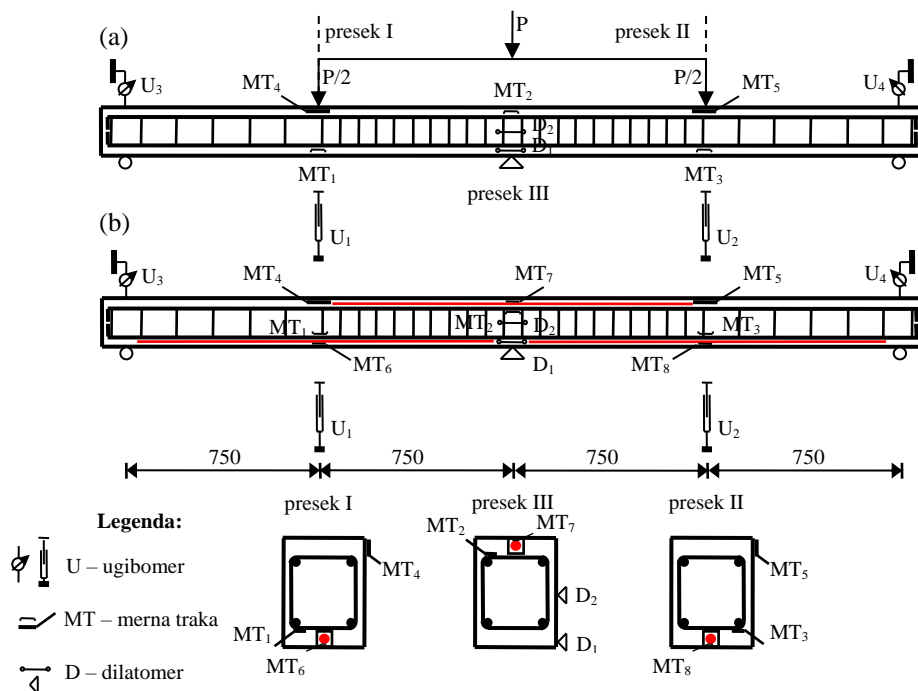
Opterećenje je nanošeno hidrauličkim uređajem, pri čemu je sila sa klipa test mašine prenešena na nosač u vidu dve koncentrisane sile, koje deluju u sredini oba polja na osovinskom razmaku od 1500 mm. Za prenos opterećenja od hidrauličkog klipa iskorišćene su dve čelične šipke dužine 2 m, a prenos opterećenja na AB gredu ostvaren je preko čeličnih ploča širine 100 mm i čeličnih valjaka Ø30 mm postavljenih između njih. Oslanjanje grednog nosača ostvareno je takođe preko čeličnih kontaktnih ploča širine 100 mm uz simulaciju dva

pokretna i jednog nepokretnog ležišta (Slika 4). Nanošenje opterećenja imalo je “vremenski mod”, odnosno konstantan priraštaj ugiba u funkciji vremena do loma. Brzina nanošenja opterećenja iznosila je 0,02 mm/s (1,2 mm/min).



Slika 4 – Dispozicija ispitivanih grednih nosača (experimental setup)

Vrednosti merenih veličina (ugiba i dilatacija), usled dejstva statičkog opterećenja, očitavane su kvazi dinamički (svake sekunde), a zatim su obrađivane u Ekselu i Ketmenu (Catman). Izlazni rezultati prikazani su u obliku dijagrama, iz kojih je moguće najpreglednije pratiti merene parametre u funkciji nanetog opterećenja.



Slika 5 – Raspored instrumenata na: (a) B-con gredi, (b) B-G i B-C gredi

Raspored primenjenih instrumenata na kontrolnoj gredi, kao i na ojačanim gredama prikazan je na *Slici 5*. Za praćenje ugiba korišćeni su induktivni pretvarači W50 (LVDT) postavljeni u sredini oba polja. Pomeranje oslonaca praćeno je ugibomer satovima sa podatkom 1/100 mm. Dilatacije u pritisnutom vlaknu betona u sredini oba polja registrovane su mernim trakama sa bazom od 50 mm. Dilatacije u čeličnoj i FRP armaturi praćene su mernim trakama prema datom rasporedu (*Slika 5*). Dilatometri sa bazom od 100 mm su postavljeni nad srednjim osloncem, po visini preseka, radi registrovanja dilatacija na betonu.

Mereni ugibi i dilatacije prikazani su u funkciji opterećenja, a kao posledica prekida u nanošenju opterećenja radi snimanja prslina, krive nisu u potpunosti glatke.

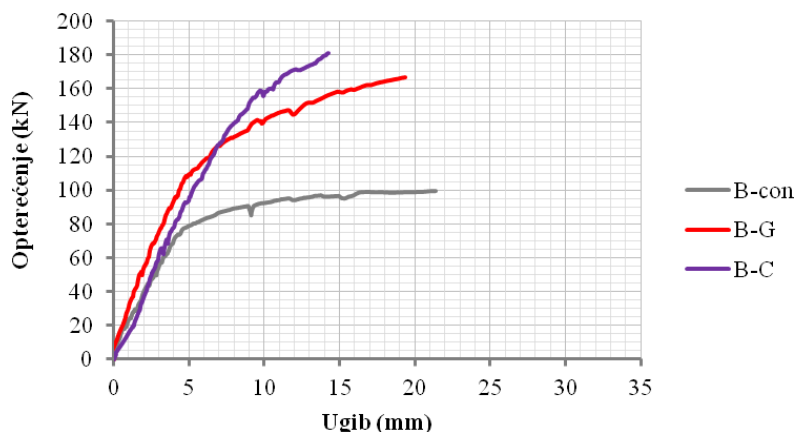
3.2 REZULTATI ISPITIVANJA I NJIHOVA ANALIZA

Dominantno mesto u sprovedenim eksperimentalnim istraživanjima zauzimaju rezultati dobijeni ispitivanjem kontinualnih grednih nosača pod dejstvom kratkotrajnog statičkog opterećenja.

3.2.1 Analiza ugiba

Jedan od najbitnijih pokazatelja ponašanja grednih nosača pod uticajem opterećenja je ugib, zbog čega je u ovom istraživanju posebna pažnja posvećena njegovoj analizi. Vrednost ugiba je integralna veličina koja objedinjuje opterećenje, raspon i krutost nosača, pa samim tim njegova analiza daje najpotpuniju sliku ponašanja nosača. Merenje ugiba je vršeno linearnim pretvaračima pomeranja (LVDT) pi čemu su vrednosti očitavane akvizicijskim sistemom (MGC plus), uz primenu odgovarajućeg softvera (Catman).

Uporedna analiza ugiba ojačanih greda izvršena je u odnosu na kontrolnu, neojačanu gredu, pri čemu je armiranje čeličnom armaturom kod svih greda bilo identično (*Slika 2*). Krive zavisnosti između opterećenja i ugiba preseka u sredini polja kontrolne grede i ojačanih greda prikazane su na *Slici 6*.

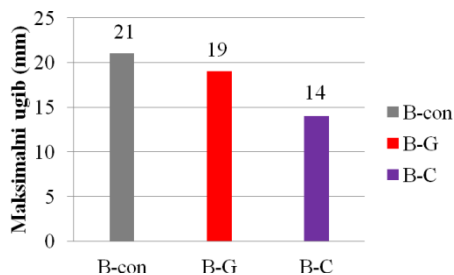


Slika 6 – Dijagram zavisnosti između opterećenja i ugiba u sredini polja kontrolne grede (B-con) i greda ojačanih FRP armaturom (B-G i B-C)

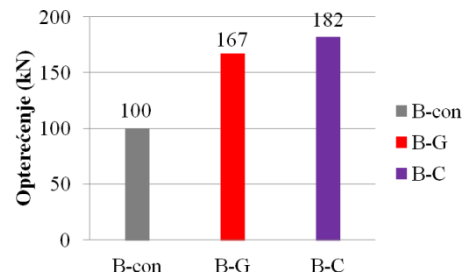
Na dijagramu se mogu uočiti karakteristične zone pri nanošenju opterećenja:

- zona pre pojave prslina,
- zona posle pojave prslina a pre pojave tečenja čelika i
- zona posle pojave tečenja čelika do otkaza nosača.

Na *Slici 7* prikazano je upoređenje maksimalnih vrednosti ugiba u sredini polja ispitanih grednih nosača.



Slika 7 – Upoređenje maksimalnih ugiba u sredini polja ispitivanih grednih nosača



Slika 8 – Upoređenje maksimalnih nosivosti ispitivanih grednih nosača

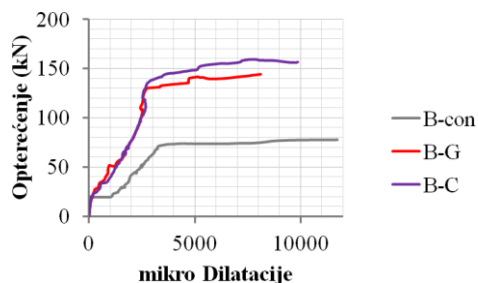
Maksimalni uočeni ugib grede ojačane GFRP armaturom iznosio je 19 mm što je za 9,52% manji ugib u odnosu na maksimalni ugib kontrolne grede koji je iznosio 21 mm, dok je maksimalni ugib grede ojačane CFRP armaturom iznosio 14 mm što je za 33% manji ugib u odnosu na ugib kontrolne grede.

Indeks duktilnosti grednog nosača ojačanog GFRP armaturom je zadovoljavajući (ID=4), dok je duktilnost grede ojačane CFRP armaturom veoma mala (ID=1,5).

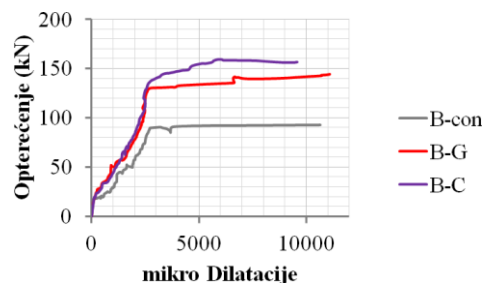
Na *Slici 8* prikazano je upoređenje maksimalnih nosivosti ispitivanih grednih nosača. Povećanje nosivosti grede ojačane GFRP armaturom je iznosilo 67% u odnosu na nosivost kontrolnog nosača dok je povećanje nosivosti grede ojačane CFRP armaturom iznosilo 82% u odnosu na nosivost kontrolnog nosača.

3.2.2 Analiza dilatacija u čeličnoj armaturi

Dijagram zavisnosti između opterećenja i dilatacija u zategnutoj čeličnoj armaturi ispitivanih grednih nosača prikazani su na *Slici 9* (presek u sredini polja) i *Slici 10* (presek nad osloncem). Iz dijagrama se može uočiti da su dilatacije u zategnutoj čeličnoj armaturi zanemarljivo male sve do pojave prslina u betonu, posle čega nelinearno rastu do pojave tečenja u čeličnoj armaturi. Poslednja faza od tečenja čelične armature do otkaza ima još izraženiju nelinearnost. Sa dijagrama na *Slikama 9* i *10* uočeno je da ojačavanjem grednog nosača GFRP armaturom postignuto povećanje opterećenja pri kome dolazi do tečenja čelične armature od 86%, dok je ojačavanjem grednog nosača CFRP armaturom postignuto povećanje opterećenja pri kome dolazi do tečenja čelične armature od 100% u odnosu na kontrolnu gredu.



Slika 9 – Dijagram zavisnosti između opterećenja i dilatacije u preseku I ispitivanih nosača

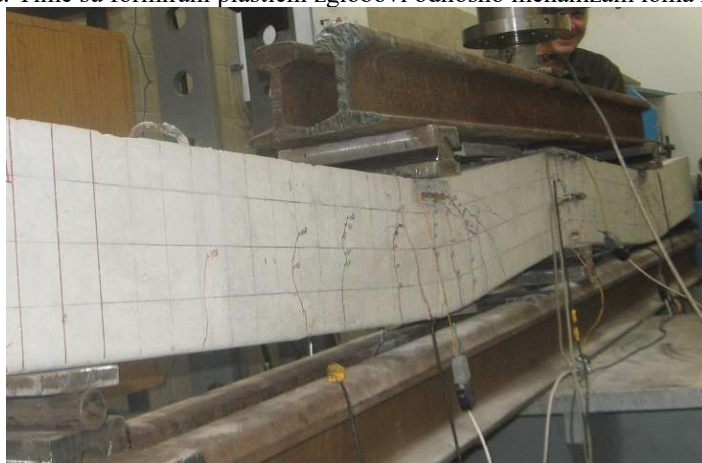


Slika 10 – Dijagram zavisnosti između opterećenja i dilatacije u preseku III ispitivanih nosača

3.2.3 Oblici otkaza (loma) nosača

B-con gredni nosač

Na Slici 11 prikazan je oblik otkaza kontrolnog grednog nosača. Lom grede je nastao dostizanjem granične dilatacije čelične armature pri zatezanju najpre nad srednjim osloncem a zatim i u polju. Time su formirani plastični zglobovi odnosno mehanizam loma nosača.



Slika 11 – Mehanizam loma kontrolne grede

B-G gredni nosač

Otkaz grednog nosača ojačanog GFRP armaturom je nastao odvajanjem na spoju epoksida i betona usled čega dolazi do odvajanja betona u visini zaštitnog sloja (Slika 12). Otkaz nosača je iznenađan uz snažan prasak i uz veliko oštećenje nosača. Poprečne prsline u betonu i sa donje i sa gornje strane grede presecaju prerez sa epoksidnom ispunom (Slika 13).



Slika 12 – Odvajanje betona u visini zaštitnog sloja betona



Slika 13 – Karakterističan raspored prslina (gornja strana nosača)

B-C gredni nosač

Na Slici 14 prikazan je oblik otkaza grednog nosača ojačanog CFRP armaturom. Do otkaza dolazi odvajanjem na spoju epoksida i betona usled čega dolazi do odvajanja betona u visini zaštitnog sloja. Nosivost ovog grednog nosača je u poređenju sa ostalim ispitivanim grednim nosačima najveća ali deformabilnost je izuzetno mala tako da do loma dolazi iznenada uz veliko oštećenje nosača.



Slika 14 – Otkaz grednog nosača B-C nastao gubitkom prijanjanja CFRP armature

4 DISKUSIJA REZULTATA I ZAKLJUČAK

Kompozitni materijali u oblasti građevinarstva, sve izvesnije, postaju materijali budućnosti. Primena NSM metode ojačanja, kao relativno nove, pruža velike mogućnosti kod sanacija i ojačanja betonskih konstrukcija i produženja njihovog eksploatacionog veka [7].

Primena FRP armature za ojačavanje AB kontinualnih grednih nosača, čak i pri maloj količini dodatne armature može značajno da poveća nosivost na savijanje. Ovo je potvrđeno sopstvenim eksperimentalnim ispitivanjem, sprovedenim 2016 god. na GAF u Nišu.

Ojačavanjem kontinualnog nosača staklenom (GFRP) armaturom postignuto je povećanje nosivosti od 67% u odnosu na kontrolnu (neojačanu) gredu dok je ojačavanjem karbonskom (CFRP) armaturom postignuto povećanje nosivosti od 82%. Pored toga, uočeno je i povećanje opterećenja pri kome dolazi do početka tečenja zategnute čelične armature od 86% i 100% u slučaju ojačanja GFRP i CFRP armaturom, redom.

Upotrebom CFRP šipki, zahvaljujući njihovoj većoj čvrstoći pri zatezanju, može se značajnije povećati nosivost kontinualnih nosača u poređenju sa upotrebom GFRP šipki. Ipak, uočeno je povećanje nosivosti od svega 9% kod grede ojačane CFRP šipkama u odnosu na nosivost grede ojačane GFRP šipkama. Razlog neznatnog povećanja nosivosti leži u prevremenom otkazu grednog nosača CFRP šipkama pri manjoj iskorišćenosti kapaciteta nosivosti dodatne armature.

Ojačavanje GFRP šipkama ne utiče negativno na duktilnost kontinualnih nosača, što potvrđuje i indeks duktilnost ispitanih nosača ($ID=4$). Za razliku od njih, duktilnost nosača ojačanog CFRP šipkama nije zadovoljavajuća ($ID=1,5$). Razlozi veće duktilnosti nosača ojačanog GFRP šipkama leži u manjem modulu elastičnosti GFRP armature, kao i prevremenom otkazu nosača ojačanog CFRP šipkama usled njihove smanjene prionljivosti.

Na osnovu svega iznetog, uzevši u obzir i da je cena GFRP armature četiri puta niža u odnosu na cenu CFRP armature, u slučajevima gde krutost nosača nije od presudnog značaja, prednost treba dati GFRP armaturi.

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UTICAJ PRSLINA NA TRAJNOST ARMIRANOBETONSKIH KONSTRUKCIJA

Rezime:

Obezbeđivanje trajnosti armiranobetonskih konstrukcija prema aktuelnim propisima vrši se na trivijalan način, propisivanjem minimalnih debljina zaštitnog sloja betona. Iako moderni koncept pruža mogućnost projektovanja konstrukcija za željeni upotrebnii vek i realistične modele procesa oštećenja, efekat postojanja prslina nije njima obuhvaćen. Osnovni cilj ovog rada je da odgovori na pitanje efikasnosti zaštitnog sloja betona sa prslinama u obezbeđivanju trajnosti armiranobetonskih konstrukcija. Analizirani su rezultati eksperimentalnih istraživanja o uticaju širine prslina na difuziju štetnih materija i koroziju armature. Pokazalo se da postoji potreba za uvođenjem efekta širine prslina kao parametra u modelima za proračun upotrebnoг века obzirom da ubrzavaju sve transportne mehanizme kroz beton.

Ključne reči: trajnost betona, prsline, difuzija hlorida, difuzija CO₂, korozija

INFLUENCE OF CRACKS ON THE DURABILITY OF REINFORCED CONCRETE STRUCTURES

Summary:

Durability of reinforced concrete structures according to the current technical legislation is carried out in an indirect manner by prescribing a minimum thickness of concrete cover. Although a modern concept provides possibility of service life design and realistic models of deterioration processes, an effect of cracks has not been covered by them. The main objective of this paper is to answer the question of efficiency of the concrete cover with cracks in ensuring durability of reinforced concrete structures. The results of experimental research on the influence of crack width on the diffusion of harmful substances and corrosion of reinforcement were analyzed. It turned out that there is a need to introduce the effect of crack width as a parameter in the models for the calculation of the service life due to acceleration of all the transport mechanisms through the concrete layer.

Keywords: concrete durability, cracks, chloride diffusion, CO₂ diffusion, corrosion

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1 UVOD

Prsline se u armiranobetonskim elementima javljaju u toku građenja, ali i u toku eksploatacije, kao posledica skupljanja, termičkog širenja, mehaničkih ili hemijskih oštećenja ili jednostavno kao posledica dejstva opterećenja. Imajući u vidu relativno nisku čvrstoću betona na zatezanje, pojava prslina na konstruktivnim elementima je gotovo neizbežna. Sa aspekta nosivosti, prisustvo prslina od opterećenja je obuhvaćeno proračunskim modelima. S druge strane, prisustvo prslina, a naročito njihova spoljašnja manifestacija, ima negativni psihološki efekat za posmatrača i još važnije – negativne posledice na trajnost armiranobetonskih elemenata kroz smanjenje otpornosti betona na različite transportne mehanizme. Jedan od glavnih transportnih mehanizama kroz beton jeste difuzija. Bilo da se radi o difuziji hloridnih jona, ili molekula CO₂, njen intenzitet je uslovljen strukturom cementne matrice u okviru zaštitnog sloja betona. Pojava prsline dovodi do narušavanja strukture zaštitnog sloja betona, a time i do povećanja koeficijenta difuzije kroz beton. Na taj način se omogućava brži prodor štetnih agenasa, što može dovesti do ubrzane korozije armature [1,2,3].

Osnovni cilj ovog rada je da odgovori na pitanja o efikasnosti zaštitnog sloja betona sa prslinama u obezbeđivanju trajnosti armiranobetonskih konstrukcija. Uticaj prslina na trajnost armiranobetonskih elemenata analiziran je kroz prizmu savremenih tehničkih propisa i preporuka, ali i kroz prikaz i analizu rezultata eksperimentalnih istraživanja o uticaju širine prslina na difuziju štetnih materija kroz zaštitni sloj betona i na koroziju armature.

2 PROJEKTOVANJE PREMA UPOTREBNOM VEKU

Tretiranje trajnosti kroz aktuelne propise se vrši na posredan način, propisivanjem minimalnih debljina zaštitnog sloja betona kako bi se usporila difuzija i ograničavanje širine prslina pod odgovarajućim opterećenjem i pri različitim uslovima izloženosti (tabele 1 i 2). Pri tome, izvršena je relativno gruba [4] ili detaljnija [5] klasifikacija uslova izloženosti, ali se u oba slučaja inženjeru daju samo iskustvene preporuke, pri čemu nije poznato šta je baza (rezulti merenja, eksperimenti, analize) za te preporuke. U uslove kojima pravilnici obezbeđuju adekvatnu trajnost konstrukcija može se pridodati i ograničenje napona u armiranobetonskim elementima [5]. Ono što je izvesna budućnost na polju trajnosti je stvarna kvantifikacija uslova izloženosti, realističniji fizičko-hemijski proračunski modeli i proračun sa koeficijentima sigurnosti, na sličan način kako se to radi za granično stanje nosivosti [6]. Ovo su osnove novog koncepta proračuna armiranobetonskih konstrukcija prema upotrebnoj dobi sadržane u okviru fib-ovog Modela propisa [7]. Konačno se osim prostog usvajanja debljine zaštitnog sloja kvantifikuje i kvalitet betona, tj. otpornost betona u okviru zaštitnog sloja i uvodi novi, ključni parametar – vreme, tj. dužina upotrebne dobi koji treba da bude obezbeđen projektovanim zaštitnim slojem. Iako nema snagu propisa, fib-ov Model propisa [7] može se smatrati obavezujućim preporukama jer obično odražava suštinu budućih izmena zvanične tehničke regulative.

Deterioracioni mehanizmi, tj. procesi oštećenja koji propagiraju kroz vreme i koji kao krajnji rezultat imaju koroziju armature su penetracija hlorida i karbonatizacija [9]. Osim njih, postoje i mnogi drugi – dejstvo mraza, kombinacija mraza i soli, alkalno-agregatna reakcija, odloženo formiranje etringita itd., ali trenutno ne postoje opšte prihvaćeni fizički ili hemijski modeli kojima bi se mogli opisati i uklopiti u koncept proračuna prema upotrebnoj dobi. Iznenađujuće, ni u jednom slučaju, prsline nisu parametar u proračunskim modelima.

Isključivo se koriste svojstva neisprskalog betona, uz ogradu da „isto važi i za situaciju sa prslinama ukoliko njihova širina ne premašuje vrednosti date u Evrokodu 2“ [9]. Ova lakonski data preporuka samo je dala na značaju potrebi da se preispita stvarni uticaj prslina na deterioracione mehanizme i upotrební vek konstrukcija.

Tabela 1 - Dozvoljene širine prslina prema EC2 [5]

Klasa izloženosti	Dozvoljena širina prslina [mm]
Nema opasnosti od korozije [XC0]	0,40
1. Korozija izazvana karbonatizacijom	
Suva ili stalno mokra [XC1]	0,30
Mokra, retko suva [XC2]	
Umereno vlažna [XC3]	
Ciklično mokra i suva [XC4]	
2. Korozija izazvana hloridima	
Umereno vlažna [XD1]	0,30
Mokra, retko suva [XD2]	
Ciklično mokra i suva [XD3]	
3. Korozija izazvana hloridima iz morske vode	
Suva sredina [XS1]	0,30
Stalno pod vodom [XS2]	
„wett and dry“ ciklusi [XS3]	

Tabela 2 - Dozvoljene širine prslina prema ACI [8]

Uslovi sredine	Dozvoljena širina prslina [mm]
Suva sredina, zaštićen element	0,40
Zemlja, vlažna sredina	0,30
Izloženost agensima za odmrzavanje	0,18
Morska voda, „wett and dry“ ciklusi	0,15
Strukture za zadržavanje vode	0,10

Osim parametara otpornosti betona koji se pojavljuju u ponuđenim proračunskim modelima Modela propisa, postoje i druga svojstva betona koji se mogu nazvati parametrima trajnosti. Oni se mogu kvantifikovati (ispitati, izmeriti) i potom dovesti u direktnu vezu sa veličinama koje su zahtevane Modelom propisa ili predstavljaju parametre nekih drugih predloženih modela za proračun prema upotrebnom veku. To se prevashodno odnosi na permeabilnost, kapilarnu sukciju i elektrootpornost materijala [10].

3 EFEKAT ŠIRINE PRSLINA

Ekperimentalna istraživanja uticaja prslina na trajnost armiranobetonskih elemenata dominantno se bave uticajem prslina na pojavu korozije armature izazvane hloridima, kao najagresivnijim deterioracionim mehanizmom. Istraživanja koja se mogu pronaći u literaturi mogla bi se podeliti na dve grupe: grupu onih koja se bave istraživanjem uticaja prslina na pojavu i tok procesa korozije [1,2,3,11] i drugu grupu koja se bavi promenama svojstava betona i transportnih mehanizama izazvanih pojavom prslina [12,13,14,15].

3.1 UTICAJ ŠIRINE PRSLINA NA PROMENU SVOJSTAVA BETONA

3.1.1 Penetracija hlorida

U tabeli 3 prikazane su vrednosti graničnih širina prslina u odnosu na transportni mehanizam difuzije hlorida koje se mogu pronaći u literaturi. Pod graničnom širinom prslina u daljoj analizi podrazumevaće se širina čije dalje povećavanje utiče na promenu određenog transportnog mehanizma. U literaturi se može pronaći mišljenje da kod prslina širih od granične vrednosti iz tabele 3 dolazi do povećanja koeficijenta difuzije sa povećanjem širine prslina, dok je kod prslina užih od datih graničnih vrednosti ova promena veoma mala [12,14,16,17]. U literaturi se mogu pronaći i granične širine za različite testove koje predlažu standardi. Tako, za dugotrajno izlaganje hloridima potapanjem (eng. *immersion*) u rastvoru NaCl ta vrednost iznosi 0,05 mm, dok za kratkotrajno izlaganje hloridima prema pravilima ubrzanog testa migracije hlorida (engl. *Rapid Chloride Migration test- RCM*) iznosi svega 0,012 mm [18]. Sve ove granične širine manje su od dopuštenih vrednosti prema evropskim i američkim propisima za date uslove izloženosti, koje iznose 0,3 i 0,15 mm respektivno (Tabele 1 i 2). Ovo bi značilo da se promena u mehanizmima difuzije i sukcije dešava pri značajno manjim širinama prslina od onih za koje se smatra da nemaju nikakvog uticaja na trajnost betona. Sa druge strane pojedini autori zaključili su da se porast koeficijenta difuzije odvija samo do granične vrednosti širine prslina nakon čega ostaje konstantan, bez obzira na dalje povećanje prslina [15,19,20].

Tabela 3 - Granične širine prslina za proces difuzije hlorida kroz isprskalu betonsku strukturu

Referenca	Granična širina prslina [mm]
[16]	0,053
[17]	0,08
[18]	0,05 (0,012)
[12]	0,135
[14]	0,04
[15]	0,20
[19]	0,08
[20]	0,08

Osim difuzije razmatrani su i drugi mehanizmi transporta hlorida kroz beton, npr. kapilarna sukcija. U radu [13] sprovedena ispitivanja su pokazala da su hloridni joni prošli kroz čitav uzorak visine 100 mm za samo tri sata, iako su prslina bile dužine 60-90 mm. To pokazuje da kapilarna sukcija može biti jedan od glavnih mehanizama transporta hlorida unutar betonske strukture, ali i da se sa porastom širine prslina ovaj mehanizam ubrzava [13,21,22,23,24]. Prisustvo prslina uzrokuje povećano početno i ukupno upijanje vode [22]. Inicijalno upijanje značajnije je uslovljeno širinom prslina nego vrednost ukupnog upijanja, čija promena je samo u manjoj meri uslovljena efektom širine. Pojava diskretnih prslina uzrokuje gotovo trenutno upijanje vode ili štetnog agensa u porozni materijal [21,24]. Međutim, većina istraživanja pojave kapilarne sukcije sprovedena je na uzorcima čije su prslina nastale dejstvom mraza, pa se malo toga zna o uticaju prslina nastalih usled opterećenja na mehanizam kapilarne sukcije.

Jedan od parametara koji je u tesnoj vezi sa otpornošću betona na difuziju hlorida je i specifična elektrootpornost materijala, koja predstavlja sposobnost materijala da se odupre transportu slobodnih jona [25,26]. Ona predstavlja indikator veličine, međusobne povezanosti i zapremine pora unutar betonske strukture [27]. Ako se uzme u obzir da veća poroznost predstavlja i veći koeficijent difuzije, onda se može zaključiti da sa povećanjem specifične elektrootpornosti raste i otpornost betona na prodor hlorida [25,26]. U literaturi se može pronaći i ispitivanje uticaja prslina na promenu specifične elektrootpornosti na postojećim konstrukcijama starosti 40 godina [28]. Ispitivanja su pokazala da je specifična elektrootpornost 3-4 puta manja na mestu prslina, u poređenju sa neisprskanim betonom. Na žalost, izostalo je ispitivanje koeficijenta difuzije hlorida istih betona. Na taj način bi se dobila veza između specifične elektrootpornosti i koeficijenta difuzije kod isprskalih preseka koji su ispitivani.

3.1.2 Difuzija CO₂

Za razliku od hloridnog transporta, uticaj prslina na difuziju CO₂ nije u dovoljnoj meri eksperimentalno ispitano. U literaturi se može pronaći zaključak da širina prslina neće imati nikakav uticaj na proces karbonatizacije [29]. Prisustvo CO₂ unutar prslina je znatno manje nego na površini betona, jer ne postoji "pogonska" sila koja će omogućiti prolaz gasa kroz prslinu. Ovaj zaključak se može uzeti sa rezervom, jer nema eksperimentalnih istraživanja kojima bi se potkrepila ova teza. Međutim, sprovedena eksperimentalna istraživanja su pokazala da nema produkata karbonatizacije unutar prslina, na osnovu čega se može zaključiti da nema njihovog uticaja na prodor ugljen dioksida kroz betonsku strukturu [14]. Pojedini autori, sa druge strane, definisali su graničnu širinu prslina (tabela 4) ispod koje ne dolazi do prodora ugljen dioksida kroz prslinu unutar betona, ili je taj prodor veoma mali [30,31]. Ta vrednost predstavlja prag uticaja širine prslina na proces difuzije CO₂, a samim tim i proces karbonatizacije betona. Ukoliko širina prslina ne prelazi graničnu vrednost, front prodora CO₂ kroz beton isti je kroz isprskali i neisprskali deo zaštitnog sloja. Poređenjem graničnih (Tabela 4) i dopuštenih (Tabele 1 i 2) širina prslina zaključuje se da projektovane širine prslina prema propisima ne obezbeđuju adekvatan proračunski model procesa karbonatizacije baziran na difuziji CO₂.

Tabela 4 - Granične širine prslina za proces difuzije CO₂ kroz isprskalu betonsku strukturu

Referenca	Granična širina prslina [mm]
[14]	nema uticaja
[29]	nema uticaja
[30]	0,01
[31]	0,10

Difuzija CO₂ kroz prslinu unutar betona pored širine zavisi i od permeabilnosti (propustljivosti) prslinskog prostora, tj. prostora između zidova prslina [32]. Reč permeabilnost u užem smislu se odnosi na transport fluida kroz pore materijala pri razlici u pritisku, što predstavlja glavni razlog degradacije betona. Permeabilnost utiče na degradaciju upravo kroz uticaj na sve transportne mehanizme štetnih agenasa koji se mogu odvijati unutar betonske strukture. S toga razlikujemo koeficijent permeabilnosti za vodu i gasove, koji su u vezi sa

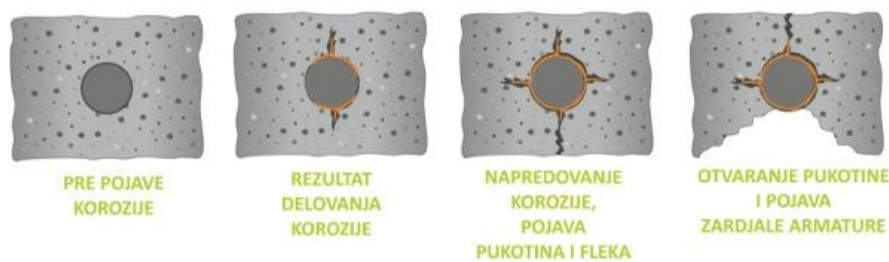
koeficijentom difuzije slobodnih hlorida i CO₂ [10]. Prsline povećavaju propustljivost betona, čineći matricu poroznijom [11]. Sa druge strane prsline uže od 0,1 mm nemaju uticaj na permeabilnost betona zbog posledice samozalečenja (eng. *self-healing*), koji se kod ovako uskih prslina javlja kao rezultat vezujućeg kalcijuma [11].

3.2 UTICAJ ŠIRINE PRSLINA NA PROCES KOROZIJE ARMATURE

3.2.1 Pojam korozije

Korozija armature je elektrohemijska reakcija oksidacije armature sa kiseonikom, uz prisustvo vlage i pri određenoj temperaturi. Konstantna dostupnost kiseonika i vode neophodna je za nastavak procesa korozije. U betonu niske relativne vlažnosti, visoka elektrootpornost materijala dovodi do odsustva korozije, dok kod betona koji je zasićen u potpunosti vodom korozija je zaustavljena usled odsustva kiseonika. U betonu izloženom periodičnom vlaženju, odnosno betonu koji je delimično zasićen vodom, korozivni proces se ubrzava [27]. Produkti korozije armature imaju i do 12 puta veću zapreminu u odnosu na čelik [33]. Pojava korozije na armaturi izazvaće smanjenje adhezije između šipke i betona, smanjenje površine poprečnog preseka šipke, ali i povećanje lokalnih napona pritiska u betonu. To će dovesti do pucanja, ljuštenja i otpadanja zaštitnog sloja betona (slika1), tj. do graničnih stanja u periodu propagacije [6].

Tanak sloj gvožđe-oksida koji nastaje u procesu oksidacije armature u alkalnoj sredini (pH>9,5) koju čini beton predstavlja zaštitu armature od korozije. Pasivizacioni sloj je nepropustan za reaktante korozije, kiseonik i vodu (O₂, H₂O). Pojava "kiselih" sastojaka okoline poput CO₂ ili SO₂, dovode do smanjivanja alkalnosti sredine betona, što dovodi do razgrađivanja zaštitnog pasivizacionog sloja armature, čime se period inicijacije završava. Reaktantima korozije sada je omogućen pristup armaturnoj šipci i period propagacije može početi [6].



Slika 1 – Proces pojave i razvoja korozije armature

3.2.2 Način merenja

Postoje različiti kriterijumi za definisanje početka procesa korozije. Svi oni se odnose na merenje električnih veličina. Dva glavna kriterijuma su:

- merenje potencijala otvorenog kola [34],
- merenje jačine (gustine) struje korozije [35].

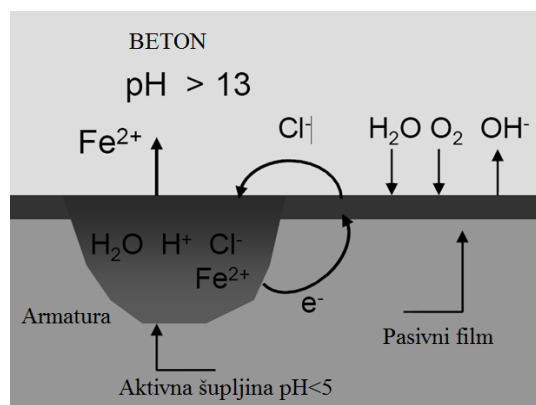
Na vrednost merenja elektropotencijala može uticati i stepen vlage unutar betona, dajući negativnije vrednosti nego što stanje korozije zapravo jeste [27]. Gustina struje korozije

određuje se na osnovu merenja linearne polarizacione otpornosti. Principi i fundamentalno znanje ove metode predstavljeni su u radu [36]. Jačina struje se može konvertovati u stepen gubitka mase armaturne šipke pomoću Faraday-ovog zakona. Ovo rezultuje konverzijom da je jačina struje od $1 \mu\text{A}/\text{cm}^2$ iznosi gubitak od $11,6 \mu\text{m}/\text{godini}$ poprečnog preseka šipke [27]. Na osnovu toga može se definisati stepen oštećenja korozijom koja ugrožavaju upotrebu konstrukcije u zavisnosti od jačine (gustine) struje korozije [37]:

- $I < 0,5 \mu\text{A}/\text{cm}^2$ ne očekuje se oštećenje armature korozijom,
- $I = 0,5 - 2,7 \mu\text{A}/\text{cm}^2$ oštećenje korozijom očekuje se za 10-15 godina,
- $I = 2,7 - 27,0 \mu\text{A}/\text{cm}^2$ oštećenje korozijom očekuje se za 2-10 godina,
- $I > 27,0 \mu\text{A}/\text{cm}^2$ oštećenje korozijom očekuje se za manje od 2 godine.

3.2.3 Korozija izazvana hloridima

Korozija izazvana hloridima je autokatalitična reakcija. Tokom procesa, Fe^{2+} , Cl^- i H^+ joni su prisutni u šupljinama i pH vrednost može pasti lokalno i do vrednosti manje od 5 (slika2). S obzirom da su kiseonik (O_2) i voda (H_2O) uvek prisutni u betonu katodna reakcija se teško može kontrolisati. Proces korozije izazvan hloridima, posebno u alkalnim sredinama, zavisi od stalnog dotoka agresivnih supstanci, propagacije u postojećim šupljinama i inicijacije novih [1].



Slika 2 – Proces korozije u betonu izazvan hloridima [38]

Ukoliko u betonskom uzorku postoji prslina moguća je pojava dve vrste korozije, mikroćelijske i makroćelijske korozije [32]. Mikroćelijska korozija u zoni prsline je proces u kome se i katodna i anodna reakcija odvijaju u maloj zoni, gde je snabdevanje kiseonikom omogućeno iz spoljašnje sredine kroz prslinu. Makroćelijska korozija predstavlja elektrohemijski proces u kome se mesto prsline ponaša kao anoda, dok se pasivna armatura između prslina ponaša kao katoda. Makroćelijska korozija izaziva znatno veća oštećenja na armaturi.

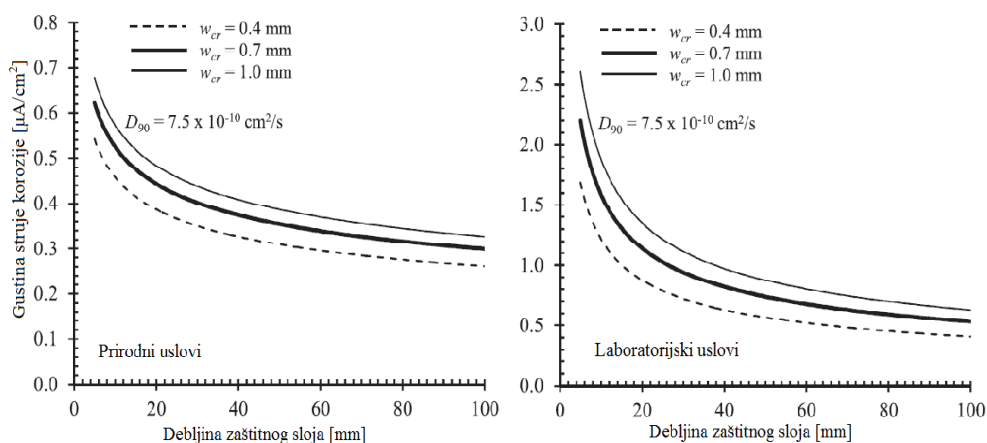
Prsline manje od granične vrednosti (tabela 5) nemaju uticaja na razvoj korozije, dok će šire prsline imati uticaj na smanjivanje perioda inicijacije i propagacije korozije armature [12,39,40,41]. Iz Tabele 5 očigledno je da su neke od definisanih graničnih širina veće [39,40] a neke manje [12,41] od dopuštenih širina prslina u evropskim i američkim propisima za date uslove izloženosti - $0,3 \text{ mm}$ [5]. Zaštita od korozije se ne može postići samo ograničenjem

širine u rasponu od 0,3 i 0,5 mm [32]. U slučaju prslina užih od 0,4 mm fenomen *self-healing*-a se često dešava kao rezultat vezivanja kalcijuma ili produkata korozije u okviru prslina, što smanjuje dalje napredovanje korozije [1]. Zbog toga će dalji prodor vode ili kiseonika biti potisnut i razvoj korozije armature smanjen. Inicijacija korozije zavisi od širine prslina, ali na njenu propagaciju bitno utiče i nivo oštećenja na mestu veze betona i armature [2].

Tabela 5 - Granične širine prslina za proces korozije izazvane hloridima kod isprskale betonske strukture

Referenca	Granična širina prslina [mm]
[39]	0,50
[40]	0,40
[41]	0,20
[12]	0,14

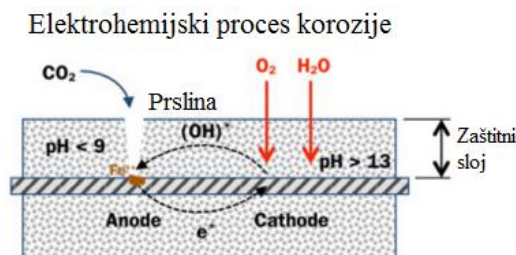
U radu [3] autori smatraju da postoji određena granična vrednost prslina preko koje, dalje širenje ne izaziva promenu u stepenu korozije. Granična vrednost ovde nije utvrđena zbog upotrebe samo dve različite širine. Predložili su određene modele veze između stepena korozije i različitih parametara betona poput vodocementnog faktora, širine prslina i koeficijenta difuzije (mera kvalitete betona). Predložena su dva modela: za laboratorijske uslove i prirodne uslove izloženosti (slika3). Na slici se može uočiti da u prirodnim uslovima debljina zaštitnog sloja igra ulogu u smanjenju stepena korozije samo do vrednosti od oko 30 mm. Veći zaštitni slojevi praktično ne utiču na stepen korozije armature, iako propisi kao jedan od glavnih mehanizama odbrane od korozije upravo vide povećanje njegove debljine. Za razliku od toga, veći uticaj predstavlja promena širine prslina. Međutim, na ovom dijagramu autori nisu prikazali stepen korozije armature kod neisprskalih preseka kako bismo mogli zaključiti koliki je zapravo uticaj pojave prslina u odnosu na neisprskale preseke. Takođe, može se primetiti da za istu debljinu zaštitnog sloja i istu širinu prslina laboratorijski uslovi pokazuju veću gustinu struje korozije a samim tim i veća oštećenja nego prirodni uslovi.



Slika 3 – Veza jačine struje korozije i debljine zaštitnog sloja u funkciji širine prslina [3]

3.2.4 Korozija izazvana karbonatizacijom

Karbonatizacija je reakcija koja smanjuje pH vrednost betona sa otprilike 12,5 do ispod 9 [27]. Pasivizacioni sloj se uništava u kontaktu sa karbonatizovanom zonom i korozija armature može početi. U isto vreme karbonatizacija smanjuje sposobnost betona da vezuje slobodne hloride, pa je na taj način reaktantima korozije omogućeno da dođu u interakciju sa armaturom [42].



Slika 4 – Proces korozije u betonu izazvan karbonatizacijom [43]

Karbonatizacija predstavlja sporiji i manje agresivan proces u odnosu na koroziju izazvanu hloridima. Međutim, nemaju sve zemlje Evrope problem sa hloridima iz morske vode, pa im karbonatizacija, iako sporiji deterioracioni proces, predstavlja glavni problem po pitanju trajnosti konstrukcija. Pojava prslina na betonskim elementima širih od granične vrednosti (0,08 mm) dovodi do pojave korozije upravo na tim mestima [30]. Sa daljim povećavanjem širine povećava se stepen korozije armature unutar betona. Inicijacija korozije izazvana karbonatizacijom je dosta brža kod isprskalih uzoraka u odnosu na neisprskale [44]. Ispitivane su prsline širine 0,01-0,6 mm i pokazalo se da je pojava prslina značajniji faktor za razvoj korozije nego njihova širina.

U budućnosti je potrebno istražiti fenomen korozije armature izazvane karbonatizacijom, pogotovo u kombinaciji sa drugim mehanizmima koji deluju na konstrukciju (hloridi, mraz sa ili bez soli za odmrzavanje).

4 ZAKLJUČAK

Kada se sagledaju istraživanja dostupna u literaturi može se zaključiti da prslina bitno utiču na trajnost armiranobetonskih konstrukcija. Prisustvo prslina utiče na ubrzanje svih transportnih mehanizama unutar betona. Takođe, prslina predstavljaju faktor koji skraćuje i vreme inicijacije i vreme propagacije oštećenja armature, što smanjuje životni vek konstrukcije. Najvažniji zaključak iz pregleda dosadašnjih eksperimentalnih ispitivanja je da postoji jasna potreba za uvođenjem efekta prslina kao parametra u modelima za proračun upotrebnoog veka konstrukcija, jer se svi transportni mehanizmi bitno menjaju i pri dosta manjim širinama od propisima dozvoljenih.

U budućnosti je potrebno sprovesti više istraživanja kako bi se utvrdio uticaj prslina na razvoja korozije pod dejstvom ugljen dioksida iz spoljašnje sredine. Takođe, potrebno je i utvrditi uticaj prslina malih širina (manjih od 0,1 mm) na razvoj korozije pod dejstvom hloridnih jona.

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Lidija Babić¹, Radomir Folić²

UTICAJ ZIDANE ISPUNE NA PERIOD VIBRIRANJA AB OKVIRNIH KONSTRUKCIJA

Rezime:

S-11

U radu je prikazan uticaj primene, vrste i položaja zidane ispune na vrednost osnovnog perioda vibriranja, koji predstavlja bitan faktor koji ukazuje na očekivani seizmički odgovor konstrukcije. Prikazane su uporedne vrednosti perioda vibriranja za jednopoljne i troljne četvoroetažne armiranobetonske (AB) okvire. Kao ispuna su primenjeni puna opeka i šuplji keramički blokovi. Tabelarno i grafički su upoređeni rezultati perioda okvira bez ispune i okvira kod kojih je ispuna raspoređena na svim etažama, kao i modela kod kojih je primenjena ispuna izostavljena u prizemlju.

Ključne reči: osnovni period vibriranja, okviri sa ispunom, AB okvirne konstrukcije

THE EFFECTS OF MASONRY INFILL ON THE PERIOD OF VIBRATION OF RC FRAME STRUCTURES

Summary:

The paper presents the effects of application, type and position of masonry infill on the value of fundamental (natural) period of vibration, which is an important factor that indicates the expected seismic response of the structure. It also presents the comparative values of periods of vibration for single-span and three-span four-storey reinforced concrete (RC) frames. Solid bricks and hollow ceramic blocks were used as infill. The results obtained for the period of frame without infill and frame in which the infill is distributed across all stories, as well as for models where the infill was left out on the ground floor are compared in tabular and graphical form.

Keywords: fundamental period of vibration, infilled frames, RC frame structures

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1 INTRODUCTORY REMARKS

In practice, the determination of periods of vibration of reinforced concrete (RC) frame structures is often conducted routinely, disregarding many influential factors. One of the factors which highly influence the value of the period is the ground on which the structure is founded. The closest results would be obtained by taking into account the structure-ground interaction, but the calculation would be a complicated and cumbersome for practical application. The presence, distribution and properties of the infill also significantly affect the period of vibration, and thereby also the seismic response of structures.

2 DETERMINING THE MODES OF VIBRATION BASED ON EUROPEAN CODES

According to the European EN 1998-1:2004 codes for the horizontal components of seismic action, the elastic response spectrum $S_d(T)$ is defined by the following expressions (Fig. 1):

$$0 \leq T \leq T_B : S_d(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2,5}{q} - \frac{2}{3} \right) \right] \quad (1)$$

$$T_B \leq T \leq T_C : S_d(T) = a_g \cdot S \cdot \frac{2,5}{q} \quad (2)$$

$$T_C \leq T \leq T_D : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2,5}{q} \cdot \left[\frac{T_C T_D}{T^2} \right] \\ \geq \beta \cdot a_g \end{cases} \quad (3)$$

$$T \leq T_D : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2,5}{q} \cdot \left[\frac{T_C T_D}{T^2} \right] \\ \geq \beta \cdot a_g \end{cases} \quad (4)$$

Where:

- a_g design ground acceleration on type A ground,
- $S_d(T)$ design spectrum for elastic analysis,,
- T_B is the lower limit of the period of the constant spectrum acceleration branch,
- T_C is the upper limit of the period of the constant spectrum acceleration branch,
- T_D is the value defining the beginning of the constant displacement response range of the spectrum,
- q behaviour factor,

β is the lower bound factor for the horizontal design spectrum. The value to be ascribed to β for use in a country can be found in its National Annex. The recommended value for the β is 0,2.

Table 1 – Values of the parameters describing the recommended Type 1 – elastic response spectra

Soil category	S	T_B (s)	T_C (s)	T_D (s)
A	1.0	0.15	0.4	2.0
B	1.2	0.15	0.5	2.0
C	1.15	0.20	0.6	2.0
D	1.35	0.20	0.8	2.0
E	1.40	0.15	0.5	2.0

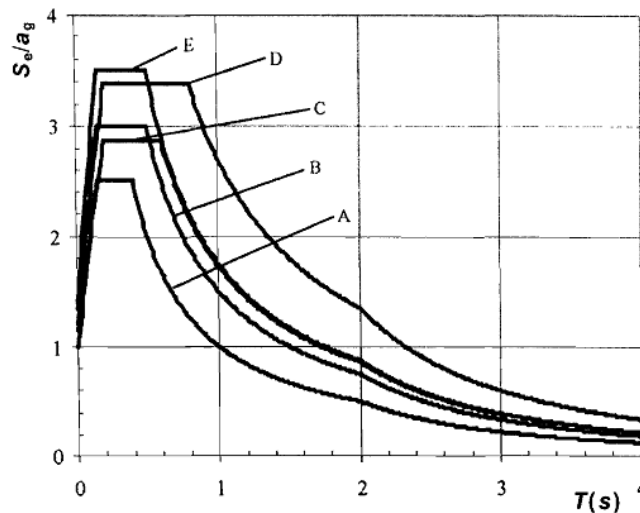


Figure 1 - Recommended Type 1 elastic spectra for ground types A to E (5% damping)

If the seismic response of the RC frame does not depend significantly on the effects of higher modes, then according to European codes planar models can be applied. The Eurocode 8 adopted the method of equivalent lateral forces as the first type of linear-elastic analysis. This type of analysis can be applied to structures which can be analyzed with two planar models and whose response does not depend significantly on the effects of higher inherent forms of free vibrations. This requirement is satisfied in structures which meet both of the following conditions:

- structures which have fundamental periods of free vibrations T_1 for each of the two main directions, which are lower than the following values:

$$T_1 \leq \begin{cases} 4T_c \\ 2,0s \end{cases} \quad (5)$$

where T_c is the upper limit of period in areas with constant spectral acceleration.

- structures that meet the criteria of regularity along the height, given in Eurocode 8.

Shear force in the base F_b for each analyzed direction is determined according to the following expression:

$$F_b = S_d(T_1)m\lambda \quad (6)$$

where:

$S_d(T_1)$ ordinate of the design spectrum for period T_1 ,

T_1 fundamental period of free vibrations for the considered horizontal direction,

m total mass of the structure above the foundation, and

λ correction factor equal to $\lambda = 0.85$ if $T_1 \leq 2T_c$ and if the structure is with more than two storeys, while in all other cases $\lambda = 1$.

Many authors have also dealt with the issues of determining the period and proposed empirical patterns some of which were included in engineering codes after being subjected to a large number of tests. It should be taken with caution, and attention should be also paid to proposals for their improvement (Pinho et al, 2009).

3 PARAMETRIC ANALYSIS OF VALUES OF PERIODS OF VIBRATIONS

The paper provides an overview of comparative values of periods of vibration for single-span and three-span four-storey RC frames of 3 m floor height and 4.8 m width without infill, infilled with ordinary 12x25x6.5 cm solid brick in compo mortar of nominal strength M2.5 (1: 3: 10) $G = 120000 \text{ kN/m}^2$, and $G/E = 0.1$, as well as for the wall of hollow 29x19x19 cm ceramic blocks in compo mortar of nominal strength M5 (1: 2: 6) $G = 550000 \text{ kN/m}^2$, and $G/E = 0.11$, which in one case is placed on all floors, and in the other case left out on the ground floor of the observed models. Dimensioning was carried out according to EN 1992 and EN 1998-1 codes. RC cross-sections were made of C25/35 of volumetric weight $f_c = 25 \text{ MPa}$ kM/m^3 . The modulus of elasticity is $E = 3.1 \text{ GPa}$. S500 reinforcement was used, whose properties correspond to our ribbed reinforcement. The dimensions of beams and columns were 30x40 cm and 50x50 cm, respectively. Effects of seismic action and gravitational load were taken into account. The calculation was conducted using nonlinear static pushover method, with the given target displacement of the top of the structure. The calculations were conducted in the SAP2000 software package, and the infill was taken into account through link elements. The properties of the infill were determined using the compressed diagonal method (Salatic et al, 2009). The plastification of beams and columns was taken into account by applying M3 plastic hinges to the end of beams, and M2-M3-P to the ends of columns.

Table 2: Comparative values of the period of the first tone of oscillation for four-storey frames of 4.8 m width and 3 m ground-floor height

Single-span 4-storey frame h=3m b=4.8m	T_1	[%]	Three-span 4-storey frame h=3m b=4.8m	T_1	[%]
Without infill	0.262371		Without infill	0.262371	
Infill 25 cm	0.201429	- 23.23	Infill 25cm	0.190653	- 27.33
Infill 19 cm	0.151626	- 42.21	Infill 19cm	0.138797	- 47.10
Infill 25 cm upper floors	0.220178	- 16.08	Infill 25cm upper floors	0.216084	- 17.64
Infill 19 cm upper floors	0.178221	- 32.07	Infill 19cm upper floors	0.174806	- 33.37

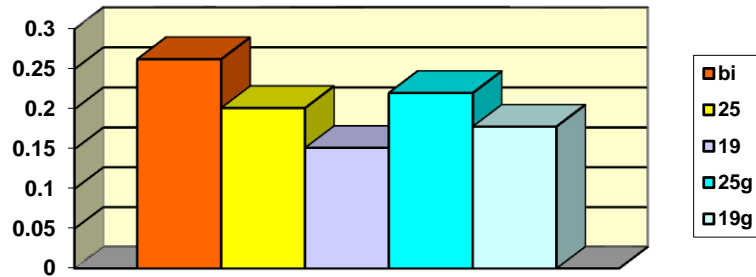


Figure 2: Comparative values of periods of the first tone of vibration for the single-span four-storey frame: 4.8 m width, 3 m ground-floor height without infill (bi), with 25 cm thick infill (25), with 19 cm thick infill (19), with 25 cm thick infill except in the ground floor (25g), and 19 cm thick infill except in the ground floor (19g)

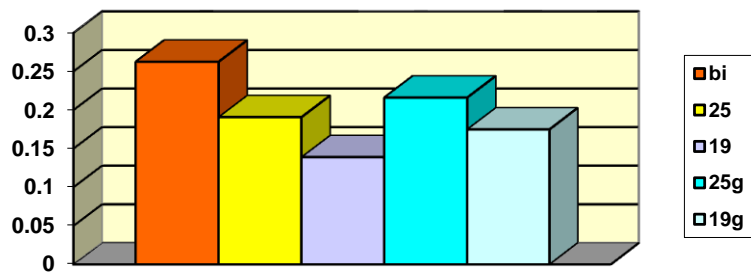


Figure 3: Comparative values of periods of the first tone of vibration for the three-span four-storey frame: 4.8 m width, 3 m ground-floor height without infill (bi), with 25 cm thick infill (25), with 19 cm thick infill (19), with 25 cm thick infill except in the ground floor (25g), and 19 cm thick infill except in the ground floor (19g)

4 DISCUSSION OF THE RESULTS

By comparing the results of analysis of the first period of vibration of 4.8 m wide single-span and three-span four-storey RC frame models for cases without infill, infilled with ordinary solid 25 cm thick brick wall, and infilled with 19 cm thick hollow ceramic block wall, all obtained values are shown in tabular and graphic forms, along with the percentage difference in relation to frames without infill. The results show that the presence of infill reduces the periods of vibration, and that the reduction depends on the position of the infill and its stiffness, but not on the thickness of the wall. The largest reduction of stiffness both in single and three-span frames occurs in the case of high-stiffness infill. In single-span frames, the applications of infill of higher stiffness (although it is a lower wall thickness) resulted in the reduction of the period of vibration for 42.21% relative to the frame without infill, while in the three-span frame this reduction was 47.10%. The next levels of reduction were induced by infills of higher stiffness left out in the ground floor, infills of lower stiffness in all floors, and infills of lower stiffness left out in the ground floor, respectively. In three-span frames periods of vibration are lower than in single-span frames, but the percentage of decrease in the value of the period is higher due to the presence of infill.

5 CLOSING REMARKS

The results of the analysis of models of RC frame structures with varying number of spans and varying types and positions of infill have shown that these parameters have a significant effect on the periods of vibration, and thus the seismic response of the structure. Comparing the periods of vibration is part of a wider analysis of observing a larger number of parameters that influence the seismic performance. In further research a larger number of models are planned to be analyzed and compared with the recommended empirical patterns of determining the period.

ACKNOWLEDGEMENTS

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Milovan Stanojev¹, Radomir Folić²

PROMENE UTICAJA U ELEMENTIMA KONSTRUKCIJA NAKON FORMIRANJA PLASTIČNIH ZGLOBOVA

Rezime:

S-12

Koristeći linearnu analizu konstrukcije prostornog modela armirano-betonske zgrade dobijaju se uticaji na osnovu kojih se vrši dimenzionisanje elemenata sa dosta rezerve. Optimizacija i racionalizacija pri projektovanju nameću potrebu za tačnijim shvatanjem i preciznijim prikazom stanja napona u konstrukciji. Nelinearnom analizom konstrukcije se dobija uvid u plastično ponašanje elemenata, pri čemu je iskorišćenost preseka mnogo veća, što za rezultat ima i racionalniju konstrukciju. U ovom radu je prikazana uporedna analiza vrednosti uticaja u linijskim elementima konstrukcije pre i nakon formiranja plastičnih zglobova u čvorovima, pod dejstvom ubrzanja tla zadatim vremenskom istorijom zemljotresa EL Centro.

Ključne reči: Uticaji, AB zgrada, nelinearna analiza, preseci, plastični zglobovi

CHANGES OF INFLUENCES IN STRUCTURE ELEMENTS AFTER PLASTIC HINGES FORMATION

Summary:

By using linear analysis of spatial model of RC building structure influences we get influences which represents basis for dimensioning elements with lot of reserve. Optimization and rationalization in structural designing imposes the need for more accurate and more precise view of stresses state in construction. Nonlinear structural analysis provides insight in plastic behaviour of elements, whereby the utilization of cross-sections is greater, which generates more rational structure. This paper presents a comparative analysis of influences in line structural elements before and after plastic hinges formation in joints, under the effect of time history acceleration of El Centro earthquake.

Key words; Influences, RC building, nonlinear analysis, cross-sections, hinges

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1 INTRODUCTION

In seismically active regions (such as entire Balkan region) earthquake induced load is usually taken as the dominant load and applicable for dimensioning reinforced concrete structural elements. Therefore, design of structures in seismically active regions has always been a challenge for structural engineers, in terms of realistic observation of seismic effects on structures. Linear analysis can still provide the correct results for dimensioning typical structures. If the effect of the earthquake on the structure is approximated as an equivalent distributed load at the ceiling level, approximate influences of the structural elements can be provided, which are often dominant at dimensioning such elements. However, the main disadvantage of this calculation is the linear capacity of elements with elastic behaviour under gravity and seismic loads and inability to fully utilize the capacity of materials (reinforcement and concrete). For taller buildings application of method of linear analysis generates much larger cross-sections of characteristic structural elements, especially for columns and beams in the lower floors. Thus higher rigidity of structural elements and construction itself are obtained, but at the same time greater seismic forces in the event of earthquake are induced in structure, after what structure is initially opposed by its rigidity. However, linear analysis is still often used. Whether it is a linear analysis (Fig.1) based on strength (linear-static analysis), or on the basis of response-spectrum (dynamic linear analysis), compared to non-linear analysis method of calculation is much simpler, with shorter calculation time.

<u>Analysis Type</u>	Linear	Nonlinear
Static	Strength-based	Static-pushover
Dynamic	Response-spectrum	Time-history

Figure 1 – Analysis methods [1]

On the other hand, if the yield and ultimate strength of the material is well-known (necessary experimental testing of materials) and if elements capacity or ultimate bearing capacity of the whole structure is needed, the application of non-linear analysis is required. Using modern computers, structures can be analyzed through a large number of iterations, and the use of iterative steps quickly come up with new stiffness matrix at each step of the nonlinear analysis. Nonlinear behaviour of the structure is given by the geometric nonlinearity (using the P- Δ effect that is by assigning the load on the deformed structure) and material nonlinearity (changing the initial stiffness of the material causes the inelastic behaviour of the element, system or connection). Ductility or individual elements can quite accurately be determined by applying Static-pushover analysis (Fig.1), through deformation parameters (translation and rotation). Thus, optimum and rational solution can be obtained. However, the most accurate (and computationally complicated) process of nonlinear analysis is certainly the Time-history analysis, because the load in this case is a function of time so the equations of motion can be set for each time step. The application of time-history analysis can simultaneously take into account material nonlinearity and P- Δ effects.

2 PERFORMENS-BASED DESIGN

After several strong earthquakes in major cities (Loma Prieta in 1989 and Northridge in 1994 in California, as well as the 1995 Kobe earthquake in Japan), human casualties with material damage (especially technical equipment in buildings), as well as costs of repairs and relocation of business and commercial activities in densely populated urban areas have become unacceptably high [3]. Seismic design based on performances represents new flexible philosophy [2] and modern comprehensive approach to seismic design of buildings and other structures [4], which enables structural performances to be ensured for several different levels of seismic hazard. In 1995, under the supervision of the Federal Emergency Management Agency (FEMA), recommendations and guidelines were issued for the first time in the United States for seismic design as a document FEMA-273, along with the later released documents the US/FEMA-350, FEMA-356, FEMA-440, ASCE-31 and ASCE-41 [8]. Although these recommendations were initially intended to assist engineers in assessing the state of existing structures after very strong seismic activity and their post-elastic behaviour, they are later included in all major international standards for seismic design of new structures [5].

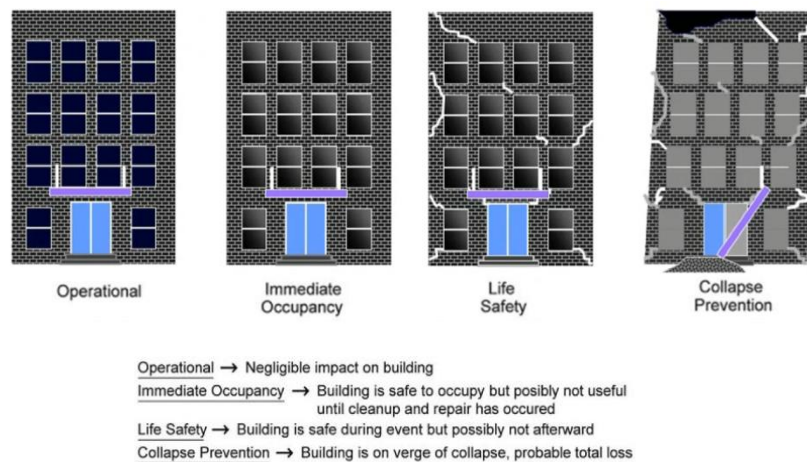


Figure 2. Qualitative definition of seismic performance levels [7]

In the context of performance-based seismic design (PBSD), multiple levels of seismic performance of structure are discreetly and qualitatively defined (Figure 1). These performance levels correspond to various limit states of structure, depending on the damage it suffered [4], [7]. Thus, after severe earthquake construction can be classified by four different states [10]:

- Operational state (negligible impact on building);
- Immediate occupancy (building is safe to occupy but possibly not useful until cleanup and repair has occurred);
- Life Safety (building is safe during event, but possibly not afterward);
- Collapse prevention (building is on verge of collapse, probable total loss) [1].

With dynamic nonlinear analysis (Time history) detailed calculation can be generated using Fast Nonlinear Analysis (FNA) based on modal analysis, or method of direct integration, where the equations of motion are set at each iterative step. Assigning seismic loads on the

structure through the ground displacement, speed or acceleration based on time histories records, behaviour of connections, elements and structure as a whole can be accurately calculated under the effect of a given earthquake. Depending on the type of building (residential buildings, hospitals, schools, public institutions, ancillary buildings, temporary facility ...) and estimated intensity of potential earthquakes, designer determines how the structure should be „protected " or to which damage are permanent deformations allowed.

3 METHOD OF CAPACITY DESIGN

Process in aseismic engineering in which structural engineer determines which structural elements are allowed to be nonlinear (ductile components), while others remain elastic (brittle components) is Capacity design [9]. If the calculation is run by capacity design, structure's performances are deliberately determined by engineer, and unrevealed in a secondary manner by computational tools. Capacity Design enables the creation of a more reliable computational model, which should lead to better structural design.

Also, when an engineer knows which elements will be permitted to yield while other elements will behave elastically, material nonlinearity need only to be modelled for ductile components, while components which will not yield need only to consider elastic stiffness properties.

Particularly sensitive main structural elements are columns, which have to be ductile, especially in lower floors. Depending on analysis method, influences in columns (in particular axial forces) may vary very much so that even tensile forces can be generated in lower columns using linear analysis under impact of severe earthquakes. Still, using nonlinear analysis, with formation of hinges in joints, variations in axial forces are less noticeable, which provides smaller cross-sections with smaller amount of reinforcement, especially in tall buildings and large objects. Column cross-sections are dimensioned so that gravitational normal force corresponds to the limit at the level of balance point, where possibility of receiving bending moment is the highest. At this level of load, the interaction between normal forces and bending moments is such that expansion in concrete and reinforcement is about the same, so the cross-section has the best possible ductility, while in the case of fracture, concrete and reinforcement are expected to suffer simultaneous failure.

For example, Fig.3 shows three same cross-sections, but with different reinforcement ratios. It can be seen that in the case of symmetric balance point is at the same axial value for both cross-section sides. In the case of asymmetrical reinforcement, balance point moves depending of stiffness, so that the stronger side of a reinforced cross-section later comes to yield in the reinforcement and concrete, but on the weak side this occurs earlier, so that the case of asymmetrically reinforced cross-section is not suggested for cyclical, harmonic and stochastic loads, such as earthquake. This is one of the reasons why the columns should be designed as a square or round, with symmetrically distributed reinforcement along the section.

Unlike columns, the beam's bending stiffness should be at least 25 to 40% lower than that of the corresponding column [6], to ensure the proper development of plastic hinges in them. Their behaviour in the post-elastic range is easier to predict than in columns in which assures interaction between axial forces and bending moments (P-M2-M3), while in beams plastic hinge represents a concentrated post-plastic (post-yield) behaviour in one or more degrees of freedom around axis 3 (M3), while other degrees of freedom remain elastic.

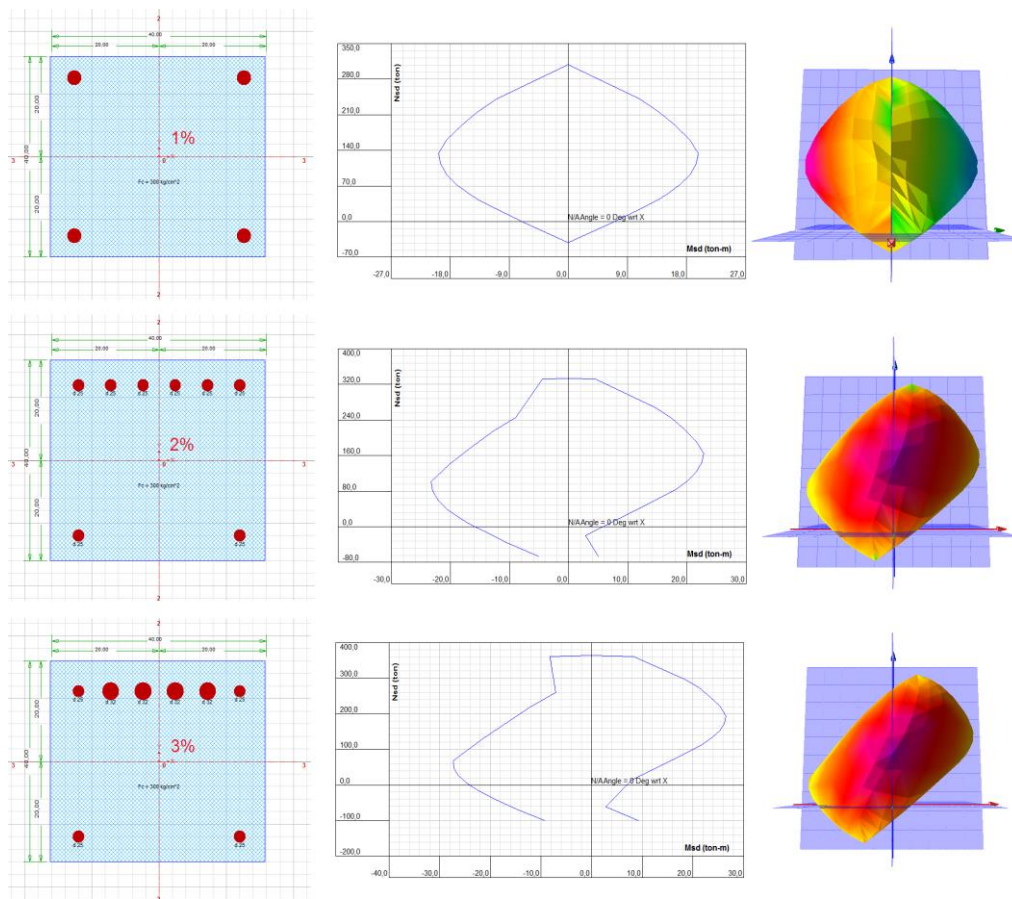


Figure 3. Interaction diagrams for square section with symmetrical and nonsymmetrical reinforcement

As for the vertical area bearing elements (walls and RC cores) they are also designed as ductile elements (especially in the case of RC cores in the building, which then absorbs the largest part of the seismic forces), while the plastic hinges are defined as regular contact elements. Unlike them, RC slabs are mainly designed as stiff solid elements without the possibility of formation of plastic hinges, which are mainly related to the transfer of lateral seismic effects on the surrounding columns and RC core.

4 CHANGES IN INFLUENCES – A CASE STUDY

Changes in influences in beams and columns on case study can be analyzed on the example of a RC building. It is a BF + GF + 5 building with a 6.0 m + 6.0 m span in the X direction, and 10.3 m + 3.0 m + 6.3 m in the Y direction, while the floor height is 3.2 m. Due to unfavourable formation of plastic hinges, up to the third floor the dimensions of columns are 60x60 cm, after that they are 40x40cm, while the cross-section of beams remains 25x40cm. The nonlinear

analysis was conducted in the CSI ETABS 2015 software using the time history of the El Centro earthquake (Imperial Valley, California, 1940), in duration of 12s, with the maximum acceleration peaks occurring between 1.5s and 2.5s of the earthquake duration (Figure 3). The time-step of applying the acceleration is 1/100.

Plastic hinges at beam ends were set for the bending moment around the major axis M3, while in columns plastic hinges up to the nodes were set for three degrees of freedom P-M2-M3. The behaviour of structural elements in the plastic range is defined using a force-displacement diagram (Fig. 4), with the marked points that correspond to limit states defined through performance-based seismic design (PBSD) as: immediate occupancy (IO), life safety (LS) and collapse prevention (CP).

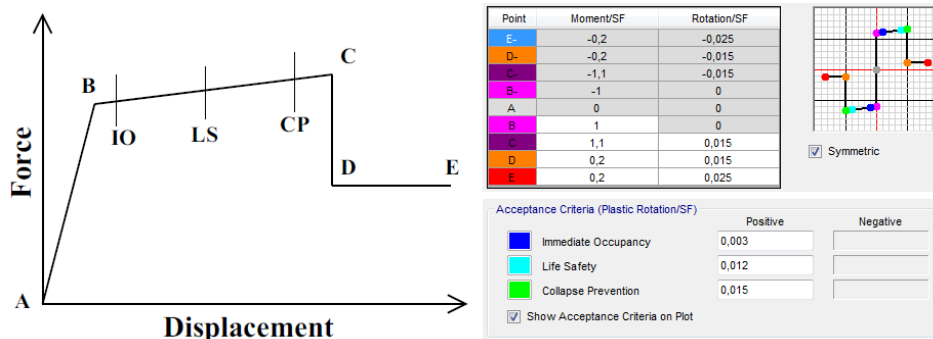


Figure 4. Force-displacement diagram used in ETABS 2015 (left), with points of acceptance criteria for seismic performance on $M-\phi$ diagram (right)

Figure 5 shows the diagrams of axial forces in columns of characteristic traverse frame across the time history of the El Centro earthquake with a peak acceleration of 0.32G. The effects are first obtained in the frame of the model without plastic hinges being set, then in the frame with plastic hinges set only in beams, and finally in the frame with defined plastic hinges in all beams and columns. It can be seen that in the model without plastic hinges significantly lesser axial forces can occur in the edge columns under strong acceleration of the structure, relative to central columns. With plastic hinges generated in beams there comes to stress redistribution so that axial pressure forces in edge columns are greater, while the stress state in the central columns remains almost the same.

However, with the formation of plastic hinges at the ends of line elements in the frame (which is best suited to the situation in reality) despite high variation in axial forces, they are almost twice as high as compared to the case without the plastic hinges, which is very important both for column dimensions and due to the global stability of the structure. With the formation of plastic hinges in joints, their stiffness decreases, so the distribution of axial forces is more uniform, and you cannot expect a as small axial forces in the edge columns as is the case with linear analysis.

An alternative direction of earthquake action leads to a high variation of axial forces in end columns. This is one of the main reasons why most regulations stipulates that stresses in columns under the effect of gravitational loads should be between 30 and 40% of capacity of

the column, providing its cross-section with capacity to counter the incidental actions, such as seismic hazard.

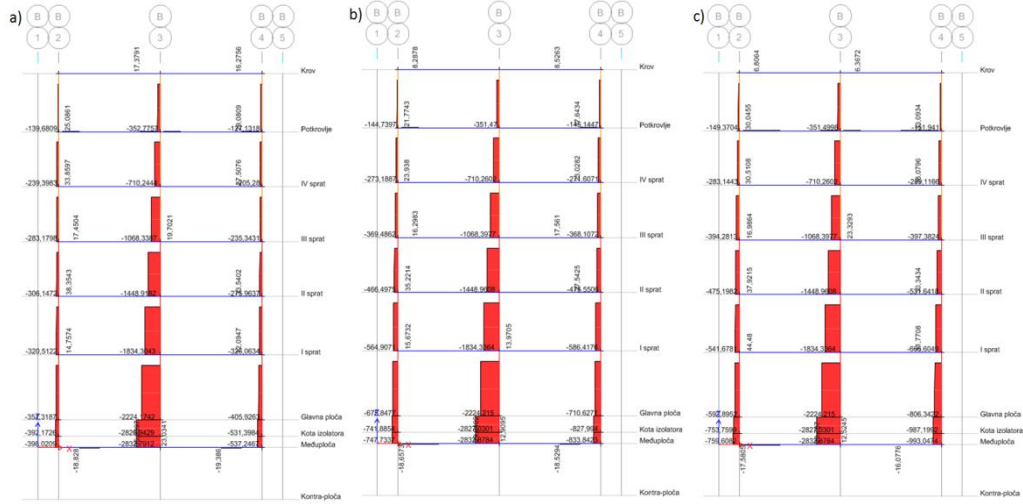


Figure 5. Axial forces in characteristic frame: a) without hinges, b) with hinges only in beams, c) with hinges in beams and columns

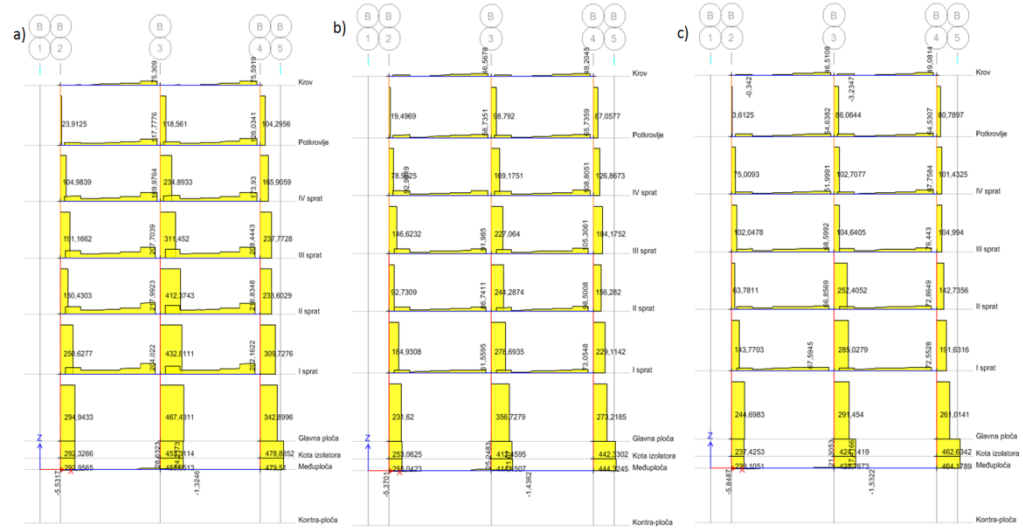


Figure 6. Shear forces in characteristic frame: a) without hinges, b) with hinges only in beams, c) with hinges in beams and columns

In Figure 6 there is a variation in transverse forces which are the largest in the linear analysis, while after the formation of plastic hinges transverse forces are reduced, both in the bearing linear elements, so that the nonlinear analysis generates less reinforcement compared to traditional linear analysis.

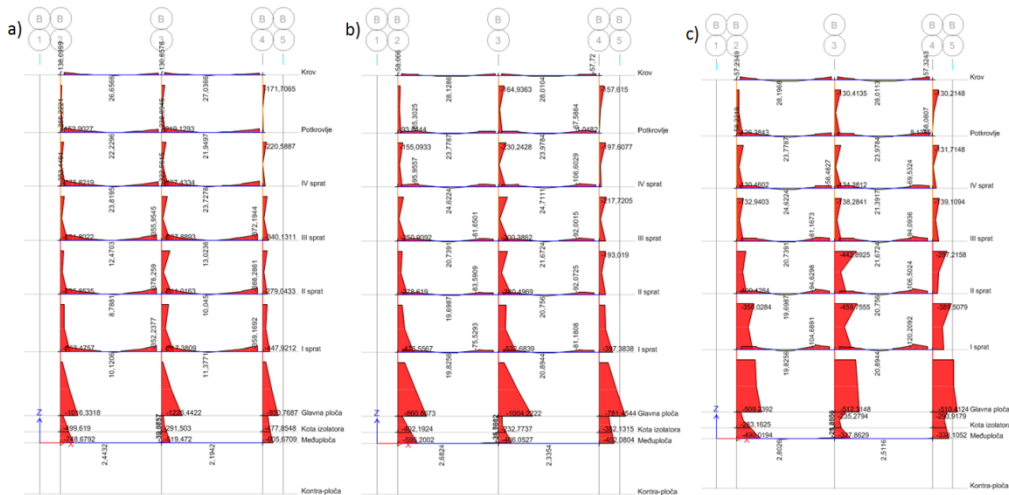


Figure 7. Bending moments in characteristic frame: a) without hinges, b) with hinges only in beams, c) with hinges in beams and columns

The largest bending moments on the lower floors appear in linear analysis (Fig.7), decreasing with the formation of plastic hinges in beams, while the lowest are in the third case, where the formation of the plastic hinge comes in columns as well. However, it can be seen that in the first case on the higher floors comes to sharply reduced bending moments in relation to the ground floor, while at the nonlinear model that transformation takes place significantly more regular. This results in a much larger cross-sections and greater reinforcement in columns on lower floors, which are calculated by linear analysis, whereas in nonlinear analysis rational cross-sections are obtained with much better distribution of stresses in the columns. As for the beams, the largest impacts at the ends of the beams are reappearing in the linear analysis and on all floors, and here draws the conclusion that nonlinear analysis, with submission of plastic hinges in the model and the proper guidance of reinforcement can lead to rationally design with a more realistic picture of stresses in the structural elements.

5 CONCLUSIONS AND SUGESTIONS

Recommendations and guidelines for achieving structural performances for different levels of hazards can be found in US documents FEMA-273, FEMA-356, FEMA-440... These performance levels correspond to various limit states of the structure, depending on the damage it suffered, so four different states are defined in which structure can be classified after earthquake hazard: Operational state (OS), Immediate occupancy (IO), Life Safety (LS) and Collapse prevention (CP).

Depending on the type of building (residential buildings, hospitals, schools, public institutions, ancillary buildings, temporary facility ...) and the estimated intensity of potential earthquakes, designer determines how the structure must be „protected" or to which damage permanent deformations are allowed.

Capacity design is a process in aseismic engineering in which structural engineer determines which structural elements are allowed to be nonlinear (ductile components), while others remain elastic (brittle components). It enables the creation of a more reliable computational model, which should lead to better structural design. Also, when an engineer knows which elements will be permitted to yield while other elements will behave elastically, material nonlinearity need only to be modelled for ductile components, while components which will not yield need only to consider elastic stiffness properties.

Column cross-sections are dimensioned so that gravitational normal force corresponds to the limit at the level of balance point, where possibility of receiving bending moment is the highest. At this level of load, the interaction between normal forces and bending moments is such that expansion in concrete and reinforcement is about the same, so the cross-section has the best possible ductility, while in the case of fracture, concrete and reinforcement are expected to suffer simultaneous failure.

In the case of asymmetrical reinforcement, balance point moves depending of stiffness, so that the stronger side of a reinforced cross-section later comes to yield in the reinforcement and concrete, but on the weak side this occurs earlier, so that the case of asymmetrically reinforced cross-section is not suggested for cyclical, harmonic and stochastic loads, such as earthquake. This is one of the reasons why the columns should be designed as a square or round, with symmetrically distributed reinforcement along the section. Unlike columns, the beam's bending stiffness should be at least 25 to 40% lower than that of the corresponding column, to ensure the proper development of plastic hinges in them. Their behaviour in the post-elastic range is easier to predict than in columns in which assures interaction between axial forces and bending moments (P-M2-M3), while in beams plastic hinge represents a concentrated post-plastic (post-yield) behaviour in one or more degrees of freedom around axis 3 (M3), while other degrees of freedom remain elastic.

Walls and RC cores are also designed as ductile elements (especially in the case of RC cores in the building, which then absorbs the largest part of the seismic forces), while RC slabs are mainly designed as stiff solid elements without the possibility of formation of plastic hinges, which are mainly related to the transfer of lateral seismic effects on the surrounding columns and RC core.

On the case study of RC frame building exposed to severe El Centro earthquake it can be seen that in the model without plastic hinges significantly lesser axial forces can occur in the edge columns under strong acceleration of the structure, relative to central columns, while in nonlinear model with generated plastic hinges axial forces in edge columns can be 100% greater relative to linear analysis, which is very important both for column dimensions and global stability of the structure.

An alternative direction of earthquake action leads to a high variation of axial forces in end columns. This is one of the main reasons why most regulations stipulates that stresses in columns under the effect of gravitational loads should be between 30 and 40% of capacity of the column, providing its cross-section with capacity to counter the incidental actions, such as seismic hazard.

Variations in transverse forces are the largest in linear analysis, while after the formation of plastic hinges transverse forces are reduced, both in the columns and the beams. This has a direct impact on the transverse reinforcement (stirrups) in bearing linear elements, so that the nonlinear analysis generates less reinforcement compared to traditional linear analysis.

In the case of linear analysis it can be seen that bending moments at the higher floors are sharply reduced in relation to the ground floor, while at the nonlinear model that transformation takes place significantly more regular. This results in a much larger cross-sections and greater reinforcement in columns on lower floors, which are calculated by linear analysis, whereas in nonlinear analysis rational cross-sections are obtained with a much better distribution of stresses in the columns. As for the beams, the largest impacts at the ends of the beams reappear in the linear analysis and on all floors, and thus it may be concluded that nonlinear analysis, with submission of plastic hinges in the model and proper guidance of reinforcement can lead to rationally design with a more realistic picture of stresses in structural elements.

Acknowledgements

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POŽARNA OTPORNOST SLOBODNO OSLOJENIH AB PLOČA NOSIVIH U JEDNOM PRAVCU

Rezime:

S-13

U radu su prikazani numerički rezultati požarne otpornosti slobodno oslonjenih AB ploča, nosivih u jednom pravcu. Analiziran je uticaj debljine zaštitnog sloja, procenta armiranja, raspona i debljine ploče. Požarna otpornost ploča je definisana primenom 'metode granične nosivosti'. Ova savremena metoda koja se koristi za proračun konstrukcija, koristi se i za definisanje požarne otpornosti. Prezentirani su i upoređeni rezultati dobiveni primenom 'uprošćene' proračunske metode, opisane u Eurocodu 2 dio 1.2, i primenom numeričkog programa FIRE.

Кljučне речи: AB ploča, požarna otpornost, zaštitni betonski sloj, procent armiranja

FIRE RESISTANCE OF ONE-WAY SIMPLY SUPPORTED REINFORCED CONCRETE SLABS

Summary:

This paper presents the numerically achieved results for the fire resistance of one-way simply supported reinforced concrete slabs. The effects of: concrete cover thickness, reinforcement ratio, slab span and thickness were analyzed. The fire resistance of the RC slabs was defined by 'ultimate strength design' criteria as used in all modern concrete design codes. Results obtained from 'simplified' calculation method, described in Eurocode 2 part 1.2, and by using the software FIRE are presented and compared in this paper.

Key words: RC slab, fire resistance, concrete cover thickness, steel ratio

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1 INTRODUCTION

The resistance of building members to the effects of fire is an important part of any structural and fire safety design. Floor slabs, as horizontal elements, have a very important role in providing bearing capacity, usability and stability of the building as a whole. Their proper selection and design, when they are exposed to different types of loads (mainly: permanent and variable), should provide stable and safe structure during the exploitation period.

In case of fire floor slabs do not have only **load bearing function**. In most cases they are used as elements for separating the fire compartment. Where compartmentation is required, the elements forming the boundaries of the fire compartment, including joints, shall be designed and constructed in such a way that they maintain their separating function during the relevant fire exposure [1]. This shall ensure, where relevant, that **integrity failure** does not occur, **insulation failure** does not occur, thermal radiation from the unexposed side is limited.

The criterion **Integrity (E)** expresses the ability of the separating element of the building construction, when exposed to fire on one side, to prevent the passage of flames and hot gases through it and to prevent the occurrence of flames on the unexposed side.

The criterion **Insulation (I)** expresses the ability of the separating element of the building construction when exposed to fire on one side, to restrict the temperature rise of the unexposed face below specified levels.

The criterion **Load bearing function (R)** expresses the ability of the structure or the member to sustain specified actions during the relevant fire, according to defined criteria. Criterion "I" may be assumed to be satisfied where the average temperature rise over the whole of the non-exposed surface is limited to 140 K, and the maximum temperature rise at any point of that surface does not exceed 180 K [1].

Does the floor structure meet the required fire resistance criteria mainly depends on: mechanical and thermal characteristics of the materials used for the construction; initial loading level; support conditions; dimensions of the cross section; steel ratio; concrete cover thickness and fire scenario.

The fire resistance of the massive simply supported reinforced concrete slabs which have load bearing capacity only in one direction with different thickness and spans, used for the multi story residential buildings, is analyzed in this paper.

A durable structure shall meet the requirements of serviceability, strength and stability throughout its design working life, without significant loss of utility or excessive unforeseen maintenance. In fire conditions, as a result of a large number of real fire tests and corresponding numerical analyzes, it was found out that the moment of failure of the floor structure is always followed by significant deformation (deflection). If the structure is close to the limit state, after the cooling phase the residual deflections are so great that it cannot be used without significant rehabilitation. For these reasons, during the fire action, the deformation (deflection) of the slab is limited to prescribed value. According to the ISO standard, this limit value is $L/30$ (L is the span of the slab) [3].

This paper presents the numerically achieved results for the fire resistance of nine solid RC slabs with different span and thickness. For comparison, all slabs were analysed as simply supported slabs and were exposed to ISO standard fire from the bottom side, as most usual fire scenario. The RC slabs were constructed without thermal insulation at the bottom side of the

slabs. Based on the numerically achieved results, certain conclusions that can be useful for meeting the prescribed fire resistance of these type of floor structures were obtained.

2 FIRE RESISTANCE OF ONE-WAY SIMPLY SUPPORTED RC SLABS

The computer program FIRE (Cvetkovska, Ss. Cyril and Methodius University in Skopje, Macedonia), based on Finite Element Method, was used for the fire resistance analysis of the simply supported RC slabs treated in this paper. This program is capable of conducting the nonlinear and transient heat flow analysis and nonlinear stress-strain response associated with fire.

The program FIRE [4] consists of two modules. The modulus FIRE-T solves the governing differential equation of heat transfer in conduction by taking into account the fire boundary conditions modeled in terms of both convective and radiating heat transfer mechanisms. The response of reinforced concrete elements and plane frame structures exposed to fire is predicted by modulus FIRE-S. This modulus accounts for: dimensional changes caused by temperature differences, changes in mechanical properties of materials with changes in temperature, degradation of sections by cracking and/or crushing and acceleration of shrinkage and creep with an increase of temperature.

A 'simplified' calculation method described in Eurocode 2 [1] was also used for the fire resistance analysis of the simply supported RC slabs treated in this paper. Wickström's formula is used to calculate the reinforcing temperature.

2.1 SIMPLIFIED CALCULATION METHOD

There are simplified methods for calculation the fire resistance of one-way slabs. Their application is possible only for simply supported and continuous slabs exposed to fire only from the bottom side.

The design procedure for concrete simply supported slabs assumes the following: concrete has no tensile strength and the parabolic compressive block in concrete can be approximated by an equivalent rectangle, which can be seen in (Figure 1). It is also assumed that the compressive block does not rise at elevated temperatures which causes reduction in material properties and that the flexural capacity is solely a function of the temperature of the reinforcing steel.

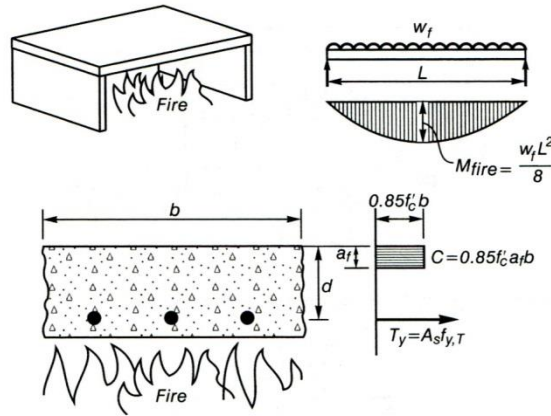


Figure 1 – One-way simply supported RC slab exposed to fire from the bottom side

There is no possibility of moment redistribution in case of simply supported slabs. The design equation for a member subjected to a bending moment M_{fire} is:

$$M_{fire} \leq M_{u,f}$$

The flexural capacity under fire conditions is given by:

$$M_{u,f} = A_s f_{y,T} (d - a_f / 2)$$

where:

A_s - the area of the reinforcing steel,

$f_{y,T}$ - the yield stress of the reinforcing steel, temperature reduced ($f_{y,T} = k_{y,T} f_y$),

d - the effective depth of the cross section (distance from the extreme compressed fiber to the centroid of the reinforcing steel),

a_f - the depth of the rectangular stress block, reduced by fire and given by:

$$a_f = \frac{A_s f_{y,T}}{0.85 f'_{cd} b}$$

where:

f'_{cd} - the design value of compressive strength of the concrete at room temperature,

b - the width of the slab strip.

Most of countries in Europe apply ISO 834 Standard fire curve, given by expression:

$$T_f = 20 + 345 \log(8t + 1)$$

Wickström's formula, which is used to calculate the reinforcement temperature, is given by expressions:

– Surface temperature:

$$T_w = \left[1 - 0.616t_h^{-0.88} \right] T_f$$

– Concrete temperature at depth 'c_e':

$$T_c = \left[0.18 \ln \left(\frac{t_h}{c_e^2} \right) - 0.81 \right] T_w$$

– Steel temperature

$$T_s = T_c$$

3 RC SLAB ANALYSIS

The following parameters, characteristics and conditions were assumed:

- The slabs were exposed to ISO 834 Standard fire only from the bottom side,
- Separate 1 m wide strips were analysed,
- The temperature dependent physical and mechanical properties of the siliceous aggregate concrete (compressive strength $f_{ck}=30\text{Mpa}$) and the reinforcement (yield strength $f_{yk}=400\text{Mpa}$) were assumed according to EC2, part 1-2,
- Dead load $G_1=1.5 \text{ kN/m}^2$ (excluding self weight),
- Live load $Q=4.0 \text{ kN/m}^2$, reduction factor in case of fire $\Psi_{2,1}=0.6$ (for category C)

All data about the slabs geometry and the intensities of the dead and live loads are given in Table 1.

Table 1 – Slab geometry and load intensity

Slab	Span (m)	Thickness (cm)	Concrete cover (cm)	G_{slab} (kN/m ²)	G_1 (kN/m ²)	Q_1 (kN/m ²)	$P_{\text{slab,cold}}$ (kN/m ²)	$P_{\text{slab,fire}}$ (kN/m ²)
S ₁	4	16	2.5	4	1.5	4	13.425	7.900
S ₂	4	16	3	4	1.5	4	13.425	7.900
S ₃	4	16	3.5	4	1.5	4	13.425	7.900
S ₄	4	17	2.5	4.25	1.5	4	13.763	8.150
S ₅	4	18	2.5	4.5	1.5	4	14.100	8.400
S ₆	5	16	2.5	4	1.5	4	13.425	7.900
S ₇	5	17	2.5	4.25	1.5	4	13.763	8.150
S ₈	5	18	2.5	4.5	1.5	4	14.100	8.400
S ₉	6	16	2.5	4	1.5	4	13.425	7.900
S ₁₀	6	17	2.5	4.25	1.5	4	13.763	8.150
S ₁₁	6	18	2.5	4.5	1.5	4	14.100	8.400

As first case study the criterion **Load bearing function (R)** was analysed. The fire resistance for all case studies is given in Table 2.

Table 2 – Fire resistance of RC slabs

Slab	Steel ratio (%)	Fire resistance (min) simplified calculation method	Fire resistance (min) program FIRE
S ₁	0.392	124.13	122.4
S ₂	0.415	158.68	151.8
S ₃	0.441	190.78	184.2
S ₄	0.369	128.06	130.8
S ₅	0.348	131.45	133.2
S ₆	0.706	140.18	138.6
S ₇	0.665	143.69	141.6
S ₈	0.628	146.70	142.2
S ₉	1.068	147.29	148.2
S ₁₀	1.005	151.05	151.2
S ₁₁	0.950	155.05	153

The results show the effect of the concrete cover thickness, the span and thickness and the steel ratio on the fire resistance of the RC slabs. The results obtained by simplified calculation method and the program FIRE are presented and compared. The results for the slabs S₁, S₂ and S₃ have shown that the concrete cover thickness has significant effect on the fire resistance of the RC slabs. In case when the concrete cover thickness is increased, the fire resistance of slab is increased too. From sets S₁,S₄,S₅; S₆,S₇,S₈ and S₉,S₁₀,S₁₁ it could be concluded that in case when the slab thickness is increased, the fire resistance of the slabs is increased too. In same time, the positive effect of the slab thickness on the criterion **Insulation (I)** is shown in Figure (9). For higher values of the slab thickness the temperature on the unexposed surface of the slab was less.

All the results for the analysed slabs are presented in Figures (2-8).

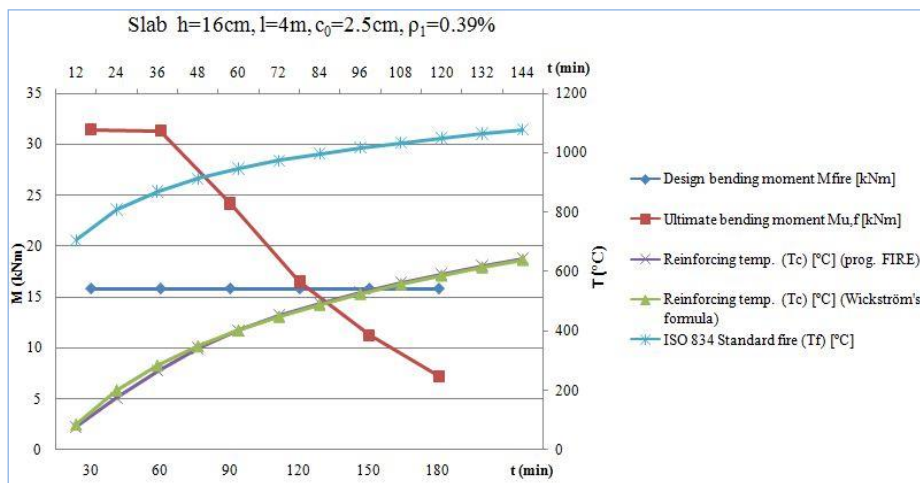


Figure 2 – Fire resistance of one-way simply supported RC slab with 4m span, 16cm thickness and 2.5cm concrete cover thickness

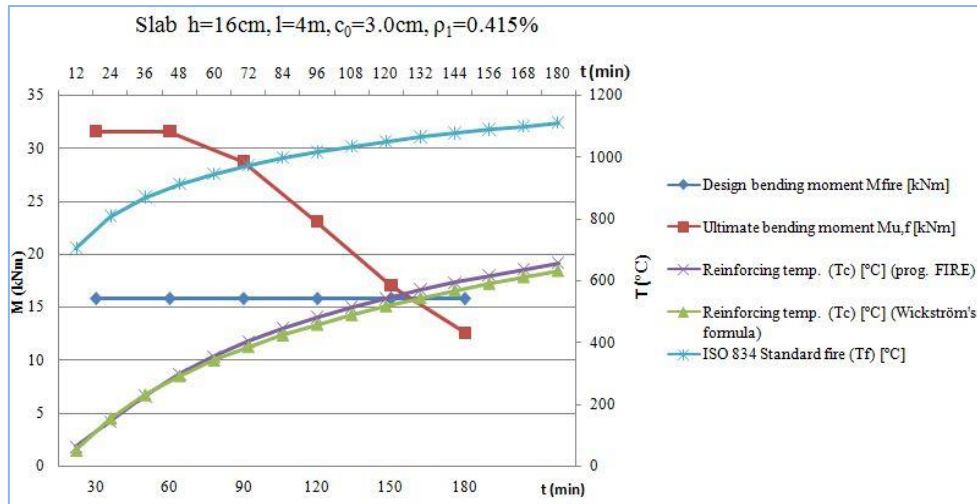


Figure 3 – Fire resistance of one-way simply supported RC slab with 4m span, 16cm thickness and 3.0cm concrete cover thickness

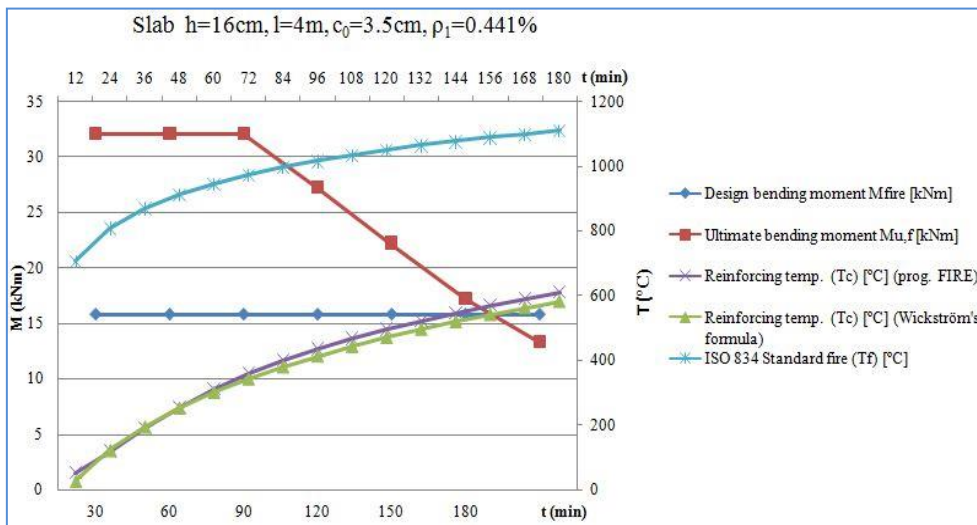


Figure 4 – Fire resistance of one-way simply supported RC slab with 4m span, 16cm thickness and 3.5cm concrete cover thickness

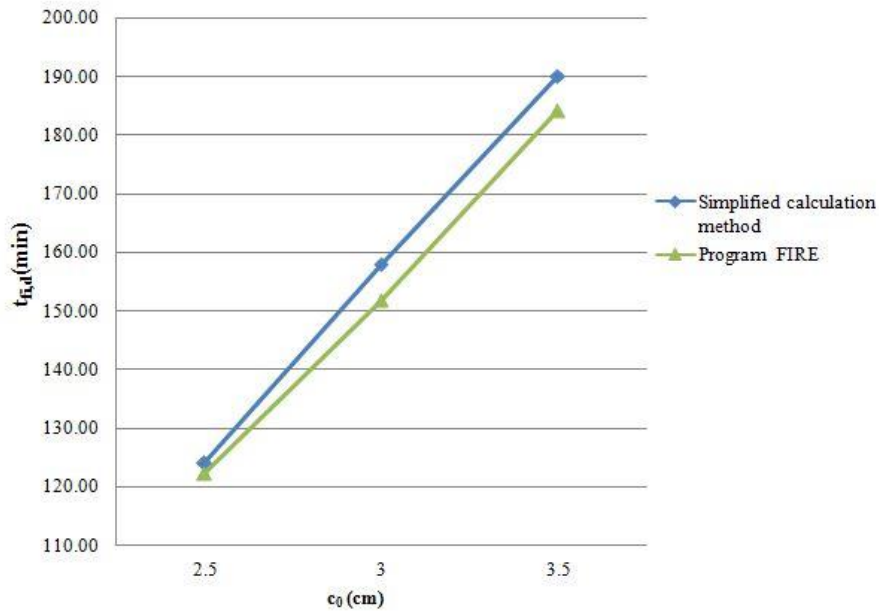


Figure 5 – The effect of concrete cover thickness on fire resistance of one-way simply supported RC slab, 4m span, 16cm thickness and comparison of the results obtained with the simplified calculation method and the program FIRE

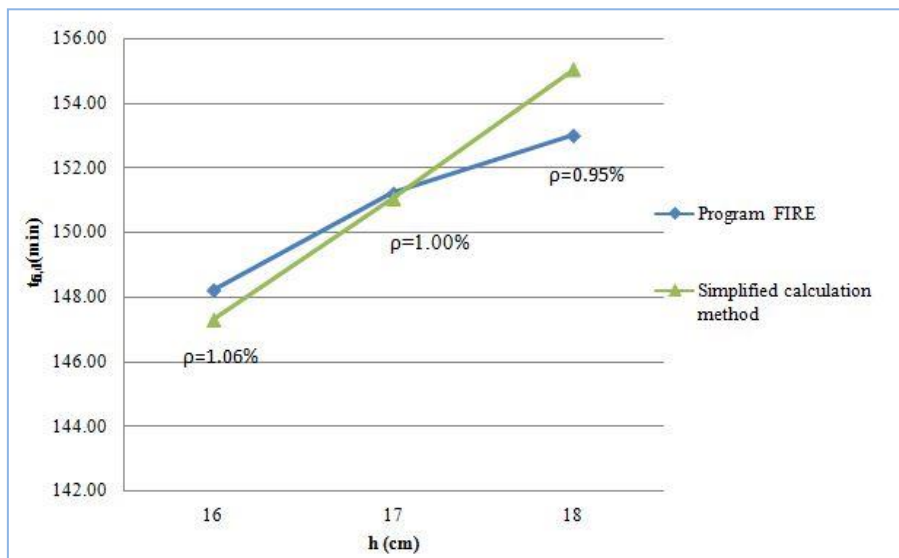


Figure 6 – Fire resistance of one-way simply supported RC slab, 5m span, 2.5cm concrete cover thickness, for different slab thicknesses, and comparison of the results obtained with the simplified calculation method and the program FIRE

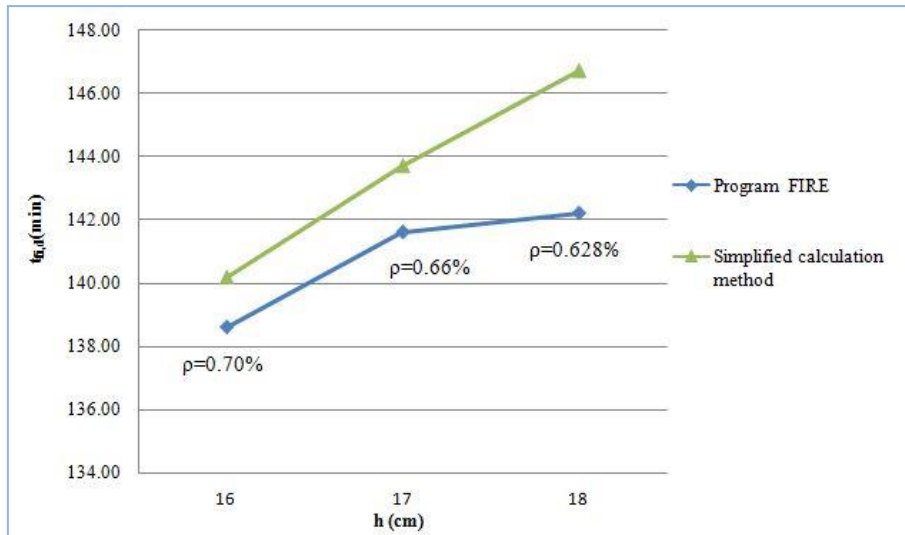


Figure 7 – Fire resistance of one-way simply supported RC slab , 6m span and 2.5cm concrete cover thickness, for different slab thicknesses, and comparison of the results obtained with the simplified calculation method and the program FIRE

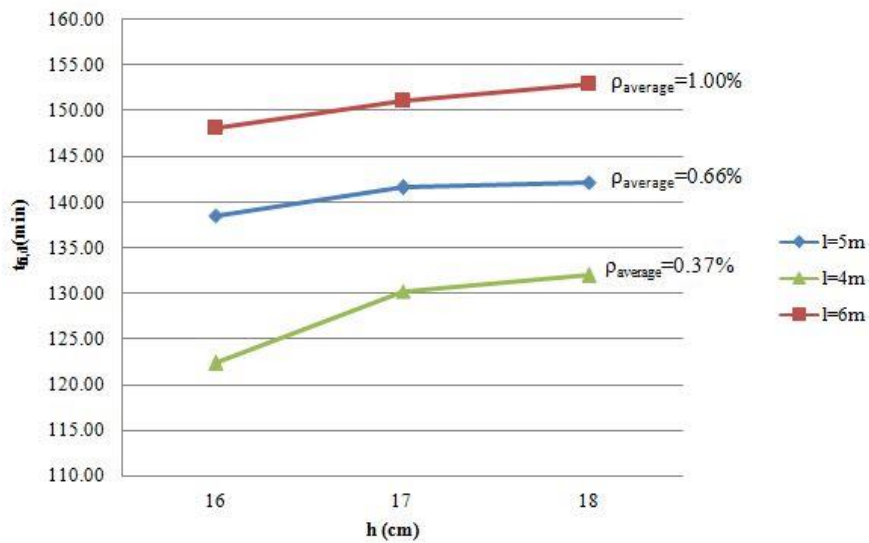


Figure 8 – Relation 'slab span-fire resistance' of one-way simply supported RC slab

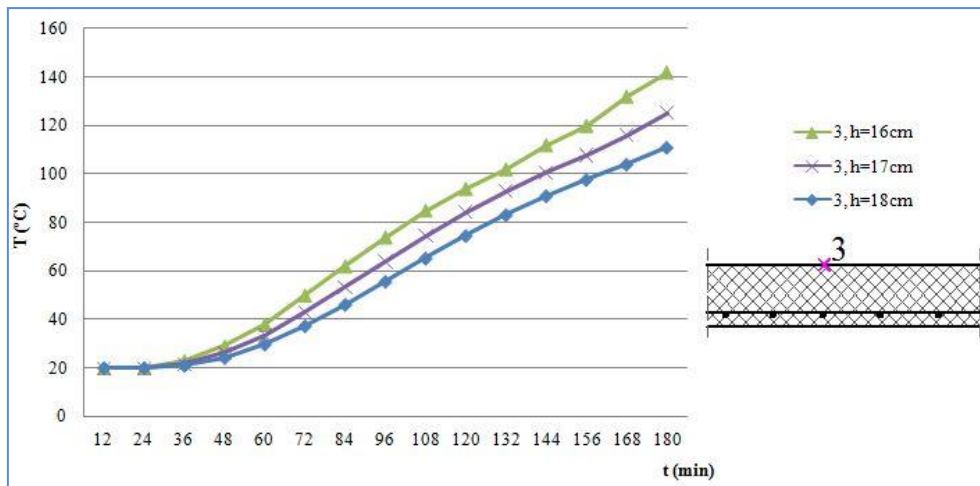


Figure 9 –The effect of slab thickness on criterion I, slab with 4m span and 2.5cm concrete cover thickness

4 CONCLUSIONS

The analysis presented in this paper show that concrete cover thickness and reinforcement ratio have positive effect on increasing the fire resistance of RC slabs. By increasing the slab thickness the fire resistance of the RC slabs is increased too.

The simplified calculation method (EN 1992-1-2, 2004), presented in this paper, is capable for defining the fire resistance of one-way simply supported reinforced concrete slab, exposed to fire from the bottom side. This method is simple for implementation, useful for single members and provides satisfactory results.

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UTICAJ POŽARNOG SCENARIJA NA POŽARNU OTPORNOST RAMOVSKIH KONSTRUKCIJA

Rezime:

S-14

Razumeti bitne karakteristike i ponašanje armiranobetonskih ramovskih konstrukcija u požaru, ovisno od požarnog scenarija, je vrlo važno za pravilno projektovanje istih. Ovaj rad prikazuje uticaj četiri različitih požarnih scenarija na požarnu otpornost trobrodnog i dvospratnog armiranobetonskog rama. Rezultati numeričke analize: temperatura u presecima pojedinih elemenata konstrukcije, požarna otpornost konstrukcije, momente savijanja i deformacija konstrukcije, dobijene su kroz termičku i mehaničku analizu primenom kompjuterskog programa SAFIR.

Ključne reči: požarna otpornost, požarno scenario, temperatura, deformacija

FIRE SCENARIO INFLUENCE ON FIRE RESISTANCE OF RC FRAME

Summary:

Understanding the performance and the response of the reinforced concrete frame structures in fires, depending on the fire scenario, is important for structural fire design. Four different fire scenarios of standard fire exposure in a three bay two storey RC frame are analyzed and the results for the: fire resistance, bending moments and deformations are presented and discussed. The temperature distribution within the structural elements and the structural behaviour due to thermo-mechanical loading were calculated in the SAFIR2014 computer program. Worst fire scenario for the frame is when the fire is assumed in the whole second floor.

Key words: fire resistance, fire scenario, temperature, deformation

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1 INTRODUCTION

The aim of the research presented in this paper is to determine the fire scenario influence on the fire resistance of a particular reinforced concrete frame.

Fires can occur in any location of the building and therefore choosing the worst fire scenario is very challenging. Fire scenarios include different locations of fire to create worst but still realistic fire conditions for the structure. In this paper four different fire scenarios were considered.

After determining the fire scenarios, the next step is to determine the design fire. According to the EN1991-1-2, for compartment fire in ordinary buildings with ordinary thermal properties of the compartment boundaries, load density and ventilation conditions, the design fires are given with the standard fire curve and the parametric fire curve. In this case the heating regime is defined with the standard temperature-time curve ISO-834 ($T=20+345\log_{10}(8t+1)$).

2 DESCRIPTION OF THE STRUCTURE

The analyzed three bay two storey RC frame, with columns fully fixed at the bottom is shown in *Figure 1*. The overall height of the frame is 6 m (storey height is 3 m) and the bay's length is 5 m. The structure is made of normal strength concrete with compressive strength $f_c=30$ MPa and reinforcing bars with a yield strength $f_y=400$ MPa. The cross-sections of all beams are 0.35×0.45 m² and the columns sections are 0.40×0.40 m². The percentage of column reinforcement is taken to be 1% and beams reinforcement are shown in *Table 1*. A uniformly distributed load of 50 kN/m² (including self weight) is applied on the beams ($q/q_u \approx 0.6$) and force of 12 kN is applied in the beam to column joints of the first floor.

Table 1 – Reinforcement in structural members

	Beam end reinforcement		Beam mid-span reinforcement		Column reinforcement
	Top rebars	Bottom rebars	Top rebars	Bottom rebars	
First floor beam	4 Φ 22	4 Φ 14	2 Φ 22	4 Φ 14	
Second floor beam	4 Φ 20	4 Φ 14	2 Φ 20	4 Φ 14	
Columns					8 Φ 19

3 NUMERICAL EXAMPLES

The calculation of the structure exposed to fire consists of two steps: calculation of temperature within the structural members (thermal analysis) and calculation of mechanical behaviour of the structure exposed to fire (structural analysis).

All analyses were carried out using the finite element SAFIR program developed at the University of Liège, Belgium. For modeling the structure BEEM elements were used.

Displacement control was made regarding maximal vertical displacements of the beam's mid spans to be lower than $L/30=16.7$ cm and axial dilatation of the columns to be lower than $0.01H$.

3.1 FIRE SCENARIOS

The numerical analyses consider four different fire scenarios for the reinforced concrete frame treated in this paper. All fire scenarios are shown in *Figure 1*. Beams are assumed to have three sides fire exposure (bottom and side faces) and for the columns it depends on the fire scenario (one or two sides).

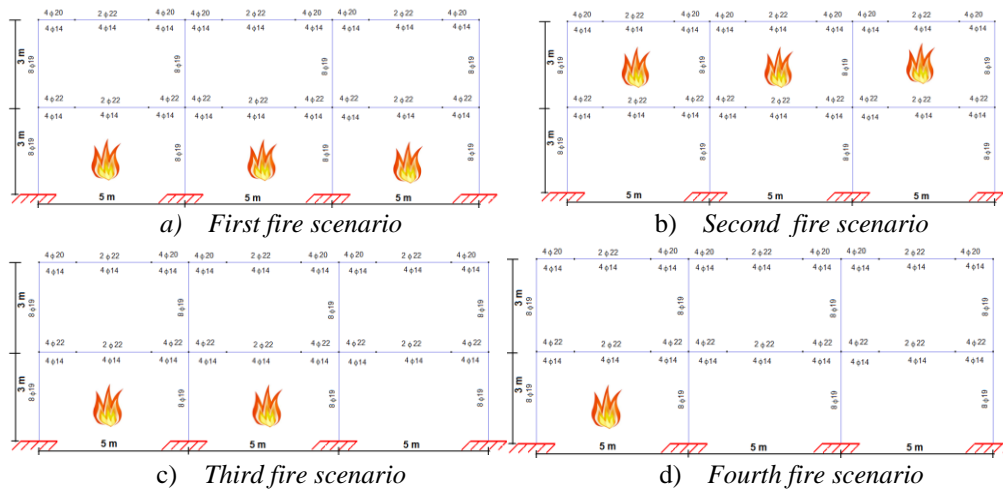


Figure 1 Fire scenarios assumed in the analyses

3.1.1 First fire scenario

In the first fire scenario the whole first floor is considered as a single fire compartment and the fire resistance of the frame is $t=11861$ sec=3.29 hours. Diagrams of bending moments (in Nm), deformation (displacements in meters) and temperature development in the cross-section of some of the structural elements are shown in the next figures.

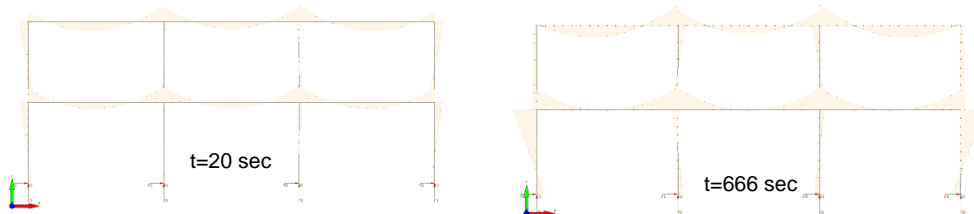


Figure 2 Bending moment diagrams at different moments of the analysis

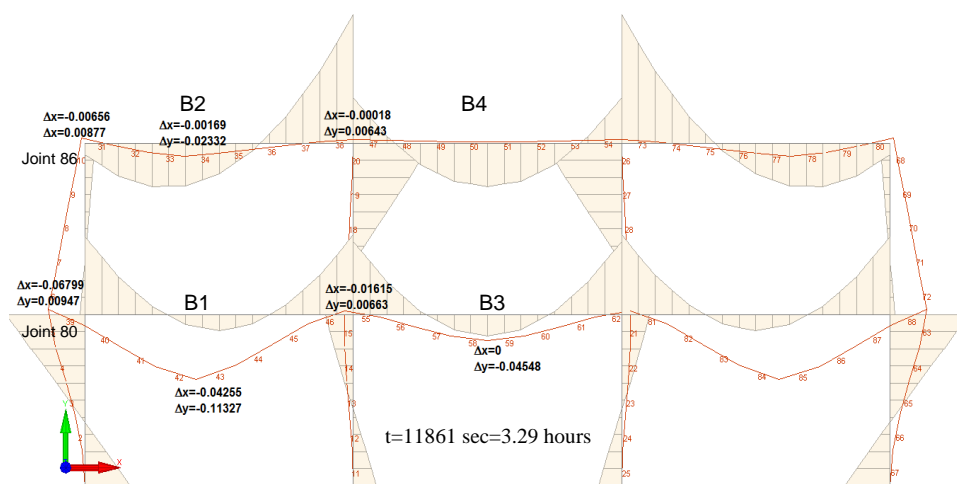


Figure 3 Bending moment diagram and failure mode at time $t=3.29$ hours

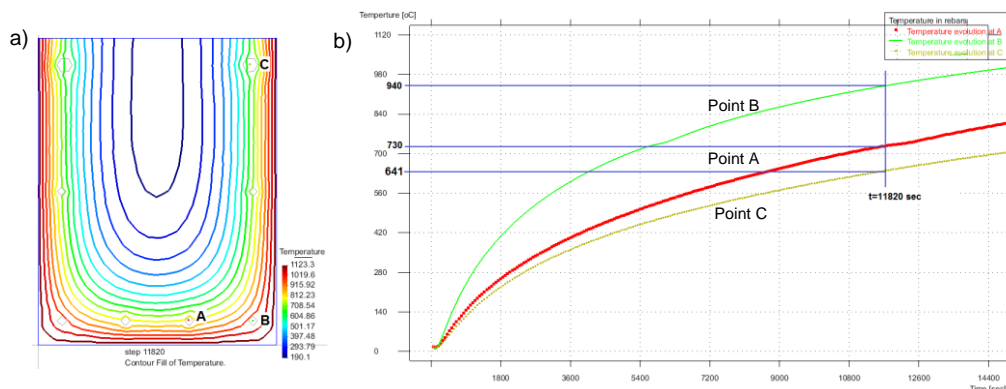


Figure 4 Temperature distribution in the cross-section of mid-span beam B1 a) Isotherms in moment $t=11820$ sec b) Line diagram of temperature evolution in rebars A, B and C

Because of the symmetry in the geometry and the loading of the structure, the behaviour of the outer columns/beams is the same.

As shown in Figure 2, bending moment diagrams are changed after application of the fire exposure. Figure 5 shows that in the first 20 minutes the positive bending moment in the mid-span of the second floor beam B2 tends to increase (the program SAFIR has opposite convention, positive bending moments have negative sign and opposite), negative left end moment tends to become positive and the right end moment tends to enlarge. At time $t=1346$ sec the redistributions have maximal values and starts the process in opposite direction, the positive moment at the mid-span decreases. After 50 minutes of the analysis bending moments become almost constant. The reason for that is the stabilization of the temperatures in the cross sections of the elements. In the first 666 seconds of the analysis, the negative bending moments at the ends of the first floor fire exposed beam B1 increase, while the positive bending

moment at the mid-span tends to become negative (*Figure 5*). After that time the bending moments are slowly reversing, except the left end negative moment which, after 1.88 hours, again starts to increase.

Temperatures in the rebars at the moment of failure of the structure, shown in *Figure 4*, are far beyond the critical (400 °C) temperature, which is causing reduction of steel yield strength.

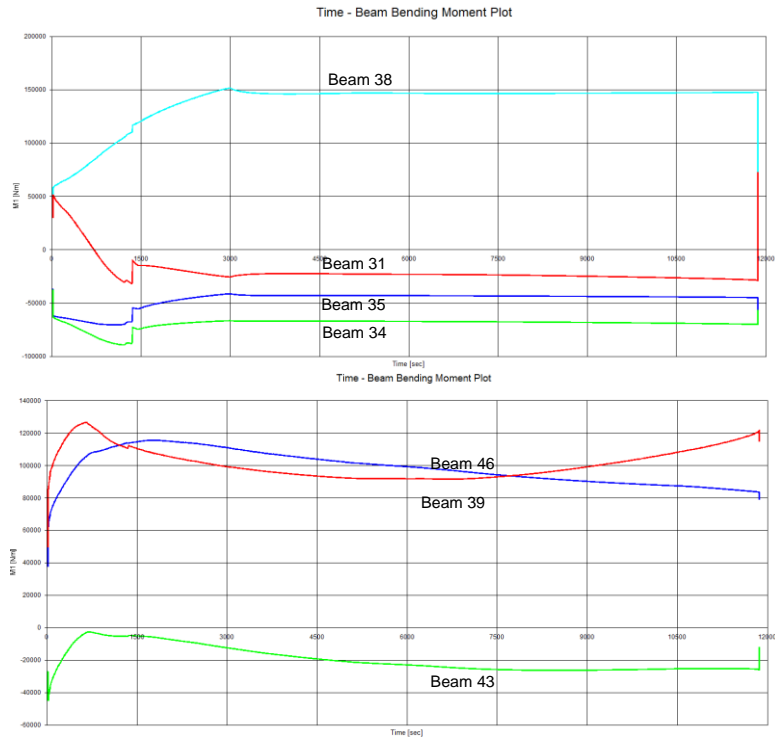


Figure 5 Time-bending moment diagrams for some of the second and first floor beam elements of beams B2 and B1, in case of first fire scenario

3.1.2 Second fire scenario

In the second fire scenario the whole second floor is considered as a single fire compartment and the fire resistance of the frame is $t=10389 \text{ sec}=2.89 \text{ hours}$.

From the beginning of fire exposure till $t=50 \text{ minutes}$, the positive mid-span bending moment of the first span first floor beam B1 is increasing because the bending moment caused by the q load is summed up with the bending moment caused by fire loading. The upper (heated) fibers of the beam are in compression and the bottom fibers are in tension. At time $t=785 \text{ sec}$ a redistribution of bending moments occurs and after 50 minutes of fire exposure mid-span bending moments become constant. Negative bending moments at the left end of the beam B1 tends to increase in time and on the right end of this beam bending moment tends to decrease. Opposite to that, in the first minutes of the fire exposure the negative bending moments in the first span second floor beam B2 are increased and the positive bending moment

(in the mid-span of the beam) is decreased, because of the compression in the lower fibers of the beam, caused by restrained thermal dilatations.

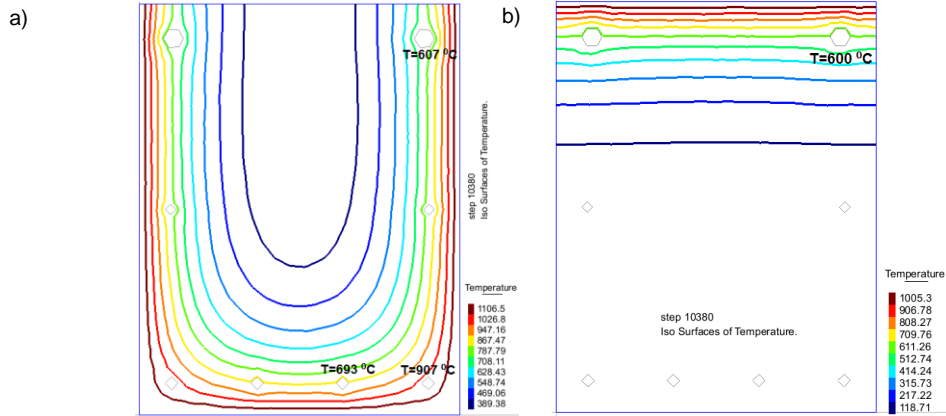


Figure 6. Isotherms in cross sections at moment=10380 sec a) mid-span of beam B2, b) mid-span of beam B1

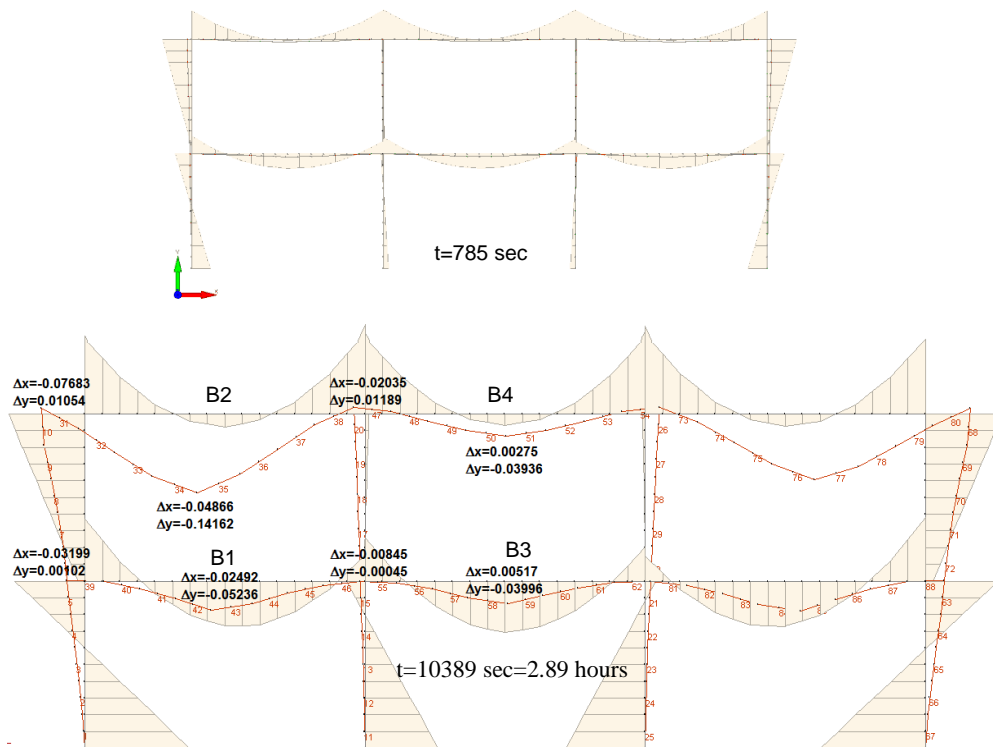


Figure 7 Bending moment diagrams at different time moments and failure mode

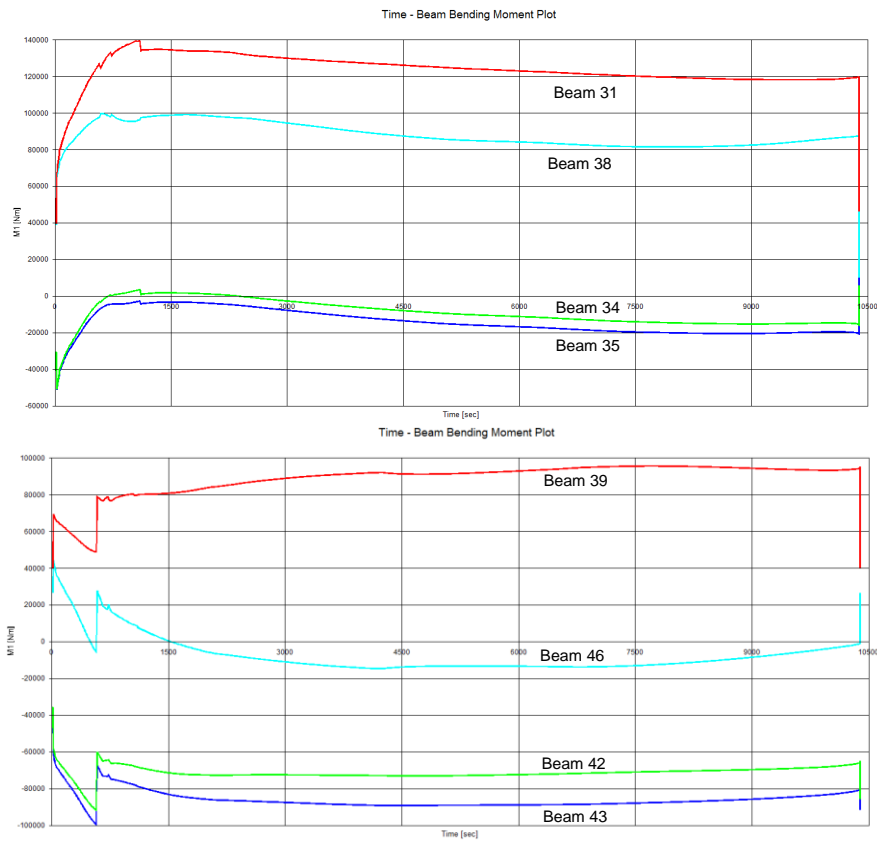


Figure 8 Time-bending moment diagrams for beam elements of beam B2 and beam B1

Steel temperatures in cross-section of mid-span beam B2 are similar to those of the heated mid-span beam B1 in first fire scenario, but in this case they are slightly lower because the moment of failure is 25 minutes less (Figure 6a). However, this doesn't make any difference because the steel started to yield much earlier.

3.1.3 Third and fourth fire scenario

In the third fire scenario, the fire exposure is only in two compartments of the ground floor, the fire resistance is $t=11869 \text{ sec}=3.30 \text{ hours}$ (Figure 9). The time-bending moment diagrams for beams B1 and B3 are similar to the corresponding in the first fire scenario (Figure 10). The right column of the middle bay is heated only from one side and the third bay is not affected by the fire. Because the left column of the middle bay is heated from two sides joint 80 has the largest horizontal displacement and it is larger than in the first and fourth fire scenario (Figure 11). This is because in the first fire scenario there is a symmetry and balance of the displacements of the outer columns, and in the fourth fire scenario the middle and outer cold bays are causing lateral restraints.

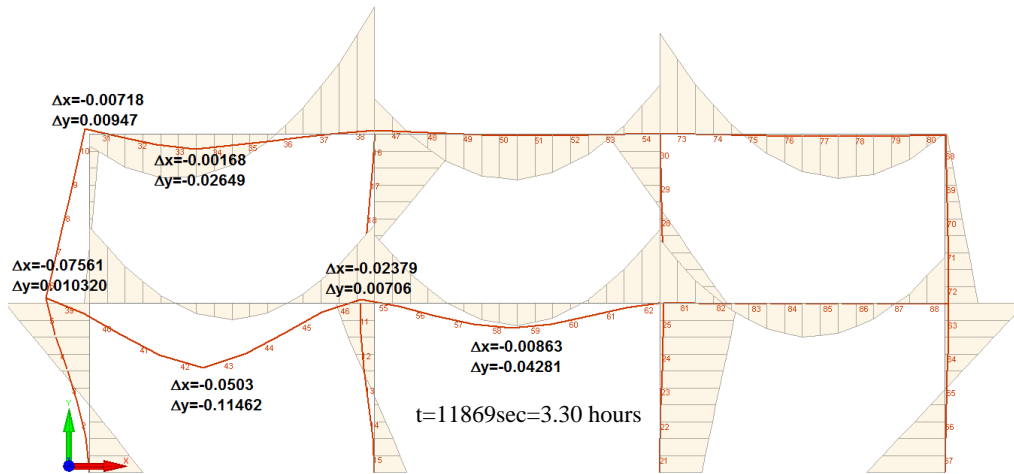


Figure 9 Bending moment diagram and failure mode at moment $t=11869$ sec in the case of the third fire scenario

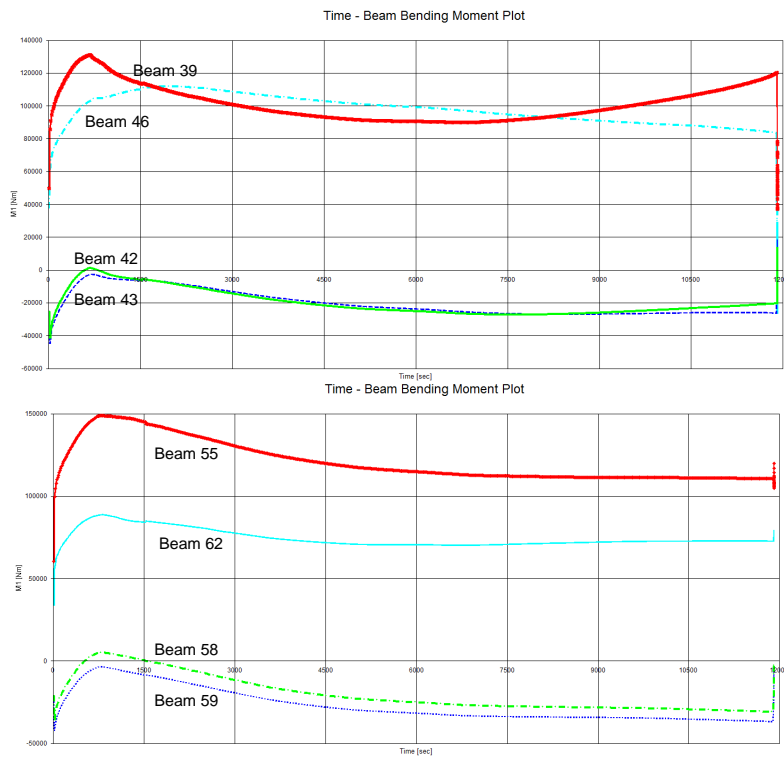


Figure 10 Time-bending moment diagrams for beam elements of beam B1 and beam B3 in the case of the third fire scenario

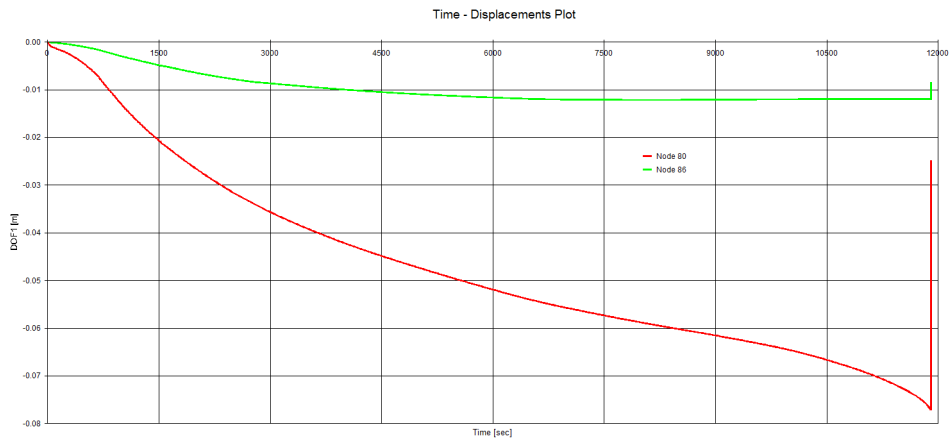


Figure 11 Time-displacement diagrams for joint 80 and 86

In the fourth fire scenario the fire involves only the left compartment of the ground floor and the fire resistance is $t=11820 \text{ sec}=3.28 \text{ hours}$ (Figure 12). This fire scenario doesn't influence the safety and stability of the whole structure, but only affects the fire compartment and the compartment above. The reason for that are the not restrained thermal dilatations in horizontal direction on the left side.

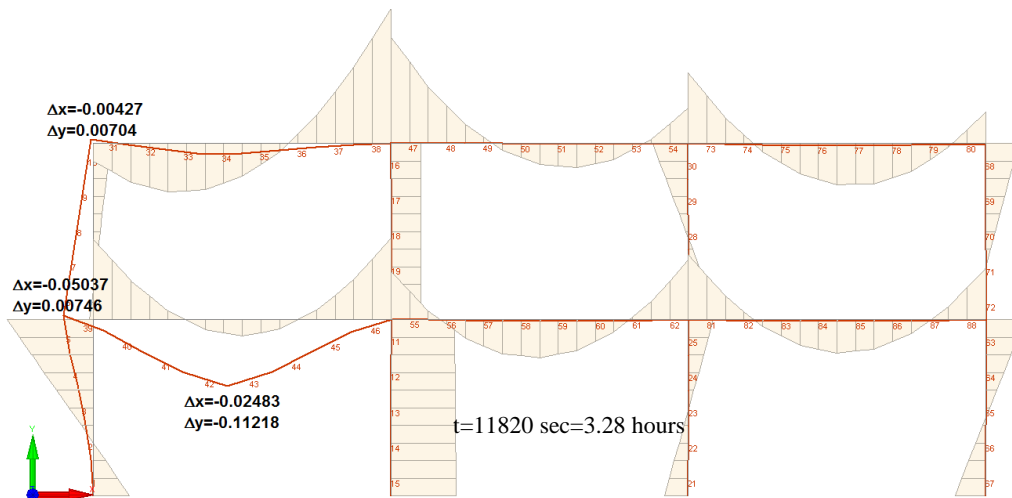


Figure 12 Bending moment diagram and failure mode at moment $t=11826 \text{ sec}$ in the case of the fourth fire scenario

4 CONCLUSION

In order to determine the fire scenario influence on the fire resistance of a reinforced concrete frame four fire scenarios were considered: two fire scenarios assuming each floor as a single fire compartment and two fire scenarios assuming compartmentation of both spans and floors.

From the numerical analysis, the following remarks could be made:

- The fire resistance of the frame in the first, third and the fourth fire scenario is almost the same (around 3.29 hours).
- The second fire scenario is the “worst” from all four analyzed fire scenarios. The fire resistance of the RC frame is $t=2.89$ hours. The smallest fire resistance is a consequence of the vertical displacement of both second floor and first floor beams. The vertical displacement in the mid-span of the beam B2 in the left second floor compartment is greatest than any other vertical displacement in the other fire scenarios (14 cm).
- The horizontal displacement of joint 80 (end of first floor left outer column) has the largest value in the third fire scenario, it is smaller in the case of the first fire scenario and it has the smallest value in the case of the fourth fire scenario. Because of the cold, stiff and strong enough central and outer right bays, causing lateral restrains and the left central column is heated only from one side, the horizontal displacement of joint 80 in the fourth fire scenario is smallest than in the other fire scenarios.
- The deflection of the first floor beam B1 has almost the same value in the first, the third and the fourth fire case scenarios (11.4 cm).
- All beams exposed to fire from the bottom side tend to move up during the first minutes of fire exposure, till the moment when the temperature difference between the heated and the cold part of the cross section will be stabilized. Because of the pressure in the lower heated parts of the beams cross sections, negative bending moments at the beam’s ends decrease and the positive mid-span bending moments tend to become negative.
- All beams exposed to fire only from the upper side tend to enlarge the positive mid-span bending moment and decrease the negative end beam bending moments because of the compressed upper heated side. These beams develop plastic hinges in the mid-span much earlier than the beams exposed to fire from the bottom side.

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ANALIZA PONAŠANJA AB STUBA OJAČANOG OMOTAČEM OD BETONA I ČELIČNIM UGAONICIMA

Rezime:

S-15

Predmet ovog rada je eksperimentalna analiza centrično pritisnutih armirano-betonskih stubova kvadratnog poprečnog preseka istovremeno ojačanih čeličnim ugaonicima, veznim limovima i oblogom od betona. Rastojanje između horizontalnih prečki varirano je za dva slučaja. Izbor dimenzija stuba je izvršen je uz simuliranje "stroge geometrijske sličnosti" sa realnom konstrukcijom u odnosu 1:3,3. Predmet rada razmatra konkretan problem konstrukcija koji se može javiti u građevinskoj praksi pri saniranju, odnosno ojačavanju betonskih stubova opterećenih aksijalnom silom pritiska.

Ključne reči: AB stub, ojačanje, sprezanje, eksperiment

ANALYSIS OF BEHAVIOR OF THE RC COLUMN STRENGTHENED WITH CONCRETE JACKET AND STEEL ANGLES

Summary:

The topic of this paper is experimental analysis of axially compressed reinforced concrete columns (squared section) simultaneously strengthened with steel angles, strips and concrete jacket. Distance between horizontal steel strips was varied in two cases. Columns dimensions were chosen to simulate pure geometric similarities with real construction in ratio 1:3,3. This paper deals with specific structural problems that can occur in civil engineering practice when retrofitting or strengthening of axially compressed concrete columns is needed.

Key words: RC column, strengthening, composite action, experiment.

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1 UVOD

Predmet rada je eksperimentalna analiza ponašanja modela aksijalno pritisnutih stubova spregnutog poprečnog preseka formiranog od čelika i betona. Analizirana su eksploataciona stanja i granično opterećenje. Spregnuti preseki formirani su ojačavanjem armiranobetonskih štapova čelikom i betonom. AB stubovi kvadratnog poprečnog preseka ojačavani su pomoću četiri čelična L profila, koji su postavljeni na uglovima betonskog stuba. Ugaonici su međusobno mestimično spojeni horizontalnim prečkama od čeličnog lima, čime je formiran "kavez" oko osnovnog AB stuba. Rastojanje između horizontalnih prečki varirano je za dva slučaja. Kako bi se ostvarilo efikasno sprezanje betona i čelika, prostor između kaveza i stuba popunjen je visoko kvalitetnim sitnozrnim betonom sa ekspanzivnim svojstvom. Na taj način izvršeno je ojačanje stuba za povećanje aksijalnog opterećenja.

Spregnuti stubovi od čeličnih cevi ispunjenih betonom predstavljaju jedan od prvih tipova spregnutih konstrukcija [8]. Omotač, odnosno čelična cev omogućava da se usled utezanja betona formira takozvani obručni mehanizam čime se znatno povećava efekat sprezanja i postiže veća nosivost. Formiranje efekta obruča izaziva pojavu složenih naponskih stanja u čeliku i u betonskom jezgru [9], [2] i [3]. Slična naponska stanja u betonu mogu se izazvati i kada se od čeličnih L profila i veznih limova formira kavez oko AB stuba, što je prikazano u [5], [6] i [7].

Kod svih pomenutih načina za povećanje pritisne čvrstoće betona usled troosnog naponskog stanja, aktiviranje dodatnih delova poprečnog preseka dešava se pri višim nivoima opterećenja, tako da se povećanje nosivosti postojećeg AB stuba može ostvariti samo za dodatno stalno i korisno – povremeno opterećenje.

Putem eksperimentalnog ispitivanja pod opterećenjem sve do loma ovih nosača (stubova) došlo se do procene uspešnosti ojačanja AB stubova uz istovremeni uvid uticaja variranih parametara pri ovakvim vrstama sanacija, odnosno ojačavanjima. U radu se razmatra konkretan problem konstrukcija koji se može javiti u građevinskoj praksi pri saniranju, odnosno ojačavanju betonskih konstrukcija opterećenih aksijalnom silom pritiska. Postupak ojačavanja AB stubova čeličnim L profilima uspešno je primenjen pri sanaciji objekta u Nemanjinoj 7 u Beogradu što je prikazano u radu [4].

Prikazani rezultati istraživanja predstavljaju izvod iz rezultata eksperimentalnih ispitivanja sprovedenih u sklopu doktorske disertaciji prvog autora [1] pod mentorstvom drugog autora ovog rada.

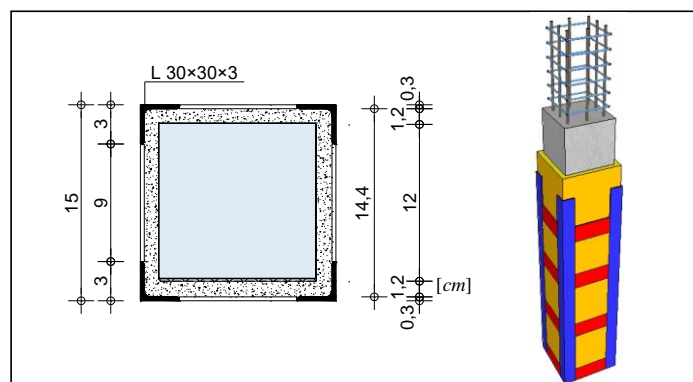
2 EKSPERIMENTALNO ISTRAŽIVANJE

Eksperimentalna analiza problema izvršena je na modelima AB stuba kvadratnog poprečnog preseka koji je zatim ojačavan čeličnim kavezom i oblogom od betona. Ispitivanje se obavljalo za centrično opterećen, obostrano zglobno oslonjen štap konstantnog poprečnog preseka. Modeli su podeljeni u nekoliko grupa. Prvu grupu od pet modela činili su kontrolni uzorci AB stubova bez ojačanja (Oznaka grupe *S*). Sledeće dve grupe od po tri modela činili su modeli AB stubova ojačani čeličnom ugaonicima, veznim limovima i oblogom od betona visokog kvaliteta (Oznaka grupe *L0* i *L1*).

Kontrolni uzorci AB stubova, tj. stubovi bez ojačanja, kvadratnog su poprečnog preseka dimenzija 12/12cm, visine 90cm. Izrađeni su od betona klase čvrstoće *C35/45*, glavne armature

8Ø6 i uzengija UØ4/6, odnosno UØ4/3, u zoni unosa sile. Kvalitet čelika upotrebljenog za izradu armature odgovara hladno vučenoj žici $f_y/f_u=500/560MPa$.

AB stubovi koji se ojačavaju su po svojim karakteristikama identični kao i stubovi osnovne grupe. Ojačavanje se vršilo na taj način što se betonski stub postavio unutar formiranog čeličnog kaveza, dok je međuprostor između čelične obloge i AB stuba popunjavao betonom čime je formirana betonska obloga. Za oblogu je upotrebljen SikaGrout® 212 koji svojim mehaničkim karakteristikama odgovara betonu klase čvrstoće C60/75. Čelični višedelni štap složenog poprečnog preseka formiran je od četiri valjana profila L30×30×3 na međusobnom razmaku 90mm. Profili su povezani spojnim limovima $\neq 90\times 40\times 3$. Vertikalni položaj horizontalnih prečki variran je za dva slučaja. Kod prva tri modela stuba ove grupe upotrebljeno je 9 spojnih limova po jednoj stranici stuba na međusobnom osnom razmaku $a=107.5mm$, dok su druga tri modela izvedena sa 7 spojnih limova po stranici stuba na razmaku $a=150mm$. Veza spojnih limova i pojasnih L profila ostvarena je zavarivanjem sučeonim 1/2V šavom. Poprečni presek ojačanog stuba, kao i prikaz elemenata stuba spregnutog preseka dat je na slici 1.



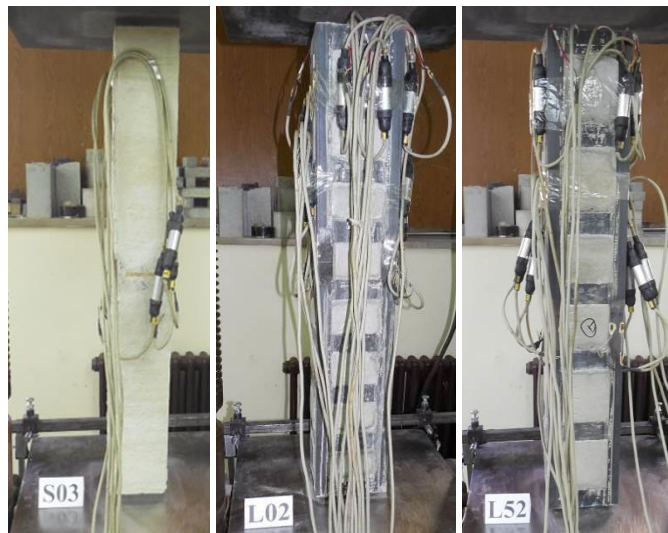
Slika 1 - Poprečni presek ojačanog stuba sa prikazom elemenata stuba spregnutog preseka

Model stuba svojim dimenzijama odgovara kratkom štapu (vitkost $\lambda < 25$), tako da provera stabilnosti usled efekta izvijanja nije vršena. Izborom dimenzija eksperimentalnog modela simulirana je "stroga sličnost" sa realnim konstrukcijama u odnosu 1:3,3 [1] i [2]. Modeliran je stub realne AB konstrukcije, bez uticaja vitkosti, dimenzija 40/40cm, visine 300cm i armiran šipkama 8Ø20 i uzengijama UØ10/20 odnosno uzengijama UØ10/10 u zoni unosa sile. Stub je izrađen od betona klase čvrstoće C35/45. Metoda ojačavanja čeličnom kavezom modelirana je za stub realne konstrukcije uz oblogu od betona debljine 5cm po obimu i sa usvojenim dimenzijama profila L100×100×10 i veznim limovima $\neq 300\times 130\times 10$. Prilikom ovoga istraživanja primenjena "stroga geometrijska sličnost" tako da je analiza eksperimentalnih podataka vršena kao da se radi o realnim stubovima, a ne njihovim modelima.

3 REZULTATI EKSPERIMENTALNOG ISTRAŽIVANJA

Eksperimentalno istraživanje obuhvatalo je registrovanje dve vrste deformacija, i to merenje lokalnih deformacija (specifičnih dilatacija) i merenje globalnih deformacija (skraćanja) stubova. Merna mesta za registrovanje specifičnih dilatacija mernim trakama

locirana su na sredini dužine štapa, a postavljena su simetrično u odnosu na uzdužnu osu stuba kako bi se postiglo takozvano opasivanje preseka. Kod ojačanih stubova registrovana je i analizirana: promena stanja napona i deformacija svih delova spregnutog preseka, granična nosivost, oblik globalne deformacije stuba pri lomu, angažovanje pojedinih delova spregnutog preseka, kao i nastanak prslina u betonu obloge. Kod modela AB stubova, pored granične sile i deformacija, registrovani su još i veličina sile pri kojoj dolazi do pojave prslina, njihov nastanak i razvoj.



Slika 2 - Modeli stubova neposredno pre ispitivanja

3.1 STUBOVI BEZ OJAČANJA

Stubovi kontrolne grupe pokazali su vrlo slično ponašanje. Analizom dijagrama sa slike 3b može se uočiti da je ponašanje modela stuba S02 gotovo linearno kroz sve faze nanošenja opterećenja, a sam dijagram je oblika plitke kvadratne parabole. Prve podužne prsline registrovane su na krajevima stuba pri nivou opterećenja 90% od sile loma. Do sloma uzorka je dolazilo usled pucanja i mrvljenja betona na mestu unosa sile, što se vidi na slici 4a. Prsline su se na svim uzorcima javljale na oba kraja, s tim da je pri lomu dolazilo do izraženijeg razaranja modela samo na jednom kraju. Granične sile stubova kontrolne grupe prikazane su u tabeli 1.

Tabela 1 - Granične sile stubova bez ojačanja

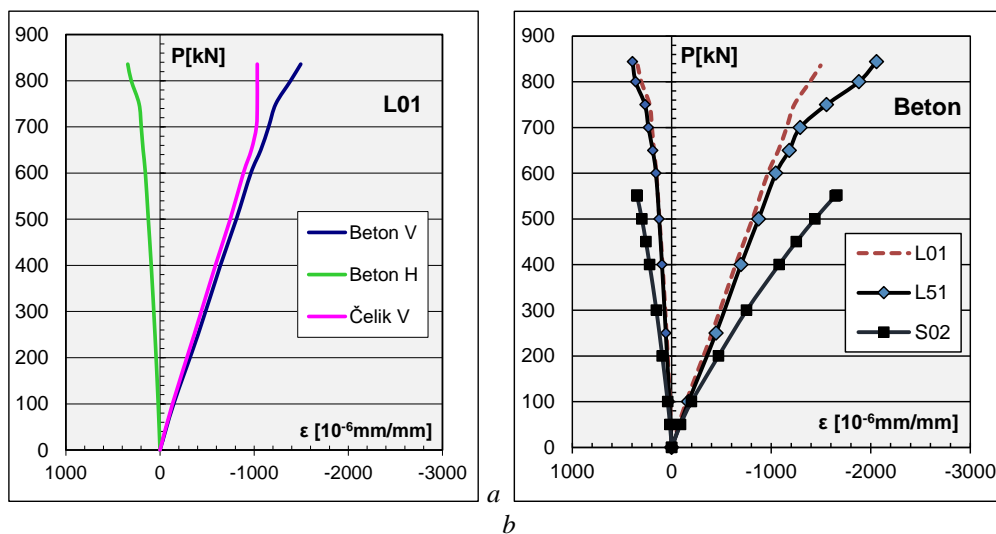
	S01	S02	S03	S04	S05	$P_{u, sr}$	S_n [kN]	C_v [%]
P_u [kN]	616	552	552	624	616	592.0	36.7	6.2

Prikazana je još i prosečna vrednost izmerenih sila, standardna devijacija i koeficijent varijacije rezultata. Nešto veće odstupanje vrednosti granična sila, uz standardnu devijaciju rezultata od $S_n=36.7$ kN i koeficijent disperzije od $C_v=6.2\%$, verovatno je posledica nešto nižih mehaničkih karakteristika betona od kojeg su izrađeni modeli S02 i S03.

3.2 STUBOVI SA OJAČANJEM

Ispitivanje šest ojačanih modela vršeno je prema sličnim fazama programa ispitivanja radi komparacije rezultata sa ispitivanjem kontrolnih modela. Na ovaj način bila je moguća direktna procena uspešnosti ojačanja AB stubova čeličnim kavezom i oblogom od betona, a takođe je bilo omogućeno i međusobno poređenje ovih modela, kako bi se uvideli uticaji različitog razmaka veznih limova. Opterećenje na modele bilo je naneto posredno preko čeličnih pločica (gornje i donje), tako da je bio opterećivan samo osnovni AB stub (jezgro). Na ovaj način se simuliraju uslovi iz realne konstrukcije, kod koje se unos opterećenja vrši samo preko ojačanog betonskog stuba

Na slici 3a prikazan je dijagram promene aksijalne sile pritiska u odnosu na uzdužne i poprečne dilatacije za merene pravce modela L01. Oznake "Beton V", "Beton H" i "Čelik V" na dijagramima odnose se na pravac glavnih dilatacija vertikalnom stubu. Iz prikazanog dijagrama uočava se da je u ponašanje svih delova spregnutog preseka linearno sve do neposredno pred lom. Gotovo potpuno sprezanje poprečnog preseka očuvano je sve do sile od oko 700kN. Pri toj sili dostignute su granične dilatacije zatezanja u betonskoj oblozi u kojoj dolazi do razvoja prslina usled čega je narušeno sadejstvo delova preseka.



Slika 3 - Dijagrami promene sile i specifičnih dilatacija modela stubova

Na slici 3b uporedno su prikazani dijagrami podužnih i poprečnih specifičnih dilatacija na betonu osnovnog AB stuba i za karakteristične modele obe grupe ojačane čeličnim kavezom i oblogom od betona. Ojačani stubovi imali su veću početnu krutost, tako da su, za iste vrednosti graničnih deformacija, postigli veću nosivost. Modeli stubova sa različitim rasporedom spojnih limova (L01 i L51) pokazali su gotovo identično ponašanje sve do 80% nosivosti uzorka tj. pri sili $P=700kN$, kada se narušava sprezanje poprečnog preseka. Razlika između ojačanih modela je relativno mala tako da se može smatrati da obe grupe imaju gotovo identično ponašanje uz izraženo linearan naponsko-deformacijski odgovor.

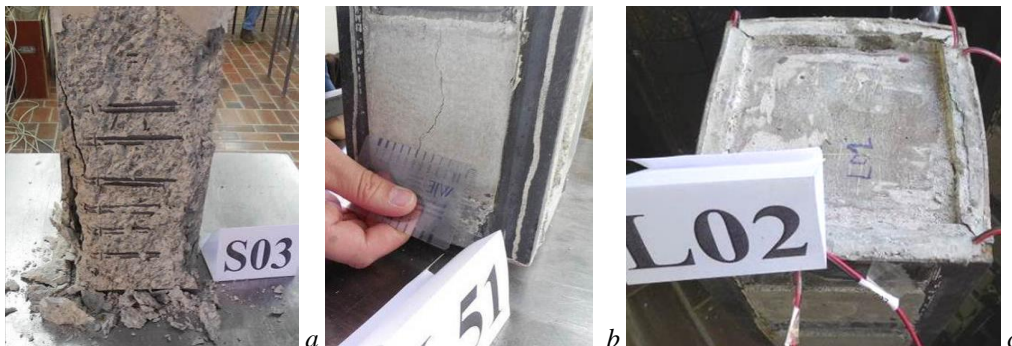
Granične sile ojačanih stubova registrovane prilikom ispitivanja prikazane su u tabeli 2. Kao i kod prethodne grupe prikazane su: prosečne vrednosti rezultata, standardna devijacija, koeficijent varijacije rezultata, kao i koeficijenta povećanja nosivosti ojačanog stuba (k_s) u poređenju sa stubom kontrolne grupe.

Tabela 2 - Granične sile ojačanih stubova

	L01	L02	L03	$P_{u,sr}$	S_n [kN]	C_v [%]	k_s [-]
P_u [kN]	836	876	856	856.0	20.0	2.34	1.45
	L51	L52	L53	$P_{u,sr}$	S_n [kN]	C_v [%]	k_s [-]
P_u [kN]	844	888	832	854.7	29.5	3.4	1.44

Analizom vrednosti uočava se da armiranobetonski stubovi ojačani sprežanjem sa čeličnim L profilima i oblogom od SikaGrout-a imaju za oko 45% veću nosivost u odnosu na kontrolnu grupu. Ukoliko se uporede vrednosti sila loma ojačanih stubova, može se uočiti da je razlika relativno mala, što navodi na zaključak da povećanje razmaka veznih limova sa $e=10cm$ na $e=15cm$ nije značajnije uticalo na smanjenje nosivosti stuba.

Na slici 8 prikazani su krajevi modela stubova nakon završenog ispitivanja. Prikazan je uzorak kontrolne grupe, kao i uzorak sa ojačanjem. Uočavaju se karakteristični načini loma modela. Kod stubova kontrolne grupe dolazilo je do podužnih prslina i pukotina koje su nastajale u zonama pri krajevima modela, kao što se može videti na uzorku S03 na slici 4a. Nakon pucanja i odvajanja delova betona došlo je do redukcije poprečnog preseka usled čega je vrlo brzo došlo i do potpunog gubitka nosivosti modela stuba.



Slika 4 - Karakteristični oblici pri lomu modela stubova

Prilikom ispitivanja svih šest modela stubova spregnutog poprečnog preseka pokazalo je gotovo identično ponašanje. Do loma, odnosno gubitka nosivosti, dolazilo je usled razvoja prslina u ispuni (Slika 4b), i usled kombinacije drobljenja betona i poprečnog razvlačenja veznih limova na mestu unosa sile pri krajevima stuba. Neposredno pre dostizanja granične nosivosti došlo je do delimičnog utiskivanja čeličnih pločica za prenos opterećenja u stub, što se jasno vidi na slici 4c.

4 ZAKLJUČCI

Rezultati eksperimenta u značajnoj meri zavise od pretpostavki modela, tako da se i izvedeni zaključci odnose na tačno definisan okvir važenja modela. Prema podacima dobijenim nakon sprovedenog eksperimentalno–teorijskog modelskog istraživanja formirani su sledeći zaključci:

- AB stubovi kontrolne grupe imaju gotovo linearnu vezu između napona i deformacija sve do neposredno pred lom. Stubovi ojačani čeličnim ugaonicima i oblogom od betona iskazuju nešto kruće ponašanje i u stanju su da pretrpe više sile u odnosu na klasične AB stubove.

- Razlike između dilatacija AB stuba i čeličnog kaveza od ugaonika i veznih limova su relativno male, sve do pred lom uzorka, što ukazuje na očuvano potpuno sprezanje svih delova preseka.

- Analizom deformacijskih dijagrama uočeno je da celokupni poprečni presek kod spregnutih stubova učestvuje u prijemu i prenosu opterećenja. Transfer opterećenja između pojedinih delova spregnutog preseka ostvaren je trenjem na kontaktu dva različita materijala.

- Opterećenje na stub se između pojedinačnih delova spregnutog preseka deli proporcionalno ostvorenoj specifičnoj dilataciji i aksijalnoj krutosti.

- Graničnu nosivost stuba ojačanog čeličnim kavezom određuju dilatacije pri zatezanju u betonu ispune, kao i nosivost samog osnovnog AB stuba. Nastankom prslina u ispuni remeti se prenos opterećenja između delova spregnutog preseka

- Nosivost AB stubova ojačanih sprežanjem sa čeličnim profilima i oblogom od betona u posmatranom slučaju povećava se za oko 1.45 puta, dok uticaj razmaka veznih limova nije mogao biti procenjen u pravoj meri.

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UTICAJ ČVRSTOĆE BETONA PRI OJAČANJU AB GREDE CFRP ARMATUROM

Rezime:

Tokom životnog ciklusa veliki broj AB konstrukcija, bez razilke dali su izvedene iz betona normalne čvrstoće ili iz betona visoke čvrstoće, pokazuju nesodvetne performanse izraženih preko smanjenje nosivog kapaciteta i pokazuju potrebu za njihovo ojačanje. Ovaj rad prezentira numeričku analizu uticaja jakosti betona na performanse CFRP ojačanoj AB gredi. Analiza je sprovedena za dva karakterističnih slučaja: kada veza između betona i spoljašne CFRP armature je kontinuirana po celoj dolžini grede i kada veza je nepravilno izvedena formirajući diskontinualne zone između AB grede i CFRP armature.

Ključne reči: ojačanje, čvrstoća betona, CFRP

INFLUENCE OF THE COMPRESSIVE STRENGTH OF THE CONCRETE ON CFRP STRENGTHENED RC BEAM

Summary:

A large number of existing RC structures, whether they have been built of concrete with usual compressive strength or of concrete with high compressive properties, exhibit non adequate performance in terms of diminished bearing capacity and there is a need of post strengthening during their service life. This paper presents numerical analysis of the influence of the compressive strength of the concrete on the performance of the CFRP strengthened RC beam. Analysis is performed for two characteristic cases: when the bond between concrete and external reinforcement is continuous along the length of the beam and when the bond is improper executed creating discontinuity zone between RC and CFRP plate.

Key words: strengthening, concrete compressive strength, CFRP

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1 INTRODUCTION

As a result of change in the environmental conditions and the increasing of the loads, which were not taken into account in the design process, contemporary structures can significantly change their performance. This may lead to decreasing of bearing capacity or structural safety during service life of the construction. Thus a number of reinforced concrete structures exhibits non-adequate performance and need of post strengthening during their service life. In the recent years, increasing the load capacity and structural safety of the flexural loaded structural members is carried out by external bonding of additional FRP (fibre reinforced polymer) reinforcement.

In order to achieve successful external strengthening of the reinforced concrete structures by FRP strips it is required a thorough understanding of the effects that FRP reinforcement has on beam failure mode. Key role in the failure of the externally strengthened construction has the bond layer between reinforced surface and FRP strip. Experimental researches show that the most often type of failure of the strengthened construction, caused from the maximal shear stresses, is followed by peeling of the FRP strip initiated at the end of the plate, where concrete is uncracked. Local shear failure is driven by a biaxial tension state composed by the interfacial stresses and the normal tension induced on concrete by the flexure [1]. The failure modes due to loss of composite action are very often observed, but due to the brittleness of the failure, they also represent a significant threat to the structural safety of the strengthened structure and therefore deserve special attention. When de-bonding starts at the end of the plate, where concrete is uncracked, local shear failure is exhibited which is driven by a biaxial tension state composed by the interfacial stresses and the normal tension induced on concrete by the flexure [1]. With the help of the theory proposed by Taljsten [2], it can be concluded that for the cases of sufficiently thin strengthening plates, the influence of the peeling stresses on the principal stresses is minute and can thus be neglected. It is clear that the bond between the concrete substrate and the FRP plate plays a crucial role in the occurrence of failure mode with loss of composite action.

It is clear that the failure behaviour of a plated beam can be very strongly influenced by the integrity of the bond between the plate and the concrete. One of the problems that can be encountered during the strengthening of reinforced concrete structures in the practice is inadequate execution of the bonding process. This may lead to weakening of the bond layer in some positions along the length of the plate, and to creating discontinuities within the bond layer. In such case significant changes of the bearing capacity may be observed, but it is extremely difficult to evaluate the influence of these weak zones within the bond layer upon structural response of the strengthened beam.

In order to determine properly with adequate accuracy the bearing capacity of the reinforced concrete structure strengthened with externally added FRP reinforcement must be used a model, which can properly describe the stresses in the bond layer [3]. Very often are used beam models based on the concept of discretization of the cross section into fibbers layers, which in the same time take into account axial and flexural influence. These models could be combined with any model of beam element based on displacement method, as well as based on the force method. With appropriate modifications fibber model could be used for the analysis of the reinforced concrete elements strengthened with externally added FRP plates.

This paper presents numerical analysis of the influence of the compressive strength of the concrete on the performance of the CFRP strengthened beam. Analysis for the influence of the

compressive strength of the concrete on the performance of the CFRP strengthened beam is performed for two characteristic cases: when the bond between concrete and external reinforcement is continuous along the length of the beam and when the bond is improper executed creating discontinuity zone between reinforced concrete and CFRP plate. The bond between reinforced concrete beam and CFRP plate in this paper is modelled using a numerical displacement-based fiber model. Discontinuous bond zone is modelled by modification in the constitutive law for description of the bond between the reinforced concrete beam and CFRP plate. A simple approach to the weak zone in the bond description is proposed. Influence of the compressive strength of the concrete on the performance of the CFRP strengthened beam is analysed by using bond stress distribution and tensile plate force distribution along the externally strengthened RC beam. Numerical analysis carried out shows that compressive strength of the concrete has not huge influence on the performance of the externally strengthened RC beam whether the bond between concrete and CFRP plate is perfect or not.

2 NUMERICAL MODEL

The numerical model used for the analysis of the strengthened beam element is based on a fibre model [4]. Beam element based on two-node displacement has been used. It has two components: a two-node concrete beam and a strengthening plate. The nodal degrees of freedom of the concrete beam and of the strengthening plate are different to permit slip. The cross section of the reinforced concrete section beam is subdivided into layers as shown in Figure 1. Cubic transverse and linear axial displacement fields are assumed for the beam, and linear axial displacements for the strengthening plate. The distribution of the bond slip is quadratic. The element is implemented in the general purpose finite element program FEAP [5].

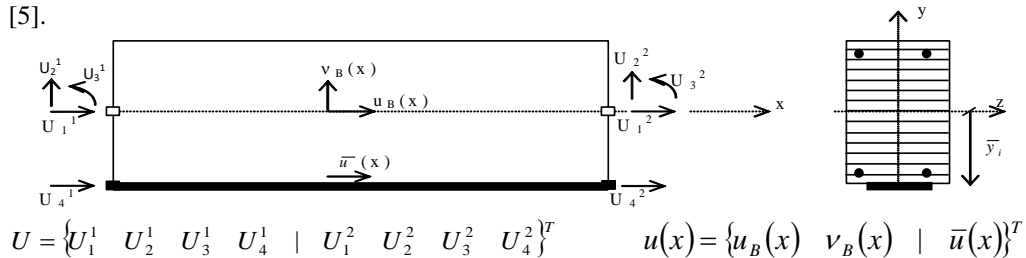


Figure 1 - Node and field displacement of the for reinforced concrete beam model with slip in plate

In the numerical model, the concrete is described by one-dimensional model proposed by Mohd-Yassina [4], while the behaviour of inner and external reinforcement for the strengthening are defined by Menegotto-Pinta model [4]. Models for the constitutive laws of the concrete and reinforcement are shown on Figure 2. Constitutive bond law between concrete and external FRP reinforcement is described by a linear relation between displacement and shear stresses in the bond layer up to the maximal bond strength. When this value of bond shear stresses is reached, slip occurs, which means that displacement is increasing while the corresponding shear stress is zero.

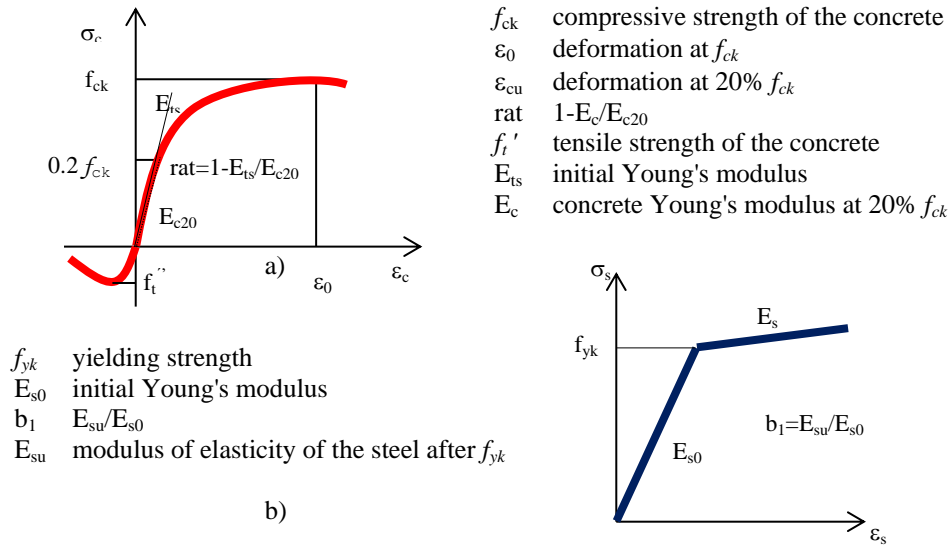


Figure 2 - Constitutive laws of the used materials: a) concrete; b) inner reinforcement

Values of the parameters for the concrete, inner, as well as external reinforcement used in the analysis are according to the Table 1.

Table 1 - Mechanical properties of the materials: a) concrete; b) reinforcement

a)

f_{ck} [MPa]	ϵ_0 [‰]	ϵ_{cu} [‰]	rat	f_t' [MPa]	E_{ts} [GPa]
-30	-0.0021	-0.01	0.1	1	100
-100	-0.0021	-0.01	0.1	1	100

b)

Reinforcement type	f_{yk} [MPa]	E_{s0} [GPa]	b_1 [%]
Inner reinforcement	460	210	0.1
External CFRP reinforcement	2400	150	0.1

In order to model weak zone in the bond layer, a modification was introduced in the original constitutive bond law. Maximal shear stress, τ_1 , remains unchanged, while displacement at slip is significantly increased. By this modification a much more flexible bond is achieved compared to the perfectly bond area. The perfect bond is described by values $u_{1,cont}=0.0013$ mm and $\tau_1=3.1$ MPa.

3 ANALYSED CASE

A reinforced concrete beam externally strengthened with CFRP strip is analysed. A 3200 mm beam element, presented in Figure 4, which has 2900 mm span and cross $b/h=800/120$ mm section is strengthened with CFRP strip with 100 mm width. The strengthened beam is subjected to four-point bending. Due to the symmetry of the case, only a half of the beam is being analyzed.

Analysis is performed for two characteristic cases: when the bond between concrete and external reinforcement is continuous along the length of the beam and when the bond is improper executed creating discontinuity zone between reinforced concrete and CFRP plate. A weak bond layer with length of 504 mm, which starts at 200 mm and ends at 704 mm from the supports, is analysed. In the two calculated cases are performed numerical analyses of the influence of two different compressive strength of the concrete on the performance of the CFRP strengthened beam and the analyses are performed for compressive strength of the concrete 30 MPa and 100 MPa.

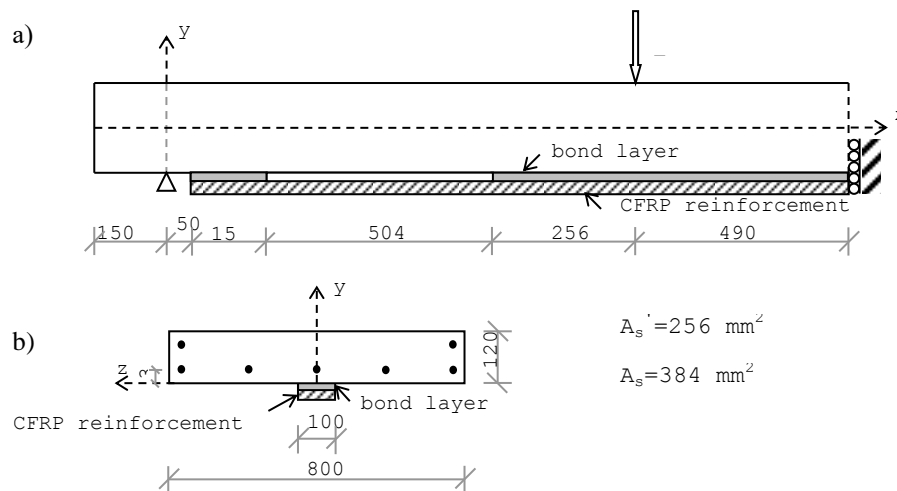


Figure 3 - Beam element strengthened with CFRP strip: a) geometry of half beam; b) cross section of the strengthened beam.

4 RESULTS AND DISCUSSION

The relationship between mid-span displacement and load for analyzed cases is presented in Figure 4. In all four analyzed cases the response can be divided into the following parts: a) no cracking occurs; b) concrete cracking occurs in the middle third of the beam, which results in the decrease of the beam stiffness; c) as loading increases, cracking spreads towards the beam supports, but bottom steel reinforcing bars are still elastic. The next stage, d), occurs when reinforcing steel yields, and the second significant drop in stiffness can be observed. At maximum load achieved, e), CFRP plate debonds. A large drop in beam strength is observed after this stage as the plate ceases to contribute to the beam stiffness.

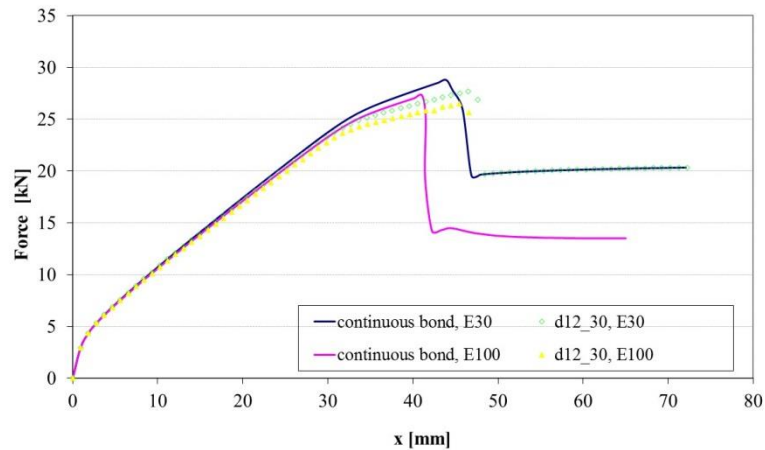


Figure 4 - Load-displacement relationship

Maximal loads of the reinforced concrete beams strengthened with CFRP strip with continuous bond along the whole length are 28.8 kN and 27.3 kN, respectively to the compressive strength of the concrete of 30 MPa and 100 MPa. Appropriate displacements are 43.9 mm and 41.1 mm. Maximal loads of the reinforced concrete beams strengthened with CFRP strip with discontinuous bond are 27.7 kN and 26.5 kN, respectively to the compressive strength of the concrete of 30 MPa and 100 MPa. Appropriate displacements are 46.5 mm and 45.4 mm. From the analysis could be concluded that the maximal load is similar regardless to the compressive strength of the concrete and bond between concrete and external reinforcement.

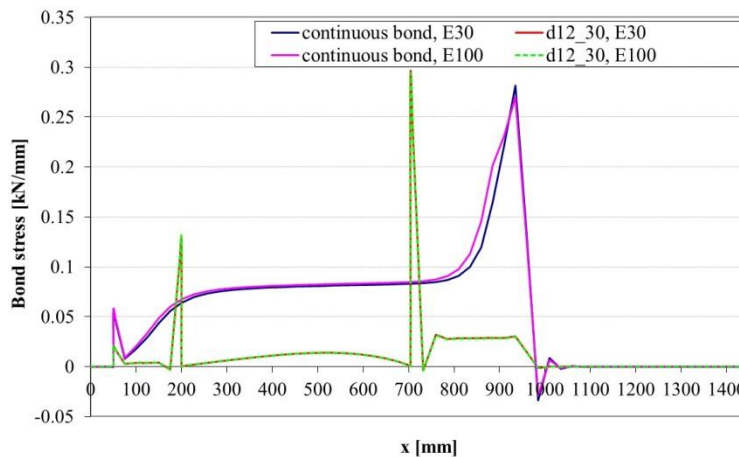


Figure 5 – Bond stress distribution between concrete and CFRP sheet

Bond stress distribution along the beam is shown on Figure 5. At the beginning of the CFRP plate can be observed a sudden increase in bond stress distribution, due to sudden change of the cross-section. Additional peaks in bond stress distribution can be observed at

both ends of the weak bond section and at the place where the beam is subjected on bending force. It can be observed that bond stress distribution mainly depends from the change of the cross-section and compressive strength of the concrete has not influence on the bond stress distribution between concrete and external reinforcement.

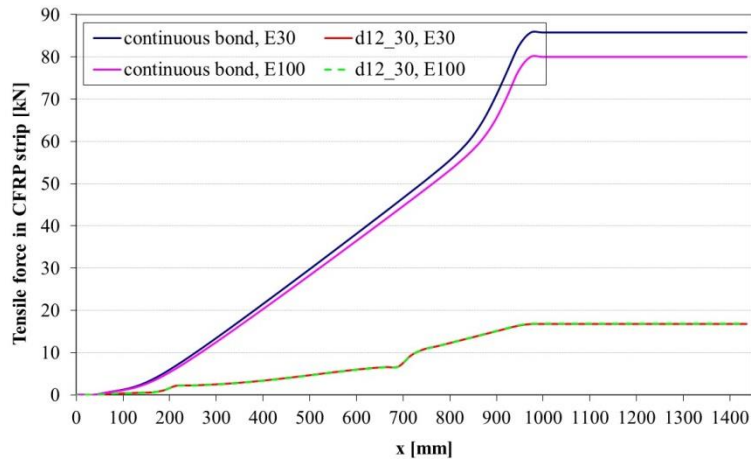


Figure 6 - Tensile force distribution in the CFRP plate along the beam

In Figure 6 is shown tensile force distribution in CFRP plate along the strengthened beam, which corresponds to the bond stress distribution depicted in Figure 5. It can be observed that in the section of the weak bond, tensile force rate is smaller and the tensile force increase with smaller intensity. This is in accordance with the obtained bond stress distribution, as bond stresses are proportional to the tensile plate force derivate. From the Figure 6 could be concluded that compressive strength of the concrete hasn't influence on the tensile force along the CFRP strip, especially when the bond between concrete and CFRP strip is imperfect.

5 CONCLUSIONS

During the strengthening of the reinforced concrete constructions with CFRP strips improper execution may occur, which can lead to the appearance of weak zones in bond layer. Influence of such phenomenon is difficult to evaluate as it can not be accounted for in the design process. This paper presents influence of the compressive strength of the concrete on the global response of the strengthened beam.

A simple approach that consists of a bond constitutive model modification which can be easily incorporated in the existing numerical mode is proposed for modeling the weak zones.

From the analysis can be concluded that the compressive strength of the concrete hasn't influence on the maximal bearing capacity of the strengthened beam. Also compressive strength of the concrete hasn't influence on the bond stress distribution between concrete and CFRP strip, as well as tensile force distribution in the CFRP sheet. Results show that local bond stress concentrations, which appear where cross-section is changed, have negative influence on the quality of the bond between concrete substrate and CFRP plate. Further

validation of the proposed model and results obtained by parametric analysis using this model, has to be carried out by the experimental researches.

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KONCEPT PRORAČUNA SEKUNDARNIH SEIZMIČKIH ELEMENATA PREMA EVROKODU 8

Rezime:

S-17

U radu je analiziran koncept proračuna armiranobetonske konstrukcije sa sekundarnim seizmičkim elementima prema zahtevima Evrokoda 8. Iako se doprinos krutosti ovih elemenata zanemaruje prilikom seizmičkog odgovora konstrukcije, primena ovog koncepta usložnjena je nizom zahteva po pitanju klasifikacije i načina proračuna statičkih uticaja. Sa ciljem da se istaknu i objasne specifičnosti primene, pokazuju prednosti, ali i kritički razmotri upotreba opcije sekundarnih elemenata, izvršen je proračun osmoetažne armiranobetonske konstrukcije. Prikazane su dve metode za klasifikaciju sekundarnih elemenata, način proračuna uticaja u njima, kao i rezultati uporedne analize u kojoj su određeni elementi konstrukcije razmatrani kao primarni i kao sekundarni.

Кljučне речи: seizmika, sekundarni elementi, beton, duktilnost, Evrokod 8

CONCEPTUAL DESIGN OF SECONDARY SEISMIC ELEMENTS ACCORDING TO EUROCODE 8

Summary:

Analysis of conceptual design of reinforced concrete (RC) structure with secondary seismic elements in compliance with Eurocode 8 is presented in this paper. The application of this concept is complex due to the requirements regarding the classification and calculation of design internal forces, although the contribution of these elements in the total structural stiffness is neglected. The basic calculations of 8-story RC structure are performed with the main goal to emphasize and explain the problems of utilization of this concept. Its advantages are clearly presented and critical analysis of application in aseismic structural design is performed. The results of comparative analysis of structural design in which some structural elements are treated as a primary or as a secondary are presented.

Keywords: seismic design, secondary elements, concrete, ductility, Eurocode 8

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1 UVOD

Projektovanje seizmički otpornih konstrukcija sa ciljem zaštite ljudskih života iako najvažniji, nije i jedini cilj analize ponašanja i projektovanja objekata u seizmičkim područjima. Osim obezbeđivanja prostorne stabilnosti predmet istraživanja su i performanse objekta tokom i nakon zemljotresa, naročito nekonstruktivnih delova – fasade, pregrada, opreme, i uopšte povredljivost objekata [1]. Ipak, imajući u vidu potrebu za predstavljanjem zahteva i mogućnosti tehničkog propisa koji reguliše ovu oblast, Evrokoda 8 [2], u svetlu predstojećeg usvajanja ovog dokumenta kao nacionalnog standarda, fokus rada biće na rasvetljavanju jednog od aspekata primene Evrokoda 8 [2] u projektovanju objekata visokogradnje.

Savremeni seizmički propisi, među kojima je i Evrokod 8 [2], nude mogućnost da se doprinos pojedinih konstruktivnih elemenata u obezbeđivanju prostorne stabilnosti objekta za dejstvo zemljotresa zanemari. Takvi delovi konstrukcije nazivaju se "sekundarnim" seizmičkim elementima [2] i na njih se primenjuju samo odredbe Evrokoda 2 [3]. Nekoliko je razloga za uvođenje mogućnosti podele konstruktivnih elemenata na "primarne" i "sekundarne" u aseizmičkom projektovanju. Pre svega, neke odredbe Evrokoda 8 kao što su geometrijski uslovi, uslovi duktilnosti i zahtevi za oblikovanje detalja ili uslovi kapaciteta nosivosti, nije moguće ispuniti poštujući arhitektonske zahteve koji se odnose na položaj i dimenzije konstruktivnih elemenata. Ukoliko nije moguće promeniti dispoziciju ili bar dimenzije preseka, opcija označavanja tih elemenata kao sekundarnih može rešiti problem. Ovo je takođe i opcija za prevazilaženje problema da neki od konstruktivnih sistema, kao što su prethodno napregnuti sistemi ili sistemi ramova sastavljenih od stubova i delova ploča oslonjenih na njih (eng. *Flat slab frames*), nisu obuhvaćeni Evrokodom 8 [2]. Naime, postojeći eksperimentalni podaci i teorijska razmatranja nisu dovoljna da bi se sa dovoljnim stepenom sigurnosti objasnilo njihovo ponašanje pri dejstvu zemljotresa i da bi se na osnovu njih formirala pouzdana pravila za primenu u praksi. Dakle, jedna od opcija je svrstavanje ovakvih sistema u sekundarne seizmičke elemente, po principu ako problem nije moguće rešiti na zadovoljavajući način, možda ga je moguće eliminisati [4]. Konačno, čest slučaj je da se konstrukcija visokogradnje dominantno sastoji od armiranobetonskih zidova, ali da iz konstruktivnih razloga (npr. prihvatanja teškog fasadnog zida) dođe do formiranja relativno malog broja ramova. Strogo i formalno gledano, prema aktuelnim domaćim propisima [5] ovakav sistem bi se klasifikovao kao mešovit i značajni deo seizmičkog opterećenja od čak 25% bi morao biti "dodeljen" ramovima. Potpuno suprotno osnovnoj ideji projektanta - zidovima se prihvata seizmika a stubovima samo gravitacija, dolazi se do značajnog povećanja dimenzija stubova i greda. Takođe, uz poštovanje pravila za obezbeđivanje duktilnosti preseka dolazi do povećanja količine armature u ovim elementima. Zato, svrstavanje pojedinih elemenata, u ovom slučaju fasadnih ramova, u grupu sekundarnih seizmičkih elemenata deluje kao primamljiva mogućnost u okviru savremenih seizmičkih propisa [2]. Ipak, iako opcija ovakve klasifikacije elemenata na prvi pogled izgleda kao jedno od najjednostavnijih rešenja, primena u proračunu konstrukcije nije trivijalna zbog niza uslova i zahteva koje treba ispuniti.

Objašnjenje koncepta, uslova i zahteva koje treba ispuniti, kao i posledica klasifikacije elemenata u grupu sekundarnih seizmičkih elemenata prema EC8 [2] osnovni je cilj ovog rada. Kako bi se detaljno objasnili svi koraci prilikom projektovanja seizmički otporne konstrukcije sa sekundarnim seizmičkim elementima, osmišljen je adekvatan numerički primer. Na bazi razmatranja rezultata analize konkretnog objekta, sprovedeno je tumačenje odredbi propisa [2] i donošenje odgovarajućih zaključaka.

2 KONCEPT PRIMARNIH I SEKUNDARNIH ELEMENATA

Osnovni koncept rada sa sekundarnim seizmičkim elementima zasniva se na zanemarenju krutosti sekundarnih elemenata pri analizi odgovora sistema u seizmičkoj proračunskoj situaciji. Da bi ovakav pristup bio moguć, doprinos krutosti sekundarnih elemenata u ukupnoj krutosti sistema je ograničen na 15% sa ciljem da se globalni odgovor konstrukcije ne promeni značajno. Iz istog razloga, označavanje nekih elemenata kao sekundarnih nije dozvoljeno sa namerom da se promeni klasifikacija konstrukcije iz neregularne u regularnu [2]. Ova odredba ima pre svega preventivni karakter i treba da suzbije mogućnost da se neregularnosti značajnog dela konstruktivnog sistema "prikriju" plaštom sekundarnih elemenata - npr. zidovi postoje na svim spratovima po visini "samo" ih nema u prizemlju.

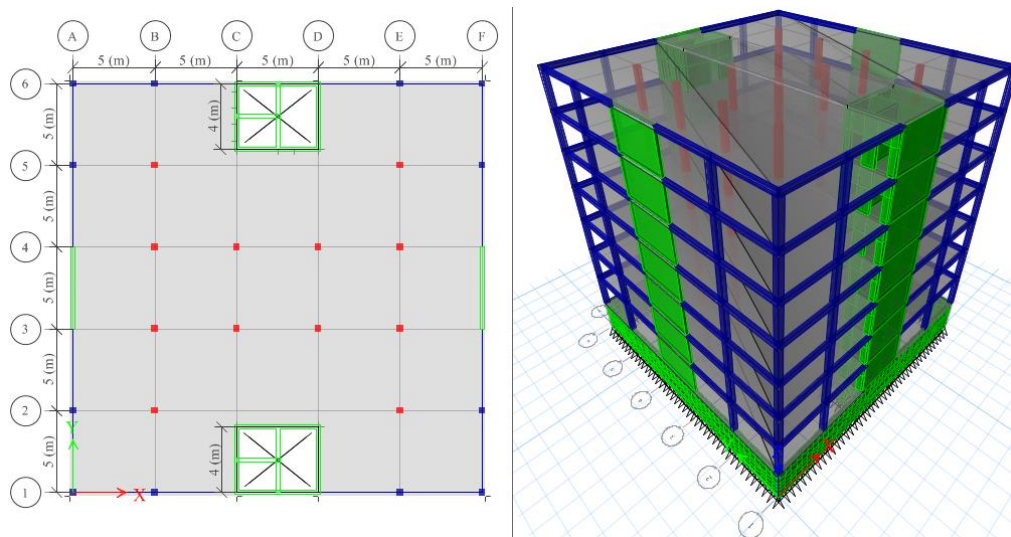
Uz uvažavanje činjenice da svi konstruktivni elementi moraju da prihvate i prenesu sva gravitaciona opterećenja u seizmičkoj proračunskoj situaciji, suštinska razlika u proračunu primarnih i sekundarnih seizmičkih elemenata leži u načinu ponašanja tih elemenata pri dostizanju istih maksimalnih pomeranja konstrukcije. Poznato je da duktilnost pomeranja konstruktivnog sistema zavisi od duktilnosti krivine preseka njegovih primarnih elemenata, koja odgovara faktoru redukcije seizmičkog opterećenja odnosno faktoru ponašanja q . Kako bi se postigla adekvatna duktilnost krivine, za takve elemente u Evrokodu 8 [2] propisani su zahtevi u pogledu armiranja preseka koji se odnose na geometrijske uslove, minimalne i maksimalne procenite armiranja podužnom armaturom, kao i osiguranja od smicanja i načina utezanja preseka u kritičnim oblastima. S druge strane, svi elementi koji su klasifikovani kao sekundarni ista pomeranja moraju izdržati pod pretpostavkom elastičnog ponašanja. To znači da takvim elementima treba obezbediti adekvatnu nosivost kojom će se sprečiti krti lom pri očekivanim, realnim pomeranjima konstrukcije. Ovakvi zahtevi rezultuju znatno većim statičkim uticajima u njima od onih koji se dobijaju uobičajenim proračunom, ali ih istovremeno oslobađa svih ograničenja i zahteva Evrokoda 8 [2] koji važe za primarne elemente.

Sigmund i ost. [6] pokazali su, primenom pushover analize na primeru kombinovanog sistema ramova i zidova (gde su ramovi klasifikovani kao sekundarni), da čak i pri zadovoljenju propisanih uslova, globalni odgovor konstrukcije može značajno da se razlikuje u zavisnosti od toga da li su ramovi označeni kao primarni ili kao sekundarni elementi. Takođe, uočeno je otvaranje plastičnih zglobova i na stubovima, kada su označeni kao sekundarni. Kako su ti elementi dimenzionisani samo prema EC2 [3], jako je važno sekundarne elemente dimenzionisati za uticaje koji se javljaju pri maksimalnim očekivanim pomeranjima konstrukcije u kojoj je krutost sekundarnih elemenata zanemarena. Fardis [7] je predložio postupak kojim je moguće proceniti ove uticaje, na osnovu odnosa relativnih međuspratnih pomeranja sistema u kome je krutost sekundarnih elemenata zanemarena i sistema u kome je krutost ovih elemenata uzeta u obzir.

3 NUMERIČKI PRIMER

3.1 ULAZNI PODACI

Postupak klasifikacije primarnih i sekundarnih seizmičkih elemenata, njihova analiza i dimenzionisanje opisani su na primeru simetrične, osmoetažne armiranobetonske konstrukcije spratne visine $h_s = 3,5$ m, prikazane na slici 1.



Slika 1 - Numerički model razmatrane AB konstrukcije, Etabs 2015 (CSI)

Elementi konstrukcije koji učestvuju u prijemu horizontalnog opterećenja su armiranobetonski zidovi, fasadni ramovi koje čine stubovi sa gredama po obimu konstrukcije i ramovi koje čine unutrašnji stubovi sa delovima ploče koja je direktno oslonjena na njih (eng. *Flat slab frames*). Dimenzije elemenata konstrukcije su: debljina ploče $h_p = 20$ cm, debljine zidova $d_z = 25$ cm, dimenzije greda $b_g/h_g = 25/40$ cm a dimenzije stubova $b_s/h_s = 40/40$ cm.

Pored sopstvene težine, u nivou tavanice deluje gravitaciono, jednakorasodeljeno dodatno stalno i povremeno opterećenje inteziteta $2,5 \text{ kN/m}^2$ i $3,0 \text{ kN/m}^2$, respektivno. Usvojena je klasa čvrstoće betona C 30/37, i armatura kvaliteta B 500 (klase duktilnosti B).

Projektno ubrzanje tla na osnovnoj steni $a_g = 0,2g$. Usvojen je projektni spektar tipa 1 za tlo kategorije B, prema EN 1998-1 [2]. Proračunom smičućih sila u zidovima utvrđeno je da sistem duktilnih zidova prihvata preko 65% ukupne seizmičke sile u oba ortogonalna pravca, što konstrukciju definiše kao sistem nevezanih zidova. Zahvaljujući regularnosti konstrukcije u osnovi i po visini, proračun seizmičkih uticaja izvršen je metodom Ekvivalentnih bočnih sila, sa usvojenim faktorom ponašanja $q = 3,0$ za klasu DCM.

Proračun stubova B1 i B2, grede u preseku ose 1 sa osama B i C, kao i njihovih veza, izvršen je primenom linearno-elastične analize prema EN 1992-1-1 [3] i EN 1998-1[2].

3.2 ANALIZA SEKUNDARNIH SEIZMIČKIH ELEMENATA

Projektovanje i oblikovanje detalja sekundarnih elemenata i njihovih veza potrebno je izvršiti za uticaje koji nastaju pri maksimalnim deformacijama koje se javljaju usled dejstva zemljotresa, kako bi imali dovoljni kapacitet nosivosti da prihvate i prenesu gravitaciono opterećenje uključeno u seizmičku proračunsku situaciju [2]. Maksimalne deformacije sistema moguće je odrediti iz analize modela u kome je doprinos bočne krutosti svih sekundarnih elemenata zamenaren, dok se fleksiona i smičuća krutost primarnih elemenata modelira sa isprskalim preseccima, pri čemu se moraju uključiti i P- Δ efekti.

Prethodni zahtevi Evrokoda 8 podrazumevaju da je potrebno izvršiti dve analize razmatrane konstrukcije za svaki pravac seizmičkog dejstva: jednu, u kojoj se uzima u obzir horizontalna krutost svih elemenata i, drugu, u kojoj je krutost svih sekundarnih elemenata zanemarena. Da bi ovakva analiza bila moguća potrebno je formirati dva numerička modela konstrukcije [7]:

- model koji obuhvata krutost primarnih i sekundarnih elemenata - SP model, i
- model koji obuhvata krutost samo primarnih elemenata - P model.

Formiranje P modela zasniva se na zanemarenju bočne krutosti elemenata konstrukcije koje projektant želi proglasiti sekundarnim. To se postiže njihovim modeliranjem bez fleksione krutosti (redukcijom momenta inercije ili modula elastičnosti) ili postavljanjem momentnih zglobova na njihovim krajevima. Na osnovu maksimalnih deformacija dobijenih iz P modela, određuju se uticaji u sekundarnim elementima u SP modelu, postupkom koji je opisan u 3.2.2.

Osim za potrebe određivanja maksimalnih deformacija sistema, P model koristi se još i za klasifikaciju sekundarnih elemenata kao i za proračun primarnih elemenata pri dejstvu seizmičkog opterećenja (slika 2a). S druge strane, SP model koristi se za proračun sekundarnih elemenata u seizmičkoj proračunskoj situaciji ali i za proračun cele konstrukcije u svim ostalim proračunskim situacijama.

Sa ciljem da se u što većoj meri pokažu specifičnosti izbora analize pojedinih elemenata kao sekundarnih, u ovom numeričkom primeru kao sekundarni elementi razmatrani su fasadni ramovi i ramovi koje čine unutrašnji stubovi sa pločom.

3.2.1 Klasifikacija sekundarnih seizmičkih elemenata

Prema odredbi 4.2.2 (4) Evrokoda 8 [2], ukupan dobrinos bočne krutosti svih sekundarnih seizmičkih elemenata ne sme da pređe 15% od doprinosa primarnih elemenata, odnosno 13,1% ukupne krutosti sistema ($15/(100+15) = 13,1\%$). Međutim, način određivanja doprinosa krutosti sekundarnih elemenata nije definisan, što omogućava dva pristupa analizi. Prva, i jednostavnija, metoda bazira se na određivanju udela seizmičkih sila u posmatranom pravcu koje ovi elementi prihvataju u nivou osnove [7]. Druga metoda podrazumeva određivanje odnosa relativnih međuspratnih pomeranja konstrukcije $d_{r,P}/d_{r,SP}$ u P i SP modelu u nivou posmatrane etaže, sračunatih prema EN 1998-1: 4.3.4 [2], koji mora biti manji od 1,15 za isti sistem horizontalnih sila u razmatranom pravcu [7], gde su:

$d_{r,P}$ relativna spratna pomeranja u P modelu, a

$d_{r,SP}$ relativna spratna pomeranja u SP modelu.

Ovakav način klasifikacije razmatra odnos krutosti sistema preko fleksibilnosti, što je jednostavniji pristup u praktičnoj primeni, pri korišćenju softvera za analizu konstrukcija. Imajući u vidu definiciju krutosti konstrukcije, akcentat je na istom sistemu horizontalnih sila - iste raspodele po visini ali i istog inteziteta. Prema preporuci autora [7] raspodela opterećenja po visini treba da odgovara seizmičkom opterećenju. Međutim, vrlo često se pri aproksimaciji krutosti sistema koristi i jednakoraspođeljeno opterećenje po visini, što može biti jednostavnije za unos u proračunski model. Ghali i Gayed [8] pokazali su, na primeru konstrukcije od 12, 25 i 50 spratova, da je uticaj primene ove dve raspodele na odnos međuspratnih pomeranja manji od 1,0%. U ovom numeričkom primeru, razlike su manje od 1,7%, pri čemu raspodela koja odgovara seizmičkom opterećenju daje konzervativnije rezultate.

Odstupanja rezultata analize primenom ove dve metode mogu biti značajna a posledica su različitih oblika deformisanja pojedinih konstruktivnih elemenata za prijem horizontalnog opterećenja po visini konstrukcije, koje druga metoda uzima u obzir. Razlika u obliku deformisanja elemenata posebno je naglašena u ovom numeričkom primeru (i to na višim

etažama), imajući u vidu izbor elemenata koji se razmatraju kao sekundarni (fasadni ramovi i unutrašnji stubovi). Analizom relativnih spratnih pomeranja u oba modela (tabela 1), koja su sračunata za isti sistem seizmičkog opterećenja, pokazano je da zbir doprinosa krutosti svih ramova ne zadovoljava propisani uslov u oba ortogonalna pravca - $d_{r,P}/d_{r,SP} > 1,15$. Poređenja radi, u nivou osnove ovi elementi prihvataju (svega) 8,9% ukupne seizmičke sile u X pravcu odnosno 8,6% u Y pravcu, čime bi propisani zahtev bio ispunjen.

Tabela 1 – Doprinos krutosti sekundarnih seizmičkih elemenata

Sprat	X PRAVAC			Y PRAVAC		
	$d_{r,P}$ [mm]	$d_{r,SP}$ [mm]	$d_{r,P}/d_{r,SP}$	$d_{r,P}$ [mm]	$d_{r,SP}$ [mm]	$d_{r,P}/d_{r,SP}$
8	18,19	15,20	1,197	18,74	15,75	1,190
7	18,43	15,52	1,188	18,79	15,95	1,178
6	18,15	15,44	1,175	18,38	15,75	1,167
5	17,18	14,78	1,162	17,26	14,95	1,155
4	15,35	13,36	1,149	15,24	13,34	1,142
3	12,47	10,99	1,135	12,15	10,76	1,129
2	8,41	7,51	1,120	7,75	6,95	1,114
1	0,93	0,90	1,031	0,95	0,91	1,044

Pored rešenja u kome bi se samo jedan sistem ramova klasifikovao kao sekundarni sistem (sistem fasadnih ramova ili ploče sa unutrašnjim stubovima), za zadovoljenje uslovljenog odnosa međuspratnih pomeranja pri klasifikaciji oba sistema ramova treba ili povećati doprinos krutosti primarnih elemenata ili smanjiti doprinos krutosti sekundarnih, ukoliko je to moguće. U ovom slučaju, smanjen je doprinos krutosti sekundarnih elemenata, smanjenjem dimenzija poprečnog preseka stubova u fasadi koje iznose $b_s/h_s = 25/40$ cm, a određene su iz uslova duktilnosti. Rezultati analize korigovanog konstruktivnog sistema, na koji deluje sistem seizmičkih sila primenjen u prvoj iteraciji, prikazani su u tabeli 2.

Tabela 2 – Doprinos krutosti sekundarnih seizmičkih elemenata (korigovan sistem)

Sprat	X PRAVAC			Y PRAVAC		
	$d_{r,P}$ [mm]	$d_{r,SP}$ [mm]	$d_{r,P}/d_{r,SP}$	$d_{r,P}$ [mm]	$d_{r,SP}$ [mm]	$d_{r,P}/d_{r,SP}$
8	18,24	15,89	1,148	18,82	16,45	1,144
7	18,48	16,19	1,141	18,93	16,63	1,138
6	18,24	16,07	1,135	18,56	16,39	1,132
5	17,29	15,36	1,126	17,47	15,52	1,126
4	15,46	13,84	1,117	15,42	13,82	1,116
3	12,61	11,37	1,109	12,30	11,12	1,106
2	8,52	7,76	1,098	7,85	7,17	1,095
1	0,94	0,92	1,022	0,96	0,93	1,032

3.2.2 Uticaji u sekundarnim seizmičkim elementima

Zahtev koji Evrokod 8 primenjuje za dimenzionisanje sekundarnih elemenata zasniva se na principu „jednakih pomeranja“, prema kom se pretpostavlja da njihovo ponašanje, kao i ponašanje njihovih veza, ostaje u elastičnoj oblasti pri dejstvu zemljotresa. Štaviše, uticaje u ovim elementima treba odrediti na osnovu maksimalnih pomeranja u fleksibilnijem sistemu (P

model), sa ciljem da se obuhvati najnepovoljniji mogući slučaj njihovog napreznja (slika 2.a). To praktično znači da će uticaji u sekundarnim elementima biti veći od onih koji bi se javili kada bi ponašanje cele konstrukcije bilo elastično pri dejstvu zemljotresa, i to srazmerno odnosu pomeranja P i SP modela. Dobra procena ovih uticaja po visini konstrukcije može se dobiti pomoću odnosa relativnih spratnih pomeranja $d_{r,P,m}/d_{r,SP,m}$ (slika 2.b), određenih za seizmičko opterećenje koje je sračunato prema dinamičkim karakteristikama odgovarajućeg modela [7], za razliku od slučaja analize njihovog doprinosa krutosti pri klasifikaciji. Koristeći definisane odnose, uticaji na m-tom spratu u svim sekundarnim elementima u SP modelu (slika 2a), dobijaju se modifikacijom kombinacije opterećenja u seizmičkoj proračunskoj situaciji [9], koeficijentom α , tako da je:

$$\sum_i G_{ki} + \alpha \cdot A_{Ed} + \sum_i \psi_{2,i} Q_{ki} \quad (1)$$

$$\alpha = q \cdot \frac{d_{r,P,m}}{d_{r,SP,m}} \cdot \frac{1}{(1-\theta_m)} \quad (2)$$

gde je:

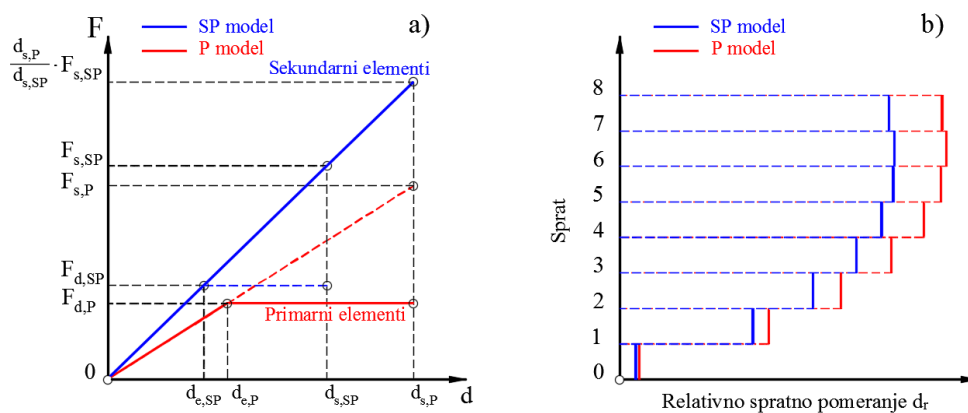
A_{Ed} seizmičko opterećenje,

q faktor ponašanja konstrukcije u posmatranom pravcu i za usvojenu klasu duktilnosti,

$d_{r,P,m}$ relativno spratno pomeranje u P modelu na m-tom spratu,

$d_{r,SP,m}$ relativno spratno pomeranje u SP modelu na m-tom spratu, a

θ_m koeficijent kojim se definišu P- Δ efekti, sračunat prema 4.4.2.2 (2) i (3) [2].



Slika 2 – Primena principa "jednakih pomeranja" na proračun sekundarnih elemenata

Opisani postupak može da se zakomplikuje pri analizi relativno krutih konstrukcija, sa osnovnim periodom oscilovanja manjim od T_c [2], gde princip "jednakih pomeranja" ne važi već princip "jednakih energija deformacija". Smatra se da je dovoljno tačno sračunati relativna međuspratna pomeranja koristeći izraz za duktilnost pomeranja μ_δ koji je dat u 5.2.3.4 (3) [2] i pomoću njih odrediti koeficijent α .

Dinamičke karakteristike P i SP modela i odgovarajuće seizmičke sile u posmatranom pravcu, dobijene primenom metode Ekvivalentnih bočnih sila, prikazani su u tabeli 3 a odnosi sračunatih relativnih spratnih pomeranja P i SP modela u tabeli 4.

Tabela 3 – Periodi oscilovanja sistema i seizmičke sile u nivou osnove

Osnovni periodi oscilovanja i seizmičke sile	X PRAVAC		Y PRAVAC	
	P model	SP model	P model	SP model
T_1 [s]	0,854	0,803	0,851	0,801
F_b [kN]	4565,63	4858,66	4585,92	4868,33

Tabela 4 – Određivanje odnosa relativnih spratnih pomeranja

Sprat	X PRAVAC			Y PRAVAC		
	$d_{r,P}$ [mm]	$d_{r,SP}$ [mm]	$d_{r,P,m}/d_{r,SP,m}$	$d_{r,P}$ [mm]	$d_{r,SP}$ [mm]	$d_{r,P,m}/d_{r,SP,m}$
8	16,95	15,43	1,099	17,45	15,97	1,093
7	17,11	15,72	1,088	17,49	16,15	1,083
6	16,83	15,60	1,079	17,10	15,91	1,075
5	15,93	14,91	1,068	16,05	15,07	1,065
4	14,22	13,44	1,058	14,17	13,42	1,056
3	11,56	11,04	1,047	11,29	10,80	1,045
2	7,79	7,53	1,035	7,20	6,96	1,034
1	0,86	0,89	0,966	0,88	0,90	0,978

Obzirom na to da se P- Δ efekti mogu zanemariti (vrednost koeficijenta $\theta_{\max} \approx 0,03$), uticaji u sekundarnim elementima dobijaju se množenjem odnosa relativnih pomeranja iz tabele 4 faktorom ponašanja $q = 3,0$, što povećava uticaje od seizmike od 3 do 3,3 puta u odnosu na uticaje dobijene za primarne elemente, izraz (2). U nastavku su analizirani rezultati proračuna pojedinih konstruktivnih elemenata, koji su razmatrani kao: (1) primarni i (2) sekundarni elementi.

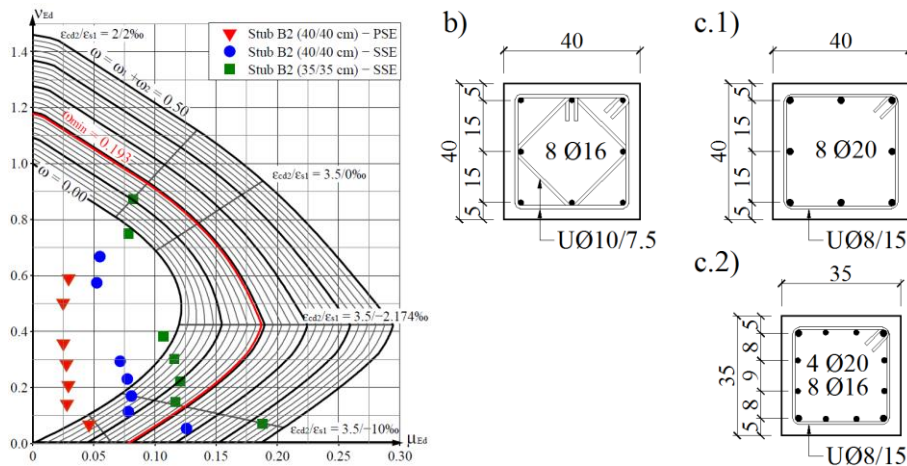
3.3 ANALIZA REZULTATA PRORAČUNA

Kako je to ranije istaknuto, pri proračunu primarnih elemenata od ključnog značaja je obezbediti njihovo duktilno ponašanje pri dejstvu zemljotresa odgovarajućim oblikovanjem detalja kako bi izdržali nelinearne deformacije koje se tom prilikom javljaju, dok se kod sekundarnih elemenata očekuje potpuno elastično ponašanje zasnovano na obezbeđivanju adekvatne nosivosti. Na primeru fasadnog stuba B1 i grede BC-1, unutrašnjeg stuba B2 i ploče koja se direktno oslanja na taj stub izvršena je uporedna analiza rezultata proračuna ovih elemenata u slučaju kada su deo primarnog odnosno sekundarnog sistema sa prihvatanje seizmičkog opterećenja.

3.3.1 Rezultati proračuna stuba B2 i njegove veze sa direktno oslonjenom pločom

Prikaz rezultata proračuna stuba B2 na slici 3, na osnovu merodavnih uticaja na pojedinim etažama, jasno pokazuje posledice njegove klasifikacije kao primarnog (PSE) odnosno sekundarnog (SSE) elementa. Kada je on razmatran kao primarni, zahvaljujući malom doprinosu unutrašnjeg stuba ukupnoj krutosti razmatrane konstrukcije, potrebne površine podužne armature su značajno manje od minimalno propisane za klasu DCM (slika 3a). Ista (minimalna) armatura dovoljna je da obezbedi zahtevanu nosivost stuba kada je razmatran i kao sekundarni, osim na poslednjoj etaži koja je merodavna za dimenzionisanje preseka. Pored toga, moguće je i smanjiti dimenzije preseka, imajući u vidu da uslov duktilnosti ne važi za sekundarne elemente tj. da se dimenzije preseka mogu odrediti iz uslova maksimalnog

dozvoljenog napona u betonu [3], što u konkretnom primeru znači smanjenje dimenzije sa 40 cm na 35 cm (tabela 5). Poređenja radi, za konstrukciju od 11 etaža sa istom dispozicijom, ovo smanjenje bi iznosilo oko 45% površine stuba.



Slika 3 – Rezultati proračuna stuba B2: a) dijagram interakcije, b) poprečni presek stuba B2 kao PSE, c) poprečni preseki stuba B2 kao SSE

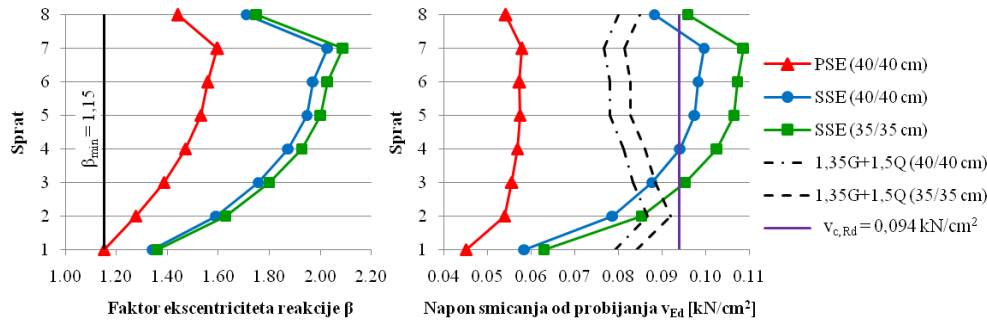
Tabela 5 – Rezultati proračuna stuba B2

Klasifikacija	b/h [cm]	$\rho_{sl,max}$ [%]	$\omega_{wd,1}$	$\omega_{wd,2-7}$
PSE	40/40	1,00	0.187	0,113
SSE	40/40	1,57	0.106	0,106
SSE	35/35	2,34	0.125	0,125

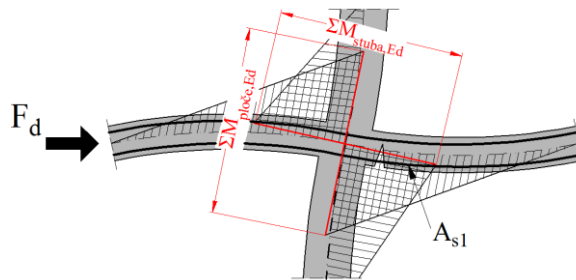
Razlike u potrebnoj količini uzengija prikazane su na slikama 3b i 3c, kao i u tabeli 5 pomoću mehaničkog zapreminskog procenta armiranja $\omega_{wd,m}$ određenog za ceo m-ti sprat. Na osnovu prikazanih rezultata može se zaključiti da zahtevi za armiranje uzengijama primarnog seizmičkog stuba u kritičnim oblastima ima za rezultat značajno povećanje količine uzengija, i do 75% u nivou osnove gde je potrebno obezbediti adekvatno utezanje preseka. Jasno je, takođe, da klasifikacija u sekundarne rezultira većom podužnom armaturom, ali kada se povede bitka za svaki kvadratni centimetar (skupog) slobodnog prostora, verovatno će opcija jače armiranih stubova manjih dimenzija dobiti prednost nad stubovima većih dimenzija sa minimalnom armaturom.

Pored dokaza nosivosti stuba, neophodno je dokazati i nosivost veze stuba sa pločom pri maksimalnim pomeranjima usled dejstva zemljotresa, kao što je navedeno u 3.2, što pre svega podrazumeva kontrolu smičućih napona od probijanja. Poznato je da ovi naponi zavise od gravitacionog opterećenja, ali se njihova vrednost značajno povećava pri dejstvu zemljotresa, što je posledica povećanja ekscentriciteta opterećenja obuhvaćenog koeficijentom β [3]. Na slici 4 prikazane su vrednosti koeficijenta β kao i smičućih napona u kritičnom preseku po visini konstrukcije, za stalnu proračunsku situaciju i seizmičku, pri različitoj klasifikaciji stuba B2. Može se zaključiti da ploča ima dovoljnu nosivost na probijanje bez armature za smicanje ($v_{c,Rd} = 0,094 \text{ kN/cm}^2$) pri dejstvu gravitacionog opterećenja u stalnoj proračunskoj situaciji. Kao rezultat povećanja momenata savijanja u sekundarnim elementima, rastu vrednosti

koeficijenta β (gotovo dva puta više od minimalne propisane vrednosti od 1,15 [3]) i smičućih napona, koje prevazilaze vrednosti sračunate u stalnoj proračunskoj situaciji kao i nosivost ploče bez smičuće armature, što rezultuje potrebom za osiguranjem ploče armaturom za smicanje od probijanja.



Slika 4 – Vrednosti koeficijenta β i napona smicanja od probijanja u funkciji klasifikacije stuba B2



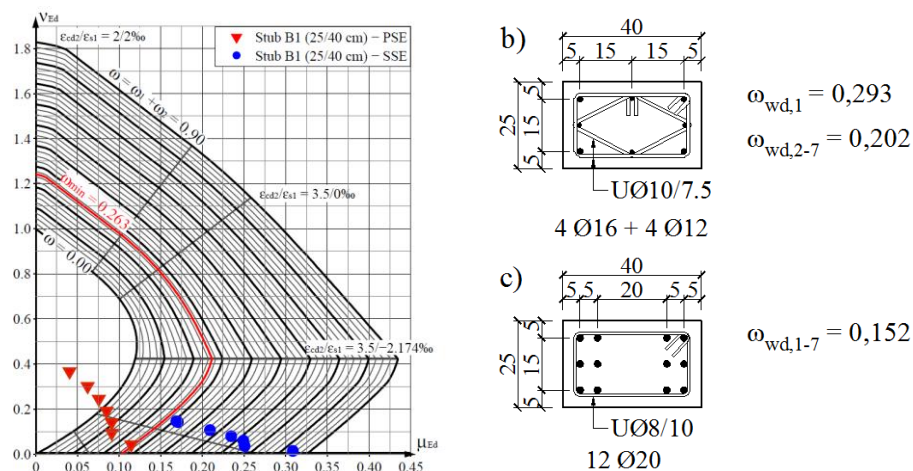
Slika 5 – Momenti u ploči usled seizmičkog opterećenja

Kako bi veza stuba i ploče u sekundarnom sistemu imala dovoljni kapacitet nosivosti da u elastičnoj oblasti prenese gravitaciono opterećenje pri dejstvu zemljotresa, od suštinske je važnosti obezbediti i odgovarajuću armaturu za savijanje na mestima oslonaca ploče, tj. na vezi ploča-stub. Imajući u vidu da se momenti velikog inteziteta na krajevima stubova uravnotežuju sa momentima u ploči (slika 5), javlja se potreba za armiranjem obe zone ploče nad osloncem usled momenata alternativnog znaka. U konkretnom slučaju, ovi momenti dostižu i do 80 % negativnih oslonačkih momenata od gravitacionog opterećenja.

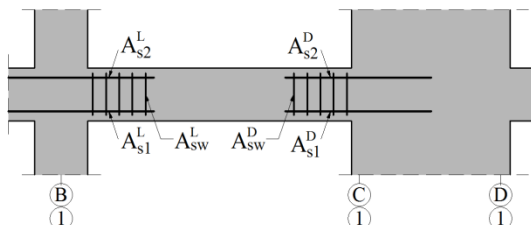
3.3.2 Rezultati proračuna rama u osi 1 - stub B1 i greda BC-1

Činjenica da je uticaj krutosti ramova na veličinu i oblik deformacije čitave konstrukcije po visini dominantan, obrazložena je u delu 3.2.1. Do istog zaključka dolazi se analizom rezultata proračuna elemenata rama u osi 1, prikazanih na slici 6 i u tabeli 6. Kao posledica ramovskog dejstva u kome je izražen uticaj aksijalnih sila u stubovima, primena izraza (1) na proračun fasadnih stubova kao sekundarnih elemenata dovodi do smanjenja aksijalnih sila uz povećanje momenata što dodatno utiče na povećanje potrebne površine armature, posebno na donjim etažama (slika 6a). Zbog smanjene širine preseka stuba (odredene iz uslova duktilnosti), normalizovane aksijalne sile u kritičnoj oblasti u osnovi su visoke ($v_{d,max} = 0,53$), što rezultuje

izraženom potrebom za utezanjem stubova kao primarnih elemenata. Vrednosti mehaničkog zapreminskog procenta armiranja $\omega_{wd,m}$ su za 33% do 93% veće od vrednosti koje odgovaraju stubovima kada su razmatrani kao sekundarni. Međutim, to nije dovoljno dobar razlog da bi se opravdala klasifikacija ovog stuba kao sekundarnog, pre svega sa ekonomskog aspekta, imajući u vidu znatno veće količine potrebne podužne armature. Smanjenje dimenzija poprečnog preseka, u ovom slučaju, nije opcija jer dovodi do prekoračenja maksimalnog koeficijenta armiranja od 4 % [3]. Očigledno je da klasifikacijom ovih stubova kao sekundarnih nije moguće postići željene rezultate i da ih je najbolje razmatrati kao deo primarnog sistema.



Slika 6 – Rezultati proračuna stuba B1: a) dijagram interakcije, b) poprečni presek stuba B1 kao PSE, c) poprečni presek stuba B1 kao SSE



Slika 7 – Armatura grede BC-1

Tabela 6 – Rezultati proračuna grede BC-1 na etaži 5

Klasifikacija	A_{s1}^L	A_{s2}^L	A_{s1}^D	A_{s2}^D	A_{sw}^L	A_{sw}^D	$\omega_{wd,5}$
PSE	3Ø16	2Ø20	3Ø16	3Ø20	UØ8/10	UØ8/10	0,227
SSE	6Ø20	3Ø25	5Ø20	5Ø25	UØ8/7.5	UØ8/10	0,265

Slični zaključci mogu se primeniti i na grede koje su deo fasadnih ramova. Rezultati proračuna grede BC-1 pokazuju očigledan uticaj povećanja momenata savijanja u sekundarnim seizmičkim gredama, dobijenih primenom izraza (1), koji rezultuje povećanjem armature i do 3 puta. U ovom primeru, uzengije u primarnim gredama određene iz uslova kapaciteta nosivosti praktično su iste kao uzengije sekundarnih greda određene iz elastičnih uticaja (tabela 6).

4 ZAKLJUČAK

Analiza proračuna armiranobetonske konstrukcije sa sekundarnim seizmičkim elementima predstavljena u ovom radu ukazala je na prednosti i nedostatke primene ovog zanimljivog koncepta u aseizmičkom projektovanju objekata visokogradnje. Iako projektantski primamljiv, jer je dimenzionisanje i oblikovanje detalja definisano "samo" Evrokodom 2 [3], sprovođenje koncepta sekundarnih seizmičkih elemenata je zametan posao sa prilično neizvesnim ishodom. Očekivana korist u vidu lakšeg proračuna kompromitovana je postupkom klasifikacije i proračuna statičkih uticaja na bazi uporedne analize dva modela. Takođe, postoje značajne posledice na ponašanje čvora stub-ploča i osiguranje ploče od proboja. Potencijalno se može očekivati smanjenje dimenzija poprečnih preseka vertikalnih elemenata ukoliko je njihov doprinos krutosti sistema relativno mali, uz "naplatu" kroz veću količinu podužne armature. Uvođenjem ovog koncepta Evrokod 8 [2] otvara mogućnosti za kompleksno tretiranje pojedinih delova konstruktivnog sistema, a tumačenje zahteva propisa svakako predstavlja istraživački i projektantski izazov.

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PRIMENA KORELACIJE DIGITALNE FOTOGRAFIJE PRI MERENJU PRSLINA ARMIRANOBETONSKIH GREDA

Rezime:

S-18

U ovom radu je prikazan princip merenja pomeranja bezkontaktnom optičkom metodom zasnovanom na korelaciji digitalne fotografije. U radu je prikazana moguća primena ove metode za merenje pomeranja, određivanja položaja i širine prslina pri eksperimentalnom ispitivanju armiranobetonskih greda. Proces formiranja i određivanje pomeranja na prslinama se može jednostavno odrediti na celokupnom osmatranom području grede primenom ovog niskobudžetnog sistema merenja.

Ključne reči: korelacija digitalne fotografije, armiranobetonska greda, prslina

APPLICATION OF DIGITAL IMAGE CORRELATION IN CRACK WIDTH MEASUREMENT OF RC BEAMS

Summary:

This paper presents the principles of a non-contact, optic metrology method based on digital image correlation (DIC) of a speckle-painted surface. The paper reports a possible application of a 2D DIC method on displacement measurement, crack mapping and crack width calculation on full-scale reinforced concrete beams. Crack development and crack kinematics can be accurately measured within the field of view by application of simple and low-cost experimental setup.

Key words: digital image correlation, reinforced concrete, cracks

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1 UVOD

U cilju razumevanja procesa i mehanizma loma armiranobetonskih elemenata, eksperimentalno ispitivanje i dalje predstavlja nezaobilaznu fazu istraživanja. Kako bi se stekla jasna slika ponašanja konstrukcije pri naprezanju, potrebno je prikupiti što je moguće više podataka. Donošenje zaključaka istraživanja zavisi od kvaliteta i količine informacija, a njihovo prikupljanje u toku eksperimentalnog ispitivanja može biti dugotrajan i osetljiv posao.

Uvid u mehanizam prenošenja opterećenja kroz armiranobetonsku isprskalu gredu se zasniva na merenju pomeranja i beleženja tačne slike prslina (mapiranje prslina). Tradicionalna metodologija merenja obuhvata vizuelno mapiranje prslina i merenje relativnog pomeranja između ograničenog broja predefinisanih tačaka. Mapiranje prslina je često dugotrajan proces i u velikoj meri zavisi od iskustva osobe koja merenja obavlja, dok je merenje relativnih pomeranja na obrazima formiranih prslina moguće u ograničenom broju tačaka i podrazumeva rasterećenje eksperimentalnog uzorka kako bi se merni instrumenti postavili na putanji prethodno formiranih prslina. U cilju prevazilaženja ograničenja tradicionalnih metoda, metode eksperimentalnog ispitivanja se razvijaju u pravcu primene novih tehnologija koje omogućavaju kontinualno i brzo prikupljanje potrebnih podataka.

U ovom radu su prikazani principi merenja bezkontaktnom metodom zasnovanom na korelaciji digitalne fotografije, koja omogućava tačno mapiranje svih formiranih prslina na osmatranom području i merenje pomeranja na celokupnoj dužini prslina u toku nanošenja opterećenja. Principi merenja i rezultati naknadne obrade izmerenih podataka su prikazani na primeru ispitivanja dve grede.

U prethodnom periodu je na Građevinskom fakultetu Univerziteta u Beogradu sproveden veći broj ispitivanja granične nosivosti grednih elemenata na savijanje i smicanje, opterećenih koncentrisanim silama u trećinama ili polovini raspona. U ovim ispitivanjima su pomeranja pri deformaciji grede merena tradicionalnim metodama i 2D DIC sistemom (DIC – digital image correlation). Kratak teorijski opis DIC metode merenja, procedura izvršenih merenja, naknadne obrade digitalne fotografije bočnog lica grede, i postupka određivanja pomeranja na obrazima formiranih prslina su prikazani u predmetnom radu.

2 KORELACIJA DIGITALNE FOTOGRAFIJE

Potreba za dobijanjem kompletne informacije o deformaciji osmatrane površine je dovela do razvoja bezkontaktnih optičkih metoda merenja polovinom prethodnog veka. Prve tehnike merenja su se zasnivale na interferometriji koherentnog svetla (lasera), poput holografske i takozvane speple interferometrije, a kasnije na poređenju promene intenziteta sive boje oslikane površine usled pomeranja [1]. Najmlađa i trenutno napopularnija bezkontaktna optička metoda je metoda korelacije digitalne fotografije, odnosno, metoda osmatranja tačkasto oslikane površine. Prva praktična upotreba DIC metode merenja pomeranja je vezana za početak razvoja digitalnih kamera i računara neophodnih za obradu fotografije, 80-ih godina prethodnog veka [2].

Sa daljim razvojem tehnologije, u narednim decenijama dolazi do značajnog napretka metode koji se ogleda u pojednostavljenju proračunskog algoritma uz povećanje tačnosti merenja, a u poslednjoj deceniji je primetan i razvoj komercijalnih sistema (Correlated Solutions, GOM Gbmh, Dantec Dynamics). Sve veća dostupnost i proširenje polja upotrebe se najjasnije ogleda u broju naučnih radova koji sadrže rezultate merenja dobijenih DIC

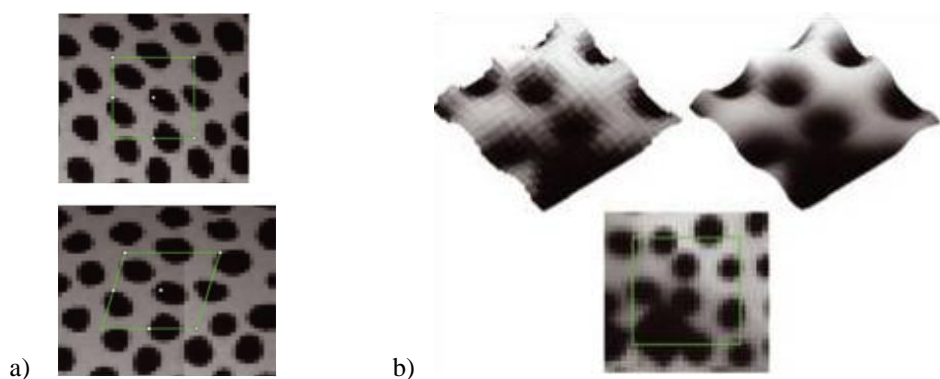
metodom, a prema [9], poslednjih godina je broj radova u eksponencijalnom porastu, tako da je 2014. godine objavljeno više od 6000 radova. Međutim, potrebno je napomenuti da je upotreba DIC metode merenja prethodnih decenija bila rezervisana na modelska ispitivanja u mašinskoj industriji, a da je tek poslednjih godina primetna sve veća primena pri ispitivanju elemenata građevinskih konstrukcija i to prvenstveno uzoraka malih dimenzija ili lokalnih naprezanja. Objavljeni radovi koji sadrže ispitivanja konstrukcija realnih dimenzija su izuzetno retki [3-7].

Osnovna karakteristika koja je dovela do široke upotrebe DIC metode je mogućnost merenja pomeranja površine elementa proizvoljne dimenzije (od mikro do makro površina) uz tačnost rezultata merenja veću od jedinične informacije fotografije, piksel-a. Tako na primer, uz kvalitetnu fotografiju je moguće postići tačnost merenja pomeranja od stotog dela piksela. Uopšteno, sistem za merenje se sastoji od jedne ili više sinhronizovanih digitalnih kamera, tako da se pomeranje površine može meriti u ravni ili prostoru. Najjednostavnijim sistemom koji se sastoji od jedne kamere, moguće je meriti samo pomeranja u ravni osmatrane površine. Kako bi se na ovaj način dobili upotrebljivi podaci, neophodno je ispuniti nekoliko zahteva: osa kamere mora biti postavljena upravno na osmatranu površinu, a deformacija elementa mora biti ograničena na pomeranje u ravni. U slučaju zakrivljene površine ili deformacije izvan ravni, neophodna je upotreba sistema sa dve ili više kamera koje moraju biti usklađene, tako da se pomeranje tačke u prostoru određuje triangulacijom. Svi komercijalni sistemi su sistemi sa više kamera.

2.1 OSNOVE I PRINCIPI KORELACIJE DIGITALNE FOTOGRAFIJE

DIC je optička metoda merenja koja se zasniva na obradi digitalne fotografije, sa ciljem određivanja globalnih pomeranja i lokalnih deformacija. Podaci o pomeranju tačaka na površini posmatranog objekta se dobijaju poređenjem digitalnih fotografija površine eksperimentalnog uzorka pre i nakon deformacije, odnosno, korelacijom referentne i tekuće slike. Osnovni princip DIC je korelacija dela fotografije, odnosno, jedinstvene predefinisane grupe piksela (subset), pri čemu se pomeranje grupe određuje nezavisno u odnosu na susedne. Podelom slike na niz grupa i određivanjem pomeranja svake od njih, dobija se kompletna informacija pomeranja na osmatranoj površini. Neophodnost sračunavanja pomeranja za grupu u odnosu na pojedinačne piksele je uslovljeno jednoznačnošću određivanja položaja grupe na različitim fotografijama. Grupa sadrži dovoljno informacija da bi imala jedinstven "otisak" koji je definisan varijacijom sivih boja.

Sa deformacijom površine uzorka deformiše se i slika grupe, odnosno, dolazi do promene njenog položaja i oblika. Na ovaj način, korelacija referentne i tekuće fotografije, slično kao i u metodi konačnih elemenata, podrazumeva upotrebu funkcija oblika (Slika 1a) i odgovarajućih interpolacionih šema pod pretpostavkom kontinualne deformacije (Slika 1b). U cilju sračunavanja pomeranja, poklapanje predefinisane grupe na referentnoj fotografiji sa istom u deformisanom stanju, se upotrebom međupikselske informacije iz interpolacije, i pomeranja i deformacije iz funkcije oblika, određuje kao ekstrem primenjene korelacione funkcije. Korišćenjem interpolacionih šema višeg reda, pomeranja tačke unutar grupe je u slučaju kvalitetne fotografije moguće odrediti sa tačnošću koja dostiže vrednost i do 100-og dela piksela, a kvalitet fotografije se meri količinom informacija koje fotografija sadrži. Pored određivanja promene položaja grupe piksela, korelacijom je određena i lokalna deformacija unutar grupe – aksijalna i smičuća dilatacija.



Slika 1: Prikaz translacije, aksijalne dilatacije i klizanja grupe piksela upotrebom funkcije oblika [12](a), Prikaz snimljenih (levo) i interpoliranih (desno) podataka grupe (dole) [11](b)

Kako je rečeno, osnovni nosilac informacije pri korelaciji je računska grupa piksela, što znači da treba da zadovoljava uslov neponovljivosti na razmatranoj fotografiji, uz količinu informacija koja je izražena varijacijom sivih boja. U cilju jedinstvenosti i dodatnog umanjenja mogućnosti ponavljanja vizuelnog oblika određene grupe na posmatranoj površini, uobičajeno je stohastičko nanošenje šare, a u cilju tačnog određivanja granica grupe, nanošenje šara na podlozi što je moguće većeg svetlosnog kontrasta.

2.2 PRINCIPI DOBIJANJA KVALITETNE FOTOGRAFIJE

Pored primenjenih procedura pri naknadnoj obradi fotografija, najbitniji deo izvođenja eksperimenta je dobijanje kvalitetne fotografije, odnosno, fotografije sa što je moguće većim brojem informacija. Ova procedura podrazumeva oslikavanje uzorka, optimizaciju osvetljenja i u slučaju sistema sa više kamera, njihovu kalibraciju. Kvalitetna fotografija omogućava izbor manjih grupa, što povećava tačnost merenja, a veličina grupe je određena količinom informacija koju nosi.

Praktično svi DIC softveri se zasnivaju na obradi fotografija koje sadrže samo nijanse sive boje, tako da je karakteristika grupe određena intenzitetom sivih boja, što garantuje njenu jedinstvenost na osmatranom području. Pravilo je da grupa sadrži nekoliko područja sa što je moguće većim kontrastom, tako da se oslikavanje površine uzorka svodi na nanošenje jednobojnih šara, najčešće na oslikavanje crnom bojom na beloj podlozi, ili obrnuto. Imajući u vidu da se ovakav zahtev u praksi najčešće ispunjava nanošenjem tačkica, DIC se u literaturi često naziva i metodom osmatranja tačkasto oslikane površine (applied painted speckle pattern).

Verovatno najteži deo postavljanja eksperimenta je oslikavanje uzorka, jer je poželjno ispuniti niz zahteva u pogledu veličine jednobojnih površina, njihove gustine i kontrasta. Kako je prethodno rečeno, visok stepen korelacije grupe na referentnoj i tekućoj fotografiji zahteva dovoljnu količinu informacija koje se nalaze na fotografiji, od čega zavisi i izbor veličine grupe. Sa jedne strane računске vrednosti su tačnije sa manjom grupom, ali su u isto vreme podložnije šumu, tako da je veličinu grupe potrebno odrediti u zavisnosti od kvaliteta fotografije. U zavisnosti od dimenzija posmatrane površine, potrebno je obezbediti i veličinu apliciranih tačkica, uz uslov da njihova gustina bude najveća moguća, a kontura jasno određena. U idealnim uslovima, veličina jednobojne površine iznosi 3-5 piksela, a odnos bele i crne boje unutar računске grupe 50%. Pri ovakvim uslovima računska grupa sadrži 3-5

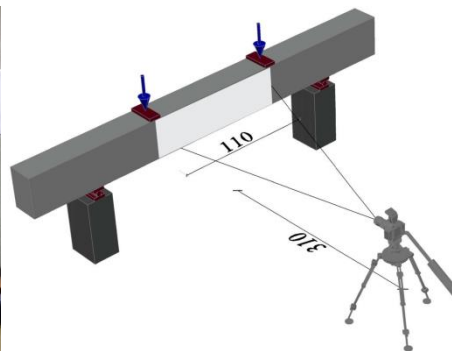
jednoboynih površina, što odgovara ukupnoj površini od 20x20 piksela. U slučaju nanošenja tačkica veće površine, potrebno je izabrati veću računsku grupu, a u slučaju nanošenja tačkica manje veličine, dolazi do preklapanja boja i nemogućnosti tačne korelacije. Pored pravilnog oslikavanja, upotrebom pravilnog osvetljenja se povećava kontrast različito obojenih površina, a samim tim i količina informacija na fotografiji. Najbolji rezultati se postižu upotrebom difuznog osvetljenja u cilju umanjenja refleksije. Više informacija o teorijskim postavkama, proračunskim algoritmima i principima eksperimentalne postavke je moguće pronaći u [1, 8], a korisne savete u seriji od 24 kratka članka u časopisu Experimental Technics [9-12].

3 EKSPERIMENTALNA POSTAVKA

U prethodnom periodu je na Građevinskom fakultetu Univerziteta u Beogradu sproveden veći broj ispitivanja granične nosivosti grednih elemenata na savijanje i smicanje, opterećenih koncentrisanim silama u trećinama ili polovini raspona. Pomeranja i deformacije su mereni tradicionalnim instrumentima (ugibomeri, deformetri, merne trake) i optičkim osmatranjem bočne stranice greda. Za prikaz rezultata merenja DIC metodom su odabrane dve grede:

- OPC-200/350-3x8 - Greda pravougaonog poprečnog preseka dimenzija 20x30cm, raspona 3.0m, opterećena parom koncentrisanih sila u trećinama raspona i
- 350-185/80-1 - Kompozitna greda sačinjena od ošupljenih prethodno napregnutih ploča visine 35cm i kompozitnog topinga visine 8cm, raspona 4.0m, opterećena koncentrisanom silom u polovini raspona

Ispitivanje prve grede je sprovedeno u cilju određivanja granične nosivosti na savijanje, a druge granične nosivosti na smicanje. Digitalnom kamerom su osmatrane srednja trećina raspona prve, odnosno, istočni smičući raspon druge grede, a dužina osmatrane bočne površine greda iznosila je do 1,1m. Na Slici 2 je prikazana šema eksperimentalne postavke i oslikana površina druge grede.



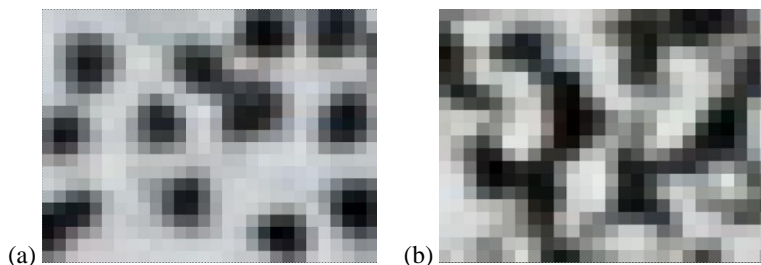
Slika 2: Prikaz oslikane bočne stranice grede (a) i dispozicija eksperimenta (b)

Cilj merenja DIC metodom u sprovedenim eksperimentima je bilo mapiranje i određivanje pomeranja na obrazima formiranih prslina. S obzirom da prsline predstavljaju zone diskontinuiteta, relativna pomeranja na prslinama (širina i klizanje) su sračunata na osnovu

izmerenih pomeranja neisprskalog betona između njih, a imajući u vidu maksimalno procenjenu širinu prsline od 0,5mm, zahtevana tačnost merenja je iznosila 0,01mm.

Optički sistem upotrebljen za merenje se sastojao od jedne DSLR (digital single-lens reflex) kamere tipa Cannon 6D (senzor veličine 32x24mm, rezolucija 5472x3648 piksela) i telefoto sočiva tipa Cannon EF 100mm 2/2.8L. Prilikom postavljanja eksperimenta je pretpostavljeno da će najveća greška merenja iznasti do 1/20 piksela. Greška u ovom iznosu se najčešće uzima za gornju granicu greške merenja izvedenog u laboratorijskim uslovima, a usvojena je prema rezultatima probnih ispitivanja (u kojima je postignuta tačnost od 1/30 piksela) i neizbežne varijacije dnevnog osvetljenja usled očekivanog trajanja eksperimenta. Sa usvojenom optičkom opremom i zahtevanom tačnosti merenja, osmatrana površina iznosi maksimalno 1100x730mm, odnosno, piksel fotografije odgovara površini od 0,2x0,2mm na oslikanoj stranici grede. S obzirom da je pri osmatranju upotrebljeno sočivo nepromenljive žižne daljine, udaljenost kamere do grede je definisana širinom osmatrane površine. U slučaju grede ispitane do loma savijanjem, udaljenost je iznosila 3,1m, (osmatrana površina širine 1000mm), a u slučaju grede ispitane do loma smicanjem 3,4m, (osmatrana površina širine 3100mm). Shodno tome, veličina piksela je odgovarala dužini od 0,181 i 0,198mm na bočnoj strani prve i druge grede, respektivno.

Najveći nedostatak primenjenog DIC sistema sa jednom kamerom je greška merenja prouzrokovana eventualnim pomeranjem upravno na osmatranu površinu. S obzirom da je ova greška proporcionalna proizvodu pomeranja i rastojanja kamere do osmatrane ravni, usvojena su sočiva sa velikom žižnom daljinom. Izmereno pomeranje upravno na posmatranu ravan je iznosilo do 0,2mm, što sa rastojanjem kamere većim od 3000mm, ukazuje da je ova greška merenja zanemarljiva. Upotrebom telefoto sočiva je umanjena i greška merenja pomeranja usled optičke distorzije, koja je kontrolisana prema proceduri preporučenoj u [10]. Rezultati kontrole su ukazali da korekcija fotografija usled distorzije sočiva nije potrebna.



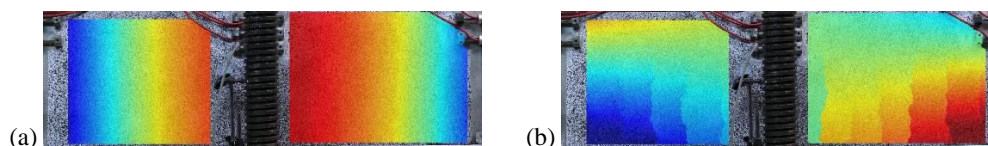
Slika 3: Prikaz dela fotografije oslikane površine šablonom od polivinila (a) i direktnim nanošenjem farbe iz boce pod pritiskom (b)

Nepomerljivost kamere tokom izvođenja eksperimenta je obezbeđena oslonjanjem na "teški" stativ, a posebna pažnja je posvećena orijentisanju kamere, u smislu obezbeđenja upravnosti na posmatranu površinu. S obzirom da je optički sistem za osmatranje formiran od komercijalnih komponenti, sve napredne funkcije kamere i objektiva poput stabilizacije slike, automatskog otvaranja blende, zumiranja i fokusiranja su deaktivirane. Difuznim LED osvetljenjem i upotrebom mat farbe pri oslikavanju osmatrane površine je obezbeđeno ravnomerno osvetljenje i umanjen je odsjaj. U toku trajanja eksperimenta, greda je fotografisana nakon svakog koraka opterećenja – neposredno nakon dostizanja predviđenog koraka opterećenja i neposredno pre otpočinjanja sledećeg.

Priprema osmatrane površine greda se sastojala u prethodnom farbanju bočne stranice grede belom bojom, na koju su kasnije nanošene crne šare. Ova faza pripreme eksperimenta je obično i najteža, jer je za dobijanje kvalitetne fotografije i merenja željene tačnosti neophodno ispuniti niz kriterijuma u pogledu veličine jednobojnih površina opisanih u prethodnom poglavlju. U predmetnom ispitivanju, piksel fotografije je odgovarao dužini od oko 0,2mm, tako da je bilo potrebno naneti crne šare veličine oko 0,6mm. U toku eksperimenta su isprobana dva načina nanošenja šara: šablonom i direktnim nanošenjem farbe iz boce pod pritiskom. Šablon je napravljen od polivinil folije debljine 0,2mm, na kojoj su laserski izbušeni nepravilno raspoređeni kružni otvori prečnika 0,5mm. Manji prečnik u odnosu na zahtevanu dimenziju tačkice je usvojen usled neizbežnog podlivanja farbe tokom nanošenja. Drugi način je obuhvatio direktno nanošenje farbe iz boce pod pritiskom, upotrebom cevastih nastavaka na izlaznom ventilu. Iako su se obe metode pokazale kao zadovoljavajuće, drugi metod je usvojen pri ispitivanju greda opisanih u ovom radu. Postignuta je srednja vrednost prečnika apliciranih crnih tačaka u iznosu od 0.7 mm, sa prosečnom vrednošću odnosa crne i bele boje u iznosu od 35%. Primer dobijenih šara je prikazan na slici 3, a pregled mogućih načina oslikavanja površine se može naći u uputstvima nekog proizvođača komercijalnih sistema. Za korelaciju snimljenih digitalnih fotografija je korišćeno nekoliko univerzitetskih softverskih rešenja (Opticist, MatchID, Ncorr), a slike prikazane u ovom radu su rezultat proračuna u Ncorr [13]. Pri obradi fotografija je usvojena veličina grupe od 30 piksela.

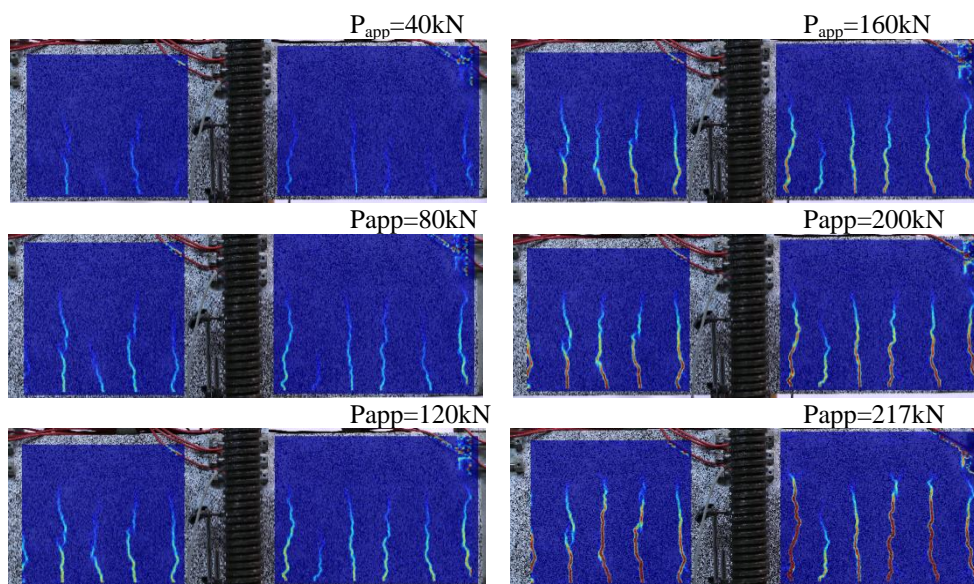
4 REZULTATI MERENJA

Rezultat korelacije digitalnih fotografija grede u različitim fazama opterećenja su pomeranja osmatrane površine u pikselima, što je osnovna jedinica mere fotografije. S obzirom da je osa kamere bila upravno postavljena na ravnu osmatranu površinu, pomeranja pri deformaciji grede su određena kao proizvod računskog pomeranja i koeficijenta uvećanja optičkog sistema. U slučaju proračuna napona, razmera nije potrebna. Na Slici 4 su prikazana pomeranja grede OPC-200/350-3x8 pri opterećenju od 217kN, neposredno pre dostizanja granične nosivosti.



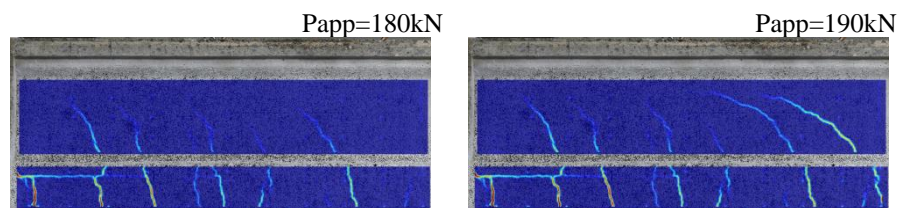
Slika 4: Vertikalno (levo) i horizontalno (desno) pomeranje bočne stranice grede OPC-200/350-3x8 pri opterećenju od 217kN

DIC metoda je stekla veliku popularnost kako zbog velike preciznosti merenja pomeranja, tako i zbog mogućnosti određivanja naponskog stanja na celokupnoj osmatranoj površini, što dalje pruža mogućnost eksperimentalnog merenja koncentracije napona i procesa plastifikacije elastičnog materijala. Prsline, kao zone diskontinuiteta, u suštini predstavljaju gubitak podataka i pri obradi fotografije bi ih trebalo ukloniti iz osmatrane površine. Međutim, u slučaju da je veličina diskontinuiteta manja od dimenzije računске grupe piksela, moguće je sračunati prosečnu vrednost napona na grupi. Ova mogućnost je u predmetnim ispitivanjima upotrebljena u cilju mapiranja prsline. Slika 5 prikazuje računске vrednosti glavnih napona zatezanja koje su sračunate na osnovu izmerenih pomeranja.



Slika 5: Prikaz formiranja prslina na gredi OPC-200/350-3x8 tokom nanošenja opterećenja

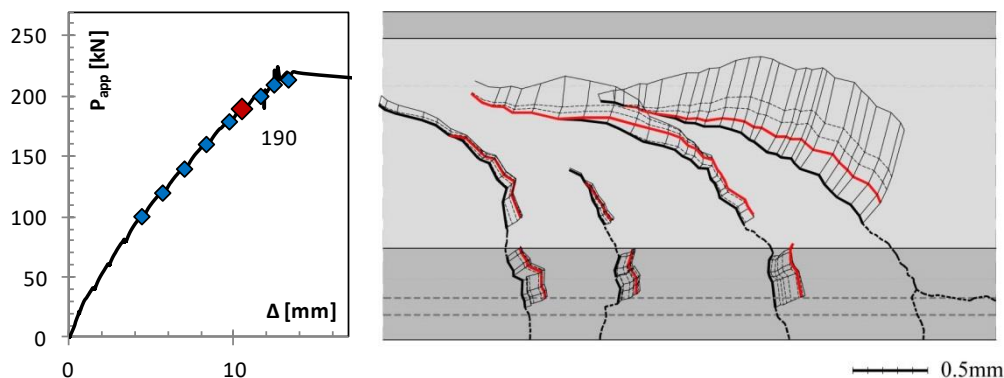
Postignuta preciznost merenja u iznosu od 0,01mm nije omogućila pouzdano određivanje dilatacija u neisprskalom betonu, ali je pružila zadovoljavajuću tačnost pri određivanju položaja prslina, procesa njihovog formiranja i relativnog pomeranja u svakoj tački prsline. Iz rezultata pomeranja, naknadnom obradom su sračunata pomeranja na prslinama usled promene opterećenja: u pravcu pružanja prsline (Δ) i u pravcu upravnom na pravac pružanja (ω) odnosno, klizanje i promena širine prsline. Za naknadu obradu podataka i proračun relativnog pomeranja na prslini napisano je softversko rešenje koje u osnovi sadrži algoritam čiji principi su dati u [13], gde je promena rastojanja između diskretnih mernih mesta određena mehaničkim deformetrom. Usvojeni algoritam proračuna je približan, sa greškom koja zavisi od relativne rotacije i koja je zanemarljiva za vrednosti izmerene u predmetnom ispitivanju.



Slika 6: Prikaz formiranja prslina na gredi 350-185/80-1 tokom nanošenja opterećenja

Primena merenja pomeranja u cilju mapiranja prslina pri ispitivanju armiranobetonskih elemenata je od naročito značaja pri beleženju procesa formiranja prslina neposredno pred lom, što je podatak koji se uobičajeno gubi usled nepoznavanja tačne vrednosti opterećenja pri kome će lom biti zabeležen, brzine procesa loma i neophodne bezbednosti lica koje merenje

obavlja. Sa druge strane, gotovo je nemoguće izmeriti promenu širine prslina neposredno pred lom, što je bio cilj pri eksperimentalnom određivanju granične nosivosti na smicanje grede 350-15/80-1 koja nije sadržala uzengije. Na slici 6 su prikazane slike prslina neposredno pre i nakon formiranja kritične smičuće prsline, a računске vrednosti pomeranja na obrazu prslina na slici 7. Sa sračunatim komponentalnim pomeranjima na obrazima formiranih prslina, moguće je odrediti i vrednost transverzalne sile koja se kroz prslinu prenosi trenjem.



Slika 7: Relativno pomeranje obraza prslina grede 350-185/80-1

5 ZAKLJUČAK

U ovom radu su prikazani principi merenja metodom zasnovanom na korelaciji digitalne fotografije. Osnovna prednost prikazane metode u odnosu na merenje pomeranja tradicionalnim instrumentima se ogleda u skraćanju vremena potrebnog za izvođenje eksperimenta i mogućnosti merenja pomeranja svih tačaka osmatrane površine. Primena metode pri ispitivanju armiranobetonskih konstrukcija se ogleda u činjenici da je moguće izmeriti širinu prslina bez potrebe poznavanja položaja na kome će prslina biti formirana. Na osnovu prikazane analize, doneti su sledeći zaključci:

- Moguća je uspešna implementacija 2D DIC metode merenja pomeranja na osmatranoj površini armiranobetonskih greda realnih dimenzija.
- Merenje pomeranja sa visokom preciznošću je moguće ostvariti i u slučaju velikih dimenzija posmatrane površine.
- Upotrebom komercijalnih DSLR kamera, odgovarajućom kalibracijom uređaja i ispravnim postavljanjem opreme, može se konstruisati niskobudžetni sistem za laboratorijsko merenje.
- Ovakav sistem merenja omogućava mapiranje prslina i precizno određivanje njihove širine na celokupnom osmatranom području.
- Jasna slika prslina formiranih na AB gredama se može dobiti kao slika maksimalnih glavnih napona zatezanja, bez narušavanja algoritma proračuna sa nastankom diskontinuiteta dimenzija manjih od dimenzije osmatrane grupe. Ovaj metod ukazuje na jasnu prednost u odnosu na ručno mapiranje prslina, umanjujući uticaj ljudske greške.

- Mapiranje, proces formiranja i određivanje pomeranja na prslinama se može jednostavno odrediti na celokupnom osmatranom području, za sve formirane prsline, u svim fazama opterećenja grede do loma.

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POŽARNA ANALIZA PROSTE ARMIRANOBETONSKE GREDE KORIŠĆENJEM ANSYS WORKBENCH

Rezime:

S-19

Evrokod standardi obezbeđuju osnovne smernice za analizu armiranobetonskih konstrukcija izloženih požaru. Za određivanje ponašanja konstrukcije, neophodna je primena naprednog metoda proračuna, kojim se obuhvataju nelinearna svojstva materijala i uticaji koji se javljaju u toku požara. Ovo podrazumeva termička i mehanička svojstva betona i čelika i vremensku termičku i mehaničku analizu. ANSYS Workbench je program baziran na metodi konačnih elemenata koji poseduje mogućnosti sprovođenja termo-naponske analize na osnovu definisanih materijala od strane korisnika. Primer proračuna ilustrovan je na slobodno oslonjenoj armiranobetonskoj gredi izloženoj standardnom požaru.

Ključne reči: požarna otpornost, prenos toplote, MKE modeliranje, ANSYS

FIRE ANALYSIS OF A SIMPLY SUPPORTED REINFORCED CONCRETE BEAM USING ANSYS WORKBENCH

Summary:

Eurocode normes provide basic guidelines for the analysis of reinforced concrete structures subjected to fire. To determine the behaviour of structure, an advanced calculation method must be applied, to comprehend nonlinear material properties and effects that occur during the course of fire. These include thermal and mechanical properties of concrete and steel and transient thermal and structural analysis. ANSYS Workbench is a finite element software capable of conducting thermal-stress analysis based on user defined material properties. An example of a simply supported reinforced concrete beam subjected to standard fire is presented.

Key words: fire resistance, heat transfer, FEM modelling, ANSYS

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1 INTRODUCTION

Problems that are recognized in the numerical analysis of reinforced concrete structures subjected to fire, using commercial finite element software ANSYS Workbench, include proper implementation of thermal and mechanical properties of both concrete and steel material at ambient and elevated temperatures, modelling of connection between steel reinforcement and the surrounding concrete, transitioning of temperature results from thermal to structural analysis and defining numerical parameters for nonlinear thermal and subsequent structural analysis.

Eurocodes EN 1992-1-2 [1] and EN 1993-1-2 [2] provide input parameters for constitutive materials, while thermal and mechanical actions for structural analysis in case of fire are presented in EN 1991-1-2. [3] Advanced calculation method should include transient heat transfer, taking into account conduction, convection and radiation effects, and a mechanical analysis that must comprehend for material degradation at elevated temperatures, which results in a highly nonlinear solution.

Numerical example of reinforced concrete simply supported beam subjected to standard ISO 834 fire is presented and the results are discussed.

2 MATERIAL PROPERTIES AT ELEVATED TEMPERATURES

With increasing temperature, concrete containing initial moisture experiences weight loss, due to evaporation of moisture and physical-chemical transformations. Specific heat of concrete, based on Eurocode, is dependent only on the initial moisture content, although the decomposition of calcium-hydroxide to calcium-oxide and transformation of quartz in some aggregates, have great influence on its values, which is recognized by the ASCE regulations. Thermal conductivity is mainly influenced by the type of aggregate, cement and moisture content in the element, and is presented by the upper and lower limit values, with former derived from steel-concrete compositions [4] and the latter corresponding to concretes made with siliceous aggregate.

Density of steel is temperature independent. The endothermic process of changing the crystalline structure of the carbon based steel at around 735°C, results in a sudden increase in form of a spike of specific heat in the range of 700-800°C. The thermal conductivity decreases gradually up until 800°C, followed by the constant value.

Thermal properties of concrete and steel based on EN 1992-1-2 and EN 1993-1-2 are presented in Figures 1-3.

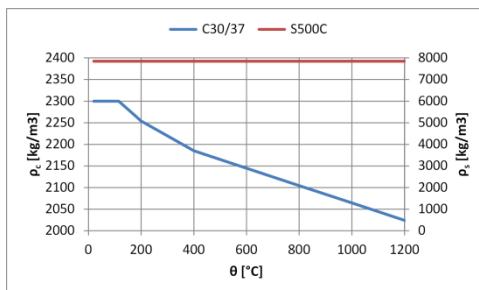


Fig 1. Density

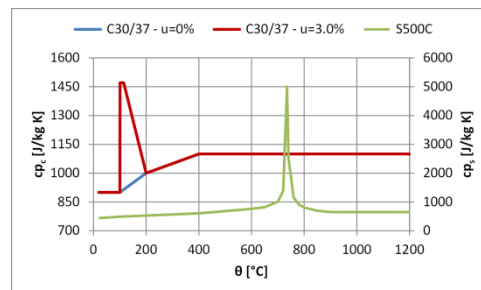


Fig 2. Specific heat

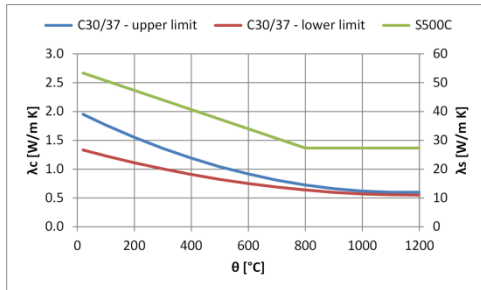


Fig 3. Thermal conductivity

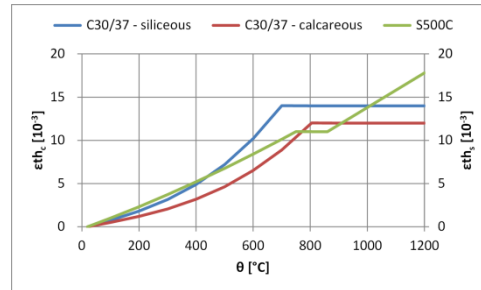


Fig 4. Thermal elongation

Coefficient of thermal expansion is temperature dependent, resulting in a nonlinear thermal expansion of both materials. According to [1], thermal expansion of concrete is defined for two types of aggregates, siliceous and calcareous. Thermal elongation is presented in Figure 4.

Increase of temperature causes drop in the slope of the stress-strain curve in the elastic range, for both concrete and steel. Reduction of strength is gradual for concrete, while steel reserves its strength up until 400°C, after which the strength decreases by 50% in the range of 400-600°C. Stress-strain curves are presented in Figure 5 and Figure 6.

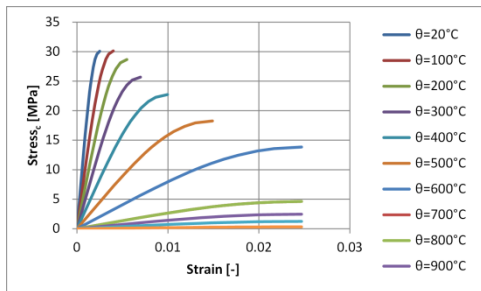


Fig 5. Stress-strain for concrete C30/37

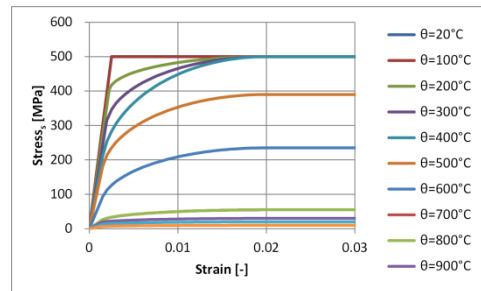


Fig 6. Stress-strain for steel S500C

3 NUMERICAL EXAMPLE

Commercial software ANSYS Workbench 16.0 [5] is used for the analysis of a simply supported reinforced concrete beam. The adopted length of a beam is 5 m, with a rectangular cross section of 30/60 cm. Due to the symmetry, only 1/4 of the beam is modelled to save the computational time. Self-weight and additional permanent load of 10 kN/m, as well as the imposed concentrated load of 50 kN at the middle of the beam span, are considered in the design. The static system and the adopted reinforcement of the beam are presented in Figure 7. The concrete part of the beam is modelled using 3D solid70 8-node brick element with a single degree of freedom at each node (temperature) for the thermal analysis, which is transformed to solid65 8-node element with 3 degrees of freedom (translations) for the structural analysis. Solid65 element is capable of cracking in tension in three orthogonal directions, and a plastic deformation in compression. Reinforcement is modelled as a uniaxial line element link33 with the ability to conduct heat between its nodes, which is replaced by the link180 element with an

isotropic hardening plasticity, for the structural analysis. Perfect bond is assumed between reinforcement and a surrounding concrete elements. Mesh element size is adopted as 1.25 cm for the thermal and 2.5 cm for the structural analysis.

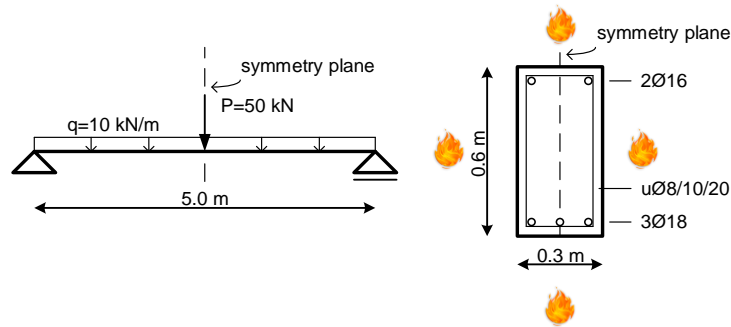


Fig 7. Static system and the adopted reinforcement

Thermal load (ISO 834 fire curve) in terms of convection and radiation is applied at each side of the beam, as described in [3]. Thermal response of the beam is calculated at every 30 seconds, and the results are later transferred as the thermal load for the structural analysis, divided in two stages. The first stage of the structural analysis considers external permanent and imposed load, after which, during the second stage, thermal load is applied. Due to the severe nonlinearity effects, time step used for the structural analysis is varied between 0.5 and 4 seconds.

4 RESULTS AND DISCUSSION

Temperature profiles are constant along the beam length. Thermal response of the arbitrary beam cross section is presented in Figure 8, while the reinforcement temperatures in time are given in Figure 9. Verification of the results is made comparing the profiles with the results provided in Annex A in [1].

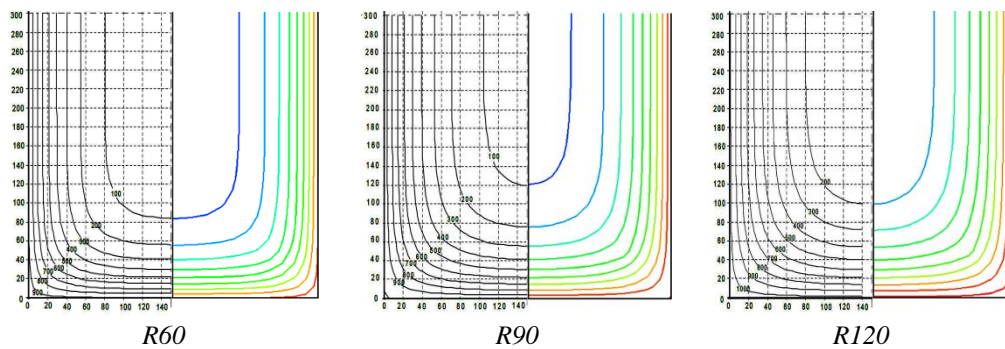


Fig 8. Temperature profiles after 30, 60 and 90 minutes (1/4 of the beam): EN 1992-1-2 (left) and ANSYS Workbench (right)

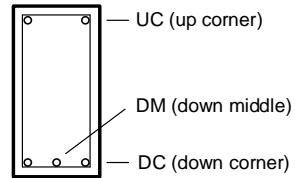
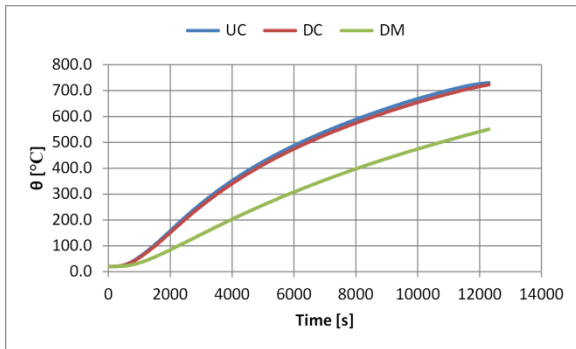


Fig 9. Temperature-time distribution in reinforcement bars

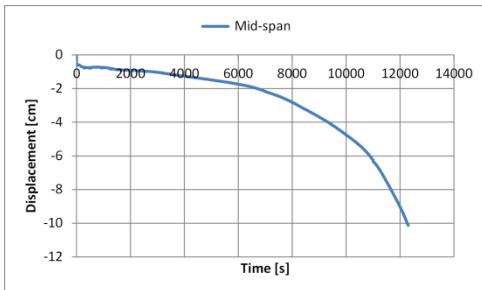


Fig 10. Mid-span vertical displacement

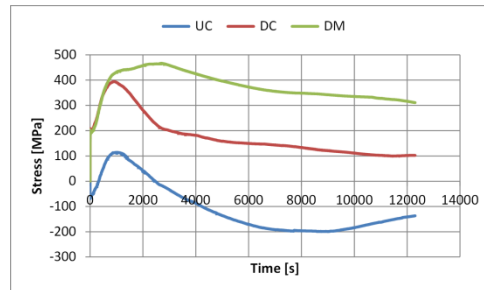


Fig 11. Stress-time distribution

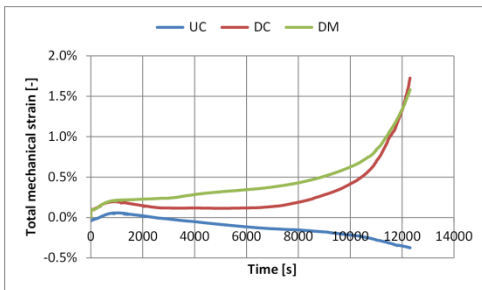


Fig 12. Total mech. strain-time distribution

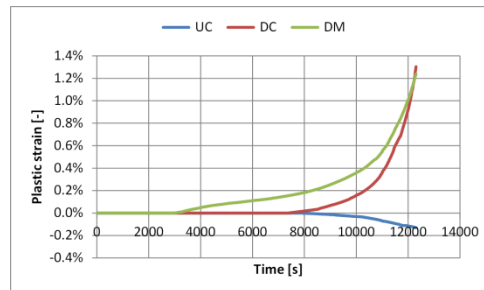


Fig 13. Plastic strain-time distribution

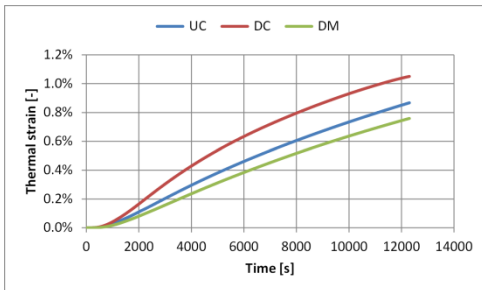


Fig 14. Thermal strain-time distribution

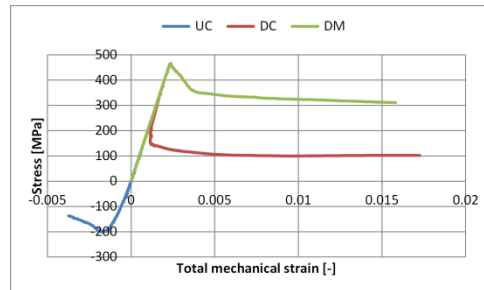


Fig 15. Stress-strain distribution

Temperatures in reinforcement bars at the corners of the cross section are considerably higher than in the bar at the middle of the cross section, due to combined heat transfer from both sides of the concrete corner surfaces exposed to fire.

Vertical displacement-time curve of the mid-span node is presented in Figure 10. Initial displacement is caused by the external forces acting on the beam prior to fire. Figures 11-15 present distribution of stress, total mechanical strain, plastic strain and thermal strain in time, and stress-strain diagram, respectively, of reinforcement bars at the middle of the beam. As the temperature inside of beam rises, degradation of mechanical properties starts, resulting in a stress reduction in corner bars and its redistribution to the middle bar (with a lower temperature). Due to concrete heating and cracking in tension, and overall stiffness reduction of the beam, the proportional limit in the middle bar is exceeded and plastic deformations occur at around 50 minutes of fire. At 120 minutes, plastic strain develops at corner bars, which eventually leads to the collapse of the beam.

5 CONCLUSION

Advanced calculation method of determining the fire resistance of reinforced concrete beam requires the application of powerful finite element software, capable of conducting both transient heat transfer and structural analysis. In this paper, results of such calculations are obtained using ANSYS Workbench platform. To overcome the numerical convergence problems, small time steps need to be applied during whole duration of fire, since cracking of concrete and plastic deformations appear at the very early stage of fire, after just few minutes. The proposed finite element based numerical model is capable of tracing the fire behaviour of reinforced concrete beams under standard fire conditions. Further research will be extended to reinforced concrete frame structures.

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PRERASPODJELA UTICAJA U KONTINUALNIM GREDAMA ARMIRANIM FRP ARMATUROM

Rezime:

S-20

U radu je prikazana osnovna problematika vezana za preraspodjelu uticaja kontinualnih greda sa opisom glavnih karakteristika FRP armature. Navedene su i osnovne razlike u ponašanju AB greda sa FRP armaturom u odnosu na grede sa čeličnom armaturom. Ukratko su prikazani rezultati dosadašnjih istraživanja u vezi preraspodjele uticaja u kontinualnim gredama sa FRP armaturom sa naznakom na parametre od kojih fenomen zavisi. Definisani su naučni ciljevi i pravci planiranih eksperimentalnih i numeričkih istraživanja koji će biti sprovedeni u okviru doktorske disertacije.

Ključne riječi: preraspodjela uticaja, FRP armatura, ciljevi istraživanja

REDISTRIBUTION OF INTERNAL FORCES IN CONTINUOUS BEAMS REINFORCED WITH FRP REINFORCEMENT

Summary:

The paper presents the basic issues related to the redistribution of internal forces in continuous beams with a description of the main characteristics of FRP reinforcement. The basic differences in the behavior of FRP concrete beams in relation to the beams with steel reinforcement are quoted. The results of previous research regarding the redistribution of internal forces in FRP continuous beams with a note on the parameters of which depends the phenomenon are briefly presented. Scientific objectives and directions of the planned experimental and numerical studies that will be conducted within the doctoral dissertation are defined.

Key words: redistribution of internal forces, FRP reinforcement, research objectives

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1 UVODNA PROBLEMATIKA O PRERASPODJELI UTICAJA

Uobičajena praksa je da se momenti savijanja i transverzalne sile u klasično armiranim betonskim konstrukcijama, za najnepovoljniju kombinaciju opterećenja, dobijaju linearno elastičnom analizom. Linearno elastično ponašanje klasično armiranih betonskih konstrukcija realizuje se samo pri veoma niskim nivoima spoljašnjih opterećenja. Nakon pojave prslina u AB elementima, dolazi do promjene njihove krutosti i razlike između stvarnih presječnih sila i onih dobijenih linearno elastičnom teorijom (stanje bez prslina), koja se posebno manifestuje razvojem plastičnih deformacija. Ova pojava je poznata kao preraspodjela statičkih uticaja, i za AB statički neodređene konstrukcije armirane čelikom može se podijeliti u dvije faze. Prva faza je uzrokovana razlikom uniformne fleksione krutosti duž elementa, koja je pretpostavljena elastičnom analizom, i stvarne krutosti koja se javlja varijacijom armature duž elementa i pojavom prslina u betonu. Preraspodjela uticaja koja je izvršena na ovaj način se često u literaturi naziva elastična preraspodjela. Druga faza je posljedica plastičnih deformacija u čeličnoj armaturi, tj. počinje nakon dostizanja granice razvlačenja u čeliku, i manifestuje se daljom promjenom vrijednosti fleksione krutosti. Ova preraspodjela se često naziva plastična preraspodjela. Dakle, presjeci koji trpe plastične deformacije će rotirati bez značajnijeg povećanja uticaja u njima, što omogućava presjecima sa nižim nivoima uticaja da prihvate dodatne uticaje. Treba napomenuti da uticaj elastične preraspodjele može imati značajan udio u ukupnoj preraspodjeli uticaja duž elementa. Može se, dakle, zaključiti da teorija elastične analize ne opisuje stvarno ponašanje ni za eksploatacioni nivo opterećenja, pa će se i značajna preraspodjela uticaja dogoditi već pri ovom nivou opterećenja.

Preraspodjela momenata je itekako korisna za svakodnevnu inženjersku praksu jer omogućava različite aranžmane armature u AB elementima konstrukcije. Koristi se kada je iz zona u kojima se očekuje veća količina armature (veze greda i stubova), poželjno izmjestiti momenat u zone u kojima se može smjestiti više armature (polje greda). Preraspodjelom momenata se obezbjeđuje unificiranje armature u serijama montažnih greda u kojima se javljaju manje razlike u momentima savijanja, i na taj način izbjegava različito armiranje svake grede posebno. Zatim, racionalnost, odnosno ekonomičnost, se može postići kada se preraspodjela momenata primjenjuje za različite kombinacije opterećenja, što rezultuje manjim vrijednostima u anvelopi momenata savijanja zadovoljavajući uslove ravnoteže. Takođe, preraspodjelom momenata se vrlo često obezbjeđuje poželjno duktilno ponašanje, sa jasno najavljenim upozorenjima prije loma. Ovo se prije svega pripisuje sposobnosti čelične armature da teče pri višim nivoima opterećenja. Preraspodjela momenata prije tečenja armature se pripisuje različitim krutostima u poprečnim presjecima duž grede, što kod kontinualnih greda konstantnog poprečnog presjeka uglavnom zavisi od procenta armiranja kritičnih zona.

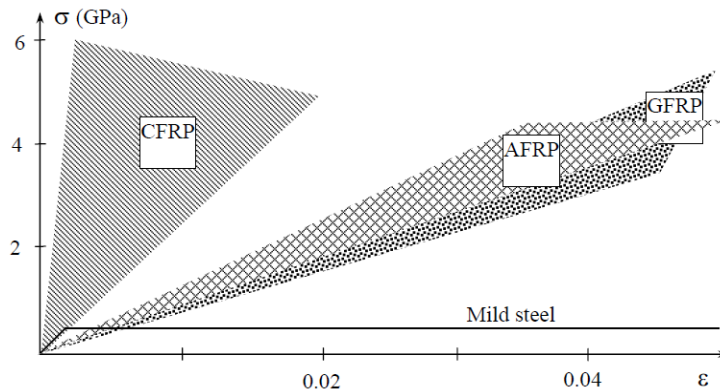
2 FRP ARMATURA

Za izgradnju građevinskih konstrukcija se danas još uvijek pretežno koristi beton armiran čeličnom armaturom. U agresivnim sredinama koje su izložene dejstvu vlage, temperature, hlorida, dolazi do redukcije alkalnosti betona, koja obično rezultuje korozijom čelične armature. Korozivni proces izaziva oštećenja betona i ugrožava funkcionalnost i upotrebljivost AB konstrukcija. Sprječavanje korozije čelične armature u AB konstrukcijama može biti skupo, a vrlo često i bez značajnijeg efekta. Iz ovog razloga se posljednjih 20 godina sve više radi na istraživanju materijala koji bi mogli zamijeniti čelik u AB konstrukcijama, posebno u

agresivnim sredinama. Tako, posljednjih godina u građevinskim konstrukcijama sve širu primjenu nalaze polimeri ojačani vlaknima (Fiber Reinforced Polymer - FRP), kao unutrašnja i spoljašnja armatura u AB elementima.

Visoka čvrstoća na zatezanje predstavlja jednu od osnovnih prednosti FRP armature u odnosu na čelik. Nisu podložni koroziji, a takođe pokazuju i potpunu električnu i magnetnu neutralnost. Lakši su od čelika, čime se može pojednostaviti njihov transport i dobiti na brzini izgradnje konstrukcije.

Pored navedenih prednosti, FRP armatura ima i određene nedostatke u odnosu na čeličnu armaturu. FRP kompoziti pokazuju linearno elastično ponašanje pri zatezanju sve do loma. Upoređujući ih sa duktilnim karakteristikama čelika, oni su krti, sa visokom čvrstoćom na zatezanje i uglavnom nižim modulom elastičnosti (slika 1). Čvrstoća na pritisak i na smicanje su znatno niži nego čvrstoća na zatezanje. Tokom eksperimentalnih istraživanja, evidentirani su i određeni problemi povezani sa prijanjanjem između FRP armature i okolnog betona.



Slika 1 – Radni dijagrami FRP i čelične armature

Kao glavni nedostatak FRP armature zapaža se njihov nizak modul elastičnosti u odnosu na čelik. Ovo podrazumijeva veće dilatacije armature u FRP AB elementima upoređujući ih sa AB elementima armiranim čelikom. Kao posljedica niskog modula elastičnosti u AB elementima se javljaju šire i dublje prsline, kao i veće deformacije. Dakle, u suprotnosti sa čeličnim AB elementima, granično stanje upotrebljivosti je vrlo često mjerodavno za dimenzionisanje elemenata sa FRP armaturom. Kod elemenata sa FRP armaturom mogu se tolerisati veće širine prslina zbog odsustva moguće korozije u AB elementu, dok se deformacije ograničavaju kao kod AB elemenata sa čeličnom armaturom.

FRP kompoziti, kao unutrašnja armatura u AB elementima, svoju primjenu nalaze u AB konstrukcijama koje su izložene agresivnom dejstvu sredine, kao što su: marinske konstrukcije, mostovi, nadvožnjaci, garaže, rezervoari, propusti, potporni zidovi, temelji i dr. U objektima sa opremom za magnetno skeniranje, bazama za velike motore, laboratorijama, aerodromskim tornjevima, MRI sobama u bolnicama, i ostalim objektima sa opremom koja zahtijeva elektro i magnetnu neutralnost, FRP armatura takođe nalazi svoju primjenu. Tokom posljednjih 20 godina postoje primjeri uspješne i praktične primjene FRP armature širom svijeta. Razlog što FRP armatura nije našla još širu primjenu, treba tražiti u još uvijek nepotpunom poznavanju ponašanja AB elemenata sa FRP armaturom.

3 PONAŠANJE AB GREDA ARMIRANIH FRP ARMATUROM

Ponašanje AB elemenata sa FRP armaturom je različito u odnosu na elemente armirane klasičnom čeličnom armaturom, što je posljedica, prije svega, različitih mehaničkih karakteristika dvije vrste armature. Tri tipa loma na savijanje se mogu javiti u AB elementima sa FRP armaturom: simultani (balans) lom po FRP armaturi i pritisnutom betonu, lom FRP armature i lom po betonu u pritisnutoj zoni. Usljed neduktilnog ponašanja FRP armature, svi tipovi loma su kruti i iznenadni, pa je iz tog razloga potrebno definisati veće koeficijente sigurnosti materijala prilikom dimenzionisanja poprečnih presjeka, nego što je to slučaj kod AB presjeka sa čelikom.

Simultani (balans) lom se u praksi veoma teško ostvaruje, jer ustvari predstavlja granicu između loma po armaturi i loma po betonu. Uslov balans loma je dostizanje granične dilatacije u pritisnutom betonu zajedno sa kidanjem FRP armature na zatezanje. Ukoliko je procenat armiranja AB presjeka FRP armaturom manji od procenta armiranja koji odgovara balans lomu, dolazi do loma po FRP armaturi. Lom koji nastaje usljed kidanja FRP armature je kruti i iznenadan, jer ne dolazi do dostizanja granične dilatacije u betonu, pa samim tim ni do kompletnog razvoja nelinearnog ponašanja betona. Međutim, ipak postoje određena ograničenja upozorenja pri lomu u vidu prslina i deformacija kao posljedica značajnih izduženja FRP armature prije loma. Sa druge strane, u slučaju većeg procenta armiranja u odnosu na procenat pri balans lomu, dolazi do dostizanja granične dilatacije u pritisnutom betonu i loma po betonu, koji može biti više poželjan, prije svega, zahvaljujući punom razvoju nelinearnog ponašanja (duktilnosti) betona. Kao takav lom po betonu može biti najavljen u vidu značajnih deformacija (ugiba) i prslina. Većina važećih propisa i smjernica zahtijeva da betonski presjeci sa FRP armaturom budu projektovani da dožive upravo lom po pritisnutom betonu, što, dakle, rezultira da oni budu prearmirani.

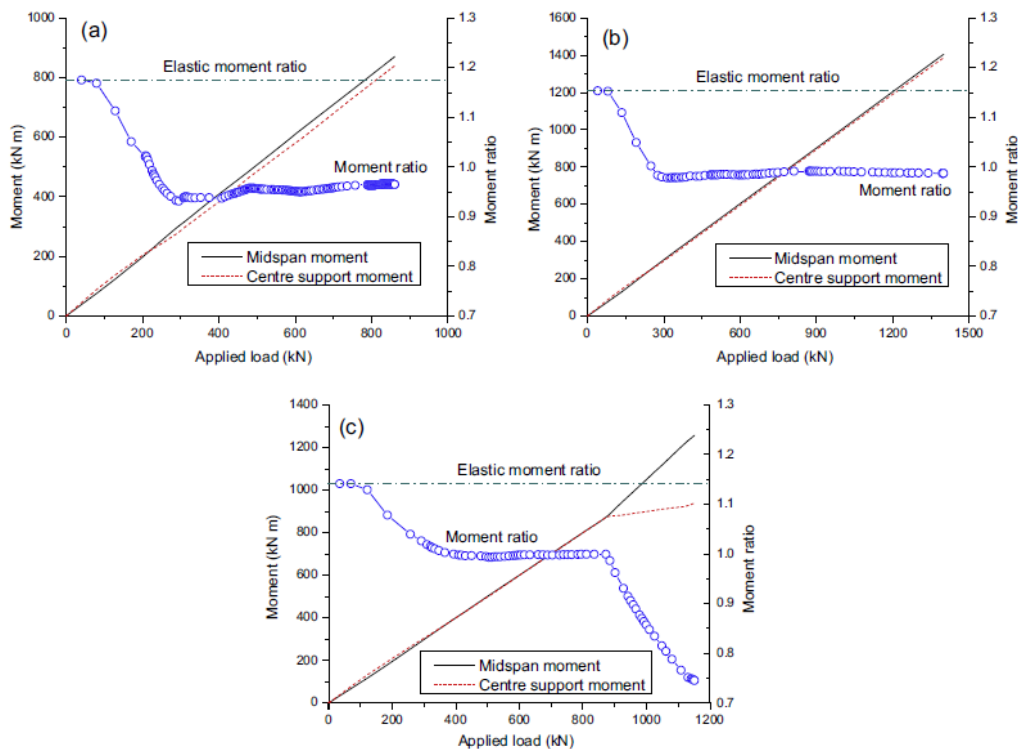
Evidentno je da su prearmirani presjeci sa FRP armaturom više poželjni u odnosu na podarmirane, zbog značajnih neelastičnih deformacija i formiranja prslina prije loma, usljed neelastičnih deformacija pritisnutog betona. Na ovaj način dolazi do poželjnih upozorenja prije loma, tj. do izvjesnog pseudo-duktilnog ponašanja. Pored toga, prearmirani presjeci vode ka značajnom smanjenju dimenzija presjeka, što je posebno važno sa arhitektonske i ekonomske tačke gledišta. Treba imati u vidu da je zadovoljenje zahtjeva upotrebljivosti u pogledu deformacija jako važno kod prearmiranih presjeka sa FRP armaturom.

4 PRERASPODJELA MOMENATA U KONTINUALNIM AB GREDEMA ARMIRANIM FRP ARMATUROM

U ovom dijelu rada je prikazana osnovna razlika u ponašanju kontinualnih AB greda sa FRP i čeličnom armaturom u uslovima preraspodjele momenata između kritičnih presjeka.

Preraspodjela momenata kod kontinualnih greda sa FRP armaturom zasniva se na elastičnoj preraspodjeli, s obzirom na odsustvo nelinearnosti (plastifikacije) FRP armature do loma. Razvoj momenata savijanja i odnos stvarnih momenata iznad oslonca i u polju na kontinualnim AB gredama sa odnosom armatura u polju i iznad oslonca od 1.5, u zavisnosti od apliciranog opterećenja za različite tipove opterećenja, prikazan je na slici 2, [2]. Na istoj slici je prikazan i odnos momenata iznad oslonca i u polju dobijen na osnovu elastične analize, radi poređenja rezultata. Za početna opterećenja momenti rastu linearno sa apliciranim opterećenjem i jednaki su elastičnim momentima, što ukazuje da ne dolazi do preraspodjele uticaja u ovoj fazi. Sa

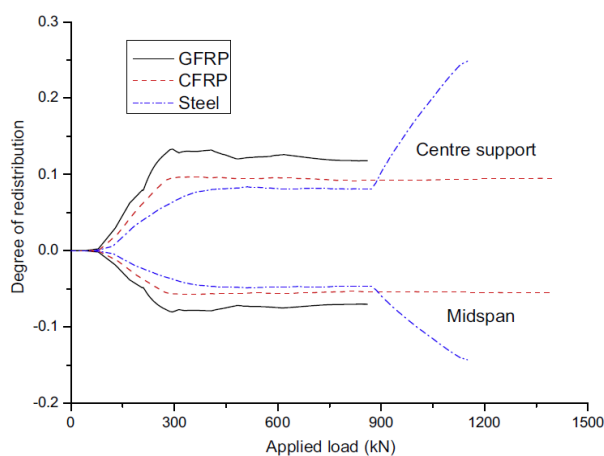
pojavom prve prslina iznad oslonca dolazi do preraspodjele momenata od oslonca ka polju, što zbog razlike u krutosti između kritičnih presjeka, rezultira bržem rastu momenta u polju i sporijem rastu momenta iznad oslonca. Ovo je posljedica usvojene veće količine armature za presjek u polju, nego za presjek iznad oslonca za sve grede, kao i različitog stepena razvoja prslina u kritičnim presjecima. Stabilizacijom propagacije prslina u oba presjeka, odnos momenata se takođe stabilizuje. Za grede sa FRP armaturom, ovaj fenomen ostaje gotovo konstantan sve do loma (slike 2a - GFRP-staklena armatura i 2b - CFRP-karbonska armatura), dok za grede sa čeličnom armaturom tečenje armature iznad oslonca uzrokuje naglo izmještanje momenta iznad oslonca u polje grede, odnosno naglo smanjenje odnosa momenta iznad oslonca i u polju. Sa slike 2 je evidentno da već i pri eksploatacionim opterećenjima dolazi do znatnog odstupanja vrijednosti momenata (preraspodjele momenata) duž kontinualne grede u odnosu na momente dobijene elastičnom analizom.



Slika 2 – Razvoj momenata savijanja i odnosa momenata iznad oslonca i u polju sa apliciranim opterećenjem za različite tipove armature a) GFRP; b) CFRP; c) čelik; [2]

Na slici 3 je prikazana varijacija stepena preraspodjele momenata sa apliciranim opterećenjem do loma za kontinualne grede sa različitim tipovima armature. Za kontinualne grede sa FRP armaturom, stepen preraspodjele momenata se sastoji iz tri različite faze, sa dvije skretne tačke koje odgovaraju pojavi prslina i stabilizaciji propagacije prslina. U prvoj fazi, prije pojave prslina, nema preraspodjele momenata u odnosu na elastičnu analizu. Nakon

pojave prslina, stepen preraspodjele naglo raste u drugoj fazi, gotovo linearno sa apliciranim opterećenjem. U trećoj fazi stepen preraspodjele momenata se stabilizuje sa gotovo konstantnom vrijednošću, što odgovara stabilizaciji propagacije prslina. Kod kontinualnih greda sa čeličnom armaturom javlja se i četvrta faza, kada usljed tečenja armature dolazi do naglog povećanja stepena preraspodjele za aplicirano opterećenje. Primjetno je da je za opterećenja niža od loma, stepen preraspodjele kod greda armiranih GFRP armaturom veći nego kod greda armiranih čelikom, i to već nakon pojave prve prslina. Ovakav trend razvoja stepena preraspodjele momenata se nastavlja sve do tečenja armature kod greda sa čeličnom armaturom. Sa slike 3 se jasno primjećuje da se preraspodjela uticaja kod kontinualnih greda sa GFRP armaturom, već i u fazi eksploatacije javlja u značajnom stepenu, [2].



Slika 3 – Razvoj stepena preraspodjele momenata sa apliciranim opterećenjem za različite tipove armature, [2]

5 REZULTATI DOSADAŠNJIH ISTRAŽIVANJA

Na osnovu skromnog broja dosadašnjih eksperimentalnih i numeričkih istraživanja došlo se do određenih zaključaka koja predstavljaju osnovnu hipotezu za planirana dalja istraživanja:

- kontinualne AB grede sa FRP armaturom pokazuju značajna upozorenja, u vidu velikih deformacija i širokih i dubokih prslina, prije loma;
- povećanje armature u donjoj zoni u polju (prearmirani presjeci) u odnosu na presjek iznad oslonca, ima pozitivne efekte na povećanje kapaciteta nosivosti greda, smanjenje deformacija i odlaganje propagacije prslina u poljima greda;
- povećanje armature u gornjoj zoni iznad oslonca, nema značajniji doprinos na povećanje kapaciteta nosivosti greda ili na smanjenje deformacija;
- kontinualne grede sa FRP armaturom imaju sposobnost preraspodjele momenata, u odnosu na momente dobijene linearno elastičnom analizom, nakon pojave prslina u betonu;
- nakon dostizanja kapaciteta nosivosti, u polju ili iznad oslonca kontinualnih greda, nema znakova preraspodjele momenata u kritičnim presjecima;

- obezbjeđenje preraspodjele momenata iz oslonca u polje, ima pozitivan efekat na smanjenje deformacija i povećanje kapaciteta nosivosti greda;
- odnos armature u polju (donja zona) i iznad oslonca (gornja zona) ima glavni uticaj na raspoloživu preraspodjelu momenata;
- procenat armiranja kritičnih presjeka može značajno uticati na vrijednosti preraspodjele momenata;
- dodatno utezanje betona, povećanjem poprečne armature iznad oslonca, bez povećanja podužne zategnute armature, povećava kapacitet nosivosti greda;
- značajan procenat preraspodjele momenata ostvaruje se već i pri eksploatacionim opterećenjima;
- preraspodjela momenata može omogućiti dostizanje većeg kapaciteta nosivosti na smicanje kontinualnih greda sa FRP armaturom.

Treba napomenuti da su pojedini zaključci izvedeni na osnovu samo jednog eksperimentalnog istraživanja, te stoga treba provjeriti njihovu opravdanost. Može se zaključiti da je trenutno vrlo malo pouzdanih podataka koji opisuju ponašanje kontinualnih greda armiranih FRP armaturom, pa je stoga potrebno nastaviti istraživački rad, kako eksperimentalni, tako i numerički. Ponašanje AB elemenata sa FRP armaturom u pogledu preraspodjele momenata još uvijek nije dovoljno istraženo, prije svega jer zavisi od velikog broja parametara. Dosadašnja istraživanja su pokazala da pristup da se kod kontinualnih greda armiranih FRP armaturom ne dozvoljava preraspodjela momenata u kritičnim presjecima, može smatrati konzervativnim, pa je isti potrebno preispitati.

6 NAUČNI CILJEVI, PRAVCI DALJIH ISTRAŽIVANJA I OČEKIVANI REZULTATI

Glavni cilj daljih istraživanja je razmatranje ponašanja kontinualnih greda armiranih FRP armaturom pri opterećenju do loma, u uslovima preraspodjele momenata između kritičnih presjeka. Takođe, istraživanjem se žele postići sljedeći naučni ciljevi:

- dati doprinos u sveukupnom razumjevanju ponašanja kontinualnih greda sa FRP armaturom, kao i doprinos u projektovanju ovih konstrukcija;
- definisati uticaj velikog broja parametara koji utiču na preraspodjelu uticaja u kontinualnim gredama sa FRP armaturom kao što su: tip loma, odnos količine armature u polju i iznad oslonca, procenat armiranja podužnom armaturom, količina poprečne armature u kritičnim presjecima, čvrstoća betona;
- pokazati da se pravilnim izborom armature u polju i iznad oslonca kontinualnih greda, obezbjeđuje odgovarajuća preraspodjela uticaja koja može dovesti do poboljšanog ponašanja kontinualne grede sa FRP armaturom, u smislu povećanja kapaciteta nosivosti i zadovoljenja zahtjeva deformacija i prslina;
- proširiti bazu podataka novim eksperimentalnim istraživanjima iz ove oblasti u cilju verifikacije tačnosti i unaprjeđenja zaključaka dosadašnjih eksperimentalnih istraživanja;
- definisati stepen dozvoljene preraspodjele, koja aktuelnim propisima za kontinualne grede armirane FRP armaturom nije dozvoljena;
- dati doprinos na unaprjeđenju smjernica i odredbi standarda i propisa, koji se primjenjuju pri projektovanju u svakodnevnoj inženjerskoj praksi, u oblasti statički neodređenih AB konstrukcija armiranih FRP armaturom.

Definisanjem ciljeva istraživanja, postavljanjem osnovnih hipoteza i definisanjem osnovnih parametara od kojih zavisi preraspodjela uticaja kontinualnih greda armiranih FRP armaturom, određiće se reprezentativni modeli (uzorci) na kojima će biti sprovedeno eksperimentalno ispitivanje. Eksperimentalno istraživanje će biti koncipirano i osmišljeno na način da dobijeni rezultati budu upotrebljivi i uporedivi sa rezultatima sprovedenih dosadašnjih istraživanja iz predmetne problematike širom svijeta. Cjelokupno istraživanje obuhvatiće:

- Definisanje parametara koji utiču na ponašanje kontinualnih AB greda armiranih FRP armaturom, sa posebnim akcentom na one parametre od kojih direktno zavisi stepen preraspodjele momenata.
- Eksperimentalnu analizu ponašanja kontinualnih greda armiranih FRP armaturom. Eksperimentalno istraživanje će biti sprovedeno u cilju definisanja mogućnosti i stepena preraspodjele uticaja, kao i uticaja već navedenih parametara na stepen preraspodjele uticaja i efekat preraspodjele na granična stanja nosivosti i upotrebljivosti.
- Numeričko modeliranje eksperimentalnih uzoraka primjenom metode konačnih elemenata radi simulacije ponašanja kontinualnih greda sa FRP armaturom. Modeli treba da verifikuju pojedine uticaje parametara koji su dobijeni eksperimentalnim putem.
- Analizu predloženih metoda proračuna kontinualnih greda armiranih FRP armaturom na savijanje i smicanje i analizu uticaja pojedinih parametara na stepen preraspodjele momenata.
- Parametarsku analizu eksperimentalnih i numeričkih rezultata i uticaj preraspodjele na granična stanja nosivosti i upotrebljivosti.
- Izradu proračunskog modela za određivanje dozvoljene preraspodjele uticaja u kontinualnim gredama sa FRP armaturom u cilju dorade aktuelnih propisa iz ove oblasti.

Planiranim istraživanjem trebalo bi da se pokaže da je preraspodjela uticaja kod statički neodređenih konstrukcija sa FRP armaturom moguća, bez obzira na linearno elastično ponašanje FRP armature sve do loma, a posebno ako se armatura duž grede izabere odgovarajuće. Takođe, ukazaće se na veći značaj preraspodjele uticaja kod greda sa FRP armaturom, nego što je to slučaj kod kontinualnih greda armiranih čeličnom armaturom, koji se posebno odnosi na kvalitetniji odgovor konstrukcije pri dejstvu spoljašnjeg opterećenja. Rezultati bi trebalo da pokažu na neophodnost uvođenja novih parametara, u odnosu na trenutno definisane propisima, u cilju što tačnijeg određivanja stepena preraspodjele.

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VEZA ČELIČNIH ELEMENATA ZAVRTNJEVIMA: KLASIČNO REŠENJE I KONTAKTNA MKE ANALIZA

Rezime:

S-21

Veze zavrtnjevima su nezaobilazan deo čeličnih konstrukcija. Klasičan metod njihovog proračuna i dimenzionisanja po metodi dopuštenih napona prepoznaje ali i zanemaruje koncentracije napona koje se javljaju na dodiru vijka i omotača rupe zavrtnja. Numeričke metode zasnovane na metodi konačnih elemenata (MKE) i primeni moćnih inženjerskih softvera omogućavaju analizu uz materijalnu nelinearnost i simulaciju kontaktnih fenomena. Na primeru jednostavne veze sa jednim zavrtnjem pokazane su razlike i prednosti između klasičnog i savremenog pristupa ovom problemu.

Ključne reči: čelične konstrukcije, veza zavrtnjevima, MKE, kontaktna analiza

BOLTED CONNECTION OF STEEL ELEMENTS: CLASSIC SOLUTION AND CONTACT FEM ANALYSIS

Summary:

Bolted connections are unavoidable part of the steel structures. Classical method of their calculation and dimensioning by the allowable stress method recognizes, but also neglects stress concentrations that occur at the contact of the bolt and the bolt hole. Numerical methods based on the finite element method (FEM) and applying of powerful engineering software enables analysis with material nonlinearity and simulation of contact phenomena. On the example of a simple single-bolted connection are presented differences and advantages between the classical and contemporary approach to this problem.

Key words: steel structures, bolted connection, FEM, contact analysis

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1 INTRODUCTION

Connections in steel structures are nowadays predominantly realized using bolts or welding. The basic difference between those two methods is in the way of force transfer. In welding, a continuity of force transfer is achieved, while in bolted connections forces are transferred discontinuously, i.e., in spots where bolts are placed.

In shear connections, according to theory, shear force is transferred by shear of the bolt shaft and by bolt bearing. In Fig. 1a) a bolted connection of three elements in shear is presented. Due to the forces in the elements and due to corresponding deformation, bending of the bolt shaft occurs, and consequently a stress concentration on bolt holes is significant (Fig. 1 b), while the shear stresses are maximal in the contact planes of the elements because of the change of the force direction (Fig. 1 c). According to the Allowable Stress Design (ASD) method, stress concentrations are ignored, and stress distribution in the bolt hole is assumed to be uniform.

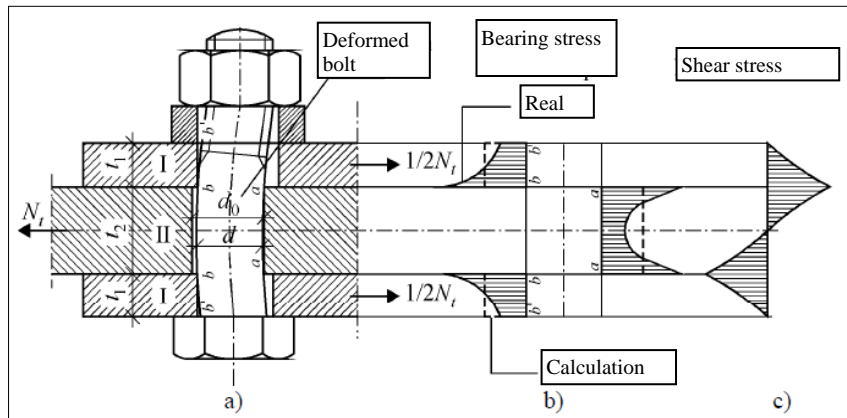


Figure 1 – Stress distribution in bolted connection [1]

The adopted assumptions are introduced for the purpose of simplification, considering that it is almost impossible to encompass all the real phenomena using classic „manual“ calculation methods. For a more real determination of stresses and deformations much more complex analysis methods are necessary. Some of those methods are experimental methods, photo-elasticity, and a few numerical methods, among them the Finite element method (FEM), as the most used one recently. Using an example of a single-bolted connection we will show possibilities and limitations of the classical vs. FEM analysis.

2 CLASSIC ANALYSIS OF A BOLTED CONNECTION

In the Fig. 2 is presented an example of one single-bolted connection often used in crane structures [2]. All dimensions are in mm, base steel material is S235, and the bolt grade is 8.8. The middle thimble (POS 4) is loaded by concentrated downward force $P=85$ kN. Load case, by ASD, is set to 1.

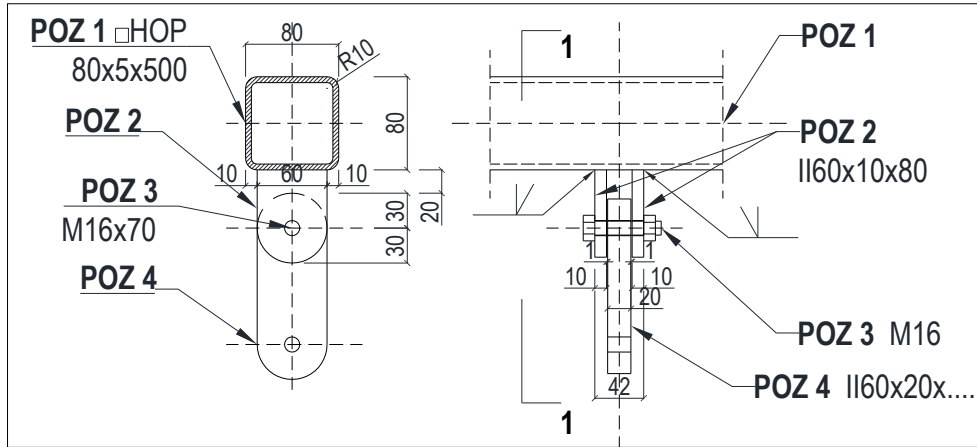


Figure 2 – Analysed example of a bolted connection

For the proposed dimensions of all elements in the connection, the stress results are given in Table 1. As one may see, all the values are under the permitted, with good effectiveness, besides of the contact pressure, which is slightly exceeded. Here it must be noted that contact pressure calculation is not a standard part of the connection calculation.

Table 1 – Stress values [MPa] in connection elements by classic analysis

POS	Element	$\sigma_{\text{allowable}} = 160$	$\tau_{\text{allowable}} = 224$	$\sigma_{b, \text{allowable}} = 270$	$\sigma_{k, \text{allowable}} = 720$
2	Left/right thimble	136.3	-	265.6	761
4	Middle thimble	145.3	-	265.6	761
3	Bolt	-	211.4	-	-

3 FEM ANALYSIS OF THE PROPOSED BOLTED CONNECTION

3.1 MODELLING OF THE STRUCTURE

Modelling of a structure in FEM means the creation of a numerical model which will in the most approximate way describe its geometry and behaviour under load. The process of modelling is developing through several phases: defining of the materials used and their characteristics, creating of geometry, setting of appropriate type(s) of finite elements, meshing, setting of boundary conditions and loads, setting of analysis parameters. A special part of the modelling in this case is defining of the contact regions where the transfer of the forces occurs from the bolt shaft to the bolt hole. Modelling and analysis was done in the ANSYS WORKBENCH software [3].

First, the analytical models of σ - ε dependence for the base and bolt material were defined. A bilinear diagram was chosen, with the modulus of elasticity $E=210$ GPa, tangent modulus $E_T=0.01E$, and yield point $f_y=235$ MPa (base material), $f_y=640$ MPa (bolts) (Fig. 3a, b).

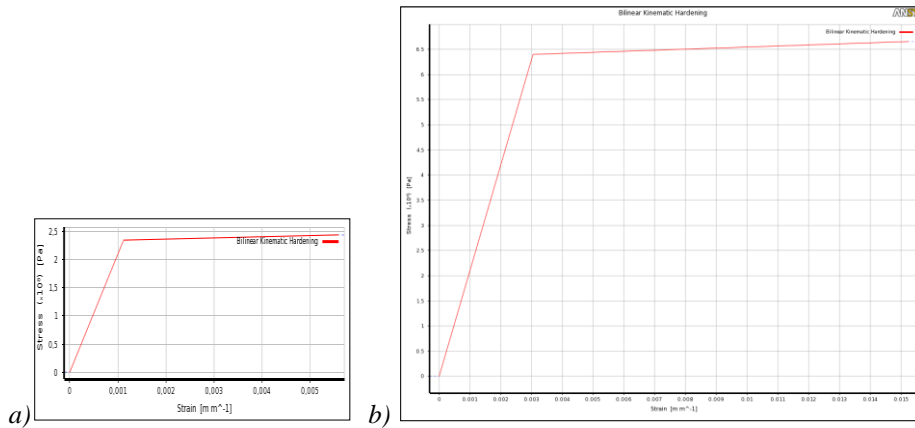


Figure 3 – Working diagram for steel material a) base material; b) bolt

Geometry of the model consists of 4 separate bodies (Fig. 4a): left thimble, right thimble, bolt, middle thimble, and fictive body „load“ (semi-cylinder used only for load application).

The next step is defining of the contact regions between bodies. Contact behaviour is a special case of nonlinearity which assumes that two bodies can experience compression stress, but alternatively, there are no tension stresses, but separation of the bodies. Contact can be in a point, along line, or over surface. In this case, four contact pairs were introduced: bolt-right thimble, bolt-left thimble, bolt-middle thimble, and middle thimble-load. All contact pairs included friction, and they consisted of one passive surface meshed with TARGET finite elements, and one active surface meshed with CONTACT finite elements. Value of friction coefficient was adopted as 0.2.

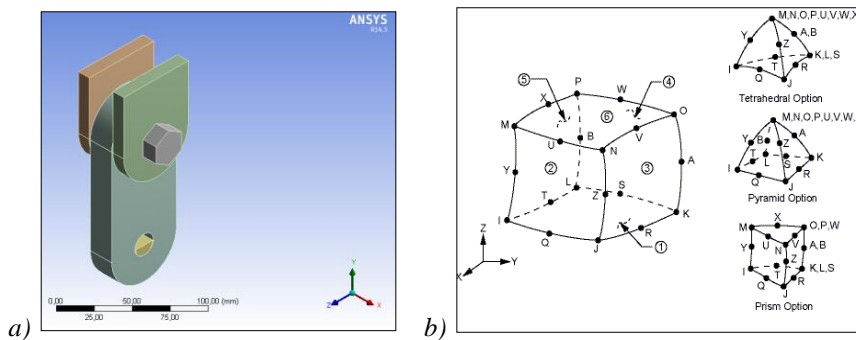


Figure 4 – a) Geometry of the model; b) SOLID186 finite element

Meshing of the structural elements was done using 3D finite elements, named SOLID186. Element has 20 nodes and three degrees of freedom per node (translations in x-, y-, and z-direction, (Fig. 4b). Degeneration of the basic shape (hexahedron) produces shapes like tetrahedron, pyramid, or three-sided prism. Such elements are suitable for describing of complex geometry shapes.

This structure was meshed using tetrahedral elements, with local densing in the contact regions. Analysis was done for two different mesh densities. The first model had 59599 nodes and 38811 elements, with max. element size of 10 mm (Fig. 5a). The second model had 70273 nodes and 45866 elements, with max. element size of od 8 mm (Fig. 5b).

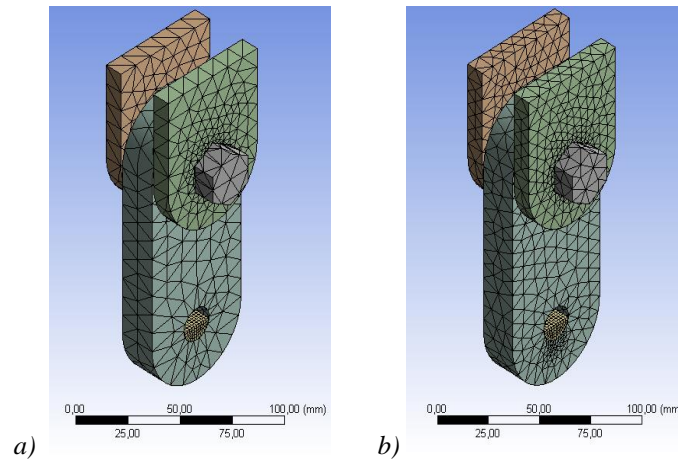


Figure 5 – FE mesh; a) coarse; b) fine

Boundary conditions were set on the top surfaces of the left and right thimble, simulating a welded, i.e., rigid connection of the thimbles to the rest of the structure. In fact, displacement of the nodes on those surfaces was prevented in all directions (Fig. 6a). Also, the nodes on the bottom line (symmetry line) of the middle thimble was prevented to move in z-direction (Fig. 6b). This prevents pendulum behaviour of the structure, but enables dilatations in x- and y-direction, and also in z-direction, since it is located on the symmetry plane.

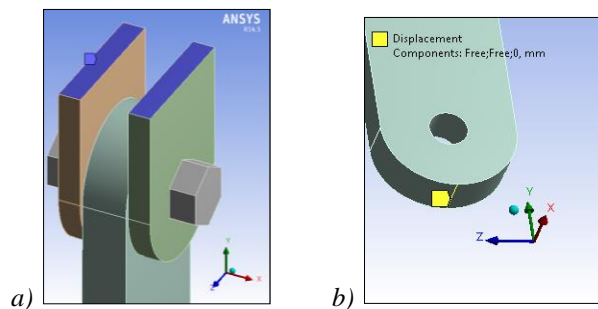


Figure 6 – Boundary conditions; a) on the left and right thimble; b) on the middle thimble

The load was modelled as force with intensity of 85 kN distributed on the top surface of the semi-cylinder which is here used only to apply the load, and to avoid any load concentration.

The problem was solved as statical, with material and contact nonlinearity, but without geometrical nonlinearity, i.e., with small deformations, considering the class of the problem.

The load was applied incrementally, with a minimum of 20, and maximum of 100 substeps. Solution convergence was achieved after 22 substeps.

3.2 ANALYSIS OF RESULTS

Fig. 7 presents resulting deformation of the whole structure. Max. deformation value is low, and occurs in the „load“ part which is not of interest here. However, in Fig. 7b one may see the form of structure deformation, i.e., bending of the bolt shaft and of the left and right thimble. This is in relatively good agreement with the classical postulates about such connections (Fig. 1), but here is also visible the bending of the thimble.

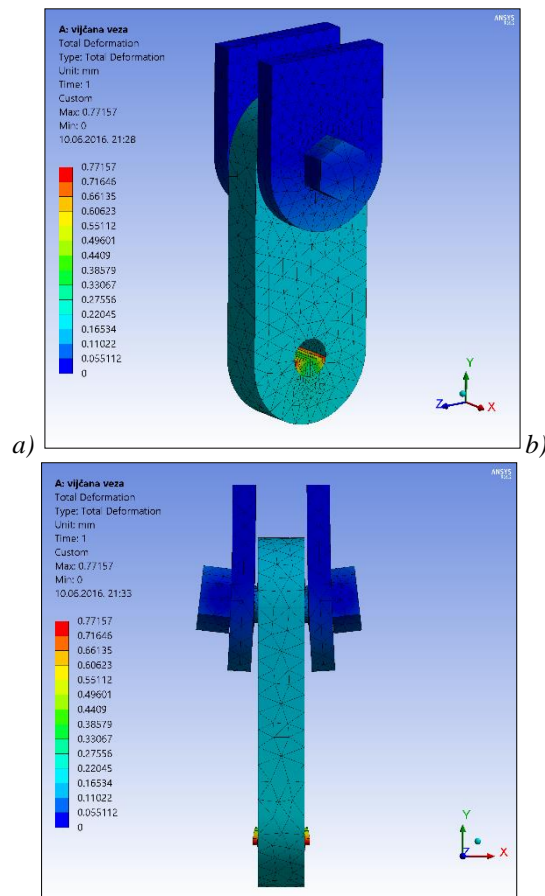


Figure 7 – Total deformation of the structure, max. 0.77 mm; a) isometric; b) front view

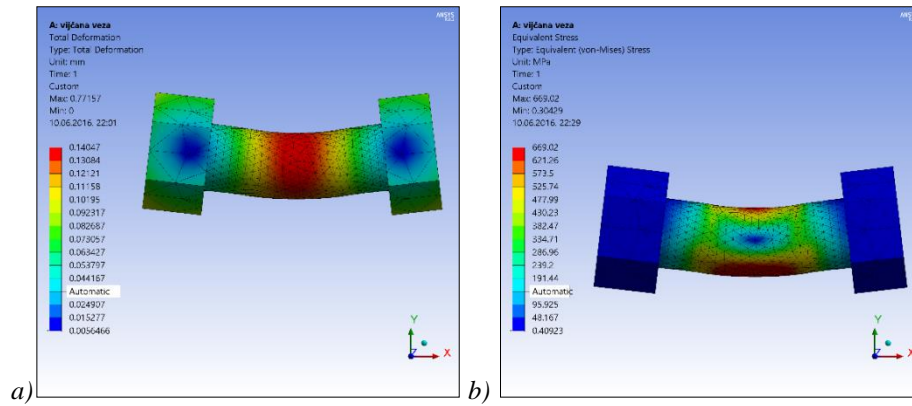


Figure 8 – Bolt. a) Total deformation, max. 0.14 mm; b) Von Mises stress, max. 669 MPa

The deformation form of the bolt clearly shows that both shear and bending are present (Fig. 8a). Max. Von Mises stress (668 MPa) slightly exceeds the yield point for the bolt material (640 MPa), (Fig. 8b).

The middle thimble underwent max. Von Mises stress of 261 MPa vs. the yield point of 235 MPa (Fig. 9a). An interesting phenomenon is shown in the Fig. 9b, where the inner surface of the hole is kneaded downward and outward (deformation magnified). This indicates the potential fracture mechanism.

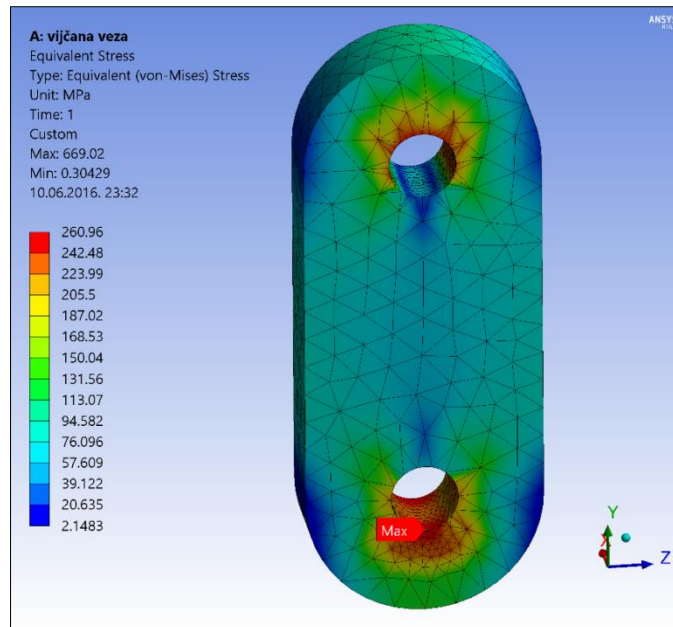


Fig. 9a – Middle thimble; Von Mises stress, max. 261 MPa

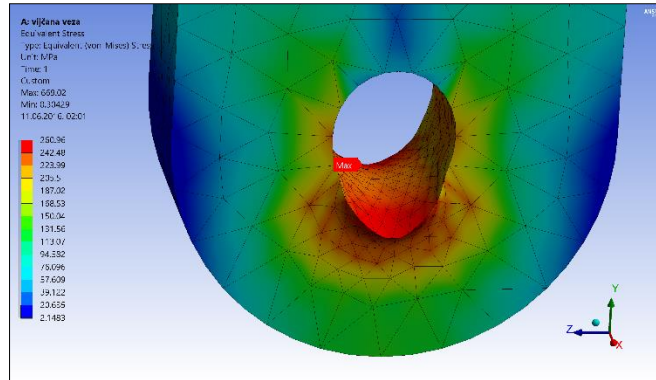


Fig. 9b – Middle thimble; total deformation, max. 0.28 mm

Contact analysis enables detecting of the phenomena occurring at the very touch between two bodies. On the Fig. 10a, b and 11b are presented contour values of the contact pressures. Those values correspond to the *Herz's* pressures in classic analysis, and here they are all below allowed values [4]. Stress patterns for the holes on the middle thimble were symmetrical, while on the right thimble they are unsymmetrical, which is as expected considering the structure assembly.

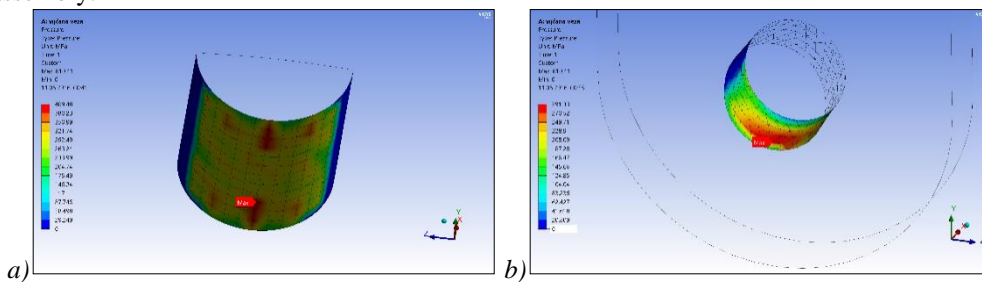


Fig. 10 – Middle thimble, contact pressures;
a) bottom hole, max. 409 MPa; b) upper hole, max. 517 MPa

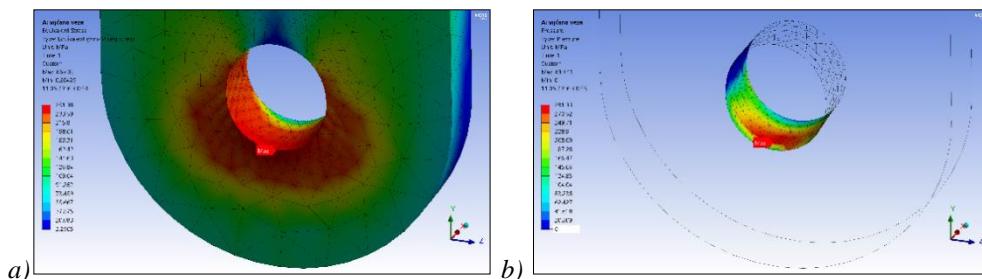


Fig. 11 – Right thimble; a) Mises stress, max. 251 MPa; b) contact pressure, max. 291 MPa

As mentioned before, plasticization occurred in all three main parts of the connection. Locations of plasticized zones are different for every part, and they reveal the dominant influence. Namely, the bolt shaft is plasticized at the bottom, proving that bending, and not shear is dominant (Fig. 12).

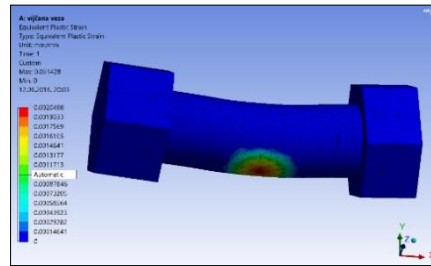


Figure 12 – Plasticization zones in the bolt shaft

On the other hand, both thimbles underwent plasticization in the regions of contact (Fig. 13a, b), meaning that pressure is the main influence here.

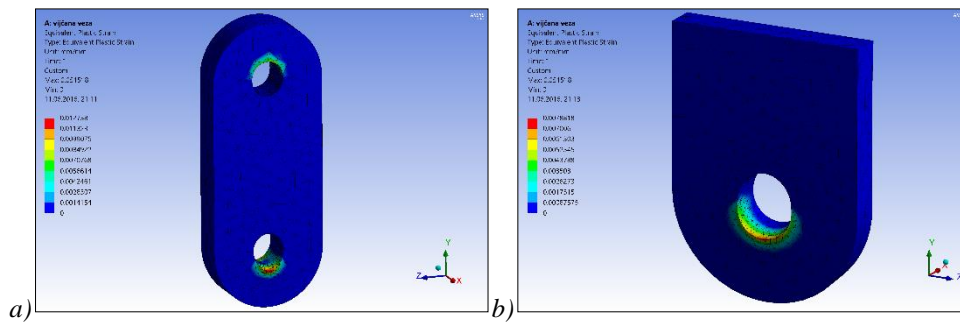


Figure 13 – Plasticization zones; a) middle thimble; b) right thimble

4 CONCLUSIONS

In the Table 2 are shortly presented extreme values of the specific results of the analysis of the given problem, performed using classical and FEM methods. The FEM analysis encompassed material nonlinearity and contact behaviour.

Table 2 – Comparison of stress values [MPa]: classic vs. FEM analysis

POS	Element	σ_{cal}	σ_{FEM}	$\Delta\%$	$\sigma_{b,cal}$	$\sigma_{b,FEM}$	$\Delta\%$	$\sigma_{k,cal}$	$\sigma_{k,FEM}$	$\Delta\%$
2	Left/right thimble	136.3	251	+84	265.6	251	-5	761	291	-62
4	Middle thimble	145.3	261	+80	265.6	261	-2	761	517	-32
3	Bolt	-	669	-	-	-	-	-	-	-

Based on the table values we can say that generally stresses obtained by FEM are much higher (+84% and +80%). Reason for this lies in the fact that FEM analysis includes stress

concentrations near holes, and classical analysis not. Bearing stresses are in very good agreement, with remark that FEM does not separate bearing stress and general structural stress. The contact pressures (σ_k) give the most interesting facts: Values obtained by FEM are significantly lower (-62% and -32%). Explanation for this is that plasticization in the FEM analysis has the effect of “adjusting” of the parts in contact which increases the contact surfaces and decreases the pressure. This confirms that plasticization in connections is present very often, and when in limited regions, like here, it can be useful [5].

Finally, the following conclusions are developed:

- difference in mesh density of two analysed models did not give significant differences in results; all differences were within the range less than 5%, which is acceptable for this method of analysis; since the model with the denser mesh gave slightly greater result values, it was used as referent;

- the most part of the analysed connection structure showed stress values below the allowed ones; exceptions are perimeters of the holes, and certain parts of the bolt, where stress concentrations cause exceeding of the allowed stress, and even plasticization;

- when the given connection is analysed using analytical ASD method, it satisfies all required conditions, with little exception regarding contact pressures calculated according to *Herz*;

- direct comparison of the analytical and numerical method is hardly possible, because they are on qualitatively different levels; although classical theory recognizes stress concentrations and bending of the bolt (Fig. 1), it deliberately ignores it and simplifies the problem because of its over-complexity for the manual calculations;

- numerical FEM analysis gives us a full 3D presentation of the behaviour in the connection, particularly in the aspect of contact.

This comparative analysis gave some answers, but also posed some questions, and the most important among them is, what results should be governing for the dimensioning of such structure. Implementation of the Limit states method, applied in most of the contemporary codes gives a partial, but not a complete answer, again due to the complexity of the problem.

ACKNOWLEDGEMENTS

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MASE ČELIKA ČELIČNIH I SPREGNUTIH DRUMSKIH MOSTOVA

Rezime:

S-22

Cilj svakog investitora i projektanta je da se izgradi što jevtiniji most. Količina čeličnog materijala za konstrukciju je vrlo važan deo ukupnih troškova izgradnje. U članku je predstavljena procena masa čelika za konstrukcije čeličnih i spregnutih drumskih mostova, prema tri izvora literature Teoretske prosečne specifične mase čelika – izražene kao kg/m^2 osnove mosta – date su kao funkcije raspona i za razne statičke sisteme konstrukcije: gredne, lučne i mostove sa kosim kablovima. Krive specifičnih masa upoređene su sa vrednostima specifičnih masa čelika 59 izgrađenih mostova u 14 zemalja. Najzad, dati su komentari i zaključci.

Ključne reči: drumski most, čelični most, spregnuti most, specifična masa čelika.

STEEL MASSES OF STEEL AND COMPOSITE ROAD BRIDGES

Summary:

The aim of every employer's and designer's effort is to build the bridge with the minimal possible costs. The volume of the structural steel is very important part of total costs. In this article is presented the estimation of the steel masses for the steel and composite road bridges, according to three literature sources. The theoretical average values of expected unit steel masses, expressed as kg/m^2 of the bridge layout, are given as a functions of the span and for various structural systems: girder, arch and cable stayed bridges. The unit mass curves are compared with the unit masses of 59 built bridges from 14 countries. The conclusion is also given.

Key words: road bridge, steel bridge, composite bridge, unit mass of steel.

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1 UVOD

Cilj svakog projektovanja i građenja bilo kog mosta je između ostalog i da bude što jevtiniji. Važan faktor u ukupnoj vrednosti mosta je količina materijala potrebna za njegovu izgradnju. Predmet ovog članka je onaj deo materijala koji se odnosi na konstrukciju čeličnih i spregnutih drumskih mostova, ne i materijala stubova i konstrukcija u fundiranju.

O količini materijala za izradu konstrukcije mora se voditi računa odmah na početku projektovanja – onda kad se utvrđuje koncepcija konstrukcije, dakle počev od faze generalnog projekta, a pogotovu u idejnom projektu. Zanemarivanje ovog faktora može dovesti do neracionalnog mosta, uprkos tome što su u svim fazama projektovanja dokazane ispunjenosti svih zahteva standarda – nosivosti, upotrebljivosti, zamora i trajnosti.

2 FAKTORI KOJI UTIČU NA KOLIČINU ČELIČNOG MATERIJALA KONSTRUKCIJE DRUMSKOG MOSTA

Količina čeličnog materijala u konstrukciji mosta uopšte zavisi od niza faktora:

- saobraćajnih opterećenja;
- statičkog sistema mosta;
- raspona konstrukcije;
- uslova granične nosivosti (ULS) i upotrebljivosti (SLS);
- uslova zamora;
- uslova trajnosti konstrukcije;
- uslova izgradnje mosta: uslovi lokacije mosta, izrade, montaže, cena materijala, radne snage i opreme.

2.1 DRUMSKA SAOBRAĆAJNA OPTEREĆENJA

Drumska saobraćajna opterećenja – podeljena i osovinska – imaju veliki uticaj na masu kolovozne konstrukcije, a nešto manji na masu konstrukcije globalnog statičkog sistema, u meri koja najviše zavisi od vrste statičkog sistema mosta.

U praksi u našoj zemlji je formalno do 1991. bio u primeni propis PTP-5:1949 koji je po sadržaju odgovarao DIN 1072:1944. Počev od oko 1960. godine primenuje se često i DIN 1072:1952, sa većim podeljenim opterećenjima i naročito izraženim osovinskim opterećenjima preko vozila SLW60 (3 x 200 kN) u glavnoj saobraćajnoj traci. Počev od 1991, od uvođenja pravilnika POM:1991 – a koji je zapravo preuzet DIN 1072:1985 – saobraćajna opterećenja se dalje povećavaju uvođenjem drugog vozila SLW30 (u našoj nomenklaturi V300) sa 3 x 100 kN.

Najzad, 2012. godine uveden je standard SRPS EN 1991-2:2012 sa drumskim saobraćajnim opterećenjima definisanim u obliku podeljenih opterećenja i tandema osovina. Iskustva primene ovih opterećenja ukazuju da na kolovoznim konstrukcijama proizvode veće presečne sile u odnosu na ista prema Pravilniku POM:1991.

Inostrana literatura o očekivanim količinama materijala za pojedine statičke sisteme drumskih mostova – nemačka [3] i [5], japanska [4] i američka [5] – poziva se na iskustva izgrađenih drumskih mostova od oko 1960. do danas, gde se saobraćajna opterećenja tih mostova ne razlikuju bitno od onih sa kojima su projektovani mostovi u Jugoslaviji, odnosno Srbiji.

2.2 STATIČKI SISTEMI MOSTOVA

Uticaoj statičkog sistema glavnog nosećeg sistema mosta je zapravo najuticajniji faktor na količinu materijala konstrukcije. Pri istim rasponima, u različitim statičkim sistemima, dobijaju se različite količine čeličnog materijala konstrukcije, prema tome i mogućnost za razliku u koštanju.

O ovoj činjenici se uveliko vodilo računa već u drugoj polovini 19. i početkom 20. veka, u vreme intenzivne izgradnje drumskih i železničkih saobraćajnica u Evropi i SAD. Razvijeni su empirijski obrasci za proračun masa konstrukcije za razne statičke sisteme i vrste kolovoznih konstrukcija. Takođe su statistički obrađivane mase izvedenih mostova i dobijali su se kao rezultat dijagrami raspon/specifična masa konstrukcije. Specifična masa konstrukcije izražavana je: za drumske mostove kao t/m^2 osnove mosta; za železničke mostove kao t/m^1 koloseka na mostu. Primeri ovih analiza mogu se naći u [1].

U novije vreme o masama konstrukcije se takođe vodi računa, kombinovano sa koštanjem mosta takođe. Primeri su nemačke analize iz 1948. i 1952, statističke analize Hercoga (Herzog) [2] i Gajslera (Geissler) [5], kao i japanske statističke analize [3] i američke [4] o čemu će biti više reči u t. 3.

2.3 OSTALI UTICAJNI FAKTORI

Ostali uticajni faktori – uslovi nosivosti, upotrebljivosti, zamora i trajnosti – imaju ograničeni uticaj na težinu masu konstrukcije mosta. Njihov uticaj se svodi na minimum pre svega pravilnim konstruisanjem.

Uslovi izgradnje mosta – uslovi lokacije mosta, izrada, montaža, cene materijala, radne snage i opreme – mogu da opredele ili bitno suze izbor statičkog sistema mosta, a i vrstu kolovozne konstrukcije.

3 MASE ČELIČNIH I SPREGNUTIH DRUMSKIH MOSTOVA PREMA LITERaturi

3.1 UVOD

Među pomenutim faktorima koji utiču na masu konstrukcije mosta, uticaj statičkog sistema glavnog nosećeg sistema je najvažniji. Nadalje će se prikazati rezime analiza iz [2], [3] i [5], kao najširim po obuhvatu mogućih statičkih sistema drumskih čeličnih i spregnutih mostova. Rezultati američkih analiza [4] su ograničeni uglavnom samo na spregnute grede manjih raspona, tj. mnogo su užeg dijapazona u odnosu na [2], [3] i [5], pa se zato nadalje i ne citiraju.

3.2 SPECIFIČNE MASE ČELIKA MOSTOVA PO HERCOGU [3]

Hercogove procene masa čelika [2] zasnivaju se na statističkoj obradi masa 154 mosta izvedena do 1975. u 18 zemalja. Izvedeni su izrazi za očekivane specifične mase [kg/m^2] osnove mosta uz definisane varijacije mase, za sledeće sisteme mostova:

- gredni (G) čelični, otvoren (o) poprečni presek: $m_{Go} = 190 + 1,50L_m$ [kg/m^2];
- gredni čelični, sandučast (b) poprečni presek: $m_{Gb} = 200 + 1,70L_m$ [kg/m^2];
- gredni spregnuti (C), otvoren (o) poprečni presek: $m_{Co} = 125 + 1,35L_m$ [kg/m^2];

- gredni spregnuti, sandučast (b) poprečni presek: $m_{Cb} = 145 + 1,40L_m$ [kg/m²];
- rešetksti (T) gredni: $m_T = 3,80L_m$ [kg/m²];
- kosi kablovi (Cst), kablovi u obliku harfe (h): $m_{Cst-h} = 240 + 0,62L_{max}$ [kg/m²];
- kosi kablovi, kablovi u obliku lepeze (f): $m_{Cst-f} = 280 + 0,69L_{max}$ [kg/m²];
- lučni (A) i ramovski (F): $m_A = m_F = 100 + 2,00L_A$ [kg/m²].

Primenjene oznake: L_m = prosečni raspon [m], L_{max} = najveći raspon, L_A = raspon luka ili rama. Koeficijenti varijacije su $\sigma = \pm 10\%$ do $\pm 16\%$.

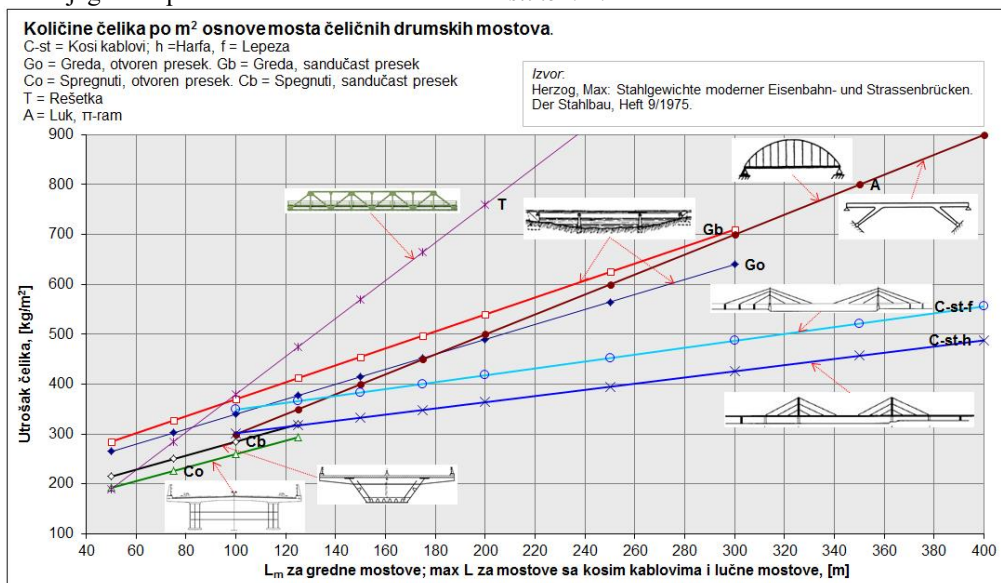
Kod mostova sa kosim kablovima može se podrazumevati da je ceo most čelični, (i greda, i piloni), što je i bila praksa u svetu u vreme [2]. Kod lučnih mostova ne precizira se o kojoj vrsti lučnih mostova je reč, pa se može smatrati da se izraz odnosi na sve vrste lukova. Kod ramovskih mostova se misli na Π -ramove (Sprengwerk u [2]).

Dijagrami specifičnih masa čelika prema navedenim izrazima dati su *slikom 1*.

Masa kablova mosta može se dobro proceniti na osnovu Fresineovih podataka:

- most sa 2 pilona, čelična (S) greda: $m_{Cst,S} = (50/630)L_{max}$ [kg/m²];
- most sa 2 pilona, spregnuta (com) greda: $m_{Cst,com} = (50/360)L_{max}$ [kg/m²];
- most sa 2 pilona, betonska (C) greda: $m_{Cst,C} = (50/170)L_{max}$ [kg/m²];
- most sa 1 pilonom = (most sa 2 pilona) x 2, za svaku vrstu greda mosta.

Dijagrami specifičnih masa kablova dati su *slikom 2*.

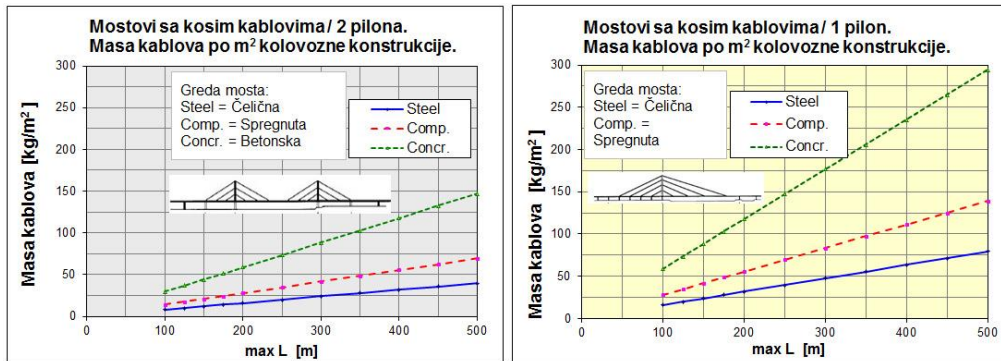


Slika 1: Specifične mase čeličnih i spregnutih mostovskih konstrukcija prema [2].

Rad [2] zasnovan je na analizi masa mostova sa kosim kablovima prve generacije – mostova kompletno čeličnih i sa dugačkim razmacima ankernih mesta kablova na gredi za ukrućenje. Od 1975. do danas mnogo toga se promenilo u konstruisanju mostova sa kosim kablovima: broj kablova je daleko veći, grede su i spregnute (ne samo čelične), piloni su prevladavajuće betonski (umesto čeličnih). Dijagrami iz *slika 2 i 3* su i tada upotrebljivi uz sledeće napomene:

- mase kablova ostaju bez izmena;

- učešće pilona u ukupnoj masi čelične konstrukcije je oko 15%, (za gredu preostaje oko 85%);
- ukoliko je greda mosta spregnuta, onda masu čelika grede treba umanjiti za najmanje 200 kg/m².



Slika 2: Specifične mase kablova mostova sa kosim kablovima.

3.3 SPECIFIČNE MASE ČELIKA MOSTOVA PO JAPANSKOJ ASOCIJACIJI JASBC [3]

Japanske procene masa [3], izvedene su na osnovu statističke obrade baza podataka JASBC (Japan Association of Steel Bridge Construction), oko 8500 mostova izvedenih u Japanu 1978-1993. Specifične mase čelika čeličnih i spregnutih mostovskih konstrukcija iz [3] odnose se na m² kolovoza.

Japanski autori iz [3] se, naime, ne obaziru na ukupnu širinu mosta već samo na širinu kolovoza zbog koga je most i izgrađen. Pod kolovozom se verovatno podrazumeva prostor između zaštitnih ograda ili ivičnjaka. Na taj način otpale su širine zona kolovozne konstrukcije koje pokrivaju razdelni pojas, zaštitne ograde, službene staze, a ponegde i delovi konstrukcije koji prodiru kroz ravan kolovoza, npr. piloni, kablovi, lukovi, zidovi rešetki. Na ovaj način, ako podatke iz [3] hoćemo da dovedemo do uporedivosti sa [2], potrebno je vrednosti iz [3] podeliti sa 1,2 do 1,5, tj. $m_{[2]} \leftrightarrow m_{[3]}/1,2$ do $m_{[3]}/1,5$.

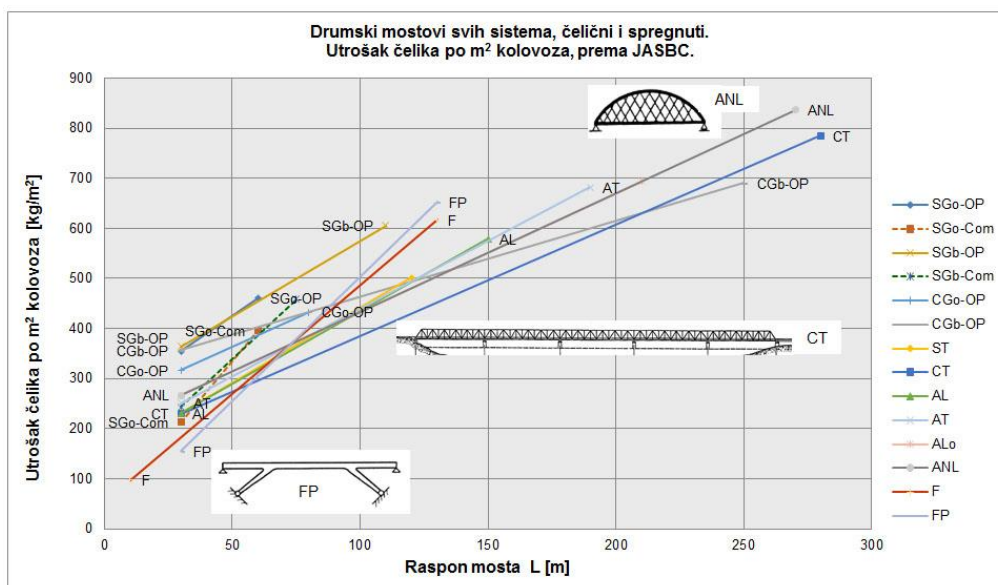
Rezultati japanskih statističkih analiza [3] daleko su detaljniji u odnosu na iste iz [2]. Vrste statičkih sistema su i sa detaljnijom podelom. S obzirom da je reč o stotinama vrednosti za svaki od statičkih sistema, specifične mase čeličnog materijala su date u dva oblika:

- kao linearna regresija: $m = aL + b$ [t/m²];
- kao parabolna regresija: $m = \alpha L^2 + \beta L + \gamma$ [t/m²],

gde su parametri a , b , α , β i γ u [3] dati tabelarno za svaki sistem mosta. L [m] je raspon mosta, a σ je standardno odstupanje od prosečne vrednosti.

U [3] su date i specifične mase čeličnih mostova sa kosim kablovima, međutim, po autorima [3], to nisu pouzdane analize. To je i razlog zašto se ovde ne navode.

Rezultati analiza [3] pretočeni u dijagrame - prikazane su krive iz linearne regresije - dati su slikom 3.



Slika 3: Dijagrami specifičnih masa drumskih mostova, čeličnih i spregnutih, prema [3].

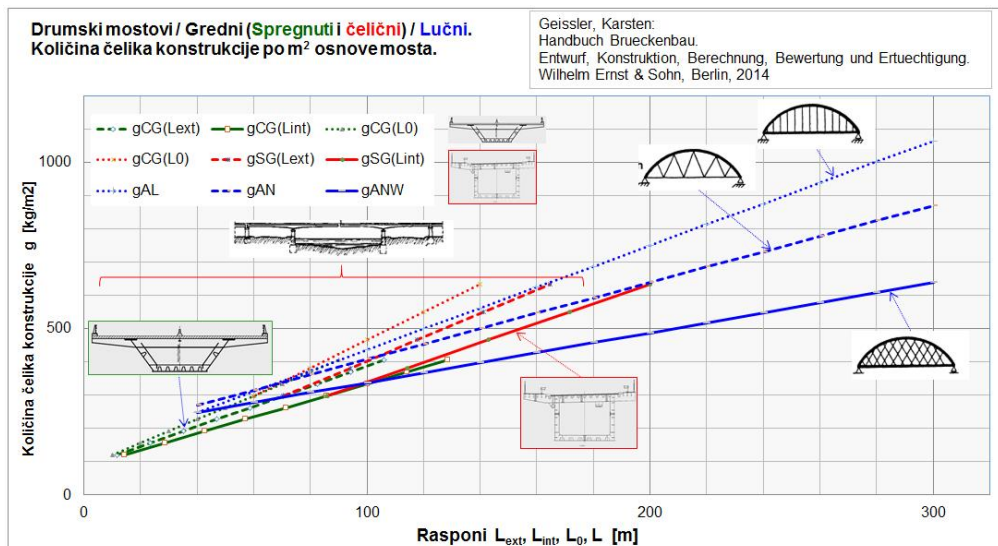
3.4 SPECIFIČNE MASE ČELIKA MOSTOVA PO GAJSLERU [5]

Krive specifičnih masa čelika drumskih mostova iz [5] zasnivaju se na nekoliko desetina izvedenih mostova u Nemačkoj, (u [5] nije naveden tačan broj). Tretirani su noviji drumski mostovi, izgrađeni u poslednjih oko 20 godina. Krive očekivanih specifičnih masa su prave. Analitički izrazi u [5] nisu navedeni, ali izvedeno iz originalnih dijagrama glase:

- gredni mostovi (indeks G), spregnuti (indeks C), sandučastog preseka:
proste grede i kontinualne grede na L_0 : $g_{GC}(L_0) = 3,571L_0 + 84$ [kg/m²],
kontinualne grede: krajnja polja: $L_{ext} = 0,85L$; srednja polja: $L_{int} = 0,70L$;
 L_0 = rastojanje nultih tačaka momenata savijanja;
- gredni mostovi (G), čelični (S), sandučastog preseka:
proste grede i kontinualne grede na L_0 : $g_{GS}(L_0) = 4,200L_0 + 46$ [kg/m²],
kontinualne grede: krajnja polja: $L_{ext} = 0,85L$; srednja polja: $L_{int} = 0,70L$;
 L_0 = rastojanje nultih tačaka momenata savijanja;
- lučni mostovi (A), sa spregnutom kolovoznom konstrukcijom:
lukovi tipa Langer (AL), (lukovi sa vertikalnim vešaljicama): $m_{AL} = 3,133L + 123$;
lukovi tipa Nilsen (AN), (rešetkaste vešaljke): $m_{AN} = 2,311L + 177$ [kg/m²];
lukovi sa dijagonalnim vešaljicama (network): $m_{ANW} = 1,500L + 188$ [kg/m²].

Slike i dijagrami iz [5] za gredne kontinualne mostove, zasnovani na L_0 , nisu najpraktičniji za primenu pošto se odnose na pojedina polja mosta, a ne na most u celini. Ako se, međutim, promenljiva L_0 zameni sa L_{ext} i L_{int} , onda se dobijene krive direktnije koriste. Uz pretpostavku o prosečnom rasponu kao u [2], (što nije navedeno u [5]), iskoristivost dijagrama je potpuna.

Rezultati analiza iz [5] i uz opisane primene L_{ext} i L_{int} dati su slikom 4.



Slika 4: Dijagrami specifičnih masa drumskih mostova, čeličnih i spregnutih, prema [5].

4 PRIMERI SPECIFIČNIH MASA ČELIKA IZVEDENIH MOSTOVA

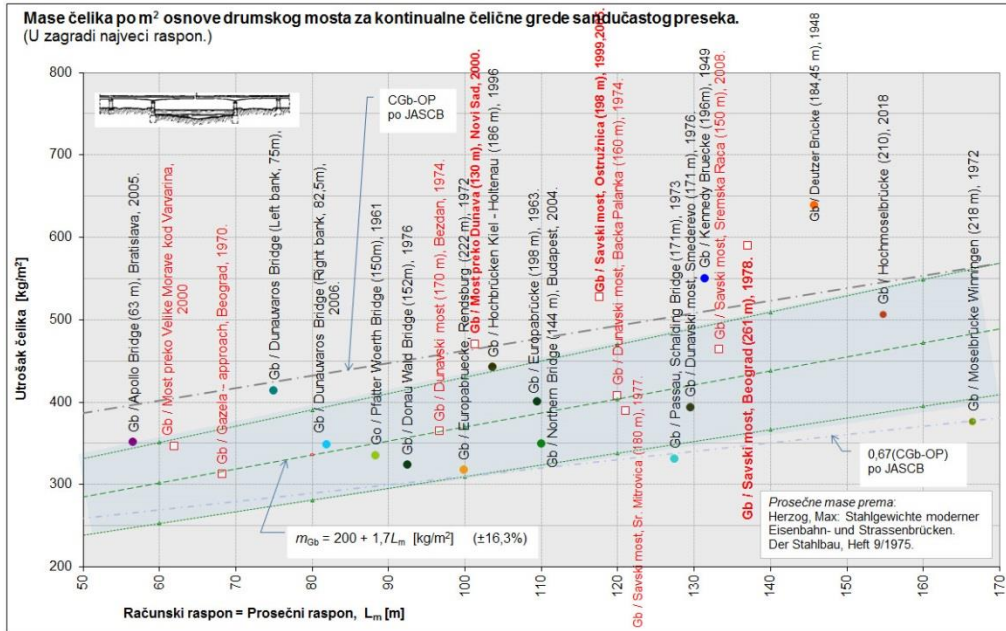
Dijagrami specifičnih masa čelika drumskih mostova prema [2], [3] i [5] razlikuju se po vrednostima koje predviđaju za istu vrstu mostova. U nastavku će se zato uporediti izrazi navedenih izvora literature sa primerima masa izvedenih mostova u Srbiji i u inostranstvu i to: grednih čeličnih – 24 mosta, grednih spregnutih – 19 mostova, lučnih – 20 mostova i mostova sa kosim kablovima – 16 mostova. Rezultati su prikazani slikama 5 do 8.

Primeri specifičnih masa 59 izvedenih mostova iz 12 zemalja pokazali su:

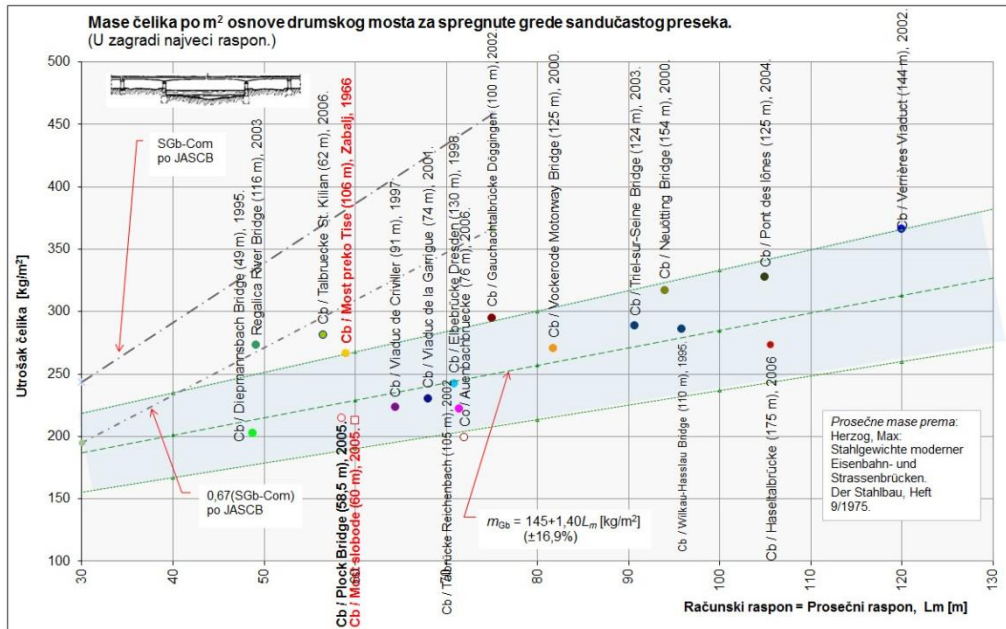
- veoma dobru iskoristivost Hercogovih [2] i Gajslerovih [5] izraza;
- uslovnu iskoristivost japanskih izraza [3] zbog pre svega nedovoljno precizno određene formulacije o površini kolovoza mosta na koju se izrazi odnose.

Primeri masa izvedenih mostova takođe su pokazali:

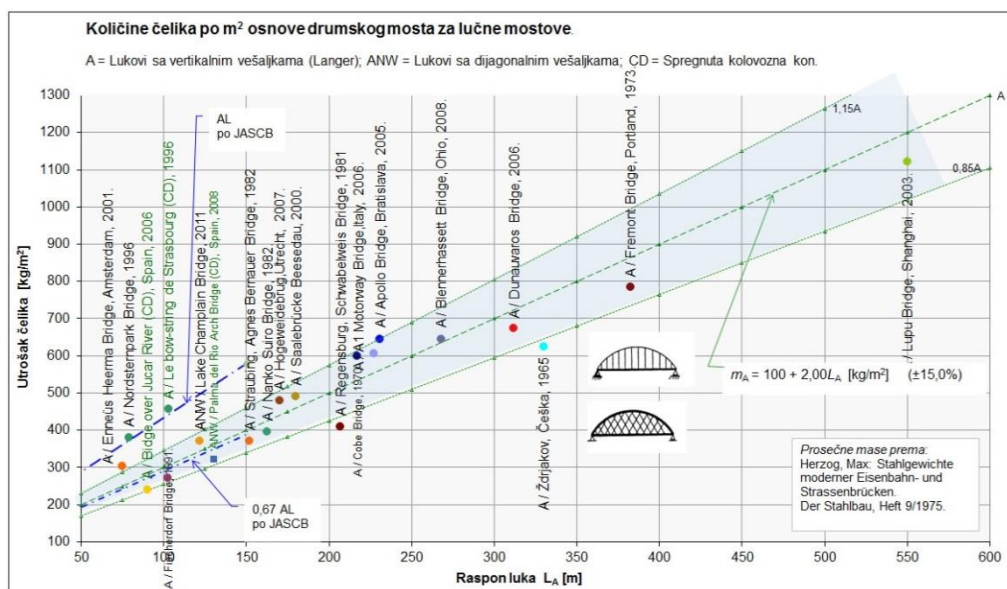
- da je masa čelika po m² osnove mosta, za isti raspon različita u različitim statičkim sistemima mostova, (videti sliku 1, sliku 3 i sliku 4);
- da most manjeg raspona jednog statičkog sistema može imati veću specifičnu masu od mosta mnogo većeg raspona i drugog statičkog sistema; primer: Varadinski most preko Dunava u Novom Sadu (130 m)/ 470 kg/m² ima veću specifičnu masu od mnogo većeg Mosta preko Visle u Plocku, Poljska (max L = 375 m), sa oko 3 puta većim saobraćajnim opterećenjem i specifičnom masom od samo 466 kg/m²!
- da je iščezavanje iz primene čeličnih greda sa ortotropnom pločom, (veoma popularnih u svetu i kod nas od oko 1955. do oko 1980.), racionalno opravdano; u oblasti raspona 50-90 m racionalnije su spregnute grede, u oblasti 90-200 m racionalni su lukovi sa spregnutom kolovoznom konstrukcijom kao i mostovi sa kosim kablovima sa betonskim pilonima i spregnutom kolovoznom konstrukcijom, već počev od raspona od oko 150 m. (Videti i sliku 9, sa približnim koštanjima mostova).



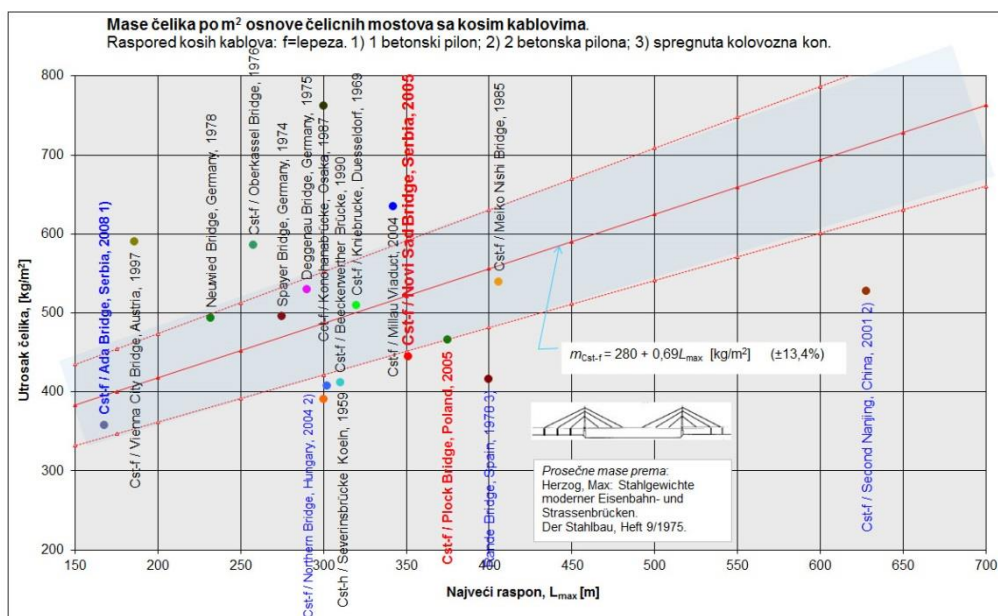
Slika 5: Mase izvedenih čeličnih grednih mostova i uklapanje u krive masa iz [2] i [3].



Slika 6: Mase izvedenih spregnutih grednih mostova i uklapanje u krive masa iz [2] i [3].



Slika 7: Mase izvedenih lučnih mostova i uklapanje u krive masa iz [2] i [3].

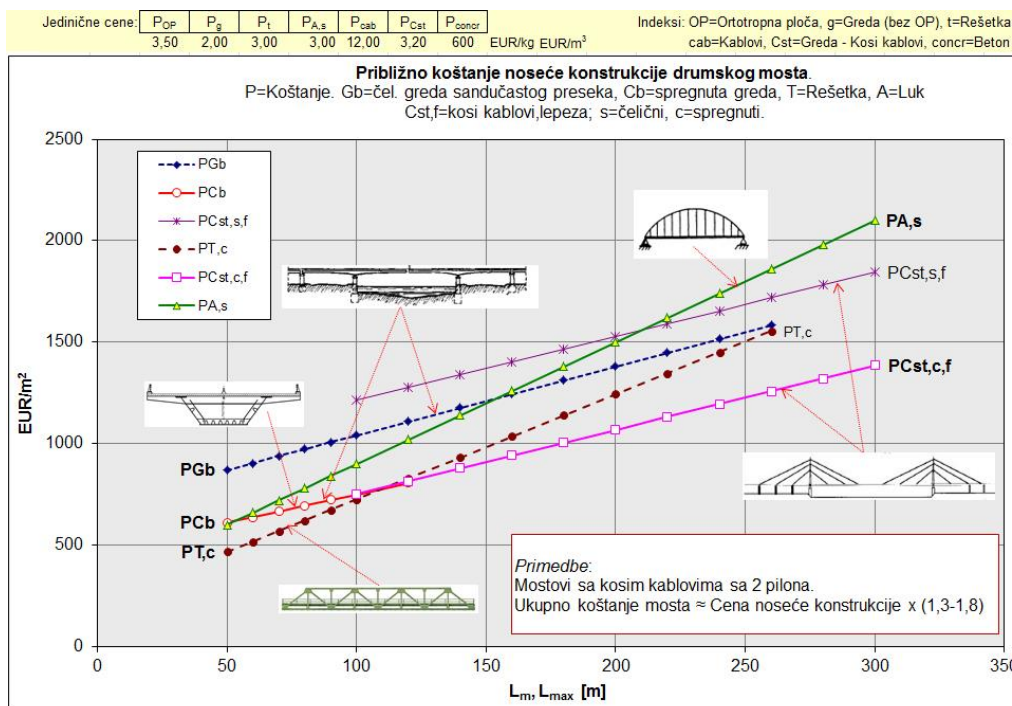


Slika 8: Mase izvedenih mostova sa kosim kablovima i uklapanje u krive masa iz [2].

Na osnovu masa iz [2] i uz pretpostavljene jedinične cene, može se dobiti i približno koštanje konstrukcije mosta, a i celog mosta, videti sliku 9.

5 ZAKLJUČAK

Izbor statičkog sistema mosta jako utiče na masu i koštanje konstrukcije i uvek se mora imati u vidu, naročito u ranim fazama projektovanja. Ovo je posebno važna tema za investitore.



Slika 9: Približno koštanje konstrukcije mosta izvedeno iz masa čelika prema [2].

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РАЧУНАРСКО 3D MODELIRANJE REZERVOARA ZA TEČNOST I SIMULACIJA DINAMIČKIH DEJSTAVA

Rezime:

S-23

U ovom radu računarskim modeliranjem objekta, za potrebe dinamičke analize, je tretiran element veze, „elastično ležište“, kao 1D prostorni konačni element. Tako tretirano elastično ležište je zamena postojeće krute veze čeličnog rezervoara i temeljne konstrukcije. Analizirane su slobodne neprigušene oscilacije i uticaj dejstva zemljotresa (spektralna – multimodalna i Time History analiza sa stvarnim ubrzanjima tla pri zemljotresu EI Centro, 1940). Rezultati analize pokazali su da uticaji od zemljotresa nisu merodavni za kriterijum nosivosti.

Ključne reči: Modeliranje, simulacija, dinamička dejstva, elastično ležište.

COMPUTER MODELLING 3D TANK FOR LIQUID AND SIMULATION OF DYNAMIC EFFECTS

Summary:

In this paper, computer modeling facility for the needs of dynamic analysis is treated element connections, "elastic support" as 1D finite element. Thus, the treated elastic support is replacing the existing rigid connection steel tank and foundation structures. We analyzed the free undamped fluctuations and the impact of earthquake (spectral – multimodal and Time History analysis with actual acceleration of soil in El Centro earthquake, 1940). Results of analysis showed that the effects of earthquake are not authoritative criteria for capacity.

Key words: Modeling, simulation, dynamic effects, elastic support.

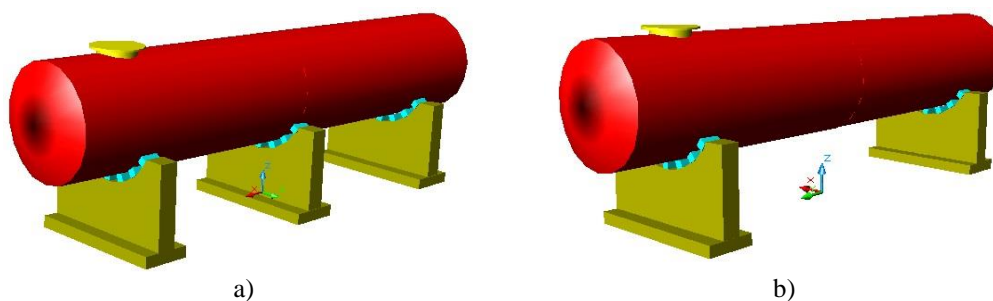
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1 UVOD

Tokom eksploatacije objekta (model sl. 1, a), kako od dejstava nastalih usled punjenja i pražnjenja bila su i druga dejstava relevantna za promatranje kriterijuma nosivosti. Na ovom objektu su utvrđena neravnomerna sleganja temelja u nivou kontaktne spojnice, odnosno naginjanja objekta na jednu stranu. Zbog toga se prvo pristupilo dijagnostifikovanju date neregularnosti, a potom je sprovedena detaljna analiza dinamičkih dejstava metodom računarskih simulacija. Predloženom metodom sanacije (sl. 1. b), predviđena je izmena postojećeg stanja konstruktivnog sistema (uklanjanjem srednjeg oslonca) kao i zamena veze rezervoara i temelja.



Slika 1 – Tretirani 3D modeli rezervoara za tečnost

2 ZADATAK DINAMIČKE SIMULACIJE

Imajući u vidu da je objekat izveden i da je u eksploataciji zapažena pojava naginjanja objekta na jednu stranu, analiziran je i predložen jedan mogući način sanacionog zahvata. Numerička simulacija sprovedena je računarskim programom za statičku i dinamičku analizu konstrukcija SAP 2000n. Pri simulaciji, elementima modela zadati su odgovarajući parametri (površina poprečnog preseka, momenti inercije preseka oko obe lokalne ose, torzioni momenti inercije itd.) karakteristike materijala (modul elastičnosti i smicanja, Poisson – ov koeficijent), a za dinamičku analizu mase elementa. Za linijske elemente masa je zadata po jedinici dužine, a za prostorne i površinske elemente po jedinici zapremine konačnog elementa. Prostornim (shell) elementima je modeliran zid i temeljna ploča fundamenta (trakasti temelj), kao i zidovi čeličnog rezervoara, dok su elementi ležišta (oslanjanje cisterne na temeljni zid) modelirani sa linijskim elementima (frame), gde su linije KE, težišne linije.

Simulacijama su tretirana dva načina sanacije, sa različitim varijantama 3D modela:

Prvi način,

- rezervoar oslonjen na tri temelja, sa krutim ležištima – postojeće stanje
- rezervoar oslonjena na tri temelja, sa elastičnim ležištima

Drugi način,

- **rezervoar oslonjena na dva temelja, sa krutim ležištima (2)**
- **rezervoar oslonjena na dva temelja, sa elastičnim ležištima (1)**
- rezervoar oslonjena na dva temelja, sa predloženim brojem krutih i elastičnih ležišta.

3 MODELIRANJE ELEMENATA SISTEMA 3D MODELOM

3.1 TEMELJ I TEMELJNA PODLOGA

Analiza nosivosti temeljnog tla:

Dužina rezervoara:	$L=15.96 \text{ m}$.
Prečnik rezervoara:	$R=3.0 \text{ m}$
Površina:	$A_R = 1.5^2 \cdot \pi = 7.086 \text{ m}^2$.
Zapreminska težina tečnosti:	$\gamma = 10.50 \text{ kN/m}^3$.
Ukupna zapremina:	$V=112.76 \text{ m}^3$.
Težina tečnosti:	$G_t = 112.76 \cdot 10.50 = 1183.98 \text{ kN} \approx 120 \text{ t}$.
Težina rezervoara:	$V = 150.34 \cdot 0.006 = 0.902 \text{ m}^3$.
	$G_c = \gamma \cdot V = 78.5 \cdot 0.902 = 70.81 \text{ kN} \approx 7.08 \text{ t}$.
Težina jednog temelja:	$G_1 = 1.30 \cdot 4.0 \cdot 0.30 \cdot 25 = 39 \text{ kN}$
	$G_2 = 4.0 \cdot 1.35 \cdot 0.50 \cdot 25 = 67.5 \text{ kN}$
	$G = 39 + 67.5 = 106.50 \text{ kN}$.
- dva temelja	$G = 2 \cdot 106.50 = 213.00 \text{ kN}$.
- tri temelja	$G = 3 \cdot 106.50 = 319.50 \text{ kN}$.
Dozvoljeni napon tla:	$\sigma_{\text{tla}} = 150 \text{ kN/m}^2$.

Kontrola napona u nivou kontaktne spojnice,

$$A_{\text{pot}} = \frac{1574.31}{150} = 10.50 \text{ m}^2.$$

na površini tri temelja,

$$P_3 = 70.81 + 1184 + 319.50 = 1574.31 \text{ kN}.$$

$$\sigma = \frac{P_3}{A_{3_temelja}} = \frac{1574.31}{15.60} = 100.92 \text{ kN/m}^2 < 150 \text{ kN/m}^2.$$

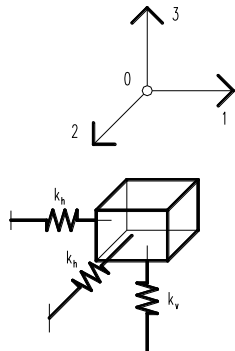
na površini dva temelja,

$$P_2 = 70.81 + 1184 + 213 = 1467.81 \text{ kN}.$$

$$\sigma = \frac{P_2}{A_{2_temelja}} = \frac{1467.81}{10.40} = 141.14 \text{ kN/m}^2 < 150 \text{ kN/m}^2.$$

Model temeljnog tla tretiran je metodom analogije kao elastična opruga, tako da je svaki čvor „shell“ elementa oslonjen na oprugu odgovarajuće krutosti. Krutosti opruga simuliraju vertikalno pomeranje i translaciju čvora u oba horizontalna pravca, dok je rotacija zanemarena. Zanemarena rotacija temelja je zbog nedostatka postupka modeliranja ove deformacijske veličine. Krutosti opruga se određuje na više načina, u svemu prema izabranom metodu

proračuna sleganja temelja. Usvojen model tla za simulaciju u ovom radu je Winklerov model, sa redukovanim modulom reakcije tla $K_0=30 \text{ MN/m}^3$ na površini temeljne spojnice „B · L”.



$$K = K_0 \cdot \frac{0.305}{B} \cdot \frac{m + 0.5}{1.5 \cdot m},$$

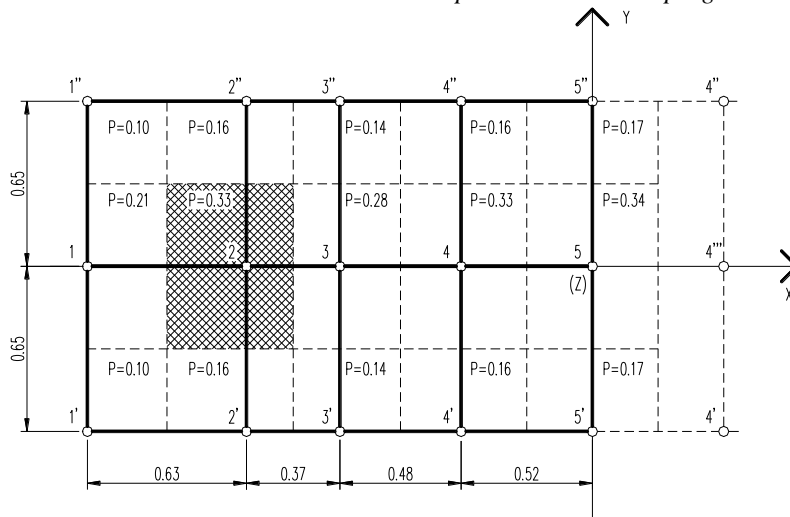
$$m = \frac{L}{B} = \frac{4.00}{1.30} = 3.08$$

$$K = 30000 \cdot \frac{0.305}{1.30} \cdot \frac{3.08 + 0.5}{1.5 \cdot 3.08} = 5454.81 \text{ kN/m}^3.$$

$$k_v = K \cdot P$$

$$k_H = \frac{k_v}{\sqrt{2}}$$

Slika 1 - Čvor "shell" elementa sa pravcima krutosti opruga



Slika 2 – Osnova temeljne spojnice sa položajem čvorova i pripadajućim površinama za redukciju modula reakcije tla – simetrični prikaz

Da bi se pri simulacijama izbegao uticaj dejstva tečnosti na deformaciju rezervoara, sopstvena težina + težina tečnosti je unesena kao povećanje zapreminske težine, odnosno gustine materijala zidova rezervoara. Tako je sada i gornji deo objekta (rezervoar) simuliran kao kruto telo, pri nametanju dva nivoa terećenja, pun rezervoar i 1/2 rezervoara.

3.2 ELASTIČNO LEŽIŠTE – VIBROIZOLATOR

Za elastično ležišta rezervoara potrebno je izvršiti proračun parametara zamenjujućeg štapa kojim se simulira vibroizolator u 3D modelu. Karakteristike zamenjujućeg štapa proračunavaju se iz uslova jednakosti krutosti sa vibroizolatorom.

Usvojene dimenzije vibroizolatora su:

- B= 50 mm, L= 50 mm, debljina sloja elastomera: hp=9 mm, broj slojeva: 3 kom.

Zamenjujući poluprečnik:

$$R^2 \cdot \pi = a^2 \qquad R = \sqrt{\frac{a}{\pi}} = \sqrt{\frac{50 \cdot 50}{\pi}} = 28.209$$

$$\zeta = \frac{h_p}{2 \cdot R} = \frac{9}{2 \cdot 28.209} = 0.16 > 0.1 \qquad \Phi = \frac{1}{4 \cdot \zeta} = \frac{1}{4 \cdot 0.16} = 1.567 < 2.5$$

Prema tome, usvaja se vibroizolator sa visokim slojevima.

Za visokoslojne vibroizolatore važe sledeće relacije:

$$E_n = \frac{3 \cdot G}{\frac{1 - \text{th}(e \cdot \zeta)}{e \cdot \zeta}}$$

e – je konstanta koja zavisi od oblika poprečnog preseka
e=4.9 za kružni presek
e=4.25 za kvadratni presek

$g_1=0.23$

$$G_n = G \cdot (1 - g_1 \cdot \zeta)$$

Kako je $G_n \approx G = 1.0 \text{ MPa}$

$$E_n = \frac{3 \cdot 1.0}{\frac{1 - \text{th}(4.25 \cdot 0.16)}{4.25 \cdot 0.16}} = 4.99 \text{ MPa}$$

3.2.1 Proračun karakteristika zamenjujućeg štapa

- **Krutost na pritisak**

$$k_1^a = \frac{E \cdot F}{h_p} = \frac{0.4994 \cdot 5 \cdot 5}{0.9} = 13.872 \text{ kN/cm. (krutost jednog sloja)}$$

$$k_e^a = \frac{k_1^a}{3} = \frac{13.872}{3} = 4.624 \text{ kN/cm. (zamenjujuća krutost izolatora)}$$

Iz uslova jednakosti krutosti ($k_e^a = k_{\text{stapa}}^a$) usvaja se da su moduli elastičnosti štapa i vibroizolatora identični ($E_n \equiv E$), tj.:

$$k_e^a = \frac{E \cdot A}{h} \qquad A = \frac{k_e^a \cdot h}{E_n} = \frac{4.624 \cdot 5}{0.4994} = 46.296 \text{ cm}^2.$$

- **Krutost na savijanje**

$$k_1^s = \frac{G_n \cdot F}{h_p} = \frac{0.1 \cdot 5 \cdot 5}{0.9} = 2.778 \text{ kN/cm.} \quad (\text{krutost jednog sloja})$$

$$k_e^s = \frac{k_1^s}{3} = \frac{2.778}{3} = 0.926 \text{ kN/cm.} \quad (\text{zamenjujuća krutost izolatora})$$

Iz uslova jednakosti krutosti ($k_e^s = k_{\text{stapa}}^s$) usvaja se da su moduli elastičnosti štapa i vibroizolatora identični ($E_n \equiv E$), tj.:

$$k_e^s = \frac{12 \cdot E \cdot I_x}{h^3} \quad I_x = \frac{k_e^s \cdot h^3}{12 \cdot E} = \frac{0.926 \cdot 5^3}{12 \cdot 0.4994} = 19.313 \text{ cm}^4. \quad I_x = I_y \text{ – kvadratni presek}$$

- **Krutost na torziju**

$$k_1^t = \frac{F \cdot (a^2 + b^2) \cdot G_n}{12 \cdot h_p} = \frac{5 \cdot 5 \cdot (5^2 + 5^2) \cdot 0.1}{12 \cdot 0.9} = 11.574 \text{ kN/cm.} \quad (\text{krutost jednog sloja})$$

$$k_e^t = \frac{k_1^t}{3} = \frac{11.574}{3} = 3.858 \text{ kN/cm.} \quad (\text{zamenjujuća krutost vibroizolatora})$$

Iz uslova jednakosti krutosti ($k_e^t = k_{\text{stapa}}^t$) usvaja se da su moduli elastičnosti štapa i vibroizolatora identični ($G_n \equiv G$), tj.:

$$k_e^t = \frac{G \cdot I_t}{h} \quad I_t = \frac{k_e^t \cdot h}{G_n} = \frac{3.858 \cdot 5}{0.1} = 192.9 \text{ cm}^4.$$

4 RAČUNARSKE SIMULACIJE DINAMIČKIH DEJSTAVA

Dinamička analiza sprovedena na 3D modelu uzima da je broj mogućih stepeni slobode kretanja čvora jednak šest. Tako su obuhvaćene podužne, vertikalne, horizontalne i torzione oscilacije i tretirane su kao spregnute. Pri proračunu podužnih, vertikalnih i horizontalnih oscilacija potrebno je odrediti raspored masa duž konstrukcije, dok je za proračun torzionih vibracija neophodno sračunati raspored polarnih momenata inercije masa. Za određivanje masa uzeta je sopstvena težina, i težina tečnosti. Proračunski model rezervoara i dinamički stepeni slobode modela birani su tako da adekvatno predstavljaju raspored masa i krutost objekta.

Za ovu analizu uzeti su svi oblici oscilovanja koji značajno doprinose ukupnom odgovoru konstrukcije, odnosno zbir efektivnih modalnih masa je 90% od ukupne mase objekta.

4.1 SLOBODNE NEPRIGUŠENE OSCILACIJE

Simulacija slobodnih neprigušenih oscilacija sprovedena je uz pretpostavku da su podužne, vertikalne, horizontalne i torzione oscilacije međusobno zavisne.

4.2 SIMULACIJA DEJSTVA ZEMLJOTRESA

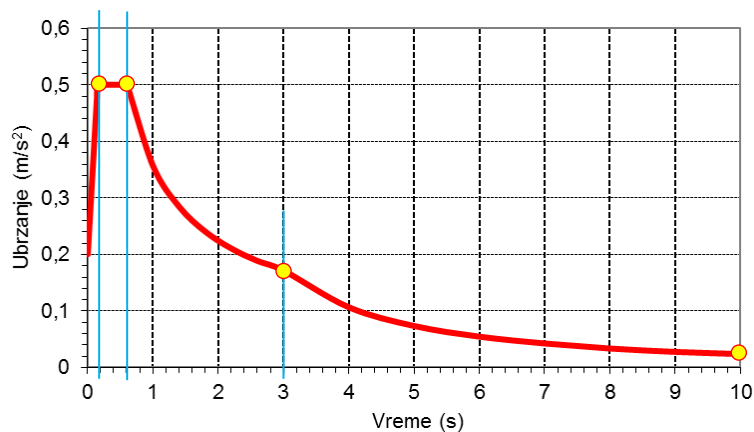
4.2.1 Spektralno modalna analiza

Spektralna analiza (multimodalna) je elastična analiza maksimalnih dinamičkih odziva svih značajnijih tonova konstrukcije, koja koristi ordinate projektnog (proračunskog) spektra ubrzanja tla u zavisnosti od lokacije objekta. Ukupni odziv konstrukcije dobija se statističkom kombinacijom maksimalnih doprinosa pojedinih tonova [4]. Za ovu simulaciju seizmički parametri određeni su prema [4] i [5]. Na osnovu izvršene kategorizacije, ova konstrukcija spada u kategoriju objekata tipa „I-kategorije“, a fundirana je na terenu II kategorije tla. Seizmička analiza sprovedena je za dejstvo projektnog zemljotresa tipa „ Z_1 “ i „ Z_2 “. Projektni zemljotres tipa „ Z_1 “ je najjači očekivani zemljotres koji se može dogoditi jedanput u 100 godina sa verovatnoćom pojave od 70%, dok je projektni zemljotres „ Z_2 “ najjači očekivani zemljotres koji može da se dogodi jedanput u 1000 godina sa istom verovatnoćom pojave.

Za lokaciju (Novi Kneževac), objekat se nalazi u području sa maksimalnim intenzitetom očekivanih zemljotresa 8⁰ skale MSK-64, i za povratni period od 100 godina (projektni zemljotres tipa Z_2). Za 8⁰ projektne seizmičnosti, prema tabeli 3 iz Pravilnika, maksimalno ubrzanje tla iznosi 20% od ubrzanja zemljine teže ($g=9.81 \text{ m/s}^2$). Da bi se metodom spektralne analize dobio odziv konstrukcije, potrebno je definisati spektralnu krivu pobude (sl. 3).

S obzirom da je za projektni zemljotres tipa „ Z_1 “ dobijena veća vrednost koeficijenta seizmičkog intenziteta ($K_s=0.05$) nego za projektni zemljotres tipa „ Z_2 “ ($K_s=0.04$), merodavan je projektni zemljotres tipa „ Z_1 “. Na osnovu datih parametara i prema EC-8, konstruisan je projektni spektar ubrzanja neophodan za unos u računarski program SAP 2000n. (Slika 3.)

Projektni spektar $S_d(T)$, za referentni povratni period je normalizovan u odnosu na ubrzanje zemljine teže „g“:



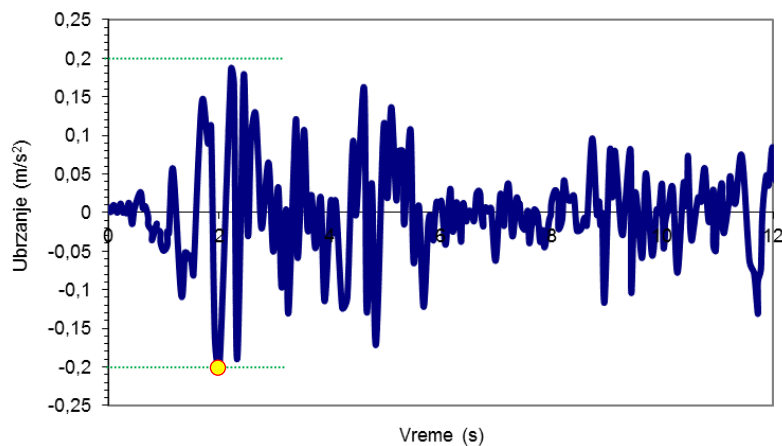
Slika 3 – Projektni spektar ubrzanja prema EC-8

4.2.2 Direktna dinamička analiza

Radi uvida u ponašanje konstrukcije u realnim seizmičkim uslovima, sprovedena je direktna dinamička analiza, tj. određen je vremenski zavisani odziv konstrukcije na dejstvo

realnih zemljotresa. Vremenska istorija dobijena je direktnom numeričkom integracijom odgovarajućih diferencijalnih jednačina kretanja, gde je oscilovanje tla prikazano kao vremenski tok ubrzanja tla za vreme zemljotresa. Zemljotresno dejstvo je opisano skupom stvarnih akceleroograma, tj. zapisa akceleroograma ranije dogođenih zemljotresa. Kao ulazni podatak u jednoj simulaciji korišćen je akceleroogram zemljotresa El Centro 1940 god. (sl. 4.). S obzirom da je korišćen prostorni model konstrukcije, seizmičko kretanje tla je opisano pomoću dva istovremena akceleroograma u „x“ i „y“ pravcu. Svi akceleroogrami su skalirani na vrednost očekivanog maksimalnog ubrzanja tla na lokaciji objekta za povratni period od 1000 godina, sa vrednosti 0.20g.

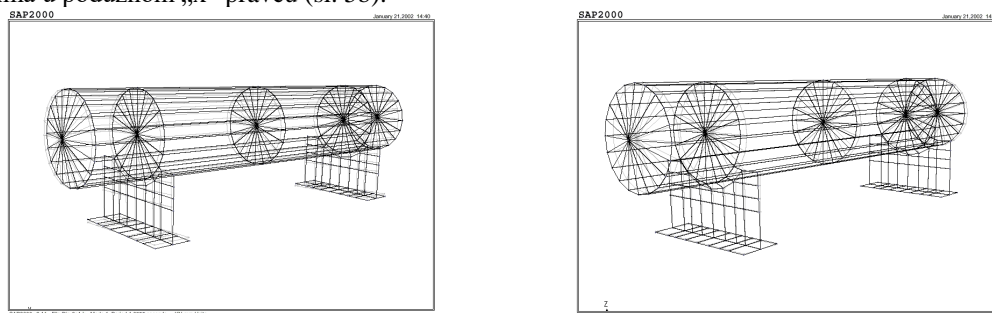
Takav odziv konstrukcije, daje vremensku zakonitost statičkih i deformacijskih veličina za vreme zemljotresa. Prikazani dijagrami su dati za pojedine veličine, jer daju očigledan uvid u kvantitativne vrednosti pojedinačnih uticaja u konstrukciji.



Slika 4 – Skalirani akceleroogram zemljotresa El Centro 1940

4.3 REZULTATI SIMULACIJA

Slobodne oscilacije 3D modela pokazuju da prvi oblik oscilovanja modela sa krutim ležištima ima pomeranja u poprečnom „y“ pravcu (sl. 5a), dok sistem sa elastičnim ležištima ima u podužnom „x“ pravcu (sl. 5b).



a) Poprečno oscilovanje (2)

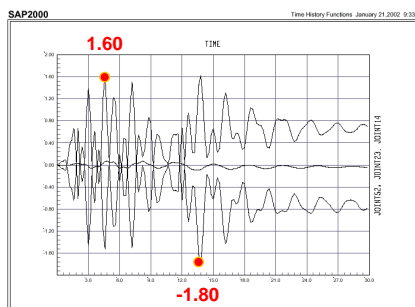
b) Podužno oscilovanje (1)

Slika 5 – Slobodne neprigušene oscilacije – prvi oblik oscilovanja

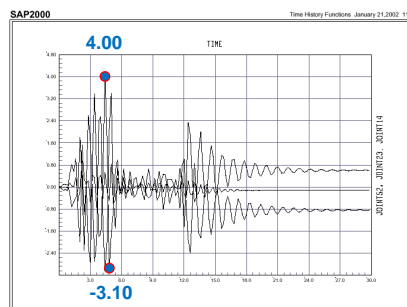
Tabela 1 – Rezultati slobodnih neprigušenih oscilacija 3D modela (1) za prvih 6 tonova

Mode	Period	Frequency	Frequency	Opis vibracije
	(time)	(cyc/time)	(rad/time)	
1	3.328896	0.300400	1.887468	podužna
2	2.636873	0.379237	2.382817	rotacija u poprečnom pravcu
3	1.636044	0.611230	3.840474	rotacija u poprečnom pravcu oslonca-2
4	1.198826	0.834150	5.241116	rotacija u poprečnom pravcu oslonca-1
5	0.958936	1.042822	6.552244	vertikalna
6	0.898871	1.112507	6.990088	vertikalna

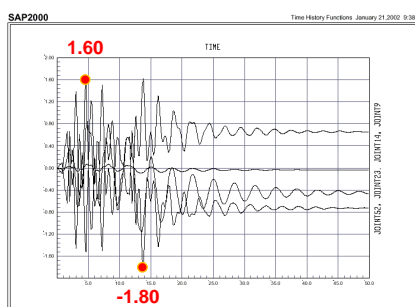
Pomeranja sa elastičnim ležištima usled dejstva zemljotresa analiziranog po metodi spektra u oba horizontalna pravca pokazuje neznatna pomeranja u odnosu na rešenje sa krutim ležištima.



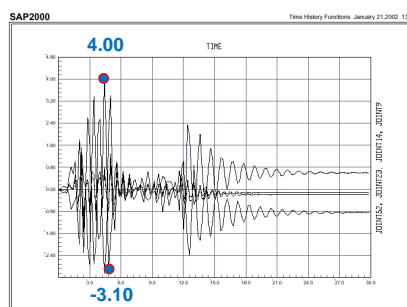
a) UZ – čvorova 14,23,52 temeljna spojnica (1)



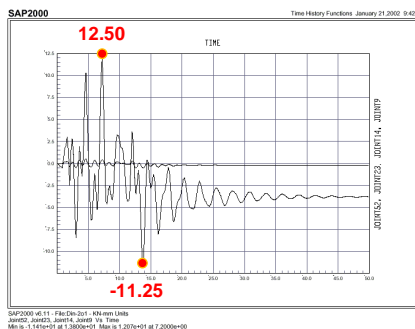
b) UZ – čvorova 14,23,52 temeljna spojnica (2)



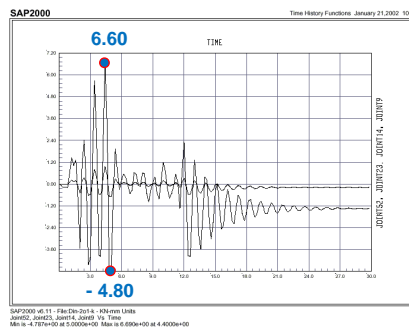
c) UZ – čvorova 9,14,23,52 (1)



d) UZ – čvorova 9,14,23,52 (2)

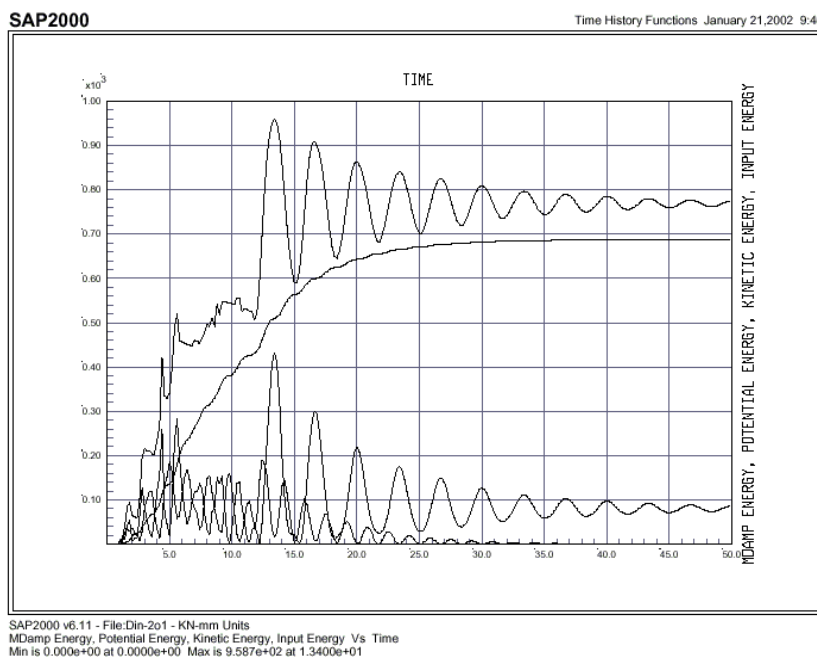


e) UY – čvorova 9,14,23,52 (1)

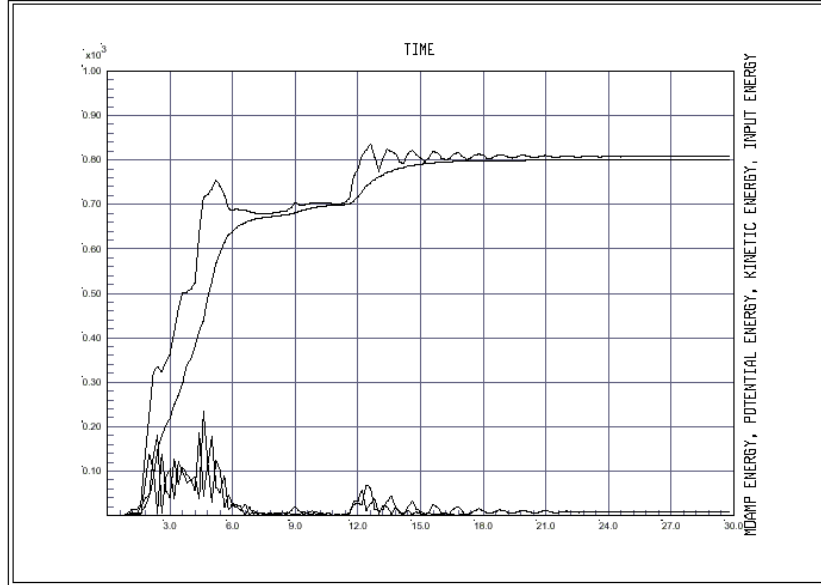


f) UY – čvorova 9,14,23,52 (2)

Slika 6 – Rezultati Time History simulacije – pomeranja čvorova



a) 3D Model sa elastičnim ležištima (1)

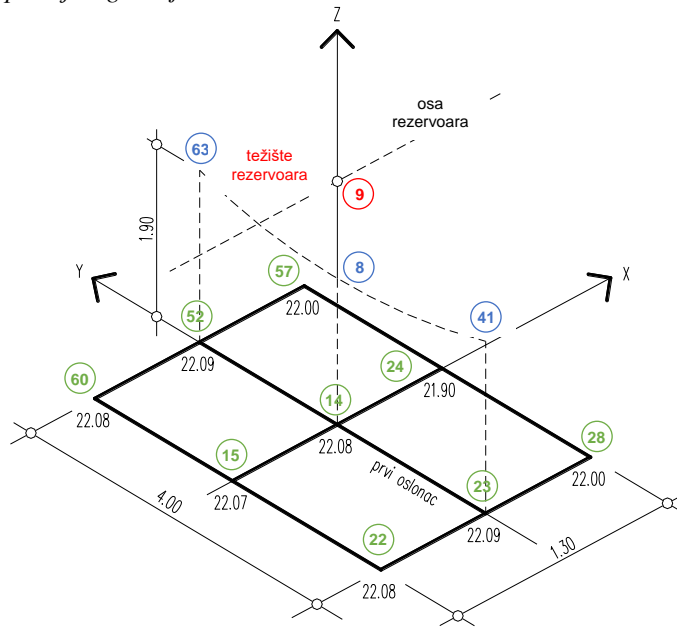


SAP2000 v6.11 - File:Din-2o1-k - KN-mm Units
 MDamp Energy, Potential Energy, Kinetic Energy, Input Energy Vs Time
 Min is 0.0000e+00 at 0.0000e+00 Max is 8.358e+02 at 1.2600e+01

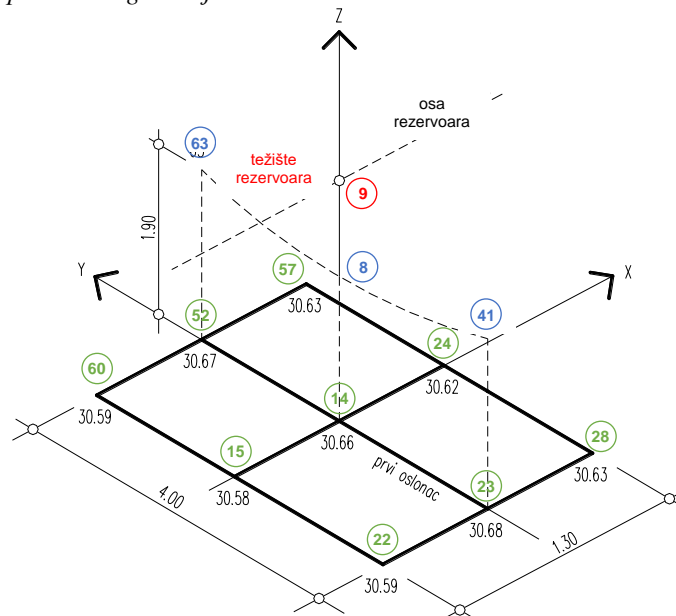
b) 3D Model sa krutim ležištima (2)

Slika 7 – Energetska analiza pri dejstvu zemljotresa El Centro 1940

- a) kod postojećeg stanja



- b) kod predloženog rešenja



Slika 8 – Pomeranja čvorova od stalnog opterećenja u nivou temeljne spojnice (mm)

Na (sl. 7,a) prikazani su rezultati Time History simulacije sa energetsom analizom simuliranog zemljotresa elastično oslonjenog rezervoara dok je na (sl. 7,b) druga varijanta sa krutim ležištima. U ovoj analizi uočava se na (sl. 6,a i sl. 6,b) da su pomeranja temeljne spojnice cca 2.5 puta veća kod rešenja sa krutim ležištima. Pomeranja u poprečnom pravcu modela „y“ pravac su date na (sl. 6,e i sl. 6,f), i pokazuju da su veće kod rešenja sa elastičnim osloncima za cca 42%. Ovde se zaključuje, da objekat pri dejstvu zemljotresa ima manju krutost u poprečnom pravcu, i zbog toga treba očekivati veća pomeranja upravo u tom pravcu, čime je model simuliran sa krutim ležištima pokazao pomeranja kao na objektu (in situ registrovano). Kako je rešenje sa elastičnim ležištima dopustilo pomeranja izazvana zemljotresom, a time model postao fleksibilan za taj pravac, postavljena ležišta su elemente koji disipiraju upravo ta pomeranja. Drugim rečima, sistem troši u većoj meri energiju prigušenja konstrukcije za savladavanje, odnosno akumulaciju energije zemljotresa, koja se pak mora odraziti u pravcu i smeru određene deformacijske veličine. U kojoj meri se izrazi deformacija ona nanosi vidne defekte ili neregularnosti na konstruktivnom sistemu ili ne. Može se reći da konstruktivni sistem modelirana na ovaj način može da izdrži zemljotres projektovanog stepena i da će ponašanje konstrukcije biti u domenu elastične oblasti. Na kraju na (sl. 8) dati su rezultati statičkog proračuna vertikalnih pomeranja temeljne spojnice označenih čvorova pri dejstvu sopstvene težine i težine tečnosti punog rezervoara.

5 ZAKLJUČAK

Na osnovu svega prikazanog u ovom radu pokazana je kompleksnost postupka modeliranja i sprovođenja simulacija za potrebe dinamičke analize u građevinskom konstrukterstvu. Prema tome, ovim istraživanjem daju se određena zapažanja i zaključci:

- analiza slobodnih neprigušenih oscilacija pokazuje da do rezonancije neće doći,
- veličine pomeranja pri dejstvu realnog zemljotresa, dobijene na osnovu direktne dinamičke analize, značajno su manji u odnosu na uticaje od stalnog opterećenja,
- uticaji od dejstva zemljotresa nisu merodavni za kriterijum nosivosti ovog objekta.

Predlaže se sanaciono rešenje kao varijanta elastično oslonjenog rezervoara za tečnost sa uklonjenim srednjim osloncem, 3D Model (sl. 1,b).

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Vladimir Živaljević¹, Dušan Kovačević²

MKE MODELIRANJE OKVIRA ZA ISPITIVANJE KONSTRUKCIJSKIH ELEMENATA PROBNIM OPTEREĆENJEM

Rezime:

S-24

U radu je opisan MKE model i numerička analiza konstrukcije čeličnog laboratorijskog okvira za ispitivanje konstrukcijskih elemenata probnim opterećenjem. Konstrukcija okvira modelirana u dve varijante: MKE model sa linijskim i sa površinskim KE. Sve veze (zavrtnji i zavareni spojevi) modelirane su kao krute, primenom specijalnih KE veze. Model je urađen u MKE softveru AxisVM[®] verzija 13.1e.

Ključne reči: laboratorijski okvir, MKE model, probno opterećenje

FEM MODELING OF THE TEST FRAME FOR TEST BY LOAD OF STRUCTURAL ELEMENTS

Summary:

This paper describes FEM model and numerical analysis of steel structure of test frame for experimental research of structural elements by test load. Modeling of the frame structure was made in two variants: FEM model with line FEs and with shell FEs. Connections (bolts and welds) are modeled as rigid by the use of the so-called link FE. Model is made in AxisVM[®] 13.1e FEM software.

Key words: test frame, FEM model, test load

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1 INTRODUCTION

For the purpose of FEM modeling and numerical analysis of the test frame structure, AxisVM® Ver 13.1e FEM software was used. Criteria that were set in order to determine geometry and topology of the test frame were easy everyday use and manipulation, as well as ensuring a safe and proper load test by achieving sufficient stiffness. Therefore, assembly-disassembly structure, with a few elements that change their position depending on the element being tested, impose itself as a reasonable solution. In consideration of aforementioned, test frame consists out of two types of structural elements:

- elements that are present the whole time during tests, no matter which type and size of structural element is tested,
- elements whose existence and position depend on the test configuration.

This characteristic of the test frame allows a great variety of structural elements to be tested, such as.

- beams (max 5.0m long),
- columns (max 2.5m high),
- slabs (max dimensions 4.2x4.2m),
- walls (max dimensions 4.0x3.0m).

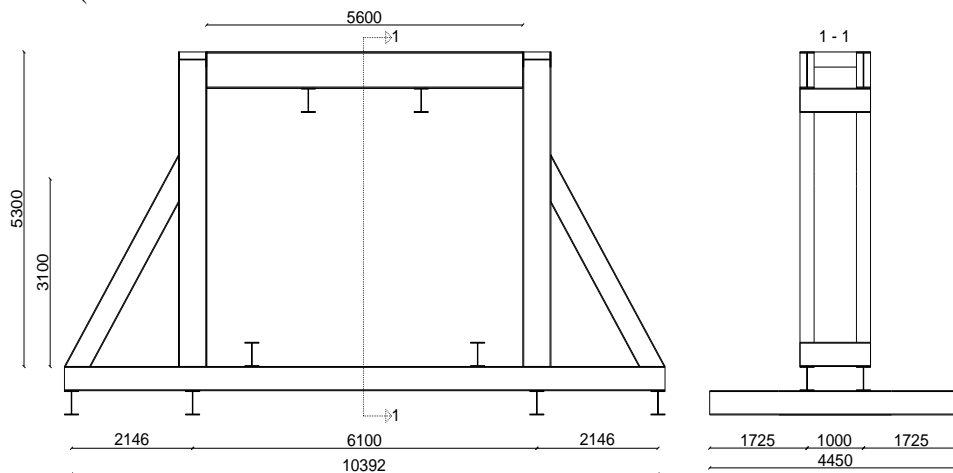


Figure 1 - General disposition with main dimensions of the test frame

In reference to above mentioned, five frame configuration are set apart: two configurations for beam tests and one configuration for each of the remaining test elements. Two configurations for beam test differ in a way that one configuration represents load test with one hydraulic jack and the other configuration represents load test with two hydraulic jacks.

2 FEM MODEL OF TEST FRAME STRUCTURE

Test frame was modelled in AxisVM® Ver 13.1e FEM software. The structural analysis was conducted in two variants:

- variant where all structural elements are presented as line FE,
- variant where all structural elements were modelled as shell FE.

In both variants, building material that was used for modelling is steel material of S355 quality with yield/ultimate strengths $f_y/f_u=355/510\text{MPa}$, modulus of elasticity of $E=210\text{GPa}$ and mass density $\rho=7850\text{kg/m}^3$.

FEM models are composed of:

- line beam FEs,
- special link FEs,
- special support FEs,
- surface shell FEs,

where all of the listed finite elements have three translational and three rotational degrees of freedom.

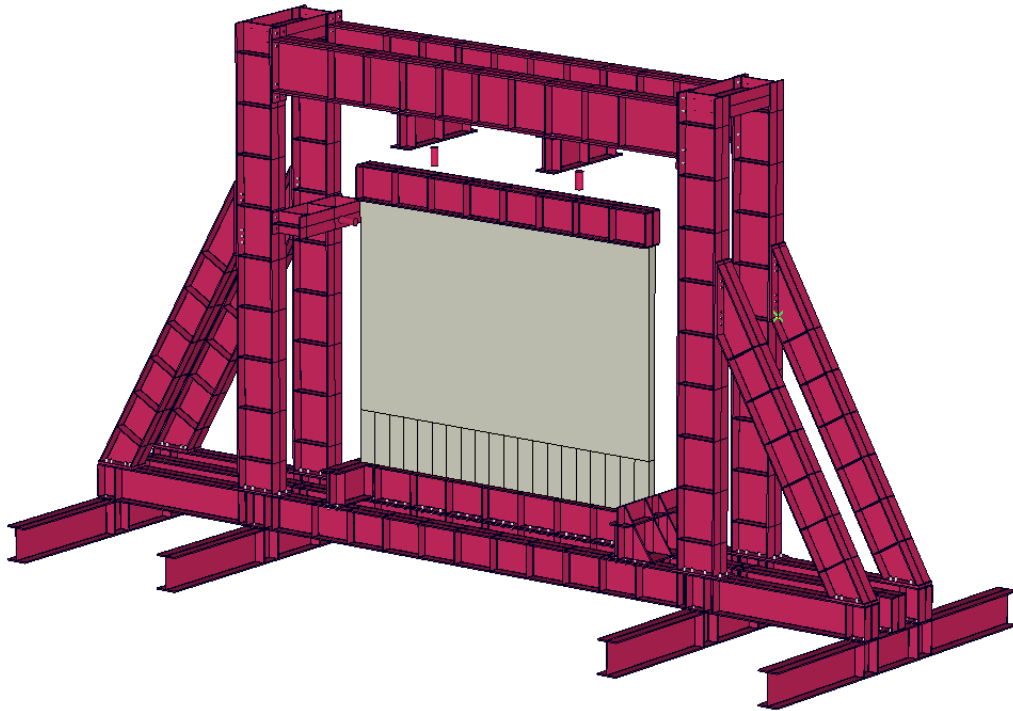


Figure 2 – FEM model of the test frame: wall test (shell FE variant)

2.1 LINE FE-VARIANT

Variant where all structural elements are treated as line FEs was used for preliminary determination and definition of cross-section properties of test frame elements. Furthermore, this model variant provided values of internal forces at end points of line FEs, which are necessary information for calculation of joints.

In figure 3 are given model information and analysis parameters of line-FE variant (test frame configuration set for wall test).

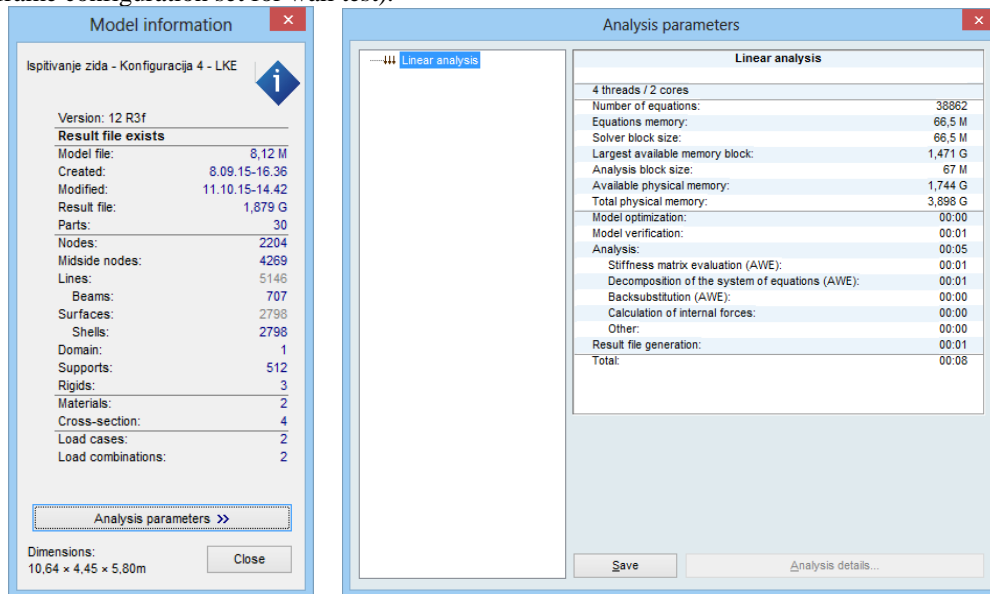


Figure 3 – FEM model data (line FEs variant)

2.2 SURFACE FE-VARIANT

Based on the results of the line FE variant, in the second variant all structural elements were modelled as shell FEs. This model was used to determine stress values in structural elements with greater precision.

In figure 4 are given model information and analysis parameters of shell-FE variant (test frame configuration set for wall test).

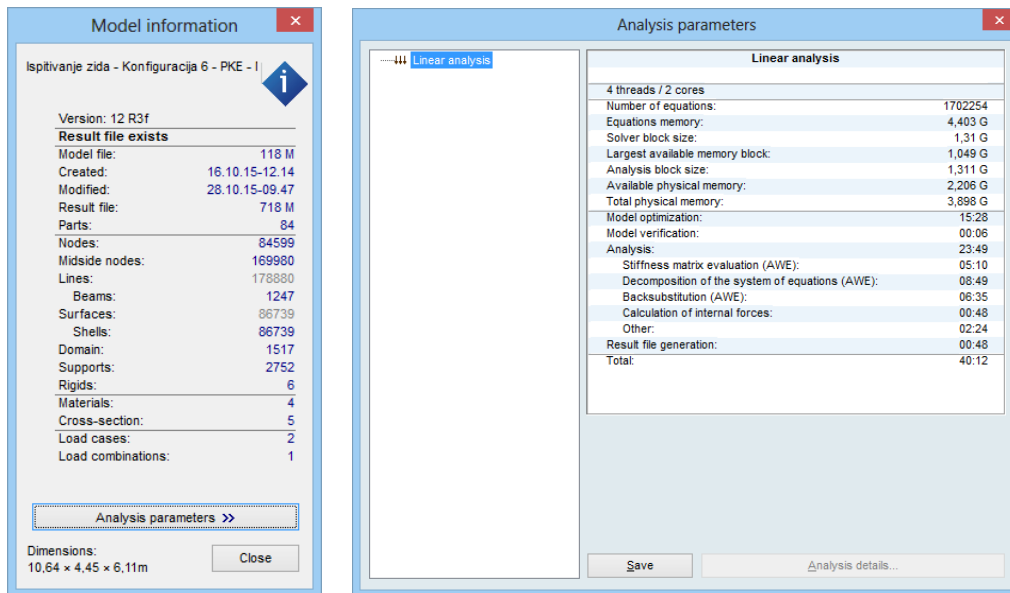


Figure 4 – FEM model data (surface FEs variant)

2.3 MODELING OF CONNECTIONS OF THE TEST FRAME STRUCTURE

In order to describe and take into consideration the eccentricity of joints as real as possible, special link FEs were used. Values that are necessary to define for analysis are values of stiffness components for every of the six global or local directions (three translational and three rotational). Elements of the test frame are connected with link FEs, whose values of stiffness components are 10^{10} kN/m for translational and 10^{10} kNm/rad for rotational directions. This means that all joints were considered as perfectly rigid.

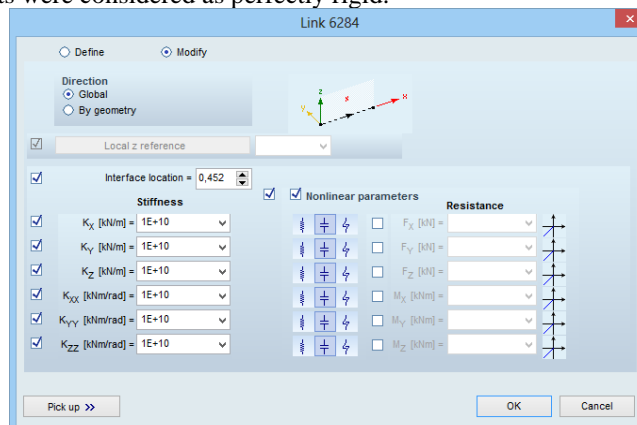


Figure 5 – Parameters of the link FEs for modeling of connections of the test frame

Connections between the test frame and elements being tested are also modeled using link FEs. However, in this case values of stiffness characteristics differ from previous case. Here, in regard to frictions that occurs in contact zone of the test frame and the element, values of stiffness components are defined as "low" values (for example, 10^2kN/m for translational and 10kNm/rad for rotational directions in the case of beam test). Exception is stiffness component of the global Z-direction (10^{10}kN/m).

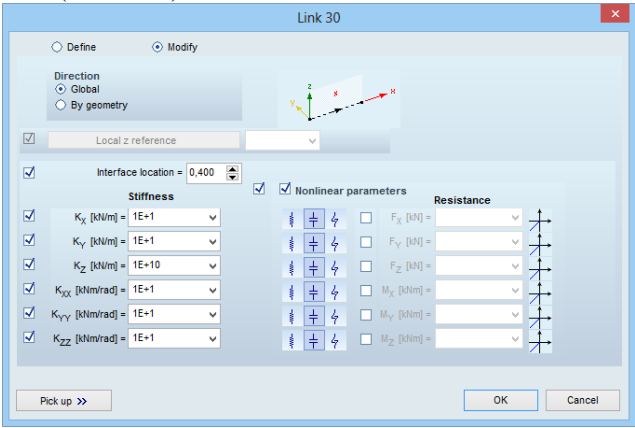


Figure 6 – Parameters of the link FEs for modeling of connections between the test frame and tested structural element

2.4 MODELING OF SUPPORTS OF THE TEST FRAME STRUCTURE

Supports were modelled as line or surface supports with stiffness characteristics that simulate simply supported beam or shell along the entire length, depending on the variant being analyzed. Furthermore, to achieve the state where supports are active only in compression, nonlinear characteristics were assigned to each direction. As a consequence, nonlinear analysis had proved that regardless of the test load and frame configuration, support reactions due to test load would always be equal to zero.

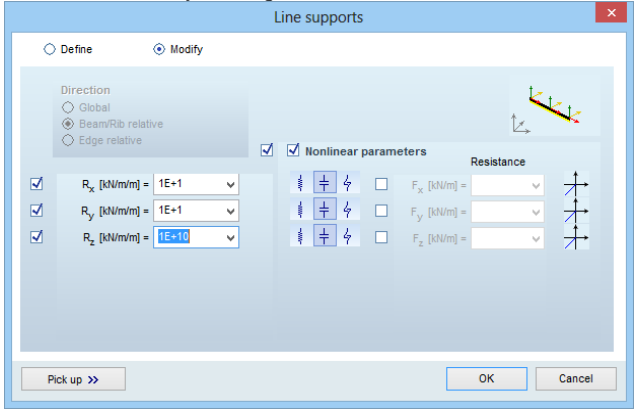


Figure 7 – Parameters of the line support

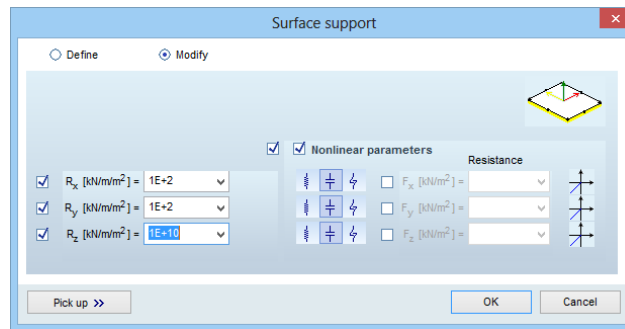


Figure 8 – Parameters of the surface support

2.5 MODELING OF LOADS AND HYDRAULIC JACKS

Load due to self-weight is automatically assigned by the software itself, depending on the material and element geometry. Hydraulic jacks were modelled as beam FE in both variants, with hinges around y and z local axes at both ends. In the line FE variant, hydraulic jack is connected with the frame in one node with the special rigid link FE. However, in shell FE variant hydraulic jack is connected with frame in four nodes with four special rigid link elements. The result is stress distribution from one node to four nodes. The load which simulates the test load of hydraulic jack was introduced as a fault in length of the beam FE.

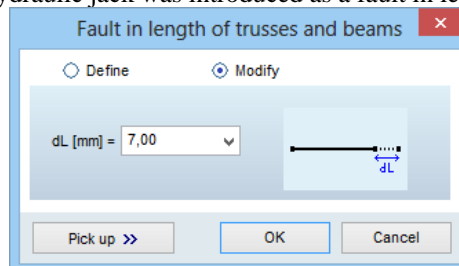


Figure 9 – Modeling the load of the hydraulic jack value (by fault in length)

3 CONCLUSION

The numerical comparison between the line and shell FE variant has been carried out. Yield strength of the steel in line FE variants not reached, whereas results of the shell FE variant confirm that the highest stress value is almost three times higher than the steel ultimate strength. Therefore, shell FE model should be used for numerical analysis, because it offers a possibility to introduce minor changes to the model that affect a great deal in results.

The assumption that vertical reactions during the load test will remain the same as the reactions due to self-weight of the frame and element being tested could only be proven by the use of nonlinear analysis. Here, nonlinear characteristics should be assigned to each support direction in a way that supports are supposed to be active only in compression. After a certain number of increments in nonlinear analysis, value of vertical reaction due to the load test converge to the value due to self-weight of the frame and specimen.

For the purpose of taking into account the eccentricity of internal force transfer in connections, special link FEs should be used.

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Doncho Partov¹, Milen Petkov² and Radoslav Nikolov³

ISTORIJA ORTOTROPNIH PLOČA KOD MOSTOVA

Rezime:

S-25

Razvoj čeličnih mostova nakon Drugog svetskog rata karakterišu: novi konstruktivni sistemi zatvorenih čeličnih nosača, novi statički sistemi i metode. Osim toga, glavni nosači, kolovoz i horizontalni nosači se statički i konstruktivno povezuju u monolitnu strukturu, zato se ostvaruje bolja raspodela unutaršnjih sila u konstrukciji. U ovom radu prikazan je istorijski razvoj teorije ortotropnih čeličnih ploča Wilhelm Cornelius-a. Primena ove teorije u Bugarskoj u periodu 1976-1992, omogućila je izgradnju nekoliko jedinstvenih po veličini čeličnih mostova sa ortotropnim ploča.

Ključne reči: ortogonalno – anizotropna ploča, ortotropna ploča

ABOUT THE HYSTORY OF ORTHOTROPIC BRIDGE DECK

Summary:

The development of steel bridges, after world second war, is characterized by: a new constructive forms-steel roadway closed sections, new static systems considering the roadway as a plate rather than a beam grill, new computational methods going from rod statics to statics a continuum and determine the internal forces of the roadway as internal forces of an orthotropic plate. Moreover, the main girders, roadway and horizontal links connect statically and constructively in an entire monolithic structure. Thus, the construction follow better the really distribution of internal forces in steel structure. Loading possibility of high-quality steel material is well used, with which the economic effect of the construction increases significantly, compared with the riveted old bridges. In this paper the historico-logical development of the theory of the orthotropic steel plate established by Wilhelm Cornelius are described. The application of this theory in Bulgaria in the period 1976-1992, allowed in Bulgarian bridge construction to build several unique for its size steel road bridges with steel orthotropic decks.

Key words: orthogonal - anisotropic plate, orthotropic steel plate

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1 INTRODUCTION

In the second half of 1945, at close of World War II all Central Europe and the European part of the USSR lay in ruins. Only in West Germany from 24,380 railway bridges were destroyed or damaged 3149, i. e. 13%. From owns of the Ministry of Railways 4827 road bridge, the 575 i.e. 12% were destroyed. From 22 railway bridges and 20 road bridges on the Rhine, along the entire length between the Swiss and the Dutch border, at the end of the war, was none. Were destroyed and all 11 bridges on the river Weser. From 24 bridge on the River Main, 23 were destroyed and from 34 bridges on the Danube River – 22 ware destroyed. Thousands more road bridges and much of spectacular bridges on the roads lay in debris. Similarly was the situation in other countries over which passed the storm of destructive war[5]. Of vital interest to these countries was fast construction of road and railway network. In (fig. 1) is visible on the process of reconstruction of railway bridges in West Germany during the period 1945 – 1955[6].

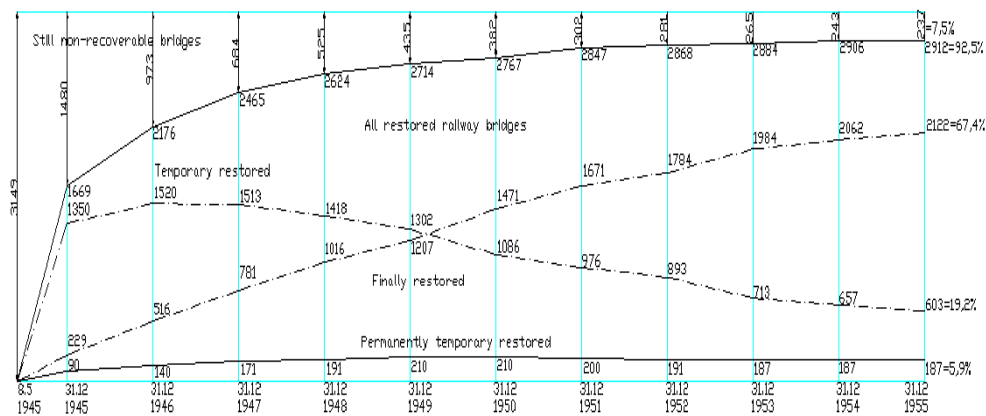


Fig.1 Diagram of remainig of railway bridges in West germany for period 1945-1955, (Damage or destroyed railway bridges on 8 May 1945 is equally around 3149)

The dire economic situation after the war and lack of the building materials necessitated to seek most - effective ways of extending the life of existing steel bridges, passed the age limit whose condition due to inadequate maintenance during the war years was bad. For steel bridges, age limit is 60 and for massive is 90 years. Hopeless economic situation forced to decisive savings in steel and other building materials. This gave new impetus to the theoretical, constructive and economic development in all fields of engineering. New directions in the development of steel bridges were expressed; a) in amending the existing rules for loading and calculation of steel railway bridges and adjusting them to the actual work of construction; b) in finding new forms static structural systems and new computational methods that cover more - better distribution of forces and allow in increased safety factor to use existing reserves in the bearing capacity of the structure and material; c) in improving the quality of construction materials and creating new kinds of steel; d) perfecting the art of welding and its wide use in the construction of road and railway steel bridges; e) in the creation of new connections.

2 NEW COMPUTATIONAL METHODS AND CONSTRUCTIVE FORM

2.1 RAILWAY BRIDGES WITH OPEN ROAD LINE

In order to use the fully load-bearing capacity of the steel, and thereby to increase the economic effect of the structure it is necessary to know well the actual distribution of the forces in parts of the construction, and their deformation. Only then can a proper and appropriate distribution of the material in the construction. To simplify decision repeatedly statically indeterminate space bridge system, it is usually seen as composed of separate planar systems, which do not interfere with each other. This proposition is completely arbitrary and not consistent with the requirement of knowledge of the actual operation of the bridge construction. Measurements on existing bridges showed that the loading of the longitudinal beams depends on the deflection of the main beam and vice versa [14,24]. In fact, between the main support system and road grill there is a significant interaction. To reduce the overhead road gridded railway bridges with open roadway, usually they predicted disruptions roadway. Striving today, however, with appropriate measures to ensure full cooperation between the longitudinal and main beams, which main beams are alleviated considerably. This requires longitudinal beams to construct a continuous beam of vertically protruding props, not a partially bent beams as before. The vertical pass the supports of the continuous longitudinal beam is a result on the one hand, to the elastic bending of the crossbars on which they are based, and on the other side of the elastic displacement (deflection) of the main beams, to which they are attached crossbars. The exact study of the longitudinal beams as continuous beams on elastic feeding props to view the interaction between the main and longitudinal beams is relatively heavy. C. Popp [31], developed for this purpose an approximate method that is used in the German prescriptions for calculation of steel railway bridges. The influence of the elasticity of the crossbars on the moments of the longitudinal beams is so greater as the distance between the crossbars is - in comparison with the supporting distance between the main beams, and the resistivity of the longitudinal beams against bending is - large in comparison with that of the crossbars. The influence of the displacements of the main beam on the moments of the longitudinal beams decreases with increasing distance of the support of the main beams. For large supporting distances it can be ignored. In (fig.2) are compared influence lines of the moment of the longitudinal beam in point 2 of the present one railway bridge spans 10,0 meters. Longitudinal beam was once viewed as a continuous beam on unmovable supports, then on the elastic supports, ignoring displacements of the main beams, and then under consideration of displacements the cross and the main beams. From the comparison of the three lines of influence seen the size of the impact of displacement on the retaining moment in point 2 of the longitudinal beams. The longitudinal beams can be regarded as continuously if indeed their continuity is established. This requires their upper and lower flange to contact with tensile and compressive flange plate and also to lie down on a console that would take the vertical load of the longitudinal beam and transmit it to the cross beam.

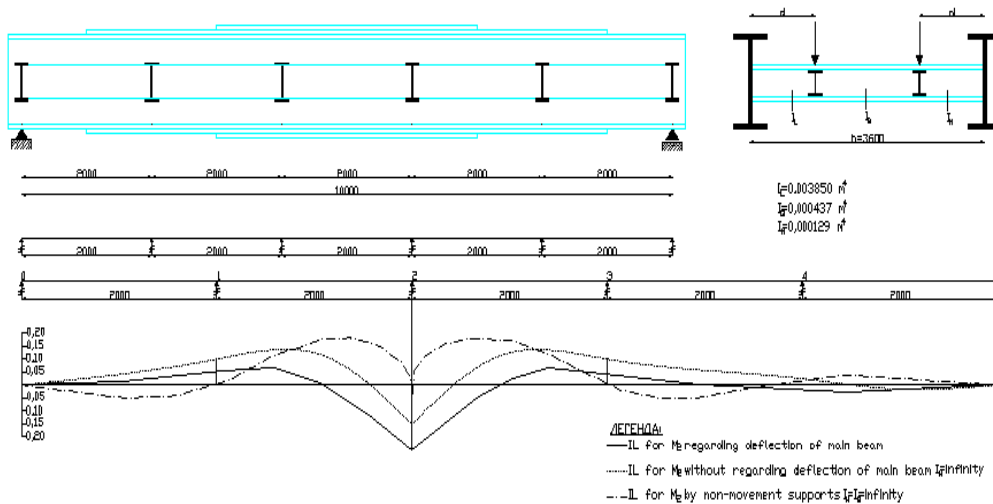


Fig.2. Railway bridge with span 10m. Comparing of Influence line for moment in longitudinal beam in point 2

3 ROAD BRIDGES AND RAILWAY BRIDGES WITH CLOSED ROADWAY

3.1 ORTHOTROPIC PLATE

In road bridges when roadway consists of road grid (longitudinal and transverse beams) and lying on her concrete slab, longitudinal and transverse beams are calculated as separate beams, independent of each other. The plate was seen also as a standalone bearing element. independent each other. Every part of the roadway performed only one function, without taking into account the interaction between the individual parts. Some progress was made, when - later reinforced concrete slab was joined by dowels with its supporting road grill. Thus ensuring collaboration between the steel beams and reinforced concrete slab, that with its effective width increased significantly moment of inertia of beam. Longitudinal and cross situated beams reviewed it independently of each other, however plate already performed two functions - took acting immediately on it and load he betrayed on its supporting beams, and also worked as an integral part of the road grid (such as upper belt of longitudinal and transverse beams). The next decisive step towards – to more accurate coverage of the forces in roadway and to move closer to the actual work of construction was made, when the road began to be seen as a beam grid. Again the plate was used as the upper belt of the beams. The calculation was conducted using a "rod statics" i.e beam grid was seen as a system composed of individual bars (beams), without taking into account the spatial effect of the plate. In fact but the roadway, consisting of orthogonally intersecting longitudinal and transverse beams, whose upper belts are connected to the deck plate, is forced to follow the deformations of the main girder and thus to work as an integral part of the main supporting system. Most decisive step in

bringing and adapting the bridge construction to the actual distribution of actual distribution of internal effort, was made when determining the internal forces of the roadway was reduced to determining the internal forces of an orthogonal anisotropic plate i.e. when they went from bar to statics of continuum (continuous medium)[3,4,21,22]. Resistivity against bending of the roadway transverse to the axis of the bridge is much greater than resistivity its longitudinal axis. On this particular different elasticity of roadway in cross and longitudinal direction follow the explanation of the anisotropy of the plate. A classic example of transition from static rods to static of the continuum is the problem of investigated of compression of upper belts of open grate bridges (with no upper stability link). F. Engesser resolve this problem with unique elegance, accepting that the resistivity of bending free-frames (two verticals associated with crossbeam) on which compression belt rests horizontally distributed evenly over the entire length of the corresponding field. Thus Engesser reduce the problem of elastic brace to specific points compression rod to solve the beam of elastic foundation (continuum). The study of F. Bleich the same problem proved by - later that at considerable distances between the transverse frames examining belt beam of elastic foundation leads to practical accurate results. In 1942 research multisectional grate systems with continuous belts (nodes are joints) was also reduced to solving a continuum as filling rods mentally replaced with a continuous wall with constant density [1,4, 25]. The calculation results were confirmed by model tests. All results of the study of a system as a continuum so on - close to reality as replaced by continuum elements are - thick distributed, i.e. as higher is the statics indeterminations. Its knowing, that the more elements comprises a structure, so its security is more high. Removal or destruction of an element in any case means destruction of the entire structure be sure you do the functions of this element will be borne by neighboring elements. For example, the security of the chain which consists of separate vertebrae is much more – less, than that of the wire-rope, which comprises a plurality of separate fibers. The new development of statics will result in -secure and economical structures, each part of the bridge construction dimensioned with different safety factor depending on the importance and her appointment. When the roadway is configured as an orthogonal - anizotropic plate, abbreviated as "orthotropic" plate, carried and at the same time used in the calculation of the bridge cross-section comprises, in addition the cross-section of the main beam, but also the cross-section of the longitudinal secondary beams and the associated deck plate. Orthotropic plate forms the upper belt of the main beams. Thus all parts of the structure, except in implementation of their immediate destination, are included in the joint execution of the functions of the main structure. For example, the deck plate works as a part of the main supporting system, as an integral part of orthotropic plate (flange of the longitudinal and transverse beams), and then as separate bearing element who takes direct current load and carry to the road grid. For that reason, loading resistance of construction and the material used is the most appropriate. The constant load (as by composite steel-concrete beams) is crucial to the economy of the bridge, especially in supporting large distances. These disadvantages of reinforced concrete roadbed drop in steel roadway, which consists of 12 - 16 mm. thick steel road plate associated with its supporting orthogonal grill of longitudinal and transverse ribs. On the steel plate is applied asphalt, 5 cm thick, whose weight is only 1,25 kN/m². Naturally in the steel roadway due to the nature of orthotropic plate, longitudinal and transverse ribs form a thick beam grid. With the construction of the steel roadway as orthotropic plate steel road bridges have entered a new stage of development. Dead load is reduced many times. Connection between orthotropic plate and main beams in a solid main supporting structure allowed to build facilities, which by their lightness and elegance courage

far outweigh known until now borders. For example, a full wall beams was thought that 100 meters is max. and more economical span. Today easily transferred supporting distances over 260 meters. Fig.3 is the cross section of the suspension bridge Köln – Mülheim, river, Rhine in Köln, where can be seen the structure and the steel orthotropic deck plate. In this mode of embodiment is considerably reduced construction height in comparison with the old embodiments, further savings in steel is huge.

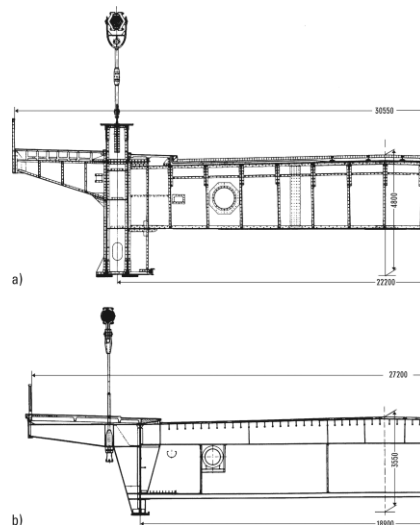


Fig. 3 Suspension road bridge over the river: Rhine, between Köln-Mülheim, with supporting distances 85.0 + 315.0 + 85.0 m; Sections: old (a) and new (b) bridge

4 GENERAL ON ORTHOTROPIC PLATE

The development of steel bridges, as we have seen, is characterized by a new constructive forms - steel roadway closed sections, new static systems - considering the roadway as a plate rather than a beam grill - with new computational methods - going from bar statics to statics a continuum and determine the internal forces of the roadway as internal forces of an orthotropic plate. Moreover, the main girders, roadway and horizontal links connect statically and constructively in an entire monolithic structure. Thus the structure is better actual allocation of domestic efforts, supporting the possibility of high-quality steel material is - well used, with which the economic effect of the construction increases significantly compared with the old bridges. Figure 3 shows the old roadway built in 1929, and suspension bridge Köln - Mülheim and the new suspension bridge built in the same place in 1951. Comparing the two carriageways shown great reduction of dead load achieved in the last twenty years. Pavement, cement bed isolation and concrete filler at the old bridge weigh on average 4,30 kN/m² versus 1,25 kN/m²-weight of 5 cm thick asphalt at the new bridge. The thickness of the steel plate varies from 12 to 16mm. In particular for structural reasons plate thickness design of the roadway is reduced to finding the most - favorable distribution of ribs and dimensioning. These ribs corresponding to the longitudinal and transverse beams of the normal road grill, as it is

shown in Fig. 3 for old and new bridge Köln - Mannheim, however they will have to adapt to their functions in orthotropic plate. When designing the roadway as valid plate are two principles, that lead to an entirely new system reinforcing ribs where used less material. The first of these principles is: way of load from the moment they take over a driveway to his surrender to the main structure, should be the shortest. On (fig.3a) the load, marked as dead load or movement load - through longitudinal beam is transferred onto the cross and from there on main beams. The distance between joists here is very large, the path to transmit the load on the main beam is long, the road is heavy grill. Moreover, the use of roadway material will be fully utilized when the entire plate is included as zones of longitudinal and transverse ribs. Therefore, the distance between the ribs has to be determined so as to be equal to the supporting width of the plate. (Fig. 3b). The second principle is that the concentrated loads must be distributed, if possible - much more on of the roadway. This can be achieved only if all the longitudinal and transverse ribs associated with lying on their road slab is continuous. The design or constructive involvement of joints will restrict load distribution. For transporting the load, which is located parallel of an abutment line, as a dead load, people, road cars etc., need a road structure have resistivity against bending only in the transverse direction. For concentrated loads, however, is required resistivity across from bending in all directions, as in isotropic plate. In reality, bridge loads are as distributed and concentrated, however, due to the large width of the bridge is dominated by the distributed. The optimum road grill must have resistivity against bending in all directions, but in the transverse direction it should be much more - greater than longitudinally. At the bridge Köln - Mülheim relationship of resistivity in the longitudinal and transverse direction is 1:20, while the old version - 1: 8[4]. To find the most - favorable distribution of the ribs and the most - their proper cross-section, it is necessary to draw diagrams for weight consumed in the road lane steel. Usually are specify the width of the bridge, the load and thickness of the plate, but varies with the distance between the longitudinal and transverse ribs, as well as their widthwise cross-sections. After these diagrams are calculated and cause, is readily determined at advantageous grill reinforcing ribs. A similar study is generally only possible if there is a calculation method which, when coverage of the actual distribution of forces and complete depletion of reserves static structure, is connected with practical affordable computing. One advantage of the theory of orthotropic plate is that it enables you to use a reserve tonnage of construction, which with known methods of calculating beam grill not taken into account. This reserve lies in the resistance of the road surface against torsion, which as a result of the eccentric arrangement of the plate relative to the parallel plane passing through the center of gravity of the overall cross-section significantly. Effective resistance to twisting roadway can be determined only on the models. To bridge Köln - Mülheim were conducted laboratory tests on one element of the actual roadway responsible manner closer to the actual load of the bridge[21]. Measured were extensions and deflections of the structure, after that in which a judge for stress and load bearing capacity of the slab was made. While isotropic plate resistivity against bending and twisting resistivity against are equal, the model resistivity against twisting accounted for approximately: $0,3\sqrt{D_x D_y}$ where D_x and D_y are resistivity against bending along the axes of x and y. The model was made of steel St 37 in thickness of the bridge deck = 1 cm, distance between the longitudinal ribs between 30 cm and 100 cm cross. The plate was loaded in two adjacent right along the entire width. The load has been passed on through the slab pads hard tire size 16/30 cm. corresponding to the contact surface of the wheels of the motor car, whose pressure according

to the norms of load is 0,0833 kN/cm². In testing the slab load 0,312 kN/cm² was measured at - large vertical displacement of the plate equal 0,24 cm, while she remained completely elasticity. Load 0,782 kN/cm² was measured at but less plastic displacement 0.8 cm. And elastic movement - 0.32 cm. In this ultimate load was not reached. On the assumption that the limit load Pult. corresponds to the measured pressure 0,312 kN/cm², the available security plate would be 0,312: 0,0833 = 3.75. It shows how great the tonnage of the plate. As the results of laboratory studies prof. Klöppel[21] came to the conclusion, that there is hardly another support system that better suits the character and qualities of steel, both anchored on four edge steel plate. For the special advantages and competitiveness of orthotropic plate over other systems roadways testifies its mass application in recent years. For the first time orthotropic plate is used in calculations of W. Cornelius of the company M.A.N. When 1945. in several mobile road bridge, and then at a large street bridge in Mannheim over the river Neckar. As announced in 1948 competition for the bridge over the river Rhine, on the site of the destroyed bridge Köln – Mülheim, were presented 39 projects of which 20 hanging bridges. Only company M.A.N. proposed suspension bridge with a steel roadway with orthotropic deck developed by W. Cornelius. This novelty in such large the facility (supporting distances 85,+ 315m + 85m.) meet with redundancy, both by the Commission and by other participating companies in the competition. After a pivotal, choice falls on orthotropic plate. How far along development end after 1948, indicates the fact that the majority of steel bridges built after 1950 have roadway with orthotropic deck. From presented in 1955 projects of competition for a fourth bridge over the river Rhine in Köln, 38 of them offer steel bridges with a steel roadway as orthotropic plate.

5 THEORY OF THE ORTHOTROPIC PLATE

5.1 THE THEORY OF MAKSYMILIAN TYTUS HUBER

Orthogonal - anisotropic plate is characterized in that its resistivity against bending in two mutually perpendicular directions is different and her resistivity against twisting can be arbitrarily large. We can look on her as composed of an isotropic plate (road, steel or concrete) and bracing with longitudinal and transverse ribs (beams) (Fig. 3). Orthotropic plate theory has developed more than 87 years of prof. M. T. Huber from University of Lvov in connection with solving the problem of the cross-reinforced concrete slab[15-19]. It is built on the same assumptions on which is built the theory of isotropic plate, namely: a) the thickness of the plate is small compared to the length and width, as a result of which, superimposed on the normal to the median plane of the plate sections remain, and then bending the plate on a straight line which is perpendicular to the deformed middle surface; b) movements perpendicular to the plane of the plate are small compared to the thickness, due to which crop up in the middle surface of the plate angular and linear deformation can be ignored. This leads to the assumption that the average surface of the plate is undeformed, i.e. it is unstretchable (movements there are equal to zero); c) the linear deformation of the plate thickness is equal to zero. XY coordinate plane right-oriented coordinate system coincides with the center of undeformed, plane of the plate, which is also plane of symmetry. In the bridge deck longitudinal and transverse ribs are situated on the underside of the plate and its middle plane is no longer plane of symmetry (Fig.4). The average plane of the isotropic plate (deck plate) does not match the parallel plane passing through the center of gravity of orthotropic plate (deck plate and ribs) and the stresses

in the median plane of the isotropic plate are zero. The premise that the average plane of the isotropic plate is not stretchable linear and angular its deformations are zero, it is not executed [7, 34].

However, the theory of Hubert used to determine the internal forces and orthotropic plates, according to Fig.4 as are common in bridge construction. For completeness theory of Hubert for orthotropic plate will be describe briefly. We know that in the second decade of the 20th century, according [23], the calculation of reinforced concrete slabs had been based on a simple structural model, essentially based on beam theory. In the method attributed to F. Grashof [13], for example, a rectangular slab is divided into two orthogonal strips and the respective deformations and internal forces in the slab strips in the in x and y directions calculated at the points of intersection based on the condition of the equality of the deflections. The torsion in the slab is neglected in this method. On the other hand, tests on reinforced concrete slabs with the same amount of reinforcement in the x and y directions confirmed the validity of Kirchhoff's plate theory for homogeneous and isotropic slabs [20]. However, it could not be applied directly to reinforced concrete slabs purely for the reason that the bending stiffness of a reinforced concrete slab, depending on the reinforcement, can assume very different values in different directions [15]. This is why Huber, in 1914, developed the general theory of reinforced concrete slabs reinforced in both directions and derived the differential equation for their deflection $w(x,y)$ [15]. In February 1929 he held a number of lectures at the Swiss Federal Institute of Technology in Zurich and in that same year these appeared in the form of a monograph in German, published in Warsaw [16]. Witold Nowacki drew attention to the origins of the theory of the orthotropic plate as early as 1951[29]. After Huber has talked about fundamental but also critical points in the theoretical foundation of tests in reinforced concrete construction, he derives the differential equation of deflection $w(x,y)$:

$$D_x \frac{d^4 w}{dx^4} + 2H^4 \frac{d^4 w}{dx^2 dy^2} + D_y \frac{d^4 w}{dy^4} = p(x, y) \quad (1)$$

due to load $p(x,y)$ with the help of the energy principle [16]. Applied to orthotropic road decks on steel bridges, Huber's differential equation contains the plate bending stiffness transverse to the axis of the bridge (bending stiffness of the road deck plate) D_x , the plate bending stiffness in the direction of the bridge axis (bending stiffness of the longitudinal stiffeners) D_y and the effective torsional stiffness:

$$H = 0,5(4C + \mu_y D_x + \mu_x D_y) \quad (2)$$

for thin, homogeneous-elastic but orthogonal-anisotropic plates. Of course, Huber's theory applies to all thin, homogeneous-elastic and orthogonal-anisotropic plates such as steel or reinforced concrete. In equation (2): $\bullet 2C$ is the pure torsional stiffness, $\bullet \mu_x$ is the lateral strain due to normal stress in the x direction; $\bullet \mu_y$ is the lateral strain due to normal stress in

the y direction. In the isotropic case the plate bending stiffnesses or lateral strains in the two directions are equal, i.e. $D_x = D_y = D$. and $\mu_x = \mu_y = \mu$. The pure torsional stiffness in this special case is: $2C = D(1 - \mu)$; and entered into equation (2) this results in the value $H = D$, which means that Huber's differential equation (1) is converted into Kirchhoff's differential equation for plates [21].

$$\frac{d^4 w}{dx^4} + 2 \frac{d^4 w}{dx^2 dy^2} + \frac{d^4 w}{dy^4} = \frac{p(x, y)}{D}; \quad (3)$$

Differential equation (1), which Huber derived in the *Journal Der Bauingenieur* [18], was used by Cornelius in his version of orthotropic plate theory [3]. Therefore, structural steelwork borrowed from reinforced concrete and during the 1950s and 1960s encouraged a far-reaching development of the theory of the orthotropic plate – driven by the technical progress in steel bridgebuilding (Figure 8a) and aircraft construction (Fig.4a,4b).

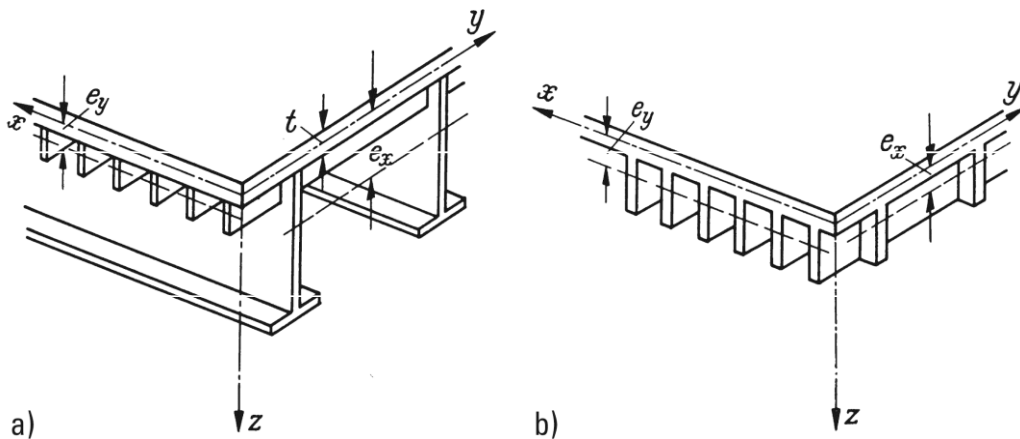


Fig.4. Plate cross-sections around 1960; a) flat steel plate (bridge-building), b) integral plate (aircraft construction) [36, p. 35].

5.2 THE METHOD OF WILLIAM CORNELIUS

Only in 1945, Wilhelm Cornelius suggested theory of orthotropic plate to determine the internal forces and displacements of practically occurring carriageways for linear and distributed loads, giving solutions in the form of Fourier rows. This path pics structural and economic development of the steel bridge was cleared. The study of the steel roadway as orthotropic plate reduce the use of ready-made formulas, allowing case to draw diagrams and weights them to determine the most economic - favorable distribution of longitudinal and transverse ribs. According[23], six months after the new Köln – Mülheim suspension bridge was opened, Klöppel's student Wilhelm Cornelius revealed the theoretical basis behind his recipe for success [3]. Cornelius consciously completed the transition from member to continuum analysis. It was important that Cornelius recognised the genesis of the loadbearing systems as an organic development from discontinuum to continuum, so to speak, which he also observed in reinforced concrete construction. Such a change in the modelling of loadbearing structures, from member to continuum analysis, was not new because “even the progress in reinforced concrete construction replaced, for example, the previous structural design of lattice structures by the structural design of shells and folded plates” [3]. Cornelius' work was based on the plate theory developed by Maksymilian Tytus Huber[15-19] for reinforced concrete construction. He solved Huber's differential equation for orthotropic plates for various types of plates such as a steel plate with a group of rolled sections and a grillage in conjunction with a concrete slab, i.e. he specified integral functions for the deformations and

internal forces and tabulated the constants for the integral functions for common types of loading.

5.3 THE METHOD OF GUYON & MASSONNET

It was in 1946 that Yves Guyon presented his theory of a zero-torsion grillage based on Huber's orthotropic plate theory [12]. Charles Massonnet (1996) generalised Guyon's [27], i.e. at the same time as the grillage theories of Leonhardt/Andrä and Homberg. Massonnet devised graphs for simply supported beams, constant moment of inertia and identical moment of inertia for all main girders. He therefore created another method for the simple analysis of grillages. Over the years 1955-1960 Konrad Sattler [32], extended the graphs to cover main girders with a varying moment of inertia, perimeter and inner beams with different moments of inertia and any structural system [20]. Finally, in 1966, the monograph of Bareš and Massonnet appeared [2], which embraced all the findings based on the Guyon/Massonnet method and delivered a series of new ideas and experience.

5.4 NUMERICAL METHODS FOR DESIGN STEEL BRIDGES WITH ORTHOTROPIC PLATES

According [23] the publications of Guyon, Massonnet, and Cornelius were followed by numerous further contributions to the theory of orthotropic plates – the papers of Trenks, Mader, Giencke, Klöppel and Schardt to name but a few. Whereas in Huber's continuum the stiffeners must be positioned symmetrically about the central plane of the isotropic deck plate, Alf Pflüger [30] grouped together the isotropic deck plate and the "distributed" stiffeners eccentric to this into a continuum that has been named after him. Ernst Giencke also used the same theoretical basis for his work on the fundamental equations for orthotropic plates with eccentric stiffeners, which was published in 1955 [7]. Mader [26] and Giencke [8] dealt with the discontinuity of the cross girders and considered the orthotropic steel bridge deck as a composite system consisting of Huber's continuum and the discontinuous cross girders below. According [23] in a later paper, Giencke analysed the hollow-rib plate, – a variation of the orthotropic plate whose success first came in the mid-1960s as the Krupp company took on a series of large bridges simultaneously and was forced to rely on large-scale production with maximum standardisation. At the same time, the steel industry switched the production of lightweight sheet piling sections from hot- to cold-rolling, which rendered possible the standardisation of deep trapezoidal profiles with transverse beams at spacings of up to 5 m. This technical progress led to the orthotropic bridge deck so typical these days: "Automatic welding and assembly plants for welding hollow ribs to deck plates rendered possible good-quality weld seams with good penetration for the typical solution. And the arrangement of the close-tolerance longitudinal rib penetrations through cut-outs in the cross girder webs, with adequate room for compensating for the tolerances of the trapezoidal profiles, plus the design of the longitudinal rib splices ensured details not susceptible to fatigue." [28]. The forerunners of the hollow-rib plate perfected in the 1960s had been produced many years before. Klöppel and Schardt achieved a graphic synthesis of the Huber [18] and Pflüger [29] continuum theory for anisotropic shell structures with the help of matrix calculations [22]. In 1960 Hans Schumann published his dissertation on the analysis of orthotropic rectangular plates [33], supervised by Pflüger. Schumann's theory, formulated in the language of matrix calculations, takes into account both the eccentricity and the discontinuous arrangement of the longitudinal

and transverse stiffeners. In his summary he notes that the matrix formulation of his theory would simplify the programming of calculations for program-controlled automatic electronic calculators [33]. The consequential matrix formulation can be regarded as equally important because it considerably simplified the transformation into algorithms for computer programs. Giencke was the driving force behind this development. In 1967 he managed to formulate a finite method for calculating orthotropic plates and slabs [10]. Three years later, Giencke, together with J. Petersen, published a finite method for calculating shear-flexible orthotropic plates [11], which at that time were being used more and more in building for sandwich constructions.

In the article not mentioned other methods related to the calculation of orthotropic road bridges named after : Pelikan W., Eßlinger M.: Die Stahlfahrbahn. Berechnung und Konstruktion. MAN-Forschungsheft 7, K.G. Kieser, Augsburg 1957; Homberg H., Trenks K.: Drehsteife Kreuzwerke. Ein Handbuch für den Brückenbau. Springer-Verlag, Berlin/Göttingen/Heidelberg 1962; Mangus, A., Shawn Sun "Chapter 14 Orthotropic Deck Bridges", Bridge Engineering Handbook, 1st ed., Chen, Wai-Fah, Duan Lian Ed., CRC Press, Boca Raton, FL, 1999; Wolchuk, R., 2006. Prefabricating standard orthotropic steel decks. Modern Steel Construction, December. Available from: http://www.modernsteel.com/Uploads/Issues\ December_2006/30760_nsba.pdf [Accessed 7 July 2010], et al., which will be discussed in other similar articles devoted to the development of orthotropic deck bridges.

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UPOREDNA ANALIZA OSETLJIVOSTI KONSTRUKCIJA “JACKET” I MONOPIL NA PARAMETRE OPTEREĆENJA

Rezime:

S-26

U današnje vreme, dosta pažnje se posvećuje razvoju obnovljivih izvora energije. Energija vetra igra značajnu ulogu u toj oblasti. Iz tog razloga, sve je veće interesovanje za poboljšanjima u mnogim aspektima dizajna vetrogeneratora. Ova studija se bavi ofšor vetrogeneratorima sa aspekta pouzdanosti. Koristeći procenu nesigurnosti, stiće se dublje razumevanje ponašanja konstrukcije. Kako su dominantna opterećenja na noseću konstrukciju vetrogeneratora talasi, proučeno je kako određene talasne karakteristike utiču na konstrukciju. Postupak je odrađen za dva tipa konstrukcija, sa ciljem zaključivanja o prednostima i manama oba tipa.

Ključne reči: energija vetra, analiza osetljivosti, noseća konstrukcija, dejstva talasa

COMPARATIVE ANALYSIS OF JACKET AND MONOPILE STRUCTURES IN SENSITIVITY TO LOAD PARAMETERS

Summary:

Nowadays, much attention is paid to the development of renewable energy resources. Wind energy plays a major role in this issue. That is why there is a growing interest for improving the design process of wind turbines at many aspects. This study deals with offshore wind turbines (OWT) from the reliability aspect. Estimating the uncertainties, a deeper understanding of behavior of the structure is obtained. As waves are the dominant load on OWT support structures, this paper addresses how specific wave characteristics affect the structure. Two structural types of support structures are studied, with the aim to evaluate pros and cons of both.

Keywords: wind energy, sensitivity analysis, support structure, wave load

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1 INTRODUCTION

1.1 GENERAL

In structural engineering practice, much effort has been made to balance between the three most important requirements that should be fulfilled: safety and structural reliability; effects of the structure on the environment; and economic efficiency [1, 2]. Renewable resources of energy make a good compromise regarding all three requirements. For that, and for the reason of growing demands for energy nowadays, there is a growing interest in renewable energy [3]. With respect to Kyoto protocol, many producers turn to renewable energy resources, which leads to a fact that more than 75% of new power capacity installations in EU in the year 2015 are renewable resources. The leading among the new renewable energy resources is wind energy [4].

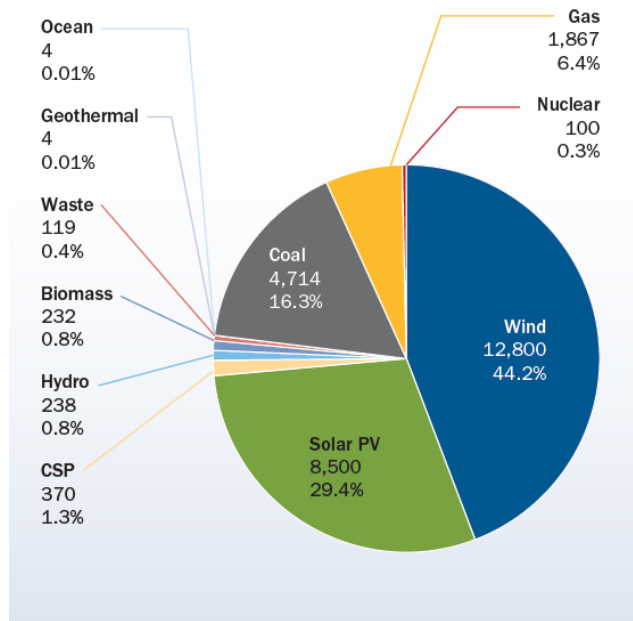


Figure 1 - Share of new energy capacity installations in EU in 2015, source: [4]

In the last decades, even more wind energy is accommodated by moving offshore. That brings up a problem of more complicated and expensive installation, but wide available locations for wind farms and higher electricity output justify the decision to go offshore. Regarding the supporting structures of offshore wind turbines (OWT), monopile is the most commonly used structure in shallow and medium water depths (0-40m), due to relatively easy installation and simple design. For higher water depths (20-50m), jacket support structures are employed due to higher stiffness, as well as smaller surface facing the wave movement compared to monopiles [5].

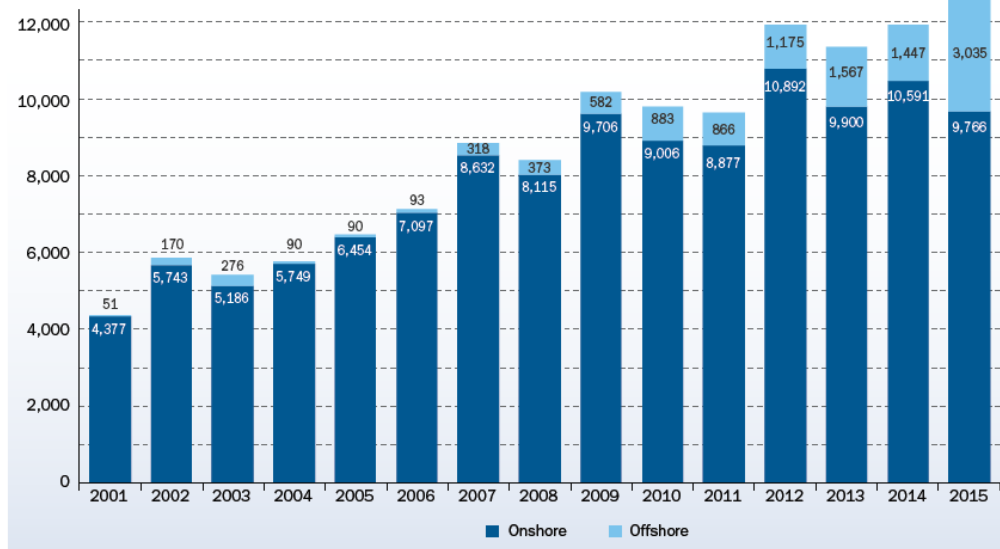


Figure 2 – Growing share of offshore WT installations from year 2001 to 2015, source: [4]

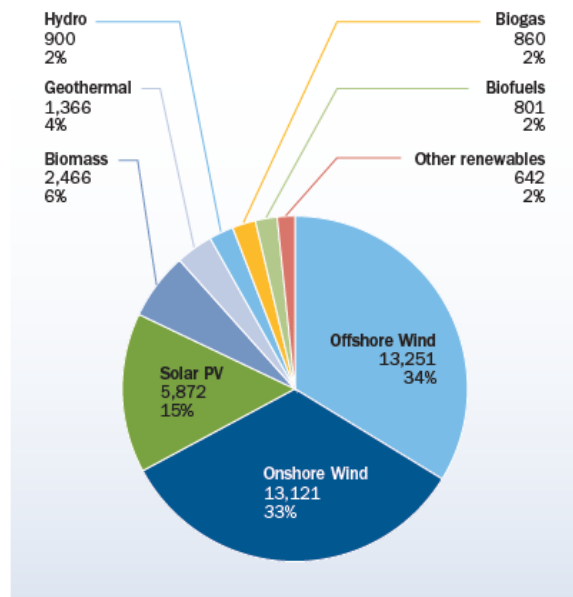


Figure 3 – Renewable energy investments in EU in year 2015 (€ million), source: [4]

1.2 TYPES OF OFFSHORE WINDENERGY SUPPORT STRUCTURES

In the design process of OWT support structures, one of the most important design driving criteria is fatigue. Damage caused by fatigue is accumulated during the structural lifetime through cyclic stresses caused by wind and wave loads [9]. The dominant load on the OWT support structure is the wave load, as it is submerged at most of its height, while the wind turbine tower is mostly affected by the wind. In this paper, the focus is on the wave loads, as only the support substructures are studied.

During the numerical modelling in most of the software packages, sea state is modelled as a superposition of a number of regular waves in order to achieve a very realistic model. This way formed sea state affects the structure and causes stresses that lead to fatigue damage. However, it is not obvious how every single waveform from the sea state contribute to the caused stress. In order to perceive how sensitive are the structures to different wave characteristics (wavelengths, frequencies), sea state is separated into single waves, and each of them is applied to the structure. Stress contribution of every wave is noted and compared with others. Finally, it is stated which of the observed structural types is more sensitive to specific wave characteristics.



Figure 4 – (a) jacket (b) monopile OWT support structure, [source: DNV GL, 2016]

2 BASIS OF INVESTIGATIONS

2.1 LOAD PARAMETERS

With the aim to study the influences of single waves on the structure and at the same time to stay in a domain close to reality, used wave characteristics are taken from “EU UpWind” project (SES6 No 019945 UPWIND) named “UpWind Design Basis” [6]. It contains 3D scatter diagrams of wave parameters for different wind speeds for two offshore sites in the Dutch North Sea. For purposes of this study, wave parameters from scatter diagrams for K13 Deep Water Site and for the most frequent wind speed of 9-11 m/s are used.

Table 1 - Scatter diagram of wave heights and periods for wind speed of 10m/s, source: [6]

Vw = 9-11 m/s		Tp [s]														
		< 0,5	1	2	3	4	5	6	7	8	9	10	11	>11,5		
Hs [m]	9,5														0,00000	
	9														0,00000	
	8,5														0,00000	
	8														0,00000	
	7,5														0,00000	
	7														0,00000	
	6,5														0,00000	
	6														0,00000	
	5,5														0,00000	
	5														0,00000	
	4,5										0,00002				0,00002	
	4									0,00002	0,00002	0,00002	0,00002	0,00002	0,00002	0,00008
	3,5									0,00006	0,00016	0,00005	0,00002			0,00028
	3								0,00006	0,00040	0,00037	0,00009				0,00093
2,5								0,00003	0,00204	0,00191	0,00061	0,00003			0,00462	
2								0,00002	0,00302	0,01006	0,00353	0,00047	0,00002		0,01711	
1,5								0,00089	0,02336	0,01448	0,00114	0,00009	0,00002	0,00002	0,03999	
1								0,00076	0,02344	0,02666	0,00366	0,00033	0,00005	0,00002	0,05492	
0,5								0,00002	0,00866	0,01416	0,00244	0,00039	0,00012	0,00002	0,00003	0,02584
<0,25								0,00026	0,00026	0,00008						0,00061
		0,00000	0,00000	0,00000	0,00002	0,00969	0,03876	0,05559	0,03069	0,00751	0,00177	0,00028	0,00005	0,00003	0,14440	

This 2D scatter is only a part of a complete scatter, which includes wind speed as a third dimension. That is why the summarized share of all possible waves is not 1 but 0.14440. For better understanding of this scatter, it is normalized for the case of the given wind speed and plotted as discrete data sequence in Figure 5. Hence, it is clearly visible which of the wave characteristics are the most frequent ones. Each of those single waves is applied to both monopile and jacket OWT support structures. Structures are numerically modelled in FE software package Poseidon, specialized for wave-induced loads [10]. For both structures, numerical simulations of every single waveform from the Table 1 with a duration of 150s are carried out using the FE Poseidon’s Wave Simulator Tool. The stress results are recorded by the set of sensors positioned on the corresponding spots on the structures with the corresponding angles.

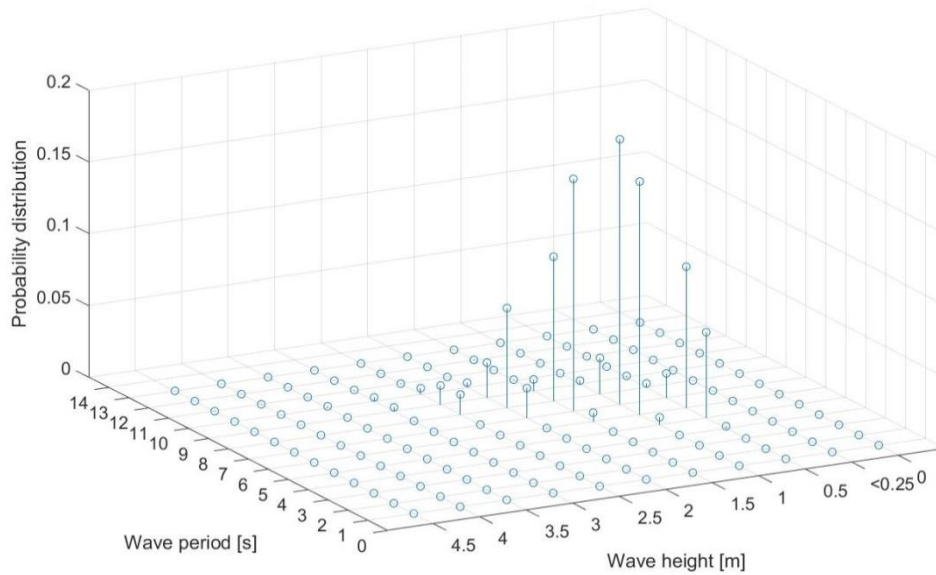


Figure 5 – Probability distribution of wave parameters (discrete data)

2.2 NUMERICAL MODELS

2.2.1 Jacket structure

The jacket structure is designed for a site in the North Sea with water depth of 50m [9]. Chords and braces are steel tubes with diameters of 1.2m and 0.8m respectively. It is supposed to be connected to the soil by four piles, which is modelled as clamped legs at the soil level. Above the soil level, the structure is submerged at a height of 50m. It continues 21.15m above the still water level, where it would be connected with the wind turbine tower by a transition piece [5].

Every wave loading from Table 1 is applied in the direction of 45° with respect to the local coordinate system, which is parallel to the diagonal of the jacket's footprint. The chosen sensor measures the stress in the structure in the same direction at the bottom of the structure (mudline) at the jacket's leg that is directly affected by the wave, and thus the mostly stressed one.

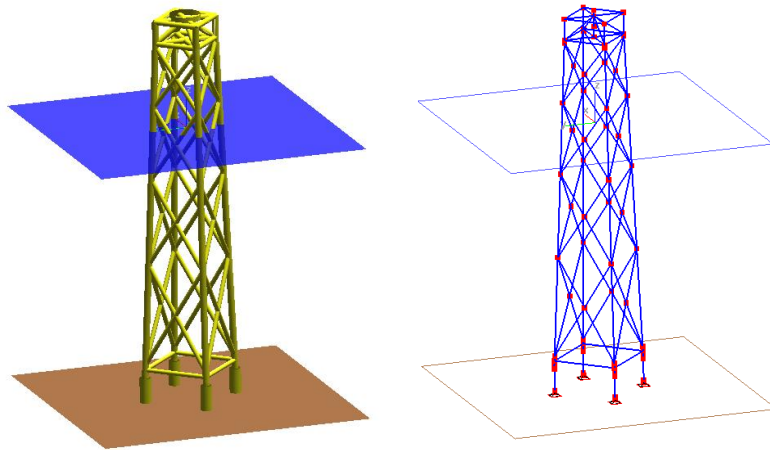


Figure 6 – Jacket (a) 3D numerical model, (b) beam model

2.1.2 Monopile structure

The monopile structure has a relatively simple design. It is made of a cylindrical steel tube with changing cross section diameter and thickness. At the soil level, it continues down into the seabed with length of 35m. The soil is modelled with springs, whose stiffness it obtained using the p-y method [7, 8]. At its bottom, it has an 8m diameter. Above the soil level, it is submerged at height of 40m, where it has a cone part, which reduces the cross section diameter to 6m. It continues 18m above the still water level, where it would be connected with the wind turbine tower that it is supporting, through transition piece [5].

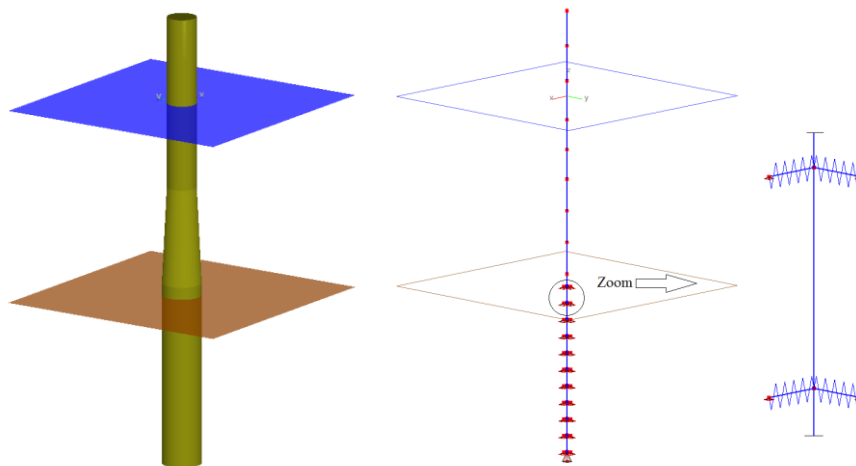


Figure 7 – Monopile (a) 3D numerical model, (b) beam model with spring detail

Every wave loading from Table 1 is applied in the direction of 0° with respect to the local coordinate system. The chosen sensor measures the stress in the structure in the same direction at the bottom of the structure.

After all the simulations have been carried out, the stress results are plotted and compared for different wave parameters for both reference structures. In Figure 8 it is shown how different wave heights influence the stress results, while the wave period is fixed to 10s. Only some of the simulations are shown.

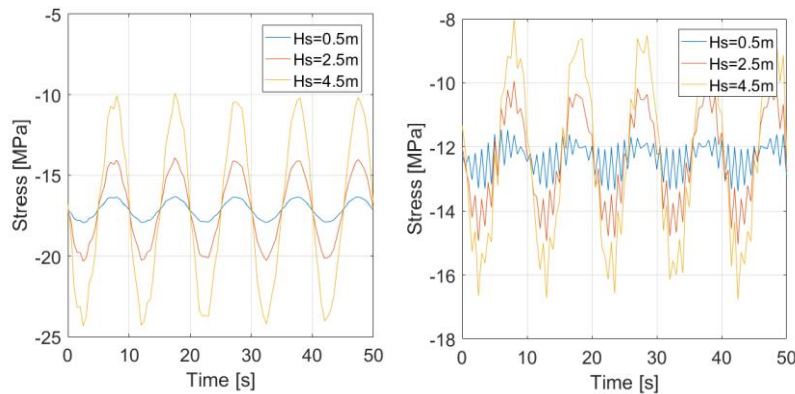


Figure 8 – Stress results for (a) monopile (b) jacket structure for different wave heights

It can be seen that both of the structures stand higher stresses with the increase of wave height, as expected. On the Figure 9, the nominal values of stress amplitudes for different wave heights for both structures are plotted. It is demonstrated that both of dependencies are nearly linear, as the simulations are carried out in the domain of linear deformations without extreme loads. However, the dependency line for monopile is steeper, which shows that monopiles are more sensitive to wave heights compared to jacket structures, due to their geometry.

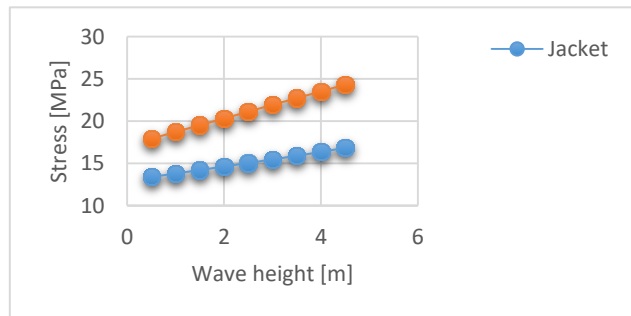


Figure 9 – Stress – wave height dependency for monopile and jacket structure

In Figure 10 it is shown how different wave periods influence the stress results, while the wave height is fixed to 0.5m. Only some of the simulations are shown.

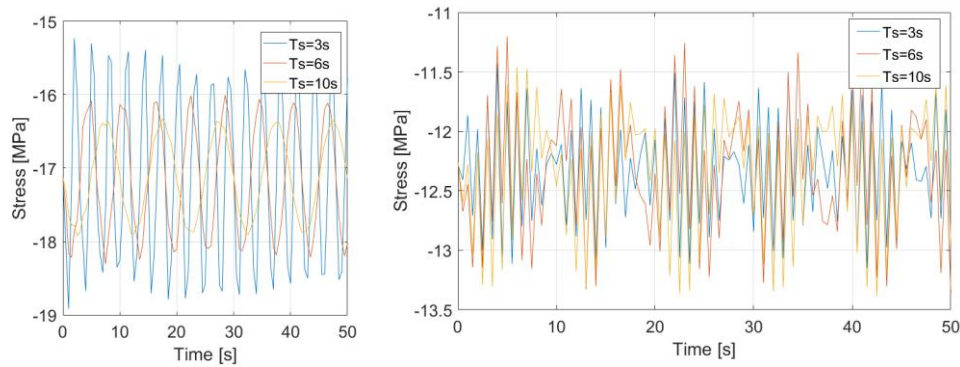


Figure 10 – Stress results for (a) monopile (b) jacket structure for different wave periods

On the Figure 11, the nominal values of stress amplitudes for different wave periods for both structures are plotted. It is demonstrated that the monopile structure has a slight increase of stresses with decrease of wave periods (increase of wave frequency). The gradient of stress increase is higher for lower wave periods (under 4s), as the wave frequencies get closer to the first eigenfrequency of the structure. The jacket structure is nearly insensitive to the examined wave periods. The reason for that is, besides its higher stiffness at footprint, also a higher first eigenfrequency compared to the monopile.

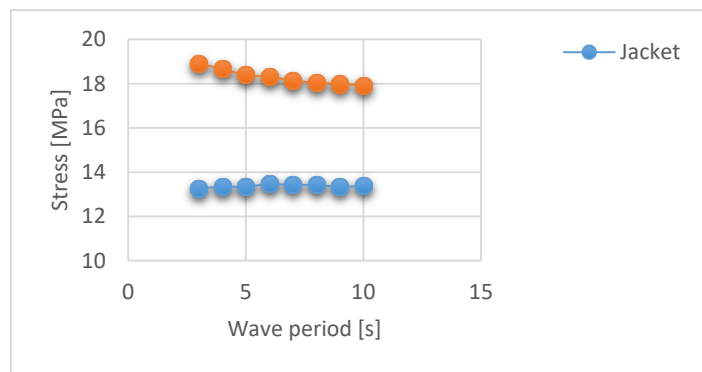


Figure 11 – Stress – wave period dependency for monopile and jacket structure

3 CONCLUSIONS

The obtained results show that the jacket structure is in general less sensitive to wave load parameters compared to the monopile structure. Due to its complex geometry and higher stiffness, the jacket structure shows low linear sensitivity to the increase of wave height and nearly no sensitivity to change of wave period, while monopile shows steeper linear sensitivity to wave heights and sensitivity to decrease of wave periods. This research is focused on one, most frequent wind speed. For a complete overview, other wind speeds as well as the extreme load cases must be taken into consideration. This work is part of a research that deals with all

reference wind speeds for the given offshore site, and take the accumulated fatigue damages from stress cycles into consideration. A reasonable choice between the reference structures has to be based on all design load cases and design driving criteria on one hand as well as overall cost efficiency for fabrication, transportation and installation of the structures on the other hand.

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MODELIRANJE I NOSAČA ZA NUMERIČKU ANALIZU GRANIČNE NOSIVOSTI PRI DELIMIČNOM OPTEREĆENJU

Rezime:

S-27

Prikazano je modeliranje limenih I nosača radi sprovođenja numeričke analize njihove granične nosivosti pod dejstvom opterećenja koje deluje na maloj dužini pojasa, a u ravni rebra. Razmatrani su nosači bez podužnih i poprečnih ukrućenja u zoni unošenja opterećenja. Izvršena je verifikacija numeričkog modela poređenjem rezultata numeričke analize sa rezultatima eksperimentalnih istraživanja. Numeričke simulacije su sprovedene koristeći softverski paket Abaqus. Granična nosivost je određena koristeći nelinearnu analizu uključujući geometrijsku i materijalnu nelinearnost

Кljučне речи: локално оптерећење, нумеричка анализа, гранична носивост

MODELING OF PLATE GIRDERS FOR NUMERICAL STUDY OF THE ULTIMATE STRENGTH UNDER PATCH LOAD

Summary:

In this paper modeling of the plate girders for the numerical research of their ultimate strength due to patch loading are presented. Girders without longitudinal and transversal stiffeners in the zone of the load introduction are considered. Verification of numerical model through comparison of numerical and experimental results is performed. The numerical simulations were carried out using software package Abaqus. Ultimate capacity was performed by nonlinear analysis including geometrical and material nonlinearities.

Keywords: patch loading, numerical study, ultimate capacity

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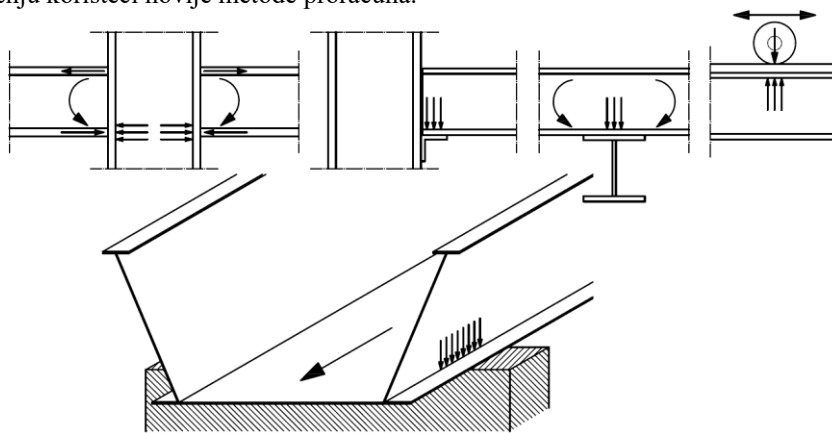
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1 UVOD

Granična nosivost limenih nosača i njihovo ponašanje u postkritičnoj fazi se intenzivno istražuju u poslednjih nekoliko decenija i predstavljaju veoma značajne oblasti istraživanja u građevinarstvu. Limeni nosači, sa ili bez vertikalnih i horizontalnih ukrućenja, predstavljaju konstruktivne elemente koji mogu biti opterećeni na različite načine. Ponašanje ovakvih elemenata pod dejstvom lokalizovanog odnosno, delimično podeljenog opterećenja (patch load) predstavlja problem koji se često javlja u inženjerskoj praksi. Delimično opterećenje se može javiti u vidu spoljašnjih sila (reakcije mostovskih stubova pri montaži glavnih mostovskih nosača) ili pri interakciji drugih strukturalnih elemenata (opterećenje od rožnjače ili kranskog točka na ramovske elemente, sile u vezama greda-stub opterećene na savijanje), Slika 1. Pored toga, mnoga druga opterećenja pri transportu ili montaži konstrukcija mogu da se klasifikuju kao delimična opterećenja.

Pri projektovanju konstrukcija ovi problemi se prevazilaze postavljanjem vertikalnih ukrućenja na mestu dejstva sile. Međutim, često ili zbog konstruktivnih (položaj sile je promenljiv), ekonomskih ili tehnoloških razloga (potreba za automatskom proizvodnjom, troškovi dodatnog zavarivanja, troškovi zaštite i budućeg održavanja) ovo nije moguće. Zbog toga, rešenja se traže u tačnijem određivanju graničnog opterećenja ili postavljanjem horizontalnih ukrućenja u blizini opterećene nožice. Navedene činjenice ukazuju na značajnost ovog problema i da bi se izbegle havarije neophodno ga je adekvatno uzeti u obzir. Odnosno, potrebno je detaljnije razumeti ponašanje i tačnije odrediti graničnu nosivost pri delimičnom opterećenju koristeći novije metode proračuna.

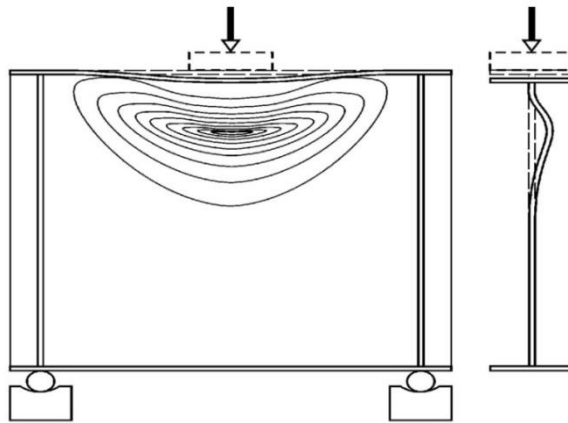


Slika 1 – Primeri delimičnog opterećenja u inženjerskoj praksi [1]

Dostizanje granične nosivosti se manifestuje pojavom lokalnog izbočavanja rebra u zoni unošenja opterećenja i deformacijom pojasa, Slika 2. Celokupno ponašanje ovih nosača u elastičnoj i plastičnoj oblasti, kao i vrednost graničnog opterećenja, zavisi od niza geometrijskih i fizičkih karakteristika koje obuhvataju: osnovne dimenzije nosača (debljina i visina rebra, dimenzija nožice, dužina nosača), rastojanje vertikalnih ukrućenja, položaj i broj horizontalnih ukrućenja, vrstu i karakteristike horizontalnih ukrućenja, početne geometrijske i strukturalne imperfekcije, napona tečenja rebra i pojasa, način nanošenja opterećenja i dužina opterećenja, ekscentričnost opterećenja u odnosu na ravan rebra, karakter opterećenja (statičko

ili dinamičko), kao i odnos pojedinih od ovih veličina (vitkost rebra i odnos debljine nožice i rebra) [2], [3], [4]. U zavisnosti od oblika loma, u literature se najčešće sreću termini tečenje, izvijanje i izbočavanje. Granična nosivost je određena ili tečenjem (gnječenjem-local yielding) materijala ispod opterećenja ili pojavom izbočavanja rebra. Pri tome može doći ili do izbočavanja po celoj visini rebra (buckling) ili do izbočavanja samo u blizini opterećenog pojasa u užoj oblasti unošenja opterećenja u nosač (web crippling) [3], [5].

Problem određivanja granične nosivosti pojavio se od uočavanja pojave lokalnog izbočavanja, ali poseban značaj dobija sa savremenim pristupima projektovanja i principima na kojima se zasnivaju propisi gde granična nosivost predstavlja važan element. Pokušaji da se dođe do jednostavnog rešenja koje će važiti za širok raspon parametara nisu u potpunosti uspešni, tako da su u svetu i dalje u toku intenzivna istraživanja različitih segmenata ovog problema.



Slika 2 - I-nosač izložen dejstvu koncentrisanog opterećenja po pojasu

Proces određivanja granične nosivosti pod dejstvom delimičnog opterećenja se uglavnom sastojao od eksperimentalnog ispitivanja (definisanog na određenom broju uzoraka), koje je kasnije služilo kao temelj daljim teorijsko-analičkim i teorijsko-empirijskim metoda. Koristeći te metode brojni istraživači su dali predlog za određivanje graničnog opterećenja, koji se uglavnom bazira na mehanizmu loma, kod ovih i sličnih problema [6], [7], [8], [9]. Međutim, kako se izrazi za graničnu nosivost razlikuju, daju različite vrednosti i odstupaju međusobno potrebno je izvršiti dodatna ispitivanja razmatrajući dodatne parametre koji utiču na nosivost nosača.

Trenutno najpopularniji pristupi u određivanju graničnog opterećenja predstavljaju numeričke metode, prvenstveno metod konačnih elemenata, koje su ugrađene u gotove softverske pakete. Razvoj novih tehnologija i teorijskih postavki metode konačnih elemenata doprineli su široj upotrebi numeričke analize u naučnim istraživanjima mnogih inženjerskih problema, pa samim tim i u oblasti stabilnosti konstrukcija. Uopšten pristup sastoji se od verifikacije numeričkog modela i određenog broja eksperimentalnih ispitivanja. Ovakav pristup je postao veoma bitan deo svakog relevantnog istraživanja. Na taj način se proširuje oblast analiziranih parametara i omogućuje se sprovođenje velikoj broja testiranja. Glavni kriterijum za pouzdanu i tačnu numeričku analizu je slaganje rezultata iz numeričkih i eksperimentalnih testova. Na osnovu toga, može se zaključiti da adekvatno i sigurno modeliranje inženjerskih

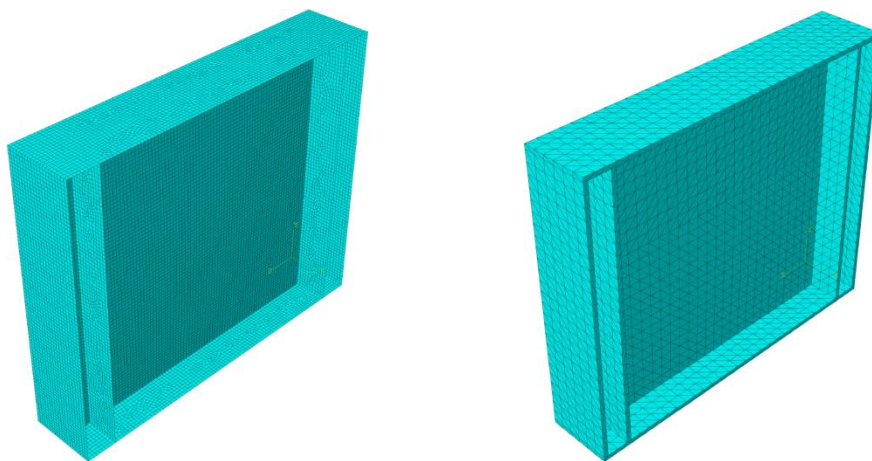
problema predstavlja važan korak u procesu istraživanja i projektovanja. Takođe, trenutni evropski standard za projektovanje čeličnih konstrukcija EN1993-1-5 [10] dopušta projektantima upotrebu numeričke analize za pločaste elemente. Na osnovu svega navedenog, jasno je da numerički pristup i modeliranje predstavljaju osnovni alat u procesu istraživanja i projektovanja.

Predmet rada je lokalno izbočavanje, postkritično ponašanje i granično opterećenje limenih nosača (zavarenih čeličnih I-nosača) bez podužnih i poprečnih ukrućenja pod dejstvom lokalizovanog opterećenja (koncentrisanog odnosno, raspodeljenog opterećenja na određenoj dužini) na jednom pojasu u ravni rebra. Pri tome, posebno se razmatra uticaj pojedinih do sada nedovoljno istraživanih parametara, prvenstveno uticaj dužine rasprostiranja opterećenja. Navedena studija se bazira na numeričkom pristupu, u kojem je korištena metoda konačnih elemenata implementirana u višenamenski softverski paket Abaqus [11]. Rad predstavlja pripremu za parametarsku analizu u kojoj je varirana dužina nanošenja opterećenja s_s , debljina rebra t_w i nožice t_f .

Cilj istraživanja je ispitivanje mogućnosti i optimalnih načina modeliranja nosača pod dejstvom delimičnog opterećenja, za određivanje njihove granične nosivosti i postkritičnog ponašanja, radi daljih numeričkih istraživanja nedovoljno proučenih parametara. Pri tome, analizira se uticaj niza različitih parametara koji utiču na rešenja, mogućnost njihovog što adekvatnijeg modeliranja, kao i njihov značaj. Da bi se uopšte analizirali pojedini parametri neophodno je prvo izvršiti verifikaciju numeričkog modela i eksperimentalnih ispitivanja. Predstavljeni numerički model je verifikovan (kalibrisan) koristeći rezultate iz eksperimentalnog istraživanja [4].

2 NUMERIČKO ISTRAŽIVANJE

Za simulacije numeričkih modela korišćen je višenamenski softverski paket Abaqus. Numerička analiza čeličnog nosača izvršena je inkrementalnom nelinearnom analizom korišćenjem geometrijske i materijalne nelinearnosti. Stanje nelinearne statičke ravnoteže tokom nestabilne faze odgovora se može odrediti na osnovu Riks-ove modifikovane metode [12], koja je implementirana u softverskom paketu Abaqus. Ova metoda je inkrementalno-iterativni postupak i pogodna je za predviđanje nestabilnog, geometrijski nelinearnog kolapsa konstrukcije, uključujući nelinearnost materijala i granične uslove. Nelinearna analiza stabilnosti se zasniva na rešavanju nelinearne jednačine ravnoteže prema kojoj se inkrementalna promena sile koja deluje na element može predstaviti inkrementom pomeranja u funkciji tangentne matrice krutosti. Ravnoteža elementa može da bude stabilna ili nestabilna. Tačka u kojoj je tangenta na krivu horizontalna je granična tačka (limit point) koja predstavlja prelazak iz stabilnog u nestabilno stanje. U okolini ovih tačaka numerički algoritam može da bude nestabilan, posebno u slučajevima kada je početni deo krive idealno linearan sa oštrim prelomom u graničnoj tački (problem bifurkacione stabilnosti). Statička nelinearna analiza je zasnovana na Newton-Raphson-ovom iterativnom konceptu primene kružnog luka (arc length) kao dodatnog parametra za kontrolu proračuna duž krive sila-pomeranje, a sve sa ciljem dobijanja intenziteta graničnog opterećenja koji predstavlja dodatnu nepoznatu veličinu u proračunu. Ovakav koncept obezbeđuje rešenje bez obzira da li je odgovor sistema u domenu stabilne ili nestabilne ravnoteže uz određena ograničenja.



Slika 3 – Numerički modeli: pločasti konačni elementi (levo), 3D konačni elementi (desno)

Nosači su modelirani u realnoj veličini, pomoću četvorougaoih pločastih (*shell*) elemenata sa redukovanom integracijom S4R. Ovaj tip konačnog elementa je usvojen na osnovu analize [13], [14], koja obuhvata različite grupe i tipove konačnih elemenata (pločasti STRI65 i 3D elementi C3D10 i C3D4, Slika 3). Osim toga, pouzdanost i tačnost modela sa usvojenim konačnim elementom su dokazani i u mnogim drugim prethodnim istraživanjima [15], [16], [17], [18], [19], [20]. Mreža konačnih elemenata je izrađena korišćenjem veličine elemenata od 10 mm na osnovu analize konvergencija prikazane u [13], [14]. U cilju sprovođenja realnih uslova opterećivanja nosača, blok opterećenja je modeliran kao poseban element, preko kojeg se opterećenje prenosi na gornju nožicu. Usvojen je četvorougli kruti konačni element R3D4. Širina ovog bloka je ista kao i širina nožice i iznosi 120 mm, dok je dužina bloka različita i iznosi $s_s = 0$ (2mm), 50 mm, 100 mm, 150 mm, 200 mm i 250 mm. Modeliranje kontakta između gornje nožice i bloka za opterećenje je sprovedeno pomoću *surface-to-surface* diskretizacione metode i *finite-sliding* formulacije.

Nosači su modelirani kao slobodno oslonjeni (sprečeno vertikalno pomeranje i pomeranje upravno na ravan rebra) na mestima vertikalnih ukrućenja što odgovara uslovima iz eksperimenta [4]. Za sprečavanje pomeranja u podužnom pravcu izabrani su čvorovi na sredini donje nožice. Bloku za nanošenje opterećenja su sprečena sva pomeranja osim u vertikalnom pravcu. Materijal je modeliran kao izotropan materijal sa von Mises-ovim kriterijumom tečenja. Young-ov modul elastičnosti i Poasonov koeficijent su 205 GPa i 0.30, respektivno. Naponi tečenja ($f_{yw} = 321$ MPa, $f_{yf} = 316$ MPa) i karakteristike ponašanja materijala (ojačanje i omekšanje) su preuzeti na osnovu ispitivanja epruveta za materijal rebra i nožice za nosač A12.

3 VERIFIKACIJA NUMERIČKOG MODELA

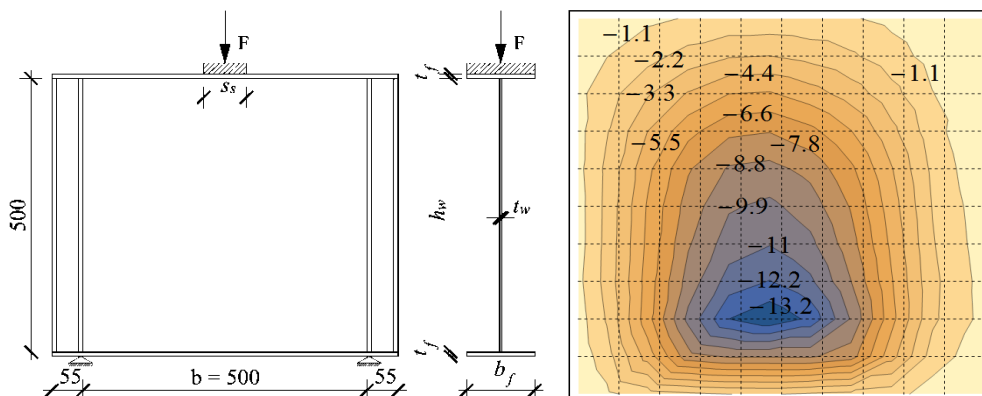
Da bi se izvršila parametarska analiza i dodatna numerička ispitivanja, neophodno je izvršiti kalibraciju (verifikaciju) numeričkog modela. Model je testiran i poreden sa eksperimentalnim ispitivanjem nosača označenog kao A12 u [4]. Eksperimentalna granična nosivost $P_{gr,eks}$, numerička granična nosivost $P_{gr,Abq}$ i geometrijske karakteristike nosača A12 su prikazani u Tabeli 1, dok je dispozicija nosača sa potrebnim oznakama prikazana na Slici 4.

Pored toga, radi što boljeg slaganja sa eksperimentalnim rezultatom korišćene su stvarne (izmerene) geometrijske imperfekcije rebra, Slika 4.

Tabela 1 – Geometrijske karakteristike nosača i vrednosti graničnih sila

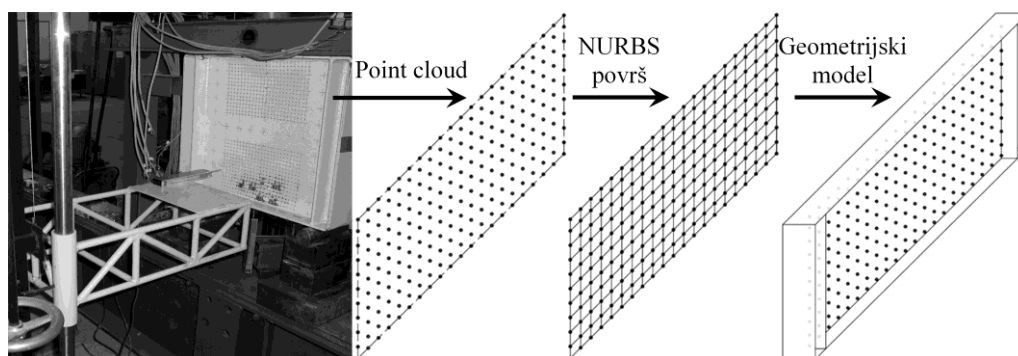
Nosač	b_f [mm]	t_f [mm]	t_w [mm]	h_w [mm]	b [mm]	s_s [mm]	$P_{gr,eks}$ [kN]	$P_{gr,Abq}$ [kN]
A12 [4]	120	8	4	500	500	25	154.60	153.86

Proces formiranja geometrije nosača sa stvarnim imperfekcijama rebra je prikazan na Slici 5. Međutim, uobičajena praksa u modeliranju geometrijskih imperfekcija predstavlja korišćenje modova oscilovanja iz linearne analize. Ovaj trend su pratili mnogi istraživači iz navedene oblasti, s obzirom da nisu imali eksperimentalnu bazu početnih imperfekcija ili nisu bili u mogućnosti da iskoriste eksperimentalne vrednosti [18], [19], [20]. Stoga usvojeni numerički model tačnije opisuje početnu geometriju nosača. Modeliranje ovakvih imperfekcija predstavlja izazov i sastoji se od sledećih koraka. Imperfekcije rebra su merene pre eksperimentalnog testiranja koristeći mrežu tačaka 50x50 mm pri čemu se dobija po 11 tačaka u podužnom i vertikalnom pravcu. Na osnovu toga, dobijaju se vrednosti imperfekcija rebra u tačkama (point cloud) koje se pretvaraju u NURBS površ koristeći CAD softvere. Nakon toga, dobijena površ rebra se importuje u softverski paket Abaqus, vrši sklapanje modela nakon čega se dobija geometrijski mode, Slika 5.



Slika 4 – Dispozicija nosača (levo) i vrednosti početnih geometrijskih imperfekcija (desno)

Modeliranje i verifikacije numeričkog modela sa eksperimentalnim testovima omogućili su parametarsku analizu koja podrazumeva izradu baze numeričkih testova. U priloženom radu [21] razmatran je uticaj dužina nanošenja opterećenja s_s , za različite debljine rebra t_w i nožice t_f .



Slika 5 – Proces pravljenja geometrijskog modela

4 ZAKLJUČAK

Rad je prikazao mogućnosti numeričkog modeliranja limenih I nosača radi sprovođenja numeričke analize njihove granične nosivosti pod dejstvom opterećenja koje deluje na maloj dužini pojasa, a u ravni rebra. Razmatrani su nosači bez podužnih i poprečnih ukrčenja u zoni unošenja opterećenja. iz navedene oblasti. Predstavljene su osnovne karakteristike numeričkog modela, njegova opravdanost u pogledu eksperimentalnog ispitivanja i način kojim se početne geometrijske imperfekcije uzimaju u obzir. Na osnovu toga, stvoreni su uslovi za dalja numerička istraživanja i simulacije. Istraživanje iz predmetne oblasti će se nastaviti razmatrajući ostale parametre (osnovne dimenzije nosača, različite odnose debljine nožice i rebra, vitkost rebra, različite karakteristike materijala, dužinu nanošenja opterećenja i dr.) u cilju definisanja predloga i izraza za graničnu nosivost nosača za praktičnu primenu.

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NUMERIČKA ANALIZA UTICAJA DUŽINE DELIMIČNOG OPTEREĆENJA NA GRANIČNU NOSIVOST I NOSAČA

Rezime:

S-28

U radu se razmatra granična nosivost čeličnih I nosača pod dejstvom opterećenja koje deluje na maloj dužini pojasa, a u ravni rebra. Izvršena je numerička analiza uticaja dužine opterećenja kod nosača bez podužnih i poprečnih ukrućenja u zoni unošenja opterećenja. U sklopu parametarske analize pored dužina nanošenja opterećenja varirana su još dva parametra: debljina rebra i debljina nožice. Granična nosivost je određena koristeći nelinearnu analizu uključujući geometrijsku i materijalnu nelinearnost. Numeričke simulacije su sprovedene koristeći softverski paket Abaqus. Uticaji razmatranih parametara na graničnu nosivost su navedeni.

Кljučне речи: lokalno opterećenje, numerička analiza, granična nosivost

NUMERICAL ANALYSIS OF INFLUENCE OF PATCH LOAD LENGTH ON ULTIMATE STRENGTH OF I GIRDERS

Summary:

In this paper results from numerical research of the ultimate strength of steel I girders due to patch loading are presented. Numerical analysis of the influence of the length of the loading for girders without longitudinal and transversal stiffeners is performed. Within the parametric analysis besides the length of applied load, thickness of web and flange are varied. Ultimate capacity was performed by nonlinear analysis including geometrical and material nonlinearities. The numerical simulations were carried out using software package Abaqus. The influences of the considered parameters on ultimate capacity are listed.

Keywords: patch loading, numerical study, ultimate capacity

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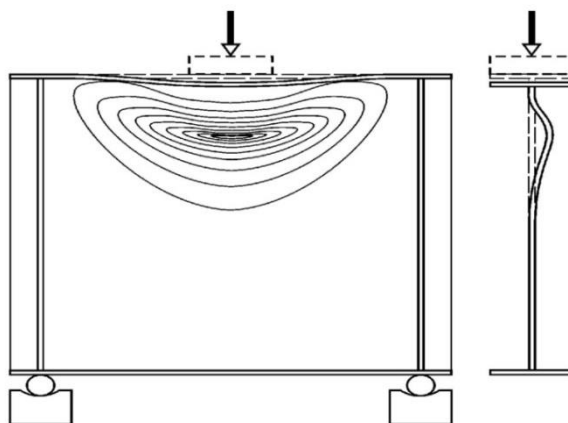
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1 UVOD

Limeni nosači, sa ili bez vertikalnih i horizontalnih ukrućenja, predstavljaju konstruktivne elemente koji mogu biti opterećeni na različite načine. Ponašanje ovakvih elemenata pod dejstvom lokalizovanog odnosno, delimično podeljenog opterećenja (patch load) predstavlja problem koji se često javlja u inženjerskoj praksi. Delimično opterećenje se može pojaviti pri montaži glavnih mostovskih nosača prevlačenjem preko privremenih ili stalnih oslonaca ili pri interakciji drugih strukturalnih elemenata. Pored toga, mnoga druga opterećenja pri transportu ili montaži konstrukcija mogu da se klasifikuju kao delimična opterećenja.

Dostizanje granične nosivosti se manifestuje pojavom lokalnog izbočavanja rebra u zoni unošenja opterećenja i deformacijom pojasa, Slika 1. Celokupno ponašanje ovih nosača u elastičnoj i plastičnoj oblasti, kao i vrednost graničnog opterećenja, zavisi od niza geometrijskih i fizičkih karakteristika koje obuhvataju: osnovne dimenzije nosača (debljina i visina rebra, dimenzije nožice, dužina nosača), rastojanje vertikalnih ukrućenja, položaj i broj horizontalnih ukrućenja, vrstu i karakteristike horizontalnih ukrućenja, početne geometrijske i strukturalne imperfekcije, napon tečenja rebra i pojasa, način nanošenja opterećenja i dužina opterećenja, ekscentričnost opterećenja u odnosu na ravan rebra, karakter opterećenja (statičko ili dinamičko), kao i odnos pojedinih od ovih veličina (vitkost rebra i odnos debljine nožice i rebra) [1], [2], [3].

Problem određivanja granične nosivosti pojavio se od uočavanja pojave lokalnog izbočavanja, ali poseban značaj dobija sa savremenim pristupima projektovanja i principima na kojima se zasnivaju propisi gde granična nosivost predstavlja važan element. Pokušaji da se dođe do jednostavnog rešenja koje će važiti za širok raspon parametara nisu u potpunosti uspeli, tako da su u svetu i dalje u toku intenzivna istraživanja različitih segmenata ovog problema.



Slika 1 - I-nosač izložen dejstvu koncentrisanog opterećenja po pojasu

Proces određivanja granične nosivosti pod dejstvom delimičnog opterećenja se uglavnom sastojao od eksperimentalnog ispitivanja (definisano na određenom broju uzoraka), koje je kasnije služilo kao temelj daljim teorijsko-analitičkim i teorijsko-empirijskim metoda. Koristeći te metode brojni istraživači su dali predlog za određivanje graničnog opterećenja, koji se uglavnom bazira na mehanizmu loma, kod ovih i sličnih problema [4], [5], [6], [7], [8],

[9], [10]. Međutim, kako se izrazi za graničnu nosivost razlikuju, daju različite vrednosti i odstupaju međusobno potrebno je izvršiti dodatna ispitivanja razmatrajući dodatne parametre koji utiču na nosivost nosača.

U Srpskoj akademiji nauka i umetnosti u okviru projekta "Teorijska i eksperimentalna istraživanja metalnih konstrukcija i njihov uticaj na savremeno projektovanje i izvođenje" pod rukovodstvom akademika Nikole Hajdine kao i na Građevinskom fakultetu u Beogradu se navedeni problem izučava već duže vremena i urađeno je više doktorskih disertacija i magistarskih radova. Neki od poslednjih rezultata ovih istraživanja su prikazani u ovom radu.

Trenutno najpopularniji pristupi u određivanju graničnog opterećenja predstavljaju numeričke metode, prvenstveno metod konačnih elemenata, koje su ugrađene u gotove softverske pakete. Razvoj novih tehnologija i teorijskih postavki metode konačnih elemenata doprineli su široj upotrebi numeričke analize u naučnim istraživanjima mnogih inženjerskih problema, pa samim tim i u oblasti stabilnosti konstrukcija. Takođe, trenutni evropski standard za projektovanje čeličnih konstrukcija EN1993-1-5 [11] dopušta projektantima upotrebu numeričke analize za pločaste elemente.

Predmet rada je lokalno izbočavanje, postkritično ponašanje i granično opterećenje limenih nosača (zavarenih čeličnih I-nosača) bez podužnih i poprečnih ukrućenja pod dejstvom lokalizovanog opterećenja (koncentrisanog odnosno, raspodeljenog opterećenja na određenoj dužini) na jednom pojasu u ravni rebra. Pri tome, posebno se razmatra uticaj do sada nedovoljno istraživanog parametra, dužine rasprostiranja opterećenja. U sklopu parametarske analize pored dužina nanošenja opterećenja varirana su još dva parametra: debljina rebra i nožice.

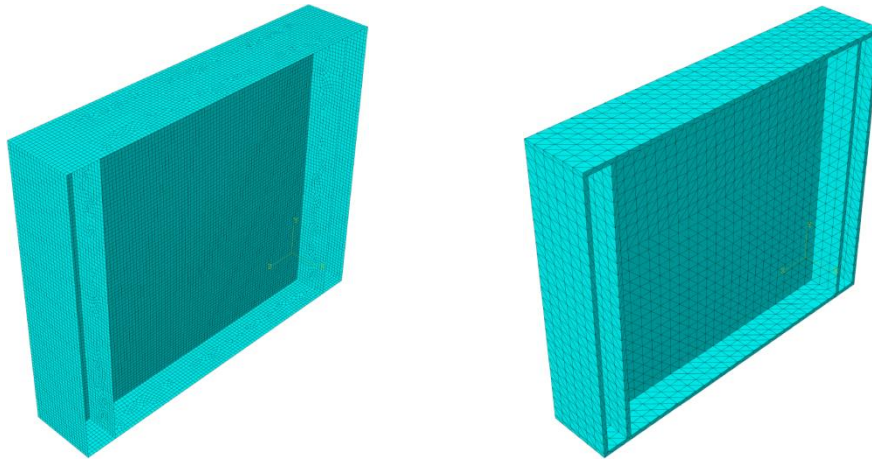
Istraživanje se bazira na numeričkom pristupu, u kojem je korištena metoda konačnih elemenata implementirana u višenamenski softverski paket Abaqus [12]. U pridruženom radu [13] je prikazano pripremno istraživanje numeričkog modeliranja. Predstavljeni i korišćeni numerički model je verifikovan (kalibrisan) koristeći rezultate iz eksperimentalnog istraživanja [3].

Rad sadrži parametarsku analizu u kojoj je varirana dužina nanošenja opterećenja s_s (0, 50, 100, 150, 200 i 250 mm), debljina rebra t_w (2, 4, 6 i 8 mm) i nožice t_f (4, 8, 12, 16 i 20 mm).

2 NUMERIČKI MODEL

Za simulacije numeričkih modela korišten je višenamenski softverski paket Abaqus. Numerička analiza čeličnog nosača izvršena je inkrementalnom nelinearnom analizom korišćenjem geometrijske i materijalne nelinearnosti. Stanje nelinearne statičke ravnoteže tokom nestabilne faze odgovora se može odrediti na osnovu Riks-ove modifikovane metode [14], koja je implementirana u softverskom paketu Abaqus. Ova metoda je inkrementalno-iterativni postupak i pogodna je za predviđanje nestabilnog, geometrijski nelinearnog kolapsa konstrukcije, uključujući nelinearnost materijala i granične uslove. Nelinearna analiza stabilnosti se zasniva na rešavanju nelinearne jednačine ravnoteže prema kojoj se inkrementalna promena sile koja deluje na element može predstaviti inkrementom pomeranja u funkciji tangentne matrice krutosti.

Nosači su modelirani u realnoj veličini, pomoću četvorougao nih pločastih (*shell*) elemenata sa redukovanom integracijom S4R. Ovaj tip konačnog elementa je usvojen na osnovu analize [15], [16], koja obuhvata različite grupe i tipove konačnih elemenata (pločasti STRI65 i 3D elementi C3D10 i C3D4, Slika 2).



Slika 2 – Numerički modeli: pločasti konačni elementi (levo), 3D konačni elementi (desno)

Osim toga, pouzdanost i tačnost modela sa usvojenim konačnim elementom su dokazani i u mnogim drugim prethodnim istraživanjima [9], [10], [17], [18], [19]. Mreža konačnih elemenata je izrađena korišćenjem veličine elemenata od 10 mm na osnovu analize konvergencija prikazane u [15], [16]. U cilju sprovođenja realnih uslova opterećivanja nosača, blok opterećenja je modeliran kao poseban element, preko kojeg se opterećenje prenosi na gornju nožicu. Usvojen je četvorougli kruti konačni element R3D4. Širina ovog bloka je ista kao i širina nožice i iznosi 120 mm, dok je dužina bloka različita i iznosi $s_3 = 0$ (2mm), 50 mm, 100 mm, 150 mm, 200 mm i 250 mm. Modeliranje kontakta između gornje nožice i bloka za opterećenje je sprovedeno pomoću *surface-to-surface* diskretizacione metode i *finite-sliding* formulacije.

Nosači su modelirani kao slobodno oslonjeni (sprečeno vertikalno pomeranje i pomeranje upravno na ravan rebra) na mestima vertikalnih ukrućenja što odgovara uslovima iz eksperimenta [3]. Za sprečavanje pomeranja u podužnom pravcu izabrani su čvorovi na sredini donje nožice. Bloku za nanošenje opterećenja su sprečena sva pomeranja osim u vertikalnom pravcu. Materijal je modeliran kao izotropan materijal sa von Mises-ovim kriterijumom tečenja. Young-ov modul elastičnosti i Poasonov koeficijent su 205 GPa i 0.30, respektivno. Naponi tečenja ($f_{yw} = 321$ MPa, $f_{yf} = 316$ MPa) i karakteristike ponašanja materijala (ojačanje i omekšanje) su preuzeti na osnovu ispitivanja epruveta za materijal rebra i nožice za nosač A12.

3 PARAMETARSKA ANALIZA

Nakon verifikacije numeričkog modela prema eksperimentalnim testovima omogućena je parametarska analiza koja podrazumeva izradu baze numeričkih testova.

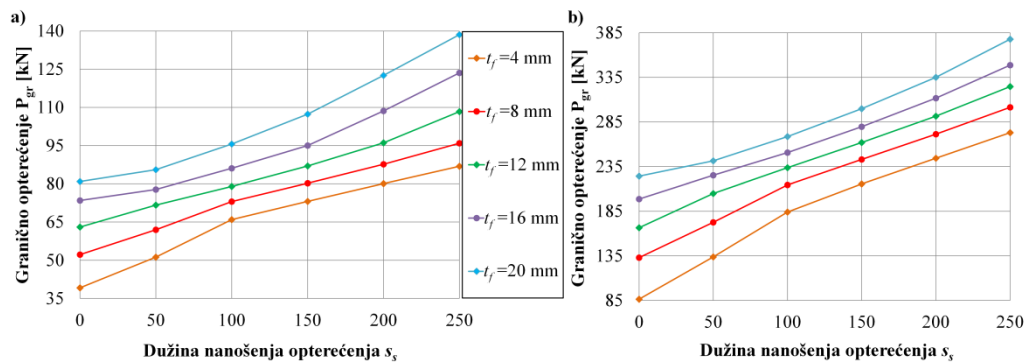
U okviru ovog rada je izvršena parametarska numerička analiza uticaja tri parametra na vrednost granične sile kod tankozidnih I-nosača bez podužnih ukrućenja opterećenih po gornjoj nožici delimičnim opterećenjem. Parametarska analiza je podrazumevala izradu kompleksne

baze numeričkih testova, koja je koncipirana sa ciljem potpunog izolovanja uticajnih parametra na graničnu silu, što nije bilo moguće sprovesti kroz eksperimentalnu analizu.

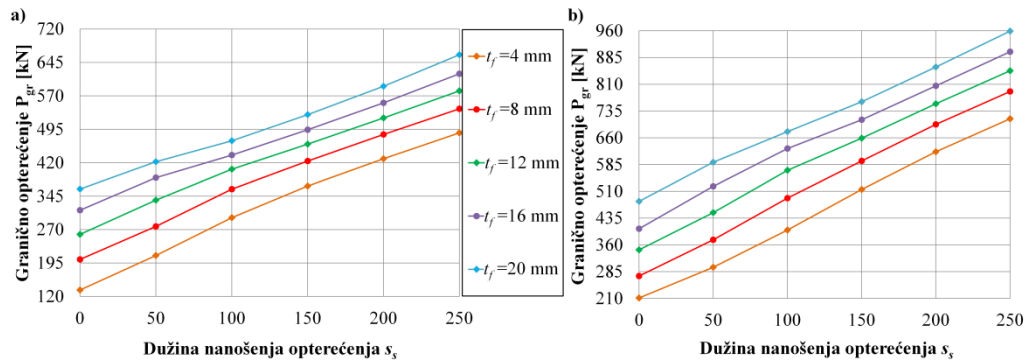
U predstavljenom radu posmatrana su tri parametra, dužina nanošenja opterećenja s_s , debljina rebra t_w i nožice t_f . Ostale dimenzije su konstantne u svim modelima:

raspon $b = 500$ mm, visina $h_w = 500$ mm, širina pojasa $b_f = 120$ mm

U cilju dobijanja što jasnijeg i izdvojenog uticaja dužine nanošenja opterećenja s_s na graničnu silu varirane su širine nanošenja opterećenja ($s_s = 0, 50, 100, 150, 200$ i 250 mm) na nosače različitih debljina rebara ($t_w = 2$ mm, 4 mm, 6 mm i 8 mm) i nožica ($t_f = 4$ mm, 8 mm, 12 mm, 16 mm i 20 mm). Na osnovu toga stvorena je numerička baza koja ima 120 različitih modela. Rezultati parametarske analize su prikazani na Slikama 3 i 4.



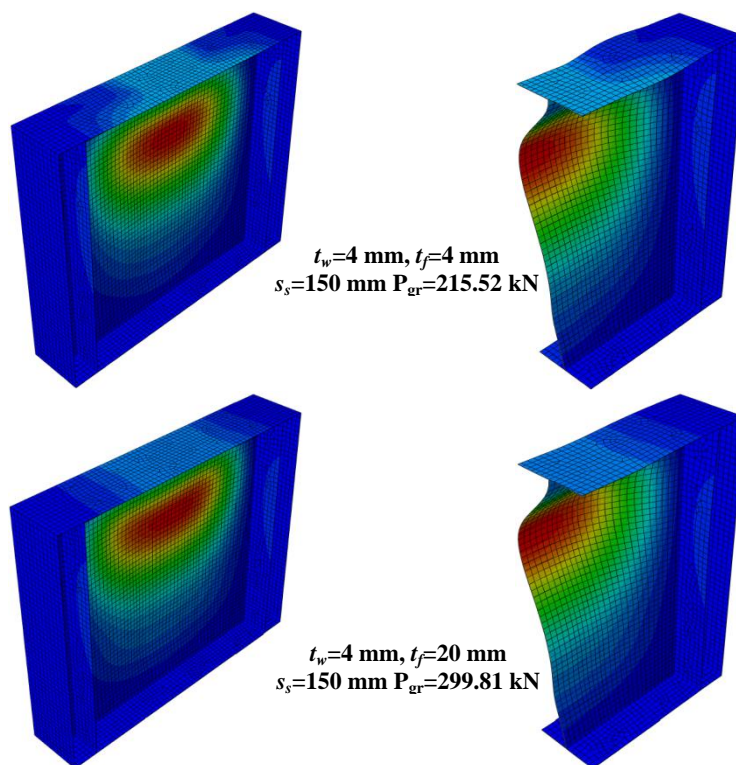
Slika 3 – Vrednosti granične nosivosti za: a) $t_w = 2$ mm. b) $t_w = 4$ mm



Slika 4 – Vrednosti granične nosivosti za: a) $t_w = 6$ mm. b) $t_w = 8$ mm

4 ZAKLJUČCI

Na osnovu priloženih dijagrama (Slika 3 i Slika 4) i numeričkih simulacija uočava se da najveći uticaj na vrednost graničnog opterećenja ima debljina rebra, dok debljina nožice utiče na dužinu oblasti rebra koja se izbočava, Slika 5. Za male odnose debljina nožice i rebra, lokalno izbočavanje obuhvata manju okolinu mesta dejstva sile nego za veće. Pažljivom obradom podataka uočava se da povećanje debljine rebra za 2 mm (pri konstantnim debljinama nožica) daje veću graničnu nosivost od 25% do 40%. Takođe, povećanje debljine nožice (pri konstantnim debljinama rebra) sa 4 mm na 20 mm doprinosi povećavanju granične nosivosti od 25% do 60%, pri čemu je veći doprinos u slučaju manjih dužina opterećenja. Odnosno, povećanje debljine nožice sa 4 mm na 8 mm daje doprinos u graničnoj nosivosti od 8% do 35%, pri čemu je takođe veći doprinos za manje dužine opterećenja.

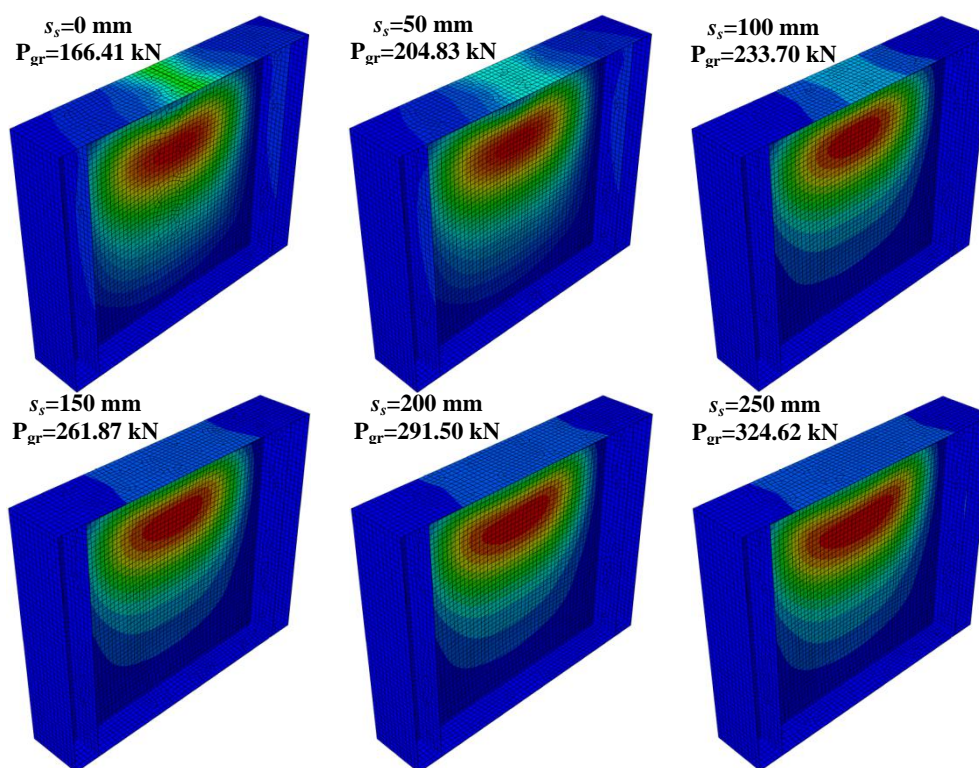


Slika 5 – Uticaj debljine nožice na ponašanje nosača pri graničnom opterećenju

Analizom rezultata utvrđeno je da sa povećanjem dužine nanošenja opterećenja takođe dolazi do povećanja granične sile, Slika 6. Povećanje granične nosivosti u slučaju rebra debljine $t_w=2 \text{ mm}$ iznosi od 5% do 23%, za $t_w=4 \text{ mm}$ od 7% do 35%, za $t_w=6 \text{ mm}$ od 14% do 36% i za $t_w=8 \text{ mm}$ od 18% do 28%. Pri tome, veći doprinos odgovara tanjim nožicama. Interesantno je napomenuta da je povećanje granične nosivosti u slučaju svih debljina rebra i

nožica za dužinu nanošenja opterećenja $s_s=250$ mm u odnosu na nosivost pri $s_s=200$ mm približno 10%.

Analizom deformisanih oblika nosača i dijagrama sila-pomeranje utvrđeno je da se pri manjim debljinama rebra a većim debljinama nožice jasno može uočiti razlika između kritične i granične sile što nije slučaj sa nosačima većih debljina rebra pri istim ostalim parametrima, Slika 7. U slučaju veoma vitkih rebara izražena je postkritična nosivost, dok u slučaju manje vitkih rebara dolazi do plastifikacije rebra lokalno u zoni unosa opterećenja mnogo pre dostizanja kritične sile.

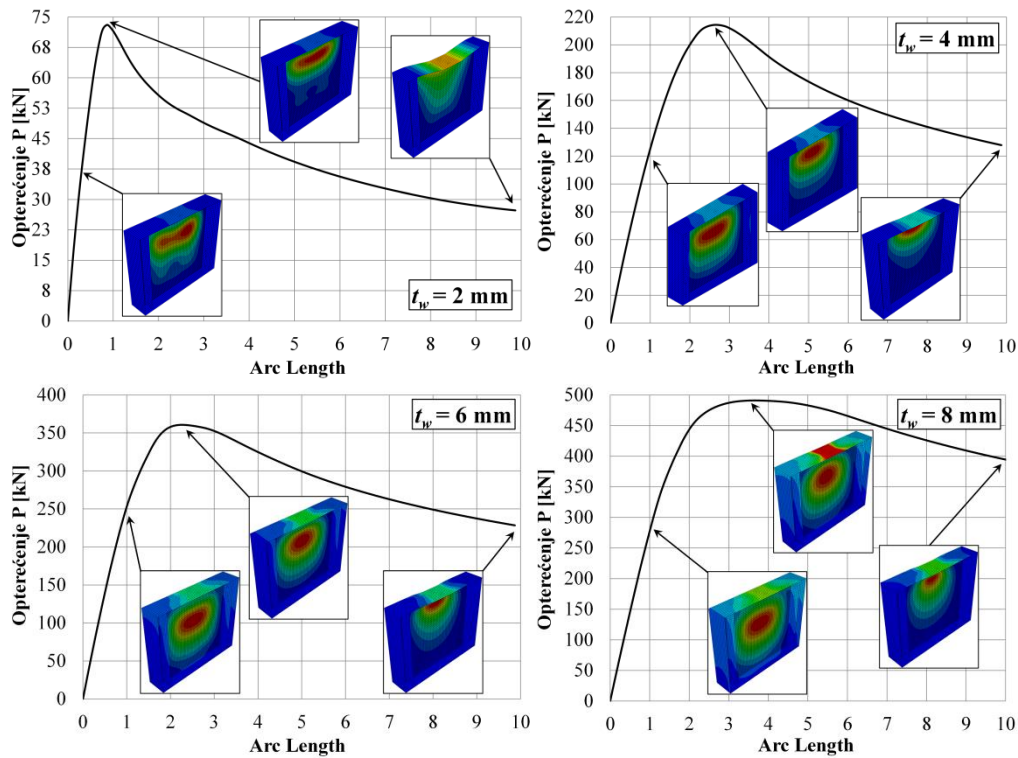


Slika 6 – Deformisani oblik i granično opterećenje za nosač $t_w=4$ mm i $t_f=12$ mm pri različitim dužinama opterećenja

Što se tiče dužine unošenja opterećenja, utvrđeno je sledeće: pri manjim dužinama opterećenja deformisanje rebra upravo na svoju ravan (izbočavanje), je lokalizovano u zoni ispod nanošenja opterećenja a takođe se rasprostire u ograničenoj uskoj zoni u podužnom pravcu nosača. Povećavanjem širine nanošenja opterećenja dolazi do pomeranja oblasti izbočavanja u vertikalnom pravcu, kao i do obuhvatanja veće zone u podužnom pravcu (deformacije se šire ka oslonačkim ukrućenjima).

Za krajnji slučaj razmatran u ovoj parametarskoj analizi (nosač debljine rebra $t_w=8$ mm, debljine nožice $t_f=20$ mm i dužine nanošenja opterećenja $s_s=250$ mm) deformacije pri graničnoj nosivosti ovog nosača se raspodjeljuju do granica vertikalnih oslonačkih ukrućenjima, tako da u ovom slučaju oslonačka ukrućenja utiču na deformaciju i nosivost nosača, na šta treba obratiti pažnju u daljim istraživanjima.

Pošto se usvojeni numerički model pokazao kao validan i tačan za opisivanje ponašanja nosača iz predmetne problematike, neophodno je izvršiti dodatna ispitivanja razmatrajući i ostale uticajne parametre kako bi se mogao doneti adekvatan zaključak u pogledu granične nosivosti razmatranih nosača.



Slika 7– Karakteristične krive opterećenje-arc length za ceo model za različite debljine rebra, $t_f=8$ mm i $s_s=100$ mm

Pored predstavljene analize rezultata potrebno je napomenuti da su u pripremnom radu [13] prikazane mogućnosti numeričkog modeliranja iz navedene oblasti. Predstavljene su osnovne karakteristike numeričkog modela, njegova opravdanost u pogledu eksperimentalnog ispitivanja i način kojim se početne geometrijske imperfekcije uzimaju u obzir. Na osnovu toga, stvoreni su uslovi za dalja numerička istraživanja i simulacije. Istraživanje iz predmetne oblasti će se nastaviti razmatrajući ostale parametre (osnovne dimenzije nosača, različiti odnosi

debljine nožice i rebra, vitkost rebra, različite karakteristike materijala i dr.) u cilju definisanja predloga i izraza za graničnu nosivost nosača za praktičnu primenu.

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NUMERIČKA ANALIZA BOČNO-TORZIONOG IZVIJANJA U PROFILA OD ALUMINIJUMSKE LEGURE

Rezime:

U radu je prikazana analiza bočno-torzionog izvijanja elemenata U poprečnog preseka od aluminijumske legure. Analizirani elementi su statičkog sistema proste grede, opterećeni linijskim opterećenjem koje deluje u nivou gornje flanše, u ravnima koje prolaze kroz težište preseka i kroz sredinu gornje flanše. Numerička analiza je sprovedena korišćenjem programskog paketa ABAQUS. Svrha ove analize je procena prikladnosti izraza za elastičan kritični moment bočno-torzionog izvijanja elemenata konstantnog poprečnog preseka, simetričnog samo u odnosu na jaču osu inercije, datog u EN 1999-1-1.

Ključne reči: kritični moment, U profil, bočno-torziono izvijanje, ABAQUS

NUMERICAL LATERAL BUCKLING ANALYSIS OF ALUMINIUM ALLOY BEAMS WITH CHANNEL SECTIONS

Summary:

This research paper presents the numerical analysis of lateral-torsional buckling of aluminium alloy beams with channel cross-section. Analysed structural elements are simply supported beams, with line load uniformly distributed over the top flange, in the direction towards the centroid of the cross-section and in the middle of the top flange. Numerical analysis is conducted using the finite elements software ABAQUS. The purpose of this analysis is estimation of appropriateness of the equation for elastic critical moment for lateral buckling of beams with uniform cross-sections, symmetrical only about the major axis, according to EN 1999-1-1.

Key words: critical moment, channel section, lateral-torsional buckling, ABAQUS

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1 INTRODUCTION

The lateral-torsional buckling phenomenon of mono-symmetric cross-sections that are symmetrical about the minor axis and of doubly-symmetric cross-sections is well understood. However, civil engineers encounter difficulties when the geometry of the cross-section does not coincide with the geometry for which the expressions are available in the standards. Example of this problem is a channel section beam. If the horizontal eccentricity of the load relative to shear centre is included, then the problem becomes even greater. The research presented here is instigated by the insufficiency of available information for determining the lateral-torsional buckling resistance of a channel section beam that is eccentrically loaded relative to shear centre. The entire paper is only one part of the Master's thesis, which, in addition to the analysis of the critical moment for lateral-torsional buckling, focuses on the ultimate bending resistance of a real channel section beam with initial imperfections.

2 DESIGN AGAINST LATERAL BUCKLING ACCORDING TO EUROPEAN STANDARDS

Lateral-torsional buckling of a structural element with cross-section asymmetrical about the minor principal axis, such as a channel section beam, is not thoroughly covered in the corresponding European standards. In EN 1993-1-1, the expression for calculation of the critical moment of C-section beams is not provided. However, the necessary equation is available in the EN 1999-1-1, in annex I, which addresses the lateral-torsional buckling phenomenon. Important to mention regarding this particular expression is the fact that it is only valid in a situation where the load is applied in the vertical plane which goes through shear centre. Considering the possible load positions relative to shear centre on the channel section structural elements, it can be concluded that this expression is relevant only for theoretically possible scenarios, not for what can be expected in real situations. The expression given in the EN 1999-1-1 has the following form:

$$M_{cr} = \mu_{cr} \frac{\pi \sqrt{EI_z GI_t}}{L} \quad (1)$$

where relative non-dimensional critical moment is:

$$\mu_{cr} = \frac{C_1}{k_z} \left[\sqrt{1 + \kappa_{wt}^2 + (C_2 \varsigma_g - C_3 \varsigma_j)^2} - (C_2 \varsigma_g - C_3 \varsigma_j) \right] \quad (2)$$

non-dimensional torsion parameter is:

$$\kappa_{wt} = \frac{\pi}{k_w L} \sqrt{\frac{EI_w}{GI_t}} \quad (3)$$

relative non-dimensional coordinate of the point of load application relative to shear centre is:

$$\varsigma_g = \frac{\pi z_g}{k_z L} \sqrt{\frac{EI_z}{GI_t}} \quad (4)$$

relative non-dimensional cross-section mono-symmetry parameter is:

$$\zeta_j = \frac{\pi z_j}{k_z L} \sqrt{\frac{EI_z}{GI_t}} \quad (5)$$

Following parameters are included in the previous equations:

E - modulus of elasticity,

L - length of the beam between lateral restraints,

I_w - warping constant,

G - shear modulus,

I_t - torsion constant,

I_z - moment of inertia about the minor principal axis,

k_z and k_w - buckling length factors, which are dependent on the boundary conditions,

C_1 , C_2 and C_3 - factors which are dependent on the shape of the bending moment diagram, end restraint conditions, type and position of the load and factor of asymmetry of the cross-section about the major principal axis,

z_g and z_j - load position coordinates relative to shear centre.

It can be demonstrated that, if the equation for critical moment of lateral-torsional buckling is rearranged in a manner that all of the dimensionless parameters and coordinates of the loading position are inserted in expression (2) in their expanded form, it becomes the same as the equation given in EN 1993-1-1:

$$\begin{aligned} M_{cr} &= \mu_{cr} \frac{\pi \sqrt{EI_z GI_t}}{L} \\ &= \frac{\pi \sqrt{EI_z GI_t}}{L} \frac{C_1}{k_z} \left[\sqrt{1 + \kappa_{wt}^2 + (C_2 \zeta_g - C_3 \zeta_j)^2} - (C_2 \zeta_g - C_3 \zeta_j) \right] \\ &= \frac{\pi \sqrt{EI_z GI_t}}{L} \frac{C_1}{k_z} \left[\sqrt{1 + \frac{\pi^2 EI_w}{(k_w L)^2 GI_t} + \frac{\pi^2 EI_z}{(k_z L)^2 GI_t} (C_2 z_g - C_3 z_j)^2} - \frac{\pi}{k_z L} \sqrt{\frac{EI_z}{GI_t}} (C_2 z_g - C_3 z_j) \right] \\ &= C_1 \frac{\pi \sqrt{EI_z GI_t}}{k_z L} \cdot \frac{\pi \sqrt{EI_z}}{k_z L \sqrt{GI_t}} \left[\sqrt{\frac{(k_z L)^2 GI_t}{\pi^2 EI_z} + \left(\frac{k_z}{k_w}\right)^2 \frac{I_w}{I_t} + (C_2 z_g - C_3 z_j)^2} - (C_2 z_g - C_3 z_j) \right] \\ &= C_1 \frac{\pi^2 EI_z}{(k_z L)^2} \left[\sqrt{\left(\frac{k_z}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(k_z L)^2 GI_t}{\pi^2 EI_z} + (C_2 z_g - C_3 z_j)^2} - (C_2 z_g - C_3 z_j) \right] \quad (6) \end{aligned}$$

This is the general formula which refers to the standard mono-symmetric cross-sections, which are symmetric about the minor axis. In this case, the only difference between critical moment of steel and aluminium element lies in Young's modulus of elasticity and in factors C_1 , C_2 and C_3 . In a situation such as this, when factors k_z and k_w are equal to 1, factors C_1 , C_2 and C_3 barely represent a difference, when comparing steel and aluminium simple beam elements with identical cross-sections, geometry and boundary conditions, loaded with uniformly distributed line load. The expression (1) is valid for cross-sections symmetrical about the minor axis; therefore, in order for it to be valid for cross-sections symmetrical only about the major axis, with load perpendicular to the major axis, directed towards the shear centre, the only

change that needs to be made is the exclusion of the z_j coordinate. If the same change is made in the expression (6), it becomes the expression for doubly-symmetric I cross-sections. Because of the commonality of these expressions - on the one hand for cross-sections symmetrical only about the major axis, with load applied perpendicular to the major axis in the plane going through the shear centre, which is given in EN 1999-1-1, and, on the other hand, for doubly-symmetric I cross-sections, where load is applied in the symmetry plane, which is given in EN 1993-1-1, the simulation of lateral-torsional buckling using commercial finite element software ABAQUS is conducted. The main purpose is determination of deviations between the values obtained using numerical method and values obtained according to the expressions available in the standard.

3 SIMULATION USING FINITE ELEMENTS SOFTWARE ABAQUS

The basic problem related to the calculation of the lateral-torsional buckling capacity of a channel section member is the eccentricity of the load in horizontal direction relative to shear centre. The position of the shear centre in the C-section is not in the material part of the cross-section; therefore, load application in the point which represents the shear centre is not as straightforward as it may seem to be. Using finite elements software ABAQUS, simply supported beams with channel cross-section are modelled. The basic material used is aluminium alloy EN-AW 6061-T6/T651, with most important physical and mechanical properties shown in Table 1.

Table 1 - Physical and mechanical properties of the material EN-AW 6061-T6/T651

Density (kg/m ³)	Melting point (°C)	Poisson's ratio (-)	Elasticity modulus (N/mm ²)	Yield strength (N/mm ²)	Tensile strength (N/mm ²)	Elongation at failure (%)
2700	582-652	0,3	70000	240	290	≈15

The structural element that is analysed is manufactured using the process of extrusion, which means that the cross-section can be shaped arbitrarily. However, the modelling process required some modifications of the cross-section's geometry (Figure 1).

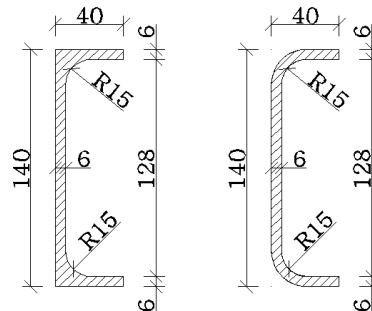


Figure 1 - Real cross-section and cross-section modelled in ABAQUS

As shown in Figure 1, the modelled cross-section has curved edges. Another option was using cross-section with sharp edges, neglecting the rounded parts; however, in a parallel

analysis of these two types of cross-section, it was observed that, in the case of the cross-section with sharp edges, the critical moment is rising with the increase of the horizontal eccentricity of the load; that kind of behaviour is not what should be counted on, because it is not safe to assume that the eccentricity is helping to increase the member's resistance against the lateral buckling. Therefore, the cross-section with curved edges was adopted because of the observed decrease of the critical moment with the increase of the horizontal eccentricity of the load relative to shear centre. The position of the shear centre is, referring to Figure 1, 12.36 mm left of the edge of the web. Parametric numerical analysis was carried out: the span lengths of the beams and position of the line load are varied. Three span lengths are analysed: 1.5 m, 2 m and 2.5 m, with the line load positioned on the top flange, directed towards the centroid and in the middle of the top flange. The full stress-strain relationship is not of particular interest in this analysis, because of the fact that the lateral-torsional buckling of the analysed structural elements is taking place in the elastic area far below the material proportionality limit, which represents the point in the stress-strain diagram in which the non-linearity effect is starting to gain influence. For material EN-AW 6061-T6/T651, the proportionality limit is in between 120 and 140 MPa, which significantly exceeds the stresses that can be achieved in the linear lateral-torsional buckling analysis of the elements described. The finite elements used for this analysis in ABAQUS software are S4R, doubly-curved four-node shell elements. The average size of the finite element is 6 mm in both directions, which remains constant throughout the analysis. Geometry was defined by sketching the cross-section, extrusion to the desired span length and by assignment of the thickness to the shell elements. The thickness of the web and flanges is 6 mm and it remains constant in all of the elements analysed. The most significant parameters for definition of the material are Poisson's ratio and Young's modulus of elasticity; for the numerical analysis purposes, it is sufficiently correct to assume the value of Poisson's ratio to be 0.3 and the value of Young's modulus of elasticity to be 70000 GPa. Assigning the material and defining the element's geometry is, generally, simple, when the topic is stability analysis of the described elements. Slightly greater challenge is defining the line load which is not applied on the free edge of the element, but which is positioned on the outside surface, in the specified position. In the finite elements software ABAQUS it is possible to apply the line load directly on the model consisted of shell elements, but only if it is applied on the free edge. In order for the load to be positioned on the outside surface – in this case on the top flange – the following steps are necessary:

- 1) Forming the partition so that it becomes possible to isolate a single line on which the load will be placed,
- 2) Creating the reference point which is positioned above the element, in the direction of the centre of the newly formed line,
- 3) Applying the concentrated force in the reference point,
- 4) Connecting the reference point and the line formed by partitioning of the element's face, using continuum distributing coupling constraints.

In this way, the concentrated force is distributed uniformly throughout the line. For purposes of the analysis, the values of the concentrated forces are equivalent to the uniformly distributed line load with magnitude of 1 kN/m, multiplied by the span length.

Besides modelling of the line load on the shell's surface, defining the material properties and forming the finite elements mesh, the next crucial step, which influences the results in a great manner, is the definition of the support conditions. The essential factor which requires attention while modelling the support conditions for the purposes of lateral-torsional buckling

analysis is the proper simulation of the cross-section's rotation about the major and minor principal axis. Acknowledging the fact that the analysed element is a simply supported beam, the support conditions should permit the rotation of the cross-section about the major and minor principal axis, as well as warping of the cross-section. Rotation about the longitudinal axis should be restrained, which corresponds to the assumption of the fork support conditions. At one end, the translation in the direction of the longitudinal axis is restrained, while on the other end it is allowed. On both ends the translation in the plane of the cross-section is restrained. One of the methods for defining the support conditions which will satisfy the above-mentioned is the application of the kinematic coupling constraints. The reference point, with purpose of kinematic connection of the support section's nodes and assignment of the unique support conditions for the entire cross-section, is defined in the centroid of the cross-section. In this case, the support conditions at one end keep the rotation about the longitudinal axis, as well as the translation in the direction of the longitudinal axis and the translation in the plane of the cross-section - restrained, while, at the other end, the same conditions remain, with the exception of the translation in the longitudinal axis direction, which is not restrained. Support conditions defined in this manner allow the requested rotations about the major and minor axis of inertia; however, deviations observed with respect to the value of the critical moment obtained from expression (1) are up to 20%, which is not negligible, considering the fact that the analysed elements are slender elements with relatively small spans. In other words, the critical moment obtained from the finite elements analysis is much greater than the one calculated using expression (1). For the purpose of explanation of these influences, the equivalent steel FE models with I cross-sections are formed. Two span lengths are analysed: 2.5 m and 3 m. The geometry is selected so that all of the cross-section's parts can be classified as class 1, 2 or 3 (Figure 2), according to the European standards, so that global instability would manifest itself as the first buckling mode, without the local buckling effects.

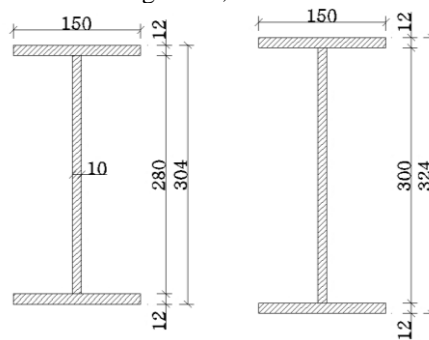


Figure 2 - Geometry of I cross-sections for 2.5m and 3m spans, respectively

In both cases, deviations observed with respect to expression (6) reach the value of 54% for both models. It means that, with supports defined as it was previously explained, the critical moment is more than two times greater than the critical moment expected, based on expression (6). In both cases, critical moments obtained from expression (6) are 100.67 kNm and 79.3 kNm, respectively. The values obtained from the finite elements analysis are 219.44 kNm and 169.52 kNm, respectively. Since this type of support conditions did not appear to be sufficiently flexible, new support conditions are designed. The reference point in the centroid of the cross-section is kinematically connected only with three nodes closest to the centroid and

the translation in the direction of the longitudinal axis is restrained in that point (Figure 3). It is possible that, in previously described support conditions, the constraints defined in such manner are restraining the longitudinal displacements of the nodes farther away from the centroid, which results in decreased rotational capacity of the cross-section. In all of the nodes of the support cross-section the translation in the plane of the cross-section and the rotation about the longitudinal axis of the element are restrained.

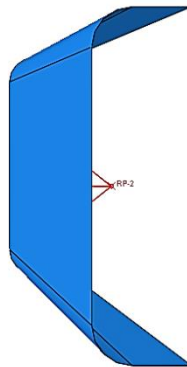


Figure 3 - Nodes connected to the centroid of the C-section beam

The support conditions, defined as described above, are tested in the analysis of I cross-section elements. The results obtained from this analysis are different from those obtained from expression (6) by no more than 2%, which is a satisfying precision. Shown in Figure 4 are the first buckling modes and the corresponding eigenvalues of the analysed I-section beams. The values of critical moments are 98.55 kNm and 78.41 kNm, respectively.

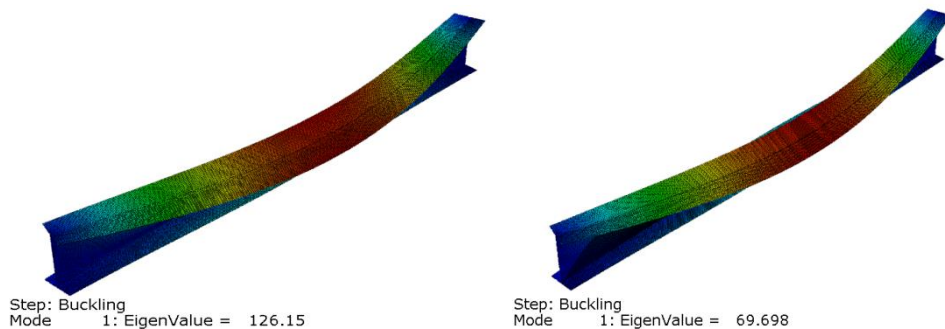


Figure 4 - First buckling modes and eigenvalues of the I-section beams

4 RESULTS AND DISCUSSION

After the conducted buckling analysis of the channel cross-section simple beam elements with spans of 1.5 m, 2 m and 2.5 m, with load position on the top flange level, directed towards the centroid and in the middle of the top flange, appropriate buckling modes and eigenvalues are obtained (Figure 5a-5c). On the left side, the results are presented for the situation when the

load is applied on the top flange, in the plane that goes through the centroid of the cross-section and on the right side - when the load is applied in the middle of the top flange.

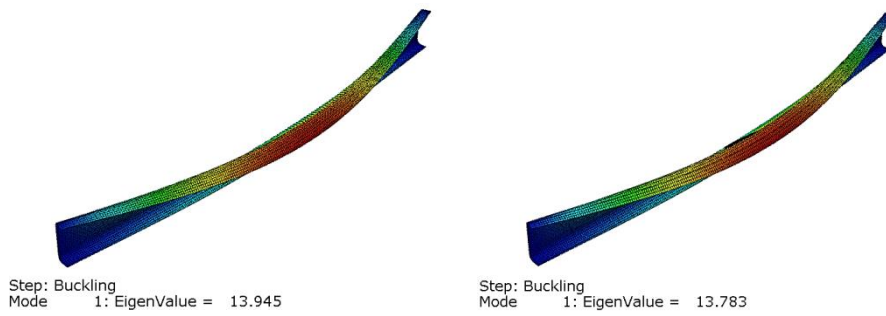


Figure 5a - First buckling modes and eigenvalues of channel beams with span length of 1.5 m

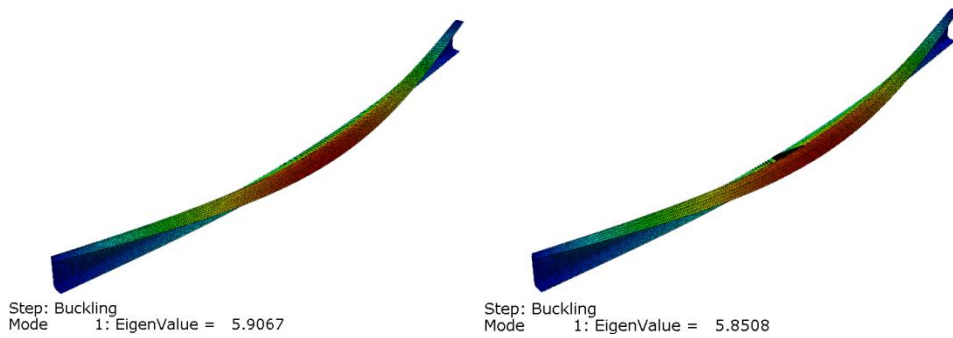


Figure 5b - First buckling modes and eigenvalues of channel beams with span length of 2 m

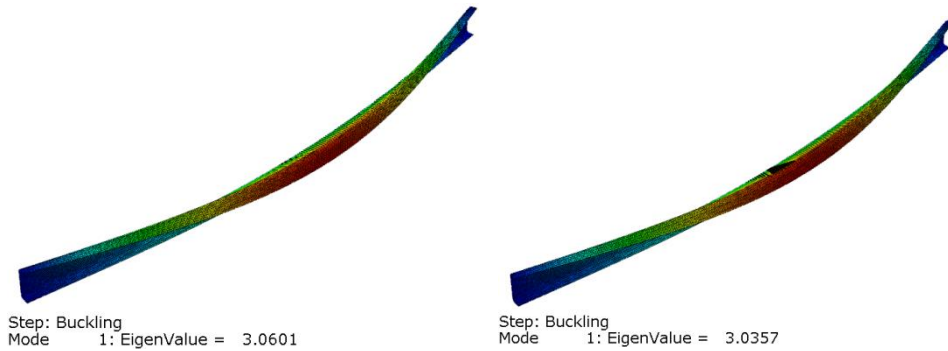


Figure 5c - First buckling modes and eigenvalues of channel beams with span length of 2.5 m

The value of the critical moment of lateral-torsional buckling can be calculated using a well-known formula for maximum bending moment in the middle of the span of a simply supported beam:

$$M_{cr} = qL^2 / 8 \quad (7)$$

where q is the line load which can be calculated by multiplying the applied line load with magnitude of 1 kN/m by the factor represented by the eigenvalue obtained from the buckling analysis. Values of the critical moments calculated using expression (7) were then compared to the values obtained from expression (1), with the remark that geometrical properties used in expression (1) correspond to the cross-section with rounded edges. In the interest of better transparency of the comparison of the results obtained using numerical method and the values calculated using expression (1), the results are shown in the Table 2.

Table 2 - Comparison of the critical moments

Span length	1,5 m		2 m		2,5 m	
Load position	Centroid	Middle of the flange	Centroid	Middle of the flange	Centroid	Middle of the flange
Abaqus (kNm)	3,922	3,876	2,953	2,925	2,391	2,372
EC9 (kNm)	3,926	3,926	2,966	2,966	2,408	2,408
Deviation (%)	0,1	1,27	0,44	1,38	0,71	1,5

As shown in Table 2, for analysed spans and load positions, deviations of the values obtained using finite elements software ABAQUS from the values calculated using expression (1) do not exceed the value of 1.5%, which can be considered as a very good precision. Critical moment values shown in Table 2 that are calculated using the European standard for aluminium structures design are only relevant for the situation when the applied load is directed towards the shear centre. The load defined in ABAQUS is horizontally eccentric, which means that, acknowledging the fact that values of the critical moment are decreasing with the increase of the eccentricity, it is logical to assume that the lateral-torsional buckling resistance will be increased if the load is applied in the plane that is going through the shear centre. In that case, it is possible that the values of the critical moment obtained from the ABAQUS software surpass the values calculated using expression (1). That means that, in theoretically possible situations, when the load is directed towards the shear centre, expression (1) could be used with a certain safety factor relative to the results obtained using the software. Considering that the deviations shown in Table 2 are relatively small, it becomes conceivable that, if the formula for the critical moment of lateral-torsional buckling is modified in a way that the horizontal eccentricity effect is included, the values calculated using the formula could have a safety factor relative to the values obtained from the numerical analysis. However, that is what further analysis should determine.

5 CONCLUSIONS

Presented in this research paper is the lateral-torsional buckling analysis of aluminium alloy channel cross-section beams. Material of which the element is comprised is EN-AW 6061-

T6/T651. The greatest difficulty encountered while calculating the critical moment using the equations provided in the standards is the horizontal eccentricity of the load relative to shear centre. European standard for design of aluminium structures EN 1999-1-1 addresses the topic, but not in a way that would remove all of the uncertainties, because of the assumption that the vertical load is directed towards the shear centre, which is primarily theoretically possible, considering the position of the shear centre in the C cross-section. The elements analysed are simple beams with span lengths of 1.5 m, 2 m and 2.5 m, with line load positioned on the top flange, directed towards the centroid, in one case, and applied in the middle of the top flange, in the other. Conclusions which can be derived from the presented analysis are:

- Expression for calculation of the critical moment of beams with cross-sections symmetrical about the major principal axis, with load applied perpendicular to the major axis in the direction of the shear centre, which is available in EN 1999-1-1, annex I, is the same expression given in EN 1993-1-1, for calculation of the critical moment of the beams with standard doubly-symmetric I cross-sections, with the load applied in the symmetry plane.
- The depicted method of modelling using finite elements software ABAQUS provides satisfying results, in terms of buckling analysis, when used for doubly-symmetric I-section beams. For models analysed, the deviations of the FEA values obtained using ABAQUS from the values calculated using the formula for critical moment provided in EN 1993-1-1 are in between 1% and 2%.
- The deviations between the FEA values of the critical moment of channel section beams and the corresponding values calculated according to EN 1999-1-1 are in range from 1% to 1.5%.
- FEA critical moment values are decreasing with the increase of the horizontal eccentricity of the load; therefore, it can be concluded that the lateral-torsional buckling capacity would increase if the load is applied in the plane directed towards the shear centre. Considering relatively small deviations from the values calculated using the standard, it is assumed that the expression for critical moment could provide results with a certain safety factor relative to the FEA values, if the expression is modified so that it includes horizontal eccentricity of the load. However, further analysis is necessary in order for that assumption to be proven.

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PREDVIĐANJE DUKTILNOG LOMA U RAZLIČITIM NAPONSKIM USLOVIMA

Rezime:

S-30

Lom duktilnog materijala predstavlja kraj plastičnog deformisanja kome prethode faza elastične deformacije i period velike plastične deformacije. Predviđanje trenutka pojave razaranja materijala ima izuzetan značaj kako za tehnologije oblikovanja materijala plastičnom deformacijom, tako i za analizu ponašanja metalnih materijala u različitim konstrukcijama (građevinskim, mašinskim, elektro i drugim). U ovom radu pokazano je kako se primenom savremenih empirijskih metoda može predvideti trenutak pojave loma, odnosno vrednost deformacije pri lomu materijala u različitim naponskim uslovima.

Key words: kriterijumi plastičnog loma, granična deformabilnos, naponsko stanje.

DUCTILE FRACTURE PREDICTION BASED ON STRESS STATE

Summary:

Ductile fracture of material represents end of plastic deformation. It is preceded by elastic deformation phase, followed by a large plastic deformation. It is of great importance to predict the moment of the material fracture in metal forming technology and to analyze behavior of metallic materials in different loaded structures (building constructions, mechanical and electrical structures and others). In this paper the application of contemporary empirical methods for determining the moment of fracture and the value of strain at that moment in different stress conditions is presented.

Key words: ductile fracture criteria, formability limit, stress state.

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1 INTRODUCTION

Ductile fracture represents material fracture preceded by large plastic deformation. Fracture stages in tensile test (Figure 1) are

- Elastic deformation,
- Uniform plastic deformation, which occurs from yield strength to ultimate tensile strength. In this stage starts void nucleation followed by necking between voids.
- Necking and non-uniform plastic deformation. In this stage void growth and void coalescence occur.
- Fracture

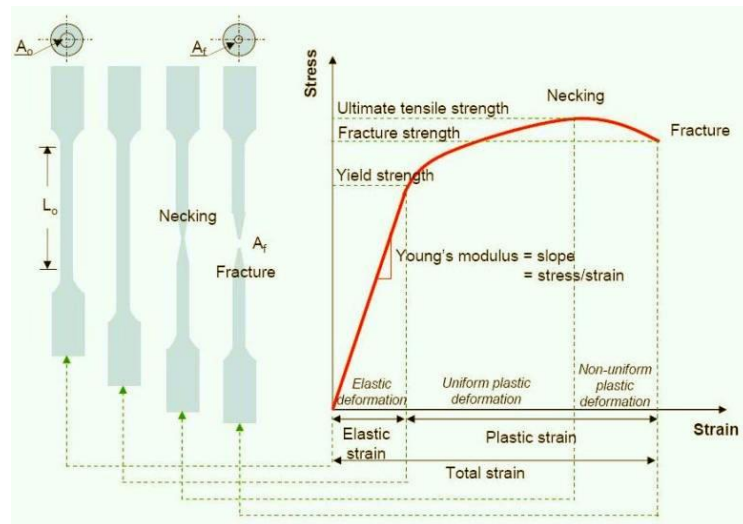


Figure 1 – Tensile test: Shape of ductile specimen at different stages [22]

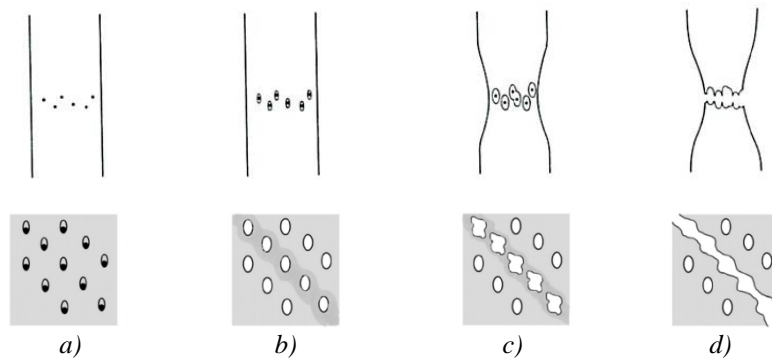


Figure 2 – Ductile fracture stages in tensile test: a) void nucleation, b) necking between voids, c) void growth and d) void coalescence [22]

Value of uniform plastic strain depends on the type of material, chemical composition, material microstructure, prior deformation of metal and heat treatment. During uniform plastic deformation voids in structure are generated and multiplied creating conditions for necking and non-uniform plastic deformation. During uniform plastic deformation stress state in material is uni axial, while in non-uniform plastic deformation (phase of strain localization) triaxial tension stress state occurs. This stress state is very unfavorable for forming processes. In this stage of process voids grow and coalesce causing material fracture [8]. Value of strain achieved to the moment of fracture represents limit strain . Various criteria are used to define the limit strain. Usually limit strain has a value at the moment when fracture occurs, but it can also be value of strain at the moment of necking (strain localization at deformation process) or strain at the moment of significant lowering of elastic modulus [19].

In tensile test limit strain can be easily predicted through standard experiment in laboratory conditions. However tensile test is a very simple in terms of stress state comparing to stress states in real metal forming processes or in complex building constructions. In these cases the fracture prediction is possible by plastic (ductile) fracture criteria application (both theoretical and empirical). The most important ductile fracture criteria are criteria Oyane, Cockcroft-Latham, Rousselier, Gurson, Needleman, Lemaitre, Tvergaard, Ruggieri, Pavankumar etc.

Empirical criteria of ductile fracture are based on experimental investigation of real processes of plastic deformation (forming processes) and they are presented as Forming Limit Diagram (FLD) in two variants: strain based criterion and stress based criterion.

1.1 STRAIN BASED CRITERION OF DUCTILE FRACTURE

Strain based criterion is forming limit diagram which represents relationship between two principal limit strains in the zone of fracture appearance: $\phi_2^l = f(\phi_1^l)$ [3,4,5,7,9]. The methodology for determining the forming limit diagram and its implementation in detail is proposed by Kuhn [7]. A formability test procedure has been developed using cylindrical upset tests and any other simple test of deformation. Variation of the upset specimen aspect ratio (height to diameter ratio) and method of lubrication of the compression dies produces a variety of stress and strain combinations at the free surface of the cylinders. Results show (Figure 3) that the surface tensile and compressive strains at fracture fit a straight line relationship having a slope of $-1/2$ and an intercept along the tensile strain axis.

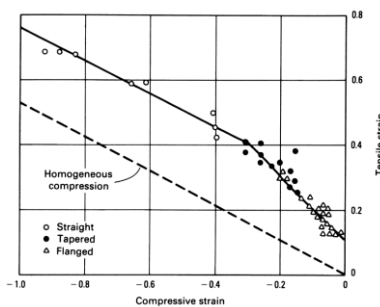


Figure 3 - FLD for steel AISI 1045 by strain criterion [7]

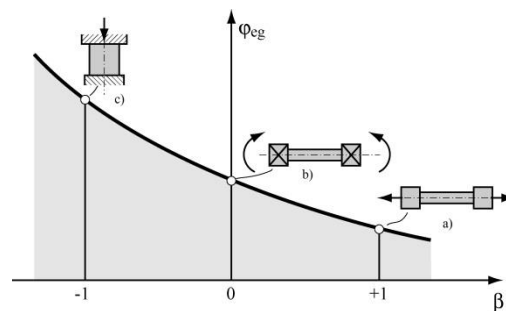


Figure 4 - Forming limit diagram by stress criterion [1]

1.2 STRESS CRITERION OF DUCTILE FRACTURE

The second variant of empirical ductile fracture criterion is stress based and represents relationship between limit strain (ϕ_e^l) and triaxiality stress indicator (β) in the critical zone of the specimen proposed by Vujović and Shabaik [1].

According to the criterion of Vujovic and Shabaik material formability (M_f) depends on material type (H), its micro structure (S), process temperature (T), strain rate ($\dot{\phi}$), stress state (T_σ) and other factors:

$$M_f = F(H, S, T, \dot{\phi}, T_\sigma, \dots) \quad (1)$$

Quantitative measure of limit formability is effective strain, ($\phi_e^l = M_f$), i.e. strain in the moment of material structure damage (fracture or strain localization). For the given material, with defined initial microstructure and in cold forming conditions by quasistatic deformation, material formability is a function only on stress state:

$$\phi_e^l = F(T_\sigma) = F(\beta) \quad (2)$$

where is:

T_σ – stress tensor

β – triaxiality stress indicator at the critical zone of specimen, i.e. at the point of structure damage. Stress indicator is defined as:

$$\beta = \frac{\sigma_x + \sigma_y + \sigma_z}{\sigma_e} = \frac{\sigma_1 + \sigma_2 + \sigma_3}{\sigma_e} \quad (3)$$

where are:

$\sigma_x, \sigma_y, \sigma_z$ – normal stress components in three orthogonal directions (x, y, z)

$\sigma_1, \sigma_2, \sigma_3$ – components of principal normal stress

σ_e – effective stress.

Graphical interpretation of the relationship of limit strain and triaxiality stress indicator, equation (2), is the forming limit diagram – FLD (Figure 4). Forming limit diagram shows that in bulk metal forming processes in which compressive stresses prevail ($\beta < 0$) higher values of limit strains can be achieved than in the processes in which tensile stresses are predominant ($\beta > 0$).

Forming limit diagram (Figure 4) is determined experimentally, by employing several basic deformation models (Vujovic and Shabaik [1]):

- a) Uniaxial tensile test, $\beta = +1$
- b) Torsion test, $\beta = 0$
- c) Uniaxial compression test, $\beta = -1$.

Alexandrov et al [17] proposed new methodology for forming limit diagram determination when stress indicator is +1 ($\beta \approx +1$). Instead of tensile test (uniaxial tension) authors suggested use of upsetting test of cylindrical billet with ring.

For more detailed construction of FLD application of more sophisticated methods is needed.

In the case of non-monotonous processes stress indicator (β) changes during deformation and its average value is inserted in the FLD diagram [15,16,17,19,21]. Average value of stress indicator is defined as:

$$\beta_{av} = \frac{1}{\varphi_e^I} \int_0^{\varphi_e^I} \beta(\varphi_e) d\varphi_e \quad (4)$$

where is:

$\beta(\varphi_e)$ – history of triaxiality stress indicator, which indicates change of stress-state as a function of effective strain during of process deformation.

By determination of triaxiality stress indicator (β), for the case that material damage occurs at free surface of the specimen, two methodologies, based upon deformation theory i.e. flow theory, are applied.

In the paper Vujovic and Shabaik [1] determination of stress components at free surface of cylinder (equatorial surface, Figure 5) was conducted by employing deformation theory, i.e. by using following relations:

a) Stress-strain relationship:

$$\frac{d\varphi_\theta}{\sigma_\theta - \sigma_m} = \frac{d\varphi_z}{\sigma_z - \sigma_m} \quad (5)$$

b) Hydrostatic stress:

$$\sigma_m = \frac{\sigma_r + \sigma_\theta + \sigma_z}{3} = \frac{\sigma_\theta + \sigma_z}{3} \quad (6)$$

c) Misses yield criterion:

$$\sigma_\theta^2 + \sigma_z^2 - \sigma_\theta \cdot \sigma_z = \sigma_e^2 \quad (7)$$

By combining of above equations stress components σ_z , σ_θ and stress indicator β at the free surface at the cylinder can be obtained:

$$\sigma_z = \pm \sigma_e \cdot \left[1 - \frac{1+2\alpha}{2+\alpha} + \left(\frac{1+2\alpha}{2+\alpha} \right)^2 \right]^{-1/2} \quad (8)$$

$$\sigma_\theta = \sigma_z \cdot \left(\frac{1+2\alpha}{2+\alpha} \right) \quad (9)$$

$$\beta = \frac{\sigma_r + \sigma_\theta + \sigma_z}{\sigma_e} = \frac{\sigma_\theta + \sigma_z}{\sigma_e} - \frac{1 + \frac{1+2\alpha}{2+\alpha}}{\sqrt{1 - \frac{1+2\alpha}{2+\alpha} + \left(\frac{1+2\alpha}{2+\alpha}\right)^2}} \quad (10)$$

where are: $\sigma_r, \sigma_\theta, \sigma_z$ – stress components at free surface of cylinder in the directions of corresponding axis (r, θ, z), whereas at free surface $\sigma_r = 0$; σ_m – hydrostatic stress; σ_e – effective stress;

$\varphi_r, \varphi_\theta, \varphi_z$ – logarithmic strain components in the directions of corresponding axis (r, θ, z),

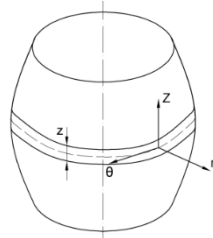


Figure 5 - Cylindrical specimen after axial compression by flat plates

In the equations (8,9,10) coefficient “ α ” is defined as:

$$\alpha = \frac{d\varphi_\theta}{d\varphi_z} \quad (11)$$

Prior to that, strain path is defined as:

$$\varphi_\theta = f(\varphi_z) = A\varphi_z + B\varphi_z^2 \quad (12)$$

A and B – coefficient of regressive curve.






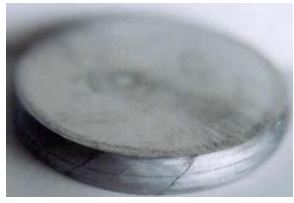



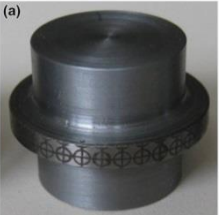

Effective strain in the moment of specimen fracture is defined:

$$\varphi_e^f = \frac{2}{\sqrt{3}} \int_0^{\varphi_z^f} \sqrt{(A + 2B\varphi_z)^2 + A + 2B\varphi_z + 1} d\varphi_z \quad (13)$$

2 EXPERIMENTAL DETERMINATION OF FLD

Analysis of ductile fracture and material formability is a permanent long-term project in Metal Forming Laboratory at Faculty of Technical Sciences, University of Novi Sad. Special attention has been paid to methodology of forming limit diagram determination. Some results were published in following papers [1, 2, 11–21]. Models of deformation (tests of deformation) used for experimental determination of FLD for steel C45, with data of average value of stress indicator (β_{av}) and limit strain (φ_e^f) values, for illustration, are presented in table 1.

Table 1 – Deformation processes for determination of limit strain of steel C45

Mark of test	Test of deformation	Specimen before deformation	Specimen after deformation	β_{av}	φ_e'
R	Rastegaev upsetting test (lubrication by stearin)			-0.96	1.74
Te	Cylinder upsetting (lubrication by Teflon)			-0,677	1.454
RP (R)	Cylinder upsetting (lubrication by mineral oil – rough surface)			-0.40	1.12
RP (F)	Cylinder upsetting (lubrication by mineral oil – fine surface)			-0.25	0.94
T	Torsion test			0	0.72
CR	Upsetting of cylinder with ring (lubrication by stearin)			+1.24	0.32

In Figure 6 experimentally determined forming limit diagram for C45 steel is presented. Following tests were used for FLD determination:

- Upsetting test of cylindrical billet with ring,
- Torsion test,
- Upsetting of cylinder with different methods of lubrication
- Uniaxial compression test (Rastegaev test that eliminates friction in upsetting test).

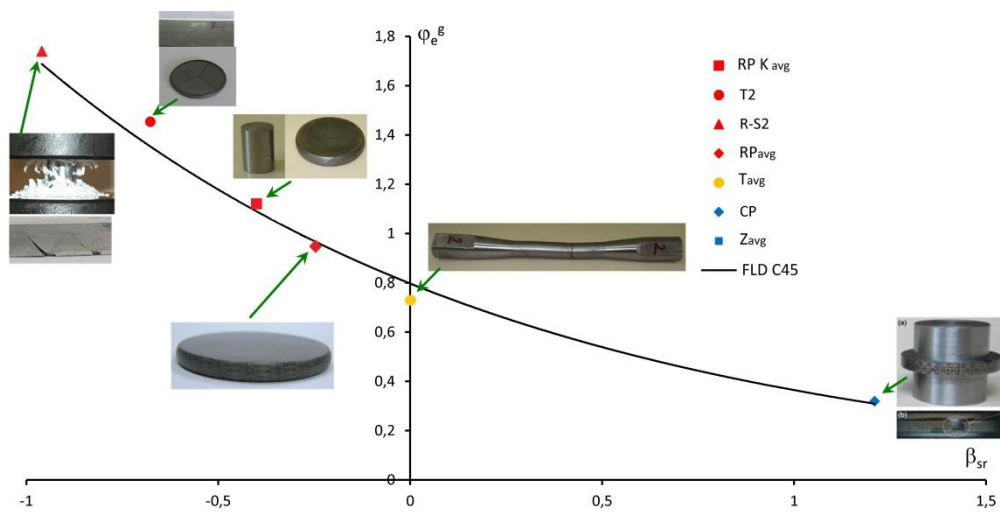


Figure 6 – Forming limit diagram of C45 steel [21]

3 CONCLUSION

Ductile fracture occurs at the end of the forming process, both in metal forming technology and in building and other technical constructions. Prediction of ductile fracture is significant in all these fields.

Based on the numerous investigations in the field of ductile fracture it can be concluded that the methodology for ductile fracture prediction based on forming limit diagram (stress based criterion) has significant advantages compared to theoretical criteria. Approach based on relationship between ductility of material and its stress state represents better solution than deformation model and theoretical criteria for plastic fracture.

Prediction of ductile fracture in the case of building construction elements and other structures can be fully carried out according to the methodology used in the failure analysis in metal forming technology. This implies experimental determination of FLD for the metal material, and analysis of stress-strain state of structures with the goal to determine the position of "critical zone", ie the zone where destruction occurs. It is also necessary to determine value of triaxiality stress indicator during deformation. Analysis of stress state can be carried out using theoretical solutions, theoretical-experimental and numerical methods.

Considering the importance of triaxiality stress indicator at the ductile fracture it is necessary to point out that the pressure load of the material is favorable comparing to tension load from the standpoint of its impact on the fracture occurrence. Shear stress, by its impact on the ductile fracture, is between compressive and tensile stresses (FLD, Figure 6).

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EKSPERIMENTALNO ISPITIVANJE X-HVB MOŽDANIKA U PREFABRIKOVANIM SPREGNUTIM PLOČAMA

Rezime:

Savremeni trendovi u građevinarstvu koji se ogledaju u povećanim zahtevima u pogledu ubrzane gradnje i što manjem obimu radova na gradilištu, uslovili su razvoj različitih vrsta prefabrikovanih ploča i grupnu raspodelu moždanika. Kratak prikaz osnovnih karakteristika X-HVB moždanika i eksera za vezivanje moždanika sa čeličnim profilom dat je u ovom radu. Ekspreimentalno ispitivanje X-HVB moždanika kroz standardni test smicanja, prema EC4:2004, prikazano je u ovom radu. Ispitivane su dve različite orijentacije moždanika u odnosu na pravac sile smicanja.

Ključne reči: X-HVB moždanik, prefabrikovane spregnute ploče, test smicanja

EXPERIMENTAL INVESTIGATION OF X-HVB SHEAR CONNECTORS IN PREFABRICATED COMPOSITE DECKS

Summary:

Increasing demands towards fast construction and smallest possible quantity of work resulted in development of different types of prefabricated composite decks and group arrangement of shear connectors. Short presentation of basic characteristics of X-HVB shear connectors and powder-actuated fasteners is given in this paper. Experimental investigation of X-HVB shear connectors through standard push-out tests, according to EC4:2004, is presented in this paper. Two different orientations of shear connectors, forward and backwards to the shear force are experimentally investigated.

Key words: X-HVB shear connectors, prefabricated composite decks, push-out test

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1 INTRODUCTION

Construction industry is constantly facing new demands towards fast construction and smallest possibly quantity of work at the construction site. Steel-concrete composite beams have been used in the construction of buildings and bridges for decades. In the field of steel-concrete composite structures, in-situ casted concrete decks often require great quantity of work at the construction site and additional time needed for concrete to achieve the necessary strength. In the recent decades, development of different types of prefabricated composite decks has taken an important place in the field of composite constructions.

Composite action between steel profile and prefabricated composite deck is achieved with group arrangement of shear connectors in envisaged openings of prefabricated decks. Less quantity of work at the construction site is related with smaller quantity of concrete needed to infill openings. The most important characteristics of X-HVB shear connectors and powder-actuated fasteners are shown in this paper. Also, results of experimental investigation of X-HVB shear connectors positioned in envisaged openings of prefabricated composite decks through standard push-out tests are presented in this paper. Different characteristic shear resistance and ductility is obtained for two examined orientations of shear connectors.

2 SHEAR CONNECTORS

Different types of shear connectors used for composite action between steel profile and concrete deck were developed during decades and are shown in Figure 9.

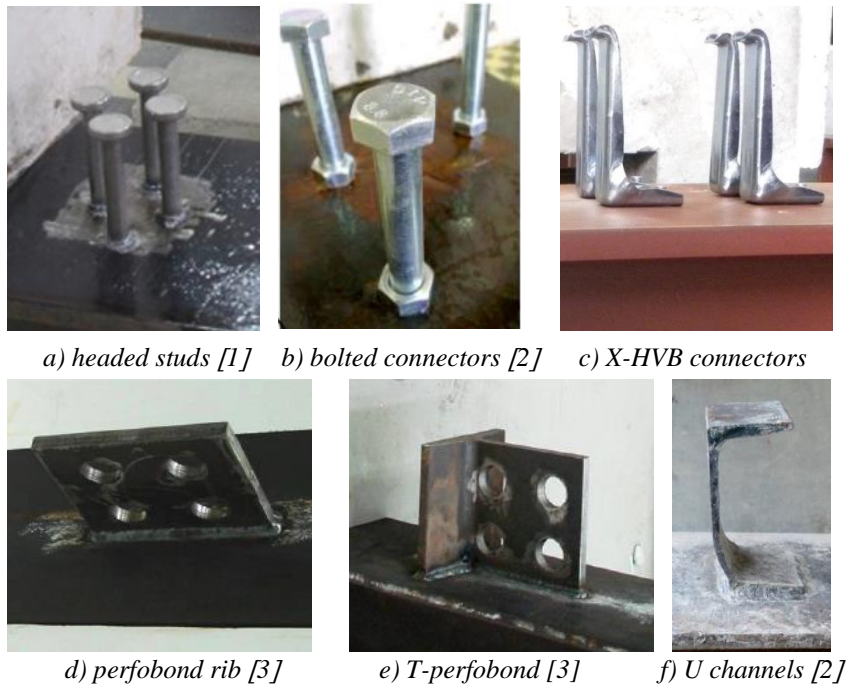


Figure 9 - Different types of shear connectors

The use of various types of shear connectors for composite constructions (see Figure 9) consider important differences regarding different quantity of work during installation, different preparation process of base material and appropriate atmospheric conditions during installation. Also, three main properties of shear connectors: shear resistance, stiffness and ductility are specific for each type of shear connector and need to be considered for their application in composite constructions.

Welded headed studs (Figure 1a) are the most often used shear connectors for steel-concrete composite decks of buildings and bridges. Design rules for those shear connectors can be found in current design codes, such as EC4:2004 [4] and ANSI/AISC-360-05 [5]. This type of connector is used worldwide, mainly due to a high degree of automation on work site. Also, it has some restrictions considering structures submitted to fatigue and requirements for specific welding equipment. Recent experimental and numerical investigations are related to the behavior of different group arrangement of headed studs positioned into envisaged openings of prefabricated composite decks through standard push-out tests [1]. The findings of the study [1] are that the reduction of the distance between the adjusted headed studs in force direction can be allowed. Experimental results confirmed that it is no need for reduction of studs' group shear resistance even in cases when the distance between the headed studs is smaller than the value prescribed by EC4:2004 [4]. Also full-scale beam experiments have been conducted in recent period in order to investigate strengthening of existing composite steel-concrete beams utilizing bolted shear connectors and welded studs [6].

Bolted shear connectors have been rarely used in construction, actually just for rehabilitation works, because there is a lack of detail research and design recommendations for this type of shear connector. Different types of bolted shear connectors can be used for composite action, such as: friction grip bolts, bolts without embedded nuts, bolts with single embedded nut (see Figure 1b) and bolts with double embedded nuts. Different stiffness, ductility and resistance is obtained for different types of bolted shear connectors and the most similarities considering welded headed studs are obtained with bolts with single embedded nut. Experimental and numerical investigation of bolted shear connectors in prefabricated composite decks, conducted in recent years, proposed connections with two nuts, one of which is embedded in the concrete as the best solution [2].

X-HVB shear connectors are used with powder-actuated fasteners for achievement of composite action between steel profile and composite decks. Basic characteristics of this type of shear connector and powder-actuated fasteners will be explained in this paper.

Perfobond rib (see Figure 1d) was developed in the late 1980s in attempt to overcome fatigue problems caused by live loads in composite bridges, and included rectangular steel plate with number of holes, welded to the beam flange. The resistance in both horizontal and vertical direction is enabled with concrete which flows through holes in steel plate. A numerous push-out and composite beam tests, as well as numerical simulations have been used in studies on the shear capacity and behavior of the perfobond rib connectors which resulted in different design recommendations of this type of connector. The structural response of perfobond rib connectors is influenced by several geometrical properties such as the number of holes, the plate height, length and thickness, the concrete compressive strength, and the percentage of transverse reinforcement provided in the concrete slab [3], [7]. T-perfobond connectors (see Figure 1e) were developed by adding a flange to the perfobond rib plate, which acts as a block combining the large strength of a block type connector with some ductility and uplift resistance arising from the holes at the perfobond rib connector. For similar longitudinal

plate geometries, the resistance and stiffness of T-perfobond connectors are higher than that of perfobond rib connectors.

Chanel shear connectors (see Figure 1f) offer higher amount of shear resistance due to its large contact area with surrounding concrete. This type of connector does not require special equipment and standard welding procedures are adequate for installation process. The results of the push-out test showed that the resistance of the composite construction can be affected by other factors apart from the concrete strength, which include flange thickness, web thickness and channel length. Design recommendations for this type of shear connector are given in current design codes, such as ANSI/AISC-360-05 [5] and CAN/CSA-S16-01 [8]. Experimental investigation of channel type shear connectors, which represent different European channel profiles named UPN profiles, with various heights and lengths and considering different concrete material properties has been performed in recent period [9], [10].

2.1 CHARACTERISTICS OF X-HVB SHEAR CONNECTORS AND POWDER-ACTUATED FASTENERS

X-HVB shear connectors (see Figure 1c) are connected to the steel profile with two powder-actuated fasteners and represent alternative for the headed studs and bolted shear connectors for construction of steel-concrete composite structures. Compared to the former mentioned types of shear connectors, X-HVB shear connectors have some advantages, such as:

- Minimal installation equipment and short installation time which are very important considering new demands towards fast construction. X-HVB shear connectors are directly fastened to steel profile with two X-ENP-21 HVB nails and require no welding. A portable, hand-held, powder-actuated fastening tool is used to drive a high-strength steel fastener directly into the base material.

- Installation quality is not affected by atmospheric conditions (such as moisture), by base material coatings (zinc coatings) or paints, resulting in less work interruptions due to atmospheric conditions or additional preparation of base material at the position of connectors. This ensures relatively simple and fast installation of X-HVB shear connectors for construction of new buildings and also for renovation of existing buildings.

- Simple check of installation quality, performed through visual checking of the stand-off over the surface of the fastened material, or for fasteners that don't allow an accurate visual check, the use of stand-off template is recommended. Installation quality recommendations for X-ENP-21 HVB nails used for X-HVB shear connectors are shown in Figure 10.

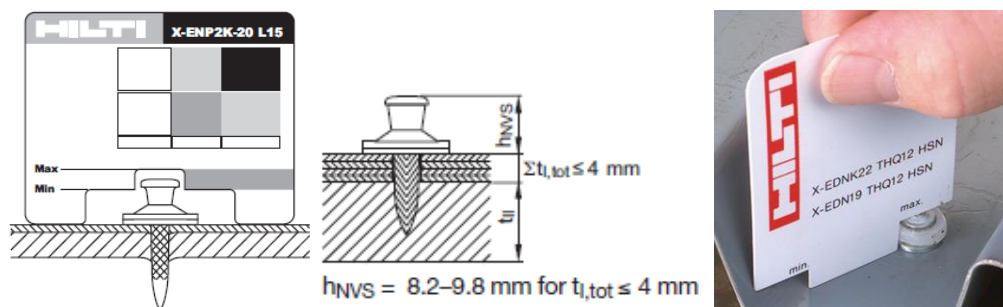


Figure 10 - Installation quality recommendations for X-ENP nails [11]

Powder-actuated fasteners are nails or threaded studs made from high-strength steel, used to fasten components to steel, concrete and masonry [12]. During installation process, depth of fastener penetration is the distance between upper surface of the base material and the point of the fastener. Fastener stand-off h_{NVS} is the distance from the head of the fastener to the surface of the fastened component materials with thickness $t_{I, tot}$. Fastener stand-off h_{NVS} is the reference dimension used to check the depth of penetration and thus the quality of the fastening installation (see Figure 10).

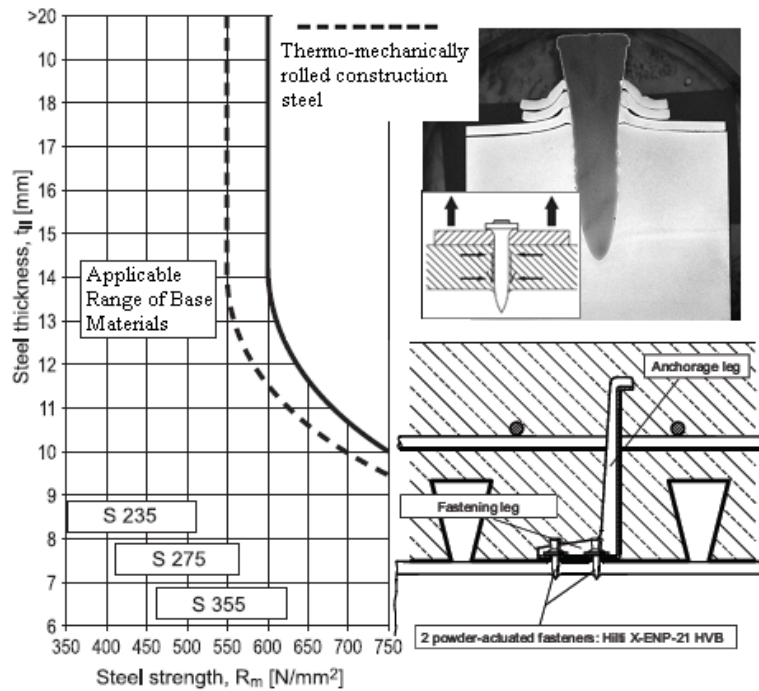


Figure 11 – Application limit diagram and clamping of the powder-actuated fastener [11]

Application range of the X-ENP-21 HVB nails used for installation of X-HVB connectors (see Figure 11) are determined by the thickness and strength of fastened component material (t_{II}) and base material.

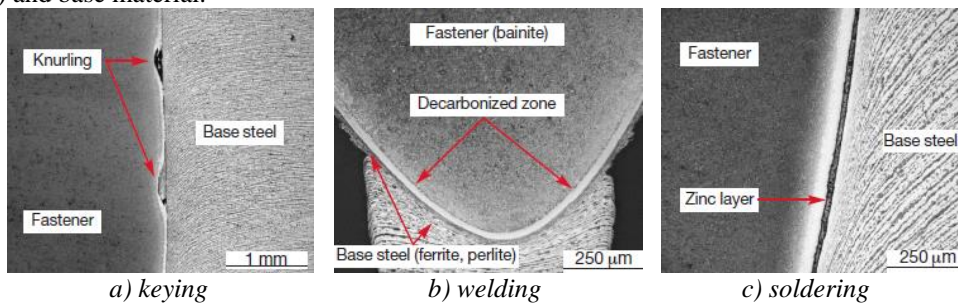


Figure 12 – Anchorage mechanisms of powder-actuated fastener [11]

The term “anchorage” refers to the hold obtained by the fastener in the base material [12]. Pull-out of the fastener from the base material is the result of the failure of the anchorage (see Figure 11). Metals with pronounced plastic behavior used for base material generally provide suitable anchorage for the powder-actuated fasteners. Different anchorage mechanisms and principles are pronounced considering powder-actuated fasteners such as clamping, keying, welding and soldering (see Figure 11 and Figure 12). Clamping is the primary anchoring mechanism. This anchoring mechanism is a result of the steel base material displacement radially and towards base material surface and fastener point during installation providing pressure on the nail surface and friction. Keying and welding are very important anchoring mechanisms for fasteners that do not penetrate through base material. Keying mechanism is characteristic for knurled fasteners, such as X-ENP-21 HVB nails. Knurled surface of the fastener enables accumulation of zinc and base material during installation process (see Figure 12a). Welding mechanism is observed mostly at the point of a fastener where the temperature during installation can be expected to be the highest and is influenced by the base material surface which is in the contact with fastener point and decarbonized zone (see Figure 12b). Soldered zinc layer between fastener and base material further form the fastener point (see Figure 12c) also contributes to the pull-out resistance and represent another anchoring mechanism.

X-HVB shear connectors are L shaped connectors which consists form fastening leg connected to the base material with the two powder-actuated fasteners (X-ENP-21 HVB nails) and anchorage leg which is cast into the concrete (see Figure 11). Shear resistance of different types of X-HVB connectors are determined through push-out tests and beam tests using solid concrete decks and composite decks and design recommendations are given in HILTI Direct Fastening Technology Manual [11]. X-HVB connectors show ductile behavior, considering recommendations given in EC4:2004 [4]. Shear resistance of this type of shear connectors is a combination of different parameters, such as hole elongation in the fastening leg, anchorage mechanisms, bending of the fasteners, deformation of the concrete in the connectors surrounding zone.

3 STANDARD PUSH-OUT TESTS

Standard push-out tests were prepared and examined in the Laboratory of Materials and Structures at the Faculty of Civil Engineering in Belgrade, according to EC4:2004 [4]. Experimental investigation of X-HVB shear connectors was performed in order to investigate the behavior of X-HVB shear connectors positioned into envisaged openings of full depth prefabricated composite decks, including influence of different orientations of connectors.

3.1 TEST SET-UP

Concrete slabs (600x650x140 mm) with standard reinforcement layout (ribbed bars $\phi 10$ mm, grade R500) are prefabricated by casting them in horizontal position. Openings with dimensions 240x240 mm in the middle of the slabs are envisaged for later assembly of shear connectors. X-HVB 110 shear connectors were installed to HEB 260 steel profile flange at the transversal distances of 50 mm and longitudinal distances of 100 mm, which are minimal distances recommended by HILTI [11]. Shear connectors are positioned in two directions,

forward (HSF specimen) and backwards (HSB specimen) to the shear force. Layout and dimensions of the specimens are shown in Figure 5.

Envisaged openings are filled in horizontal position with three-fraction concrete. In order to minimize initial cracks due to shrinkage, specimens are kept in wet condition during first three days. After three days, half assembled specimens are turned and second assembling phase is performed in the same way as the first one.

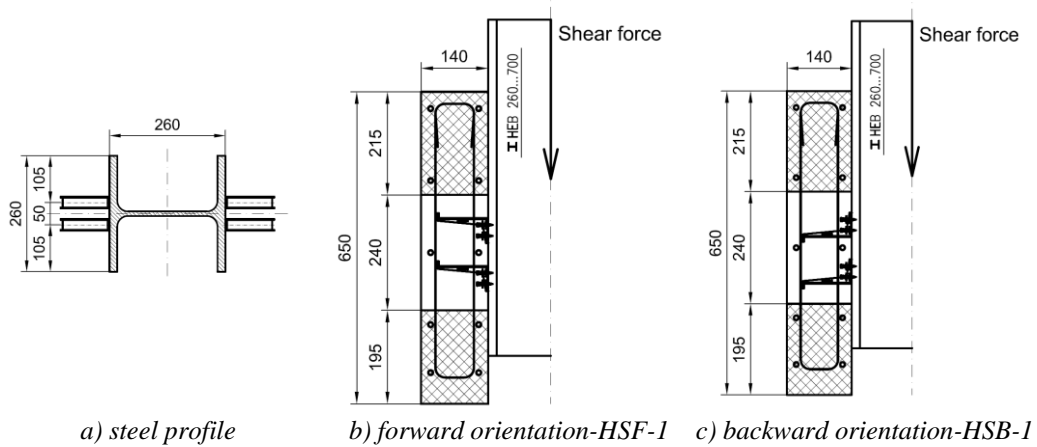


Figure 13 – Specimens layout

Test set-up for both specimens is shown in Figure 6. Longitudinal slip between the steel profile and both concrete slabs is measured with 4 sensors, two on each side. Separation between steel profile and concrete slabs is measured on the front side, as close as possible to connectors. Separation between prefabricated slabs is measured on the front side, 15 cm above the slab support.



a) testing frame

Figure 14 – Test set-up

The loading regime is adopted as specified in EC4:2004, Annex B [4]. Force controlled cycling load is applied in 26 cycles ranging from $F_{\min}=15$ kN to $F_{\max}=110$ kN, corresponding to approximately 5% and 40% of expected shear resistance. Shear resistance of eight connectors in one specimen is 280 kN, based on characteristic shear resistance of one X-HVB 110 connector $F_{Rk}=35$ kN, according to HILTI Direct Fastening Technology Manual 2014 [11].

3.2 EXPERIMENTAL RESULTS

Experimental investigation included different orientation of shear connector's (HSF-1 and HSB-1 specimens). Results of push-out tests for both specimens are shown in Figure 15 and Table 1.

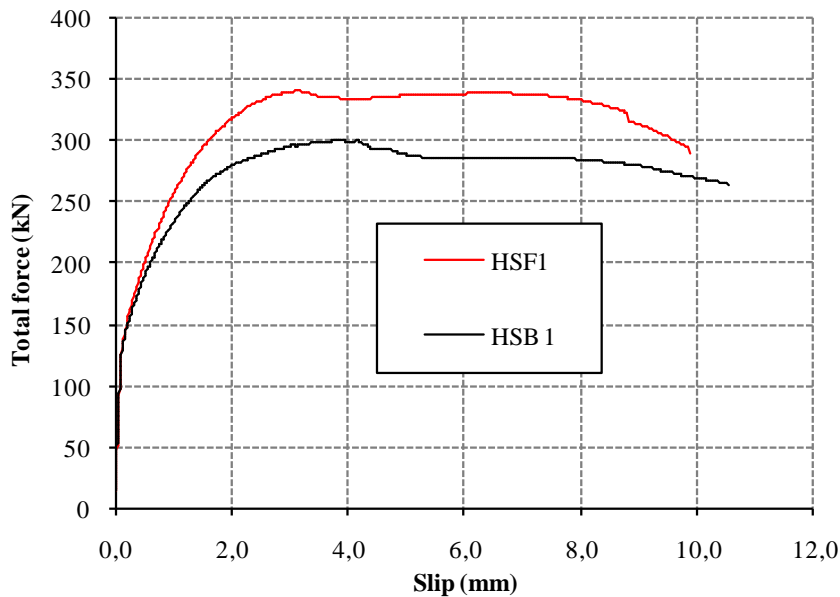
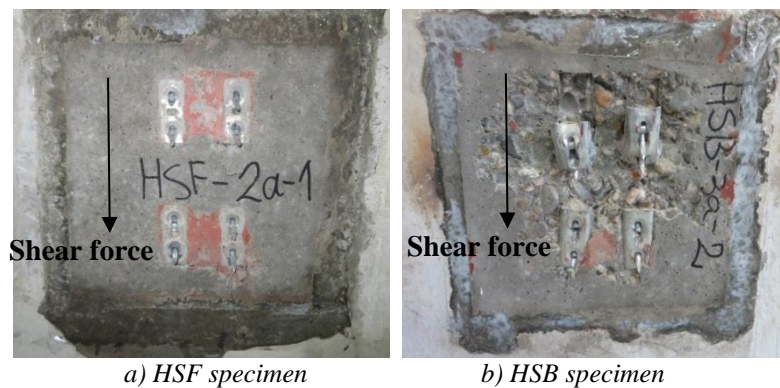


Figure 15 – Force-slip curves for HSF-1 and HSB-1 specimens



a) HSF specimen

b) HSB specimen

Figure 16 – Infill concrete after push-out tests

Shear force P_{ult} is defined as total ultimate force for all shear connectors of one specimen. Longitudinal slip between steel section and concrete slabs is measured with four sensors and divided in initial slip during cyclic loading δ_{init} and slip to failure δ_u , as total slip $\delta_{u,tot} = \delta_{init} + \delta_u$.

Table 2 – Results of standard push-out tests

Series	Ultimate force (kN)	Slip (mm)			Separation (mm)	
		initial	to failure	total	between slabs	steel to concrete
	P_{ult}	δ_{init}	δ_u	$\delta_{u,tot}$		
HSF-1	341.7	0.12	9.69	9.81	2.32	1.69
HSB-1	301.3	0.15	10.19	10.34	2.82	2.40

Higher ultimate shear resistance P_{ult} is obtained for HSF specimen in comparison to the HSB specimen. Infill concrete zones of both specimens are shown in Figure 16. Higher damage of concrete and connector deformation is obtained for HSB-1 specimen than for HSF-1 specimen. The initial stiffness is the same for both specimens, HSF-1 and HSB-1.

For both experimentally investigated orientation of shear connectors slip to failure δ_u is higher than $\delta_{uk} = 6$ mm which is the minimum required in EC4:2004 [4] to consider this type of shear connection as ductile.

4 CONCLUSIONS

HILTI X-HVB 110 connectors with X-ENP-21 HVB nails in full depth prefabricated concrete slabs are investigated for the purpose of possible application in prefabricated steel-concrete composite construction. Approximately 13 % higher ultimate shear resistance is obtained for HSF-1 specimen (forward orientation of shear connectors) in comparison to the HSB-1 specimen (backwards orientation of shear connectors). Different failure modes are obtained for different orientations of shear connectors, considering much higher deformation of shear connectors and concrete damage for HSB specimen. For both experimentally investigated specimens minimal required slip to consider the shear connection as ductile is achieved.

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PRORAČUN SPREGNUTIH MEĐUSPRATNIH NOSAČA NA DEJSTVO VIBRACIJA IZAZVANIH AKTIVNOSTIMA LJUDI

Rezime:

S-32

Najnoviji trendovi u građevinarstvu u pogledu projektovanja vitkih konstrukcija sa velikim rasponima, velikim otvorenim unutrašnjim prostorima, nekonvencionalnim oblicima i konstrukcijskim rešenjima, često dovode do izraženih problema u pogledu graničnih stanja upotrebljivosti. Izražene vibracije međuspratnih konstrukcija izazvane ljudskim aktivnostima mogu značajno da utiču na funkcionalnost objekta, komfor ljudi i kvalitet života unutar objekta. U ovom radu dat je kratak prikaz proračuna spregnutih nosača na dejstvo vibracija, kao i preporuke za proračun vibracija izazvanih ljudskim aktivnostima.

Ključne reči: vibracije tavanica, sopstvene frekvencije, kriterijumi prihvatljivosti

DESIGN OF COMPOSITE STEEL-CONCRETE BEAMS DUE TO VIBRATIONS INDUCED BY HUMAN ACTIVITIES

Summary:

The latest demands in construction design towards slender structures with large spans, large open interiors, unconventional shapes and constructional solutions leading to the extensive problems related to the serviceability limit state. Annoying vibrations induced by human activities can significantly affect the structure's functionality, comfort of people and quality of life. This paper presents a short overview of design procedures for composite steel-concrete beams considering vibrations and design recommendations of floor vibrations induced by human activities.

Key words: floor vibrations, natural frequencies, acceptance criteria

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1 UVOD

Trenutni zahtevi pri projektovanju konstrukcija u zgradarstvu najčešće uslovljavaju primenu konstrukcija velikih raspona, sa velikim otvorenim unutrašnjim prostorima, sa spregnutim međuspratnim konstrukcijama od čelika i betona visokih kvaliteta. Takođe, često se zahteva mogućnost naknadne prenamene prostora u druge svrhe, odnosno fleksibilnost prostora koji se može postići većim rasponima. Ovakvi trendovi dovode do toga da konstrukcija ima niže vrednosti sopstvenih frekvencija oscilovanja i manji koeficijent prigušenja, što značajno utiče na dinamički odgovor konstrukcije kada je ona izložena dinamičkom opterećenju, kao što je na primer kretanje ljudi unutar objekta.

Konstrukcija, osim kriterijuma sigurnosti, koji se dokazuje kroz granična stanja nosivosti, mora da zadovolji i kriterijume funkcionalnosti. Kriterijumi funkcionalnosti zavise od namene objekta, a dokazuju se kroz kontrolu graničnih stanja upotrebljivosti koja obuhvataju kontrolu deformacija (ugibi, horizontalna pomeranja i obrtanja preseka), kontrolu vibracija, kao i naponske kontrole u slučaju pojedinih, dinamički opterećenih konstrukcija.

2 IZVORI VIBRACIJA MEĐUSPRATNIH KONSTRUKCIJA

Pojam vibracija podrazumeva pojavu oscilatornog kretanja pojedinih delova konstrukcije određenom amplitudom i frekvencijom oscilovanja. U zgradarstvu se problem vibracija uglavnom odnosi na vertikalne vibracije međuspratnih konstrukcija, ali u pojedinim slučajevima, posebno kod visokih i vitkih konstrukcija, mogu se pojaviti i horizontalne vibracije usled dejstva vetra.

Kretanje ljudi unutar objekta predstavlja najčešći i najvažniji unutrašnji izvor vertikalnih vibracija međuspratnih konstrukcija. Unutrašnji izvor vibracija međuspratnih konstrukcija može biti i dinamičko opterećenje izazvano radom mašina i opreme unutar objekta. Vertikalne vibracije međuspratnih konstrukcija mogu biti izazvane i spoljašnjim dinamičkim opterećenjem kao što je drumski i železnički saobraćaj, kao i dejstvom zemljotresa ili udara vozila.

Vibracije međuspratnih konstrukcija mogu značajno da utiču na kvalitet života, komfor ljudi, a samim tim i na funkcionalnost objekta. Osetljivost ljudi na pojavu vibracija međuspratnih konstrukcija je veoma velika, odnosno nivo prihvatljivosti vibracija je veoma nizak. Reakcija ljudi na pojavu vibracija međuspratnih konstrukcija najčešće zavisi od toga kojom aktivnošću se osoba bavi u datom trenutku. Pored toga što vibracije mogu značajno da umanje komfor ljudi i kvalitet života unutar određenog objekta, drugi bitan faktor u pogledu ocene prihvatljivosti vibracija međuspratnih konstrukcija je funkcionalnost objekta. Funkcionalnost objekta može posebno biti ugrožena pojavom vibracija u slučaju kada se u objektima obavljaju specifične aktivnosti kao što je na primer slučaj sa operacionim salama, ili u slučaju kada prevelike vibracije međuspratnih konstrukcija mogu da ugroze rad određenih mašina ili tačnost opreme sa kojom se radi unutar objekta.

Jednom projektovana i izgrađena konstrukcija teško se može naknadno modifikovati u pogledu poboljšanja kriterijuma vibracija, izuzev ako se ne razmatraju značajne promene u pogledu mase konstrukcije, krutosti konstrukcije i povećanja koeficijenta prigušenja. Stoga je veoma bitno da se nivo prihvatljivosti vibracija međuspratnih konstrukcija objekta definiše na početku, u fazi projektovanja, u zavisnosti od namene objekta i zahteva investitora.

3 PRORAČUNSKI KRITERIJUMI ZA VIBRACIJE SPREGNUTIH KONSTRUKCIJA

Preporuke za projektovanje objekata u cilju zadovoljavanja kriterijuma vibracija mogu se pronaći u različitim standardima, priručnicima za proračun i stručnim publikacijama. Problemom vibracija međuspratnih konstrukcija izazvanih ljudskim aktivnostima bavili su se mnogi autori koji daju različite preporuke, koje se najčešće odnose na ograničavanje sopstvenih frekvencija oscilovanja međuspratnih konstrukcija, u zavisnosti od namene objekta i vrste dinamičkog opterećenja. Tradicionalno, mnogi autori smatraju da dovoljno visoka sopstvena frekvencija oscilovanja konstrukcije omogućava da ona bude izvan opsega frekvencije pobude, odnosno frekvencije dinamičkog opterećenja usled različitih aktivnosti ljudi.

Prema Evrokodu kriterijumi upotrebljivosti u pogledu vibracija treba da se propišu za svaki projekat i dogovore sa investitorom, ili se mogu definisati u okviru Nacionalnog priloga. Prema SRPS EN 1994:2012 [1], dinamičke karakteristike međuspratnih konstrukcija trebaju da zadovolje preporuke date u SRPS EN 1990:2012 [2]. SRPS EN 1990:2012, Prilog A1 [2] definiše da se za zadovoljavajuće ponašanje konstrukcije u pogledu vibracija mora uzeti u obzir komfor ljudi koji borave unutar objekta i funkcionalnost objekta, a ostali aspekti treba da se usaglase u skladu sa namenom objekta. Kako granična stanja upotrebljivosti u pogledu vibracija ne bi bila prekoračena, neophodno je da sopstvena frekvencija oscilovanja konstrukcije ili delova konstrukcije bude iznad određenih granica, koje zavise od namene objekta i izvora vibracija. SRPS EN 1990:2012 [2] definiše da su za ostale informacije referentni standardi: SRPS EN 1991:2012 [3], SRPS EN 1991:2012 [4] i ISO 10137 [5]. SRPS EN 1990:2012, Prilog A2 [2] preporučuje maksimalne vrednosti ubrzanja za bilo koje delove međuspratne konstrukcije od $0,7 \text{ m/s}^2$ za vertikalne vibracije i $0,2 \text{ m/s}^2$ za horizontalne vibracije.

Tabela 1 – Indikativne vrednosti sopstvenih frekvencija konstrukcije zgrade i ograničenja ubrzanja [6]

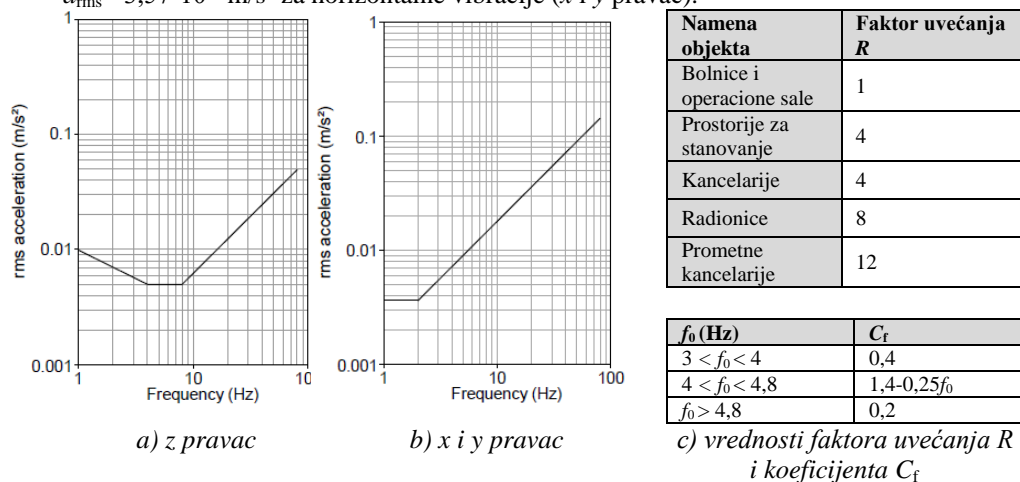
Namena objekta	Obično zadovoljavajuće ponašanje	Često nezadovoljavajuće ponašanje	Preporučena granična vertikalna ubrzanja (u % od g)
Sportske dvorane, javni prostori	$n_e > 10 \text{ Hz}$	$n_e < 6 \text{ Hz}$	10 %
Stambene zgrade	$n_e > 8 \text{ Hz}$	$n_e < 5 \text{ Hz}$	0,1 %
Poslovne zgrade	$n_e > 8 \text{ Hz}$	$n_e < 5 \text{ Hz}$	0,2 %

Nacionalni prilog SRPS EN 1990/NA [6] propisuje indikativne vrednosti sopstvenih frekvencija oscilovanja i ograničenja vertikalnih ubrzanja izražena u procentima ubrzanja zemljine teže g , koje bi u većini regularnih slučajeva trebalo da obezbede prihvatljivo ponašanje međuspratne konstrukcije (tabela 1). Međutim, sopstvena frekvencija nije jedini parametar koji utiče na prihvatljivost vibracija, pa u određenim slučajevima kontrola graničnog stanja upotrebljivosti usled vibracija ne može da se svede samo na ograničenje sopstvene frekvencije [7]. Tada je neophodno da se sprovedu složeniji postupci analize koji mogu da se nađu u specijalizovanoj literaturi, a koji se uglavnom zasnivaju na direktnoj ili indirektnoj

kontroli vertikalnog ubrzanja međuspratne konstrukcije, koje najviše utiče na komfor korisnika [7].

BS 6472 [8] pokriva različite izvore vibracija u međuspratnim konstrukcijama, a nivoi prihvatljivosti dati su u obliku težinskih funkcija za osnovne krive i serije faktora za uvećanje u zavisnosti od namene objekta, koje su date na slici 1. Osnovne krive za vibracije u pravcu z ose (vertikalni pravac) i x i y ose (horizontalni pravac) izvedene su na osnovu sledećih osnovnih vrednosti srednjeg kvadratnog ubrzanja a_{rms} (m/s^2):

- $a_{rms} = 5 \cdot 10^{-3} m/s^2$ za vrtikalne vibracije (z pravac),
- $a_{rms} = 3,57 \cdot 10^{-3} m/s^2$ za horizontalne vibracije (x i y pravac).



Slika 1 – Osnovne krive za vibracije i faktor uvećanja prema BS 6472 [9]

Faktor uvećanja R određuje se u zavisnosti od vrednosti sopstvene frekvencije oscilovanja međuspratne konstrukcije [10]. Ukoliko je sopstvena frekvencija oscilovanja konstrukcije manja od 7 Hz, konstrukcija se smatra nisko frekventnom i faktor uvećanja R se određuje prema izrazu:

$$R = \frac{68000C_f}{m_g S_{eff} L \zeta} \quad (1)$$

Za visoko frekventne konstrukcije (sopstvena frekvencija oscilovanja konstrukcije je veća od 7 Hz), faktor uvećanja R određuje se prema izrazu:

$$R = \frac{3000}{m_g b_e L} \quad (2)$$

gde je:

- m_g masa konstrukcije koja osciluje (kg/m^2),
- L raspon spregnutog grednog nosača (m),
- I_s moment inercije spregnute ploče (m^4),
- ζ koeficijent prigušenja,

$b_e = \min(b, 40h_p)$ gde je b rastojanje grednih nosača (m), a h_p visina betonske ploče (m),

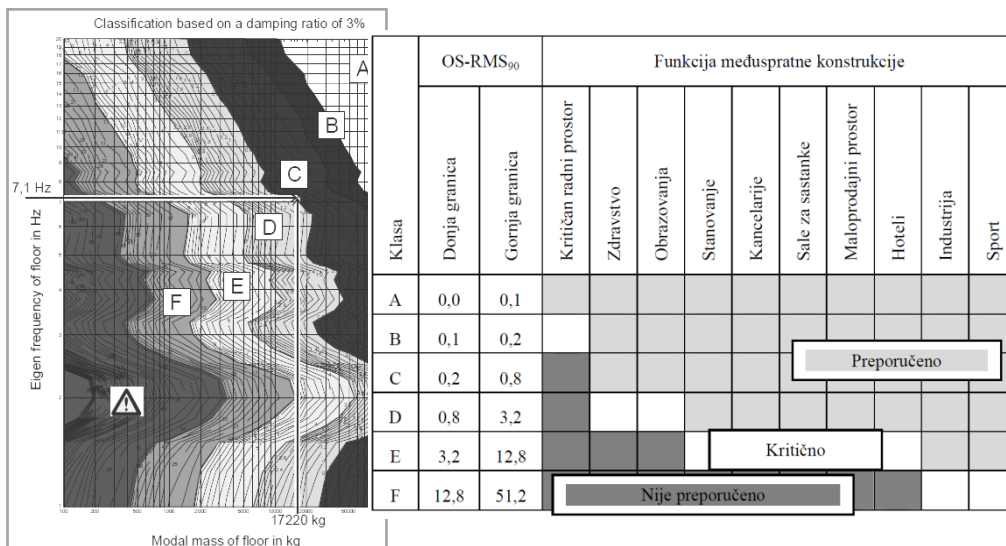
$S_{eff} = 4,5 \left(\frac{E_a I_s}{m_g f_0^2} \right)^{1/4}$ efektivna širina međuspratne konstrukcije (m).

Koeficijent C_f određuje se u zavisnosti od sopstvene frekvencije oscilovanja konstrukcije, kako je prikazano na slici 1.

Na osnovu rezultata istraživanja u okviru ECCS-a (projekat JRC55118 [11]) definisane su preporuke za projektovanje međuspratnih konstrukcija usled dejstva vibracija izazvanih ljudskim hodom. Izvod iz ovih istraživanja publikovao je i *Arcelor-Mittal* u vidu preporuka [12]. Tri osnovna parametra koja utiču na vibracije međuspratnih konstrukcija su sopstvena frekvencija konstrukcije f , prigušenje D i modalna masa M_{mod} . U cilju ocene komfora i definisanja kriterijuma prihvatljivosti u pogledu vibracija, uvedena je veličina OS-RMS₉₀ (*One Step - Root Mean Square 90%*) koja predstavlja brzinu (ili ubrzanje) reprezentativnog pojedinačnog koraka koja odgovara 90-procentnom fraktilu svih različitih ljudskih koraka.

Ukupno prigušenje konstrukcije određuje se kao zbir tri različite komponente prigušenja: prigušenja konstrukcije, prigušenja usled opreme i nameštaja unutar objekta i prigušenja usled završne obrade međuspratnih konstrukcija. Procedura ocene prihvatljivosti međuspratne konstrukcije usled vibracija sastoji se u:

- određivanju osnovnih karakteristika konstrukcije, kao što su sopstvena frekvencija, modalna masa i prigušenje;
- utvrđivanju kategorije kojoj pripada međuspratna konstrukcija, na osnovu dijagrama koji su dati za različite nivoe prigušenja, na osnovu vrednosti sopstvene frekvencije i modalne mase;
- proveriti da li je dobijena kategorija prihvatljiva ili nije za zahtevanu namenu, odnosno funkciju međuspratne konstrukcije.




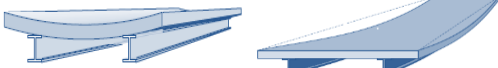
Slika 2 – Klasifikacija međuspratnih konstrukcija (*Arcelor-Mittal* [11])

Definisano je 6 klasa međuspratnih konstrukcija u zavisnosti od opsega vrednosti OS-RMS₉₀, i različiti nivoi prihvatljivosti vibracija međuspratnih konstrukcija u zavisnosti od funkcije (namene) objekta, kako je prikazano na slici 2.

4 ODREĐIVANJE DINAMIČKIH KARAKTERISTIKA SPREGNUTIH MEĐUSPRATNIH KONSTRUKCIJA

Za definisanje nivoa prihvatljivosti međuspratne konstrukcije na dejstvo vibracija potrebno je odrediti osnovne karakteristike konstrukcije kao što su sopstvena frekvencija, modalna masa i prigušenje. Sopstvena frekvencija oscilovanja međuspratne konstrukcije može se odrediti koristeći pojednostavljene metode proračuna date u literaturi, ili pomoću odgovarajućih programa. Pojednostavljene metode proračuna (tabela 2) obuhvataju proračun sopstvene frekvencije oscilovanja i modale mase, koristeći izraze za gredne nosače, ili izraze za ortotropnu ploču, kada se spregnuta međuspratna konstrukcija posmatra kroz frekvencije oscilovanja spregnute betonske ploče i grednog nosača.

Tabela 2 – Izrazi za određivanje sopstvenih frekvencija oscilovanja i modalnih masa spregnutih međuspratnih konstrukcija

Gredni nosači (self weight approach) [10]		$\delta_m = 5m_g L^4 / (384 E_a I_{i0})$ $f_0 = 17,8 / \sqrt{\delta_m}$
Ortotropna međuspratna konstrukcija [11]	 deformacija ploče+deformacija grednog nosača	$\frac{1}{f_{0s}} = 3,56(E_a I_s / m_p b^4)^{1/2}$ $\frac{1}{f_{0b}} = (\pi/2)(E_a I_{i0} / m_g b L^4)^{1/2}$ $\frac{1}{f_0^2} = \frac{1}{f_{0s}^2} + \frac{1}{f_{0b}^2}$
Određivanje modalne mase međuspratne konstrukcije [11]	$M_{mod} = M_{total} \left(\frac{\delta_x^2 + \delta_y^2}{2\delta^2} + \frac{8}{\pi^2} \frac{\delta_x \delta_y}{\delta^2} \right)$ $\delta_y = \delta_g = \frac{5}{384} \frac{m_g b L^4}{E_a I_{i0}}$ $\delta_x = \delta_p = \frac{5}{384} \frac{m_p b^4}{E_a I_s}$	
f_0, f_{0s}, f_{0b} δ_m E_a I_{i0} m_p M_{total} * ostale veličine imaju značenja data u izrazima (1) i (2).	sopstvena frekvencija oscilovanja posmatrane konstrukcije (gredni nosač ili ortotropna ploča), ploče i spregnute grede, respektivno, ugib grednog nosača (mm), modul elastičnosti čelika, moment inercije spregnutog grednog nosača, masa spregnute ploče, ukupna masa međuspratne konstrukcije.	

5 PRIMERI

Primena različitih metoda proračuna prihvatljivosti spregnutih međuspratnih konstrukcija na dejstvo vibracija sprovedena je kroz nekoliko numeričkih primera. Analizirana su četiri spregnuta nosača raspona 8, 10, 12 i 15 metara, na međusobnom rastojanju od četiri metra, čije su karakteristike prikazane u tabeli 3.

Tabela 3 – Karakteristike spregnutih nosača

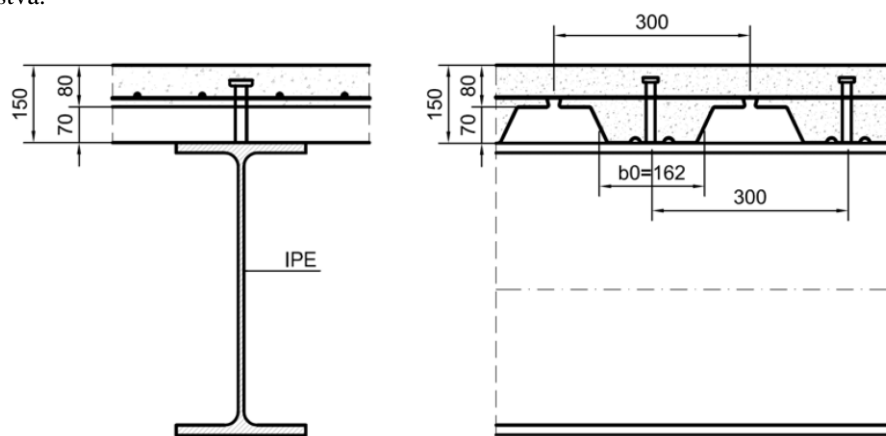
Nosač	Profil	Raspon (m)	Moment inercije spregnute ploče	Moment inercije spregnutog nosača	Efektivna širina međuspratne konstrukcije
			I_s (cm ⁴ /m)	I_{i0} (cm ⁴)	S_{eff} (m)
SN1	IPE 360	8	881,21	65040	14,61
SN2	IPE 400	10	881,21	88800	16,46
SN2-1	IPE 600	10	881,21	269000	13,03
SN3	IPE 500	12	881,21	165200	16,91
SN3-1	HEB 500	12	881,21	245972	15,47
SN4	IPE 550	15	881,21	275500	18,46

Tabela 4 – Sopstvene frekvencije oscilovanja spregnutih nosača

Nosač	Gredni nosač			Ortotropna ploča (čelični nosač+AB ploča)				
	m_g (kN/m)	δ_m (mm)	f_0 (Hz)	m_p (kg/m ²)	m_g (kg/m ²)	f_{0s} (Hz)	f_{0b} (Hz)	f_0 (Hz)
SN1	19,00	7,41	6,54	468,91	483,44	13,98	6,52	5,87
SN2	19,06	13,31	4,88	468,91	485,73	13,98	4,87	4,61
SN2-1	19,62	4,52	8,37	468,91	500,00	13,98	8,35	7,26
SN3	19,31	15,03	4,59	468,91	492,10	13,98	4,58	4,34
SN3-1	20,27	10,60	5,47	468,91	516,57	13,98	5,45	5,06
SN4	19,50	22,22	3,78	468,91	496,94	13,98	3,77	3,63

Čelični nosač (standardni vruće valjani IPE i HEB profili) spregnuti su sa armirano-betonskom pločom na profilisanom limu CF70 (pravac profilacije lima je upravan na pravac nosača), videti sliku 3. Nosači SN2-1 i SN3-1 su varijantna rešenja sa jačim nosačima u odnosu na osnovne nosače SN2 i SN3, respektivno. Kod nosača SN2-1 povećana je visina poprečnog preseka u odnosu na SN2, dok je u slučaju nosača SN3-1 IPE profil je zamenjen HEB profilom iste visine. Za sprezanje se koriste zavareni moždanici sa glavom, prečnika 19 mm za nosače SN1 i SN2, odnosno 22 mm za ostale nosače. Visina moždanika je 120 mm. Ukupna visina betonske ploče na profilisanom limu je 150 mm, a visina betona iznad gornje ivice profilisanog

lima je 80 mm, videti sliku 3. Kvalitet čelika od kojeg su izrađeni vruće valjani profili i profilisani lim je S235, a klasa čvrstoće betona je C30/37. U svim analiziranim numeričkim primerima, pored sopstvene težine konstruktivnih elemenata, u obzir je uzeto i korisno opterećenje od $2,5 \text{ kN/m}^2$, opterećenje u toku gradnje od $0,75 \text{ kN/m}^2$ i opterećenje od završnih radova i instalacija od $1,0 \text{ kN/m}^2$. Proračun graničnih stanja nosivosti i upotrebljivosti izvršen je prema preporukama datim u SRPS EN 1994: 2012 [1], pri čemu je nosač raspona 15 m proračunat kao nosač koji je poduprt u toku gradnje. Za sve nosače predviđeno je nadvišenje čeličnog profila za vrednost koja odgovara deformaciji nosača pre dostizanja spregnutog dejstva.



Slika 3 – Geometrija analiziranih podnih nosača

Prigušenje konstrukcije usvojeno u svim analiziranim primerima je 3%. Prilikom proračuna sopstvene frekvencije oscilovanja, prema preporukama datim u literaturi [9], [10], [11], masu konstrukcije koja osciluje čini stalno opterećenje i 10% korisnog opterećenja. Za analizirane gredne nosače, sopstvene frekvencije oscilovanja prikazane su u tabeli 4.

Tabela 5 – Nivoi prihvatljivosti vibracija

Nosač	BS 6472 [8]		Arcelor-Mittal [12]					Klasa
	C_f	R	M_{mod} (kg)	δ_p (mm)	δ_g (mm)	δ_{total} (mm)	M_{total} (kg)	
SN1	0,20	8,03	15470	8,29	7,41	15,69	7050	D
SN2	0,26	7,34	19430	8,29	13,31	21,60	8844	D
SN2-1	0,20	0,15	20000	8,29	4,52	12,81	9135	C
SN3	0,31	7,13	23620	8,29	15,03	23,32	10790	D
SN3-1	0,20	4,73	24800	8,29	10,60	18,88	11240	C
SN4	0,40	6,59	29820	8,29	22,22	30,50	13790	C

Nivoi prihvatljivosti spregnutih grednih nosača koji su prikazani u tabeli 5, pokazuju da prema BS 6472 [8] svi analizirani gredni nosači, izuzev nosača SN2-1, imaju vrednost faktora R između 4 i 8, odnosno razmatrani gredni nosači u okviru spregnutih međuspratnih konstrukcija se mogu koristiti u slučajevima prostorija predviđenih za radionice i kancelarije. Nosač SN2-1 zadovoljava kriterijume vibracija za upotrebu u objektima svih namena, pa tako i u međuspratnim konstrukcijama bolnica i operacionih sala, za koje su definisani najstrožiji uslovi ($R < 1$) BS 6472 [8].

Prema preporukama koje je dao *Arcelor-Mittal* [12], analizirani gredni nosači pripadaju klasi C i D. Nosači raspona 8, 10 i 12 metara (SN1, SN2 i SN3) pripadaju klasi D i mogu se koristiti kod međuspratnih konstrukcija prostorija za stanovanje, kancelarija, sportskih i industrijskih objekata, što omogućava raznovrsniju primenu ovih nosača u pogledu namene objekta u odnosu na BS 6472 [8]. Nosači SN2-1 i SN3-1 sa značajno većim čeličnim profilima su klase C, što omogućava da budu prihvatljivi u pogledu vibracija u objektima skoro svih namena (izuzev kritičnog radnog prostora). Analizirani gredni nosači ne zadovoljavaju indikativne preporuke koje propisuje Nacionalni prilog SRPS EN 1990/NA [6], za uobičajeno zadovoljavajuće ponašanje.

Kada međuspratne konstrukcije treba da zadovolje strožije kriterijume vibracija, nije opravdano korišćenje čelika kvaliteta višeg od S275. Izborom čelika boljih mehaničkih svojstava dobijaju se manji poprečni preseči čime se smanjuje krutost nosača, što za rezultat ima manje vrednosti sopstvenih frekvencija i pogoršanje karakteristika sa aspekta prihvatljivih vibracija. Step en iskorišćenja nosivosti poprečnog preseka analiziranih podnih nosača, u ovom radu, je od 69% do 77 % (nosači SN1, SN2, SN3 i SN4) od plastičnog momenta nosivosti spregnutog poprečnog preseka. Na osnovu dobijenih rezultata može se izvesti zaključak da u slučajevima kada se zahteva da konstrukcija ima bolje dinamičke karakteristike u pogledu vibracija, granično stanje upotrebljivosti postaje merodavan kriterijum za dimenzionisanje.

Na osnovu analize rezultata prethodnih primera može se zaključiti da značajna promena dinamičkih karakteristika konstrukcije i poboljšanje komfora u pogledu vibracija zahteva rešenje sa nosačima znatno većih visina, što podrazumeva i povećanje spratnih visina.

6 ZAKLJUČCI

Pri projektovanju spregnutih međuspratnih konstrukcija, koje se po svojim dinamičkim karakteristikama mogu svrstati u vitke konstrukcije, posebnu pažnju treba posvetiti proveru graničnih stanja upotrebljivosti, koja često mogu biti merodavna za dimenzionisanje. Konstrukcije koje ne zadovoljavaju kriterijume vibracija za zahtevanu namenu objekta jako je teško i neekonomično naknadno preprojektovati, a često i neizvodljivo. Jasno definisanje nivoa prihvatljivosti vibracija međuspratnih konstrukcija u fazi projektovanja, u zavisnosti od namene objekta i zahteva investitora, veoma je važno u cilju zadovoljenja predviđenih kriterijuma, ispunjenja uslova funkcionalnosti i komfora ljudi koji borave unutar objekta. Ovo je veoma važno jer zahtevani nivo komfora, kako je pokazano na primerima, direktno utiče na dimenzije čeličnih nosača, a samim tim i na vrednost investicionih troškova.

Postojeća domaća regulativa, SRPS EN 1990:2012 [2] i SRPS EN1990/NA [6], daje samo okvirne (indikativne) preporuke za ocenu prihvatljivosti konstrukcija u pogledu vibracija međuspratnih konstrukcija. Evrokod takođe ne daje preciznije preporuke za proračun vibracija međuspratnih konstrukcija. Takođe treba naglasiti da sopstvena frekvencija konstrukcije nije jedini parametar za ocenu prihvatljivosti u pogledu vibracija, i da slepo praćenje

konzervativnih uslova u pogledu frekvencija može dovesti do neracionalnih rešenja konstrukcija. U različitoj stručnoj literaturi mogu se pronaći preporuke i pojednostavljeni postupci za proračun dinamičkih karakteristika konstrukcije, što je veoma značajno za uobičajenu inženjersku praksu, u cilju lakšeg sagledavanja različitih kriterijuma u pogledu vibracija, koji moraju biti zadovoljeni prilikom projektovanja konstrukcija.

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ПОЖАРНА ОТПОРНОСТ ЧЕЛИК-BETON СПРЕГНУТИХ GREDA U ZAVISNOSTI OD STATIČKOG SISTEMA GREDE

Rezime:

S-33

U radu je prikazana numerička analiza požarne otpornosti spregnutih greda čelik-beton u zavisnosti od usvojenog statičkog sistema grede. Greda sa delimično ubetoniranim čeličnim IPE profilom spregnuta je sa armiranobetonskom pločom i analizirana za slučaj statičkog sistema proste grede i kontinualne grede na dva polja. Iako se, zbog jednostavnijeg izvođenja i proračuna, u praksi, za međuspratne tavanice visokogradnje, češće koristi sistem prostih spregnutih greda, ovde je, na jednom primeru, prikazana prednost statički neodređenog sistema spregnutih greda, u uslovima dejstva visokih temperatura (požara).

Ključne reči: čelik-beton spregnuta greda, požarna otpornost, statički sistem

STRUCTURAL SYSTEM INFLUENCE ON FIRE RESISTENCE OF STEEL-CONCRETE COMPOSITE BEAMS

Summary:

Fire resistance numerical analysis of steel-concrete composite beams as function of the defined structural system are presented in this paper. Partially encased steel profile IPE is coupled with the reinforced concrete slab and two different structural systems are analysed: simply supported beam and continuous beam over two spans. In practice, as a result of a simpler calculation and construction, in case of steel-concrete composite floor structures, the simply supported beams are more often used than the continuous one. Nevertheless, the advantage of the continuous steel-concrete composite beams in comparison to the simply supported beams in case of high temperatures and fire exposure is presented in this paper.

Key words: steel-concrete composite beam, structural system, fire resistance analysis

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1 UVOD

U savremenim konstrukcijama objekata visokogradnje, u svetu, uglavnom se koristi sistem spregnutih čelik-beton međuspratnih tavanica. Ove tavanice imaju bezbroj prednosti u odnosu na ostale vidove međuspratnih konstrukcija, od kojih se mogu izdvojiti sledeće:

- veća nosivost a time i postizanje većih rasona od armiranobetonskih klasičnih tavanica,
- manja sopstvena težina i sa time i veća iskorišćenost u smislu prihvatanja opterećenja,
- jednostavnije i brže izvođenje uz minimalnu oplatu (pogotovu kod tavanica sa profilisanim limom),

Nedostaci ovih tavanica su u preciznijoj izvedbi, potrebi za stručnom radnom snagom pri izvođenju a kod nas i cena i ponuda čelika kao materijala koji znatno učestvuje u ovom vidu konstrukcije.

No, i pored pomenutih nedostataka, ove konstrukcije su nezamenljive i nezaobilazne u objektima gde se traže veći rasponi odnosno, gde je izražena potreba za većim slobodnim površinama.

2 SPREGNUTA MEĐUSPRATNA TAVANICA

Spregnuta međuspratna tavanica je tavanica koja se formira sprežanjem armiranobetonske ploče sa čeličnim nosećim elementima (samostalnim ili ubetoniranim profilima).

Dva osnovna vida spregnutih međuspratnih tavanica čelik-beton su :

- tavanice sa klasično izvedenom armiranobetonskom pločom i
- tavanice sa armiranobetonskom pločom izvedenom sa profilisanim limom.

Prednost spregnutih tavanica sa profilisanim limovima je u činjenici da za iste nije potrebna posebna oplata, već profilisani lim služi kao oplata u fazi izvođenja i kao nosivi element u fazi eksploatacije. Danas se uglavnom koristi ovaj drugi tip spregnute tavanice.

2.1 ELEMENTI SPREGNUTE MEĐUSPRATNE TAVANICE

Osnovni elementi spregnutih međuspratnih konstrukcija, čelik-beton, su armiranobetonske ploče i čelične grede. Ranije pomenute armirano betonske ploče mogu biti livene na licu mesta, polumontačne i montažne ploče.

Grede spregnutih tavanica su ili samostalni čelični profili ili ubetonirani čelični profili spregnuti sa armiranobetonskom pločom tavanice. Ubetonirani (potpuno ili delimično) čelični profili imaju znatne prednosti u odnosu na samostalne čelične profile: veća krutost grede, manja opasnost od bočnog i torzionog izvijanja, bolja zaštita profila od dejstva požara i dr. Nedostaci ubetoniranih greda su u većoj sopstvenoj težini i komplikovanim izvođenju. Ali ako se radi o objektima sa većim korisnim opterećenjima i većim rizicima od požarnog dejstva u svakom slučaju izbor će pasti na ubetoniranje čelične grede. Na taj način će se izbeći dodatni konstruktivni zahvati na sprečavanju torzije i bočnog izvijanja čeličnog profila veće dučine kao i skupa protivpožarna zaštita.

2.2 PROJEKTOVANJE I PRORAČUN SPREGNUTIH TAVANICA

Projektovanje i proračun spregnutih tavanica obuhvaćeni su nacionalnim propisima pojedinih zemalja odnosno, savremenim evropskim propisima-Evrokodovima. U Evrokodu EC 4 data su pravila i smernice za proračun i projektovanje svih elemenata spregnutih konstrukcija

u celini. Sam proračun ovih konstrukcija može biti izveden uz pomoć teorije dozvoljenih naprezanja ili pomoću teorije granične nosivosti. Pored proračuna nosivosti pojedinih preseka elemenata spregnutih tavanica (dimenzionisanja), potrebno je obavezno sprovesti proveru uslova upotrebljivosti ovih elemenata i konstrukcije u celini. Ovde se posebno misli na zadovoljenje uslova dozvoljenih deformacija pojedinih elemenata i cele konstrukcije.

Pri projektovanju jedne spregnute tavanice posebnu pažnju treba obratiti na pravilan odabir raspona ploča i spregnutih greda, a da se ne naruše osnovni arhitektonski zahtevi. Ekonomičnost i jednostavnost u projektovanju podrazumeva usvajanje optimalnih raspona ploča i greda.

2.3 SPREGNUTE KONSTRUKCIJE ČELIK-BETON U POŽARNIM USLOVIMA

Sama činjenica da je čelik, kao materijal, osnovni noseći element greda u spregnutim međuspratnim konstrukcijama, i da je isti veoma osetljiv na dejstvo povišenih temperatura, nameće potrebu za istraživanjima ponašanja spregnutih konstrukcija (tavanica) pri dejstvu požara. Ovde posebno, ako se radi o spregnutim tavanicama, treba obratiti pažnju na smanjenje nosivosti greda, jer kod armiranobetonske ploče, beton znatno štiti armaturi ploče od dejstva temperature. Zato je potrebno dobro zaštititi čelične profile greda ili ubetoniravanjem ili posebnim sredstvima za izolaciju.

U svetu je poslednjih nekoliko decenija izvršeno mnogo istraživanja (eksperimentalnih i teoretskih) u oblasti požarne otpornosti spregnutih konstrukcija. Pokazalo se da postoji mnogo faktora koji utiču na nosivost spregnutih konstrukcija pri dejstvu požara kao što su: tipovi preseka spregnutih elemenata, požarni scenario, modeli požarnog dejstva, statički sistemi konstrukcije, načini ispitivanja i dr. Uglavnom se u analizama koristio standardni model požara, ali se u zadnje vreme analizira i dejstvo ostalih modela požara (prirodni, realni, parametarski požari i dr.).

Problem eksperimentalnih istraživanja se sastojao (sastoji) u nemogućnosti opterećenja požarnim dejstvom realnih konstrukcija zbog visokih troškova eksperimenata. Uglavnom su se laboratorijski analizirali izdvojeni elementi (stubovi, grede). U nekoliko skorijih ispitivanja na realnim modelima zgrada pokazalo se da rezultati znatno odstupaju od rezultata dobijenih na pojedinačnim elementima. Ova odstupanja su rezultat prostorne preraspodele opterećenja i povećanja nosivosti pojedinih elemenata spregnute konstrukcije u celini. U tom cilju se nametnula potreba da se pomoću dobijenih rezultata iz eksperimenata, termodinamike, statike konstrukcija i drugih nauka, iznađu numerički moduli koji će moći da simuliraju ponašanje građevinskih konstrukcija u slučaju dejstva požara.

Postignuti su značajni rezultati u ovom poduhvatu tako da danas ima nekoliko numeričkih softvera koji mogu sa vrlo zadovoljavajućim rezultatima da procene ponašanje armiranobetonskih, čeličnih, spregnutih i drugih konstrukcija pri požarnom opterećenju. Ovi moduli se uglavnom baziraju na metodi konačnih elemenata kako pri termičkoj tako i pri statičkoj analizi preseka, elemenata i konstrukcije u celini.

U radu je korišćen numerički softver SAFIR kreiran u Liježu, Belgija. Softver je verifikovan na velikom broju rezultata eksperimenata, pokazao je veoma zadovoljavajuće rezultate, tako da se može koristiti u numeričkoj analizi procene nosivosti elemenata i konstrukcija u celini.

2.4 SAFIR – MODUL ZA PRORAČUN POŽARNE OTPORNOSTI

SAFIR je namenski računski program za analizu konstrukcija i konstruktivnih elemenata u uslovima ambijentne temperature i povišenih temperatura za vreme dejstva požara [5]. Program je baziran na metodi konačnih elemenata (MKE). Razvijen je na Univerzitetu u Liježu, Belgija. Odnos napon-deformacija je uglavnom linearno-elasto-plastični za čelik i nelinearni za beton. Prvi korak analize je proračun raspodele temperature unutar preseka konstruktivnih elemenata – “termička analiza”. Drugi korak je “statička analiza” čija svrha je određivanje odgovora konstrukcije usled statičkog i toplotnog opterećenja.

Numerička analiza ponašanja spregnute grede (sprezanje čelik-beton), međuspratne tavanice, sa varijacijom statičkog sistema grede, izvršena je pomoću opisanog numeričkog modula.

3 NOSIVOST SPREGNUTIH GREDA U POŽARU U ZAVISNOSTI OD STATIČKOG SISTEMA GREDE

Poznato je da različiti statički sistem konstrukcije ili elementa konstrukcije može bitno da utiče na izbor preseka, dimenzije preseka, noseće raspone, ekonomičnost i dr. pri projektovanju iste. Tako u normalnim uslovima eksploatacije, za usvojena projektna opterećenja u statički neodređenim elementima ili konstrukcijama dolazi do preraspodele statičkih uticaja čime se povećava nosivost usvojenih preseka i elemenata. Iz tog razloga kod sistema greda na dva, tri ili više polja (statički neodređene grede), preraspodela statičkih uticaja utiče na manje dimenzije preseka i veće raspone od sistema proste grede. S druge strane spregnute grede (sprezanje čelik-beton), u međuspratnim konstrukcijama, zbog jednostavnosti pri izvođenju najčešće se izvode u sistemu prostih greda. Na ovaj način se dobijaju malo veće dimenzije preseka greda ali se izbegava konstruisanje i armiranje oslonaca sistema kontinualnih greda.

U uslovima dejstva visokih temperatura potrebno je konstrukciju i elemente konstrukcije projektovati tako da isti ne samo održe svoju nosivost i celovitost, već da zadrže određeni nivo upotrebljivosti. To znači da deformacije (ugibi) greda moraju biti što duže vremena u granicama dozvoljenih vrednosti. Ovo je lakše postići kod statičkih sistema greda gde postoji mogućnost preraspodele statičkih uticaja u njima.

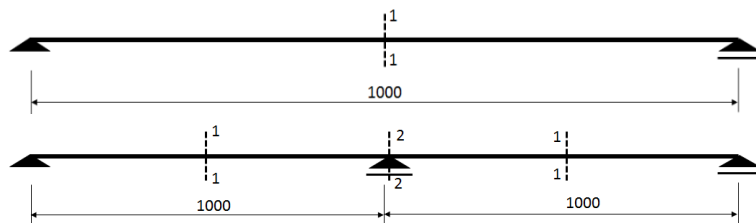
Postoji u zadnjim decenijama mnogo eksperimentalnih i numeričkih ispitivanja ponašanja raznih statičkih sistema u uslovima požara. Rezultati se mogu pronaći u obimnoj literaturi iz ove oblasti. Ipak u ovom radu je dat jedan jednostavan prikaz ponašanja spregnute grede međuspratne konstrukcije u slučajevima da je ista izložena dejstvu standardnog požara, a za slučaj da je ista projektovana kao prosta greda, odnosno kao greda na dva polja.

Svi usvojeni materijali, njihove nosive karakteristike, termičke karakteristike, koeficijenti opterećenja i drugo, usvojeni su prema evropropisima EC2, EC3 i EC4.

3.1 KARAKTERISTIKE USVOJENIH I ANALIZIRANIH SPREGNUTIH GREDA

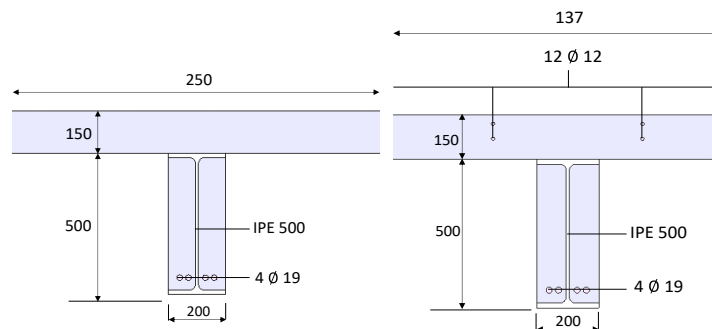
Na slici br.1, prikazani su statički sistemi analiziranih spregnutih greda : prosta greda raspona $L=10$ m, i grede na dva polja raspona $L_1=L_2=10$ m.

Poprečni preseki u poljima spregnute proste i grede na dva polja, i poprečni presek u osloncu grede na dva polja, prikazani su na slici 2. Presek grede čini čelični IPE 500 profil koji je delimično ubetoniran. Ovaj presek je usvojen jer je vrlo praktičan za izvođenje, a veoma dobro se ponaša na uticaje torzije i bočnog izvijanje.



Slika 1 – Statički sistemi analiziranih greda : prosta greda (gore); greda na dva polja (dole);

Iznad oslonca za prihvat negativnog momenta savijanja usvojena je armatura 12 ϕ 12. Kod obe grede usvojen je čelik kvaliteta S 235, beton kvaliteta C 30/37 sa vlažnošću 4% (40 l vode na m³ betona) i armatura B 400/500.



Slika 2 – Poprečni presecci spregnutih greda: presek u polju greda 1-1 (levo); presek u osloncu greda na dva polja 2-2 (desno)

3.2 POŽARNI SCENARIO, OPTEREĆENJE I KRITERIJUMI POŽARNE ANALIZE

Obe grede su izložene standardnom požaru, samo sa donje strane, po celom obimu preseka kako je to prikazano na slici 3.

U požarnim uslovima opterećenja su znatno manja od maksimalno projektovanih opterećenja pod normalnim temperaturama. Opterećenja su manja iz dva razloga. Prvi razlog je činjenica da se faktori opterećenja, za uslove normalnih temperature, razlikuju od faktora opterećenja pri požarnim uslovima. Za požarne uslove ovaj faktor za stalno i promenljivo opterećenje je 1,00. Na običnoj temperaturi imamo :

$$q_{sd} = (\gamma)_{stal} \cdot q_{s,stal} + (\gamma)_{prom} \cdot q_{s,prom} = \gamma^* \cdot q_{s,20^{\circ}C} = \gamma^* \cdot (q_{s,stal} + q_{s,prom}) \text{ gde je :}$$

- q_{sd} – projektovano opterećenje;
- $q_{s,20^{\circ}C}$ - ukupno realno opterećenje u normalnim uslovima;
- $(\gamma)_{stal}$ - faktor kombinacija za stalno opterećenje, $(\gamma)_{stal} = 1,35$;
- $(\gamma)_{prom}$ - faktor kombinacija za promenljivo opterećenje, $(\gamma)_{prom} = 1,50$.

Opterećenje u požarnim uslovima je : $q_{fi} = q_{s,stal} + q_{s,prom}$ (faktor $\gamma^* = 1,00$)

Tada sledi da je : $q_{fi} / q_{sd} = 1 / \gamma^*$. Ako usvojimo da je: $\gamma^* = (1,35 + 1,50) / 2 \approx 1.43$,

dobija se : $q_{fi} / q_{sd} = 1 / \gamma^* = 1 / 1,43 = 0,70$

Sa druge strane, kako je $q_{fi} = q_{pl,Rk}$, gde je $q_{pl,Rk}$ - opterećenje koje izaziva karakterističnu plastičnu otpornost preseka, a u normalnim uslovima se zahteva da je $q_{sd} \leq q_{pl,Rd}$, dobija se $q_{pl,Rd} = q_{pl,Rk} / \gamma^{**}$, gde je γ^{**} faktor za pojedine materijale i to : za čelik 1,10, za beton 1,50 i za armaturu 1,15. Za usvojenu prosečnu vrednost faktora materijala $\gamma^{**} = (1,10+1,15+1,50)/3=1,25$ i $q_{sd} = q_{pl,Rd}$, konačno dobijamo da je:

$$q_{fi} / q_{pl,Rd} = 0,70 / 1,25 \approx 0,60$$

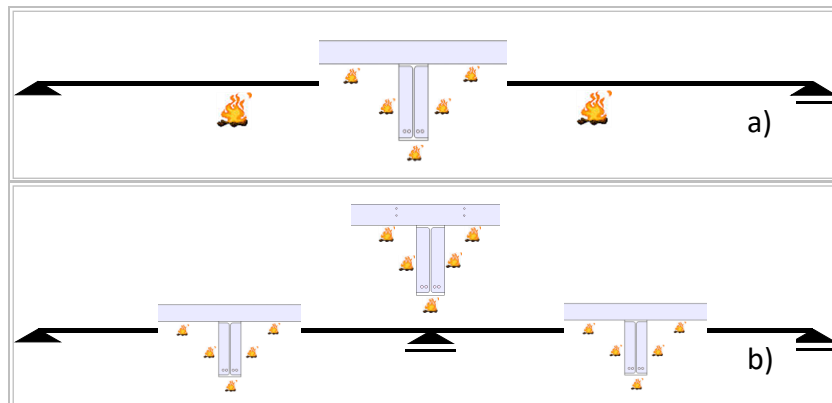
Pri proračunu požarne otpornosti, predmetnih spregnutih greda, usvojena je granica odnosa opisanih opterećenja (0,40-0,50-0,60),

Kao kriterijum krajnjeg vremena otpornosti na požar (u sekundama), usvojen je strožiji kriterijum ugiba od dve vrednosti :

- $u_{max} = L^2 / (400 * d) = 38,46 \text{ cm} = 0,385 \text{ m}$ i
- $u_{max} = L / 30 = 1000 / 30 = 33,33 \text{ cm} = 0,33 \text{ m}$;

a to je L/30. Realno eksploataciono opterećenje $q_{real}=48 \text{ KN/m}$ (na gredi), nalazi se približno na 40% od plastične nosivosti grede ($q_{pl,Rd}$).

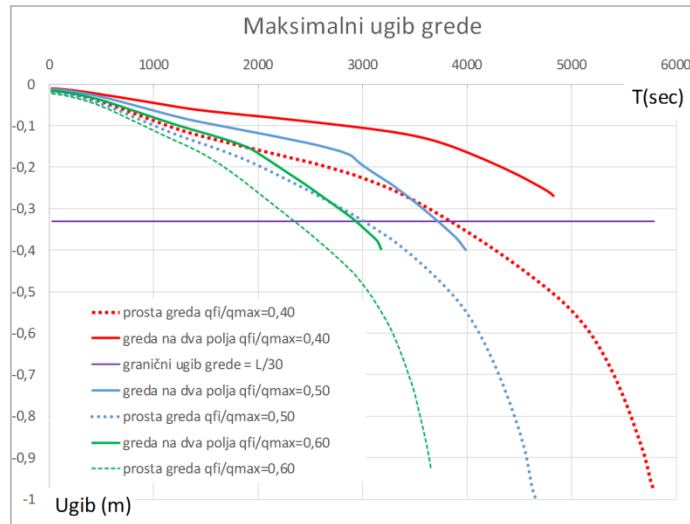
U oba statička sistema grednih nosača, omogućeno je nesmetano termičko širenje preko pokretnih oslonaca. Ovo je usvojeno iz razloga da se ne bi javila dodatna naprezanja, usled sprečavanja aksijalne ekspanzije elementa, pri dejstvu visokih temperatura.



Slika 3 – Položaj poprečnih preseka spregnutih greda sa prikazom opožarenosti preseka:
a) prosta greda ; b) greda na dva polja;

3.3 REZULTATI NUMERIČKE POŽARNE ANALIZE SPREGNUTIH GREDA

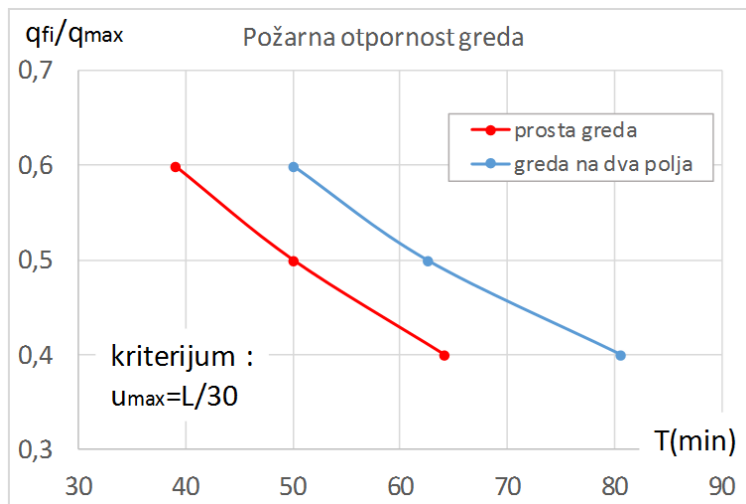
Numeričkom analizom ponašanja dva statička sistema spregnutih greda sa slika 1 i 2, za tri odnosa požarnog opterećenja prema graničnom opterećenju (0,40-0,50-0,60), dobijen je dijagram deformacija tih greda do trenutka gubitka nosivosti grede, slika 4. Na dijagramu je u okviru iste boje, koja predstavlja jedan odnos opterećenja, punom i isprekidanom linijom, predstavljen maksimalni ugib u toku zagrevanja za gredu na dva polja (puna linija) i prostu gredu (isprekidana linija).



Slika 4 – Deformacije opožarene proste i grede na dva polja (spregnute) za različite odnose opterećenja pri požaru prema graničnom opterećenju ;

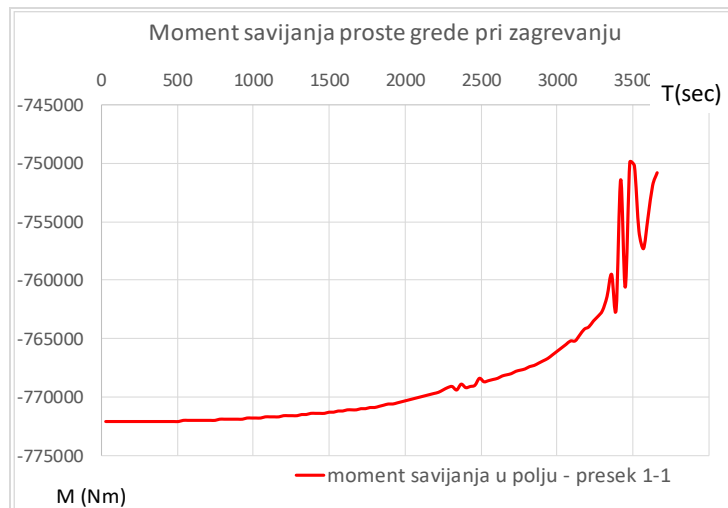
Sa dijagrama se može videti da gubitak nosivosti, kod grede sa dva polja, nastaje za manje vrednosti ugiba, ali da se kod sistema proste grede znatno ranije dostiže granična vrednost ugiba.

Na slici 5, prikazana je razlika u vremenu dostizanja granične vrednosti ugiba, za dva analizirana statička sistema spregnutih greda.



Slika 5 – Požarna otpornost proste i grede na dva polja (spregnute) za različite odnose opterećenja, i za kriterijum graničnog ugiba

Različito ponašanje dva statička sistema spregnute grede, pod ravnomerno raspodeljenim opterećenjem, pri zagrevanju, može se najbolje sagledati iz ponašanja momenata savijanja grede. Slika 6, prikazuje moment savijanja proste grede izložene dejstvu standardnog požara. Opterećenje grede iznosi 60% od graničnog opterećenja u ambijentalnim uslovima.

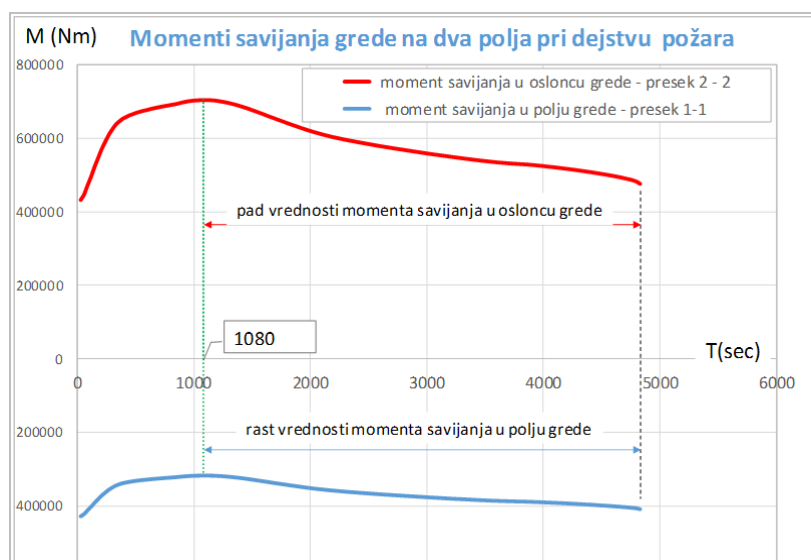


Slika 6 – Dijagram momenta savijanja sredine grede, u toku zagrevanja, za odnos požarnog i graničnog opterećenja - 0,60

Na dijagramu se može primetiti pad vrednosti momenta savijanja pri zagrevanju spregnute proste grede. Razlog pada vrednosti momenta savijanja je neravnomerno zagrevanje gornjeg i donjeg pojasa grede. Hladnija gornja vlakna grede se suprotstavljaju ekspanziji donjih zagrejanih vlakana, pa se javlja zatezanje na gornjoj strani grede, odnosno, pritisak na donjoj strani. Ovaj takozvani “termički”, moment savijanja, umanjuje moment savijanja koji izaziva ravnomerno raspodeljeno opterećenje grede.

Kod spregnute grede na dva polja javljaju se slični efekti, ali je situacija drugačija zbog statičke neodređenosti grede. Na slici 7, prikazan je dijagram momenta savijanja u osloncu i polju grede na dva polja, u toku zagrevanja iste. Može se primetiti da pojava “termičkog” momenta, u početku zagrevanja, izaziva slične efekte kao i kod proste grede, ali se ovde menjaju vrednosti momenta savijanja i u osloncu i u polju grede. Momenat u osloncu raste, a momenat u polju grede opada do određenog vremena zagrevanja. U jednom trenutku, posle $t=18$ min (1080 sec) zagrevanja, presek u osloncu ne može dalje prihvatiti rast dodatnog “termičkog” momenta savijanja, i dolazi do popuštanja nosivosti preseka. Nastaje plastifikacija preseka i preraspodela momenta savijanja u osloncu na moment savijanja u polju grede. Dok moment savijanja u osloncu grede opada, moment savijanja u polju grede počinje da raste sve do trenutka iscrpljenosti nosivosti preseka.

Preraspodela statičkih uticaja (u ovom slučaju momenta savijanja) kod grede na dva polja, pri dejstvu standardnog požara, omogućava duže vreme otpornosti grede pri zagrevanju. Dokaz za ovu tvrdnju je prikazano vreme otpornosti dva statička sistema spregnute grede na slici br.5.



Slika 7 - Dijagram momenta savijanja u osloncu i polju grede, u toku zagrevanja, za odnos požarnog i graničnog opterećenja - 0,60

4 ZAKLJUČAK

Na osnovu svega predhodno izloženog u radu može se izvesti zaključak da, pri projektovanju spregnutih greda u međuspratnim konstrukcijama objekata visokogradnje, treba težiti ka projektovanju statički neodređenih sistema greda (jednostrano ili dvostrano uklještene grede, kontinualne grede na dva i više polja i sl.).

Kod ovih statički neodređenih sistema greda preraspodela statičkih uticaja znatno utiče na povećanje požarne otpornosti greda. Kod statičkog sistema proste grede, dodatni „termički“ moment od zagrevanja nosača, svojim suprotnim dejstvom, neznatno utiče na brzi gubitak nosivosti (lom) i rast deformacija nosača, u toku dejstva požarnog opterećenja.

Kod grede na dva polja javlja se prvo plastifikacija nosača u osloncu i preraspodela statičkih uticaja iz oslonca grede na polje grede, čime se usporava razvoj deformacija i lom samoga nosača.

Poznato je da je proračun i izvođenje statički neodređenih greda, u sprezanju čelika i betona, znatno komplikovanije od izvođenja prostih greda. Međutim, ovakve analize kao iverliki broj eksperimenata, pokazali su da kod projektovanja spragnutih tavanica (sa gredama), kod objekata sa velikim rizikom od požara, ipak treba razmisliti i primeniti statički neodređene sisteme greda.

Treba računati da će se, po istom principu koji je prikazan za grede, i ostali elementi jedne tavanice, a misli se na kontinualne armirano betonske ploče, ponašati slično, čime će se znatno produžiti otpornost spregnute međuspratne tavanice u slučaju pojave požara.

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KOMPARATIVNA ANALIZA PRORAČUNA REŠETKASTOG TORNJA PREMA SRPS I EN 1993

Rezime:

S-34

Pravila za proračun čeličnih rešetkastih tornjeva za nošenje antena data su u okviru evropskog seta propisa u standardu EN 1993-3-1. Svrha ovog rada je da se, na primeru realne noseće konstrukcije antenskog stuba, uporedi pomenuti standard sa SRPS standardom i ukaže na specifičnosti njegove primene u različitim fazama projektovanja. I pored razlika u konceptima proračuna nosivosti u ova dva standarda i drugačijih postupaka za utvrđivanje inteziteta dominantnih dejstava, pokazano je da doskorašnji srpski propis daje neznatno konzervativnije rezultate sa razlikama koeficijenta iskorišćenosti glavnih elemenata konstrukcije koje su u opsegu do 10%.

Ključne reči: rešetkasti čelični stub, vetar, led, pojasevi, ispuna, nosivost

COMPARATIVE ANALYSIS OF LATTICE TOWER DESIGN ACCORDING TO SRPS AND EN 1993

Summary:

Rules for the design of steel lattice antenna towers are given within the sets of Eurocode standards in EN 1993-3-1. The purpose of this paper is, base to the case of the real antenna tower support structures, to comper the aforementioned standard with SRPS standards and to emphasize the specificity of its application in various stages of design. Despite to the differences in the concepts for design resistnace and differences in procedures for calculation of the intensity of action, in these two standards, it is shown that the recently Serbian standard provides a slightly more conservative results, with differences in utilization factor for main elements of the structure in the range of up to 10%.

Key words: lattice steel tower, wind, ice, legs, bracings, resistance

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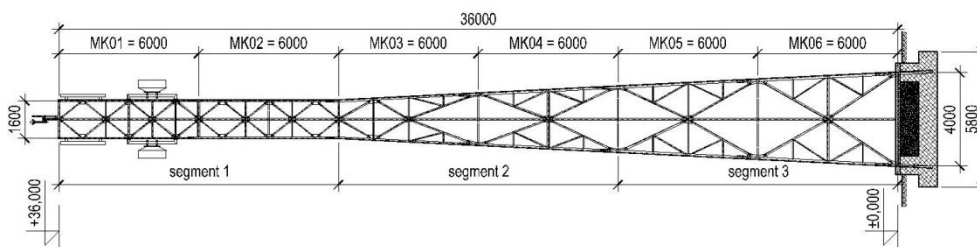
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1 UVOD

Ovaj rad prikazuje komparativnu analizu proračuna rešetkastog antenskog stuba prema povučenom srpskom standardu SRPS U.H2.110:1992 [1] i zamenjujućem standardu SRPS EN 1993-3-1:2012 [2], [3]. Akcenat analize je na utvrđivanju inteziteta dominantnih dejstava vetra i leda i osobenostima proračuna pojasnih elemenata i elemenata ispune. Predmet analize je čelični rešetkasti toranj za mobilnu telefoniju visine 36 m [4]. U konstruktivnom smislu stub je četvoropojasna rešetkasta konstrukcija sa kvadratnom osnovom dimenzija 4x4 m na koti oslanjanja. Pojasevi su u konstantnom nagibu od kote +0,00 m do kote +24,00 m, sa širinom osnove od 1,6 m na koti +24,00. Na delu između kote +24,00 m i vrha stuba poprečni presek stuba je konstantan. Konstrukcija je fundirana na betonskom temelju kvadratne osnove dimenzija 5,8 m. Osnovna brzina vetra je 23 m/s, a teren pripada II kategoriji. Na antenskom stubu je predviđen sledeći sistem antena: 3 panel antene na koti +35,00 m i 3 mini linka na koti +32,00 m. Za potrebe održavanja i montaže antena predviđene su radne platforme na kotama +30,00 m i +33,00 m, kao i odmorište na visini +15,00 m.

Za elemente konstrukcije antenskog stuba usvojeni su oštroični vrućevaljani profili. Usvojeni poprečni preseći pojasnih štapova se menjaju, prateći tok unutrašnjih presečnih sila, po montažnim komadima stuba od L60x60x6 na vrhu stuba preko L80x80x8, L90x90x9, L100x100x10, L110x110x12 do L120x120x13 u podnožju stuba. Štapovi ispune su poprečnog preseka L70x70x7, L60x60x6 i L50x50x5. Slika 1 prikazuje dispoziciono rešenje konstrukcije stuba.



Slika 1 - Dispoziciono rešenje antenskog stuba

Antenski stub je dimenzionisan na dejstvo sledećih opterećenja: stalna opterećenja (sopstvena težina konstrukcije, težina antena i opreme), dejstvo vetra, dejstvo leda i korisna opterećenja na platformama. Dejstvo leda na konstrukciju ima dvostruki uticaj: povećanje gravitacionog opterećenja usled težine leda i kao promena dejstva vetra usled ledenih naslaga na konstruktivnom elementu.

2 OPTEREĆENJE VETROM

U okviru analize dejstva vetra u ovom radu su istaknuti delovi proračuna koji su karakteristični za čelične antenske stubove i ogledaju se prvenstveno u utvrđivanju veličine koeficijenta sile c_f . Početni deo proračunskog algoritma koji je jedinstven za sve konstruktivne sisteme i pokriven u bazičnom standardu za proračun dejstva vatra SRPS EN 1991-1-4 [5] nije prikazan. Promena inteziteta dejstva vetra po visini konstrukcije je analizirana podelom

konstrukcije stuba na tri segmenta: "vršni - segment 1" od 24 - 36 m, "centralni - segment 2" od 12-24 m i "oslonički - segment 3" od 0-12 m.

2.1 KOEFICIJENTI SILE

Prema poglavlju B.2, SRPS EN 1993-3-1[2], koeficijent sile c_f u pravcu dejstva vetra je zbir koeficijenta sile za konstruktivne delove $c_{f,S}$ i koeficijenta sile za nekonstruktivne (pomoćne) delove $c_{f,A}$ analiziranog segmenta konstrukcije stuba:

$$c_f = c_{f,S} + c_{f,A} \quad (1)$$

Ukoliko su, za analiziranu ravan stuba, projekcije površina nekonstruktivnih elemenata (*ancillary components*) na vertikalnu ravan manje od 10% površine projekcije konstruktivnih elemenata, nekonstruktivni elementi se mogu tretirati kao konstruktivni pri proračunu ukupnog koeficijenta sile.

Sledeća novina koju evropski standard donosi je zavisnost vrednosti koeficijenta sile za konstruktivne elemente $c_{f,S}$ od ugla delovanja vetra:

$$c_{f,S} = K_\theta \cdot c_{f,S,0} \cdot \frac{A_s}{\Sigma A} \quad (2)$$

gde su:

ΣA referentna površina konstrukcije (A_{ref}) – projekcija površine konstruktivnih elemenata na vertikalnu ravan upravnu na pravac delovanja vetra uključujući i čvorne limove,

$c_{f,S,0}$ opšti koeficijent sile bez uticaja kraja,

K_θ faktor koji uzima u obzir ugao θ pod kojim vetar deluje na konstrukciju.

Za razliku od povučenog propisa SRPS U.H2.110 [1], Evrokod [2] dopušta da se odredi jedinstveni koeficijent sile za segment stuba koji se istovremeno sastoji od oštroičnih i kružnih profila. Kružni profili u okviru konstrukcije stuba mogu biti u potkritičnom i natkritičnom režimu strujanja. Izraz za opšti koeficijent sile bez uticaja kraja $c_{f,S,0}$ za posmatrani segment j , koji se sastoji od oštroičnih i kružnih profila u potkritičnom režimu strujanja i kružnih profila u natkritičnom režimu strujanja dat je u sledećem obliku:

$$c_{f,S,0,j} = c_{f,0,f} \cdot \frac{A_f}{A_s} + c_{f,0,c} \cdot \frac{A_c}{A_s} + c_{f,0,c,sup} \cdot \frac{A_{c,sup}}{A_s} \quad (3)$$

gde su:

$c_{f,0,f}$; $c_{f,0,c}$; $c_{f,0,c,sup}$ koeficijenti sile za sekcije sastavljene od oštroičnih, potkritičnih kružnih i natkritičnih kružnih profila, respektivno, čije vrednosti zavise od globalnog oblika poprečnog preseka antenskog stuba i koeficijenta ispunjenosti φ ,

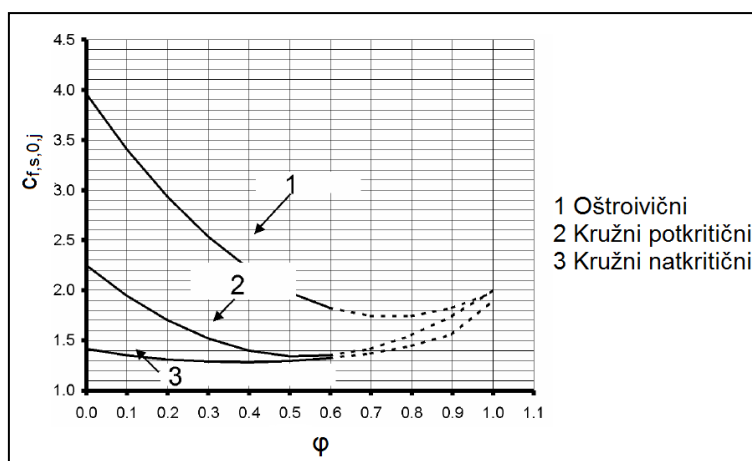
A_f ; A_c ; $A_{c,sup}$ površine projekcija štapova na vertikalnu ravan upravnu na pravac delovanja vetra za oštroične profile, kružne profile u potkritičnom režimu strujanja i kružne profile u natkritičnom režimu strujanja vetra, respektivno,

φ koeficijent ispunjenosti $\varphi = \frac{A_{ref}}{A}$,

A_s zbir projektovanih površina svih štapova ($A_s = A_f + A_c + A_{c,sup}$),

A ukupna površina unutar spoljašnje konture sekcije.

Slika 2 prikazuje dijagram putem kojeg se mogu odrediti aproksimativne vrednosti ovog koeficijenta.



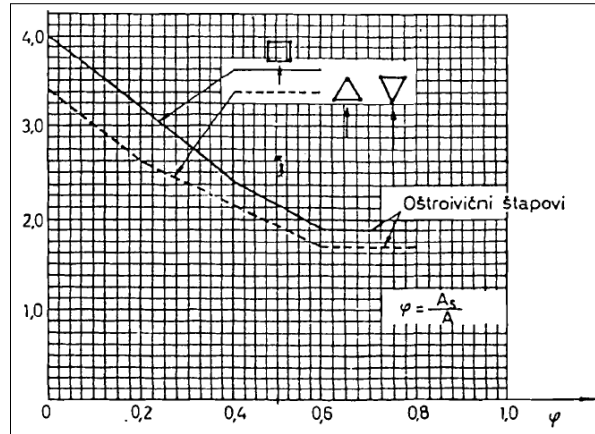
Slika 2 – Opšti koeficijent sile za kvadratni oblik preseka stuba prema SRPS EN 1993-3-1[2]

Koeficijent sile za nekonstruktivne linearne elemente, kao što su penjalice, vodovi kablova i slično zavisi od odnosa površina nekonstruktivnih A_A i konstruktivnih elemenata ΣA , opšteg koeficijenta sile za nekonstruktivne elemente $c_{f,A,0}$, zaklonjenosti konstrukcijom u funkciji koeficijenta K_A i položaja u prostoru u funkciji ugla ψ :

$$c_{f,A} = c_{f,A,0} \cdot K_A \cdot \sin^2 \psi \cdot \frac{A_A}{\Sigma A} \quad (4)$$

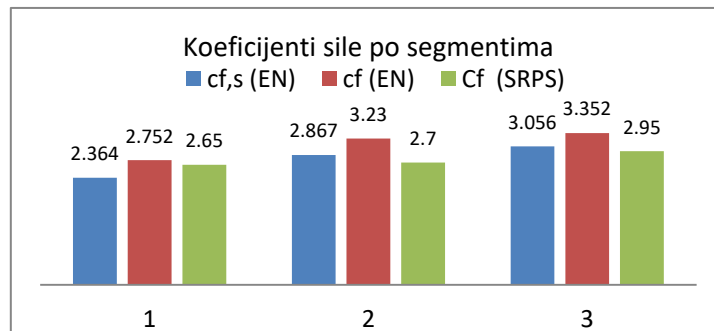
Za diskretno raspoređene nekonstruktivne elemente kao što su antene i elementi za osvetljenje, vrednost koeficijenta sile, koja je definisana na osnovu rezultata ispitivanja dejstva vetra u tunelu, data je od strane proizvođača. Proizvođači antena raspolažu sopstvenim softverskim programima ili tablicama putem kojih se direktno može izvršiti proračun sila vetra na antene.

Prema SRPS U.C7.113 [7] koeficijent sile za čelične tornjeve izgrađene od oštroivičnih profila zavisi samo od koeficijenta ispunjenosti φ - slika 3. Pri proračunu izložene površine konstrukcije A_s , obično se uzimaju u obzir linearni nekonstruktivni elementi, npr. penjalice, ili se opterećenje vetrom na takve elemente računa posebno. Bitna razlika u odnosu na Evrokod [2] je da pomenuti standard [7] ne obuhvata tornjeve čiji su segmenti sastavljeni istovremeno od oštroivičnih i kružnih profila. Takođe, standard [7] nalaže da u okviru jednog segmenta tornja svi kružni profili moraju biti u istom režimu strujanja. Nadalje, koeficijent sile prema SRPS U.C7.113 ne zavisi od pravca delovanja vetra. Slika 3 prikazuje dijagram putem kojeg se u standardu [7] određuje vrednost koeficijenta sile C_f u funkciji koeficijenta ispunjenosti i oblika poprečnog preseka tornja u celini.



Slika 3 - Koeficijent sile prema SRPS U.C7.113[7]

Vrednosti koeficijenata sile za konstrukciju antenskog stuba analiziranu u ovom radu prema SRPS EN 1993-3-1 [2] i prema SRPS U.C7.113 [7] su prikazane na slici 4 za svaki segment konstrukcije posebno.



Slika 4 - Koeficijenti sile po segmentima konstrukcije tornja [8]

Koeficijenti sile za konstruktivne elemente $c_{f,s}$ prema Evrokodu [2] se razlikuju od 4% do 10% od koeficijenata sile C_f prema starim srpskim propisima [7], dok je razlika između vrednosti ukupnih koeficijenata sile c_f [2] i C_f [7] od 4% do 20%.

2.2 OPTEREĆENJE OD VETRA

Krajnji izrazi za utvrđivanje inteziteta dejstva vetra prema SRPS EN 1991-1-4 [5] i SRPS U.C7.113 [7] su slični, s tim što se prema [7] koeficijent sile C_f množi površinom A_s u koju je uračunata površina nekonstruktivnih elemenata, dok se prema [5] ukupni koeficijent sile c_f množi površinom konstruktivnih delova objekta A_{ref} . Takođe prema Evrokodu [5] ugao dejstva vetra na konstrukciju se uzima u obzir pri proračunu koeficijenta sile za konstruktivne elemente, dok se prema starom SRPS standardu [7] uzima u obzir u krajnjem izrazu za određivanje sile vetra preko koeficijenta kosog delovanja ζ .

U standardu SRPS U.C7.113 [7] opterećenje vetrom na rešetkasti toranj kvadratne osnove, sastavljen od oštroičinih profila, se računa pomoću sledećeg izraza:

$$W = \xi \cdot q_{m,t,z} \cdot G_z \cdot C_f \cdot A_s \quad (5)$$

gde su:

$q_{m,t,z}$ osrednjeni aerodinamički pritisak vetra,
 G_z dinamički koeficijent konstrukcije.

Prikazani izraz je sličan izrazu za opterećenje vetrom koji je dat u SRPS EN 1991-1-4 [5], gde se umesto dimamičkog koeficijenta G_z i osrednjenog aerodinamičkog pritiska vetra $q_{m,t,z}$ koriste koeficijent konstrukcije $c_s c_d$ i udarni pritisak vetra $q_p(z_e)$ na referentnoj visini z_e :

$$F_w = c_s c_d \cdot c_f \cdot q_p(z_e) \cdot A_{ref} \quad (6)$$

Međutim, prema evropskom propisu za proračun čeličnih tornjeva [1] nije dovoljno samo izračunati silu od vetra prema izrazu (5), već se u obzir mora uzeti povećanje sile vetra usled globalnog dinamičkog odgovora tornja. Odgovor tornja se može analizirati na dva načina, jednostavnom ekvivalentnom statičkom metodom, ili kompleksnijom dinamičkom analizom.

Da bi se mogla koristiti ekvivalentna statička metoda, potrebno je da konstrukcija zadovolji sledeći uslov:

$$\frac{7 \cdot m_T}{\rho_s \cdot c_{f,T} A_T \cdot \sqrt{d_b \cdot \tau_0}} \cdot \left(\frac{5}{6} - \frac{h_T}{h} \right)^2 < 1 \quad (7)$$

U slučaju čeličnog tornja [8] koji je predmet ovog rada, uslov (7) je bio zadovoljen, pa je primenjena ekvivalentna statička metoda. Takođe, urađena je i parametarska analiza u cilju utvrđivanja validnosti primene ekvivalentne statičke metode, kroz promenu visine stuba, koncentrisanja mase na vrhu i promenu brzine vetra. Analiza je izvršena za tri tornja: toranj visine 45 m sa velikom koncentrisanom masom u vrhu u vidu platforme i "igle" za brzinu vetra od 23 m/s; toranj visine 45 m za brzinu vetra od 35 m/s i toranj visine 60 m za brzinu vetra od 23 m/s. Za granične vrednosti koeficijenta sile za konstruktivne elemente u gornjim segmentima antenskih stubova usvojene su vrednosti 2,5 i 3,0 i dobijene vrednosti ostalih parametra koji figurišu u uslovnoj jednačini (7). Tabela 1 prikazuje rezultate parametarske analize.

Tabela 1 – Analiza ispunjenosti uslova za primenu ekvivalentne statičke metode

h/v	$c_{f,s}$	φ	$A(m^2)$	$A_{ref}(m^2)$	$A_A(m^2)$	$c_{f,A}$	c_f	$m_t(kg)$	$h_t(m)$	Услов	
45/23	2.5	0.3	28.8	8.64	3.3	0.61	3.11	8540.9	15	0.79	<1
	3.0	0.188	28.8	5.41	3.3	0.98	3.98	8540.9	15	0.99	<1
45/35	2.5	0.3	28.8	8.64	3.3	0.61	3.11	7980.5	15	0.74	<1
	3.0	0.188	28.8	5.41	3.3	0.98	3.98	7980.5	15	0.92	<1
60/23	2.5	0.3	34.2	10.26	3.96	0.62	3.12	9395.5	18	0.83	<1
	3.0	0.188	34.2	6.43	3.96	0.99	3.99	9395.5	18	1.04	<u>≈1</u>

Na osnovu rezultata prikazanih u tabeli 1 se može zaključiti da je ekvivalentna statička metoda primenljiva za uobičajene antenske stubove visine do 45 m, dok se u slučaju viših stubova i stubova sa velikim koncentrisanim masama u vrhu uočava divergencija rezultata, ili čak prekoračenje granične vrednosti.

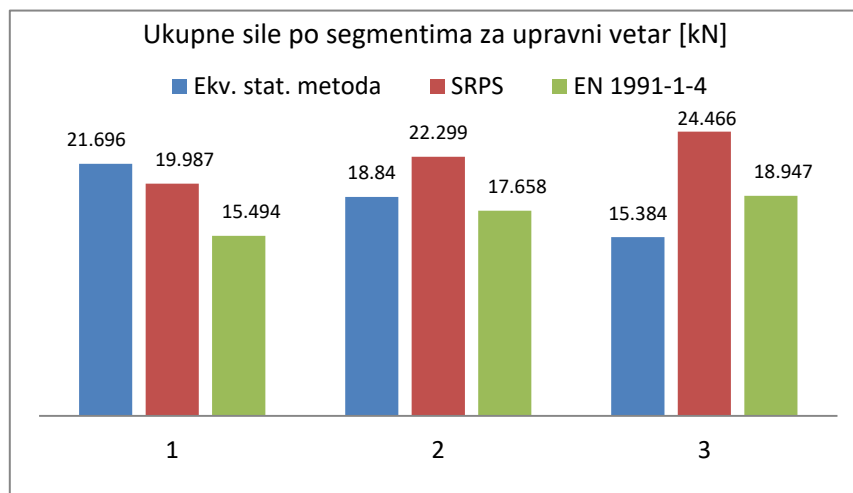
Za određivanje srednjeg opterećenja vetrom prema ekvivalentnoj statičkoj metodi koristi se sledeći izraz:

$$F_{m,w}(z) = \frac{q_p(z)}{1 + 7 \cdot I_v(z_e)} \cdot \sum c_f A_{ref} \quad (8)$$

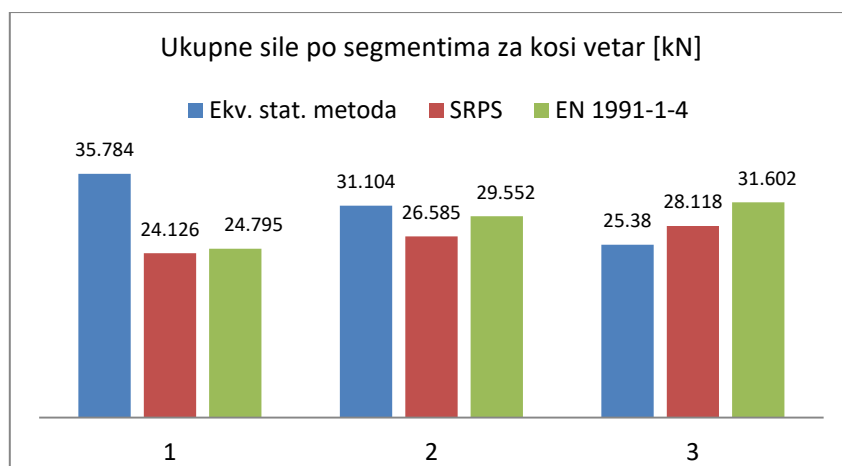
Ekvivalentno udarno opterećenje vetrom u pravcu vetra, se dobija kada se srednje opterećenje vetrom uveća usled uticaja inteziteta turbulencije I_v , koeficijenta konstrukcije $c_s c_d$, koeficijenta topografije c_o , visine tornja h i visine konstrukcije na kojoj se analizira udar vetra z_m :

$$F_{T,w}(z) = F_{m,w}(z) \left[1 + \left(1 + 0.2 \left(\frac{z_m}{h} \right)^2 \right) \cdot \frac{[1 + 7I_v(z_e)] \cdot c_s c_d - 1}{c_o(z_m)} \right] \quad (9)$$

Na dijagramima sa slika 5 i 6 su uporedo prikazane vrednosti rezultujućeg opterećenja vetrom za predmeni antenski stub po segmentima, za upravan i kos pravac dejstva prema ekvivalentnoj statičkoj metodi u SRPS EN 1993-3-1 [2], SRPS U.C7.113 [7] i EN 1991-1-4 [5].



Slika 5 - Ukupne sile po segmentima za upravni vetar [8]



Slika 6 - Ukupne sile po segmentima za kosi vetar [8]

2.3 UTICAJ LEDA NA PROMENU DEJSTVA VETRA

Dejstvo leda na konstrukciju se ispoljava na dva načina, kroz povećanje težine konstrukcije i promenu inteziteta dejstva vetra na konstrukciju.

Oba razmatrana standarda prepoznaju različite tipove ledenih naslaga, kao što su inje (*soft rime*), mraz (*hard rime*), led nastao zamrzavanjem vlažnog snega (*wet snow ice*) i led usled ledene kiše (*glaze ice*). Ovi oblici pojave leda imaju različite fizičke karakteristike kao što su gustina, adhezija, kohezija, boja i oblik. Led nastao zamrzavanjem vlažnog snega i led usled ledene kiše formiraju približno ravnomerne naslage po obimu štapova konstrukcije, dok se inje i mraz talože većim delom na navetrenoj strani konstrukcije menjajući na taj način oblik štapova i unoseći asimetriju u vertikalno opterećenje konstrukcije.

Za predele sa malom i umerenom opasnošću od pojave ledenih naslaga oba standarda [2] [7] dozvoljavaju uzimanje leda u konstantnoj debljini po obimu profila štapova. Standard SRPS EN 1993-3-1 [2] se poziva na standard ISO 12494 u kom su date osnovne smernice za određivanje potrebnih karakteristika ledenih naslaga, čak i za predele sa ozbiljnijim ledenim naslagama, kao i za inje i mraz. Domaći nacionalni prilog Evrokoda 3 [3] definiše debljinu leda na konstrukciji u funkciji nadmorske visine. Prema Evrokodu 3 [2],[3] uski rešetkasti stubovi (širine manje od 2 m), koji se nalaze na nadmorskim visinama većim od 1000 m, treba da se provere za slučaj potpunog zaleđavanja. U povučenom standardu SRPS U.H2.110 [1] su date tabele koje obuhvataju samo ledene naslage konstantne debljine po obimu štapova, dok je za predele u kojima je opasnost od pojave ledenih naslaga velika, podatke potrebno prikupiti od odgovarajuće meteorološke službe.

Sila od vetra za "zaleđeni toranj" u slučaju leda čije su naslage konstantne po obimu štapova, se određuje na isti način kao i za "nezaleđeni toranj", uzimajući u obzir povećanje dimenzija štapova usled ledenih naslaga. U predelima u kojima dolazi do pojave leda na konstrukciji, kombinacija opterećenja od vetra i leda često može da bude merodavna za proračun tornja.

Prema SRPS U.H2.110 [1] opterećenje vetrom zaleđene konstrukcije je dopunsko opterećenje, pa je dopušteni napon za kombinacije sa opterećenjem usled dejstva leda veći

nego za osnovna opterećenja. U Evrokodu se pri kombinovanju opterećenja od vetra i leda smatra da je karakteristični pritisak vetra manji kada dolazi do pojave leda na konstrukciji nego u ostalim situacijama. Ova pretpostavka se u proračunu uzima u obzir redukcijom inteziteta opterećenja od vetra množenjem faktorom k čija je vrednost, u zavisnosti od klase leda, data u ISO 12494.

Prema poglavlju C.6 standarda SRPS EN 1993-3-1 [2] treba da se razmatraju sledeće dve kombinacije dejstva vetra i leda na konstrukciju:

- dominantan led i odgovarajući vetar:

$$\gamma_G \cdot G_k + \gamma_{ice} \cdot Q_{k,ice} + \gamma_w \cdot k \cdot \psi_w \cdot Q_{k,w} \quad (10)$$

- dominantan vetar i odgovarajući led:

$$\gamma_G \cdot G_k + \gamma_w \cdot k \cdot Q_{k,w} + \gamma_{ice} \cdot \psi_{ice} \cdot Q_{k,ice} \quad (11)$$

U Nacionalnom aneksu SRPS EN 1993-3-1/NA [2] definisane su preporučene vrednosti koeficijenata kombinacije $\psi_w = \psi_{ice} = 0,5$.

3 SPECIFIČNOSTI PRORAČUNA STABILNOSTI PRITISNUTIH ŠTAPOVA PREMA SRPS EN 1993-3-1

Prema SRPS U.H2.110 [1] dimenzionisanje nosećih elemenata konstrukcije tornja se vrši primenom uobičajenih postupaka kontrole maksimalnih napona u merodavnim presecima, odnosno kontrole stabilnosti pritisnutih elemenata na izvijanje, za odgovarajuće kombinacije opterećenja. Sa druge strane, Evrokod [2] dopunjava standardne postupke proračuna nosivosti centrično pritisnutih elemenata, koji su dati u bazičnom standardu za proračun čeličnih konstrukcija SRPS EN 1993-1-1 [9], definisanjem koeficijenta efektivne vitkosti k kojima se uvodi uticaj naknadnih imperfekcija usled fleksibilnosti ili ekscentriciteta veza. Koeficijent efektivne vitkosti k se množi odgovarajućom relativnom vitkošću elementa u svakoj relevantnoj ravni poprečnog preseka, pa se ovako dobijena efektivna, relativna vitkost koristi u daljem postupku proračuna. Vrednosti ovih koeficijenata date su u Prilogu G, i u slučaju pojasnih elemenata zavise od oblika rešetkaste ispune u refrenetnoj ravni stuba, odnosno konstruktivnog oblikovanja veze na krajevima i oblika poprečnog preseka u slučaju elemenata ispune:

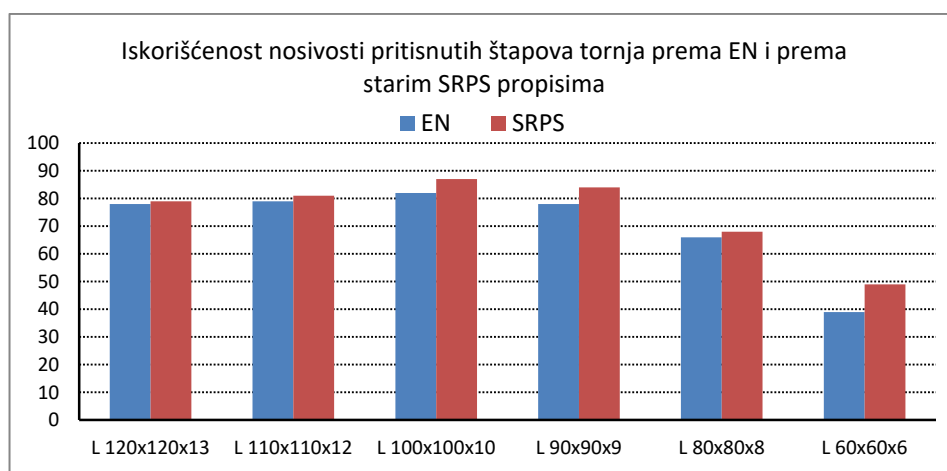
$$k = 0.8 + \frac{\bar{\lambda}}{10} \begin{cases} \geq 0.9 \\ \leq 1.0 \end{cases} \text{ za pojasne štapove} \quad (12)$$

$$k = 0.7 + \frac{0.35}{\bar{\lambda}_w} \text{ za štapove ispune} \quad (13)$$

SRPS EN 1993-3-1 [2] dopušta vezu elemenata ispune za pojaseve samo sa jednim zavrtnjem, pod uslovom da se proračunska vrednost nosivosti pritisnutog elementa na izvijanje redukuje koeficijentom η koji ima vrednost 0,8 ukoliko su oba kraja ispune vezana sa jednim zavrtnjem, odnosno 0,9 u slučaju da je veza na jednom kraju fleksibilna i vezana sa jednim zavrtnjem, a na drugom kraju kruta.

4. ZAKLJUČAK

Uz sve suštinske razlike između navedenih propisa koje su detaljno opisane u radu, kao krajnji rezultat dobijaju se bliske vrednosti koeficijenta iskorišćenosti za sve merodavne konstruktivne elemente stuba. Generalno, može se reći da su povučeni srpski propisi [1] blago konzervativniji u odnosu na evrokod, jer se dobija veće iskorišćenje nosivosti pritisnutih štapova (za 1% do 10%), a zategnutih do 10% (videti sliku 7).



Slika 7 – Koeficijenti iskorišćenosti nosivosti pritisnutih elemenata u %

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POŽARNA OTPORNOST SPREGNUTIH STUBOVA PREMA EVROKODU

Rezime:

S-35

U radu je prikazan i analiziran proračun požarne otpornosti stuba višespratne poslovne zgrade. Analiziran je spregnuti stub sa delimično ubetoniranim čeličnim profilom i spregnuti stub sa potpuno ubetoniranim čeličnim profilom. Požarna otpornost spregnutih stubova upoređena je sa požarnom otpornošću čeličnog stuba iste nosivosti. Nosivost stubova na dejstvo požara određena je za standardni požar koji je definisan standardom ISO 834. Opterećenja i svojstva materijala su u skladu sa Evrokodom. Dejstvo standardnog požara na stub simulirano je pomoću programa ABAQUS. Pomoću FEM modela određene su vrednosti temperature u delovima poprečnog preseka pri standardnom požaru.

Ključne reči: spregnuti stub, vatrootpornost, proračun, Evrokod, ISO 834

FIRE RESISTANCE OF THE COMPOSITE COLUMNS ACCORDING TO EUROCODE

Summary:

In this paper the calculation and analysis of the fire resistance for typical column of the multi-story office building was presented. Composite columns with partially encased steel section and with fully encased steel section were analyzed. Fire resistance of the composite columns was compared to fire resistance of the steel column of the same bearing capacity. Bearing capacity of the columns in fire condition was obtained for the standard fire according to ISO 834. Load and properties of materials are in all according to Eurocode. Action of the standard fire to the column was simulated using the ABAQUS software. The values of the temperature in parts of the cross section of the column in the standard fire was obtained by FEM model.

Key words: composite column, fire resistance, design, Eurocode, ISO 834,

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1 UVOD

Pod pojmom požar se podrazumeva nekontrolisano gorenje koje ugrožava ljudske živote i nanosi materijalnu štetu. Osnovni ciljevi zaštite građevinskih konstrukcija od požara su smanjenje rizika za živote korisnika tj. povećanje bezbednosti ljudi, ograničenje širenja požara kao i smanjenje nastale štete. Oslobođanje velikih količina toplote koje su praćene visokim temperaturama utiče na nosivost građevinske konstrukcije. Problematika požara je veoma bitna za metalne konstrukcije i zauzima značajno mesto u okviru evrokodova za konstrukcije. Evrokod 1 Deo 1-2 se bavi problematikom požarnog opterećenja, a Evrokod 3 Deo 1-2 [6] i Evrokod 4 Deo 1-2 [4] definišu postupke proračuna nosivosti čeličnih i spregnutih konstrukcija pri dejstvu požara.

Kako požar predstavlja pojavu koja zavisi i od velikog broja parametara, standardizacija modeliranja dejstva požara nije ni najmanje jednostavna. Uslovi ispitivanja elemenata u peći, u cilju određivanja njihove otpornosti u uslovima dejstva požara precizno su propisani međunarodnim standardom ISO 834 [3]. Standardna požarna kriva koja definiše promenu temperature vazduha tokom trajanja požara, definisana je izrazom:

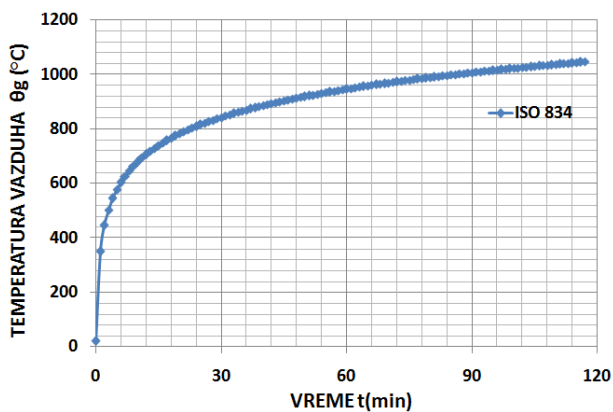
$$\theta_g = 345 \cdot \log_{10}(8t + 1) + 20$$

gde je:

θ_g temperatura vazduha u požarnom sektoru [$^{\circ}\text{C}$],

t vreme trajanja standardnog požara [min].

Kriva definisana prethodnim izrazom prikazana je na slici 1.



Slika 1 – Standardna požarna kriva prema ISO 834

Problem određivanja požarne otpornosti konstrukcija se može razdvojiti na dva dela: termički i mehanički. Termička analiza se može podeliti na dva dela: jedan se odnosi na prenos toplote od požara na površinu elementa, koji predstavlja kombinaciju konvekcije i zračenja, a drugi deo se odnosi na dalje provođenje toplote kondukcijom kroz konstruktivni element. Provođenje toplote opisuje se Furijeovom diferencijalnom jednačinom u kojoj figurišu

termičke karakteristike materijala: toplotna provodljivost λ i specifična toplota c , veličine zavisne od temperature materijala. Promena temperature značajno utiče na mehanička svojstva materijala, tako što sa porastom temperature dolazi do pogoršavanja ovih svojstava. Od posebnog značaja je poznavanje oblika i karakterističnih vrednosti σ - ε dijagrama, granice razvlačenja, modula elastičnosti zagrejanog materijala.

Tabela 1- Mehaničke karakteristike čelika i betona na različitim temperaturama

Temperatura čelika θ_a [°C]	$k_{E,\theta} = \frac{E_{a,\theta}}{E_a}$	$k_{y,\theta} = \frac{f_{ay,\theta}}{f_{ay}}$	Temperatura betona θ_c [°C]	$k_{c,\theta} = \frac{f_{c,\theta}}{f_c}$	$\varepsilon_{cu,\theta} \cdot 10^3$
20	1	1	20	1	2,5
100	1	1	100	1	4
200	0,9	1	200	0,95	5,5
400	0,7	1	400	0,75	10
500	0,6	0,78	500	0,6	15
...
1100	0,0225	0,02	1100	0,01	25
1200	0	0	1200	0	-

Prema Evrokodu [6] i [4], proračun građevinskih konstrukcija izloženih dejstvu požara se zasniva na metodi graničnih stanja. Granični kriterijum koji analizirana konstrukcija, ili njen element treba da zadovolji može se definisati u domenu vremena, u domenu temperature i u domenu nosivosti.

Primena čeličnih konstrukcija u objektima zgradarstva vrlo često je ograničena zahtevanim kriterijumima za nosivost konstrukcije pri dejstvu požara. Čelični stubovi bez protivpožarne zaštite mogu zadovoljiti nosivost na dejstvo požara do 30 min. Troškovi pasivne ili aktivne zaštite čeličnih konstrukcija mogu biti značajni. Ako se zahteva veća nosivost na dejstvo požara racionalnije je projektovati spregnute stubove sa kojima se može postići i nosivost za dejstvo požara od 240 minuta.

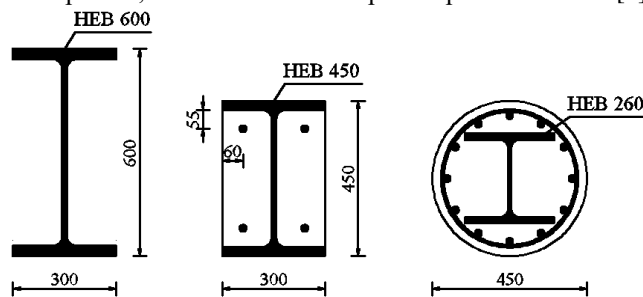
2 PRORAČUN NOSIVOSTI STUBOVA PRI DEJSTVU POŽARA

Proračun stubova u objektima visokogradnje u uslovima normalnih temperatura (20°C) svodi se na određivanje nosivosti preseka i kontrolu stabilnosti stuba kao elementa konstrukcije. Nosivost i stabilnost stuba pri dejstvu požara mogu se dokazati na isti način kao i za stub na sobnoj temperaturi, uzimajući u obzir pad mehaničkih svojstava materijala preko redukcionih parametara koji zavise od temperature u elementu u razmatranom trenutku.

S obzirom na to da je termička provodljivost betona mala, toplota se kroz betonski deo preseka širi mnogo sporije nego kroz čelik. Visoka požarna otpornost stubova može se postići primenom spregnutih stubova od čelika i betona. U praksi se najčešće koriste tri tipa spregnutih stubova: potpuno ubetonirani čelični profili (*fully encased steel sections - FES*), delimično ubetonirani čelični profili (*partially encased steel sections - PES*) i betonom ispunjeni šuplji profili (*concrete filled tubular sections - CFS*) [4]. Čelični stubovi su izuzetno osetljivi na

direktnu izloženost požaru, dok kod potpuno ubetoniranih čeličnih profila beton prekriva čelični profil i štiti ga od intenzivnog zagrevanja (beton deluje kao izolator). U slučaju delimično ubetoniranih čeličnih profila nožice čeličnog profila su direktno izložene požaru, pa je smanjen efekat betona kao zaštite od požara. Kod šupljih profila ispunjenih betonom čelični profil je direktno izložen uticaju visokih temperatura, dok betonsko jezgro ostaje hladno i ponaša se kao element koji apsorbuje toplotu.

Analiza požarne otpornosti, prikazana u ovom radu urađena je za jedan najopterećeniji unutrašnji stub sedmospratne poslovne zgrade. Analizirani stub je sa svih strana izložen požarnom opterećenju. Spratna visina je 3,2 m, pripadajuća površina osnove je 86,4 m². Usvojene karakteristike materijala stubova koji su analizirani u radu: čelik S355, armatura B500 i beton C40/50. Iz uslova nosivosti poprečnog preseka i nosivosti elementa na izvijanje pri sobnoj temperaturi (20°C), usvojena su tri različita tipa poprečnih preseka: čist čelični presek HEB 600, spregnut presek sa delimično ubetoniranim čeličnim profilom HEB 450 i spregnut presek sa potpuno ubetoniranim čeličnim profilom HEB 260, videti sliku 2. Za svaki od navedenih stubova odnos proračunske vrednosti aksijalne sile i proračunske nosivosti pritisnutog elementa na izvijanje ($N_{Ed} / N_{b,Rd}$) je 0,8±3%. Zatim je analizirana nosivost ovih stubova u standardnom požaru, za standardnu krivu požara prema ISO 834 [4].



Slika2 – Poprečni preseki stubova

2.1 NOSIVOST ČELIČNOG STUBA

Za određivanje požarne otpornosti čeličnog stuba (HEB600) korišćen je uprošćen proračunski model koji je dat u Evrokodu 3 [6]. Pretpostavljena je zahtevana požarna otpornost stuba od 30 minuta, a zatim je provereno da li stub zadovoljava kriterijume nosivosti pri dejstvu požara.

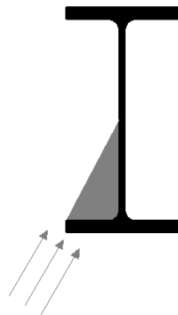
Proračunska nosivost pritisnutog elementa na izvijanje pri dejstvu požara u vremenu t , za klase poprečnog preseka 1, 2 i 3, u slučaju ravnomerne raspodele temperature u poprečnom preseku određuje se prema izrazu:

$$N_{b,fi,Rd} = \chi_{fi} \cdot A \cdot k_{y,\theta} \cdot f_y / \gamma_{M,fi}$$

gde je:

- χ_{fi} bezdimenzionalni koeficijent izvijanja za temperaturu θ dostignutu u vremenu t ,
- A površina poprečnog preseka stuba,
- $k_{y,\theta}$ redukциони faktor granice razvlačenja čelika za temperaturu θ dostignutu u vremenu t ,
- f_y granica razvlačenja čelika pri sobnoj temperaturi (20°C),
- $\gamma_{M,fi}$ parcijalni koeficijent, za svojstva materijala pri dejstvu požara, jednak je 1.0.

Brzina zagrevanja poprečnog preseka zavisi od njegove mase i površine obima. Nivo zagrevanja poprečnog preseka opisuje se faktorom preseka koji predstavlja odnos površine A izložene zračenju i konvekciji i mase V koja direktno zavisi od površine poprečnog preseka. U zavisnosti od toga da li je element zaštićen ili ne koristi se faktor preseka za zaštićene A_p/V , odnosno A_m/V za nezaštićene elemente. Ukoliko postoji efekat senke faktor preseka može da se redukuje korekcionim faktorom k_{sh} , videti sliku 3. Kako razmatrani stub nije zaštićen, pri određivanju temperature θ uzeti su u obzir samo vreme t i faktor preseka A_m/V redukovan korekcionim faktorom k_{sh} .



Slika3 – Efekat senke

Pri dejstvu požara dolazi do smanjenja krutosti stuba koji je zahvaćen požarom [4]. Sa druge strane, stubovi koji se nalaze iznad ili ispod razmatranog stuba zaštićeni su od porasta temperature međuspratnim pločama, tako da njihova krutost nije ugrožena. Na taj način, stubovi zahvaćeni požarom u slučaju ukrućenih okvira bivaju u određenoj meri uklješteni u spratove ispod i iznad njih. Imajući ovo u vidu dužina izvijanja stuba na spratu koji je u požaru je manja od sisteme dužine stuba. Pri određivanju vitkosti na granici razvlačenja koriste se vrednosti svojstava materijala u funkciji temperature poprečnog preseka. Granica razvlačenja f_y i modul elastičnosti E menjaju se tokom trajanja požara usled zagrevanja poprečnog preseka.

Da bi se proverila nosivost stuba na dejstvo požara, potrebno je odrediti proračunske vrednosti uticaja u stubu pri dejstvu požara. Prema uprošćenom postupku proračuna (EN 1991-1-2) da bi se dobile reakcije oslonaca i presečne sile za incidentnu proračunsku kombinaciju opterećenja u slučaju požara, uticaji koji su dobijeni iz merodavnih kombinacija opterećenja za analizu konstrukcije na sobnoj temperaturi množe se koeficijentom redukcije. Koeficijent redukcije predstavlja odnos vrednosti uticaja u konstrukciji za stalne i prolazne proračunske situacije i incidentnu proračunsku kombinaciju sa požarom. Pretpostavlja se da su uticaji u trenutku $t=0$ nepromenjeni tokom čitavog perioda trajanja požara [4]. Vrednosti uticaja u konstrukciji pri dejstvu požara mogu se odrediti prema sledećem izrazu:

$$E_{fi,d} = \eta_{fi} \cdot E_d$$

gde su:

E_d proračunska vrednost odgovarajućeg uticaja za proračun elementa na sobnoj temperaturi, za merodavnu kombinaciju dejstava,

η_{fi} redukcioni faktor za uticaje u konstrukciji pri dejstvu požara.

U slučaju analiziranog stuba višespratne poslovne zgrade vrednost redukcionog faktora η_{fi} iznosi 0,52, a određen je prema sledećoj formuli:

$$\eta_{fi} = \frac{\gamma_{GA} \cdot G_k + \Psi_{1,1} \cdot Q_{k,1}}{\gamma_G \cdot G_k + \gamma_{Q,1} \cdot Q_{k,1}}$$

gde je:

- G_k karakteristična vrednost uticaja usled stalnog dejstva,
- γ_G parcijalni koeficijent sigurnosti za stalno dejstvo,
- $Q_{k,1}$ karakteristična vrednost uticaja usled promenljivog dejstva,
- $\gamma_{Q,1}$ parcijalni koeficijent sigurnosti za promenljiva dejstva,
- γ_{GA} parcijalni koeficijent sigurnosti za stalno dejstvo za incidentne proračunske situacije,
- $\Psi_{1,1}$ koeficijent kombinacije za promenljiva dejstava.

Usvojeni poprečni presek stuba HEB 600, bez protivpožarne zaštite, nije zadovoljio kriterijume nosivosti i stabilnosti pri dejstvu požara od 30 min. Proračunski je određeno vreme trajanja požara za koje je nosivost analiziranog stuba zadovoljena. Na osnovu proračuna dobijeno je da analizirani stub zadovoljava kriterijume nosivosti i stabilnosti za dejstvo standardnog požara u trajanju do 22 min.

2.2 NOSIVOST SPREGNUTOG STUBA – PARCIJALNO UBETONIRANI PROFIL

Sledeći stub za koji je analizirana nosivost pri dejstvu požara je spregnuti stub prikazan na slici 2b. Za parcijalno ubetonirani čelični profil usvojen je vruće valjani poprečni presek HEB 450. U Evrokodu 4 [4] su definisane tri metode proračuna nosivosti na dejstvo požara spregnutih stubova sa parcijalno ubetoniranim čeličnim profilom: tabelarna, pojednostavljena i napredna.

Tabela 2- Kriterijumi za primenu tabelarne metode prema EN 1994-1-2 za $0,47 \leq \eta_{fi} \leq 0,66$

Dozvoljeni parametri R60	Postojeći parametri	Ispunjeni uslovi
$e_w / e_f > 0,5$	$14 / 26 = 0,538$	DA
h i $b > 400$	$h = 450\text{mm}$ i $b = 300\text{mm}$	NE
u_1 i $u_2 > 70$	$u_1 = 55\text{mm}$ i $u_2 = 60\text{mm}$	NE
$A_s / (A_c + A_s) > 4\%$	$1134 / (112066 + 1134) = 1\%$	NE

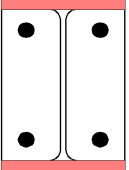
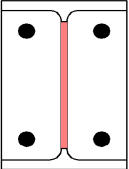
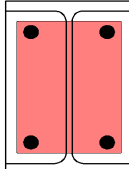
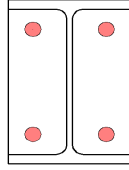
Analizirani stub nije zadovoljio kriterijume, videti tabelu 2, za primenu tabelarne metode proračuna, pa je proračun sproveden prema pojednostavljenoj metodi. Prema ovoj metodi nosivost na dejstvo požara posebno se određuje za svaki deo poprečnog preseka: nožice čeličnog profila $N_{fi,pl,Rd,f}$, rebro čeličnog profila $N_{fi,pl,Rd,w}$, beton $N_{fi,pl,Rd,c}$ i armaturu $N_{fi,pl,Rd,s}$, videti tabelu 3, pri čemu je ukupna nosivost poprečnog preseka:

$$N_{fi,pl,Rd} = N_{fi,pl,Rd,f} + N_{fi,pl,Rd,w} + N_{fi,pl,Rd,c} + N_{fi,pl,Rd,s}$$

Nosivost stuba na dejstvo požara dobija se množenjem nosivosti poprečnog preseka bezdimenzionalnim koeficijentom izvijanja χ_z čija vrednost je u funkciji temperature:

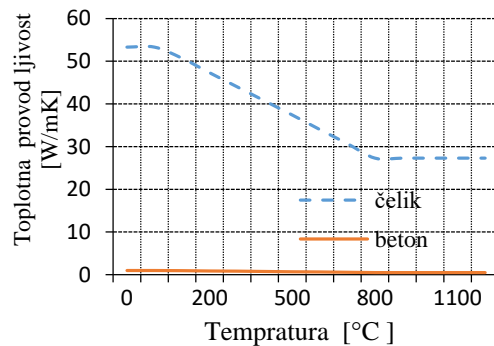
$$N_{fi,Rd,z} = \chi_z \cdot N_{fi,pl,Rd}$$

Tabela 3- Proračun nosivosti delova poprečnog preseka prema pojednostavljenoj metodi

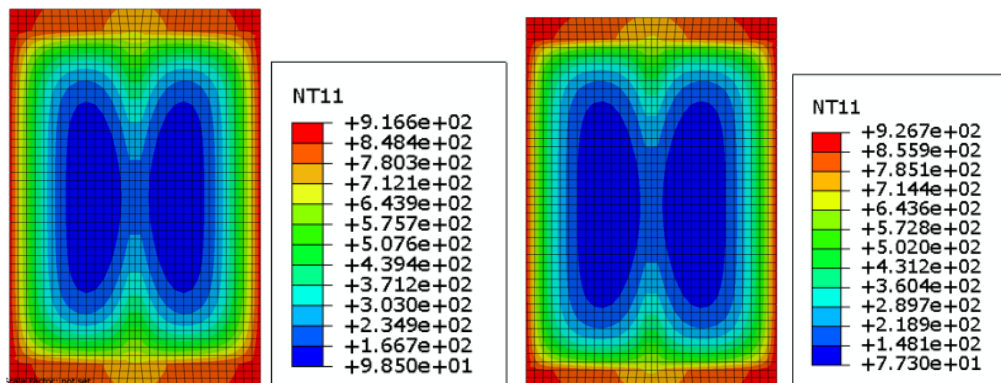
<p>NOŽICE PROFILA</p> 	$N_{fi,pl,Rd,f} = 2 \cdot \frac{b \cdot t_f \cdot f_y \cdot k_{y,\theta}}{\gamma_{M,fi}}$ <p> b, t_f širina, odnosno debljina nožice čeličnog profila f_y karakteristična vrednost granice razvlačenja čeličnog profila $k_{y,\theta}$ redukcioni faktor granice razvlačenja čelika u funkciji temperature </p>
<p>REBRO PROFILA</p> 	$N_{fi,pl,Rd,w} = \frac{(t_w \cdot (h - 2 \cdot t_f - 2 \cdot h_{w,fi}) \cdot f_{y,w,t})}{\gamma_{M,fi}}$ <p> h, t_w visina, odnosno debljina rebra čeličnog profila $h_{w,fi}$ redukovana visina rebra profila </p> $h_{w,fi} = 0.5 \cdot (h - 2 \cdot t_f) \cdot \left(1 - \sqrt{1 - 0.16 \cdot \left(\frac{H_t}{h}\right)}\right)$ <p> H_t nalazi u tablici datoj u Evrokodu 4 [2] $f_{y,w,t}$ redukovana granica razvlačenja čelika </p> $f_{y,w,t} = f_y \cdot \sqrt{1 - \left(\frac{0.16 \cdot H_t}{h}\right)}$
<p>BETONSKI DEO PRESEKA</p> 	$N_{fi,pl,Rd,c} = \frac{0.86 \cdot \{(h - 2 \cdot t_f - 2 \cdot b_{c,fi}) \cdot (b - t_w - 2 \cdot b_{c,fi}) - A_s\} \cdot f_{c,\theta}}{\gamma_{M,fi}}$ <p> $b_{c,fi}$ redukovana debljina betona A_s ukupna površina armature $f_{c,\theta}$ redukovana čvrstoća betona u funkciji temperature </p>
<p>ARMATURA</p> 	$N_{fi,pl,Rd,s} = \frac{A_s \cdot k_{y,t} \cdot f_{s,y}}{\gamma_{M,fi}}$ <p> $f_{s,y}$ granica razvlačenja čelika za armaturu $k_{y,t}$ redukcioni faktor za granicu razvlačenja čelika za armaturu </p>

Nosivost stuba na dejstvo požara određena je i na osnovu temperaturnog polja dobijenog pomoću FEM modela u programu ABAQUS. Pri modeliranju stuba u softveru korišćeni su

solid – heat transfer konačni elementi (DC3D8). Usvojeni broj konačnih elemenata u poprečnom preseku iznosi 8280. Analiza je pokazala da dalje povećanje broja konačnih elemenata ne utiče značajno na rezultate proračuna. Imajući u vidu činjenicu da toplotna provodljivost, kao termička karakteristika materijala, opada pri povećanju temperature, videti sliku 4, napravljena su dva modela: jedan sa konstantnom toplotnom provodljivošću materijala i drugi sa toplotnom provodljivošću koja zavisi od temperature. Dijagram zavisnosti toplotne provodljivosti u funkciji temperature materijala za čelik i beton usvojene su prema Evrokodu. Za parametar toplotne provodljivosti između betona i čelika u programu ABAQUS usvojena je vrednost $k = 1000$, postoji kontakt između čelika i betona po celoj površini [6]. Dobijene vrednosti temperature za određene delove poprečnog preseka za trajanje standardnog požara od 60 minuta prikazane su na slici 5. Rezultati proračuna nosivosti pojednostavljenom metodom datom u Evrokodu i proračunom na osnovu vrednosti temperature delova preseka dobijenih FEM modelom dati su u tabeli 4.



Slika 4 – Zavisnost toplotne provodljivosti materijala od temperature



Slika 5 – Temperaturno polje za $t=60$ min sa konstantnom toplotnom provodljivošću i toplotnom provodljivošću koja zavisi od temperature

Tabela 4- Rezultati dobijenih uprošćenim postupkom i pomoću softvera

	Nožice T [°C]	Rebro T [°C]	Beton T [°C]	Nosivost poprečnog preseka [kN]
EN 1994-1-1 sobna temperatura	20	20	20	10772
EN 1994-1-2 uprošćen postupak	786	/	317	4836
ABAQUS $\lambda = \text{const}$ (60 min.)	800	337	281	5174
ABAQUS $\lambda \neq \text{const}$ (60 min.)	818	328	249	5271

Nosivost poprečnog preseka dobijena pojednostavljenom metodom proračuna prema Evrokodu se razlikuje za 7% od rezultata dobijenih FEM analizom za model sa konstantnom toplotnom provodljivošću, odnosno za 9% od rezultata iz FEM analize za model sa promenljivom toplotnom provodljivošću. Razlike koje su dobijene u analiziranom slučaju pokazuju da za ovakav tip poprečnog preseka promena toplotne provodljivosti u funkciji temperature nema značajan uticaj na konačne rezultate nosivosti stuba pri dejstvu požara. Rezultati dobijeni sa konstantnom toplotnom provodljivošću su na strani sigurnosti.

Pri proračunu nosivosti stuba na požar armatura je zanemarena. Proračunom prema Evrokodu kao i na osnovu rezultata FEM analize dokazna je nosivost i stabilnost stuba za trajanje standardnog požara od 60 minuta.

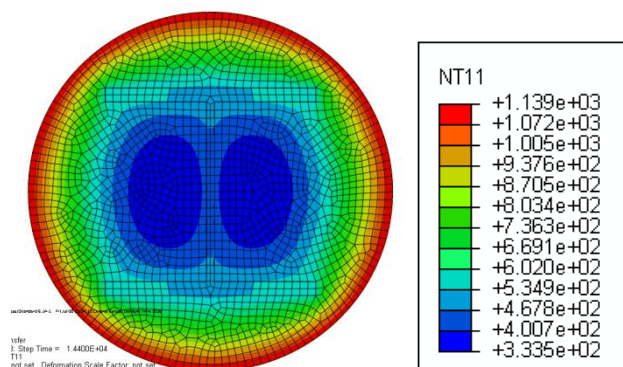
2.3 NOSIVOST SPREGNUTIH STUBOVA – POTPUNO UBETONIRANI PROFIL

Određivanje nosivosti na dejstvo požarna spregnutog stuba sa potpuno ubetoniranim čeličnim profilom prema Evrokodu 4 [4] moguće je samo tabelarnom metodom i to za pravougaoni poprečni presek. Nosivost na dejstvo požara određuje se očitavanjem iz tabele na osnovu geometrije poprečnog preseka. Analiziran je stub kružnog poprečnog preseka, slika 2c. Nosivost preseka je određena pomoću FEM modela u programu ABAQUS, pomoću koga je dobijeno temperaturno polje prikazano na slici 6. Rezultati proračuna prikazani su u tabeli 5.

Tabela 5- Rezultati proračuna nosivosti dobijeni pomoću FEM modela

	Nožice T [°C]	Rebro T [°C]	Beton T [°C]	Nosivost poprečnog preseka [kN]
EN 1994-1-1 sobna temperatura	20	20	20	8534
ABAQUS (240 min.)	544	452	590	4518

Na osnovu proračunskih vrednosti uticaja koji se javljaju u stubu u uslovima delovanja požara, proračunom je dokazano da analizirani stub zadovoljava požarnu otpornost za trajanje požara od 240 min.



Slika 6 –Temperaturno polje za $t=240$ min sa konstantnom toplotnom provodljivošću za potpuno ubetonirani čelični profil HEB 260

3 ZAKLJUČAK

Pri visokim temperaturama koje su rezultat delovanja požara dolazi do smanjenja nosivosti elemenata konstrukcije. Smanjenje nosivosti je rezultat promene svojstava materijala pri visokim temperaturama. Na osnovu rezultata proračuna prikazanih u ovom radu zaključuje se:

Vrednost redukcije nosivosti stuba pri dejstvu požara bitno zavisi od tipa poprečnog preseka;

Kada se zahteva veća nosivost na dejstvo požara treba projektovati spregnute stubove.

Stubovi sa delimično ubetoniranim čeličnim profilom su jednostavni za izvođenje i konstruktivno oblikovanje detalja. Sa ovim tipom stubova može se postići nosivost na dejstvo požara od 60 min bez PP zaštite. Ako se zahteva veća nosivost ona se može postići PP zaštitom površine nožica.

Najveća nosivost stubova na dejstvo požara se postiže sa poprečnim presecima koji su formirani sa potpuno ubetoniranim čeličnim profilom.

Za potpuno ubetonirane čelične delove poprečnog preseka, nosivost na dejstvo požara od 240 min bez PP zaštite može se postići usvajanjem minimalno propisane vrednosti zaštitnog sloja betona koja je data u Evrokodu 4.

LITERATURA

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Dorđe Đuričić¹, Duško Lučić², Dragoslav Arsović³

GRANIČNA NOSIVOST ŠTAPOVA ISPUNE REŠETKASTIH KONSTRUKCIJA

Rezime:

S-36

U radu je data analiza proračunske nosivosti štapova ispune rešetkastih konstrukcija od CHS profila. Određivanje proračunske nosivosti kod rešetkastih konstrukcija se svodi na proračun prema EN1993-1-1, za centrično pritisnute štapove sa uticajem izvijanja i proračun prema EN1993-1-8 za proračunsku nosivost zavarenih veza u čvoru rešetkaste konstrukcije. U radu je prikazana međusobna zavisnost ova dva proračuna i preko odgovarajućih dijagrama dat postupak za utvrđivanje da li je za dostizanje granične nosivosti štapa ispune merodavno njegovo izvijanje ili plastifikacija pojasnog štapa u čvoru rešetke.

Ključne reči: Rešetkaste konstrukcije, K - čvor, izvijanje, granična nosivost

ULTIMATE LOAD BEARING CAPACITY OF WEB MEMBERS OF LATTICE STRUCTURES

Summary:

This paper presents analysis of carrying capacity calculation of steel lattice CHS web elements. Designing of lattice elements is based on EN1993-1-1, and assumes calculating carrying capacity, taking in to consideration elastic buckling of centrally pressed elements, and calculating carrying capacity of lattice chord welded connection, based on EN1993-1-8 standard. Both of this two criteria are discribed in the paper and some diagrams are developed for better understanding wich of them is critical for designig of lattice web elements.

Key words: Lattice structures, K - joint, Buckling, Ultimate strength

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1 OPŠTE

Za elemente rešetkaste konstrukcije od šupljih profila koriste se:

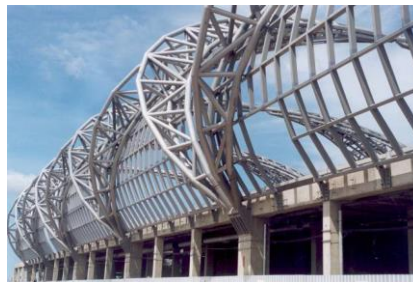
- Kružni šuplji profili, eng. circular hollow sections (CHS),
- Pravougaoni šuplji profili, eng. rectangular hollow sections (RHS) i
- Kvadratni šuplji profili, eng. square hollow sections (SHS).

Kružni cevasti profili se sve više koriste u izgradnji rešetkastih konstrukcija. Odlične osobine kružni profil pruža u pogledu nosivosti na pritisak, savijanje i uvijanje. Takođe kružni oblik je neprevaziđen u pružanju otpora dejstvu vetra i talasa. Zahvalan je oblik koji daje velike mogućnosti atraktivnog arhitektonskog oblikovanja. Kako nema oštih ivica kružni profili se bolje koroziono zaštićuju i trajnija je nanescena zaštita.

Kružni oblik daje mogućnost direktnog spajanja profila međusobno tako da se omogućuje izrada lakih i elegantnih konstrukcija koje su i ekonomski isplativije od konstrukcija sa otvorenim profilima. Primeri takvih konstrukcija su dati na slikama 1 i 2 [3].



Slika 1- Paviljon u Sevilji, Španija

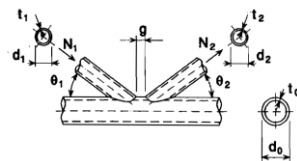


Slika 2- Aeodrom Bangkok, Tajland

2 PROJEKTOVANJE KONSTRUKCIJA OD ŠUPLJIH CEVASTIH PROFILA

2.1 KARAKTERISTIKE K-ČVORA, PROJEKTOVANJE I PRORAČUN ČVOROVA REŠETKE

U radu je analiziran K-čvor rešetkastog nosača od CHS profila. Na slici 3 su date oznake geometrijskih veličina K-čvora.

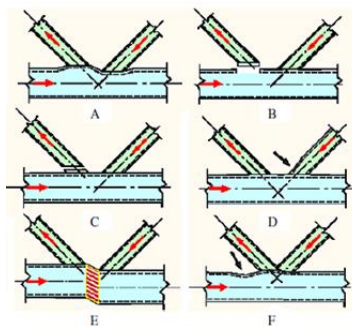


Slika 3- Oznake dimenzija K čvora

Pojasni štap ima prečnik d_0 i debljinu zida t_0 , pritisnuta dijagonala prečnik d_1 i debljinu zida t_1 , zategnuta dijagonala prečnik d_2 i debljinu zida t_2 . Napadni uglovi dijagonala su θ_1 i θ_2 . Razmak između dijagonala je g (eng. gap).

2.1.1 Modeli loma K čvora rešetkaste zavarene konstrukcije od cevastih profila

Packer i dr. [3] imaju šest režima loma za vezu tipa K prikazanih na slici 4.



- Režim A: Plastifikacija zida pojasnog štapa ;
- Režim B: Lom pojasnog štapa po obodu pritiskute ili zategnute dijagonale;
- Režim C: Lom štapa dijagonale ili pucanje šava na dijagonali;
- Režim D: Lokalno izvijanje pritiskute dijagonale;
- Režim E: Smicanje pojasnog štapa usled velikog razmaka između cevi u čvoru;
- Režim F: Plastifikacija zida pojasnog štapa kod preklopljenih štapova.

Slika 4 - Režimi loma K čvora

Za rešetkaste konstrukcije sa K čvorovima najčešći slučaj loma je režim A, a kontrola se vrši i za režim B [3-4].

2.1.2 Proračun granične sile u štapovima ispune K čvora

Za potpunu analizu pritisnutih štapova ispune rešetkaste konstrukcije potrebno je odrediti graničnu nosivost u dva nezavisna slučaja, a to su:

- granična nosivost štapa uzimajući u obzir uticaj izvijanja,
- kontrola granične nosivosti u štapu ispune iz uslova da ne dođe do plastifikacije ili prekomerne deformacije pojasnog štapa u čvoru.

Sličnu analizu treba sprovesti i za zategnute štapove, samo bez uticaja izvijanja.

Proračun pritisnutih i zategnutih štapova prema Evrokodu definisan ju u EN1993 deo 1-1 [1] a proračun granične sile u čvoru je definisan u EN1993 deo 1-8 [2].

2.1.3 Proračunska nosivost štapova ispune K čvora rešetkaste zavarene konstrukcije od cevastih profila (CHS) prema EN 1993- 1-8 [2]

Proračunska nosivost zavarenih veza između elemenata ispune i pojasnih štapova kod kružnih šupljih profila je data izrazom.

$$N_{1,Rd} = \frac{k_g k_p f_{y0} t_0}{\sin \theta_1} \left(1.8 + 10.2 \frac{d_1}{d_0} \right) / \gamma_{M5} \quad (1)$$

U izrazu (1) koeficijent k_g je koeficijent zavistan od razmaka profila (g) i parametara pojasne cevi d_0 i t_0

$$k_g = \gamma^{0.2} \left[1 + \frac{0.024 \gamma^{0.24}}{1 + \exp(0.5g/t_0 - 1.33)} \right] \quad (2)$$

gde je :

$$\gamma = \frac{d_0}{2t_0} \quad (3)$$

U izrazu (1) koeficijent k_p je koeficijent napona u pojasnom štapu

za $n_p > 0$ - pritisak

$$k_p = 1 - 0.3n_p(1 + n_p), \quad k_p \leq 1 \quad (4)$$

gde je: n_p -koeficijent napona

$$n_p = \frac{\sigma_{p,Ed}}{f_{y,0}} \quad (5)$$

gde je:

$$\sigma_{p,Ed} = \frac{N_{p,Ed}}{A_0} \quad (6)$$

u izrazu (6) A_0 je površina pojasnog štapa, a $N_{p,Ed}$ proračunska vrednost sile pritiska u pojasnom štapu.

za $n_p < 0$ - zatezanje, $k_p = 1$.

2.1.4 Proračunska nosivost štapova ispune sa uticajem izvijanja prema EN1993 -1-1 [1]

Granična nosivost je data izrazom:

$$N_{b,Rd} = \chi \cdot N_{c,Rd} = \chi \cdot \frac{A \cdot f_y}{\gamma_{M1}} \quad (7)$$

gde je:

$N_{b,Rd}$ -granična nosivost štapa sa uticajem izvijanja

Bezdimenzionalni koeficijent izvijanja χ dat je izrazom

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} \quad (8)$$

gde je:

$$\Phi = 0.5 \cdot \left[1 + \alpha \cdot (\bar{\lambda} - 0.2) + \bar{\lambda}^2 \right] \quad (9)$$

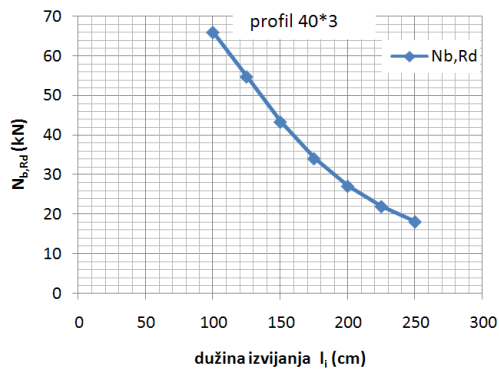
krive izvijanja i parametri za proračun su dati u EN 1993 1-1 [1].

2.2 PRIMER ODREĐIVANJA PRORAČUNSKE NOSIVOSTI ŠTAPOVA ISPUNE

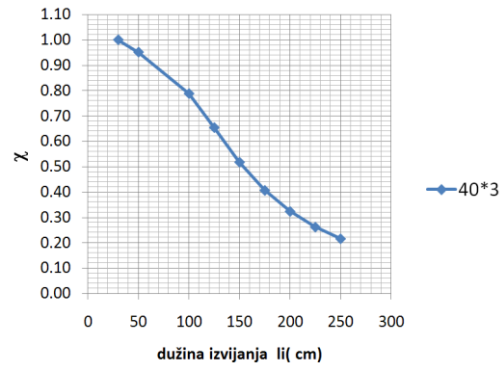
2.2.1 Proračun granične sile u štapu ispune sa uticajem izvijanja - $N_{b,Rd}$

Analiziran je štap ispune od cevastog profila $\phi 40 \times 3$. Proračunska nosivost $N_{b,Rd}$ prema (7-9) zavisi samo od vrste profila i koeficijenta izvijanja, a koji zavisi samo od vitkosti profila, odnosno od njegove dužine izvijanja. Na osnovu toga zaključujemo da se za svaki profil može konstruisati dijagram zavisnosti granične nosivosti $N_{b,Rd}$ i dužine izvijanja profila u konstrukciji. Na slici 5, date su vrednosti proračunske nosivosti $N_{b,Rd}$ profila $\phi 40 \times 3$ za dužine izvijanja 100, 125, 150, 175, 200, 225 i 250 cm i prikazane na dijagramu.

Takođe, može se za određeni profil odrediti dijagram zavisnosti dužine izvijanja i koeficijenta χ . Za analizirani profil $\phi 40 \times 3$ ta zavisnost je data na slici 6.



Slika 5- Zavisnost l_i i $N_{b,Rd}$



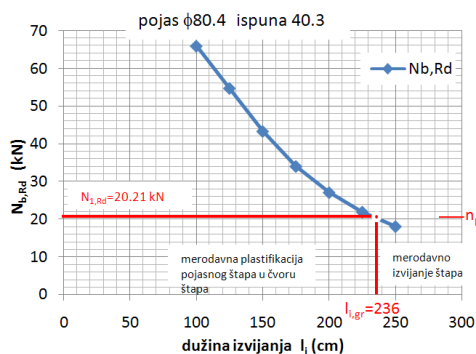
Slika 6- Zavisnost l_i i χ

2.2.2 Proračun granicne sile u štapu ispune za plastifikaciju pojasnog štapa – $N_{1,Rd}$

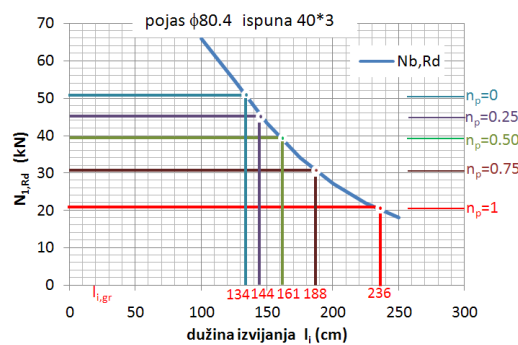
Izrazi potrebni za proračun granicne nosivosti dati su od (1) do (6). Vrednost granicne nosivosti profila, pored određenih konstantnih geometrijskih i fizičko mehaničkih parametara, zavisi od dva koeficijenta k_g i k_p i nagibnog ugla dijagonale. Usvojen je nagibni ugao od $\theta=45^\circ$. Koeficijent k_g dat u (2) je zavistan od razmaka profila (g) i dimenzija pojasne cevi d_0 i t_0 . Razmak između profila je uzet tako da je ekscentricitet veze u čvoru jednak nuli. Takvi, centrisani čvorovi se i preporučuju prilikom projektovanja rešetkastih konstrukcija. Na osnovu tako usvojene vrednosti za g i ugao θ dobija se konstantna vrednost koeficijenta k_g . Promena vrednosti granicne sile $N_{1,Rd}$ sada samo zavisi od koeficijenta k_p . Koeficijent k_p zavisi od vrednosti napona u pojasnom štapu. Ako uzmemo granicni slučaj, kada je $n_p=1.0$, a to se dešava pri potpunom iskorišćenju napona u pojasnom štapu, dobijamo najmanju vrednost granicne nosivosti $N_{1,Rd}$ u slučaju plastifikacije pojasnog štapa.

2.2.3 Zajedničko razmatranje kriterijuma za određivanje granicne nosivosti

Za pojasni štap od profila $\phi 80 \times 4$ određena je nosivost $N_{1,Rd}$ za vrednost $n_p=1$ i naneta na dijagram prikazan na slici 6. Sa dijagrama se vidi gde je granicna dužina izvijanjanja $l_{i,gr}$. Granicna dužina izvijanjanja je vrednost dužine izvijanjanja pri kojoj se istovremeno događa granicna nosivost po oba uslova, i izvijanjanjem štapa i plastifikacijom čvora. Ako je dužina izvijanjanja štapa manja od granicne dužine izvijanjanja merodavna je plastifikacija čvora rešetke, ako je dužina izvijanjanja štapa veća od granicne dužine izvijanjanja merodavno je izvijanjanje štapa. To je prikazano na slici 7.



Slika 7- Vrednost $l_{i,gr}$ za $n_p=1$



Slika 8- Vrednost $l_{i,gr}$ za $n_p=1$

Na slici 8 date su vrednosti granične dužine izvijanja $l_{i,gr}$ za vrednosti koeficijenta $n_p=0$, $n_p=0.25$, $n_p=0.5$, $n_p=0.75$, $n_p=1$. Sa smanjenjem koeficijenta n_p povećava se proračunska nosivost čvora, a smanjuje vrednost granične dužine izvijanja, odnosno povećava se uticaj izvijanja štapa na proračunsku nosivost. Na osnovu dijagrama možemo zaključiti da se poznavajući koeficijent n_p može odrediti granična dužina izvijanja $l_{i,gr}$ i ustanoviti kako se dostiže granična nosivost, izvijanjem štapa ili plastifikacijom čvora.

2.2.4 Proračun primenom koeficijenta redukcije napona χ

Koeficijent redukcije napona kod izvijanja štapa označimo sa χ_b (indeks b nam govori da je ovo koeficijent dobijen preko proračunske nosivosti za izvijanje štapa). Oba koeficijenta χ_1 , koji se odnosi na plastifikaciju čvora i χ_b , koji se odnosi na izvijanje daju odnos proračunske nosivosti prema maksimalnoj nosivosti preseka štapa.

Šta se dešava kada su koeficijenti redukcije isti $\chi_1=\chi_b$. Tada je $N_{1,Rd} = N_{b,Rd}$. Odnosno, merodavna proračunska nosivost je ista i za plastifikaciju čvora i za izvijanje štapa. Sa tim je definisana, kao što smo prethodno naglasili i granična dužina izvijanja $l_{i,gr}$. Kako prikazujemo sve na jednom dijagramu pišaćemo χ bez indeksa.

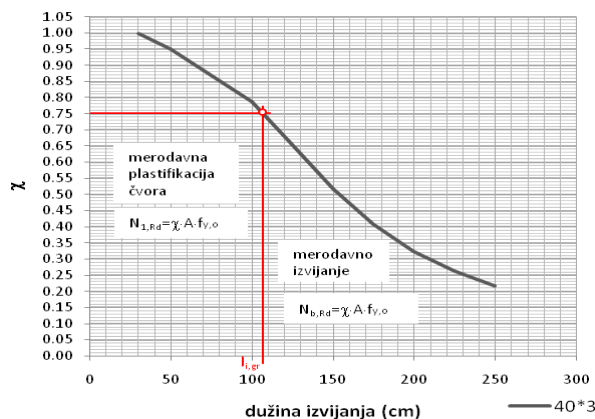
Ako je na primer $\chi_1=0.75$, nanošenjem na dijagram sa slike 6, dobija se granična dužina od 108 cm. Za manje dužine štapa od granične dužine ($l_i < l_{i,gr}$) za određivanje proračunske nosivosti merodavna je plastifikacija čvora, a za veće dužine od granične dužine ($l_i > l_{i,gr}$) merodavno je izvijanje. To je prikazano na slici 9.

Ako je dužina štapa ispune kraća od 108 cm tada je merodavna plastifikacija čvora i vrednost granične proračunske nosivosti štapa ispune je

$$N_{1,Rd} = \chi \cdot N_{c,Rd} = \chi \cdot \frac{A \cdot f_y}{\gamma_{M1}} = 0.75 \cdot \frac{3.49 \cdot 24}{1} = 62.82 \text{ kN} \quad (10)$$

i ta sila mora biti veća od računске sile u štapa ispune.

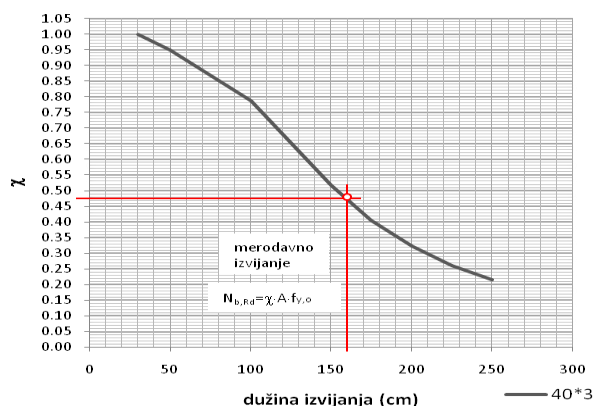
U slučaju da je dužina štapa ispune veća od 108 cm tada je merodavno izvijanje štapa i vrednost granične proračunske nosivosti štapa ispune je $N_{b,Rd}$ prema (7).



Slika 9- Određivanje granične dužine $l_{i,gr}$ za $\chi_l=0.75$

Na primer, dužina izvijanja štapa ispune je 160 cm, a to je veće od 108 cm. Merodavno je izvijanje. Povlačimo liniju sa $l_i=160$ cm na dijagram, a zatim za tu vrednost očitamo vrednost parametra χ i na osnovu njega izračunamo vrednost proračunske nosivosti na izvijanje. Čitamo sa dijagrama $\chi=0.46$ pa je proračunska nosivost

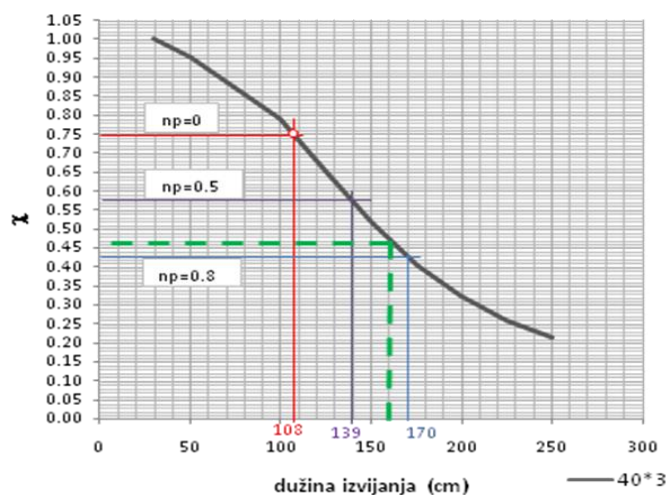
$$N_{b,Rd} = \chi \cdot N_{c,Rd} = \chi \cdot \frac{A \cdot f_y}{\gamma_{M1}} = 0.46 \cdot \frac{3.49 \cdot 24}{1} = 38.53 \text{ kN} \quad (11)$$



Slika 10- Određivanje koeficijenta redukcije χ za $l_i=160$ cm

Za vrednost $n_p=0.5$ sledi da je $\chi=0.58$ i dobijamo da je druga vrednost granične dužine izvijanja. Ali i dalje je l_{gr} manje od dužine izvijanja štapa koja je usvojena (160 cm) što znači da je merodavno izvijanje štapa, gde bi opet bilo $\chi=0.46$.

Za vrednost $n_p=0.8$ sledi da je $\chi=0.43$ i dobijamo da je granična dužina izvijanja $l_{gr}=170$. Sada je l_{gr} veće od 160 cm i merodavna je plastifikacija čvora odnosno sila $N_{1,Rd}=0.43 \cdot A \cdot f_y / \gamma_{M1}$. To je prikazano na slici 11.



Slika 11- Vrednosti koficijenta χ i dužine $l_{i,gr}$ za $n_p=0$, $n_p=0.5$ i $n_p=0.8$

Ako izračunamo vrednosti χ za razne vrednosti n_p , za sve štapove koji mogu da se kombinuju sa štapom ispune 40x3 a čiji je zid debljine 4 mm dobijamo sledeću tabelu 1.

Tabela 1. Vrednost χ u odnosu na razne vrednosti n_p

40*3	χ										
n_p	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1
160*4	0.41	0.40	0.38	0.36	0.34	0.32	0.29	0.26	0.23	0.20	0.19
120*4	0.58	0.56	0.54	0.51	0.48	0.45	0.41	0.37	0.33	0.28	0.23
100*4	0.64	0.62	0.59	0.56	0.53	0.49	0.45	0.41	0.36	0.31	0.25
80*4	0.75	0.73	0.70	0.67	0.63	0.58	0.54	0.48	0.43	0.37	0.30
60*4	1.00	0.96	0.92	0.88	0.83	0.77	0.71	0.64	0.56	0.48	0.40

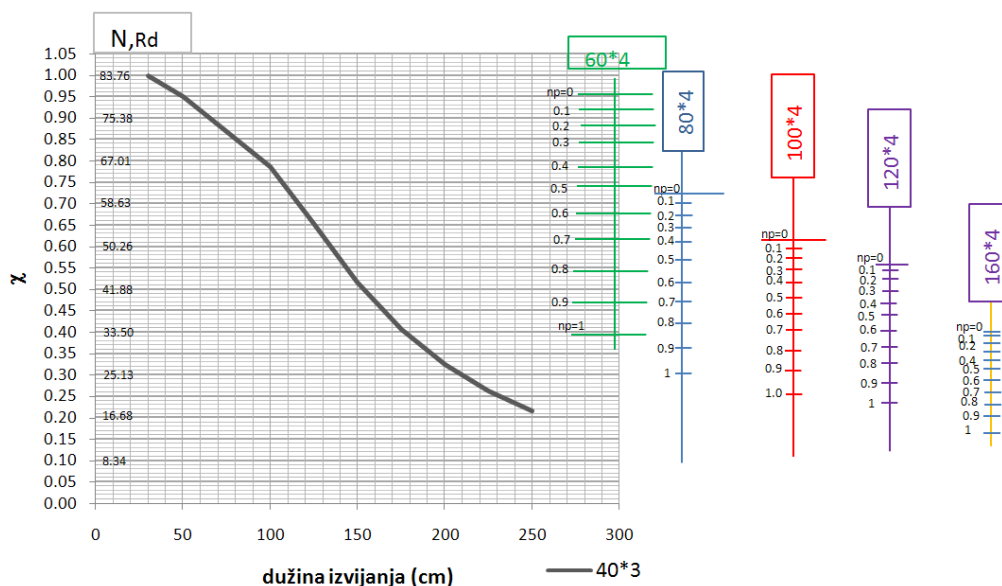
Ako dobijene vrednosti uskladimo i nanesimo na dijagram u obliku nonograma dobijamo podlogu sa koje direktno možemo da proverimo graničnu nosivost štapa ispune po oba kriterijuma kao što je prikazano na slici 12.

Primer:

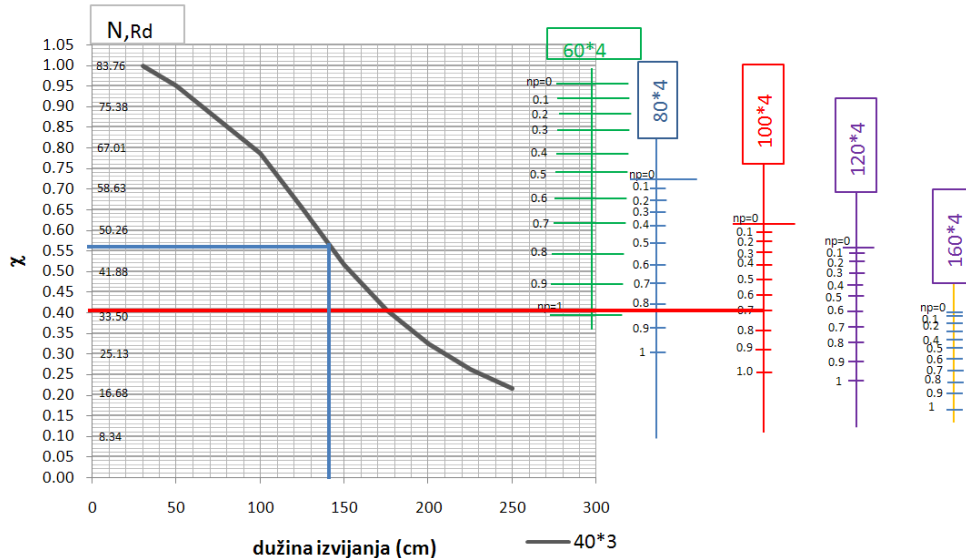
Pojas rešetke je od profila $\phi 100 \times 4$ a štapovi ispune su $\phi 40 \times 3$ mm. Uglovi su $\theta_1 = \theta_2 = 45^\circ$. Ekscentricitet je $e = 0$. Sile u dijagonalnim štapovima su $N_{1,Ed} = 30$ kN i $N_{2,Ed} = -30$ kN. Koficijent $n_p = 0.7$. Dužina pritisnutog štapa ispune je 140 cm. Izvršićemo određivanje proračunske nosivosti pritisnutog štapa ispune pomoću dijagrama na slici 13.

Rešenje:

Za $n_p = 0.7$ i pojasni profil $\phi 100 \times 4$ na dijagramu povlačimo horizontalnu liniju i očitavamo vrednost bezdimenzionalnog koficijenta χ . Vrednost je $\chi = 0.40$. Za dužinu izvijanja $l_1 = 140$ cm, povlačimo liniju do krive a zatim horizontalno i odredimo vrednost koficijenta χ . U ovom slučaju $\chi = 0.55$. Znači, merodavna je plastifikacija pojasa, granična sila je oko 33.50 kN, što je veće od proračunske nosivosti $N_{1,Ed} = 30$ kN. Postupak određivanja granične nosivosti štapa ispune prikazan je na slici 13.



Slika 12- Određivanje granične nosivosti N_{Rd} za razne kombinacije profila



Slika 13- Prikaz postupka određivanja granične nosivosti štapa ispune

Koeficijent redukcije napona za veći odnos prečnika pojasnog štapa i štapa ispune je konstantno mali i presek nije iskorišćen pri malim dužinama štapova. Potpuno je neracionalano usvajati takav odnos prečnika cevi. Racionalan odnos štapova pojasa i ispune je u slučaju kada su vrednosti za koeficijent redukcije χ približne.

3 ZAKLJUČCI

Korišćenjem dijagrama i tabela odnosa koeficijenta redukcije napona χ i dužine izvijanja za pojedine štapove ispune može se na vrlo brz način odrediti merodavni koeficijent redukcije χ i samim tim i proračunska nosivost štapa N_{Rd} , kao i na koji način je ona dostignuta, plastifikacijom čvora ili izvijanjem.

Postojanje ovakvih dijagrama nam daje pregledniju sliku o kombinacijama štapova u rešetkastim konstrukcijama. Pavilnim izborom štapova (a to je kada su koeficijenti redukcije po oba kriterijuma slične vrednosti) racionalise se konstrukcija.

Projektantima se nudi lak postupak za projektovanje, čitajući sa dijagrama koeficijente za čije određivanje bi bilo potrebno dosta vremena. Samim tim i mogućnost računskih grešaka je svedena na minimum.

Dijagrami se mogu koristiti i kada je potrebno proračunati štap samo na izvijanje, jednostavno čitajući za određenu dužinu štapa koeficijent redukcije napna χ .

Za vrednost koeficijenta redukcije napona $\chi=1$ dobija se granična nosivost štapa na zatezanje.

Nedostatak je što se dijagrami i tabele odnose na jedan profil štapa, pa je potrebno napraviti dijagrame za sve štapove iz kataloga i za njihove veze sa raznim pojasnim štapovima. Ovo bi se jednom uradilo i uvek bi služilo za proračun. Olakšanje su preporuke za izbor kombinacija debljina zida i prečnika cevi, što smanjuje broj kombinacija.

Analizirani dijagrami se odnose na ugao $\theta=45^\circ$ i $e=0$, za druge uglove i ekscentricitete veze u čvoru potrebno je uraditi posebne dijagrame ili uvesti faktor promene koeficijenta redukcije χ u odnosu na već definisane dijagrame.

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NUMERIČKA ANALIZA NOSIVOSTI NEUNIFORMNIH PRITISNUTIH ELEMENATA NA FLEKSIONO IZVIJANJE

Rezime:

Rad prikazuje parametarsku linearno-elastičnu analizu izvijanja idealizovanog neuniformnog elementa i nelinearnu analizu izvijanja istog elementa sa zadatim ekvivalentnim geometrijskim imperfekcijama urađenu u programu Abaqus. Analizom su obuhvaćeni zglobno oslonjeni elementi sa stepenastom promenom poprečnog preseka kod kojih je variran odnos krutosti gornjeg i donjeg segmenta i odnos aksijalnih sila na vrhu, odnosno mestu promene preseka. Cilj rada je da se na osnovu relevantne i pouzdane baze podataka, definišu aproksimativni nomogrami za određivanje koeficijenata dužine izvijanja neuniformnih elemenata i utvrdi pouzdanost opšte metode date u EC3 za proračun fleksionog izvijanja.

Ključne reči: neuniformni element, izvijanje, FEA, opšta metoda, Evrokod 3

NUMERICAL ANALYSIS OF FLEXURAL BUCKLING RESISTANCE OF COMPRESSED NON-UNIFORM MEMBERS

Summary:

This paper presents parametric linear-elastic buckling analysis of idealized non-uniform members and non-linear analysis of the same member with the given geometric imperfections created in a software Abaqus. The analysis included hinged ends members with stepped change in cross-section in which was varied the ratio of rigidity and the ratio of axial force of the top and bottom segment. Purpose of this paper is to define the approximate nomograms for determining coefficients of buckling of individual non-uniform member and determine the reliability of the general method for flexural buckling resistance according to EC3.

Key words: non-uniform member, buckling, FEA, general method, Eurocode 3

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1 UVOD

Opšta metoda proračuna nosivosti pritisnutih elemenata na fleksiono izvijanje, koja je data u SRPS EN 1993-1-1 [1], se zasniva na analitičkoj zavisnosti između bezdimenzionalnog koeficijenta χ , odnosno odnosa proračunske nosivosti elementa na fleksiono izvijanje i proračunske nosivosti njegovog poprečnog preseka $N_{b,Rd}/A_{f_y}$ sa jedne strane i bezdimenzionalne relativne vitkosti $\bar{\lambda}$, odnosno odnosa nosivosti poprečnog preseka i elastične kritične sile izvijanja sa druge strane $\sqrt{A_{f_y}/N_{cr}}$. Ova zavisnost, u osnovi zasnovana na Ayrton-Perijevoj funkciji, predstavljena je familijom pet krivih izvijanja kojima su obuhvaćene početne nesavršenosti realnih konstruktivnih elemenata. Kod elemenata sa neuniformnom promenom poprečnog preseka i/ili aksijalne sile pritiska, ovoj metodi prethodi postupak određivanja elastične kritične sile izvijanja, koji nije propisan standardom. Tačnost u određivanju vrednosti kritične sile izvijanja zavisi od pouzdanosti interpretacije geometrijskih karakteristika po dužini elementa, graničnih uslova oslanjanja i postojanja eventualnih ekscentriciteta u položaju sistemnih osa susednih segmenta elemenata sa stepenastom promenom preseka. Aktuelni standard SRPS EN 1993-1-1 [1] ne daje preporuke koje odlikavaju matematičko rešenje kritične sile izvijanja, što stvara poteškoće inženjerima u praksi. U opštem slučaju, ona se može odrediti koristeći teorijske analize koje zahtevaju nalaženje rešenja nelinearne diferencijalne jednačine izvijanja ili primenom neke od metoda naprednih numeričkih analiza. U jednostavnijim slučajevima mogu se koristiti uprošćeni postupci proračuna u kojima se vrednost elastične kritične sile izvijanja određuje jednim od osnovnih Ojlerovih izraza za izvijanje uniformnih elemenata, ali sa ekvivalentnim momentom inercije ili ekvivalentnom dužinom izvijanja kojima se uzima u obzir geometrijska neuniformnost analiziranog elementa.

Ovaj rad prikazuje rezultate parametarske numeričke analize neuniformnih, obostrano zglobno oslonjenih elemenata koja je sprovedena u softveru Abaqus [2]. Neuniformnost se ogledala u promeni poprečnog preseka kroz dva segmenta i stepenastoj promeni aksijalne sile pritiska. Variiran je odnos poprečnih preseka gornjeg i donjeg segmenta i odnos aksijalnih sila na vrhu i mestu promene poprečnog preseka elementa. Dužina elemenata je 10 m, visina gornjeg segmenta 4 m, visina donjeg segmenta 6 m. Poprečni presek donjeg segmenta je HEA 300 dok je poprečni presek gornjeg segmenta variran u opsegu: HEA 160, HEA 180, HEA 200, HEA 220 i HEA 240. Analiziran opseg odnosa reaktivnih sila na krajevima FE modela, $P_1/(P_1+P_2)$ je od 0,05 do 0,50 sa korakom od 0,05. Ukupan broj analiziranih FE modela je 50. Svrha ovog rada je da se na osnovu zadatih parametara i rezultata linearne analize izvijanja utvrdi opseg koeficijeneta dužine izvijanja oko jače ose inercije pojedinačnih segmenata i razviju nomogrami za njihovo aproksimativno određivanje. Takođe, da se proceni tačnost prediktivnih računskih vrednosti fleksione stabilnosti prema opštoj metodi proračuna [1] kroz njihovo poređenje sa rezultatima statičke nelinearne analize.

2 OPIS NUMERIČKIH MODELA

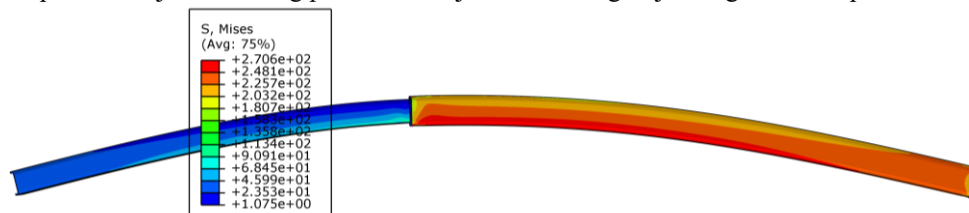
Dve različite vrste analiza su urađene za svaki numerički model: linearna elastična analiza i nelinearna statička analiza fleksionog izvijanja oko jače ose inercije, koristeći Riksov solver. Linearna elastična analiza, zasnovana na problemu bifurkacione stabilnosti, daje procenu

elastične kritične sile izvijanja idealno pravog elemenata bez strukturnih imperfekcija. Odgovor elementa je praćen zanemarljivo malim bočnim deformacijama, a kada sila dostigne graničnu (kritičnu) vrednost dolazi do naglog izvijanja koje je praćeno velikim deformacijama. Kako je modul elastičnosti u linearno-elastičnoj oblasti konstantan, vrednost kritične sile izvijanja isključivo zavisi od vitkosti elemenata i graničnih uslova na krajevima ili duž elementa. Sa druge strane, materijalna i geometrijska nelinearnost kao i strukturne imperfekcije realnih konstruktivnih elemenata ograničavaju njihovu sposobnost u dostizanju teoretske elastične sile izvijanja. U takvim slučajevima linearna elastična analiza se koristi za procenu osnovnih oblika izvijanja, a njeni rezultati za interpretaciju početnih geometrijskih imperfekcija u kasnijim fazama analize stabilitetnih problema realnih elemenata. U Abaqusu postoji nekoliko različitih numeričkih metoda za rešavanje nelinearnih statičkih problema. Metoda Riksa ili metoda kružnog luka je osnovna i najviše korišćena metoda u analizi ponašanja pritisnutih elemenata za različite vrste izvijanja. Uslov za njenu primenu jeste da kriva sila-pomeranje bude glatka i bez grananja. Tačnost procene vrednosti granične sile značajno zavisi od veličine početnih geometrijskih imperfekcija realnog elementa. Ukoliko su ova odstupanja idealizovana, odnosno zanemarljivo mala, početni, uzlazni deo krive je "strm" sa naglim prelaskom u nestabilno ravnotežno kada primena Riksove metode može dovesti do divergencije rešenja.

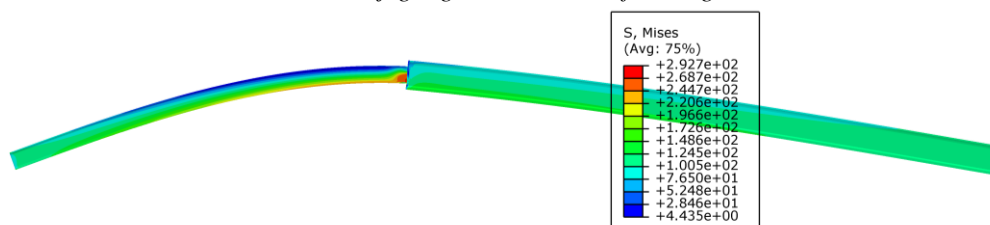
FE modeli su realizovani sa „*solid*“ elementima i mrežom „*wedge*“ konačnih elemenata dimenzija 15 mm. U težištima poprečnih preseka, na oba kraja FE modela, definisane su referentne tačke koje su povezane sa površinama krajnjih poprečnih preseka preko opcije „*kinematic coupling type*“. U obe referentne tačke dopuštena je rotacija oko glavnih osa poprečnog preseka, sprečena torziona obrtanja oko podužne ose i sprečena translatorska pomeranja. Koncentrisane sile pritiska u pravcu podužne ose FE modela su zadate u težištu krajnjeg gornjeg poprečnog preseka, odnosno na mestu stepenaste promene poprečnog preseka, respektivno. Nominalne vrednosti krive napon-dilatacija koja je dobijena eksperimentalnim ispitivanjem epruveta uzetih iz finalnog vruće-valjanog profila HEB 260 sa kvalitetom čeličnog materijala S275 [3] usvojena je za opisivanje mehaničkih svojstava materijala svih FE modela. Eksperimentalne vrednosti su transformisane u stvarne vrednosti napon-dilatacija. Elastični domen naprezanja određen je modulom elastičnosti $E = 210000 \text{ N/mm}^2$ i Poasonovim koeficijentom $\nu = 0,3$. Rezidualni naponi nastali kao rezultat proizvodnog procesa nisu modelirani. Evropski standard SRPS EN 1993-1-5 [3] u Prilogu C sugeriše da se geometrijske imperfekcije mogu bazirati na obliku kritičnog (prvog) oblika izvijanja i dopušta redukciju geometrijskih tolerancija fabričke izrade u iznosu od 80% u interpretaciji početnih nesavršenosti. Međutim, obzirom da rezidualni naponi nisu uključeni u analizi statičkog odgovora elementa, za amplitudu početne imperfekcije usvojena je dopuštena proizvodna tolerancija koja uključuje odstupanje ose elementa od vertikalnosti u punom iznosu od $L/750$, u skladu sa standardom SRPS EN 1090-2 [5]. Normalizovane vrednosti koordinata deformisanog modela za kritični oblik izvijanja su skalirane i učitane su naredbom „*Imperfection*“ u opciji „*Edit keyword*“ za svaki pojedinačni model. Uticaj lokalnih imperfekcija poprečnog preseka je zanemaren u okviru analize.

3 PRIKAZ REZULTATA NUMERICKE ANALIZE

Slike 1 i 2 prikazuju raspodelu Misesovih napona pri graničnom stanju nosivosti analiziranog neuniformnog elementa usled fleksionog izvijanja oko jače ose inercije koji je određen plastifikacijom kritičnog preseka u donjem, odnosno gornjem segmentu, respektivno.



Slika 1 – Lom donjeg segmenta FE neuniformnog modela



Slika 2 – Lom gornjeg segmenta FE neuniformnog modela

Ključni rezultati elastične linearne i statičke nelinearne analize fleksionog izvijanja sumirani su redom u tabelama od 1 do 5, gde su: N_{cr} elastična kritična sila izvijanja; β koeficijent dužine izvijanja pojedinačnih segemenata, λ bezdimenzionalna relativna vitkost, $N_{b,FEA}$ numerička granična nosivost na fleksiono izvijanje oko jače ose preseka, $N_{b,EC}$ računaska granična nosivost na fleksiono izvijanje oko jače ose prema opštoj metodi [1].

Tabela 1 - Rezultati numeričke analize za odnos $I_2/I_1 = 0,09$

$I_2/I_1=0,09$		$P_2/(P_1+P_2)$									
		0,05	0,1	0,15	0,2	0,25	0,3	0,35	0,4	0,45	0,5
Donji segment HEA300	N_{cr} (kN)	1996	1828	1686	1563	1456	1363	1280	1207	1141	1082
	β	2,30	2,40	2,50	2,59	2,69	2,78	2,87	2,95	3,04	3,12
	λ	1,24	1,29	1,35	1,40	1,45	1,50	1,54	1,59	1,64	1,68
	Lom	-	-	-	-	-	-	-	-	-	-
	$N_{b,FEA}$ (kN)	1583	1444	1324	1221	1135	1046	976	917	870	823
	$N_{b,EC}$ (kN)	1400	1315	1238	1168	1105	1048	997	949	906	866
	$N_{b,FEA}/N_{b,EC}$	1,13	1,10	1,07	1,05	1,03	1,00	0,98	0,97	0,96	0,95
Gornji segment HEA160	N_{cr} (kN)	99,8	182,8	252,9	312,6	364,1	408,8	448,1	482,7	513,5	541,1
	β	4,66	3,44	2,92	2,63	2,44	2,30	2,20	2,12	2,05	2,00
	λ	3,24	2,39	2,04	1,83	1,70	1,60	1,53	1,47	1,43	1,39
	Lom	DA	DA	DA	DA	DA	DA	DA	DA	DA	DA
	$N_{b,FEA}$ (kN)	79	144	199	244	284	314	342	367	391	411
	$N_{b,EC}$ (kN)	90	159	213	256	292	322	348	369	388	404
	$N_{b,FEA}/N_{b,EC}$	0,88	0,91	0,93	0,95	0,97	0,97	0,98	0,99	1,01	1,02

Tabela 2 - Rezultati numeričke analize za odnos $I_2/I_1 = 0,14$

$I_2/I_1=0,14$		$P_2/(P_1+P_2)$									
		0,05	0,1	0,15	0,2	0,25	0,3	0,35	0,4	0,45	0,5
Donji segment HEA300	N_{cr} (kN)	2760	2541	2353	2189	2045	1919	1807	1706	1616	1535
	β	1,95	2,03	2,11	2,19	2,27	2,34	2,41	2,48	2,55	2,62
	λ	1,05	1,10	1,14	1,18	1,22	1,26	1,30	1,34	1,37	1,41
	Lom	DA	-	-	-	-	-	-	-	-	-
	$N_{b,FEA}$ (kN)	2146	1968	1805	1649	1527	1424	1325	1240	1185	1088
	$N_{b,EC}$ (kN)	1723	1641	1563	1491	1424	1361	1303	1249	1199	1152
	$N_{b,FEA}/N_{b,EC}$	1,25	1,20	1,15	1,11	1,07	1,05	1,02	0,99	0,99	0,94
Gornji segment HEA180	N_{cr} (kN)	138	254	353	438	511	576	632	682	727	767
	β	4,85	3,58	3,04	2,73	2,52	2,38	2,27	2,18	2,11	2,06
	λ	2,98	2,19	1,86	1,67	1,55	1,46	1,39	1,34	1,30	1,26
	Lom	-	DA	DA	DA	DA	DA	DA	DA	DA	DA
	$N_{b,FEA}$ (kN)	107	197	271	330	382	427	464	496	515	544
	$N_{b,EC}$ (kN)	123	217	291	350	398	438	472	500	524	545
	$N_{b,FEA}/N_{b,EC}$	0,87	0,91	0,93	0,94	0,96	0,97	0,98	0,99	0,98	1,00

Tabela 3 - Rezultati numeričke analize za odnos $I_2/I_1 = 0,20$

$I_2/I_1=0,20$		$P_2/(P_1+P_2)$									
		0,05	0,1	0,15	0,2	0,25	0,3	0,35	0,4	0,45	0,5
Donji segment HEA300	N_{cr} (kN)	3670	3404	3170	2964	2782	2619	2473	2342	2224	2117
	β	1,69	1,76	1,82	1,88	1,94	2,00	2,06	2,12	2,17	2,23
	λ	0,91	0,95	1,01	1,01	1,05	1,08	1,11	1,14	1,17	1,20
	Lom	DA	DA	DA	-	-	-	-	-	-	-
	$N_{b,FEA}$ (kN)	2557	2498	2363	2187	2029	1904	1779	1639	1558	1463
	$N_{b,EC}$ (kN)	1994	1925	1858	1793	1731	1671	1614	1559	1507	1458
	$N_{b,FEA}/N_{b,EC}$	1,28	1,30	1,27	1,22	1,17	1,14	1,10	1,05	1,03	1,00
Gornji segment HEA200	N_{cr} (kN)	184	340	476	593	695	786	866	937	1001	1058
	β	5,10	3,75	3,17	2,84	2,62	2,47	2,35	2,26	2,19	2,13
	λ	2,81	2,07	1,75	1,57	1,45	1,36	1,30	1,25	1,20	1,17
	Lom	-	-	-	DA	DA	DA	DA	DA	DA	DA
	$N_{b,FEA}$ (kN)	128	250	354	437	507	571	623	655	701	732
	$N_{b,EC}$ (kN)	163	287	385	464	528	580	623	660	691	717
	$N_{b,FEA}/N_{b,EC}$	0,78	0,87	0,92	0,94	0,96	0,99	1,00	0,99	1,02	1,02

Tabela 4 - Rezultati numeričke analize za odnos $I_2/I_1 = 0,30$

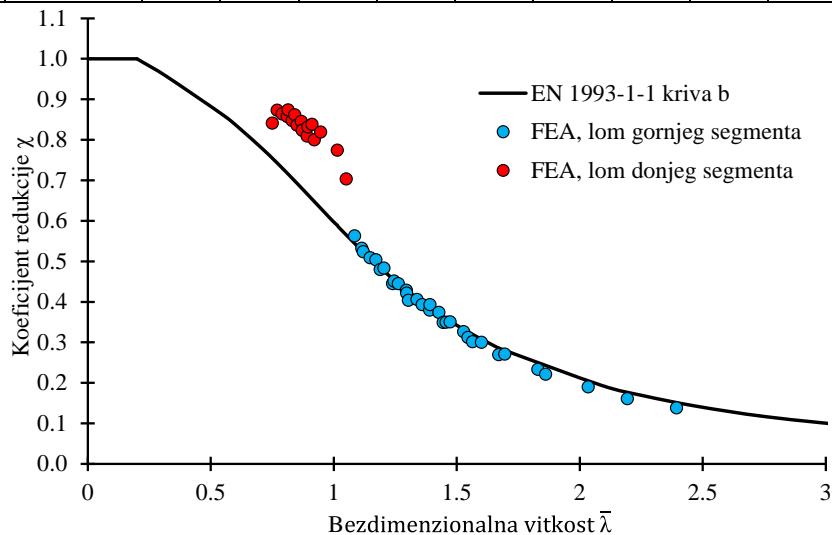
$I_2/I_1=0,30$		$P_2/(P_1+P_2)$									
		0,05	0,1	0,15	0,2	0,25	0,3	0,35	0,4	0,45	0,5
Donji segment HEA300	N_{cr} (kN)	4594	4304	4043	3807	3595	3403	3229	3070	2926	2793
	β	1,51	1,56	1,61	1,66	1,71	1,76	1,80	1,85	1,90	1,94
	λ	0,81	0,84	0,87	0,90	0,92	0,95	0,97	1,00	1,02	1,05
	Lom	DA	DA	DA	DA	DA	-	-	-	-	-
	$N_{b,FEA}$ (kN)	2665	2628	2578	2537	2441	2337	2205	2080	1963	1849
	$N_{b,EC}$ (kN)	2182	2130	2078	2027	1976	1925	1876	1828	1781	1735
	$N_{b,FEA}/N_{b,EC}$	1,22	1,23	1,24	1,25	1,24	1,21	1,18	1,14	1,10	1,07

Tabela 4 - Nastavak

$I_2/I_1=0,30$		$P_2/(P_1+P_2)$									
		0,05	0,1	0,15	0,2	0,25	0,3	0,35	0,4	0,45	0,5
Gornji segment HEA220	N_{cr} (kN)	230	430	606	761	899	1021	1130	1228	1317	1397
	β	5,52	4,04	3,40	3,03	2,79	2,62	2,49	2,39	2,31	2,24
	λ	2,75	2,01	1,69	1,51	1,39	1,30	1,24	1,19	1,15	1,11
	Lom	-	-	-	-	-	DA	DA	DA	DA	DA
	$N_{b,FEA}$ (kN)	133	263	387	508	610	701	772	832	884	924
	$N_{b,EC}$ (kN)	203	361	487	588	670	738	794	841	880	914
	$N_{b,FEA}/N_{b,EC}$	0,65	0,73	0,79	0,86	0,91	0,95	0,97	0,99	1,00	1,01

Tabela 5 - Rezultati numeričke analize za odnos $I_2/I_1 = 0,42$

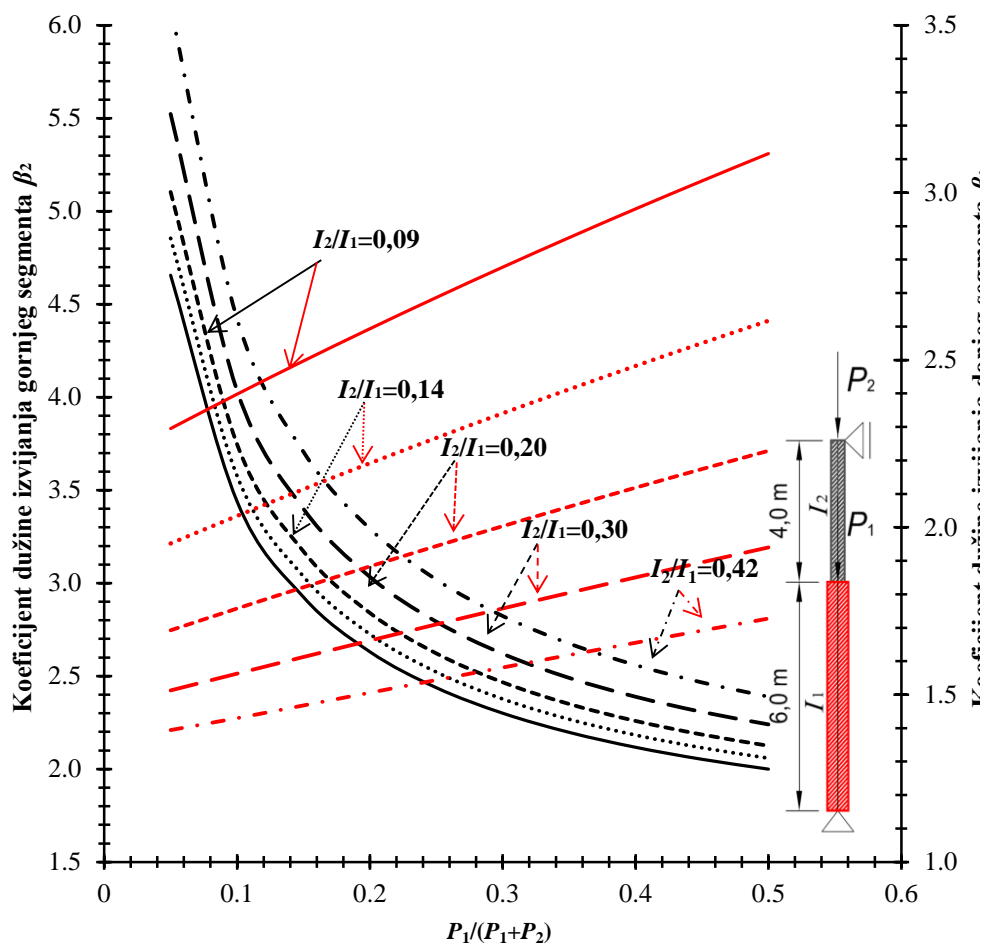
$I_2/I_1=0,42$		$P_2/(P_1+P_2)$									
		0,05	0,1	0,15	0,2	0,25	0,3	0,35	0,4	0,45	0,5
Donji segment HEA300	N_{cr} (kN)	5408	5142	4879	4636	4412	4205	4014	3837	3674	3523
	β	1,39	1,43	1,47	1,51	1,54	1,58	1,62	1,66	1,69	1,73
	λ	0,75	0,77	0,79	0,81	0,83	0,85	0,87	0,89	0,91	0,93
	Lom	DA	DA	DA	DA	DA	DA	DA	DA	-	-
	$N_{b,FEA}$ (kN)	2566	2663	2635	2614	2585	2551	2511	2468	2413	2334
	$N_{b,EC}$ (kN)	2301	2266	2228	2189	2150	2111	2072	2034	1995	1957
		$N_{b,FEA}/N_{b,EC}$	1,12	1,18	1,18	1,19	1,20	1,21	1,21	1,21	1,21
Gornji segment HEA240	N_{cr} (kN)	270	514	732	927	1103	1261	1405	1535	1653	1762
	β	6,10	4,42	3,71	3,29	3,02	2,82	2,67	2,56	2,47	2,39
	λ	2,77	2,01	1,68	1,50	1,37	1,28	1,21	1,16	1,12	1,08
	Lom	-	-	-	-	-	-	-	-	DA	DA
	$N_{b,FEA}$ (kN)	128	266	395	523	646	765	879	987	1086	1167
	$N_{b,EC}$ (kN)	240	431	586	713	817	903	975	1035	1085	1129
	$N_{b,FEA}/N_{b,EC}$	0,54	0,62	0,67	0,73	0,79	0,85	0,90	0,95	1,00	1,03



Slika 3 – Poređenje numeričkih i računskih vrednosti nosivosti na fleksiono izvijanje

Numeričke vrednosti graničnih sila fleksionog izvijanja oko jače ose inercije, $N_{b,FEA}$ normalizovane su sa vrednošću sile pri kojoj dolazi do plastifikacije poprečnog preseka Af_y za svaki pojedinačni segmenat i upoređene sa krivom izvijanja b na slici 3. Kriva izvijanja b reprezentuje računске vrednosti graničnih sila fleksionog izvijanja prema opštoj metodi proračuna u EN 1993-1-1 [1] koje su normalizovane istom vrednošću sile Af_y .

Na osnovu zadatih ulaznih parametara koji se ogledaju u odnosu momenata inercije gornjeg i donjeg segmenta I_2/I_1 i odnosa sila na krajevima analiziranih FE modela $P_1/(P_1+P_2)$ sa jedne strane i dobijenih vrednosti kritičnih sila izvijanja N_{cr} sa druge strane definisani su nomogrami putem kojih se mogu odrediti približne vrednosti koeficijenta izvijanja β pojedinačnih segmenata. Nomogrami su prezentovani na slici 4 i važe u slučaju obostrano zglobno oslonjenih elemenata sa gornjim i donjim segmentom dužine 6 m odnosno 4 m.



Slika 4 – Koeficijent dužine izvijanja gornjeg i donjeg segmenta u funkciji odnosa momenta inercije I_2/I_1 i odnosa sila na krajevima elementa $P_1/(P_1+P_2)$

4 ZAKLJUČCI

Pregledom rezultata sprovedene numeričke analize koji su prezentovani u tabelama od 1 do 5 i grafički na slikama 3 i 4, mogu se izneti sledeći zaključci.

Vrednost koeficijenta dužine izvijanja za gornji segment elementa je u opsegu od 1,4 do 3,1 i ima veću vrednost pri većim vrednostima odnosa sile u vrhu i sile na kontaktu segmenta. Vrednost koeficijenta se smanjuje povećanjem odnosa krutosti gornjeg i donjeg segmenta.

Vrednost koeficijent dužine izvijanja za donji segment elementa je od 2 do 6,1 i opada sa povećanjem odnosa sile u vrhu i sile na spoju segmenata, a raste sa povećanjem odnosa krutosti gornjeg i donjeg segmenta.

Kada je krutost gornjeg segmenta značajno manja u odnosu na krutost donjeg segmenta (odnos $I_2/I_1=0,09$) do loma elemenata dolazi izvijanjem gornjeg segmenta Povećanjem odnosa krutosti gornjeg i donjeg segmenta do loma elemenata dolazi izvijanjem donjeg segmenta.

Opšta metoda proračuna u Evrokodu 3 sa visokom tačnošću daje predikciju granične nosivosti neuniformnog elementa na fleksiono izvijanje za slučaj loma gornjeg segmenta, sa odstupanjima u opsegu od +12% do -1%. U slučaju loma donjeg segmenta sa većim poprečnim presekom, odstupanja su znatno konzervativnija i nalaze se u opsegu od +12% do +30%.

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OJAČANJE DRVENIH NOSAČA PRIMENOM FRP ŠIPKI

Rezime:

S-38

Potreba za ojačanjem drvenih konstrukcija usled oštećenja ili promene namene objekata dovela je do razvoja savremenih metoda ojačanja i upotrebe novih materijala u tu svrhu. Poslednjih godina, upotreba polimera ojačanih vlaknima (FRP) u oblasti sanacija i ojačanja građevinskih konstrukcija omogućena je zahvaljujući povećanoj dostupnosti i sve nižoj ceni. U radu se razmatra mogućnost primene FRP šipki kao materijala za ojačanje drvenih nosača. Ojačanjem drveta kompozitima na bazi karbonskih, staklenih ili bazaltnih vlakana mogu se obezbediti bolje karakteristike, kao što su poboljšana nosivost, krutost i duktilnost.

Ključne reči: drveni nosači, ojačanje, FRP kompoziti, šipke

STRENGTHENING OF TIMBER BEAMS USING FRP BARS

Summary:

The growing need for the reinforcement of timber beams (required due to deterioration or damage to the material or change of use) has led to the development of new methods of reinforcement with modern materials. In the recent years the use of fibre reinforced polymers (FRP) as reinforcement materials for structures has been made possible thanks to the increased availability and lower costs. This paper presents FRP bars as products for strengthening timber structures. Strengthening timber with glass, carbon and basalt FRP can provide better features of timber beams, such as improved load capacity, rigidity and ductility.

Key words: timber structures, strengthening, FRP composites, FRP bars

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1 UVOD

Drvo kao jedan od najstarijih građevinskih materijala nalazi primenu i u savremenoj građevinskoj praksi zahvaljući tome što predstavlja prirodan, obnovljiv, biorazgradiv i estetski atraktivan materijal. Potreba za ojačanjem drvenih konstrukcija može nastati iz različitih razloga, kao što su mehanička oštećenja, destruktivni uticaji okruženja ili povećanje korisnog opterećenja. U ovom kontekstu, razvoj efikasnih metoda ojačanja je od velike važnosti.

Poslednjih godina, primena polimera ojačanih vlaknima (*Fiber Reinforced Polymer - FRP*) u oblasti sanacija i ojačanja građevinskih konstrukcija omogućena je zahvaljujući povećanoj dostupnosti i sve nižoj ceni. FRP materijali su grupa naprednih kompozita u okviru kojih su prisutna vlakna izraženih mehaničkih karakteristika (najčešće staklena ili karbonska) povezana izuzetno čvrstom, hemijski otpornom i trajnom sintetičkom smolom (kao matricom). Ovi kompozitni materijali su dostupni kao gotovi fabrički proizvodi najčešće u formi traka, tkanina ili šipki. Povezvanje FRP ojačanja za konstrukcijske elemente izvodi se uglavnom lepljenjem uz primenu odgovarajućih polimernih lepkova. FRP kompoziti su idealno ojačanje za drvene elemente zbog njihovih izuzetnih karakteristika, kao što su odlična mehanička svojstva, mala sopstvena težina, izuzetna trajnost, velika mogućnost oblikovanja i fleksibilnost.

U radu je predstavljena upotreba FRP šipki kao materijala za ojačanje drvenih nosača. Postavljanje FRP šipki unutar poprečnog preseka nosača ima nekoliko značajnih prednosti u odnosu na uobičajenu primenu FRP traka sa spoljne strane preseka, kao što su: veća sigurnost pri požaru, bolja estetska svojstva, efikasnija veza drvo-FRP (veća površina lepljenja) i smanjena mogućnost pojave delaminacije ojačanja.

2 FRP KOMPOZIT

Zavisno od zahtevanih fizičkih i mehaničkih karakteristika, kao i od ekonomskih razmatranja, FRP kompoziti mogu biti sačinjeni od različitih tipova vlakana i polimernih matrica, i prilagođeni da obezbede potrebnu nosivost i krutost u željenim pravcima. U okviru kompozita vlakna i matrica zadržavaju svoj fizički i hemijski identitet, a ipak zajedno proizvode određena svojstva koja ne mogu biti dostignuta kada deluju samostalno.

2.1 VLAKNA

Izbor vlakana u mnogome utiče na karakteristike kompozita. Za primenu u građevinarstvu dominiraju tri tipa veštačkih vlakana: karbonska, staklena i aramidna, a u poslednje vreme primenu nalaze i prirodna bazaltna vlakna. Vlakna imaju različite karakteristike, uključujući i cenu, što čini jednu vrstu više pogodnom od druge vrste za različite namene. Sva vlakna imaju generalno veliki kapacitet nosivosti (veći od običnog čelika) i linearno elastično ponašanje do loma. U tabeli 1 su prikazane osnovne karakteristike vlakana.

Staklena vlakna imaju dobre mehaničke karakteristike, visoku hemijsku otpornost i odlična izolaciona svojstva uz nisku cenu u odnosu na druge tipove vlakana. Nedostaci ovih vlakana su relativno nizak modul elastičnosti, mala otpornost na zamor, osetljivost na habanje i na vlagu, kao i sklonost ka deformacijama tečenja.

Karbonska vlakna imaju visoke mehaničke karakteristike u pravcu vlakana i znatno niže u poprečnom pravcu. Prednosti karbonskih vlakana su dobar odnos između čvrstoće i težine, odlična trajnost, dobra reološka svojstva, otpornost na zamor. Najveći nedostatak karbonskih vlakana je njihova cena.

Aramidna vlakna imaju najmanju težinu i najveću čvrstoću na zatezanje naspram težine u poređenju sa staklenim i karbonskim vlaknima. Kompoziti sa aramidnim vlaknima imaju dobru otpornost na dinamička i udarna opterećenja. Aramidna vlakna imaju visoku termičku stabilnost i dobru hemijsku otpornost. Mane aramidnih vlakana su nepostojanost na povišenim temperaturama, kao i osetljivost na vlažnost i UV zračenje.

Bazaltna vlakna imaju odličnu otpornost na koroziju i hemijske uticaje. Po svom ponašanju su najbližnja staklenim vlaknima pa se često koriste kao zamena za njih. Najveća prednost ovog tipa vlakana je u tome što su prirodan materijal pa su negativni uticaji na životnu sredinu tokom proizvodnje i upotrebe ovih vlakana svedeni na minimum [1].

Tabela 1 – Poređenje prosečnih vrednosti karakteristika vlakana [1–4]

Vlakna	Zapreminska masa (g/cm ³)	Modul elastičnosti (GPa)	Čvrstoća na zatezanje (MPa)
Staklena	2,6	70-80	2000-3500
Karbonska	1,75-1,95	240-760	2400-5100
Aramidna	1,45	62-180	3600-3800
Bazaltna	2,8	90	4800

2.2 MATRICA

Matrica je vezivni materijal, sa osnovnim zadatkom da drži vlakna zajedno i sačuva njihovu orijentaciju. Takođe, matrica ima ulogu da štiti vlakna od uticaja okruženja i mehaničkog habanja. Veoma je važno da matrica da bude hemijski i termički kompatibilna sa vlaknima, kao i da ima malu zapreminsku masu kako ne bi povećavala težinu kompozita [5]. Najčešće korišćeni polimer za FRP materijale u građevinarstvu je epoksid. Poliester ili vinilester se takođe upotrebljavaju. U Tabeli 2 su prikazane osnovne karakteristike matrica.

Tabela 2 – Karakteristike matrica [4]

Materijal	Zapreminska masa (g/cm ³)	Modul elastičnosti (GPa)	Čvrstoća na zatezanje (MPa)
Epoksid	1,1-1,4	2,0-6,0	35-130
Poliester	1,1-1,5	1,2-4,5	40-90
Vinilester	1,5	3,0-4,0	65-90

2.3 KOMPOZIT

Karakteristike FRP materijala se ne mogu predvideti jednostavnim sumiranjem karakteristika njegovih sastavnih delova. Vlakna i matrica deluju komplementno da obezbede željene karakteristike obe komponente. Na primer, većina matrica na bazi polimera ima malu čvrstoću na zatezanje, ali izuzetnu tvrdoću i savitljivost, dok tanka vlakna imaju veliku čvrstoću na zatezanje, ali osetljivost na oštećenja. Generalno, karakteristike FRP kompozita zavise od karakteristika materijala vlakana i matrice, orijentacije vlakana (kod šipki obično vlakna postavljena u jednom pravcu), zapreminskog udela vlakana, itd.

Pultruzija je tehnologija koja se uglavnom koristi za proizvodnju FRP šipki koje mogu biti u obliku kružnog ili kvadratnog poprečnog preseka, glatke ili rebraste kao i peskarene (slika 1).



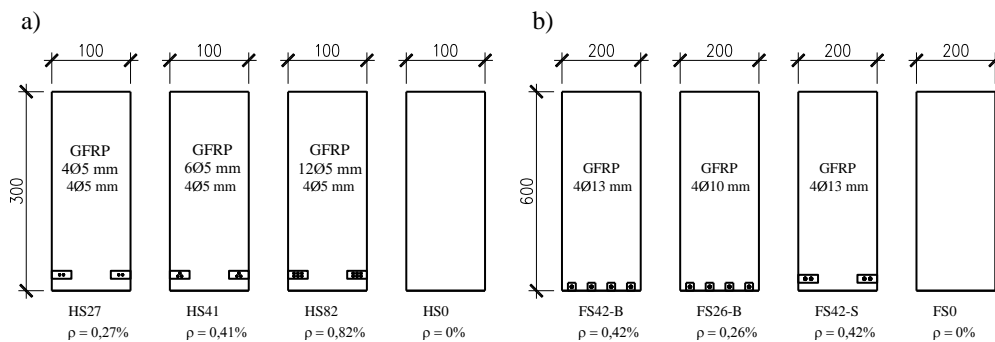
Slika 1 – Tipovi FRP šipki

3 OJAČANJE DRVENIH ELEMENATA IZLOŽENIH SAVIJANJU

Kod drvenih elemenata napregnutih na savijanje, inicijalni lom nastaje uglavnom u zategnutoj zoni u blizini kvrga, defekta ili na mestima poprečnog nastavka lamela kod lepljenih lameliranih nosača. Lom usled zatezanja drveta izloženog savijanju je krt, nasumičan i teško predvidiv. Stoga, drveni nosači se uglavnom ojačavaju na zategnutoj strani, čime se povećava nosivost i krutost na savijanje i postiže znatno duktilniji lom u pritisnutoj zoni.

U različitim radovima do sada je ispitivano ponašanje monolitnih i lepljenih lameliranih drvenih nosača ojačanih FRP šipkama. U nastavku su prikazana neka od istraživanja.

Gentile, Svecova i Rizkalla [6] su sproveli eksperimentalno ispitivanje sa ciljem procene ponašanja na savijanje 30 godina starih drvenih nosača (Duglasova jela) ojačanih šipkama na bazi staklenih vlakana (GFRP). Dvadeset dve grede (10 x 30 x 430 cm) isečene iz glavnih nosača starog drvenog mosta, od kojih je 15 ojačanih, ispitano je na savijanje. Grede su bile ojačane GFRP šipkama postavljenim u zategnutoj zoni sa bočnih strana (slika 2a). Uticaj površine ojačanja u poprečnom preseku je bio razmatran kroz tri procenta ojačanja: 0,27, 0,41 i 0,82%. Pored greda, četiri cela glavna nosača mosta (20 x 60 x 1040 cm), od kojih su tri ojačana, ispitano je sa ciljem utvrđivanja uticaja efekata veličine uzorka na rezultate ojačanja. Ovi nosači su ojačani GFRP šipkama, koje su postavljene u zategnutoj zoni odozdo ili sa bočnih strana (slika 2b). Procenti ojačanja kod nosača bili su 0,26 i 0,42%.

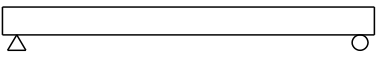
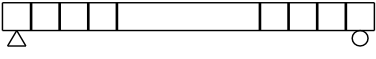
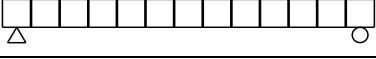
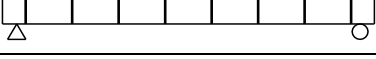
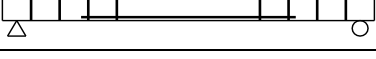
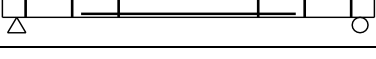
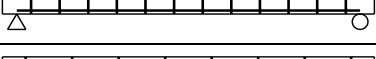
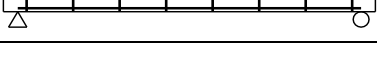


Slika 2 – Poprečni preseci ispitanih ojačanih i neojačanih uzoraka [6]

Istraživanje je pokazalo da su GFRP šipke efikasna tehnika ojačanja na savijanje monolitnih drvenih nosača. Slično ponašanje, u smislu oblika loma, dijagrama opterećenje-ugib, raspodele dilatacija i granične čvrstoće, je zabeleženo kod obe grupe ispitanih uzoraka. Nije evidentiran nikakav efekat veličine uzorka. Za procenute ojačanja između 0,27 i 0,82%, granično opterećenje se povećalo 48-60%. Ojačanjem nosača prosečna vrednost granične dilatacije zatezanja drveta se povećala za 64%, što pokazuje da prisustvo ojačanja znatno umanjuje uticaj defekata u drvetu. Osim ovoga, kod 60% ojačanih uzoraka zabeleženi oblik loma je duktilni lom u pritisnutoj zoni.

Svecova i Eden [7] su sproveli istraživanje kako bi doprineli razvoju praktične metodologije ojačanja postojećih drvenih mostova primenom GFRP šipki. Povod za ovo istraživanje je težnja da se umesto skupe zamene starih i oštećenih drvenih mostova ojačaju postojeće konstrukcije i na taj način im se produži upotrebnii vek. Eksperimentalni program obuhvatio je ispitivanje na savijanje do loma 45 drvenih greda (Duglasova jela), isečenih iz glavnih nosača drvenih mostova oblasti Manitoba u Kanadi. Grede su bile dimenzija 10 x 30 x 200 cm. Generalno, dve šeme ojačanja su primenjene u okviru programa ispitivanja. Prva grupa, Grupa S, obuhvatila je 16 greda, koje su ojačane samo vertikalnim šipkama (prečnika 16 mm) kao ojačanjem na smicanje, dok je druga grupa od 20 greda, Grupa SF, pored vertikalnih imala i ojačanja u vidu dve horizontalne šipke sa bočnih strana (prečnika 5 mm), što je predstavljalo kombinaciju ojačanja na savijanje i na smicanje. Položaj i rastojanje vertikalnih šipki, kao i dužina horizontalnih šipki su parametri koji su varirani. Rezultati ispitivanja ojačanih greda upoređeni su sa rezultatima ispitivanja grupe od 9 neojačanih greda (Grupa C). Program eksperimentalnog ispitivanja dat je u tabeli 3.

Tabela 3 – Program eksperimentalnog ispitivanja [7]


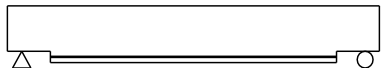
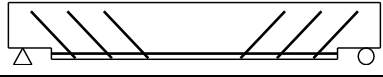
Oznaka uzoraka	Šema ojačanja	Broj uzoraka
C		9
S-S150		5
S-C150		6
S-C300		5
SF-S150		5
SF-S300		5
SF-C150		5
SF-C300		5

Eksperimentalno ispitivanje dovelo je do sledećih zaključaka:

- upotreba GFRP šipki se pokazala kao primenljiva za ojačanje drvenih greda;
- optimalno rastojanje vertikalno postavljenih šipki za povećanje smičuće nosivosti treba da bude jednako širini poprečnog preseka;
- grede ojačane samo na smicanje su pokazale povećanje nosivosti od 33%, dok su kombinovano ojačane grede pokazale povećanje od 47% do 52%;
- upotreba kombinovanog ojačanja dovodi do loma u pritiskutoj zoni, kome prethode velike deformacije što može poslužiti kao upozorenje pre loma grede;
- upotreba ojačanja smanjuje prirodnu varijabilnost mehaničkih karakteristika drveta u različitim pravcima;
- veze između GFRP, lepka i drveta nisu pokazale znake popuštanja pre nastupanja loma čitave grede;
- dijagrami kombinovano ojačanih greda opterećenje-ugib su pokazali duktilno ponašanje, pri čemu je 60% greda imalo dva puta veće ugibe pri lomu u odnosu na neojačane grede.

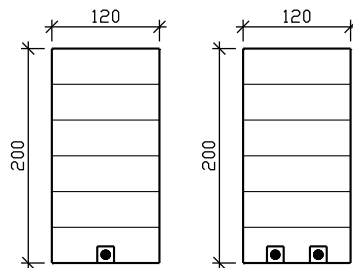
Istraživanje *Amy i Svecova* [8] predstavlja nastavak eksperimentalnog programa ojačanja glavnih nosača drvenih mostova. Sva istraživanja do tada su sprovedena na pravougaonim gredama bez zasečenih krajeva. Međutim, većina drvenih nosača u okviru mostova u oblasti Manitomba (Kanada) je imala redukovanu visinu na krajevima. Zbog koncentracije napona na mestu nagle promene visine, na zasečenim delovima nosača, ovaj eksperimentalni program je obuhvatio ojačanje zasečenih drvenih nosača. Ukupno 26 drvenih nosača (10 x 40 x 340 cm) ispitano je na savijanje do loma: 8 neojačanih (kontrolnih) uzoraka (Grupa C), 12 ojačanih horizontalnim GFRP šipkama (prečnika 12 mm) u oblasti najvećih napona savijanju u zategnutoj zoni (Grupa F) i 6 ojačanih horizontalnim GFRP šipkama u zategnutoj zoni i kosim GFRP šipkama, pod uglom od 60° prema horizontalnoj ravni, kao ojačanje na smicanje (Grupa FD). Program eksperimentalnog ispitivanja dat je u tabeli 4.

Tabela 4 – Program eksperimentalnog ispitivanja [8]

Grupa uzoraka	Šema ojačanja	Broj uzoraka
C		8
F		12
FD		6

Primena samo ojačanja na savijanje se ne preporučuje za zasečene nosače, jer je dominantan smičući lom na zasečenom delu, koji znatno redukuje nosivost nosača. Primena ojačanja i na savijanje i na smicanje za ovaj tip nosača vodila je ka povećanju graničnog opterećenja od 22%, uz promenu oblika loma (pritisak upravno na vlakna u pritiskutoj zoni). Duktilnost nosača je, takođe, povećana primenom GFRP ojačanja. Znatno veće povećanje duktilnosti je zabeleženo kod nosača ojačanih i na savijanje i na smicanje u odnosu na nosače ojačane samo na savijanje.

Micelli, Scialpi i La Tegola [9] su razmatrali mogućnost upotrebe šipki na bazi karbonskih vlakana (CFRP) kao ojačanja lepljenih lameliranih drvenih nosača. Šest nosača od lepljenog lameliranog drveta (smreka), od čega četiri ojačana, je ispitano na savijanje. Nosači (12 x 20 x 500 cm) su ojačani karbonskim šipkama prečnika 12,5 mm (jednom ili dve) postavljenim u zategnutoj zoni, u neposrednoj blizini donje površine (slika 3).







Slika 3 – Poprečni preseki ojačanih nosača [9]

Eksperimentalni rezultati su pokazali da se postavljanjem CFRP šipki sa zategnute strane poprečnog preseka može značajno poboljšati nosivost i krutost nosača. Za procenete ojačanja 0,51% i 1,03%, zabeleženo je povećanje graničnog momenta od 26% i 82%, odnosno povećanje krutosti od 8% i 19% u odnosu na neojačane nosače. Oblici loma i eksperimentalni podaci pokazali su odlične karakteristike spoja između drveta i karbonskih šipki. Lom kako neojačanih, tako i ojačanih nosača zavisio je pre svega od čvrstoće drveta na zatezanje.

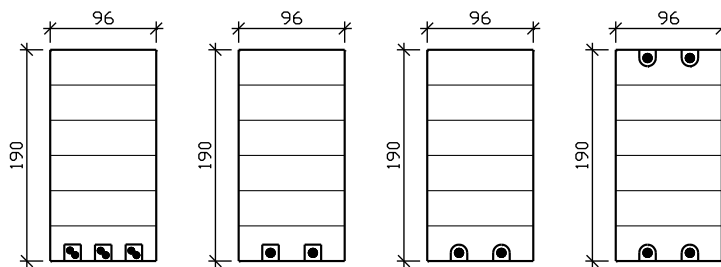
Johnsson, Blanksvard i Carolin [10] su istraživali ojačanje nosača od lepljenog lameliranog drveta pomoću CFRP šipki. Posebna pažnja bila je usmerena ka utvrđivanju potrebne minimalne dužine sidrenja ojačanja, pri kojoj neće doći do pojave prevremenog loma. Ukupno je ispitano 10 lepljenih lameliranih nosača (smreka), porečnog preseka 9 x 22,5 cm i dužine 350 cm. Karbonske šipke (pravougaonog poprečnog preseka, 10 x 10 mm) su postavljene unutar preseka, u neposrednoj blizini donje površine. Program eksperimentalnog ispitivanja dat je u tabeli 5.

Tabela 5 – Program eksperimentalnog ispitivanja [10]

Serijski uzorak	Tip	Ojačanje	Broj uzoraka
1		Bez ojačanja	3
2		1 šipka, centrano postavljena u zategnutoj zoni, celom dužinom nosača	3
3		2 šipke, simetrično postavljene u zategnutoj zoni, celom dužinom nosača	3
4		1 šipka, centralno postavljena u zategnutoj zoni, kraće dužine	1

Rezultati su pokazali da pored povećanja nosivosti na savijanje od 40-60% i povećanja krutosti od 5-25% ovaj metod ojačanja menja tip loma iz krtoq u zatežućoj zoni u duktilni lom u pritisnutoj zoni. Kao posledica duktilnog ponašanja ojačanih nosača, ugib u sredini nosača pri lomu povećao se do 80%. Rezultati eksperimentalne i teorijske analize dužine sidrenja CFRP šipke pokazali su da je minimalna potrebna dužina 150 mm. Takođe, ojačanje postavljanjem šipki blizu površine u pripremljene proreze ne menja visinu nosača dok istovremeno štiti šipke od spoljašnjih uticaja.

Raftery i Whelan [11] su u svom radu istraživali različite dispozicije ojačanja lepljenog lameliranog drveta niže klase sa GFRP šipkama (slika 4). Ispitano je 5 nosača svake serije sa rasponom od 342 cm i dimenzijama poprečnog preseka 9,6 x 19 cm. Kao što se može videti na slici 4 ispitivan je uticaj upotrebe više šipki manjeg prečnika naspram šipki većeg prečnika, zatim oblik proreza u koji se postavljaju šipke, kao i ojačanje u obe zone (pritisnuta i zategnuta zona) nosača. Procenti ojačanja za razmatrane šeme ojačanja su iznosili redom: 1,05, 1,4, 1,4 i 2,8%.

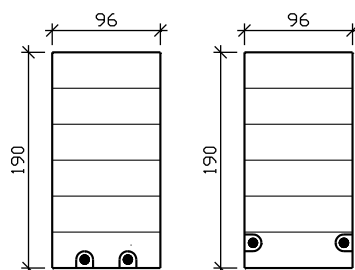


Slika 4 – Poprečni preseki ojačanih nosača [11]

Zaključci dobijeni eksperimentalnim ispitivanjem su sledeći:

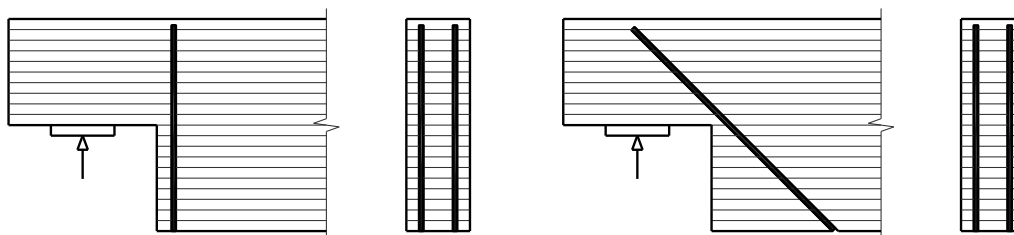
- veći kapacitet nosivosti i veća krutost se dostižu primenim kružnih u odnosu na kvadratne proreze za postavljanje šipki usled smanjenja koncentracije napona na ivicama proreza;
- upotreba više šipki manjih prečnika nije pokazala poboljšanje u nosivosti i krutosti bez obzira na povećanje površine lepljenja između lepka i šipki;
- ojačani nosači su dostigli duktilni lom u pritisnutoj zoni za razliku od neojačanih koji su krti lom dostizali u zategnutoj;
- sa većim procentom ojačanja se postiže i veće iskorišćenje mehaničkih svojstava drveta u pritisnutoj zoni;
- upotrebom procenta ojačanja od 1,4% u zategnutoj zoni dostiže se povećanje krutosti od 11,2% i nosivosti od 68%. Ojačanje od 1,4% u zategnutoj i 1,4% u pritisnutoj zoni dovodi do povećanja krutosti od 22% i nosivosti od 98,5%.

Raftery i Kelly [1] su istraživali primenu šipki na bazi bazaltnih vlakana (BFRP). U svom radu su pokazali da BFRP šipke imaju nešto bolje karakteristike od GFRP šipki. Ispitano je 5 nosača svake serije sa rasponom od 342 cm i dimenzijama poprečnog preseka 9,6 x 19 cm. Rezultati ojačanja su dosta slični onim koji se dobijaju ojačavanjem GFRP šipkama. Pored postavljanja ojačanja na donjoj zategnutoj strani nosača, ispitano je i postavljanje šipki u proreze sa strane (slika 5), što je povoljnije sa estetske strane. Međutim, ovako postavljeno ojačanje daje manje povećanje nosivosti nosača, jer položaj ojačanja nije optimalan u odnosu na neutralnu osu poprečnog preseka.



Slika 5 – Poprečni preseki ojačanih nosača [1]

Lokalno ojačanje drvenih nosača su prikazali *Franke, Franke i Harte* [12]. U slučaju kada se oslanjanje nosača izvodi sa redukovanom visinom nosača iznad oslonca, koncentracije napona koje se javljaju u uglu dovode do otvaranja pukotina na tom mestu i njihove brze progresije kroz poprečni presek elementa, što može dovesti do loma. Pored smičućih napona javljaju se i naponi zatezanja upravni na vlakna. Predložene metode ojačanja prikazane su na slici 6 [12]. Ojačanje može biti postavljeno upravno na vlakna ili pod uglom od 45° .



Slika 6 – Ojačanje na mestu redukovane visine nosača – neposredno uz oslonac [12]

Primena ojačanja pod uglom od 45° dala je znatno bolje rezultate, pre svega zbog velikih smičućih napona u ovim preseccima. Pored povećanja nosivosti, ojačanje ovog tipa omogućilo je i duktilniji lom nosača. Istraživanje je pokazalo da FRP šipke treba da bude postavljeno što bliže uglu zasecanja, pri čemu je veoma važno da budu ispoštovana njihova minimalna odstojanja.

4 ZAKLJUČAK

Primena FRP šipki kao materijala za ojačanje drvenih elemenata pruža velike mogućnosti kod sanacije postojećih konstrukcija, ali i kod projektovanja novih objekata. Pored svoje jednostavnosti, ova tehnika omogućava značajno povećanje nosivosti i krutosti ojačanih konstrukcijskih elemenata. Takođe, čini konstrukciju pouzdanijom, redukujući mogućnost pojave krtog loma. Prisustvo FRP ojačanja sprečava otvaranje pukotina, ograničava lokalna oštećenja i premošćava lokalne defekte u drvetu.

Bez obzira na visoke mehaničke karakteristike i druge pogodnosti, posebnu pažnju treba obratiti na pitanje trajnosti, funkcionalnosti i ekonomičnosti, što se postiže pravilnim izborom i primenom odgovarajućeg sistema. Buduća istraživanja treba definitivno da utvrde uticaj tipa, položaja i površine ojačanja na nosivost i krutost drvenih elemenata kako za granično stanje

nosivosti, tako i za granično stanje upotrebljivosti. Osim toga, da bi bi ovaj metod ojačanja bio praktičan i ekonomičan za svakodnevnu primenu potrebno je usavršiti proračunski postupak i implementirati ga u dogovarajući standard.

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Nemanja Marković⁵, Stefan Conić⁶*

RAČUNSKA ANALIZA NEKIH METODA SPREZANJA DRVETA I BETONA

Rezime:

S-39

U radu je dat kratak postupak proračuna spregnutih konstrukcija drvo-beton na osnovu postavki teorije elastičnosti i u skladu sa aktuelnim evropskim i našim propisima u ovoj oblasti poznatijim kao Evrokod 5. Pregled raznih inovativnih spojnih sistema i sredstava koja se koriste za povezivanje ova dva materijala izložen je u određenoj meri. Suština rada se zasniva na uporednoj analizi proračuna nosivosti proste spregnute grede tipa drvo-beton određenih geometrijskih karakteristika. Sprezanje je u prvoj varijanti izvedeno klasičnim postupkom, pomoću zavrtnjeva, a u drugoj, kombinacijom zavrtnjeva i žlebova izvedenim u drvetu.

Ključne reči: proračun, spregnute konstrukcije, drvo, beton, zavrtnaj, žleb.

COMPUTATIONAL ANALYSIS OF SOME METHODS COUPLING TIMBER AND CONCRETE

Summary:

This paper gives a brief procedure for the design of composite timber-concrete structures based on the theory of elasticity and in accordance with current European and national codes in this area, known as Eurocode 5. Review of various innovative coupling systems used to connect these two materials is exposed to a certain extent. The essence of the work is based on comparative analysis of the load bearing capacity simply supported composite timber concrete beams with specific geometric characteristics. Coupling, in the first variant, is performed by classical procedure, using bolts, and the other, a combination of bolts and notches executed in timber.

Key words: computation, composite structures, timber, concrete, bolt, notch.

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1 UVOD

Spregnute konstrukcije drvo-beton predstavljaju inovativne konstruktivne elemente koji su poslednjih decenija postali predmet istraživanja i praktične primene u našoj zemlji, a i u inostranstvu, prvenstveno zbog svojih atraktivnih mehaničkih osobina. Drvo ima relativno veliku čvrstoću na zatezanje i njegovim sprežanjem sa betonom postiže se optimalno iskorišćenje nosivosti ova dva materijala u nosaču. Sreću se u novim konstrukcijama stambenih zgrada, mostova, industrijskih i sportskih objekata

Sprežanjem drveta i betona u različitim eksploatacionim uslovima može da ima veoma korisne efekte. Ako ovaj metod koristimo pri reparaciji klasičnih drvenih tavanica, nakon intervencije nosivost istih može biti i do dva puta uvećana, a krutost upravno na ravan tavanice poboljšana 3-4 puta. Korišćenjem spregnutih konstrukcija tipa drvo-beton, prisutna je manja upotreba oplata, smanjuju se troškovi fundiranja zbog manje težine konstrukcije, bolja je zvučna izolacija, objekat je sigurniji na seizmička dejstva. Ukoliko se koristi ovaj tip sprežanja pri izgradnji mostova, treba imati u vidu da betonska ploča pokriva drvenu gredu i štiti je od negativnih atmosferskih uticaja.



Slika 1 - Vihantasalmi most, Finska. Raspon: $21+3 \times 42+21=168\text{m}$, širina: $11+3=14\text{ m}$

Jedan od osnovnih elemenata nosača sastavljenog od dva različita materijala jesu spojna sredstva pomoću kojih se oni međusobno povezuju u jednu funkcionalnu celinu. Spojna sredstva pomoću kojih se ostvaruje sprežanje drveta i betona, u ovom slučaju, treba da prihvate smičuće sile koje se javljaju duž spojne ravni spregnutih elemenata od različitog materijala i da obezbede odgovarajuću nosivost nosača na različite vrste naprežanja.

U formiranju hibridnog preseka, a u cilju poboljšanja elastomehaničkih karakteristika, nije isključeno korišćenje i drugih savremenih materijala kao što su karbonska i staklena vlakna, čelik i posebne vrste betona.

2 TEORIJSKI KONCEPT PRORAČUNA SPREGNUTIH KONSTRUKCIJA TIPRA DRVO-BETON. EUROCODE 5

Pri proračunu elastično spregnutih nosača drvo-beton uvode se sledeće pretpostavke:

- drvo, beton i spojna sredstva su izotropni, elastični materijali i važi Hukov zakon;
- važi Bernulijeva hipoteza, odnosno ravni presecci i posle deformacije ostaju ravni i upravni na deformisanu osu nosača. Ova hipoteza kod elastično spregnutih nosača ne važi za ceo presek nosača, ali važi za svaki od pojedinih elemenata spregnutog preseka;
- spojna sredstva postavljena su na konstantnom razmaku i mogu se smatrati kao ekvivalentna kontinualna veza sa konstantnom krutošću spoja duž celog nosača;
- poprečni presecci betona i drveta su konstantni celom dužinom nosača;
- drvo i beton imaju jednake ugibe u svakoj tački spoja.

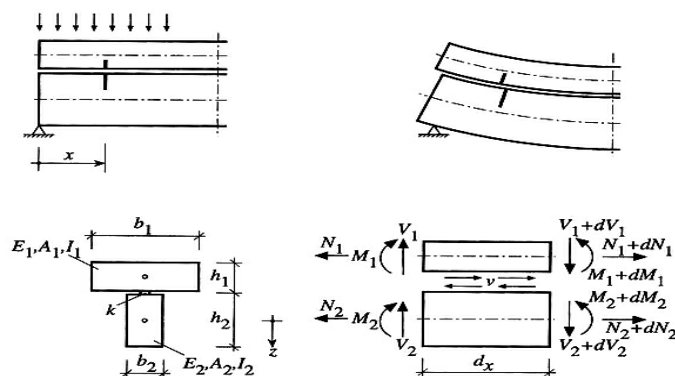
Kao najopštiji slučaj pri statičkoj analizi, razmatra se spregnuti nosač drvo-beton sistema proste grede opterećen proizvoljnim spoljnim opterećenjem $q(x)$. Iz uslova ravnoteže diferencijalno malog elementa dx i na osnovu pretpostavki o jednakosti krivine drvenog i betonskog dela preseka i zanemarenju deformacije usled dejstva smičućih sila krivina se može definisati izrazom:

$$w'' = -\frac{M_1}{E_1 I_1} = -\frac{M_2}{E_2 I_2} \quad (2.1)$$

gde su $E_1 I_1$, $E_2 I_2$ krutosti na savijanje betonskog, odnosno drvenog dela preseka.

Korišćenjem uslova kompatibilnosti aksijalnih pomeranja u spoju drveta i betona i određenim matematičkim transformacijama dobija se diferencijalna jednačina četvrtog stepena po pomeranjima w za spregnuti nosač drvo-beton sistema proste grede koji je opterećen u svemu prema slici 3.

$$w'''' - \alpha^2 w'' = -\frac{M_x''}{(EI)_0} + \alpha^2 \frac{M}{(EI)_\infty} \quad (2.2)$$



Slika 3-Deformacija spregnute grede, sile u preseccima element dx (H. J. Blass)

Kada je poznato rešenje za w , za date granične uslove, unutrašnje presečne sile mogu se lako sračunati u pojedinim elementima spregnutog preseka, a na osnovu njihovih vrednosti i

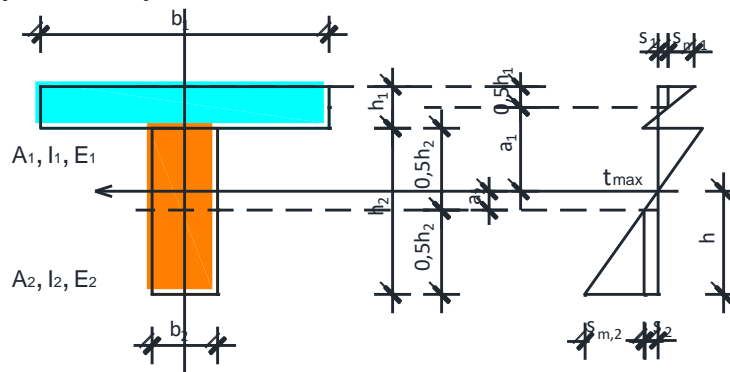
odgovarajuća naponska stanja u bilo kojoj tački površine poprečnog preseka spregnutog nosača, tj.

$$\sigma_1(x, y) = \frac{M_1(x)}{I_1} y + \frac{N_1(x)}{A_1} \quad \sigma_2(x, y) = \frac{M_2(x)}{I_2} y + \frac{N_2(x)}{A_2} \quad (2.3)$$

$$\tau_1(x, y) = \frac{V_1 S_1}{b_1 I_1} + \frac{v}{b_1} \left(\frac{A_1}{I_1} - \frac{S_1}{I_1} r_1 \right) \quad \tau_2(x, y) = \frac{V_2 S_2}{b_2 I_2} + \frac{v}{b_2} \left(\frac{A_2}{I_2} - \frac{S_2}{I_2} r_2 \right) \quad (2.4)$$

Proračun spregnutih greda tipa drvo-beton okvirno je definisan u aneksu B Evrokoda 5. Proračun napona po visini betonskog i drvenog dela spregnutog preseka, kao i nosivost mehaničkih spojnih sredstava podrazumevaju princip elastičnog spreznjanja.

Prema preporukama iz aneksa B Evrokoda 5, za spregnuti poprečni presek određenih geometrijskih karakteristika, dat na slici 4, stanje normalnih i smičućih napona u karakterističnim tačkama poprečnog preseka i mehanička svojstva sredstava ili sistema mogu se sračunati korišćenjem sledećih jednačina:



Slika 4-Spregnuti poprečni presek

Efektivna krutost na savijanje spregnutog nosača može se sračunati jednačinom:

$$(EI)_{ef} = \sum_{i=1}^n (E_i I_i + \gamma_i E_i A_i a_i^2), \quad \text{gde je} \quad (2.5)$$

i broj elemenata u poprečnom preseku spregnutog nosača.

E prosečna vrednost modula elastičnosti betona, odnosno drveta.

$$A_i = b_i h_i, \quad I_i = b_i h_i^3 / 12, \quad \gamma_2 = 1, \quad (2.6)$$

$$\gamma_i = \left[1 + \pi^2 E_i A_i s_i / (K_i \ell^2) \right]^{-1} \quad \text{za } i = 1, \quad i = 3, \quad (2.7)$$

$$a_2 = \frac{\gamma_1 E_1 A_1 (h_1 + h_2) - \gamma_3 E_3 A_3 (h_2 + h_3)}{2 \sum_{i=1}^3 \gamma_i E_i A_i}. \quad (2.8)$$

Vrednosti normalnih napona u pojedinim elementima spregnutog preseka mogu se odrediti jednačinama:

$$\sigma_i = \gamma_i E_i a_i M / (EI)_{ef} \quad \sigma_{m,i} = 0,5 E_i h_i M / (EI)_{ef} \quad (2.9, 2.10)$$

Maksimalna vrednost napona smicanja u određenoj tački površine poprečnog preseka drvenog dela poprečnog preseka spregnutog nosača može se odrediti izrazom:

$$\tau_{2,max} = (\gamma_3 E_3 A_3 a_3 + 0,5 E_2 b_2 h^2) V / (b_2 (EI)_{ef}). \quad (2.11)$$

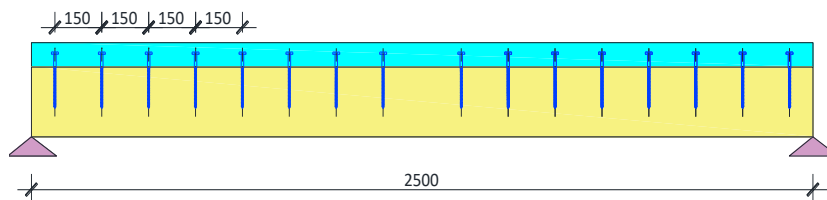
Opterećenje štapastog spojnog sredstva može se sračunati korišćenjem jednačine:

$$F_i = \gamma_i E_i A_i a_i s_i V / (EI)_{ef}, \quad \text{pri čemu je } i=1 \text{ i } 3, \quad (2.12)$$

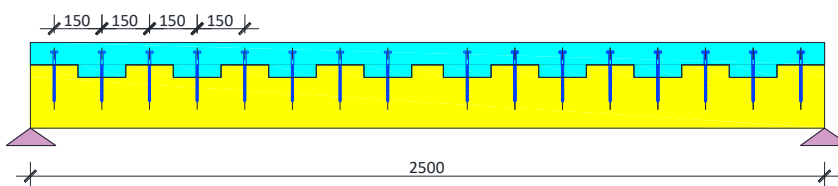
$s_i = s_i(x)$ razmak spojnih sredstava određen u poglavlju B1.3 i $V = V(x)$.

3 PRORAČUN NOSIVOSTI SPREGNUTIH GREDA

Dve grede tipa drvo-beton, spregnute različitim spojnim sredstvima, određenih geometrijskih karakteristika (slike 5, 6 i 7), analizirane su na osnovu napred izloženog koncepta elastičnog sprezanja. U prvom postupku sprezanja korišćen je zavrtanj prečnika 8mm. Duž spoja drveta i betona zavrtnjevi su ugrađeni upravno na spojnu ravan drvenog i betonskog dela preseka na međusobnom rastojanju $s=150mm$. Drugi model grede kao sredstvo sprezanja koristi kombinaciju pomenutih zavrtnjeva i žlebova izvedenih u drvetu u širini jednakoj širini drvene grede (100mm), dubini od 13mm, dužini od 150mm. Žlebovi su izvedeni na osovinsko rastojanju od 300mm.

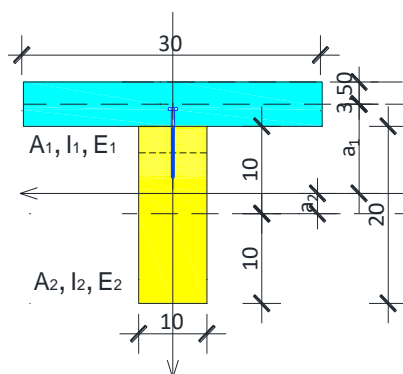


Slika 5-Spregnuta greda zavtnjima



Slika 6-Spregnuta greda zavtnjima i žlebovima određene geometrije

Mehaničke karakteristike betona i drveta usvojene su iz odgovarajućih standarda koji su navedni u spisku literature. Geometrijske karakteristike poprečnog preseka spregnutih greda date su na slici 7.



Slika 7-Geometrijske karakteristike poprečnog preseka spregnutih greda

3.1 MEHANIČKE KARAKTERISTIKE ELEMENATA SPREGNUTE GREDE

Ploča: beton klase čvrstoće C25/30, prema ENV 206 :

$$f_{ck,cube} = 30 \text{ N/mm}^2 \quad f_{cm} = 2,6 \text{ /mm}^2 \quad E_{cm} = 30000 \text{ N/mm}^2;$$

C25/30, RH=80% i $2Ac/u \approx 200 \Rightarrow \varphi_{28,\infty} = 2,12$ and $\varphi_{3year,\infty} = 1,10$

Greda: lepljeno lamelirano drvo, GL24, SRPS EN 338 (2012):

$$\begin{aligned} f_{m,k} &= 24 \text{ N/mm}^2 & f_{t,0,k} &= 16,5 \text{ N/mm}^2 & f_{v,k} &= 2,7 \text{ N/mm}^2 \\ \rho_{0,k} &= 380 \text{ kg/m}^3 & f_{h,0,k} &= 29,42 \text{ N/mm}^2 & E_{0,mean} &= 11600 \text{ N/mm}^2 \\ f_{c,0,k} &= 24 \text{ N/mm}^2 & f_{c,90,k} &= 2,70 \text{ N/mm}^2 & & \end{aligned}$$

Eksploataciona klasa 1: $k_{mod} = 0,6$; $k_{def} = 0,6$ (stalno opterećenje), (EN14080)

Spojno sredstvo: zavrtanj, (EN 10080), Eksploataciona klasa 1, prečnik, $d = 8 \text{ mm}$.
Efektivni razmak spojnih sredstava duž spoja drveta i betona:

$$s_{eff} = 0,75s_{min} + 0,25s_{max} = 15 \text{ cm} = 150 \text{ mm}, \quad f_{u,k} = 500 \text{ N/mm}^2,$$

$$M_{y,k} = 0,3 \cdot f_{u,k} \cdot d^{2,6} = 0,3 \cdot 500 \cdot 8^{2,6} = 33429,14 \text{ Nmm},$$

$$f_{h,0,k} = 0,082(1 - 0,01d) \rho_k = 0,082(1 - 0,01 \cdot 8) \cdot 390 = 29,42 \text{ N/mm}^2$$

Modul pomerljivosti:

$$K_{ser} = \rho_m^{1,5} d / 23 = 420^{1,5} \cdot 8 / 23 = 2993,89 \text{ N/mm} \quad (\text{GSU})$$

$$K_u = 2K_{ser} / 3 = 1995,92 \text{ N/mm} \quad (\text{GSN}).$$

Računske vrednosti mehaničkih karakteristika elemenata spregnutog preseka prema EC 2, EC 4 i EC 5 su za beton i drvo, redom:

$$f_{cd} = \frac{0,83 \cdot 0,85 f_{ck,cube}}{1,5} = 14,1 N/mm^2 \quad f_{ctmd} = \frac{0,85 f_{ctm}}{1,5} = 1,47 N/mm^2$$

$$f_{m,d} = \frac{0,6 \cdot 24}{1,3} = 11,07 N/mm^2 \quad f_{t,0,d} = \frac{0,6 \cdot 16,5}{1,3} = 7,61 N/mm^2$$

$$f_{v,d} = \frac{0,6 \cdot 3,5}{1,3} = 1,61 N/mm^2 \quad f_{h,0,d} = \frac{0,6 \cdot 29,42}{1,3} = 13,57 N/mm^2$$

$$f_{c,0,d} = \frac{0,6 \cdot 24}{1,3} = 11,07 N/mm^2 \quad f_{c,90,d} = \frac{0,6 \cdot 2,7}{1,3} = 1,25 N/mm^2$$

Nosivost zavrtnja M8 u odnosu na beton (EN 14555):

$$R_d = P_{Rd} = 0,29 \alpha d^2 \sqrt{f_{ck} \frac{E_{cm}}{\gamma_v}} = 0,29 \cdot 1,0 \cdot 8^2 \sqrt{30 \cdot \frac{30000}{1,25}} = 15748,68 N = 15,70 kN$$

d prečnik zavrtnja,
 f_{ck} karakteristična čvrstoća na pritisak betonske kocke ($f_{ck} = 1,5 f_{cd}$),
 γ_v parcijalni koeficijent sigurnosti za materijal ($\gamma_v = 1,25$),
 E_{cm} srednja vrednost modula elastičnosti betona.

Nosivost zavrtnja na smicanje u vezi sa betonom (EN 14555):

$$R_d = P_{Rd} = 0,8 f_u \frac{\pi d^2}{4 \gamma_v} = 0,8 \cdot 500 \cdot \frac{3,14 \cdot 8^2}{4 \cdot 1,25} = 16076,8 N = 16,08 kN$$

Nosivost zavrtnja u vezi sa drvenom gredom:

$$R_k = 2,3 \sqrt{M_{y,k} f_{h,2,k} d} = 2,3 \sqrt{500 \cdot 29,42 \cdot 8} = 789 N = 7,89 kN$$

3.2 PRORAČUN PREMA GRANIČNOM STANJU NOSIVOSTI

Geometrijske karakteristike poprečnog preseka prikazanog na slici 7:

$$A_1 = 21000 mm^2 \quad A_2 = 20000 mm^2$$

$$I_1 = 8575000 mm^4 \quad I_2 = 66666666,67 mm^4$$

$$\gamma_1 = 0,013 \quad \gamma_2 = 1,0$$

$$a_1 = 130,36 mm \quad a_2 = 4,64 mm$$

$$E_1 = E_{cm} = 30000 N/mm^2 \quad E_2 = E_{0,mean} = 11500 N/mm^2$$

$$\gamma_1 = \left[1 + \pi^2 E_1 A_1 s_{eff} / (K_u \ell^2) \right]^{-1} = 0,013 \quad a_2 = 4,64 mm$$

$$(EI)_{ef} = E_1 I_1 + \gamma_1 E_1 A_1 a_1^2 + E_2 I_2 + \gamma_2 E_2 A_2 a_2^2 = 4,78 \cdot 10^{11}$$

3.2.1 Računske vrednosti odgovarajućih normalnih napona:

Betonski deo preseka:

$$\sigma_{c,1,d} = \gamma_1 E_1 a_1 M_d / (EI)_{ef} = 1,03 M_d \cdot 10^{-7} N / mm^2$$

$$\sigma_{m,1,d} = 0,5 E_1 h_1 M_{d,1} / (EI)_{ef} = 2,19 M_d \cdot 10^{-6} N / mm^2,$$

$$\sigma_{c,d} = \sigma_{c,1,d} + \sigma_{m,1,d} \leq f_{cd} = 14,1 N / mm^2 \Rightarrow q_d = 7,83 kN / m' \Rightarrow F_d = 19,56 kN$$

$$\sigma_{c,t,d} = \sigma_{m,1,d} - \sigma_{c,1,d} \leq f_{cmd} = 1,47 N / mm^2 \Rightarrow q_d = 0,89 kN / m \Rightarrow F_d = 2,25 kN$$

Drveni deo preseka:

$$\sigma_{t,2,d} = \gamma_2 E_2 a_2 M_d / (EI)_{ef} = 11500 \cdot 4,64 M_d / 4,78 \cdot 10^{11} = 1,12 M_d \cdot 10^{-7} N / mm^2,$$

$$\sigma_{m,2,d} = 0,5 E_2 h_2 M_d / (EI)_{ef} = 0,5 \cdot 11500 \cdot 200 \cdot M_d / 4,78 \cdot 10^{11} = 24 M_d \cdot 10^{-7} N / mm^2,$$

$$\frac{\sigma_{t,2,d}}{f_{t,0,d}} + \frac{\sigma_{m,2,d}}{f_{m,d}} \leq 1,0 \Rightarrow q_d = 5,50 kN / m' \Rightarrow F_u = 13,76 kN$$

Računski napon smicanja u gredi:

$$\tau_{2,max} = 0,5 E_2 h^2 V_d / (EI)_{eff} \leq f_{v,d} = 1,61 N / mm^2 \Rightarrow q_u = 2,67 N / mm \Rightarrow F_u = 6,70 kN$$

Nosivost zavrtnja:

$$F_{1,d} = \frac{\gamma_1 E_1 A_1 a_1 s_{min} V_d}{(EI)_{eff}} \leq R_d = 7890 N \Rightarrow q_u = 18,8 kN / m \Rightarrow F_u = 47 kN.$$

3.3 PRORAČUN PREMA GRANIČNOM STANJU UPOTREBLJIVOSTI

Geometrijske karakteristike poprečnog preseka prikazanog na slici 7:

$$A_1 = 21000 mm^2$$

$$A_2 = 20000 mm^2$$

$$I_1 = 8575000 mm^4$$

$$I_2 = 66666666,67 mm^4$$

$$\gamma_1 = 0,013, \quad a_1 = 130,36 mm$$

$$\gamma_2 = 1,0, \quad a_2 = 6,68 mm$$

$$E_1 = E_{cm} = 30000 N / mm^2 \quad E_2 = E_{0,05} = 5 / 6 E_{0,mean} = 9583,33 N / mm^2$$

$$K_{ser} = \rho_m^{1,5} d / 23 = 420^{1,5} \cdot 8 / 23 = 2993,89 N / mm$$

$$\gamma_1 = \left[1 + \pi^2 E_1 A_1 s_{eff} / (K_{ser} \ell^2) \right]^{-1} = 0,019, \quad a_2 = 6,68 mm$$

$$(EI)_{ef} = E_1 I_1 + \gamma_1 E_1 A_1 a_1^2 + E_2 I_2 + \gamma_2 E_2 A_2 a_2^2 = 4,78 \cdot 10^{11}$$

Vrednost elastične deformacije usled stalnog opterećenja:

$$u_{1,inst} = \frac{5}{384} \cdot \frac{q_k \ell^4}{(EI)_{ef}} = \frac{5}{384} \cdot \frac{q_k \cdot 2500^4}{4,78 \cdot 10^{11}} \leq \frac{l}{300} = 8,33mm \Rightarrow q_k = 7,2kN/m \Rightarrow F_k = 19,57kN$$

4 PRORAČUN NOSIVOSTI GREDE SA ŽLEBOVIMA

U drugom postupku sprežanja primenjen je spojni sistem koji se sastoji od zavrtnjeva prečnika $d=8mm$ i žlebova od betona formiranih pogodnom obradom drvenog dela spregnutog preseka. Žlebovi su su dubine 35mm i dužine od 150mm. Osovinsko rastojanje samih žlebova je 300mm. U osnovi, postupak proračuna nosivosti ovako kreirane spregnute grede je u svemu analogan prethodno pokazanom postupku. Razliku donosi druga vrednost modula popustljivosti spojnog sistema K_u , odnosno K_{ser} koja se može jedino eksperimentalno odrediti na osnovu EN 26891.

$$K_u = 0,6F_m / v_{0,6} \quad K_{ser} = 0,4F_m / v_{0,4} \quad (4.1a,b)$$

F_m čvrstoća na smicanje betona armiranog zavrtnjem određenog prečnika u spregnutoj žleboj vezi drveta i betona

$v_{0,6}$ i $v_{0,4}$ odgovarajuća pomeranja u žleboj vezi drveta i betona pri dejstvu sile F_m u vrednosti od 40 % odnosno 60% vrednosti koja se može sračunati izrazom:

$$F_{conc, shear} = \beta \cdot 0,5b_n l_n v f_c + n_{ef} (\phi_{cs} d_{ef} \pi)^{0,8} f_w \quad (4.2)$$

U sledećoj tabeli date su analitičke vrednosti nosivosti sprenute grede prikazane na slici 5, određene iz odgovarajućih uslova iskorišćenja računskih čvrstoća i deformacija, po konceptu graničnih stanja nosivosti i upotrebljivosti. Nosivost spregnute grede zavrtnjima i žlebovima kao i uporedna analiza biće komentarisani nakon eksperimentalno određenih parametara popustljivosti takvog spojnog sistema.

Tabela 1 – Uporedne vrednosti nosivosti dveju spregnutih greda različitim postupcima

	I postupak	II postupak
$\sigma_{c,d} \leq f_{cd}$	$q_d = 7,83kN/m' \Rightarrow F_d = 19,56kN$	<i>eksperimentalna procedura</i>
$\sigma_{c,t,d} \leq f_{ctmd}$	$q_d = 0,89kN/m \Rightarrow F_d = 2,25kN$	
$\frac{\sigma_{t,2,d}}{f_{t,0,d}} + \frac{\sigma_{m,2,d}}{f_{m,d}} \leq 1,0$	$q_d = 5,50kN/m' \Rightarrow F_u = 13,76kN$	
$\tau_{2,max} \leq f_{v,d}$	$q_u = 2,67N/mm \Rightarrow F_u = 6,70kN$	
$F_{1,d} \leq R_d$	$q_u = 18,8kN/m \Rightarrow F_u = 47kN$	
$u_{1,inst} \leq 8,33mm$	$q_k = 7,2kN/m \Rightarrow F_k = 19,57kN$	

ZAKLJUČAK

Prikazanom postupkom data je analiza dveju spregnutih greda tipa drvo-beton određenih, istih geometrijskih karakteristika, koje su formirane korišćenjem različitih sistema za sprezanje. Sračunate su odgovarajuće nosivosti u odnosu na granične vrednosti karakterističnih (računskih) čvrstoća na pritisak i zatezanje u betonskom delu preseka i na smicanje, savijanje i zatezanje u drvenom delu spregnutog preseka. Sagledan je i kapacitet spojnog sredstva u prvom postupku sprezanja. Popustljivost spojnog sredstva ili sistema u vezi drveta i betona karakteriše se određenom vrednošću koja se, u prvom postupku, može analitički, dovoljno tačno, odrediti jednačinom iz EC 5, a u drugom, na osnovu standarda EN 26891, kojim je definisana određena eksperimentalna procedura. Eksperimentalna verifikacija dobijenih rezultata kroz ispitivanje određenog broja greda do sloma uz praćenje i merenje niza parametara koji karakterišu mehaničko ponašanje spregnutih konstrukcija drvo beton neophodna je za dalji iskorak u istraživanjima.

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Dragan Manojlović¹, Tatjana Kočetov Mišulić²

RACIONALNI IZBOR PROPORCIJA ELEMENATA U SPREGNUTOM SISTEMU DRVO - BETON

Rezime:

S-40

Mehanička svojstva i dimenzije konstitutivnih elemenata u spregnutom sistemu drvo-beton (drvena greda, betonska ploča i sredstva za sprezanje) svojim proporcijama i rasporedom direktno utiču na krutost i nosivost konstrukcije. U očekivanju dela Evrokoda za spregnute konstrukcije drvo-beton, mnogobrojne studije eksperimentalnim i parametarskim istraživanjima doprinose proširivanju baze znanja, kao i generalizaciji i optimizaciji neophodnih izraza za proračun. Rad obuhvata relevantne izvode iz dostupnih standarda i istraživanja, kao i prikaz racionalnog izbora dimenzija elemenata pri projektovanju ovakvih konstrukcija.

Ključne reči: proporcije elemenata, kompozit drvo-beton, efektivna krutost, modul pomerljivosti

RATIONAL CHOICE OF ELEMENTS' PROPORTIONS IN TIMBER - CONCRETE COMPOSITE SYSTEMS

Summary:

The mechanical properties and dimensions of members in timber-concrete composites (wooden beams, concrete slab and connectors), with their proportions and arrangement, directly affect on effective bending stiffness and structural resistance. In expectation of EC part dedicated to timber-concrete composites, many experimental and parametric studies contributes to knowledge database extension, as well as to generalization and optimization of the expressions necessary for design. The paper presents the relevant outcomes of structural codes and research studies, as well as the overview of parametric study for rational choice of elements' proportions in the optimized design of TCC structures.

Key words: optimal design, TCC composite, effective stiffness, slip modulus

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1 TIMBER-CONCRETE COMPOSITE SYSTEM

Originally, the timber-concrete composite (TCC) system has been developed to strengthen existing timber joist floors with a concrete slab, but this construction technique is now also applied in new (multi-storey) buildings or for decks of timber/glulam bridges. TC composites represent very effective combination of the two construction materials (the concrete is placed in the pressure zone and the timber in the tension zone) and show many advantages comparing to pure timber or pure concrete slabs (greater rigidity, higher load-bearing capacity, less self-weight, improved fire safety and sound insulation,). The advantages and performance of mentioned construction technique are deeply scientifically investigated and practically applied in reinforcement of many existing floor structures in Serbia by Stevanovic (1991-2005), [1]. The reasons for rather poor application of this effective construction technique in Serbian practice lay in lack of codes, in lack of tradition and experience, lack of general information and research about relevant connector's behavior, that consequently leads to inability of adequate computational modeling, what is unavoidable tool in modern structural design.

The aim of the paper is to promote TCC as effective and sustainable construction system, as well as to point out the facts about rational proportions' choice.

1.1 SHEAR CONNECTORS AND EFFECTIVE BENDING STIFFNESS OF TCC

The degree of composite action in TCC, that may vary from no composite action to full composite action, Fig 1, directly depends on the load-slip behavior of connectors type, arrangements and positions, the way of application i.e. on achieved connection stiffness. The shear connectors are the key elements of a composite system because they have to provide effective shear transfer interconnecting the concrete slab with timber beam.

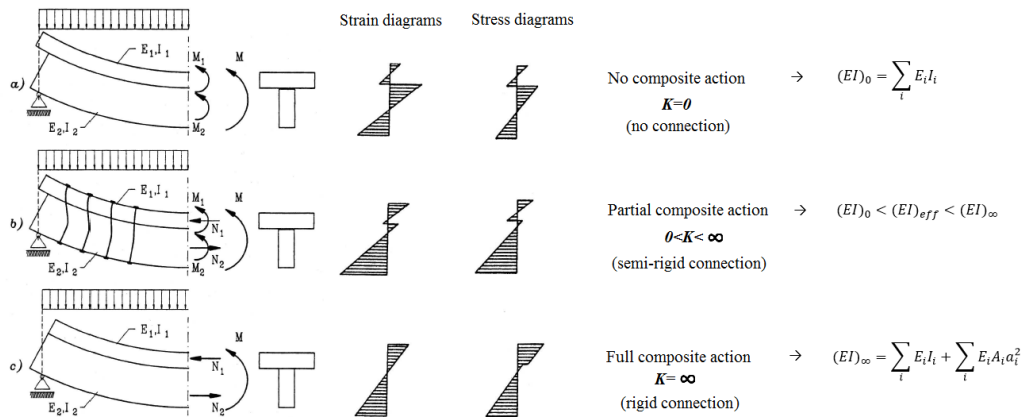


Figure 1 – No composite, Partial and Full composite action in TCC system

Types of fasteners and methods of their applications as shear connectors in TCC systems varied from typical mechanical fasteners for wood (nails, screws, bolts, studs, ...) as discrete, relatively flexible dowels to technologically advanced continual connections (punched metal plates, steel glued lattices, perforated steel "T" ribs, glued-in, etc...) as rigid ones. In Serbian

practice, the doweled type connectors, such as nails, reinforced steel bars - studs, long and self-tapping screws, in vertical and/or inclined position, are the commonly used connectors, Fig 2. Their advantages lay in a facts that they are inexpensive, practical, easy to handle and install, with the sufficient degree of composite action related to purpose and function of considered floor or bridge deck structures.

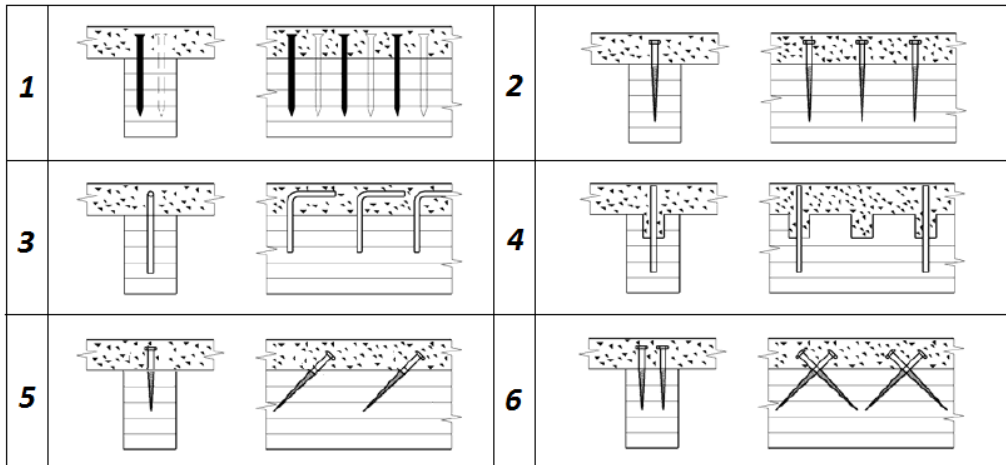


Figure 2 – Commonly used TCC joining methods in Serbia [5]

1.2 EUROCODE 5 : "γ" METHOD

Analytical models for calculating stresses and deformations in TCC systems, Fig 3, are generally based on semi-rigid connections between two layers. A simple linear model, widely used and present in Eurocode 5 [2] is based on Newmark's and Mohler's works developed in 60's and is so called "γ" (gamma) method.

The mechanical performance of TCC systems, is more influenced by joint slip modulus than the joint ultimate load-bearing capacity. The dominant factor in "γ" method, i.e. in Eqs (1-3), is the slip modulus for connections "K_i" - the target value of numerous experimental and theoretical verification.

$$(EI)_{ef} = E_1 \cdot I_1 + \gamma_1 \cdot A_1 \cdot a_1^2 + E_2 \cdot I_2 + \gamma_2 \cdot A_2 \cdot a_2^2 \quad (1)$$

$$\gamma_1 = \left[1 + \frac{\pi^2 \cdot E_1 \cdot A_1 \cdot s}{K_i \cdot L^2} \right]^{-1} \quad (2)$$

$$a_2 = \frac{E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + E_2 \cdot A_2)} \quad (3)$$

where:

$(EI)_{ef}$ is the effective bending stiffness,

- γ_1 is gamma coefficient (parameter describing composite action),
 a_2 is distance between the centroid of the timber-concrete composite section to the centroid of the timber section.

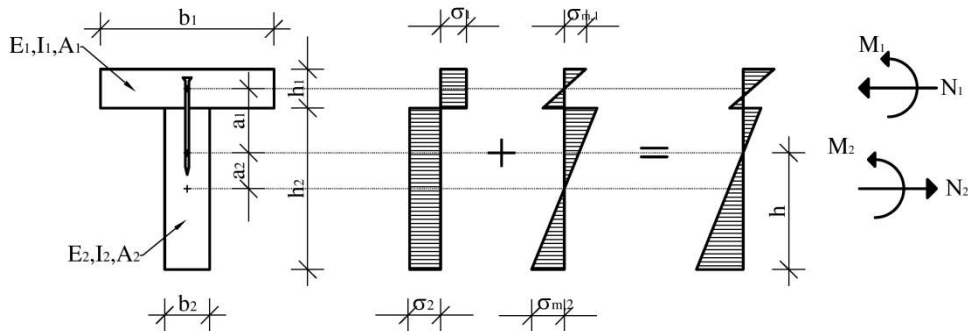


Figure 3 – Cross-section of TCC in "γ" method

Eurocode 5 suggests values for "slip modulus" as a function of connector's type, its diameter and specific mass weight of wood, in cases when experimental data are not available. According to EC5, the " K_{ser} " value for screws and predrilled nails - dowel type fasteners, is given by Eq (4) for SLS and by Eq (5) for ULS. Suggested equations are derived for timber-to-timber joints, and for TCC modelling the values for slip modulus have to be doubled.

$$K_{ser} = 2 \cdot \rho_m^{1.5} \cdot \frac{d}{23} \quad (4)$$

$$K_u = \frac{2}{3} \cdot K_{ser} \quad (5)$$

where:

K_{ser} is slip modulus for serviceability limit states,

K_u is instantaneous slip modulus for ultimate limit states.

1.3 EXPERIMENTAL RESEARCH ABOUT "SLIP MODULUS"

Extensive experimental campaigns conducted in last two decades (Gelfi & alt., 2002 [3], Dias, 2005 [4], Lukaszewska, 2009 [5], etc..) conflict with EC5 suggestions for "slip modulus", finding it quite conservative and underestimated for TCC systems. Some of researchers suggest that the slip modulus should be determined in terms of fastener diameter and timber's modulus of elasticity.

The proposed values in EC5 do not include the presence of formwork in cross-section, Fig 4a), so the additional research (Gelfi & alt., 2002) [3], gives the theoretical background and experimental results for stud connector's stiffness with existing formwork, Fig 4b), Eq (6):

$$K_s = \frac{124000 \cdot d}{(4.34 + \frac{t}{d})^3} \quad (6)$$

where:

$K_s = K_{ser}$ is connectors stiffness - slip modulus for serviceability limit states,
 t is depth of existing formwork,
 d is stud (dowel) diameter.

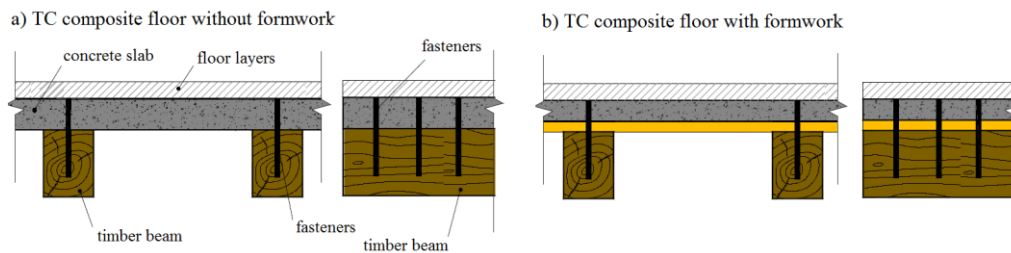


Figure 4 – TC composite floor without/with presence of formwork

EC5 gives the constant value for slip modulus and bearing capacity of connections, while model proposed by Gelfi & alt. introduce the variable value of slip modulus bearing capacity of connections, depending on planks thickness, Fig 5. Results obtained by EC5 show the decrease of cross-sectional stresses in cases when plank depth increase, while Gelfi's model for K_s gives more realistic description for stress distribution related to applied plank thickness [6].

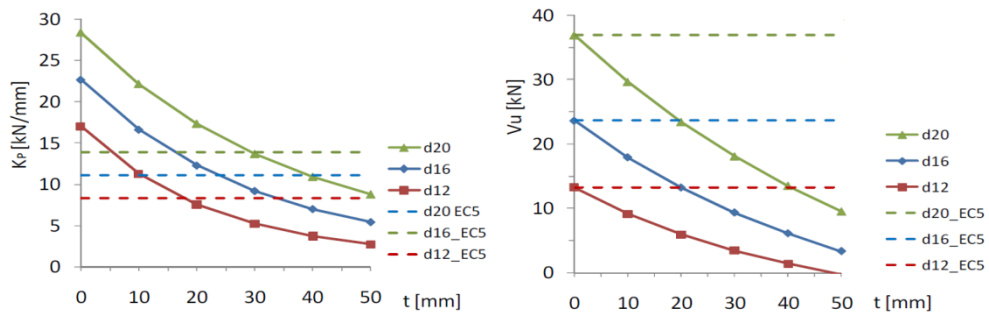


Figure 5 - Stiffness variation (K_{ser}) and the bearing capacity (V_u) of connection in function of the thickness of planks (t)

Many other experimental and numerical studies have also highlighted how the increase of screw inclination provides an increase of the resistance and stiffness of the joints, Fig 6. In the case of crossed screws subjected to shear-tension stress or X-shape positioned, the EC5 formula is completely unsuitable because it significantly underestimates the slip modulus.

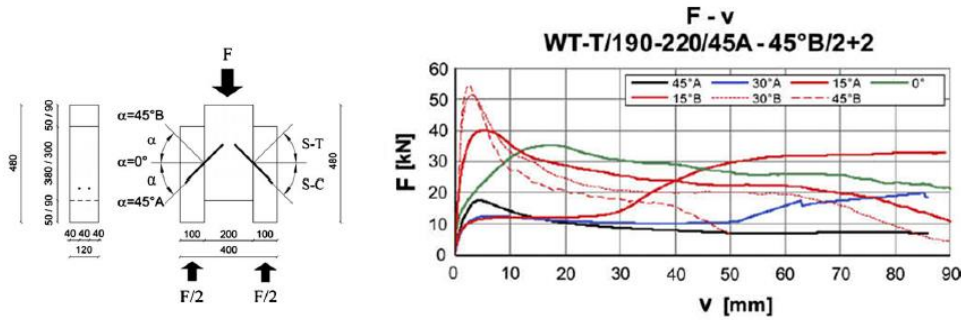


Figure 6 – Average force versus displacement, $F-v$, curves for various inclinations α , symbols are: S-T and $\alpha^\circ B$ for shear-tension screws; S-C and $\alpha^\circ A$ for shear-compression screws [7]

Suggested expressions, Eqs (4-5) by EC5, represent the slip modulus in shear, i.e. $K_{lat,ser} = K_{ser}$ that considers only the vertical position (perpendicular to the timber grain direction) of mechanical fastener, Fig 2. For the very common practical cases of inclined and crossed dowel type fasteners in TCC systems, Fig 2, there is no suggested values for axial slip modulus $K_{ax,ser}$ in EC5. Tomasi & alt., 2010 [7] proposed expressions for calculation "slip modulus" of inclined screws $K_{ser,\alpha}$, which depends on the axial and lateral stiffness, Eq (7):

$$K_{ser,\alpha} = K_{lat,ser} \cdot \cos \alpha \cdot (\cos \alpha - \mu \cdot \sin \alpha) + K_{ax,ser} \cdot \sin \alpha \cdot (\sin \alpha - \mu \cdot \cos \alpha) \quad (7)$$

where:

$K_{lat,ser}$ is lateral (shear) stiffness,

$K_{ax,ser}$ is axial stiffness,

α is angle of inclination,

μ is friction coefficient at the interface between concrete and timber.

Findings by Tomasi & alt., 2010 [7], shown that:

- for timber in shear-compression the slip modulus value could be adopted as slip modulus in shear, Fig 7(a), i.e. according to Eq (4),
- for timber in shear-tension the stiffness grows with angle, reaching maximum in $\alpha=45^\circ$, Fig 7(b), i.e. according to Eq (7),
- X crossed fasteners give the maximum values of stiffness, Fig 7(c), i.e. according to Eq (7), where is $\mu=0$.



Figure 7 – Inclined fasteners - arrangements: a) compression, b) tension, c) cross X [7]

2 RELATIONSHIP BETWEEN SLIP MODULUS, EFFECTIVE BENDING STIFFNESS AND DEGREE OF COMPOSITE ACTION

From the Eq (1-2) presented in "γ" method, it becomes evident that the degree of composite action increases with the increase of the slip modulus. However, the effective bending stiffness of composite structure is not in linear correlation with connection stiffness. Bending stiffness of TCC system could be increased up to max 4 times (Van der Linden, 1999 [8]), Fig 8 (left).

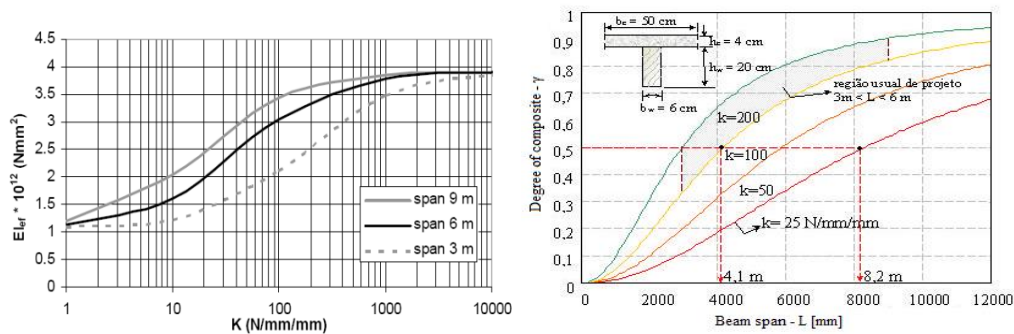


Figure 8 – Correlation between effective bending stiffness and connections stiffness of TCC floors [4] and relationship between span and degree of composite action [10]

This value occurs, however, only for certain combinations of geometric and material properties i.e. in cases of adequate choice of timber and concrete element's proportions. The proper choice of proportions could lead directly to decrease of the deformations (max 75%), what is the important issue in design of TCC, as well as to the increased bending stiffness. From the Fig 8 (left) is clear that above certain levels the increase of the slip modulus becomes useless due to the small effect that it has on the bending stiffness of the composite structure. On the other hand, below certain values of the slip modulus the composite action becomes negligible and thus is not worth to use composite systems.

Degree of composite action provided by the shear connection and spacing is measured using the Eq (8) proposed by Lukaszewska, 2009 (Piazza, 1983):

$$\gamma_s = \frac{EI_{eff} - EI_{min}}{EI_{max} - EI_{min}} \quad (8)$$

where:

EI_{min} is the bending stiffness for a perfect composite action,

EI_{eff} is the bending stiffness for the partial composite action,

EI_{max} is the bending stiffness for a zero composite action.

Another phenomenon, highlighted by the Almeida et al. [10], is the influence of beam span in the connection performance between the elements of the composite section. In Fig 8 (right) demonstrates the effect of the beam span on the degree of composite "γ" in TCC. For smaller spans than two meters it is very difficult to achieve a degree of composite "γ" higher than 0.33, which means that achieves a gain in stiffness of 33% in the composite section,

regardless of the type of connector and adopted spacing. For longer spans between 6m and 12m, it becomes much easier to achieve full interaction between the components.

In optimum design, increased stiffness is governed by K_{ser} and will in practice always less than the theoretical maximum of four. The maximum value can be obtained by the expression Eq (9), found in [10]:

$$\left(\frac{EI_{eff}}{EI_{min}} \right)_{max} = 1 + 3 \cdot \gamma_s \quad (9)$$

where:

γ_s is degree of composite action,

EI_{min} is the bending stiffness for a perfect composite action,

EI_{eff} is the bending stiffness for the partial composite action.

3 RELATIONSHIP BETWEEN PROPORTIONS OF COMPOSITE MEMBERS

As the main flexural members, glulam beams contribute significantly to the effective bending stiffness EI_{eff} of the composite TCC system. Van der Linden, 1999 [8] has found that adding a thin concrete slab to a deep timber beam or connecting a shallow timber beam to a heavy concrete slab does not lead to effective cooperation between the materials. The diagrams in Fig 9 (left) show that for a certain width ratio $n \cdot bc/bt$, there is a specific depth ratio hc/ht that will generate the optimum bending stiffness of a fully composite beam ($n = Ec/Et$).

This finding is also valid in the case of a partially composite beam. Shown in Fig 9 (right), for a partially composite beam with a given span L , connection spacing s and width ratio $n \cdot bc/bt=16$ (optimum), the depth ratio that gives the optimum bending stiffness is approximately constant (about 0,3) regardless of variations in the connection stiffness K_{ser} .

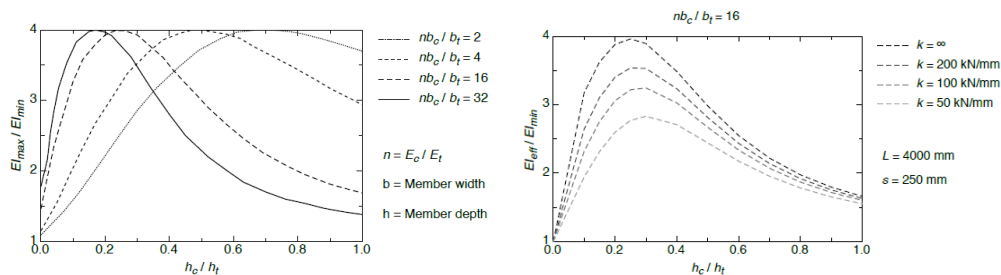


Fig 9 - Relationship between the proportions of the members - influence to the effective system bending stiffness: for a fully composite beam (left); for a partially composite beam (right) [9]

In order to better understand the behavior of the composite system Dias, 2005 [4] reports the trend of the shear force acting on the connector element and the maximum bending stress in the timber beam in function of the thickness of the slab, depth of the beam and the spacing of the screws. Fig 10 shows that: (a) if the thickness of the concrete slab is increased, the mid-span deflection and the maximum shear force acting on the connector will decrease, (b) if the

depth of beam is increased the mid-span deflection will be reduced and the shear force on the connector will decrease as well, (c) by reducing either the number of the screws or the spacing between the screws the mid-span deflection and the shear force on the connectors increase.

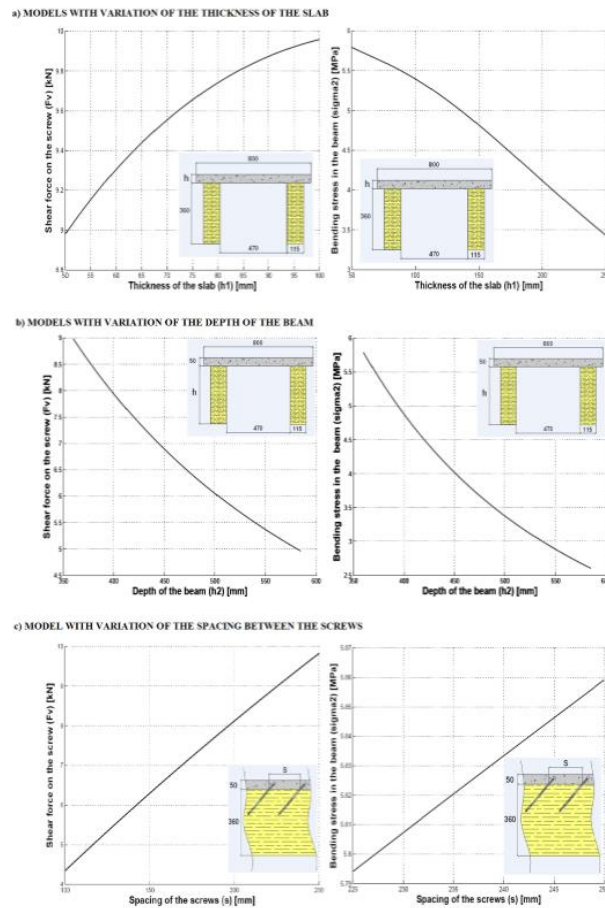


Fig 10 - The trend of the shear force acting on the connector element and the maximum bending stress in the timber beam [4]

4 CONCLUSION

The main conclusions of the paper are:

1. EC 5 suggestions for TCC slip modulus are conservative, applicable only for vertical positions of dowel fasteners and do not take into account the presence of formwork;
2. Additional experimental and theoretical researches are contributing to form a database necessary for developing optimal and practical expressions for slip modulus determination for all type of shear connectors in TCC;

3. "Gamma" method, although it is simplified linear procedure, is very acceptable and efficient method of calculation for practical purposes and on the safe side;
4. Geometric cross-section proportions of TCC members, as well as length of the beams, are very important issue because they could significantly influence the level of composite action apart from types, stiffness and arrangement of used fasteners;
5. Integrated knowledge about all aspects of TCC systems will result with particular part of EC for timber-concrete composites, that will promote and make available this sustainable and effective construction technique in the practice.

ACKNOWLEDGEMENT

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PRIMJENA ANALITIČKOG RJEŠENJA U ANALIZI SLOJEVITIH KOMPOZITNIH PLOČA

Rezime:

S-41

Analitičko rješenje problema savijanja slojevitih kompozitnih ploča u Teoriji slojeva dobija se primjenom dvostrukih trigonometrijskih redova. U ovom radu su prikazani i analizirani rezultati dobijeni primjenom programa u FORTRAN-u ANSLACOP (ANalitical Solution of LAMinated COMposite Plates) koji je napravljen od strane autora ovog rada sa teorijskim osnovama datim u Reddy-jevoj teoriji slojeva. U radu je prikazana promjena ugiba i napona u sredini i blizu ivica slobodno oslonjene kvadratne ploče u zavisnosti od debljine ploče, broja i rasporeda slojeva. Na osnovu sprovedenih razmatranja i analize rezultata na kraju rada dati su zaključci.

Ključne reči: Slojevita kompozitna ploča, teorija slojeva, sloj, analitičko rješenje

ANALYSIS OF LAMINATED COMPOSITE PLATES USING ANALYTICAL SOLUTION

Summary:

Analytical solution to the problem of bending for laminated composite plates in the Layerwise theory is obtained by using double trigonometric series. This paper presents results and analysis of the results calculated using program ANSLACOP (ANalitical SOLUTION of LAMinated COMposite Plates) created by the author of this paper. Program is based on assumptions given in Raddy's Layerwise theory. Changes of the deflection and stresses in the middle plate and stresses close to the edges of a simply supported square plate, depending on the plate thickness, number and layout of the layers are presented. Based on analysis of results conclusions are derived and presented at the end of the paper.

Key words: Laminated composite plates, Layerwise theory, layer, analytical solution

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1 UVOD

Kompozit je materijalna struktura koja se dobija kombinacijom dva ili više materijala na makro nivou. I ako je ideja kombinovanja materijala, u cilju dobijanja materijala sa boljim karakteristikama, poznata od ranije, istorija savremenih kompozita je stara svega nekoliko decenija. Jedan od najčešće primjenjivanih klasičnih kompozitnih materijala u građevinarstvu je armirani beton koji se dobija kombinacijom betona i armature. Za razliku od konstrukcija napravljenih od klasičnih materijala, kompozitne konstrukcije napravljene od savremenih materijala (pr. staklena ili ugljena vlakna u osnovnoj epoksi masi) posjeduju manju težinu, veću nosivost i veću otpornost. Pored dobrih i povoljnijih karakteristika, ove konstrukcije imaju i nedostatke vezano za pojavu prslina i praznina koji mogu nastati na vezama slojeva u fazi proizvodnje i eksploatacije, a koji značajno utiču na nosivost zbog značajno manje smičuće krutosti u odnosu na krutost na zatezanje.

Za proračun uticaja u presjeku kod umjereno debelih i debelih savremenih slojevitih kompozitnih ploča, sa slojevima koji nose u jednom pravcu i koje su napregnute na savijanje, mogu se primijeniti 3D teorije, jednoslojne teorije ESLT (Equivalent Single Layer Theory) [1-6] i teorije slojeva LWT (Layerwise theory) [7-9]. Zbog ekonomičnosti, slojevite kompozitne ploče napregnute na čisto savijanje obično se modeluju primenom ESLT teorija i LWT teorije slojeva. U ESLT jednoslojnim teorijama ploča se razmatra kao cjelina ekvivalentne krutosti zbog čega se ovim teorijama ne može prikazati stvarno naponsko deformacijsko stanje unutar ploče, a ne mogu se ni obuhvatiti veći broj problema koji se mogu javiti na nivou sloja. U ESLT teorije spadaju CLPT (Classical Laminated Plate Theory), FSDT (First-order Shear Deformation Theory) i HSDT (High-order Shear Deformation Theory) teorije. U LWT teorijama razmatranja se sprovode na nivou sloja zbog čega se, u zavisnosti od nivoa zahtjevana tačnosti, mogu uvesti u proračun i veći broj uočenih problema na nivou sloja. U ovom radu se primjenjuje teorija slojeva koju je razvio J.N.Reddy [7-9].

2 TEORIJSKE OSNOVE

Razmatra se pravougaona slojevita kompozitna ploča sa N elastičnih ortotropnih i ortogonalnih slojeva opterećena upravno na svoju ravan. U radu je primijenjena Reddy-jeva Parcijalna teorija slojeva, a sva razmatranja se sprovode u pravouglom koordinatnom sistemu (x,y,z).

Za k-ti ortotropni sloj veze napona i defomacija definisane su na sljedeći način:

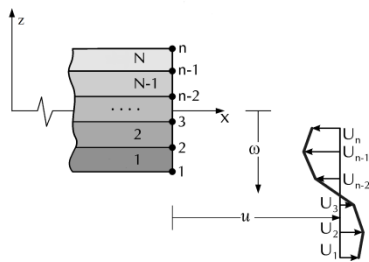
$$\begin{Bmatrix} \sigma_{xx} \\ \sigma_{yy} \\ \sigma_{xy} \\ \sigma_{yz} \\ \sigma_{xz} \end{Bmatrix}^k = \begin{bmatrix} \bar{Q}_{11} & \bar{Q}_{12} & \bar{Q}_{16} & & & \\ & \bar{Q}_{12} & \bar{Q}_{22} & \bar{Q}_{26} & & \\ & \bar{Q}_{16} & \bar{Q}_{26} & \bar{Q}_{66} & & \\ & & & & \bar{Q}_{44} & \bar{Q}_{45} \\ & & & & \bar{Q}_{45} & \bar{Q}_{55} \end{bmatrix}^k \begin{Bmatrix} \varepsilon_{xx} \\ \varepsilon_{yy} \\ 2\varepsilon_{xy} \\ 2\varepsilon_{yz} \\ 2\varepsilon_{xz} \end{Bmatrix}^k \quad (1)$$

gdje su \bar{Q}_{ij}^k transformisane materijalne karakteristike k-tog sloja [7].

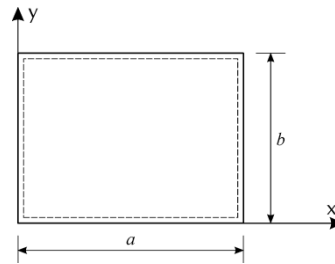
Prepostavlja se da je izduženje upravno na ravan ploče zanemarljivo iz razloga što se razmatra problem savijanja ploče zbog čega je komponentalno pomjeranje $w(x,y)$ konstantno po debljini ploče dok su preostala dva komponentalna pomjeranja $u(x,y,z)$ i $v(x,y,z)$ promjenljiva po debljini ploče [10]. S obzirom da se radi o linearnoj teoriji ukupno pomjeranje

u ravni ploče dobija se kao zbir pomjeranja srednje ravni ploče $u(x,y)$, $v(x,y)$ i dodatnih pomjeranja po debljini ploče $U(x,y,z)$, $V(x,y,z)$, slika 1:

$$\begin{aligned} u(x,y,z) &= u(x,y) + U(x,y,z) \\ v(x,y,z) &= v(x,y) + V(x,y,z) \\ w(x,y,z) &= w(x,y) \end{aligned} \quad (2)$$



Slika 1: Dodatno komponentalno pomjeranje U po debljini ploče



Slika 2: Geometrija ploče

Promjena dodatnih pomjeranja pretpostavlja se u obliku sume

$$U(x,y,z) = \sum_{J=1}^n u^J(x,y)\psi^J(z) \quad V(x,y,z) = \sum_{J=1}^n v^J(x,y)\psi^J(z) \quad (3)$$

gdje su u^J i v^J nepoznate veličine, a $\psi^J(z)$ neprekidne funkcije koje zadovoljavaju uslov $\psi^J(0)=0$ za $J=1,n$. Pretpostavka (3) predstavlja diskretizaciju po debljini ploče kojom se 3D model prevodi u kvazi 3D model. U ovom izrazu $\psi^J(z)$ je opšta interpolaciona funkcija dok su u^J i v^J čvorne vrijednosti pomjeranja po debljini razmatrane ploče.

U radu se usvaja diskretizacija pomjeranja po debljini ploče linearnim interpolacionim funkcijama duž dva susjedna sloja J i $J-1$:

$$\phi_z^J = \begin{cases} \psi_2^{J-1}(z) = \frac{z - z_{J-1}}{z_J - z_{J-1}}, & z_{J-1} < z < z_J \\ \psi_2^J(z) = \frac{z_{J+1} - z}{z_{J+1} - z_J}, & z_J < z < z_{J+1} \end{cases} \quad (4)$$

gdje su: z - promjenljiva, z_J - koordinata čvora J , $J=1,n$.

Zbog usvojene pretpostavke vektor deformacije ima pet članova. Vektor deformacija, zbog pretpostavljene promjene pomjeranja koja je definisana relacijom (2), definiše se preko vektora deformacije srednje ravni ploče i vektora deformacije definisanog po debljini ploče:

$$\{\mathcal{E}\}^T = \{u_{,x}; v_{,y}; (u_{,y} + v_{,x}); w_{,x}; w_{,y}\} \quad \{\mathcal{E}_k\}^T = \{U_{K,x}; V_{K,y}; (U_{K,y} + V_{K,x}); U_K; V_K\} \quad (5)$$

3 JEDNAČINE PROBLEMA I ANALITIČKO RJEŠENJE

Diferencijalne jednačine problema savijanja slojevitih kompozitnih ploča se dobijaju primjenom principa virtualnih pomjeranja [9-10]:

$$\begin{aligned}
N_{x,x} + N_{xy,y} = 0 \quad N_{xy,x} + N_{y,y} = 0, \quad Q_{x,x} + Q_{y,y} + q = 0 \\
N_{x,x}^j + N_{xy,y}^j - Q_x^j = 0 \quad N_{xy,x}^j + N_{y,y}^j - Q_y^j = 0
\end{aligned} \tag{6}$$

gdje je j oznaka sloja, $j=1,\dots,N$, a q zadato opterećenje upravno na ravan ploče. Nepoznate sile u proizvoljnom presjeku se određuju kao integrali napona:

$$\begin{aligned}
(N_x, N_y, N_{xy}) = \int_{-h/2}^{h/2} (\sigma_x, \sigma_y, \sigma_{xy}) dz \quad (Q_x, Q_y) = \int_{-h/2}^{h/2} (\sigma_{xz}, \sigma_{yz}) dz \\
(N_x^j, N_y^j, N_{xy}^j) = \int_{-h/2}^{h/2} (\sigma_x, \sigma_y, \sigma_{xy}) \mu^j(z) dz \quad (Q_x^j, Q_y^j) = \int_{-h/2}^{h/2} (\sigma_{xz}, \sigma_{yz}) \mu^j(z) dz
\end{aligned} \tag{7}$$

Da bi presječne sile bile definisale u funkciji nepoznatih pomjeranja potrebno je kombinovati relacije (1), (2), (3), (5) i (7). Dobijeni izrazi predstavljaju vezu sila i pomjeranja iz kojih se, kada se uvrste u jednačine (6), dobija sistem od $3+2n$ jednačina sa isto toliko nepoznatih pomjeranja (u, v, w) i (U^j, V^j) za $J=1, n$):

$$\begin{aligned}
A_{11} u_{,xx} + A_{12} v_{,yx} + A_{66} (u_{,yy} + v_{,xy}) + \sum_{J=1}^n [B_{11}^J u_{,xx}^J + B_{12}^J v_{,yx}^J + B_{66}^J (u_{,yy}^J + v_{,xy}^J)] = 0 \\
A_{12} u_{,xy} + A_{22} v_{,yy} + A_{66} (u_{,yx} + v_{,xx}) + \sum_{J=1}^n [B_{12}^J u_{,xy}^J + B_{22}^J v_{,yy}^J + B_{66}^J (u_{,yx}^J + v_{,xx}^J)] = 0
\end{aligned} \tag{8a}$$

$$\begin{aligned}
A_{55} w_{,xx} + A_{44} w_{,yy} + \sum_{J=1}^n [B_{55}^J u_{,x}^J + B_{44}^J v_{,y}^J] + q = 0 \\
B_{11}^J u_{,xx} + B_{12}^J v_{,yx} + B_{66}^J (u_{,yy} + v_{,xy}) - B_{55}^J w_{,x} + \sum_{J=1}^n [D_{11}^{II} u_{,xx}^J + D_{12}^{II} v_{,yx}^J + D_{66}^{II} (u_{,yy}^J + v_{,xy}^J) - D_{55}^{II} u^J] = 0
\end{aligned} \tag{8b}$$

$$B_{11}^J u_{,xy} + B_{212}^J v_{,yy} + B_{66}^J (u_{,yx} + v_{,xx}) - B_{44}^J w_{,y} + \sum_{J=1}^n [D_{12}^{II} u_{,xy}^J + D_{22}^{II} v_{,yy}^J + D_{66}^{II} (u_{,yx}^J + v_{,xx}^J) - D_{44}^{II} v^J] = 0$$

date jednačine se definišu za tačku u ravni ploče sa koordinatama (x, y) . U jednačinama (8a,b) veličine:

$$A_{ij} = \sum_{k=1}^N \bar{Q}_{ij}^k (z_{k-1} - z_k) \tag{9a}$$

$$B_{ij}^J = \bar{Q}_{ij}^{J-1} \frac{(z_J - z_{J-1})}{2} + \bar{Q}_{ij}^J \frac{(z_{J+1} - z_J)}{2} \text{ za } (i, j = 1, 2, 6), \quad B_{ij}^J = \bar{Q}_{ij}^{J-1} - \bar{Q}_{ij}^J \text{ za } (i, j = 4, 5)$$

$$\text{za } J = I \quad D_{ij}^{II} = \bar{Q}_{ij}^{J-1} \frac{(z_J - z_{J-1})}{3} + \bar{Q}_{ij}^J \frac{(z_{J+1} - z_J)}{3} \quad (i, j = 1, 2, 6), \quad D_{ij}^{II} = \frac{\bar{Q}_{ij}^{J-1}}{(z_J - z_{J-1})} + \frac{\bar{Q}_{ij}^J}{(z_{J+1} - z_J)} \quad (i, j = 4, 5)$$

$$\text{za } J = I \pm 1 \quad D_{ij}^{II} = D_{ij}^{II} = \bar{Q}_{ij}^J \frac{(z_{J+1} - z_J)}{6} \quad (i, j = 1, 2, 6), \quad D_{ij}^{II} = D_{ij}^{II} = -\frac{\bar{Q}_{ij}^J}{(z_{J+1} - z_J)} \quad (i, j = 4, 5) \tag{9b}$$

predstavljaju krutosti materijala koje su date u zavisnosti od \bar{Q}_{ij}^J redukovane krutosti J -tog sloja (1), [5].

Za pravougaonu slobodno oslonjenu ploču sa N ortotropnih slojeva (slika 2) granični uslovi su:

$$\begin{aligned} v = w = V^J = N_x = N_x^J = 0 & \quad x = 0, a; J = 1, n \\ u = w = U^J = N_y = N_y^J = 0 & \quad y = 0, b; J = 1, n \end{aligned} \quad (10)$$

Uslovi (10) su identički zadovoljeni za pomjeranja pretpostavljena u obliku dvostrukih trigonometrijskih redova [10-12]:

$$\begin{aligned} u = \sum_{m,n} X_{mn} \cos \alpha x \sin \beta y \quad v = \sum_{m,n} Y_{mn} \sin \alpha x \cos \beta y \quad w = \sum_{m,n} W_{mn} \sin \alpha x \sin \beta y \\ U^J = \sum_{m,n} R_{mn}^J \cos \alpha x \sin \beta y \quad V^J = \sum_{m,n} S_{mn}^J \sin \alpha x \cos \beta y \quad \text{gdje su } \alpha = \frac{m\pi}{a} \quad \beta = \frac{n\pi}{b} \end{aligned} \quad (11)$$

gdje je $J=1, n$ je oznaka čvora.

Transverzalno opterećenje prikazuje se u obliku dvostrukog trigonometrijskog reda:

$$q(x, y) = \sum_{m,n} Q_{mn} \sin \alpha x \sin \beta y \quad (12)$$

Q_{mn} su koeficijenti koji se dobijaju integracijom i zavise od funkcije promjene opterećenja.

Kada se odrede izvodi izraza (11), i sa relacijom (12) uvrste u jednačine ravnoteže (8a) i (8b) nakon sređivanja za svaki Furie-ov par (m,n) ispisuje se sistem od $3+2n$ jednačina sa nepoznatim koeficijentima X_{mn} , Y_{mn} , W_{mn} , R_{mn}^J i S_{mn}^J , $J=1, n$. Sljedeći korak je određivanje pomjeranja primjenom izraza (3) i (2) a onda i deformacija prema (5) i napona prema (1). Postupak se ponavlja za svaki par (m,n). Proračun tačnijeg rasporeda smičućih napona $\bar{\sigma}_{xz}$ i $\bar{\sigma}_{yz}$ po visini presjeka, nego onog datog relacijom (5) u prikazanom modelu autor ovog rada je koristio poseban poluanalitički postupak [11], [13].

Prikazano analitičko rješenje za proračun pomjeranja i napona je automatizovan izradom sopstvenog kompjuterskog programa napisanog u programskom jeziku Fortran, program ANSLACOP. Vrijednosti pomjeranja i napona prikazane u ovom radu su sračunate primjenom ovog programa i imaju dobro poklapanje sa vrijednostima datim u literaturi.

4 NUMERIČKI PRIMJERI I VERIFIKACIJA REZULTATA

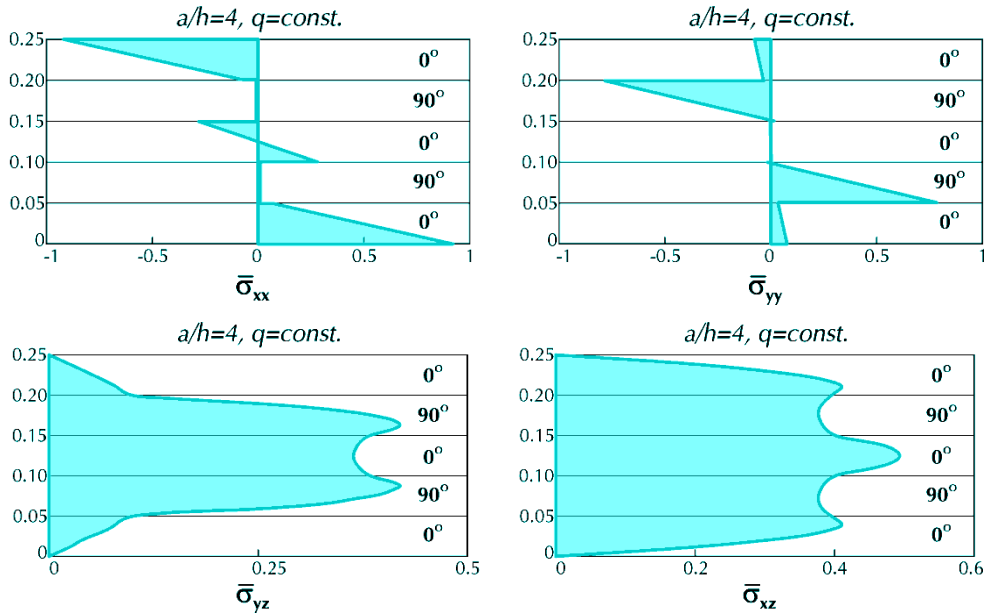
Razmatra se kvadratna ploča dimenzija $axa=1x1$ sa pet i šest naizmenično poredanih slojeva ugla orijentacije 0^0 i 90^0 . Usvaja se materijal sljedećih karakteristika: $E_x=25$, $E_y=1$, $G_{xy}=G_{xz}=0.5$, $G_{yz}=0.2$, $\nu_{xy}=\nu_{xz}=0.25$.

Pomjeranja i naponi prikazuju se u bezdimenzionom obliku:

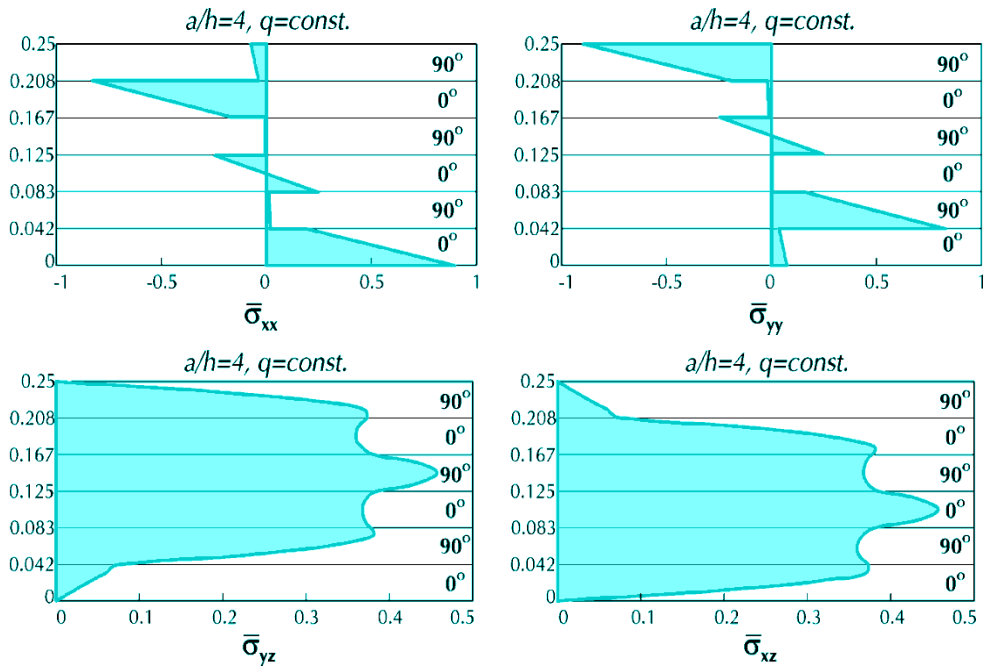
$$\bar{w} = \frac{100E_2}{q_0hs^4} w \quad \bar{\sigma}_{xx} = \frac{1}{s^2q} \sigma_{xx} \quad \bar{\sigma}_{yy} = \frac{1}{s^2q} \sigma_{yy} \quad \bar{\sigma}_{xy} = \frac{1}{sq} \sigma_{xy} \quad \bar{\sigma}_{xz} = \frac{1}{sq} \sigma_{xz} \quad \bar{\sigma}_{yz} = \frac{1}{sq} \sigma_{yz} \quad (13)$$

gdje je $s=a/h$, u tačkama sa koordinatama:

$$\bar{\sigma}_{xx} = \bar{\sigma}_{xx}(A, A, z), \bar{\sigma}_{xy} = \bar{\sigma}_{xy}(B, B, z), \bar{\sigma}_{xz} = \bar{\sigma}_{xz}(B, A, z), A = 1.105662(a/2), B = 1.894338(a/2) \quad (14)$$



Slika 3: Promjena napona $\bar{\sigma}_{xx}$, $\bar{\sigma}_{yy}$, $\bar{\sigma}_{yz}$, $\bar{\sigma}_{xz}$ po debljini ploče $0^\circ/90^\circ/0^\circ/90^\circ/0^\circ$, $a/h=4$



Slika 4: Promjena napona $\bar{\sigma}_{xx}$, $\bar{\sigma}_{yy}$, $\bar{\sigma}_{yz}$, $\bar{\sigma}_{xz}$ po debljini ploče $0^\circ/90^\circ/0^\circ/90^\circ/0^\circ/90^\circ$, $a/h=4$

Na slikama 3 i 4 grafički je dat prikaz promjene bezdimenzionih napona $\bar{\sigma}_{xx}$, $\bar{\sigma}_{yy}$, $\bar{\sigma}_{yz}$ i $\bar{\sigma}_{xz}$ po debljini kvadratnih ploča $0^0/90^0/0^0/90^0/0^0$ i $0^0/90^0/0^0/90^0/0^0/90^0$, za odnos dužine i debljine $a/h=4$, u definisanim tačkama (14) i usled dejstva ravnomjerno podijeljenog jediničnog opterećenja q .

U Tabeli 1 dato je upoređenje bezdimenzionih vrijednosti ugiba \bar{w} za troslojnu kvadratnu ploču $0^0/90^0/0^0$ odnosa $a/h=4$ i $a/h=10$, koje su dobijene prezentiranim analitičkim postupkom PLWT teorije sa vrijednostima dobijenim primjenom klasičnih teorija CLPT i FSDT i TSDT (Third-order shear deformation plate theory), [1]. Razlika između vrijednosti dobijene prezentiranom PLWT teorijom i jednoslojnom teorijom smicanja trećeg reda TSDT za odnos $a/h=4$ je 2,298%, a za odnos $a/h=10$ je 3,742%. Razlika između vrijednosti dobijene prezentiranim modelom PLWT teorije i jednoslojne teorije smicanja prvog reda FSDT za odnos $a/h=4$ je 2,278%, a za odnos $a/h=10$ je 6,120%. Najveća razlika se dobija poređenjem vrijednosti dobijenih prezentiranim modelom sa CLPT jednoslojnom teorijom i iznosi za 78,073% za odnos $a/h=4$, dok je razlika 41,731% za odnos $a/h=10$.

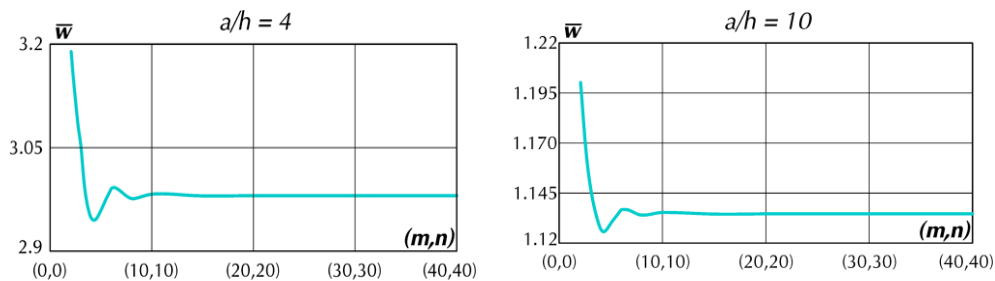
Tabela 1: Ugib $\bar{w}(a/2;b/2)$ usled dvostrukog sinusoidalnog opterećenja, slojevita troslojna ploča $0^0/90^0/0^0$

a/h	4	10
CLPT*	0.4313	0.4313
FSDT*	1.9122	0.6949
TSDT*	1.9218	0.7125
PLWT (predstavljeno analitičko rješenje)	1.9670	0.7402

* vrijednosti preuzete iz literature, [1], str. 612, Tabela 11.4-1.

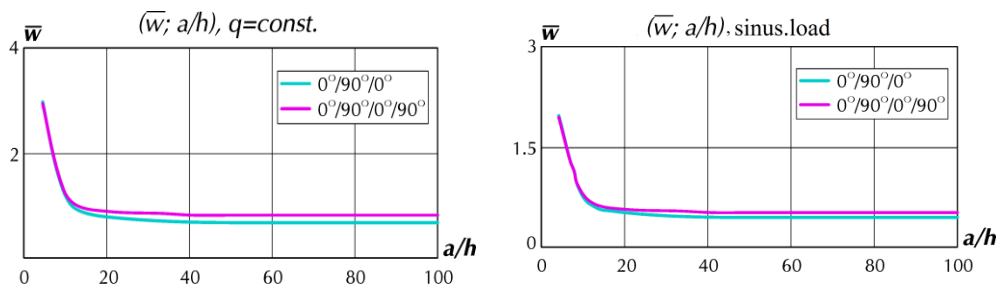
5 ANALIZA REZULTATA

Konvergenција analitičkog rješenja u zavisnosti od broja članova dvostrukog reda prikazana je na primjeru promjene bezdimenzionog ugiba \bar{w} u sredini kvadratne troslojne ploče $0^0/90^0/0^0$ dimenzija axa i odnosa $a/h=4$ i $a/h=10$. Zaključuje se da analitičko rješenje za \bar{w} veoma brzo konvergira tačnom rješenju, slika 5.



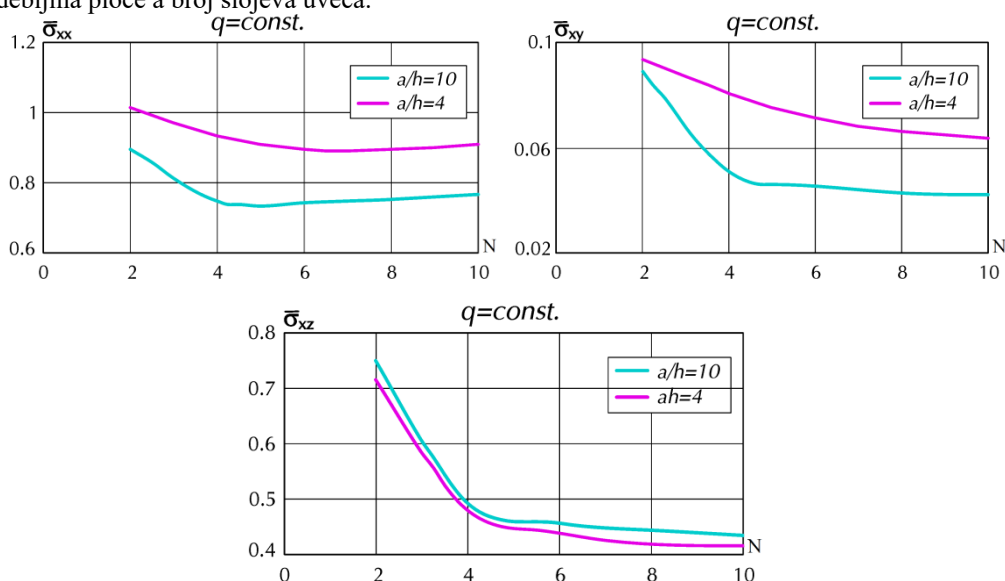
Slika 5: Konvergenција max.ugiba \bar{w} u funkciji broja članova reda (m,n) , $0^0/90^0/0^0$

Na slici 6 prikazana je promjena maksimalnog ugiba \bar{w} u zavisnosti od odnosa a/h za ploče sa tri sloja simetričnog rasporeda i četiri sloja antimetričnog rasporeda, i za slučaj dejstva ravnomjerno i sinusoidalno podijeljenog opterećenja. Za manji odnos a/h dobijaju se veće vrijednosti maksimalnog bezdimenzionog ugiba.



Slika 6: Promjena max.ugiba \bar{w} u funkciji odnosa a/h za ploče $0^\circ/90^\circ/0^\circ$ i $0^\circ/90^\circ/0^\circ/90^\circ$

Na osnovu analize promjene ugiba \bar{w} u sredini slojevite ploče, za unaprijed odabrane materijalne karakteristike, može se zaključiti da na maksimalan ugib utiču debljina ploče, broj slojeva, raspored slojeva (simetričan ili antimetričan) i vrsta opterećenja. Pokazuju se da za tanke ploče rješenje konvergira rješenju klasične jednoslojne teorije ploča. Ugib u sredini ploče se može smanjiti za 46,4% za odnos $a/h=10$ i 11,95% za odnos $a/h=4$, ako se zadrži ista debljina ploče a broj slojeva uveća.



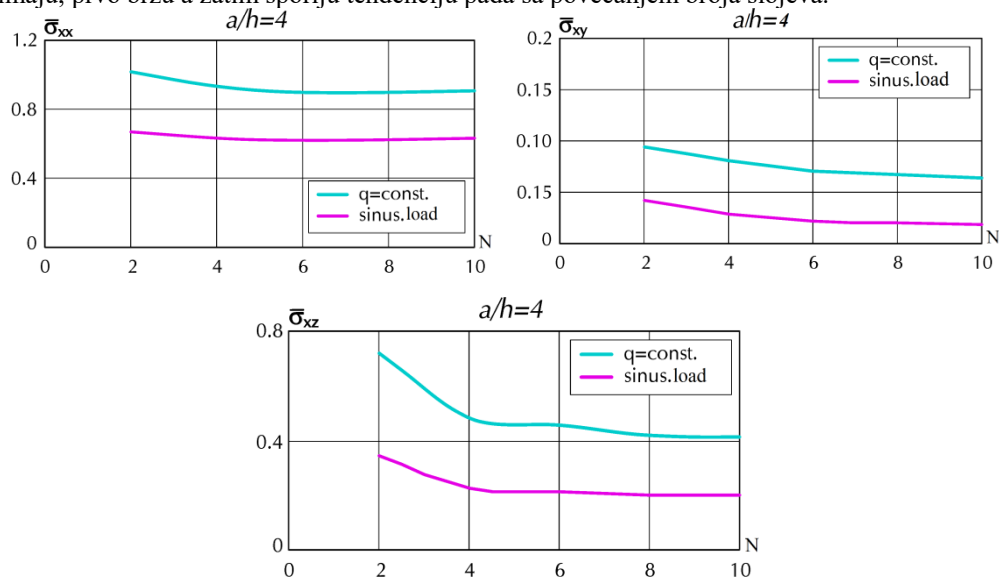
Slika 7: Promjena maksimalnih napona $\bar{\sigma}_{xx}$, $\bar{\sigma}_{xy}$, $\bar{\sigma}_{xz}$ u odnosu prema broju slojeva N za $a/h=4$ i $a/h=10$

Na slici 7 prikazana je promjena maksimalnih vrijednosti napona $\bar{\sigma}_{xx}$, $\bar{\sigma}_{xy}$, $\bar{\sigma}_{xz}$ u funkciji broja slojeva N , za ploče debljine $h=0,1a$ i $h=0,25a$, usled dejstva ravnomjernog

opterećenja, dok je na slici 8 prikazan raspored istih napona za slučaj dejstva sinusoidalno podijeljenog opterećenja u oba pravca. Analizom dobijenih rezultata zaključuje se da se povećanjem broja slojeva, a zadržavajući istu debljinu ploče, maksimalne vrijednosti napona u presjeku mogu značajno smanjiti.

Vrijednosti bezdimenzionih napona $\bar{\sigma}_{xx}, \bar{\sigma}_{xy}, \bar{\sigma}_{xz}$ u karakterističnim tačkama zavise od materijalnih karakteristika slojeva, broja i rasporeda slojeva, debljine ploče i vrste opterećenja. Dodavanjem novih slojeva opadaju maksimalne vrijednosti bezdimenzionih napona.

Umanjenje napona je ravnomjernije i manjeg intenziteta kod debljih ploča. Za tanke ploče vrijednost napona se ne mijenja naglo nakon dodatog četvrtog sloja. Smičući naponi imaju, prvo bržu a zatim sporiju tendenciju pada sa povećanjem broja slojeva.



Slika 8: Promjena maksimalnih napona $\bar{\sigma}_{xx}, \bar{\sigma}_{xy}, \bar{\sigma}_{xz}$ u odnosu na broj slojeva N za dejstvo sinusoidalnog i ravnomjerno raspodijeljenog opterećenja, $a/h=4$

6 ZAKLJUČAK

Rezultati dobijeni primjenom programa ANSLACOP, koji je napisan na osnovu Redy-jeve teorije slojeva, pokazuju da je analitičko rješenje stabilno i da veoma brzo konvergira tačnom rješenju. Pretpostavka da svaki sloj ima različitu promjenu pomjeranja omogućava da se dobije realniji raspored napona po debljini.

Analizom sračunatih vrijednosti zaključuje se da se brojem i rasporedom slojeva, za istu debljinu ploče i unaprijed odabrane materijalne karakteristike, vrijednosti pomjeranja i napona u presjeku mogu značajno umanjiti. Ovo omogućava projektantima da odaberu najpogodniju kompoziciju slojeva u zavisnosti od uslova definisanih projektним zadatkom.

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PRORAČUN ODVAJANJA VRTLOGA KOD VITKIH KONZOLNIH KONSTRUKCIJA PREMA EC1 METODAMA

Rezime:

S-42

Odvajanje vrtloga i uticaj koji ovaj fenomen ima na vitke čelične konstrukcije sa malim prigušenjem vrlo često predstavlja kritično i nedovoljno ispitano polje dinamičkog dejstva vjetrov kojim treba obratiti posebnu pažnju. Ovaj rad ima za cilj da, analizirajući dostupne metode proračuna iz Evrokoda, ukaže na dva fenomena. Prvi se odnosi na nepoznanice i nedostatke u primjeni spektralne metode, pogotovo u analizi vibracija konstrukcije u višim tonovima oscilovanja, dok se drugi odnosi na poboljšanje postojeće harmonijske metode u domenu sračunavanja inercijalnih sila koje pobuđuju konstrukciju i dovode do rezonantnog – lock in stanja.

Ključne riječi: odvajanje vrtloga, lock-in, rezonantna metoda, harmonijske sile

VORTEX SHEDDING CALCULATION OF SLENDER CANTILEVER STRUCTURES BASED ON EC1 METHODS

Summary:

The vortex shedding and the impact of this phenomenon on slender steel structures with small damping is often a critical and insufficiently examined area of dynamic wind action that needs particular attention. This paper aims to analyze the available calculation methods of Eurocode, and to point out to two phenomenons. The first relates to the uncertainties and weaknesses in the application of the spectral method, especially in the analysis of the structural vibrations in the higher modes of oscillation, while the second relates to the improvement of existing method in the field of calculation of inertial harmonic forces that give rise to the structural vibrations and lead to resonance – lock-in state.

Key words: vortex shedding, lock-in, resonant method, harmonic forces

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1 UVOD

Odvajanja vrtloga od vitkih konstruktivnih elemenata kao što su dimnjaci, vertikalni nosači za osvetljenje, dalekovodi, tornjevi i piloni predstavlja jedan od specifičnih a najmanje ispitanih fenomena u oblasti inženjeringa. Problem rezonancije u lock-in fazi često izaziva amplitude koje, iako sa aspekta nosivosti kratkotrajno ne mogu ugroziti nosač na globalnom ili lokalnom nivou, potencijalno predstavljaju opasnost usled efekta zamora materijala zbog prirode cikličnog oscilovanja konstrukcije u pobuđenom stanju. U cilju osiguranja ovakvih konstrukcija od neželjenih efekata koji mogu ugroziti njenu nosivost i stabilnost potrebno je sprovesti detaljna teorijska, numerička i eksperimentalna istraživanja.

Iako je veliki broj istaknutih naučnika i istraživača u poslednjih sto godina doprinio boljem razumijevanju ovog fenomena, nedoumice vezane za vrijednosti pojedinih parametara koje definišu ovakve oscilacije i dalje postoje. Ovakvoj tvrdnji govori u prilog činjenica da na globalnom nivou postoji barem pet različitih metoda proračuna [1] koje su implementirane u standarde pojedinih zemalja, dok Evrokod [2] definiše dvije metode (harmonijska i spektralna) ali bez jasnih preporuka o tome koju metodu kada primijeniti.

Ovaj rad će, pozivajući se na dosadašnja istraživanja ovog fenomena, pokušati da ponudi odgovor na pitanje ograničenosti upotrebe Spektralne metode (Metoda2) iz Evrokoda. U radu je dat kratak pregled dosadašnjih zapažanja i teorijske osnove sa posebnim osvrtom na izmjerene amplitude već izvedenih konstrukcija. Harmonijski model (Metoda1) je analiziran pomoću dva različita proračunska pristupa. Prvi pristup se zasniva na statičkom proračunu prema klasičnoj teoriji metode konačnih elemenata uz primjenu rešenja problema svojstvenih vrijednosti, izvođeci presječne sile iz sračunate kvantitativne forme oscilovanja [3]. Drugi pristup, koji predstavlja predlog autora ovog rada, zasniva se na teoriji oscilovanja linearno-elastične konstrukcije pri viskoznom prigušenju. U ovom proračunskom modelu, pojava vrtložnog dejstva se analizira uz pretpostavku određenog oblika odgovora sistema, u vremenski zavisnom dinamičkom proračunu, pri čemu je krajnji cilj sračunavanje naponskog stanja, tj. presečnih sila koje dovode do sračunate kvantitativne forme oscilovanja. Rezultati se prikazuju tabelarno i grafički na odabranim primjerima uz prikaz najznačajnijih rezultata.

2 PRORAČUN ODVAJANJA VRTLOGA PREMA EC1

2.1 HARMONIJSKA METODA – METODA 1

Harmonijska metoda iz EC1 bazirana je na radovima Scrutona, Rummana i Ruscheweyha. Zasniva se na pretpostavci da fenomen odvajanja vrtloga stvara sinusoidalne sile harmonijske prirode koje su upravne na pravac vjetra. Za razliku od spektralne metode determinističke je prirode i zasniva se na jasno definisanoj mehanici.

Prema [4] jednačina maksimalne amplitude bilo koje tačke konstrukcije u toku oscilovanja prouzrokovanog odvajanjem vrtloga glasi :

$$y_{\max}(z) = a_{\max} \phi_j(z) = \frac{\rho_a C_l b^3 \phi_j(z) \int_{z_1}^{z_2} \phi_j(z) dz}{16\pi^2 G_j \eta_j S_l^2} \quad (1)$$

gdje su: ρ_a - gustina vazduha; ϕ_j - vrijednost normirane forme oscilovanja u tački j ; b - širina presjeka upravno na pravac vjetra; S_i - Strouhalov broj; C_l - amplituda sinusoidalne sile po jedinici dužine; G_j - generalisana masa po jedinici dužine; η_j - kritično prigušenje j -tog oblika oscilovanja; z_1 i z_2 - gornja i donja granica oblasti u kojoj djeluje sila izazvana vrtožnim dejstvom.

Prema EC1 uticaj odvajanja vrtloga treba ispitati u slučaju kada je $v_{crit,i} < 1,25 \cdot v_m$ gdje su: v_m - srednja brzina vjetra, a $v_{crit,i} = b \cdot f / S_i$ kritična brzina vjetra

Za slučaj konzolnog nosača sa uniformnom masom, uz izražavanje amplitude vibracije kao dijela prečnika kritičnog presjeka u kom odvajanje vrtloga nastaje, formula (1) dobija jednostavniji oblik:

$$\frac{y_{max}}{b} = \frac{K \cdot K_w \cdot c_{lat}}{S_i^2 \cdot S_c} \quad (2)$$

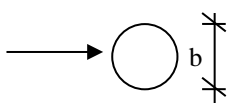
koji se prema [3] može prikazati u sljedećem obliku:

$$\frac{y_{max}}{b} = c_{lat} \frac{1}{S_i^2 S_c} \frac{\sum_{i=1}^n \left| \sum_k I_3^k \right|}{4\pi^2 \phi_r^T \phi_r} \frac{\phi_r^T \bar{P}}{\sum_{i=1}^n \left| \sum_k I_3^k \right|} \quad (3)$$

pri čemu je K_w koeficijent korelacije predstavljen u funkciji blok-sile definisane pomoću vrijednosti c_{lat} , kritične brzine koja dovodi do rezonantnog stanja i dimenzije kritičnog presjeka. Jednostavnim poređenjem se može zaključiti da su relacije (2) i (3) identične i da su u saglasnosti sa (1), [5]. U relaciji (2) c_{lat} predstavlja koeficijent bočne sile definisan Evrokodom. Razjašnjenje ostalih oznaka datih u jednačini (2) biće dato u tekstu koji slijedi.

U tabeli E1 iz [2]. dati su Strouhalovi brojevi za različite tipove poprečnih presjeka. Uočava se da je za kružni poprečni presjek ova vrijednost fiksna i iznosi 0.18 nezavisno od vrijednosti Reynoldsovog broja. Kako se ovakva tvrdnja ne slaže sa zaključcima iz dostupne literature [4,6,7,8], to se u ovom tekstu predlaže iterativna procedura data u [6] sve do dostizanja konvergencije u funkciji Reynoldsovog broja prema Tabeli 1.

Tabela 1 –Vrijednost Strouhalovog broja u funkciji Reynoldsovog broja

	$10^5 \leq R_e \leq 4 \cdot 10^5$	0,19
	$4 \cdot 10^5 \leq R_e \leq 10^6$	$-0,7674 + 0,1709 \log_{10}(R_e)$
	$10^6 \leq R_e \leq 1,6 \cdot 10^6$	$1,3752 - 0,1862 \log_{10}(R_e)$
	$R_e \geq 1,6 \cdot 10^6$	0,22

Vrijednost Scratonovog broja S_c se računava prema relaciji:

$$S_c = \frac{2\delta_s m_{i,e}}{\rho b^2} \quad (4)$$

Vrijednost S_c data je u funkciji ekvivalentne mase $m_{i,e}$ i prigušenja konstrukcije izraženog preko logaritamskog dekrementa prigušenja δ_s . Ukupno prigušenje sistema sadrži samo dio usled strukturalnog prigušenja dok se, za razliku od spektralne metode, aerodinamičko

prigušenje zanemaruje. Fundamentalni problem detaljnijeg proračuna u višim tonovima oscilovanja se odnosi na činjenicu da logaritamski dekrement prigušenja definisan Evrokodom daje smjernice samo za prvi, osnovni, ton oscilovanja (Tabela F2 iz [2]). Obzirom na navedeno, autori rada predlažu dopunsku metodu koja se zasniva na vremenski zavisnoj analizi uz pretpostavku viskoznog prigušenja prema Rayleigh-ovoj teoriji. Treba istaći da je preteča Evrokoda u domenu ovog proračuna definisala granicu za analizu uticaja odvajanja vrtoga u funkciji vrijednosti Scratonovog broja ali i same geometrije [10,11]. *Pri proračunu posebna pažnja treba da se obrati na konstrukcije kod kojih je ova vrijednost u prva 2 tona oscilovanja manja od 5.* U tom slučaju, savjetuju se *posebne konsultacije* sa stručnjacima iz oblasti dinamičkih uticaja pri opterećenju vjetrom.

K i K_w u formuli (2) predstavljaju koeficijent oblika tona oscilovanja i koeficijent efektivne dužine korelacije, respektivno. Dok za vrijednost koeficijenta oblika tona oscilovanja ne postoji dilema o načinu sračunavanja, za koeficijent K_w , sračunat kod konične konstrukcije prema [3] ili [2], treba istaći da dužina korelacije L_l za kritičnu oblast na vrhu mora biti sračunata iterativno kao poditeracija glavnog iterativnog proračuna koji podrazumijeva kontrolu amplitude u funkciji pretpostavljenih dužina korelacije, Tabela 2.

Tabela 2 – Efektivna dužina korelacije kao f-ja amplitude oscilovanja

$y_f(s_i)/b$	L_l/b
< 0,1	6
Od 0,1 do 0,6	$4,8 + 12 + y_f(s_i)/b$
>0,6	12

Ova procedura se predlaže zbog činjenice da Evrokod 1 definiše srednju brzinu vjetra v_m u zoni blizu vrha konstrukcije u sredini dužine korelacije, za šta se predlaže primjena početnog uslova kritičnog presjeka koji je definisan u funkciji koniciteta konstrukcije k i prečnika na vrhu konstrukcije d_{top} :

$$b_{crit,1} = b_{top} / (1 - 0,003 \cdot k) \quad (5)$$

Iz sračunate amplitude oscilovanja, uz definisanje diskretizovane mase $m(s)$ i kvantitativne forme oscilovanja sračunate iz normiranog oblika tona oscilovanja $\phi_{i,y}$, primjenom jednačine:

$$y_{i,y}(s) = y_{F,max} \phi_{i,y}(s) \quad (6)$$

moгуće je sračunati inercijalne sile po jedinici dužine, prema formuli iz Evrokoda:

$$F_w(s) = m(s) \cdot (2\pi f_{i,y}) y_{i,y}(s) \quad (7)$$

Na konkretnom primjeru parametarske analize može biti pokazano kako za cilindričnu konstrukciju definisanu parametrima $L=15$ m, $t=6$ mm, $\delta = 0,012$, $S_t = 0,18$, zona vjetra 2, kat. terena II prema njemačkom nacionalnom aneksu, sračunate maksimalne amplitude u prva 2 oblika oscilovanja imaju vrijednosti predstavljene na Slici 1.

Kako je dosadašnja praksa pokazala da relacija (7) koju definiše Evrokod 1 ne odgovara sračunatim diskretizovanim amplitudama pomjeranja svake tačke nosača u višim tonovima oscilovanja (6) kod nosača promjenljive geometrije ili sa koncentrisanim masama, ovim radom se predlažu dvije metode kojima se na adekvatniji način mogle sračunate nepoznate presječne sile.

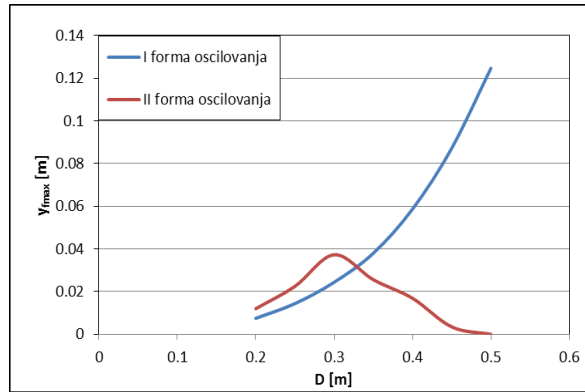
2.1.1 Proračun presječnih sila primjenom MKE

Nakon određivanja $y_{f,max}$ [3,9] moguće je sračunati presječne sile bez primjene jednačine (7). Sračunati oblik oscilovanja zadovoljava poznatu formu problema svojstvenih vrijednosti:

$$K\phi_r - \omega_r^2 M\phi_r = 0 \quad (8)$$

gdje je rešenje predstavljeno u obliku:

$$X_r = \phi_r e^{i\omega_r t} \quad (9)$$



Slika 1 – Sračunate amplitude oscilovanja u funkciji prečnika

Zadržavajući pretpostavku definisanog vektora presječnih sila u funkciji matrica krutosti, masa i pomjeranja sistema u obliku:

$$s = k \cdot v + m \frac{\partial^2 v}{\partial t^2} \quad (10)$$

gdje su: $s^T = [Q_{ij}(t) \ M_{ij}(t) \ Q_{ji}(t) \ M_{ji}(t)]$ presječne sile, $v^T = [w_i(t) \ \psi_i(t) \ w_j(t) \ \psi_j(t)]$ vektor pomjeranja štapa, k i m su matrice masa i krutosti izvedene za interpolacione funkcije u obliku Hermitovih polinoma za štap dužine l , krutosti EI i μ težine po m':

$$k = \frac{EI}{l^3} \begin{bmatrix} 12 & 6l & -12 & 6l \\ \cdot & 4l^2 & -6l & 2l^2 \\ \cdot & \cdot & 12 & -6l \\ \cdot & \cdot & \cdot & 4l^2 \end{bmatrix} \quad m = \frac{\mu l}{420} \begin{bmatrix} 156 & 22l & 54 & -13l \\ \cdot & 4l^2 & 13l & -3l^2 \\ \cdot & \cdot & 156 & -22l \\ \cdot & \cdot & \cdot & 4l^2 \end{bmatrix} \quad (11)$$

možemo primjenom (9) izvesti zaključak da važi $\ddot{v}_r = -\omega_r^2 v_r$, što uz primjenu (10) vodi ka lokalnim presječnim silama elementa s_r u r -tom obliku tona oscilovanja :

$$s_r = \max y_f [k - \omega_r^2 m] \phi_r^{ij} \quad (12)$$

Potrebno je dokazati da su sračunate presječne sile i raspodijeljeno spoljašnje opterećenje predstavljeno preko blok-sile u ravnoteži, [3]. Za konične elemente, moguće je izvršiti određivanje vrijednosti krutosti na krajevima štapa uz diskretizaciju na male elemente, što je i primijenjeno u konkretnim proračunskim primjerima na način da je $I = 1/((2I_1)+(2I_2))$, dok je zamjenjujuća vrijednost μl u matrici masa usvojena kao razlika normalne sile na krajevima

elementa vodeći računa o doprinosu koncentrisanih masa. Za slučaj kada je potrebno uzeti u obzir uticaj normalnih sila matrici k je potrebno dodati (ili oduzeti) matricu $k(N_{ij})$, prema [3]. Alternativno, moguće je primijeniti matrice izvedene za spomenutu geometriju (konični KE).

2.1.2 Proračun presječnih sila primjenom modela linearno elastične konstrukcije pri viskoznom prigušenju

Prethodno je rečeno da jednačina (7) kojom su definisane inercijalne sile statičke prirode ne odgovara uvijek sračunatoj kvantitativnoj formi oscilovanja (6) pa je potrebno izvršiti detaljniju analizu i predstaviti tačniji model proračuna. Za razliku od modela iz prethodnog poglavlja novi predlog se odnosi na dinamički model u kojem su sile, shodno osnovnim teorijskim pretpostavkama, harmonijske prirode.

Diferencijalna jednačina kretanja sistema pod spoljašnjim opterećenjem koje je promjenjivo u vremenu se može izraziti jednačinom:

$$[M]\{\ddot{y}(x)\} + [C]\{\dot{y}(x)\} + [K]\{y(x)\} = \{F(t)\} \quad (13)$$

Klasičnom jednačinom (13) se definiše sistem u kom su spoljašnje sile funkcija matrice mase $[M]$, prigušenja $[C]$ i krutosti $[K]$ sistema. Ranije je definisano kako je strukturalno prigušenje sistema EC standardom definisano samo za osnovni ton oscilovanja. Stoga je, u skladu sa potrebama jednačine (13), potrebno usvojiti određenu pretpostavku kojom će se strukturalno prigušenje predstaviti u funkciji prirodne frekvencije.

Najjednostavniji model koji se ovim radom preporučuje je model Rayleigh-ovog prigušenja. Ovim modelom viskoznog prigušenja, tražena vrijednost je izražena kao kombinacija mase i krutosti, na sljedeći način:

$$[C] = \alpha[M] + \beta[K] \quad (14)$$

Određivanje koeficijenata α i β će biti sprovedeno zadržavanjem pretpostavke o prigušenju u osnovnom tonu oscilovanja, logaritamski dekrement će biti preračunat u kritično prigušenje uz dopunsko zadavanje maksimalne vrijednosti prigušenja u iznosu od 0,5 % za ton u kome je participacija mase veća od 90%. Ovo ima za posledicu da je, obzirom na definisano maksimalno prigušenje ζ ,

$$\omega_d = \omega\sqrt{1-\zeta^2} \approx \omega \quad (15)$$

moguće jednačinu (13) posmatrati u istom obliku za prigušen i neprigušen sistem. Vrijednosti prigušenja u tonovima koji se nalaze u rasponu između osnovnog i tona m , sa participacijom mase koja je veća od 90%, pri čemu je ω kružna frekvencija, se sračunavaju interpolacijom:

$$\zeta_i = \frac{\zeta_m - \zeta_1}{\omega_m - \omega_1} (\omega_i - \omega_1) + \zeta_1 \quad (16)$$

Kako je prosta linearna interpolacija često nedovoljna, i potencijalno ne uzima u obzir mogući nelinearni pad prigušenja pri nižim frekvencijama u skladu sa Rayleigh-teorijom, potrebno je proširiti set podataka ekstrapolacijom rezultata do $2,5m$ tonova oscilovanja. Za opseg frekvencije iznad tona m primjenom formule:

$$\zeta_i = \frac{\zeta_m - \zeta_1}{\omega_m - \omega_1} (\omega_{m+i} - \omega_m) + \zeta_m \quad (17)$$

moguće je izvršiti ekstrapolaciju. Konačno, rješavanjem jednačina:

$$\beta = \frac{2\zeta_1\omega_1 - 2\zeta_m\omega_m}{\omega_1^2 - \omega_m^2} \quad \alpha = 2\zeta_1\omega_1 - \beta\omega_1^2 \quad (18)$$

prigušenje sistema je u potpunosti definisano.

Kritičan korak proračuna predstavlja sračunavanje presječnih sila koje dovode do sračunate kvantitativne forme oscilovanja. Pođimo od opšte jednačine harmonijskih sila upravnih na pravac vjetra vrtložnog rezonantnog modela [7] :

$$F(z, t) = q(z)b(z)c_F(z)\sin(2\pi f \cdot t + \gamma(z)\pi) \quad (19)$$

gdje su : $q(z)$ - brzina fluida koji vrši pritisak na konstrukciju; $b(z)$ – dimenzija presjeka upravno na pravac fluida; $c_F(z)$ – faktor amplitude sile; f – prirodna frekvencija; $\gamma(z)$ - faktor doprinosa znaka.

Pojednostavljajući ovu jednačinu i zanemarišvi faktor doprinosa znaka, obzirom na to da je iz sračunate kvantitativne forme poznat smjer svake harmonijske sile, možemo napisati:

$$F(z, t) = \pm F_i^0 \sin(2\pi f \cdot t) \quad (20)$$

Ova metoda podrazumijeva iterativni proračun sproveden u nekom od programskih paketa koji omogućava integraciju sila u vremenski zavisnom proračunu, u cilju zadovoljenja jednačine (13), a pretpostavljajući inercijalne sile u obliku (19) tj. (20). Varijable iterativnog koraka su veličina vremenskog koraka Δt i početne vrijednosti inercijalne sile za svaku diskretizovanu tačku nosača - F_i^0 . Uslov za završetak proračuna je dostizanje konvergencije amplitude oscilovanja za svaku od diskretizovanih tačaka, prema pretpostavljenom odgovoru sistema koji će biti prikazan kroz dva numerička primjera.

2.2 SPEKTRALNA METODA – METODA 2

Spektralna metoda, poznatija kao Hansenova metoda za proračun odvajanja vrtloga iz Evrokoda ima korijene u radovima naučnika Vickery i Basu (1983). Rješenje jednačina amplitude oscilovanja na vrhu konzolnog nosača prema Vickery-Basu-ovom modelu:

$$\frac{\bar{y}}{b} = g \frac{[n_1 S_{cl}(n_1)]^{1/2} (\rho_a b^2 / m)}{16\pi^{3/2} \eta^{1/2} S_i^2} f(\phi) \quad (21)$$

gdje su : $S_{cl}(n)$ – spektralna gustina koeficijenta sile koja je upravna na pravac toka; $f(\phi)$ - funkcija oblika oscilovanja; g – faktor maksimuma amplitude koji zavisi od rezonantne frekvencije; η -kritični koeficijent prigušenja koji uzima u obzir strukturno i aerodinamičko prigušenje, predstavljeno je u Evrokodu u zatvorenom obliku za *I oblik oscilovanja* kao metoda 2 za proračun odvajanja vrtloga:

$$\left(\frac{\sigma_{\max}}{b}\right)^2 = c_1 + \sqrt{c_1^2 + c_2} \quad c_1 = \frac{a_L^2}{2} \left(1 - \frac{Sc}{4\pi K_a}\right) \quad c_2 = \frac{a_L^2 \rho b^2 C_c^2 b}{K_a m S_i^4 h} \quad (22)$$

gdje su : ρ -gustina vazduha, m -masa po jedinici dužine; S_c -Scratonov broj; S_i -Strouhalov broj; $a_L = \gamma_L a_{L,ref}$ -limitirajuća/granična amplituda; $C_c = \gamma_c C_{c,ref}$ -aerodinamički parametar

pobuđivanja vrtloga (funkcija Reynoldsovog broja); K_a -parametar aerodinamičkog prigušenja; h -dužina konstrukcije izložena vrtložnom dejstvu.

Kako navedena metoda prognozira izuzetno rijetke i ekstremne vrijednosti amplituda oscilovanja, to nije pogodna za jednostavno primjenu pri proračunu zamora materijala kod konstrukcija sa dužim vijekom trajanja. Do sada zapažene amplitude koje su rezultati mjerenja koje je trajalo duže od 25 godina, nisu u saglasnosti sa rezultatima sračunatim primjenom predmetne teorije [12,13]. Ovakva diskrepancija u rezultatima je prevelika da bi se ignorisala, što je navelo na isključivanje ove metode iz njemačkog nacionalnog aneksa.

3 NUMERIČKI PRIMJERI

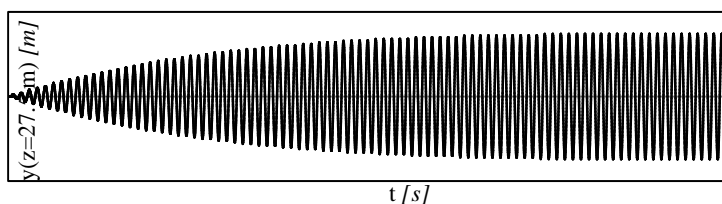
Primjer1: Razmatra se konzolni nosač visine 27.5 m sastavljen iz 3 elementa jednakih dužina sa spojevima na preklap dužine 1,0 m. Nosač je kružnog presjeka, prečnika na vrhu $b_{top} = 360$ mm i koniciteta $k = 10$ mm/m, a debljina nosača je 4mm. Koncentrisana masa težine 1 kN se nalazi na visini od 7.5 m. Parametri koji definišu proračun su: $\delta = 0.012$, $S_t = 0.18$, zona vjetra II, kat. terena 2 prema njemačkom nacionalnom aneksu.

Cilj proračuna je analiza mogućnosti sračunavanja tačnog odgovora sistema primjenom teorije date u tački 2.1.2 kao i sračunavanje momenta savijanja u prvom obliku tona oscilovanja uz poređenje rezultata (Tabela3).

Tabela 3 – Sračunate vrijednosti za osnovni oblik oscilovanja

Ton	f [Hz]	H_{crit} [m]	v_{crit} [m/s]	c_{lat} [-]	S_c [-]	Re [-]	K [-]	K_w [-]	$y_{f,max}$ [m]
I	0,902	26,53	1,87	0,7	5,78	46663	0,126	0,21	0,037

Na osnovu navedenog daje se analiza sračunatih vrijednosti prema navedenim metodama. Za usvojene vrijednosti $\alpha = 0,03376$ i $\beta = 0,00085$, odgovor sistema (pomjeranje tačke na vrhu konstrukcije) prema 2.1.2 je predstavljen na Slici 2.



Slika 2 – Pomjeranje u vremenu za tačku na vrhu konstrukcije –prema 2.1.2

Tabela 4 – Poređenje sračunatih momenata savijanja

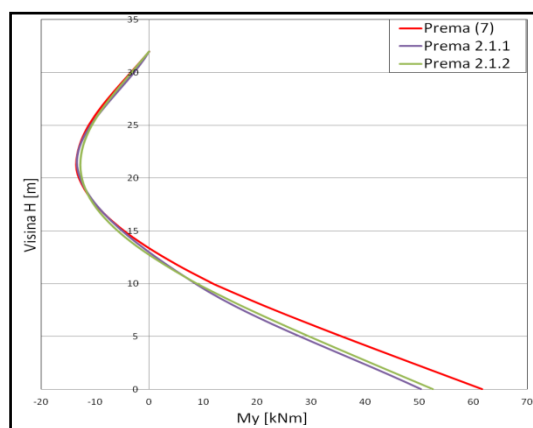
H [m]	M_y prema (7) [kNm]	M_y prema 2.1.1 [kNm]	M_y prema 2.1.2 [kNm]
27,5	0	0	0
15,85	2,601	2,528	2,476
0	10,570	10,277	10,053

Za diskretizaciju nosača na 17 elemenata, primjenom (7), kao i na osnovu prikazane teorijske osnove, sračunati momenti savijanja u pojedinim tačkama po visini nosača dati su u Tabeli 4. Uočava se da već u prvom obliku oscilovanja postoje mala odstupanja u sračunatom

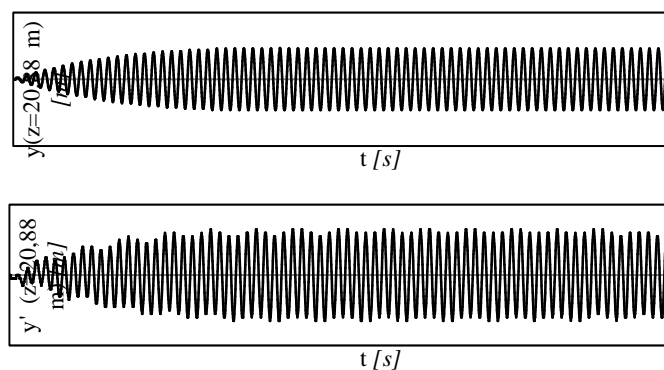
momentu savijanja. Detaljnija analiza ovog fenomena u drugom obliku oscilovanja će biti prikazana u primjeru koji slijedi.

Primjer 2: Za konzolnu konstrukciju visine 32m koja se sastoji od 3 dijela dužina $L_1=L_2=12$ m, $L_3=9,9$ m sa definisanim preklopima dužine 0,8 tj. 1.1 m potrebno je sračunati presječne sile usled odvajanja vrtloga u prva 2 oblika oscilovanja. Debljina 12-kantovskih presjeka iznosi $t_1=t_2=4$ mm, $t_3=5$ mm. Konicitet konstrukcije je $k = 20$ mm/m, prečnik na vrhu konstrukcije iznosi $b_{top} = 230$ mm. Na visini od 32 m, tj. 10m nalaze se koncentrisane mase u iznosu 3.8 kN, tj. 0.5 kN respektivno. Ostali parametri koji definišu proračun su: $\delta = 0.015$, $S_r = 0.20$, zona vjetra II, kat. terena 2 prema DE nacionalnom aneksu. Za II oblik oscilovanja je Tabela 5 – Sračunate vrijednosti za prva dva tona oscilovanja

Ton	f [Hz]	H_{crit} [m]	v_{crit} [m/s]	clat [-]	Sc [-]	Re [-]	K [-]	K_w [-]	$y_{f,max}$ [m]
I	0,625	31,35	0,76	0,7	40,3	12473	0,135	0,145	0,0021
II,1	2,929	31,35	3,58	0,7	24,4	58456	0,106	0,197	0,0037
II,2	2,929	20,88	6,65	0,7	7,09	201253	0,106	0,197	0,0235



Slika 3 – Poređenje sračunatog momenta savijanja, ton II, $H_{crit} = 20,88m$



Slika 4 - Pomjeranje i rotacija u vremenu za tačku $H = 20,88m$

potrebno izvršiti analizu uzimanjem u obzir dva kritična presjeka, sračunate vrijednosti date u Tabeli 5.

Za kritičnu proračunsku situaciju u drugom tonu sa maksimalnom amplitudom poređenje sračunatog momenta savijanja je prikazano na Slici 3. Analizom slike 3 moguće je zaključiti da razlika sračunatog momenta savijanja za presjek na visini $z=0$, poredivši teoriju iz Evrokoda i predložene metode iz ovog teksta, iznosi 25 %. Ne samo da ovakva razlika ima velike posledice u daljem ispitivanju zamora materijala u spomenutom presjeku, već i sama kvalitativna forma oscilovanja sračunata primjenom (7) ne odgovara sračunatom obliku (6).

Za usvojene vrijednosti $\alpha = 0,02942$ i $\beta = 0,00108$, odgovor sistema prema tački 2.1.2 u drugom obliku oscilovanja, pri odvajanju vrtloga na visini od 20,88 mm, je predstavljen na Slici 4.

4 ZAKLJUČAK

U radu je prikazan novi postupak proračuna za određivanje uticaja usled odvajanja vrtloga koji je dobijen proširivanjem postupka, Metode 1, koju predlaže Evrokod 1. Parametri koji definišu fenomen odvajanja vrtloga su vrlo često nedovoljno jasno definisani pa se zbog navedenog posebna pažnja treba posvetiti proračunskom modelu prilikom projektovanja konstrukcija sa pojavom ovog efekta. Za detaljne parametarske analize i tačnije proračunske modele potrebna je primjena specijalizovanog softvera, zasnovanog na predloženoj metodi, koji bi varijacijom odgovarajućih parametara, omogućio adekvatne i upotrebljive rezultate.

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SLOBODNE VIBRACIJE PLOČA SA UKRUĆENJIMA PRIMENOM METODE DINAMIČKE KRUTOSTI

Rezime:

S-43

U okviru ovog rada analizirane su slobodne vibracije ploča sa ukrućenjima primenom metode dinamičke krutosti. Razvijena je transformisana dinamička matrica krutosti potpuno slobodne pravougaone Mindlin-ove ploče korišćenjem matrice transformacije. Takođe, izvedena je globalna dinamička matrica krutosti sistema ploča koristeći sličan postupak kao u metodi konačnih elemenata. Određene su sopstvene frekvencije ploča sa ukrućenjima za različite konturne uslove i upoređene sa vrednostima dobijenim po metodi konačnih elemenata primenom programskog paketa Abaqus. Dobijeni su rezultati visoke tačnosti.

Ključne reči: slobodne vibracije, dinamička matrica krutosti, ploče sa ukrućenjima

DYNAMIC STIFFNESS ELEMENTS FOR FREE VIBRATION ANALYSIS OF STIFFENED PLATES

Summary:

The free vibration analysis of stiffened plate assemblies is presented in this paper by using the dynamic stiffness method. The transformed dynamic stiffness matrix of completely free rectangular Mindlin plate is derived by using the transformation matrix. In addition, the global dynamic stiffness matrix of plate assembly is derived by using similar assembly procedure as in the finite element method. The natural frequencies of stiffened plate assemblies with arbitrary boundary conditions are computed and validated against the results obtained by using the finite element software Abaqus. High accuracy of the results is demonstrated.

Key words: free vibrations, dynamic stiffness matrix, stiffened plates

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1 UVOD

Primena građevinskih materijala visokih mehaničkih karakteristika, pre svega čelika, uzrokuje upotrebu ploča sa ukrućenjima. Time se postiže veća nosivost za manju količinu materijala, kao i ušteda u ceni. Takođe, ukrućenja ojačavaju ploču i smanjuju njenu osetljivost na izbočavanje i torziju. Ploče sa ukrućenjima imaju široku primenu u građevinarstvu, posebno pri projektovanju mostova većih raspona i manjih poprečnih preseka, izradu paluba brodova, konstrukcije aviona, itd. Tokom svog radnog veka, ove konstrukcije su često izložene dinamičkom opterećenju, te je precizan proračun njihovog dinamičkog odgovora od velikog značaja u inženjerskoj praksi. U takvim slučajevima neophodno je predvideti osnovne dinamičke karakteristike sistema, kao što su sopstvene frekvencije i oblici oscilovanja.

Dinamički odgovor tankih ploča može se odrediti primenom Kirchhoff-ove klasične teorije ploča (classical plate theory - CPT). U slučaju debelih ploča ova teorija ne daje adekvatne rezultate zbog zanemarivanja deformacije smicanja, pa je potrebno primeniti Mindlin-ovu teoriju (first order shear deformation theory - FSDT), koja uzima u obzir uticaj deformacije smicanja pretpostavljajući da je klizanje konstantno po visini ploče. Paralelno sa razvojem različitih teorija ploča, razvijale su se i analitičke metode [1]. Ove metode se zasnivaju na tačnom rešenju diferencijalne jednačine kretanja i pokrivaju samo specijalne uslove oslanjanja. Leissa [2] je dao sveobuhvatan pregled analitičkih rešenja slobodnih vibracija ploča različitih oblika, zasnovane na Kirchhoff-ovoj teoriji. Liew i ostali [3] su analizirali slobodne vibracije debelih ploča sa proizvoljnim konturnim uslovima, primenom Rayleigh-Ritz-ove metode. Primena pomenutih metoda je ograničena na analizu slobodnih vibracija individualnih ploča i ne može se lako proširiti na analizu složenijih sistema ploča sa različitim geometrijskim i materijalnim karakteristikama, kakvi se najčešće sreću u inženjerskoj praksi (ploče sa ukrućenjima, sendvič ploče i sl.). U takvim slučajevima u analizi se primenjuju numeričke metode, kao što je metoda konačnih elemenata (MKE) [4]. U dinamičkoj analizi primenom MKE tačnost rezultata zavisi i od najviše frekvencije koja se javlja u analizi. Minimalan broj konačnih elemenata direktno je proporcionalan najvišoj razmatranoj frekvenciji, pa kod složenih konstrukcija kod kojih je u analizi potrebno u obzir uzeti i više tonove oscilovanja, potreban broj konačnih elemenata postaje veliki, čime se povećava ukupno trajanje proračuna.

U poslednje vreme se za analizu slobodnih vibracija ploča sve češće koristi metoda dinamičke krutosti (MDK) [5-13]. MDK kombinuje karakteristike MKE, kao što su fizička diskretizacija i mogućnost povezivanja elemenata u jedinstveni globalni sistem, sa rešenjem polja pomeranja koje predstavlja tačno rešenje diferencijalne jednačine slobodnih vibracija. Kako interpolacione funkcije, kojim se opisuje polje pomeranja u MDK, predstavljaju tačno rešenje diferencijalne jednačine kretanja u frekventnom domenu, greške usled diskretizacije su eliminisane. Podela ploče na manje dinamičke elemente je neophodna samo ukoliko unutar ploče postoji neki geometrijski i/ili fizički diskontinuitet. Time se smanjuje broj elemenata u analizi, broj stepeni slobode, kao i vreme potrebno za rad i mogućnost javljanja greške, u poređenju sa MKE.

U okviru ovog rada prikazan je numerički model za analizu slobodnih vibracija Mindlin-ovih ploča sa ukrućenjima sa proizvoljnim graničnim uslovima, primenom MDK. Na osnovu dinamičkih matrica krutosti za analizu slobodnih poprečnih vibracija i vibracija u ravni, izvedena je matrica transformacije za različite položaje ploča koje su pod pravim uglom u odnosu na referentnu ravan [14]. Primenjen je sličan postupak kao u MKE za formiranje globalne dinamičke matrice krutosti ploče sa ukrućenjima i razvijen je računarski program u MATLAB-u [15] za analizu slobodnih vibracija sistema ploča. Verifikacija prikazanog

postupka izvršena je upoređivanjem rezultata sa rezultatima dobijenim primenom programskog paketa Abaqus[16].

2 POSTUPAK FORMIRANJA DINAMIČKE MATRICE KRUTOSTI PRAVOUGAONE PLOČE

Postupak formiranja dinamičke matrice krutosti pravougaonog elementa ploče za poprečne i vibracije u ravni detaljno je prikazan u radovima [11, 13], dok će ovde biti prikazani osnovni koraci u postupku formiranja dinamičke matrice krutosti. Polaznu tačku predstavljaju jednačine kretanja elementa Mindlin-ove ploče u vremenskom domenu. Pretpostavlja se da su pomeranja harmonijske funkcije frekvencije ω , tj.:

$$\mathbf{u}(x, y, t) = \hat{\mathbf{u}}(x, y, \omega)e^{i\omega t} \quad (1)$$

gde je:

$\hat{\mathbf{u}}(x, y, \omega)$ amplituda polja pomeranja u frekventnom domenu.

Na osnovu ove pretpostavke, jednačine kretanja se iz vremenskog transformišu u frekventni domen. Na slici 1a prikazano je polje pomeranja pravougaone ploče po Mindlin-ovoj teoriji. Polje pomeranja se prikazuje kao superpozicija rešenja za četiri slučaja simetrije u odnosu na x i y koordinatne ose: simetrija-simetrija (SS), simetrija-antimetrija (SA), antimetrija-simetrija (AS) i antimetrija-antimetrija (AA):

$$\hat{\mathbf{u}}(x, y, \omega) = \hat{\mathbf{u}}^{SS}(x, y, \omega) + \hat{\mathbf{u}}^{SA}(x, y, \omega) + \hat{\mathbf{u}}^{AS}(x, y, \omega) + \hat{\mathbf{u}}^{AA}(x, y, \omega) \quad (2)$$

Na slici 1b prikazana je simetrična deformacija ploče (SS) oko obe koordinatne ose. Na ovaj način moguće je analizirati samo jednu četvrtinu ploče, čime se značajno umanjuje red dinamičkih matrica krutosti i ubrzava proračun. Rešenje jednačina kretanja u frekventnom domenu pretpostavlja se u obliku beskonačnog Fourier-ovog reda u sledećem obliku:

$$\hat{\mathbf{u}}^{ij}(x, y, \omega) = \sum_{m=0,1}^{\infty} C_m f_m(x) g_m(y) \quad (3)$$

gde su:

$f_m(x), g_m(y)$ bazne trigonometrijske funkcije,

C_m integracione konstante.

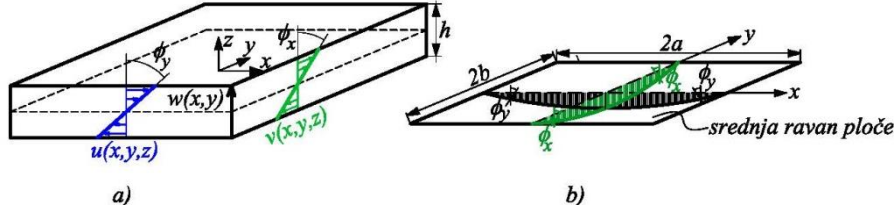
U jednačini (3) bazne funkcije zavise od slučaja simetrije i rešenja odgovarajućih jednačina kretanja, a $i, j = S, A$. Na osnovu kinematičkih i konstitutivnih relacija ploče, kao i jednačine (3), vektor sila u preseku u proizvoljnoj tački ploče može se napisati u obliku:

$$\hat{\mathbf{f}}^{ij}(x, y, \omega) = \sum_{m=0,1}^{\infty} C_m f_m^f(x) g_m^f(y) \quad (4)$$

gde su:

$f_m^f(x), g_m^f(y)$ izvodi baznih funkcija u zavisnosti od usvojene teorije ploče.

U praktičnoj primeni, beskonačni red u jednačinama (3) i (4) potrebno je prekinuti u tački M , tako da tačnost rešenja praktično zavisi samo od usvojenog broja članova reda.



Slika 1 – a) Polje pomeranja u Mindlin-ovoj toriji, b) Simetrična deformacija ploče (SS)

Sledeći korak predstavlja formiranje vektora pomeranja $\hat{\mathbf{q}}^{ij}$ na konturama $x = a$ i $y = b$ četvrtine ploče za svaki od četiri slučaja simetrije, koji se dobijaju zamenom koordinata kontura u jednačinu (3):

$$\hat{\mathbf{q}}^{ij} = \begin{bmatrix} \hat{\mathbf{u}}^{ij}(a, y, \omega) \\ \hat{\mathbf{u}}^{ij}(x, b, \omega) \end{bmatrix} \quad (5)$$

Slično, vektor sila $\hat{\mathbf{Q}}^{ij}$ na konturama ploče dobija se zamenom koordinata kontura u jednačinu (4):

$$\hat{\mathbf{Q}}^{ij} = \begin{bmatrix} \hat{\mathbf{f}}^{ij}(a, y, \omega) \\ \hat{\mathbf{f}}^{ij}(x, b, \omega) \end{bmatrix} \quad (6)$$

S obzirom da su komponente vektora pomeranja i sila na konturama ploče funkcije prostornih koordinata x i y , nije moguće direktno uspostaviti vezu između tih vektora sa jedne strane i vektora integracionih konstanti \mathbf{C} sa druge strane. Ovaj problem se može rešiti pomoću metode projekcije, koja se bazira na predstavljanju funkcija pomeranja i sila na konturi ploče u vidu Fourier-ovog reda:

$$\begin{aligned} \tilde{\mathbf{q}}^{ij} &= \frac{2}{L} \int_s \mathbf{H}^{ij} \hat{\mathbf{q}}^{ij} ds = \mathbf{D}^{ij} \mathbf{C} \\ \tilde{\mathbf{Q}}^{ij} &= \frac{2}{L} \int_s \mathbf{H}^{ij} \hat{\mathbf{Q}}^{ij} ds = \mathbf{F}^{ij} \mathbf{C} \end{aligned} \quad (7)$$

gde je:

\mathbf{H}^{ij} matrica baznih funkcija za odgovarajući slučaj simetrije.

U jednačinama (7) $L = a$ za konturu paralelnu sa x osom, a $L = b$ za konturu paralelnu sa y osom. Eliminacijom vektora integracionih konstanti iz jednačina (7) dobija se dinamička matrica krutosti četvrtine ploče $\tilde{\mathbf{K}}_D^{ij}$ za svaki od četiri slučaja simetrije:

$$\tilde{\mathbf{Q}}^{ij} = \mathbf{F}^{ij} (\mathbf{D}^{ij})^{-1} \tilde{\mathbf{q}}^{ij} = \tilde{\mathbf{K}}_D^{ij} \tilde{\mathbf{q}}^{ij} \quad (8)$$

Dinamička matrica krutosti cele ploče $\tilde{\mathbf{K}}_D$ može se dobiti primenom transfer matrice [11, 13].

3 PLOČE SA UKRUĆENJIMA

Poprečne vibracije i vibracije u ravni za jednu izotropnu ploču predstavljaju dva nezavisna stanja. Stoga se dinamička matrica krutosti ploče može napisati kao:

$$\tilde{\mathbf{K}}_D = \begin{bmatrix} \tilde{\mathbf{K}}_{Dt} & 0 \\ 0 & \tilde{\mathbf{K}}_{Di} \end{bmatrix} \quad (9)$$

gde je:

$\tilde{\mathbf{K}}_{Dt}$ dinamička matrica krutosti ploče izložene poprečnim vibracijama,

$\tilde{\mathbf{K}}_{Di}$ dinamička matrica krutosti ploče za vibracije u ravni.

Saglasno jednačini (9), vektor projekcija pomeranja i sila na konturi ploče može se prikazati u sledećem obliku:

$$\tilde{\mathbf{q}} = \begin{bmatrix} \tilde{\mathbf{q}}_t \\ \tilde{\mathbf{q}}_i \end{bmatrix}$$

$$\tilde{\mathbf{Q}} = \begin{bmatrix} \tilde{\mathbf{Q}}_t \\ \tilde{\mathbf{Q}}_i \end{bmatrix}$$

(10)

Kod ploča sa ukrućenjima gde su ploče međusobno spojene pod pravim uglom, poprečne vibracije jedne ploče izazivaju vibracije u ravni druge ploče i obrnuto. Zbog toga je potrebno uspostaviti vezu između vektora projekcija pomeranja i sila $\tilde{\mathbf{q}}$ i $\tilde{\mathbf{Q}}$ u lokalnom i vektora $\tilde{\mathbf{q}}^*$ i $\tilde{\mathbf{Q}}^*$ u globalnom koordinatnom sistemu (Slike 2 i 3) pomoću matrice transformacije \mathbf{T} :

$$\tilde{\mathbf{q}} = \mathbf{T}\tilde{\mathbf{q}}^* \quad (11)$$

$$\tilde{\mathbf{Q}} = \mathbf{T}\tilde{\mathbf{Q}}^*$$

U skladu sa definisanim relacijama projekcija u lokalnom i globalnom koordinatnom sistemu, dinamička matrica krutosti ploče u globalnom koordinatnom sistemu dobija se kao:

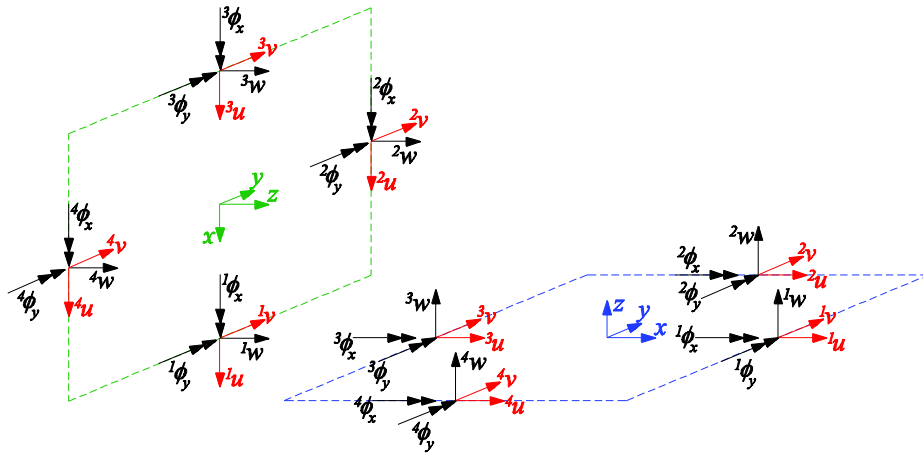
$$\tilde{\mathbf{K}}_D^* = \mathbf{T}^T \tilde{\mathbf{K}}_D \mathbf{T} \quad (12)$$

Dinamičke matrice krutosti pojedinačnih ploča sabiraju se u globalnu dinamičku matricu krutosti sistema ploča, slično kao u MKE, s tom razlikom što su ploče povezane duž kontura umesto u čvorovima. U analizi je moguće primeniti proizvoljne granične uslove uklanjanjem iz globalne dinamičke matrice krutosti sistema redove i kolone koje se odnose na sprečena pomeranja.

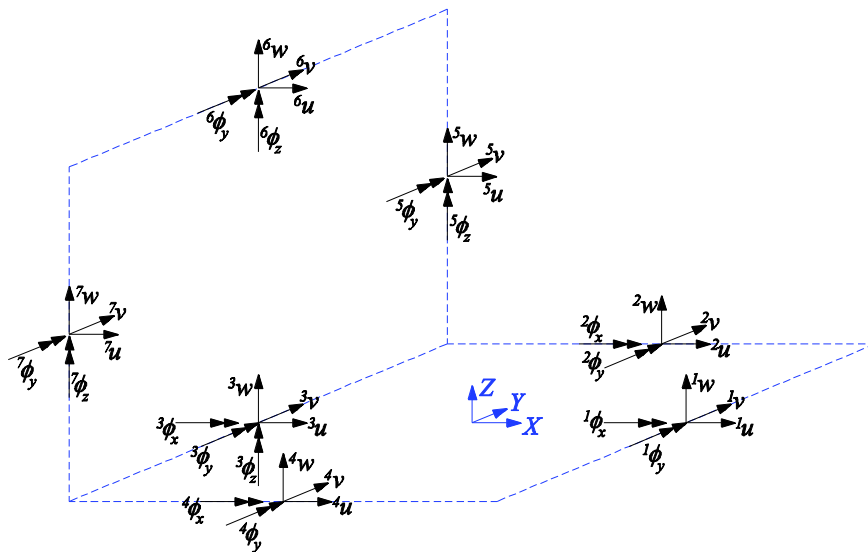
Dinamička matrica krutosti je kvadratna, frekventno zavisna matrica čiji red zavisi od broja članova reda M usvojenog rešenja. Sopstvene frekvencije određuju se iz sledeće jednačine:

$$\det \left| \tilde{\mathbf{K}}_{DG,m}^* (\omega) \right| = 0 \quad (13)$$

gde je:



Slika 2 - Komponente pomeranja na konturama ploča u lokalnom koordinatnom sistemu



Slika 3 - Komponente pomeranja na konturama ploča u globalnom koordinatnom sistemu

$\tilde{\mathbf{K}}_{DG,mn}^*$ globalna dinamička matrica krutosti sistema uz nepoznata pomeranja.

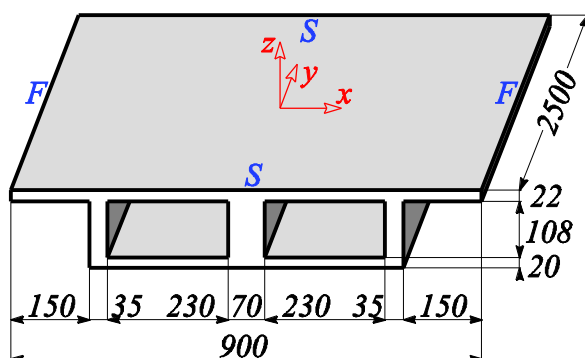
Pošto je jednačina (13) transcendentna, rešenja se mogu dobiti primenom neke od tehnika pretraživanja. Kako bi se izbegle numeričke poteškoće prilikom određivanja nule jednačine (13), sopstvene frekvencije se mogu odrediti kao maksimumi izraza:

$$g(\omega) = \log \frac{1}{\det |\tilde{\mathbf{K}}_{DG,mn}^*(\omega)|} \quad (14)$$

4 NUMERIČKI PRIMERI

Primena metode dinamičke krutosti u analizi slobodnih vibracija ploča sa ukrućenjima ilustrovana je u narednim primerima. Na osnovu izloženog postupka napisan je program u MATLAB-u [15] za određivanje sopstvenih frekvencija i oblika oscilovanja ploča sa ukrućenjima za različite uslove oslanjanja. Dobijeni rezultati su upoređeni sa rezultatima dobijenim primenom komercijalnog software-a Abaqus [16].

U prvom primeru razmatran je armirano-betonski nosač sandučastog poprečnog preseka, ($E = 31.5GPa$, $\nu = 0.2$ и $\rho = 2500kg/m^3$), čija je geometrija i granični uslovi prikazana na Slici 4. Ploča je diskretizovana minimalnim brojem od 9 elemenata i 26 kontura. Konture paralelne sa x osom su slobodno oslonjene (S), dok su konture paralelne sa y osom slobodne (F). Prvih 10 sopstvenih frekvencija $f = \omega/2\pi$ sračunato je primenom različitog broja članova reda kako bi se utvrdila konvergencija rešenja. Rezultati su prikazani u Tabeli 1 i upoređeni sa numeričkim rešenjem dobijenim primenom 46250 konačnih elemenata tipa S4R u Abaqus-u (dimenzija elementa 0.1 m). Za rešenje sa $M=7$ članova reda sračunato je odstupanje Δ od rešenja u Abaqus-u.



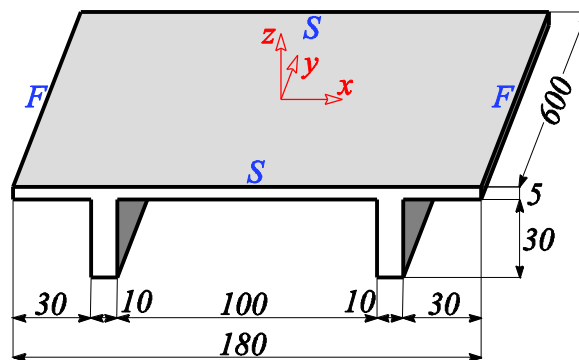
Slika 4 - Geometrija i granični uslovi armirano-betonskog nosača sandučastog preseka

Tabela 1 - Sopstvene frekvencije [Hz] armirano-betonskog nosača sandučastog preseka

MDK	Ton									
	1	2	3	4	5	6	7	8	9	10
M=1	7.8	8.6	9.7	10.2	12.1	13.3	13.6	14.0	16.0	17.6
M=3	10.0	16.8	24.3	27.7	27.9	31.0	33.8	34.0	36.3	38.4
M=5	9.9	16.7	23.9	27.4	27.8	29.1	37.9	38.1	39.0	43.7
M=7	9.9	16.6	23.8	27.3	27.7	29.0	37.8	38.0	38.9	43.7
Abaqus	9.865	16.552	23.676	27.177	27.713	28.884	37.578	37.860	38.742	43.683
Δ [%]	0.35	0.29	0.52	0.45	-0.05	0.40	0.59	0.24	0.41	0.04

Prvih 10 sopstvenih frekvencija se odlično poklapaju sa rešenjem dobijenim primenom MKE (prosečno odstupanje je 0.33%), što potvrđuje izuzetne mogućnosti primene izvedenih dinamičkih matrica krutosti u analizi vibracija armirano-betonskih ploča, čak i kada se uzme u obzir mali broj članova reda. Diskretizacija modela svedena je na minimum, čime se smanjuje ukupno trajanje proračuna u odnosu na klasičnu metodu konačnih elemenata.

U drugom primeru razmatrane su slobodne vibracije armirano-betonske korube ($E = 31.5GPa$, $\nu = 0.2$ и $\rho = 2500kg/m^3$), čija je geometrija i granični uslovi prikazana na Slici 5. Ploča je diskretizovana sa 5 elemenata (16 kontura). Prvih 10 sopstvenih frekvencija $f = \omega/2\pi$ sračunate su primenom različitog broja članova reda kako bi se utvrdila konvergencija rešenja. Rezultati su prikazani u Tabeli 2 i upoređeni sa MKE rešenjem dobijenim primenom 23520 konačnih elemenata tipa S4R u Abaqus-u (dimenzija elementa 0.025m). Za rešenje sa $M=7$ članova reda sračunato je odstupanje Δ od rešenja u Abaqus-u. Kao i u prethodnom primeru, sračunate frekvencije primenom prikazanog modela se odlično poklapaju sa numeričkim rešenjem (prosečno $\Delta = 0.22\%$).



Slika 5 - Geometrija i granični uslovi armirano-betonske korube

Tabela 2 - Sopstvene frekvencije [Hz] armirano-betonske korube

MDK	Ton									
	1	2	3	4	5	6	7	8	9	10
M=1	33.9	36.4	40.7	52.2	52.6	60.0	67.1	78.4	79.4	82.8
M=3	33.0	35.5	55.4	77.8	87.2	91.7	96.9	109.7	129.3	133.9
M=5	32.9	35.4	58.0	76.9	86.2	90.5	96.3	108.6	133.1	136.1
M=7	32.9	35.3	57.9	76.7	85.9	90.1	96.1	108.3	132.7	135.6
Abaqus	32.788	35.251	57.936	76.459	85.532	89.843	95.844	108.250	132.320	135.580
Δ [%]	0.34	0.14	-0.06	0.32	0.43	0.29	0.27	0.05	0.29	0.01

5 ZAKLJUČAK

U ovom radu prikazana je primena metode dinamičke krutosti u analizi slobodnih vibracija ploča sa ukrućenjima. Prikazan je postupak dobijanja dinamičke matrice krutosti ploče, kao i matrica transformacije za različite položaje ploča koje su pod pravim uglom u odnosu na referentnu ravan. Primenjen je sličan postupak kao u MKE za formiranje globalne dinamičke matrice krutosti ploče sa ukrućenjima. Razvijen je računarski program u MATLAB-u za analizu slobodnih vibracija sistema ploča. Sopstvene frekvencije i oblici oscilovanja određeni su za različite tipove konturnih uslova. Verifikacija dobijenih rezultata izvršena je upoređivanjem sa rezultatima dobijenim primenom programskog paketa Abaqus. Tačnost rezultata dobijenih primenom MDK ne zavisi od broja elemenata, već isključivo od broja članova reda usvojenog rešenja. Uočena je brza konvergencija rezultata dobijenih po MDK.

Već sa tri do pet članova reda dobijaju se rezultati visoke tačnosti. Međutim, za više tonove oscilovanja povećava se broj potrebnih članova reda.

Na osnovu izvedenih zaključaka, može se zaključiti da MDK poseduje veliki potencijal u dinamičkoj analizi konstrukcija koji se može proširiti na kompozitne ploče, ploče zasnovane na teoriji višeg reda, kao i ploče spojene pod proizvoljnim uglom.

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IMPLEMENTACIЈA VLAKNASTOG „STUB-GREDA“ ELEMENTA U АКАДЕМСКИ CAD SOFTVER - MATRIX 3D

Rezime:

S-44

U radu su prikazane teoretske osnove linijskog konačnog elementa koji je implementiran i testiran u okviru akademskog softvera za analizu konstrukcija razvijenog na Fakultetu tehničkih nauka. Sam element je formulisan na osnovu interpolacije unutrašnjih sila i baziran na diskretizaciji poprečnog preseka na vlakna, kao i podeli štapa na proizvoljan broj preseka. Pored materijalne obuhvaćena je i geometrijska nelinearnost. Razmatrana su i neka numerička pitanja neophodna za izvođenje opisanog inkrementalno-iterativnog proračuna. Konačno, prikazani su rezultati proračuna i upoređeni sa dostupnim rezultatima.

Ključne reči: vlaknasti model, stub-greda, akademski softver, nelinearna analiza

FIBER BEAM-COLUMN ELEMENT IMPLEMENTATION IN ACADEMIC CAD SOFTWARE MATRIX 3D

Summary:

Theoretical foundations of beam-column finite element implemented (and tested) within academic CAD software developed on FTN (department of civil engineering) are presented in this paper. Aforementioned FE is force-based fiber element, divided into a discrete number of monitored sections. Besides of material nonlinearity, finite-element is cable of capturing geometrical nonlinearity. Some of numerical issues needed for performing incremental-iterative solution procedures with those elements are addressed in the paper. Finally, results and comparison with available data are shown.

Key words: fiber element, beam-column, academic software, nonlinear analysis

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1 INTRODUCTION

Developments in earthquake engineering and nonlinear analysis have led to need for more sophisticated finite-elements (FE). Even though 2D or 3D FEs are certainly more accurate for precise nonlinear analysis, there are many drawbacks of its use. Some of the most relevant are complicated interpretation of results, cumbersome numerical analysis, and substantial time for modelling. All of these obstacles are surpassed with the use of fiber line elements, obviously provided frame-type structure is analysed. Fiber FE can be divided into force-based (FB) and displacement-based elements. Although great majority of software is displacement based, efficient algorithm for implementation of FB elements is developed in [1, 2]. Basic formulation of FB element used in this paper is designed by a group of scientists led by Fillipou [1-3] and is afterwards improved by same group to include some other aspects such as shear deformations [4,5], effects of bond deterioration [6], and composite structures [7]. Recently, De Souza presented in [8] extension of their work introducing corotational formulation, thus enabling analysis with large displacements and large rotations. Different approaches led to other improvements on including shear deformations [9, 10]. All of the mentioned fiber elements are used primarily for dynamic analysis of concrete structures. Thai and Kim [11, 12] included stability functions and nonlinear springs, and used FB elements for analysis of steel structure. In this paper, formulation very similar to one used in [10] will be presented, discussed and tested.

2 FIBER BEAM-COLUMN ELEMENT FORMULATION

There are two options for describing nonlinear material behaviour in beams and columns: lumped and distributed plasticity. Briefly, lumped plasticity, most often modelled with hinges, has two major shortcomings. Firstly, precise position of expected plastic hinge must be known a priori, and secondly nonlinear relation is specified as relation between force and deformation. These types of relations are known from experiments, further simplified and specified in various codes such as [13], textbooks etc. Much more realistic way of performing materially-nonlinear-only analysis (MNO) is through the constitution of material nonlinear law, where fiber elements come superior.

First fiber elements were used in the similar way as hinges, positioned in the area of expected plastic hinge, only to represent more realistic force/deformation dependence through integration of stresses. Later the possibility of using only one FE per element became possible. Hence, pre-processing became less time-consuming and, if the goal is to use 1 element per member, FB elements are more suitable. Still, FB elements may be scarcely found in commercial CAD software. Main obstacle in the widespread use of this type of elements is the difficulty of integrating nonlinear element state determination in an analysis program based on direct stiffness method [13]. This obstacle is overcome by a state determination procedure, proposed by [1] that iteratively determines the element resisting forces and stiffness matrix, while strictly satisfying element equilibrium and compatibility in each iteration. With this, analysis becomes incremental-iterative, which is discussed in 2.4. Since a pure flexibility formulation is not feasible in multi-degree freedom structure model [14], most of the CAD software is displacement-based. This means that linear elements are based on appropriate interpolation functions for member displacements. Conventional frame elements are based on Hermitian polynomials for transverse displacement fields, and Lagrange shape functions for

axial displacements. On the contrary, formulation of FB elements is based on interpolation functions for the internal forces. The main difference is that in classical stiffness-based 2-node element, curvature distribution is assumed linear (and in real member it becomes highly nonlinear as material softening occurs), while in FB element without element load, moment distribution is assumed linear, which is true. As result, only one element per member can be used, with accuracy depending on adopted tolerance of numerical error.

Since the goal of using one fiber FE per member is to accurately represent true behaviour of a member, two numerical choices are crucial. First one is type of numerical integration, and second one is number of sections per element. Obviously, the greater number of sections, greater is the accuracy, but this leads to greater cost of analysis. Even more important is the proper type of integration. Here, advantages of Gauss-Lobatto integration surpasses the ones of Gauss integration. Even though the latter is derived optimising both the weights and the integration points stations, former has stations at the ends of interval (hence minimum 3 stations) where the highest bending moments/curvatures are expected.

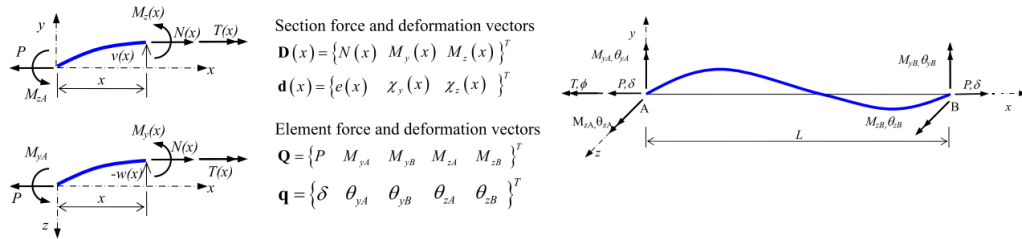


Figure 1 – Natural forces at element and section levels

Process of calculation for a single load increment is achieved on three different levels: structure level, element and section level. After calculating structure deformations using global stiffness matrix, another set of element degrees of freedom is adopted, and deformations are recalculated using transformation matrix \mathbf{L} given in [3]. On the element level, there is only 5 degrees of freedom, derived by eliminating rigid body modes, and presented in Figure 1. These element deformations are denoted by \mathbf{q} , and corresponding internal forces by \mathbf{Q} .

$$\mathbf{F} = \int_0^L \mathbf{b}^T(x) \mathbf{f}(x) \mathbf{b}(x) dx \approx L \sum_{sec=1}^{n_{sec}} w_j (\mathbf{b}^T(sec) \mathbf{f}(sec) \mathbf{b}(sec)) \quad (1)$$

$$\Delta \mathbf{Q} = \mathbf{K}_{ele} \Delta \mathbf{q}, \quad \delta \Delta \mathbf{D}(x) = \mathbf{b}(x) \Delta \mathbf{Q}, \quad \delta \Delta \mathbf{d}(x) = \mathbf{r}(x) + \mathbf{f}(x) \Delta \mathbf{D}(x) \quad (2)$$

Element internal forces are calculated using element stiffness matrix \mathbf{K} which is inverted flexibility matrix \mathbf{F} (1). For constructing \mathbf{F} , section flexibility matrix $\mathbf{f}(x)$ must be determined, as presented in (5).

$$\mathbf{b}(x) = \begin{bmatrix} 1 & 0 & 0 & 0 & 0 \\ 0 & \xi - 1 & \xi & 0 & 0 \\ 0 & 0 & 0 & \xi - 1 & \xi \end{bmatrix}, \quad \xi = x/L \quad (3)$$

$$\mathbf{f}(x) = \mathbf{k}_{sec}(x)^{-1}, \quad \mathbf{k}_{sec}(x) = \sum_{i=1}^{n_{fib}} \begin{bmatrix} E_i A_i & E_i A_i z_i & -E_i A_i y_i \\ E_i A_i z_i & E_i A_i z_i^2 & -E_i A_i z_i y_i \\ -E_i A_i y_i & -E_i A_i z_i y_i & E_i A_i y_i^2 \end{bmatrix} \quad (4)$$

Assumptions of internal force's distribution in element are represented in matrix \mathbf{b} . For absence of element loads \mathbf{b} is presented in (6). Section flexibility matrix is obtained by

inverting section stiffness matrix, \mathbf{k}_{sec} (5). At the fiber level, uniaxial state is assumed, and fiber strain \mathbf{e}_{fib} is calculated from section deformation vector $\mathbf{d}(x)$ (2c).

$$\delta\Delta\mathbf{e}_{fib} = \mathbf{a}_{fib(x)}\delta\Delta\mathbf{d}(x), \quad \mathbf{a}_i = \{1 \quad z_i \quad -y_i\}, \quad E_i = \Delta\sigma_i/\Delta\varepsilon_i \quad (5)$$

By using selected stress-strain relation, stresses of each fiber are found, after which the calculation of section resisting forces \mathbf{D}_R is possible (7).

$$\mathbf{D}_R(x) = \sum_{i=1}^{n_{fib}} \sigma_i A \mathbf{a}_i^T, \quad \mathbf{D}_U(x) = \mathbf{D}(x) - \mathbf{D}_R(x), \quad \mathbf{r}(x) = \mathbf{f}(x) \mathbf{D}_U(x) \quad (6)$$

When difference between resisting forces and applied forces \mathbf{D}_U (4) for the element is reached, element state determination is completed, and global residual forces \mathbf{P}_R are determined (7). Residual forces in the element are used for self-balance mechanism through \mathbf{r} (4c) and s (6).

$$\mathbf{s} = \sum_{sec=1}^{n_{sec}} [w_{sec} \mathbf{b}(sec)^T \mathbf{r}(sec)] , \quad \Delta\mathbf{Q}^{j>1} = -\mathbf{K}_{ele} \mathbf{s}, \quad j \text{ is NR iteration} \quad (7)$$

$$\mathbf{P}_R = \mathbf{L}_{ele} \mathbf{Q}, \quad \mathbf{P}_U = \mathbf{P} - \mathbf{P}_R \quad (8)$$

Vectors denoted with δ are calculated in every iteration of element state determination, while vectors denoted only with Δ are calculated once per Newton-Raphson (NR) iteration. . Hence, algorithm consists of 3 loops: load increment loop, NR structure loop, and element state determination one. Inside the last one there are section and fiber loops, but they do not bring unbalanced quantities since they depend on values passed by the main 3 loops.

One of the most important aspects of implementation of presented FE is convergence criterion enforced on both the element and structure loop. Element loop is terminated when all of the sections achieve convergence, or when prescribed number of iterations is performed (in Matrix 3D [15] it is adopted as 25). If latter is the case, outer (structure) loop is also aborted, and smaller load increment is applied. Structure loop continues NR iterations until the tolerance is achieved. Convergence criteria may be displacement-based, force-based or energy-based. Criesfield in [16] explains that energy criterion can be misleading, while Fillipou in [1] proposes this type of criterion. In Matrix 3D, for inner loop, force criterion is defined, while for structure loop, both energy and force criterion have to be satisfied to proceed. Adopted tolerances are 10^{-6} for force, and 10^{-12} for energy criterion.

2.1 MATERIAL NONLINEARITY - MATERIAL MODELS

There are few material models extensively used in numerical researches of steel structures lately. To perform dynamic, or cyclic analysis, hysteretic properties need to be specified. Usually isotropic, kinematic or mixed strain hardening is chosen. All of them are implemented in aforementioned software. Besides, any multilinear material rule can be specified. However, multilinear type of stress-strain relation may lead to numerical obstacles. Since \mathbf{k}_{sec} is calculated with tangent stiffness of each fiber, during collapse analysis (such as pushover, especially close to buckling), if the difference between two adjoining slopes is considerable, convergence will be difficult to achieve. Thus, material models described with smooth curves are preferred. This condition is fulfilled with either of Ramberg-Osgood, Monti-Nuti or Menegotto-Pinto model. Among these models Ramberg-Osgood has an important disadvantage. Since described algorithm requires calculation of stresses from strains, and this model is described with bijective function of stress, it means that equation of material law must be solved for each strain. Because there are many fibers, sections, iteration and increments, it becomes vastly time-consuming. Menegotto-Pinto model is improved by Fillipou et al [17] to

include isotropic strain hardening. For the analysis performed in this paper, regular model is used with the following values: $R_0=20$, $a_1=18.5$, $a_2=0.15$, $b=0.01$.

2.3 GEOMETRICAL NONLINEARITY

Two effects regarding geometrical nonlinearity are represented through implemented FB formulation: $P-\Delta$ and $P-\delta$. Former is introduced complementing element stiffness matrix in global degrees of freedom. It accounts for the effects of axial force acting through the relative transverse displacement of member ends, Δ . It is denoted by \mathbf{K}_G , and calculated by (8), where \mathbf{Z}_0 is zero matrix 3×3 . This way of including $P-\Delta$ effect in analysis is more accurate, and also more suitable for presented formulation in comparison with calculating only shear due to second order effects, and then reiterate with this force as node load (as is used in SAP2000 e.g.).

$$[K_g]_{12 \times 12} = \begin{bmatrix} [K_s] & -[K_s] \\ -[K_s] & [K_s] \end{bmatrix}, \quad [K_s]_{6 \times 6} = \begin{bmatrix} [K_g] & \mathbf{Z}_0 \\ \mathbf{Z}_0 & \mathbf{Z}_0 \end{bmatrix}, \quad [K_g]_{3 \times 3} = \begin{bmatrix} 0 & a & -b \\ a & c & 0 \\ -b & 0 & c \end{bmatrix}$$

$$a = \frac{M_{zA} + M_{zB}}{L^2}, \quad b = \frac{M_{yA} + M_{yB}}{L^2}, \quad c = -\frac{P}{L}, \quad P \text{ is considered negative for compression} \quad (9)$$

$$[K_{ele}] = [L]_{5 \times 12}^T [K_e]_{5 \times 5} [L]_{5 \times 12} + [K_g]_{12 \times 12} \quad (10)$$

Effect of axial force acting through the lateral displacement of the beam-columns element (usually called P-small delta), is captured using stability functions in the element stiffness matrix (5×5 - natural degrees of freedom). Derivation of these functions (12) can be found in [18, 19] and they were introduced by James in 1935. They are derived for a propped cantilever, introducing lateral displacement due to rotation on one side of beam-column, and hence can be used only for continuous beam-columns. For different boundary conditions (i.e. brace member) different set of functions should be used. They are implemented in Matrix3D, but not shown here for brevity.

$$\begin{bmatrix} \Delta P \\ \Delta M_{yA} \\ \Delta M_{yB} \\ \Delta M_{zA} \\ \Delta M_{zB} \end{bmatrix} = \begin{bmatrix} K_{e1,1} & 0 & 0 & 0 & 0 \\ 0 & S_{1y}/4 * K_{e2,2} & S_{2y}/2 * K_{e2,3} & 0 & 0 \\ 0 & S_{2y}/2 * K_{e3,2} & S_{1y}/4 * K_{e3,3} & 0 & 0 \\ 0 & 0 & 0 & S_{1z}/4 * K_{e4,4} & S_{2z}/2 * K_{e4,5} \\ 0 & 0 & 0 & S_{2z}/2 * K_{e5,4} & S_{1z}/4 * K_{e5,5} \end{bmatrix} \begin{bmatrix} \Delta \delta \\ \Delta \theta_{yA} \\ \Delta \theta_{yB} \\ \Delta \theta_{zA} \\ \Delta \theta_{zB} \end{bmatrix} \quad (10)$$

$$S_{1n} = \begin{cases} \frac{k_n L \sin(k_n L) - (k_n L)^2 \cos(k_n L)}{2 - 2 \cos(k_n L) - k_n L \sin(k_n L)} \\ \frac{(k_n L)^2 \cosh(k_n L) - k_n L \sinh(k_n L)}{2 - 2 \cos h(k_n L) + k_n L \sinh(k_n L)} \end{cases} \quad S_{2n} = \begin{cases} \frac{(k_n L)^2 - k_n L \sin(k_n L)}{2 - 2 \cos(k_n L) - k_n L \sin(k_n L)} \\ \frac{k_n L \sinh(k_n L) - (k_n L)^2}{2 - 2 \cos h(k_n L) + k_n L \sinh(k_n L)} \end{cases} \quad (11)$$

where $k_n^2 = |P|/EI_n = |P|/\sum_{j=1}^{nsec} w_j (\sum_{i=1}^{nifb} E_i A_i z_i^2)_j$.

Upper set of equations with trigonometric functions in (11) is used when axial force is compressive, while bottom set with hyperbolic functions is used if \mathbf{P} is tensile.

Additionally, lateral displacements are included in calculation of incremental section forces, (2b). Force interpolation matrix (3) is altered so $\mathbf{b}_{2,1}$ becomes $-\delta \mathbf{y}(x)$ instead of 0, and $\mathbf{b}_{3,1}$ becomes $-\delta \mathbf{z}(x)$ [11,19].

$$\delta_y(x) = \delta_{y,0}(x) - \frac{M_{zA}}{EI_z k_z^2} \left[\frac{\sin(k_z x)}{\tan(k_z L)} - \cos(k_z x) - \frac{x}{L} + 1 \right] - \frac{M_{zB}}{EI_z k_z^2} \left[\frac{\sin(k_z x)}{\sin(k_z L)} - \frac{x}{L} + 1 \right] \quad (12a)$$

$$\delta_z(x) = \delta_{z,0}(x) + \frac{M_{nA}}{EI_n k_n^2} \left[\frac{\sin(k_y x)}{\tan(k_y L)} - \cos(k_y x) - \frac{x}{L} + 1 \right] + \frac{M_{yB}}{EI_y k_y^2} \left[\frac{\sin(k_y x)}{\sin(k_y L)} - \frac{x}{L} + 1 \right] \quad (12b)$$

where $\delta_{i,0}(x) = L/1000 * \sin(\pi x/L)$, which represents initial bow-type imperfection of member.

2.4 NUMERICAL TECHNIQUES FOR NONLINEAR SOLUTION PROCEDURES

Among several numerical methods used for solving systems of nonlinear equation, there is no one exceptionally more suitable than the others. Several methods are sufficient for analysis in which limit point is not reached, such as full or modified Newton-Raphson, bisection method or combination with NR, etc. In Matrix3D, full NR is used since the biggest difficulty of the method is calculation of tangent stiffness in each iteration. But since the element stiffness matrices are calculated either way, pure assembling of global matrix is much more beneficial than using modified NR method. In explained algorithm, limit of 20 iterations is imposed as maximum, before load increment is decreased.

Although aforementioned numerical methods are very efficient, in the proximity of limit point they become insufficient. NR method will not be able to surpass limit point, and also is not able to handle snap-through behaviour. For these situations, which are experienced with buckling or with material softening (concrete), continuation techniques are needed. The most popular, and very possibly the most robust one is the arc length method [16, 20]. Few variations are most often: Criesfield's (cylindrical or spherical), Ramm's or Riks-Wemper. Criesfield arc length method is probably the most used one, and it belongs to group of variation with constant arc length. Problem with this method is that two solutions are found from quadratic equation, hence it needs predictor and corrector phase of solution, latter being more problematic. Ramm's method is based on finding intersection between desired field of resistance with updated hyperplane. For all mentioned methods load increment is variable, which requires another equation to be solved. Their main differences are in that equation. Both Ramm's and Criesfield methods are implemented in Matrix3D in conjunction with full NR method, but only Ramm's equation (13) will be shown, since none of the advantages of Criesfield's method is uncovered in present use by the authors. It however should be noted that authors' experience with its use led to conclusion that β factor, which is a value that weights the importance of the contribution of displacements and loads, preferably be taken nonzero, contrary to what many authors suggests.

$$d\lambda = \frac{\Delta u_0^T \delta \bar{u}}{\Delta u_0^T \delta u_t + \Delta \lambda_0 \beta^2 P_{ef}^T P_{ef}} \quad (13)$$

where:

$d\lambda$ is iterative change of load-level parameter,

$\Delta \lambda_0$ is incremental load parameter changed in every iteration.

3 VERIFICATIONS

Implementation of above-mentioned formulation is verified for accuracy and efficiency by the comparison of predictions with accessible results and those generated by commercial finite element packages through two following examples. First one investigates a column with all end

restrains fixed except for longitudinal displacement at the upper end, as shown in Figure 2, with the aim to verify the accuracy and efficiency of the proposed element in capturing the buckling loads of columns with imperfections. Second example is the Vogel's portal frame, which is usually adopted for calibrating advanced second-order inelastic analysis. Since the frame collapse is determined by the column's inelastic buckling, this example is a good benchmark test for any inelastic formulation [22].

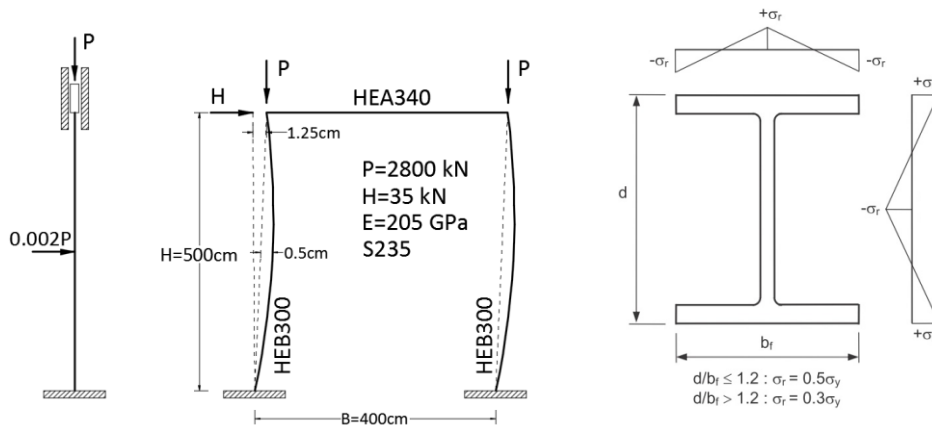


Figure 2 – Column clamped on both ends (left), Vogel's frame (centre), residual stress pattern (right)

Both models include initial out-of-plumbness imperfections, modelled by equivalent load in first example, and rearrangement of nodes in the second. Both models include already mentioned out-of-straightness initial imperfection throughout their length. Even though residual stresses incorporation is not so convenient for both flexibility approach and Menegotto-Pinto model, it may be overcome with few coding improvisations. Residual stress pattern of I section used for this research is proposed by ECCS as illustrated in Figure 2 and assigned directly to fibers as initial stresses.

The load-deflection curves of the column obtained by the proposed element (Figure 3) are compared with those generated by SAP2000 and SeismoStruct. Material model used in Matrix 3D (PW) and SeismoStruct (SS) is Menegotto-Pinto with initial modulus of 210 GPa and post-elastic hardening of 1%, while bilinear material with the same hardening is used for SAP2000 model. Fiber element of SAP2000 overpredicts buckling load by about 18% if two elements are used, and 6% if 4 elements per member are used. Critical force obtained by SeismoStruct and proposed element are almost identical, while the difference in displacement can be partly explained by the absence of residual stresses in SS model. It should be noted that SeismoStruct has FB element implemented while for SAP2000 model, displacement-based fiber element is used. Design buckling resistance of a column determined by EC3 rules of design is marked on the graph for comparison.

Table 1 – Vogel’s frame: collapse load and drift

Collapse point	Vogel [20]	Clarke [21]	Kim&Lee [22]	Alvarenga [23]	Present work
λ [%]	102,2	102,3	102,9	100,7	102,8
u_{Ah} [mm]	17,3	17,1	18,9	17	14,3

Figure 3 (right) shows the comparison of the load-deflection curves for Vogel’s frame predicted by the proposed element with results of several researchers. It was first analysed by Vogel in 1985. [21], whose plastic-zone (PZ) model is unknown to authors, and later by Clarke in 1994. [22] who used 50 PZ elements per column and 20 for the beam. Kim and Lee [23], using the Abaqus commercial software, modelled the frame with 8952 3D-shell elements. Alvarenga and Silveira [24] in 2009. used fiber model with 8 elements per column and 6 for the beam. Comparison of ultimate load (limit point) and maximum horizontal displacement of upper right joint is presented in Table 1.

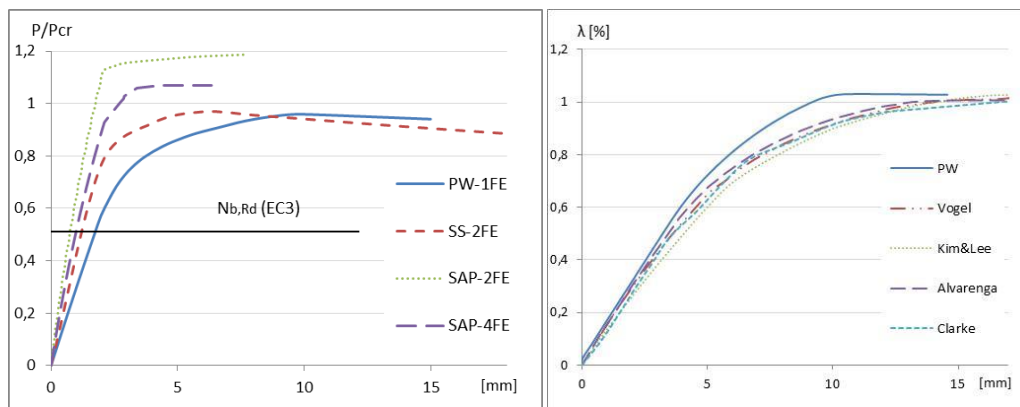


Figure 3 – Load-displacement curves of column (left) and Vogel’s frame (right)

As it can be recognized from Table 1, proposed FB need only one element per member to match the results generated by Abaqus with two-dimensional finite elements used in [22] or 8 fiber elements as in [23]. Again, precision in determination of limit load is considerably greater than for a displacement. Some divergence of results can be attributed to different post-elastic modulus. In all cited analyses elastic-perfectly plastic bilinear model was used, while in present study post-elastic modulus of 2.05 GPa is adopted in order to avoid numerical difficulties and hence somewhat stiffer response. Altogether, presented data illustrate considerable agreement between results obtained using only 1 proposed FB element per member and analyses higher order of discretization.

4 CONCLUSION AND DISSCUSION

Fiber elements has the essential advantage of representing very accurate behaviour of structures, while need simple pre-processing and capable of delivering results common for engineers, i.e. internal force diagrams. This actually means that no extra time is needed for structural modelling with fiber elements than with commonly used line elements. Beside this,

many advantages of more sophisticated and accurate analysis are on the reach with introduction of fiber elements. Such as, for example, introducing real material behaviour, finding a limit load and critical configuration, or even exploring the structural behaviour beyond limit point. And in the case of nonlinear dynamic analysis, elements with distributed plasticity need no presumptive positioning of hinges, and are able to show „true“ spreading of plasticity.

Considering steel structures, complete GMNIA analysis can be performed with this type of element and setting in Matrix3D. Including advanced heuristic algorithm Matrix3D uses for load stepping, limit load of the structure can be very precisely determined. This fact inspire the new concept of safety coefficients. In Eurocode 3, design of compressed elements includes many approximations that had to be done in the past. Considering present concept of combining loads and safety coefficients, it is unbiased to say that computer aid is unavoidable. When such, analysis presented can determine safety coefficients more accurately which is the aim of design in general.

Formulation and numerical procedures presented here are just a small part of academic CAD software Matrix3D. It is highly developed multipurpose software capable of different types of nonlinear analysis, including 2D and 3D finite elements, nonlinear dynamic analysis, impact loads, accelerogram inputs etc. On top of that, it is user friendly as everything can be set and viewed from user interface. Future work that is already in progress will include introducing bond-slip model (for reinforcement and composite sections), including shear in FB formulation, warping, adaptive pushover etc.

Acknowledgments

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OSNOVNI PRINCIPI ASEIZMIČKOG PROJEKTOVANJA I GRAĐENJA

Rezime:

S-45 U radu je prezentovana suština aseizmičkog projektovanja i građenja sa aspekta racionalne i adekvatne zaštite objekata od dejstva zemljotresa. Ukazano je na najnepovoljnije slučajeve koje seizmičko opterećenje kao posebna vrsta opterećenja izaziva u konstrukciji objekta ili u samom objektu. Pokazana je značajna razlika u veličini seizmičkih sila sračunatih prema našim važećim propisima i prema Evrokodu 8. Za navedeni slučaj potenciran je vrlo nepovoljan uticaj likvefakcije, pa takve terene treba izbegavati po svaku cenu jer je njen uticaj katastrofalan po objekte kako je na slikama pokazano. Prikazane su histerezisne petlje eksperimentalno ispitanog modela do loma u prirodnoj veličini.

Ključne reči: zemljotres, seizmičke sile, aseizmička zaštita, eksperiment

BASIC PRINCIPLES OF ASEISMIC DESIGN AND CONSTRUCTION

Summary:

The paper presents the essence of aseismic design and construction from the view of rational and adequate protection of structures against earthquakes. It is pointed to the most unfavorable damages that seismic load as special load has caused to the carrying structure or to the other parts of some buildings. Examples of significant difference between design seismic forces intensity calculated according to Serbian current regulations and according to Eurocode 8 are presented too. It is emphasized and shown by the pictures how adverse effect of liquefaction is, and that such terrains should be avoided because of disastrous influence of liquefaction on buildings. Presented are the hysteresis loops of experimentally tested full size model of the connection column-to-foundation loaded to the failure.

Key words: earthquake, seismic forces, seismic protection, experiment

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1 UVOD

Zemljotres je prirodna pojava čiji se uticaj neizostavno mora uzeti pri projektovanju (jer je najčešće i merodavan) kao i pri izgradnji građevinskih objekata, tj. svih vrsta građevina. Od pamtiveka je poznato da zemljotresi mogu izazvati (a najčešće i izazivaju) katastrofalne posledice, tj. enormne štete, a posebno gubitke ljudskih života. Zato je aseizmičko projektovanje i građenje, odnosno zaštita od zemljotresa veoma važna u savremenom urbanizovanom društvu. Iz tog razloga je aseizmičko projektovanje i građenje potrebno svim stručnjacima koji učestvuju u stvaranju objekta, a posebno građevinskim inženjerima pa je to deo njihovog redovnog školovanja [1]. Intenzivna istraživanja iz oblasti zemljotresnog inženjerstva i inženjerske seizmologije, kao i iskustva nakon jakih zemljotresa, posebno poslednjih decenija kada se ti uticaji i mere velikim brojem seizmometra, doprinose dragocenim saznanjima iz ove oblasti. Definisane seizmičkog hazarda i smanjenje seizmičkog rizika doprinosi i smanjenju teških oštećenja objekata, povreda ljudi i gubitka ljudskih života. Pod projektovanjem u širem smislu ovde se podrazumeva: proračun, dimenzionisanje i oblikovanje detalja noseće konstrukcije [2]. Aseizmičko građevinarstvo je povezano sa nizom suprotnosti i kontradiktornosti za koje je potrebno pronaći adekvatan kompromis koji nije na štetu sigurnosti i bezbednosti ljudi. Osim novogradnje inženjeri sve češće imaju zadatak da proveravaju aseizmičku otpornost postojećih građevinskih objekata, gde posebno mesto zauzimaju kulturno istorijski spomenici.

Posledice zemljotresa se mogu podeliti na direktne usled samog podrhtavanja tla i dinamičkog odgovora građevine za vreme trajanja potresa. Indirektne posledice usled požara, sleganje i/ili klizanja terena, poplava, odnosno poplava koje je izazvao sam zemljotres (cunami talas). Najnoviji primer je zemljotres od 11.03.2011. sa obale Japana, magnitude od 9 stepeni Rihterove skale. Cunami talasi visine od 30m su ostavili katastrofalne posledice, brišući sve pred sobom, dopirući do 10km u dubinu kopna. Potres je ošteti i nuklearna postrojenja u elektrani u Fokušimi (udaljene oko 120km od epicentra potresa) što je izazvalo dodatne probleme i zabrinutost na celoj Zemljinoj kugli, pa su mnoge nuklearke ugašene, a odustalo se od izgradnje mnogih planiranih nuklearnih elektrana. Zaštita od dejstva zemljotresa iziskuje dodatna materijalna sredstva. To je u razvijenijim državama stepen zaštite od potresa uopšteno veći. Za ilustraciju poređena su dva zemljotresa iste magnitude, sličnih topografskih i geoloških uslova, slične gustine naseljenosti u dve socijalno-ekonomski gledano različite društvene sredine u Armeniji i u Kaliforniji. Posledice su bile približno 350 puta više poginulih i 70 puta više je ljudi ostalo bez krova nad glavom u društveno siromašnijoj sredini [3]. Celokupna teritorija naše zemlje nalazi se u seizmički aktivnom području [1].

1.1 SEIZMIČKE PRORAČUNSKE METODE

Seizmički proračun je samo korak u postupku projektovanja neke građevine sa posebnim osvrtom na dejstvo zemljotresa. Da bi se sproveo proračun prethodno se mora izabrati konstruktivni sistem i definisati seizmičko opterećenje, a nakon proračunske analize sledi dimenzionisanje prilagođeno specifičnim uslovima potresa, a kao najvažniji korak za konkretnu građevinu oblikovanje konstrukcije, posebno detalje. Seizmičke metode proračuna se kreću u rasponu od jednostavnih i racionalnih inženjerskih postupaka do teorijskih komplikovanih i detaljnih numeričkih analiza.

Dinamička jednačina ravnoteže sistema sa više stepeni slobode izloženog ubrzanju tla od potresa glasi:

$$M\ddot{u} + K\dot{u} + Cu = -M\ddot{u}_g \quad (1)$$

Gde je:

- M matrica masa sistema,
- K matrica prigušenja sistema,
- C matrica krutosti sistema,
- u* vektor pomeranja
- \dot{u} vektor brzine
- \ddot{u} vektor ubrzanja
- \ddot{u}_g ubrzanje tla posmatranog sistema
- I jedninični vektor.

Osnovne metode seizmičkog proračuna su: ekvivalentna statička metoda, spektralna analiza, direktna dinamička analiza (time history) i nelinearna statička (push-over) analiza.

Ekvivalentna statička i spektralna analiza su linearne metode proračuna, a druge dve su uopšteno nelinearni postupci analize. Prva i četvrta metoda su statičke dok su druge dve dinamičke. Ekvivalentna statička metoda ili postupak sa zamenjujućim silama na mestima gde su skoncentrisane najveće mase je najjednostavniji i najstariji postupak proračuna neke konstrukcije na dejstvo zemljotresa.

Metoda spektralne analize je u novije vreme uobičajena metoda za projektovanje seizmički otpornih konstrukcija. To je u osnovi dinamički proračun uz pretpostavku linearno-elastičnog ponašanja materijala.

Metoda direktne dinamičke analize ili dinamička analiza u vremenu podrazumeva dinamički proračun uz pretpostavku nelinearnog rada materijala. U određenim vremenskim intervalima Δt integrali se sistem diferencijalnih jednačina kretanja konstrukcije. Proračun se ne može obaviti bez računarske tehnike i odgovarajućih kompjuterskih programa.

Nelinearna statička analiza ili push-over analiza se značajnije koristi u seizmičkoj analizi građevina u poslednjih nekoliko decenija. Konstrukcija je izložena statičkom opterećenju koje se monotono povećava i traži se odgovor konstrukcije a pretpostavlja se nelinearno ponašanje materijala. Naziv push-over je povezan sa granicom kada je kompletna konstrukcija dostigla ili prešla granicu statičke nestabilnosti.

Metode proračuna se razlikuju kako kvalitativno, tako i po potrebnom vremenu, ali i znanju za sprovođenje i razumevanje proračuna. Modalno spektralna analiza predstavlja osnovni metod proračuna na osnovu čijih rezultata sledi dimenzionisanje konstrukcije [3]. U metodi ekvivalentne statičke analize pretpostavlja se dejstvo horizontalnih statičkih sila u nivou tavanica, a zgrada zgrada je pretpostavljena zamenjujućim konzolnim štapom. Prema relaciji iz statike konstrukcija:

$$F_{\max} = C \cdot u_{\max} \quad (2)$$

Za određeni period oscilovanja jednak spektralnom pomeranju sledi:

$$F_{\max} = C \cdot S_d \quad (3)$$

Imajući u vidu da je $S_d = \frac{S_a}{\omega^2}$ dobija se:

$$F_{\max} = C \frac{S_a}{\omega^2} = m \cdot S_a \quad (4)$$

Gde je:

F_{\max} maksimalna sila naprezanja u konstrukciji,

u_{\max} maksimalno relativno pomeranje,

S_d spektralno pomeranje,

S_a spektralno ubrzanje,

m masa sistema.

Ukupna ekvivalentna seizmička sila jednaka je ukupnoj horizontalnoj statičkoj sili, tj.

$$F = M \cdot S_a \quad (5)$$

Gde je:

F ukupna ekvivalentna seizmička sila,

M ukupna masa (sračunata prema Pravilniku [1])

Raspored ukupne zamenjujuće sile od zemljotresa po visini zgrade glasi:

$$F = (F - \Delta F_n) \frac{m_j \cdot h_j}{\sum_{j=1}^n m_j \cdot h_j} \quad (6)$$

Gde je:

F_j ekvivalentna sila na j-tom spratu.

ΔF_n posebna koncentrisana sila na vrhu zgrade,

m_j masa j-tog sprata,

h_j visina j-tog sprata iznad temelja.

Prema Pravilniku [1] sila na vrhu zgrade iznosi 15%F, a recimo Evrokod 8 [2] ne predviđa koncentrisanu silu na vrhu zgrade.

Ekvivalentna statička metoda prema Evrokodu 8 važi za osnovne vrednosti perioda $T_1 \leq 4T_c$ ili $T_1 \leq 2.0s$. Gde je T_c period oscilovanja uzet iz odgovarajuće tabele date u EC8 za odgovarajuću kategoriju tla. Ukupna seizmička sila (base shear force) određuje se za svaki od horizontalnih pravaca za koji se zgrada analizira, prema izrazu:

$$F_b = S_d(T_1) m \lambda \quad (7)$$

Gde su :

$S_d(T_1)$ Ordinata projektnog spektra za period T_1 ,

m Ukupna masa zgrade iznad temelja,

λ Korekcionni faktor koji je $\lambda=0.85$, ako je $T_1 \leq 2T_c$, za zgrade više od dva sprata $\lambda=1.0$.

File F_i se apliciraju u visinama spratova, a računaju se prema:

$$F_i = F_b \frac{z_i \cdot m_i}{\sum_{j=1}^n z_j \cdot m_j} \quad (8)$$

Gde je:

F_i horizontalna sila koja deluje na visini i-tog sprata.

F_b ukupna horizontalna seizmička sila,

m_i, m_j mase pojedinih spratova,

z_i, z_j visina i-tog i j-tog sprata iznad temelja.

Za šestospratnu zgradu dat je proračun seizmičkih sila prema EC8 i Pravilniku. Rezultati ekvivalentne statičke metode daju $S=1534.3\text{kN}$. Seizmička sila sračunata prema EC8 iznosi $S_b=3748.9\text{kN}$. Očigledno je da je ukupna seizmička sila prema EC8 2.44 puta veća od one dobijene preme važećem Pravilniku [5]. Međutim, treba napomenuti da ukoliko se izvrši dimenzionisanje armiranobetonskog zida prema odgovarajućim propisima razlika u ukupnoj armaturi je nešto manja od razlike u ukupnoj poprečnoj sili. Razlog je primena dva različita koncepta sigurnosti pri dimenzionisanju i oblikovanju. Rezultati ekvivalentne statičke metode prema EC8 su $M_i=47832.6\text{kNm}$, a premanašim važećem Pravilniku $M_i=20782.8\text{kNm}$, pa je odnos 2.3 puta. Može se uočiti da su razlike u ukupnim momentima savijanja u dnu zgrade manje nego u ukupnoj poprečnoj sili zbog toga što EC8 ne propisuje izdvajanje posebne koncentrisane sile na vrhu zgrade.

3 OŠTEĆENJA GRAĐEVINSKIH OBJEKATA USLED DEJSTVA ZEMLJOTRESA

Za sigurnost neke građevine na seizmička dejstva najvažnija je koncepcija konstruktivnog sistema i kvalitetno građenje. Ma koliko bili detaljni, proraču i dimenzionisanje ne mogu nadomestiti nedostatke u koncepciji i realizaciji nekog građevinskog objekta. Od njih najviše zavisi povoljan odgovor konstrukcije na potres i moguća oštećenja. Od krutosti i nosivosti vertikalnih elemenata zavisi horizontalna krutost građevine izložene potresu, a od posebnog značaja je njihova duktilnost. Jedan od veoma čestih uzroka teških oštećenja pa i rušenja zgrada za vreme potresa su fleksibilno prizemlje ili sprat ili naglo smanjenje krutosti. Veliki ekscentricitet između centra masa i centra krutosti mogu uzrokovati značajne torzione deformacije cele zgrade. Iskustva su pokazala da se nepovoljan odgovor na potres i značaj oštećenja mogu očekivati kod zgrada kod kojih se mešaju okvirni sistem i zidovi od opeke, zbog izrazito različitih svojstava dvaju materijala (armirani beton, opeka ili dva nosiva sistema okvir i zid), zbog različite duktilnosti. Ovo ne treba mešati sa zidovima od opeke kao osnovnim nosivim sistemom za preuzimanje i horizontalnog i vertikalnog opterećenja, a koje se nakon zidanja uokviruju vertikalnim i horizontalnim armiranobetonskim ukrućenjima poznatim pod nazivom „serklaži“.

Širina dilatacione ili seizmičke razdelnice, odnosno razmak između zgrada mora minimalno biti jednak zbiru maksimalnih horizontalnih pomeranja spratova susednih zgrada. Prema važećem pravilniku dilatacija se sračunava prema izrazu:

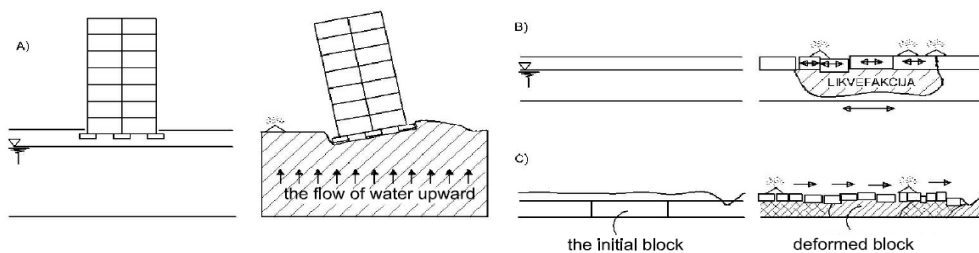
$$\delta = 3.0 + \frac{H - 5.0}{3} \quad (9)$$

Znači da je minimalna razdelnica 3cm i to važi za visine do 5m, a nadole za svako povećanje visine zgrade za 3m širina razdelnice se povećava za 1cm. Na slici 1 prikazan su sudari dve susedne zgrade, Mexico City 1985.



Slika 1 – Sudari dve susedne zgrade Mexico Sity 1985.

Prilikom potresa može doći i do poremećaja u ravnoteži tla neposredno ispod građevine, pogotovo kod tla podložnog likvefakciji, recimo peskovito tlo zasićeno vodom. Može se reći i da je tlo izgubilo nosivost. Jedna od posledica je da usled gubitka ravnoteže u tlu dođe do prevrtanja građevine, pri čemu nije bilo nekih značajnih prethodnih oštećenja. Ovo je zabeleženo za vreme mnogih zemljotresa u istoriji, recimo Filipini 1954, Skopje 1963, zapadna Turska 1998 i 1999. Prethodno opisana pojava je šematski prikazana na slici 2. Moguće je delimično potonuće zgrade uz blago naginjanje ili prevrtanje cele zgrade usled gubitka stabilnosti u tlu.



Slika 2 – Types of damage caused by the effect of liquefaction



Slika 3 – Zgrada nagnuta i oslonjena na susednu zgradu Adapazari Turska 1999 (levo), potonuće zgrade usled likvefakcije (desno)

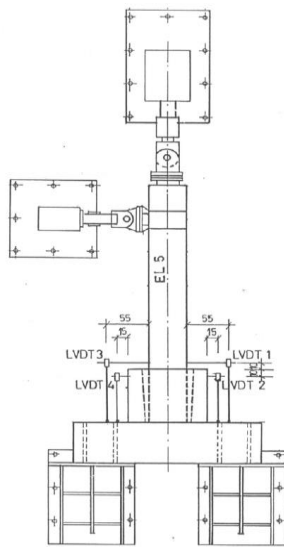
4 KONSTRUKTIVNI SISTEMI SEIZMIČKI OTPORNIH ZGRADA

Prvi koraci u projektovanju su ispravno postavljanje dilatacionih razdelnica i definisanje konstruktivnog sistema za preuzimanje horizontalnog i vertikalnog opterećenja od najvišeg sprata do temelja. Jednostavni konstruktivni sistem sa razumljivim tokom prenosa opterećenja je najbolji sistem za ukrućenje zgrade izložene zemljotresu. Greške učinjene u globalnoj koncepciji noseće konstrukcije ne mogu se popraviti proračunskim analizama, bez obzira koliko one bile detaljne. Osnovni problem koji treba rešiti je preuzimanje horizontalnih sila od dejstva zemljotresa. Oblikovanje građevine za preuzimanje i dalje prenošenje vertikalnog opterećenja je obično jednostavniji zadatak i one poseduju dovoljnu rezervu za dodatna naprezanja usled vertikalnih seizmičkih sila. Preuzimanje horizontalnih seizmičkih sila se takođe vrši preko vertikalne nosive konstrukcije koju treba osmisliti i za taj, teži deo projektnog zadatka. Pri tome se nenosivi elementi uglavnom zanemaruju jer je većina njih relativno fleksibilno povezana sa nosivom konstrukcijom. Međutim, treba voditi računa da i nenosivi elementi prate odgovor nosivog sistema, tako da prva oštećenja nastaju upravo na pregradnim zidovima i drugim nenosećim konstruktivnim elementima.

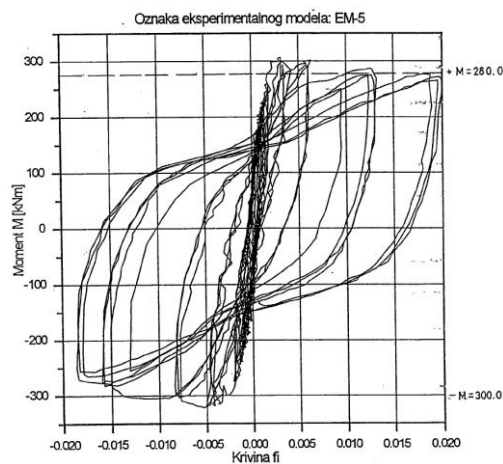
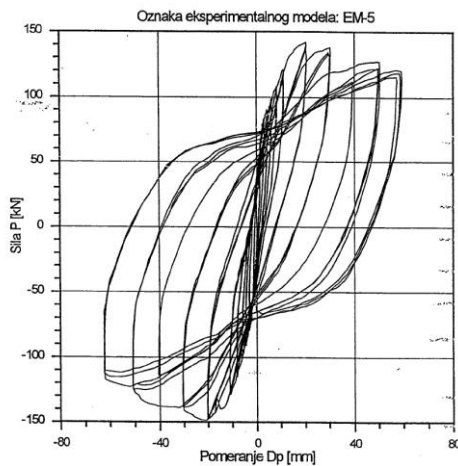
Pravilno projektovanja aseizmička konstrukcija treba da ispuni minimalne zahteve u pogledu: krutosti, nosivosti i duktilnosti. Ova tri svojstva su veoma važna za postizanje potrebnih ciljeva. Krutost nosive konstrukcije je značajna za ograničenje relativnih horizontalnih pomeranja spratova koja mogu nastati pri potresima slabijeg intenziteta. Nosivost bi trebala biti tolika da za učestale potrese manje jačine odgovor konstrukcije ostane uglavnom u linearno-elastičnom području [4]. Pri snažnijim potresima dozvoljava se nelinearno ponašanje konstrukcije i formiranje plastičnih zglobova na najnapregnutijim mestima konstruktivnih elemenata. Duktilnost konstrukcije je važna za izbegavanje naglog rušenja građevine usled delovanja najačih zemljotresa koji se očekuju na nekom području. Veća duktilnost znači i veću sposobnost deformisanja u nelinearnom području odgovora [3]. Krutost, nosivost i duktilnost imaju različito relativno značenje u zavisnosti od mogućih posledica dejstva zemljotresa i kakva vrsta zaštite se želi od njih postići.

5 EKSPERIMENTALNA ISTRAŽIVANJA

Eksperimentalna istraživanja su veoma važna u zemljotresnom inženjerstvu u sklopu analiza šteta nastalih prilikom razornih zemljotresa imaju odlučujuću ulogu za razvijanje uputstava za seizmičko projektovanje, građenje i unapređenje seizmičkih propisa. Ispitivanje delova nosivih konstrukcija vrši se najčešće do loma pošto obično prelaze u nelinearno područje odgovora. Ispitivanje se vrši do stanja naprezanja i deformacija kada se može reći da je dostignut kapacitet nosivosti ili gotovo iscrpljena nosivost, a u nekim slučajevima i stabilnost konstrukcije. Na slici 4 prikazan je sistem za kvazi-statičko ispitivanje. Svojtvo konstrukcije koje karakteriše nelinearno ponašanje a da pri tome ne dođe do njenog rušenja naziva se duktilnost. Duktilne konstrukcije mogu svojim nelinearnim radom rasipati energiju unetu potresom. Pri tome dolazi do formiranja "histerezisne petlje" (slika 5). Površina koju zatvaraju ove krive je mera disipacije energije putem nelinearnog rada nosive konstrukcije [5].



Slika 4 – a) dispozicija eksperimentalnog modela EM-5, b) model pripremljen za ispitivanje



Slika 5 – a) relacija ciklična sila pomernje kraja stuba EM-5, b) relacija moment krivina

U razmatranom slučaju armiranobetonskog stuba sa čašicom duktilno pomeranje se označava kao globalno, a u slučaju zakrivljenosti kao lokalna duktilnost [6]. Dve duktilnosti su međusobno spregnute ali im se apsolutne brojčane vrednosti razlikuju tj. koeficijent zakrivljenosti je znatno veći. Za isti pretpostavljeni zemljotres može se projektovati nosiva konstrukcija sa velikom nosivošću i malom duktilnošću i obrnuto niska nosivost i velika duktilnost. Projektovanje objekta da se ponaša linearno elastično, pri jakom zemljotresu nema ekonomskog a ni tehničkog opravdanja. Zato treba usaglasiti duktilnost i nosivost. Dogodio se

lom stuba dok je čašica ostala neoštećena (detaljno ispitivanje ovog sistema, s obzirom na prostor i temu rada, nije prikazano).

6 ZAKLJUČAK

Na osnovu svega napred rečenog mogu se izvući vrlo značajni zaključci o projektovanju i građenju objekata seizmičkim područjima. S obzirom na kompleksnost i nedovoljna pouzdanost početnih podataka o karakteristikama tla i konstrukcije, a naročito nepoznatom dejstvu opterećenja zato je vrlo teško obezbediti sigurnost objekata sa zadovoljavajućim nivoom seizmičke bezbednosti. Na ovako malom prostoru za prikaz rada nemoguće je prikazati sve uticaje čije je dejstvo nedvosmisleno značajno za bezbednost objekata, tj. da je to što manje gubitaka ljudskih života i povreda, a da su materijalne štete takve da su ekonomski opravdane a finansijski prihvatljive. Seizmičke metode proračuna se kreće u rasponu od jednostavnih i racionalnih inženjerskih postupaka do kompleksnih numeričkih analiza. Osnovne metode proračuna su: ekvivalentna statička metoda, spektralna analiza, dinamička analiza u vremenu, nelinearna statička analiza (push-over) analiza i dr. Date su vrednosti seizmičkih sila sračunate prema našem važećem pravilniku za objekte visokogradnje i prema Evrokodu 8, ali je razlika vrlo velika (2.44), što može ukazivati na zastarelost naših propisa, što znači da ih treba prilagoditi Evropskim normama jer razlika nije na strani sigurnosti.

Kraljevački zemljotres 2010 magnitude $M=5.4$ stepeni Rihterove skale izazvao je velika oštećenja zgrada što ukazuje na potrebu za bezbedniju aseizmičku izgradnju novih i ojačanje postojećih zgrada. Seizmičkim razdelnicama se treba posvetiti naročita pažnja jer pri srednje jakim i jakim zemljotresima dolazi do sudara zgrade i značajnih oštećenja što se može videti sa priloženih slika zemljotresa (Adapazari Turska 1999. i Mexico Sity 1978.). O katastrofalnom uticaju navedenih zemljotresa, gde su na slikama prikazane nagnute i potonule zgrade usled uticaja likvefakcije, vodom zasićenih peskova je jasno vidljiva.

Na kraju rada je ukazano na veliki značaj eksperimentalnih istraživanja u zemljotresnom inženjerstvu. Na slici je prikazana armiranobetonski stub u čašici koji je opterećen gravitacionim i bočnim opterećenjem do loma. Na histerezisnoj petlji duktilnog pomeranja i duktilne zakrivljenosti ispitanog stuba u prirodnoj veličini pokazano je njegovo ponašanje do loma. Jasno da je ovo samo mali deo izvedenih eksperimentalnih istraživanja i ispitivanja.

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VIBRACIJE VISKOELASTIČNOG POLUPROSTORA

Rezime:

S-46

Numerički model tla predstavlja važan deo analize dinamičke interakcije tla i objekta. Model tla treba da obuhvati različite fenomene koji se javljaju u tlu pri dinamičkom opterećenju, pre svega radijacijsko prigušenje. U ovom radu, fenomen radijacijskog prigušenja je analiziran primenom dva različita pristupa: analitičkim i diskretnim. Analitički model, razvijen od strane autora, koristi Metodu integralne transformacije koja se bazira na rešenju Lamé-ovih diferencijalnih jednačina. Diskretni model se bazira na metodi konačnih elemenata i primeni komercijalnog softverskog paketa Abaqus. U oba slučaja, tlo je predstavljeno kao elastičan, homogen i izotropan poluprostor. Posmatrane su vertikalne vibracije usled prinudne sile. Prikazani rezultati za 3 karakteristične frekvencije pokazuju dobro slaganje.

Ključne reči: Dinamika tla, Metoda integralne transformacije, Abaqus

VIBRATIONS OF VISCOELASTIC HALFSPACE

Summary:

Numerical model of the soil is an essential part of the dynamic soil – structure interaction analysis. The soil model should take into account various phenomena that may occur due to dynamic loading, first of all radiation damping. In this paper, the damping phenomena is investigated by comparing the results of two different modeling approaches: analytical and discrete. Analytical model, developed by author, is obtained by using Integral transform method, based on Lamé's differential equations solution. Discrete model is based on the finite element method and commercial software package Abaqus. Soil is considered an elastic, homogeneous and isotropic halfspace. Vertical vibrations due to dynamic force are considered. The results, presented for 3 characteristic frequencies, are in good agreement.

Keywords: Soil dynamics, Integral Transform Method, Abaqus

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1 INTRODUCTION

Dynamic Soil – structure interaction (SSI) has become important research topic over last few decades, especially with the fast development of cities and transportation systems. Structural health, laboratory equipment maintenance and comfort of living could be highly improved by performing a thorough SSI analysis. The analysis is usually carried out in parts, observing the soil and the structure independently. It is fair to assert that the structure model is less demanding than the soil model, since the structure is designed by engineers. Therefore, one could claim that the soil model is an essential part of the SSI analysis. It has to take into account all the phenomena that may occur in soil. Radiation damping is a phenomenon that drew the most research attention to it. It was discovered by Reissner [1] who investigated the response of the cylindrical disk on an elastic halfspace. It states that wave propagates from the source toward infinity. Radiation damping is well modeled by using analytical methods that consider the soil as a semi – infinite medium. One of the most used analytical method is Boundary element method (BEM) [2]. It is based on solving linear partial differential equations which are formulated as integral equations in boundary integral form. Unlike analytical, the discrete methods are applicable to problems of irregular and complex geometry. However, a medium has to be finite in order to be discretized. Therefore, the soil is considered as a bounded box instead of a halfspace. The distortion of the response could be avoided by setting the boundaries far enough from the source of vibrations. That results in robust models especially in the case of high frequency range excitations where the discretization has to be fine. This problem is solved by introducing boundaries that are able to transmit the wave energy outside the model. Kausel and Wass [3] were the first to invent such a boundary. They called it transmitting boundary. Nowadays, various types of boundaries are in use. One of them are infinite elements that were used in acoustics initially. A commercial software package Abaqus [4] uses infinite elements for solving infinite medium problems. In this paper, the results of two different approaches, analytical and discrete, are compared.

2 SOIL MODEL - FORMULATION AND SOLUTION

The vertical response of the surface of the halfspace due to the surface distributed harmonic load of length 1m and amplitude 10 kN/m is calculated by using two different approaches, analytical and discrete. The soil is considered as an elastic, isotropic and homogeneous halfspace. The analytical model is based on Integral transform method. The solution is obtained by using own computer program developed in Mathworks Matlab [5]. The discrete model is carried out by using commercial finite element program Abaqus with the help of infinite elements.

2.1 ITM

The Integral Transform Method is based on the analytical solution of Lamé's differential equations of motion of the continuum:

$$\mu \nabla^2 \mathbf{u} + (\lambda + \mu) \nabla \nabla \cdot \mathbf{u} = \rho \ddot{\mathbf{u}} \quad (1)$$

where:

ρ is mass density of the material,
 u is displacement vector,

$$\nabla^2 = \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2} + \frac{\partial^2}{\partial z^2}, \quad \nabla = \left[\frac{\partial}{\partial x} \quad \frac{\partial}{\partial y} \quad \frac{\partial}{\partial z} \right] \text{ and}$$

λ, μ are Lamé's material constants through which the damping model is introduced:

$$\mu = E \frac{1+2i\zeta}{2(1+\nu)}, \quad \lambda = 2E\nu \frac{1+2i\zeta}{1-2\nu} \quad (2)$$

where:

E is elasticity modulus,
 ν is Poisson's coefficient and
 ζ is damping ratio.

Lamé's equations of motion (1) can be brought into the form of wave equations:

$$\nabla^2 \varphi = \frac{1}{c_p^2} \ddot{\varphi}, \quad \nabla^2 \boldsymbol{\psi} = \frac{1}{c_s^2} \ddot{\boldsymbol{\psi}} \quad (3)$$

where the displacement vector is expressed by the scalar field φ and the vector field $\boldsymbol{\psi}$ that have to satisfy the relation (4), according to the Helmholtz's principle:

$$\nabla u = \nabla \varphi + \nabla \times \boldsymbol{\psi} \quad (4)$$

In Eqs. (3) c_p and c_s are the velocities of the dilatational and shear waves, respectively:

$$c_p^2 = \frac{\lambda + 2\mu}{\rho}, \quad c_s^2 = \frac{\mu}{\rho} \quad (5)$$

The system of equations (3) are transferred from the space/time domain (x, y, z, t) into the wave number/frequency domain (k_x, k_y, z, ω) by using threefold Fourier transform:

$$\hat{f}(k_x, k_y, \omega) = \int_{-\infty}^{+\infty} \int_{-\infty}^{+\infty} \int_{-\infty}^{+\infty} f(x, y, t) e^{-i(k_x x + k_y y + \omega t)} dx dy dt \quad (6)$$

where k_x and k_y are wavenumbers in x and y direction, respectively.

By assuming that $\boldsymbol{\psi}_z=0$, the system of partial differential equations (3) becomes a system of three decoupled ordinary differential equations with six unknown coefficients of integration, \mathbf{C} .

The coefficients of integration are obtained taking into account Sommerfeld's radiation condition and the boundary conditions at the surface of the halfspace. In the case of vertically applied load $p(x)$ at the surface of the halfspace, boundary conditions are given as shown on Figure 1.

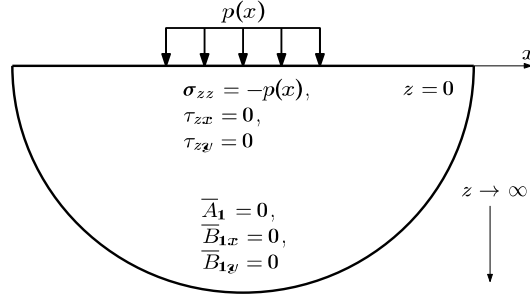


Figure 1. ITM – boundary conditions

The relation between the displacement vector and the coefficients of integration is derived from Eqs (3) and (4) in the following form:

$$\hat{\mathbf{u}} = \mathbf{A}^u \cdot \mathbf{C} \quad (7)$$

where

$$\hat{\mathbf{u}} = \begin{Bmatrix} \hat{u}_x \\ \hat{u}_y \\ \hat{u}_z \end{Bmatrix}, \quad \mathbf{A}^u = \begin{bmatrix} ik_x & 0 & \lambda_2 \\ ik_y & -\lambda_2 & 0 \\ -\lambda_1 & -ik_y & ik_x \end{bmatrix}, \quad \mathbf{C} = \begin{Bmatrix} A_2 \\ B_{2x} \\ B_{2y} \end{Bmatrix} \quad (8)$$

and

$$\lambda_1^2 = k_x^2 + k_y^2 - k_p^2, \quad k_p = \frac{\omega}{c_p}, \quad \lambda_2^2 = k_x^2 + k_y^2 - k_s^2, \quad k_s = \frac{\omega}{c_s}. \quad (9)$$

Following the stress – strain relationship for elastic, isotropic and homogeneous material, one could derive the relation between the stress vector and coefficient of integration [6]:

$$\boldsymbol{\sigma} = \mathbf{A}^\sigma \cdot \mathbf{C} \quad (10)$$

where

$$\boldsymbol{\sigma} = \begin{Bmatrix} \sigma_{zz} \\ \tau_{zx} \\ \tau_{zy} \end{Bmatrix}, \quad \mathbf{A}^\sigma = \begin{bmatrix} 2(k_x^2 + k_y^2) - k_s^2 & 2ik_y\lambda_2 & -2ik_x\lambda_2 \\ -2ik_y\lambda_1 & \lambda_2^2 + k_y^2 & -k_xk_y \\ -2ik_x\lambda_1 & k_xk_y & -\lambda_2^2 - k_x^2 \end{bmatrix}, \quad \mathbf{C} = \begin{Bmatrix} A_2 \\ B_{2x} \\ B_{2y} \end{Bmatrix} \quad (11)$$

Once the displacements, $\hat{\mathbf{u}}$, in the transformed domain are obtained from Eqs. (7) and (10), the displacements, \mathbf{u} , in the original domain are calculated by using threefold inverse Fourier transform:

$$f(x, y, t) = \frac{1}{(2\pi)^3} \int_{-\infty}^{+\infty} \int_{-\infty}^{+\infty} \int_{-\infty}^{+\infty} \hat{f}(k_x, k_y, \omega) e^{i(k_x x + k_y y + \omega t)} dk_x dk_y d\omega. \quad (12)$$

In the case of plain strain analysis k_y is set to be zero.

The vertical response of the surface of the halfspace due to the surface distributed harmonic load of length 1m and amplitude 10 kN/m, Figure 2, are obtained for three different frequencies of excitation, 10, 50 and 100 Hz and presented in Figures 5-7 together with the response obtained by FEM.

2.2 FINITE ELEMENT MODEL IN ABAQUS

In finite element method (FEM), excitation produces waves which remains trapped in the model. These waves reflect from the boundary of finite elements and increase the response of the model, Figure 2.

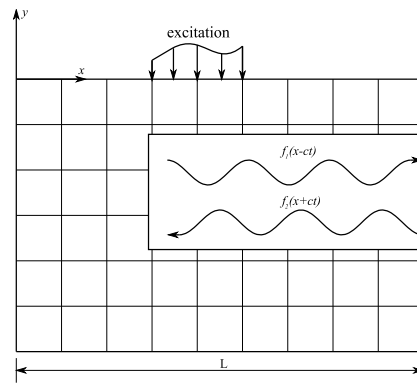


Figure 2. Finite element mesh of soil model with trapped waves

This can be avoided by using a large area of soil in the finite element model, so that waves could attenuate before reach the boundary. More elegant way to solve this issue is to use infinite elements as a “quiet” boundary, which does not reflect waves back into the model. This approach enables only a small portion of soil to be modeled, which has a significant impact on time duration of analysis. Abaqus provides infinite elements that are based on the works of Zienkiewicz et al. [7] for static analysis and of Lysmer and Kuhlemeyer [8] for dynamic response. The standard finite elements, which model the area around the region of interest, are used in conjunction with the infinite elements modeling the far-field region. Because the solution in the far field is assumed to be linear, only linear behavior is provided in the infinite elements. The general behavior of the infinite elements is based on modeling displacement \mathbf{u} , with respect to distance r measured from a “pole” of the solution, so that $\mathbf{u} \rightarrow 0$ as $r \rightarrow \infty$ and $\mathbf{u} \rightarrow \infty$ as $r \rightarrow 0$. Further, the dynamic response of the infinite elements is based on consideration of plane body waves travelling orthogonally to the boundary.

We consider plane waves travelling along the x -axis, Figure 2. Two body wave solutions exist. One describes longitudinal waves, which have the form:

$$u_x = f(x \pm c_p t), u_y = 0 \quad (13)$$

The other is the shear waves:

$$u_y = f(x \pm c_s t), u_x = 0 \quad (14)$$

The solution $f(x - ct)$ describes a displacement wave travelling at velocity c in the positive x -direction and another, $f(x + ct)$, travelling at the same speed in the negative x -direction. We consider a boundary at $x=L$ of a medium modeled by finite elements, Figure 2, and introduce distributed damping on this boundary, such that there are no forces at boundary:

$$\sigma_{xx} + d_p \dot{u}_x = 0, \quad \sigma_{xy} + d_s \dot{u}_y = 0 \quad (15)$$

It is necessary to choose the damping constants d_p and d_s in such a way that reflection of longitudinal and shear wave energy back into the medium is zero. Plane, longitudinal waves approaching the boundary have the form $u_x = f_1(x - c_p t)$, $u_y = 0$. If they are reflected completely as plane, longitudinal waves, their reflection will be of the form $u_x = f_2(x + c_p t)$, $u_y = 0$. Since the problem is linear, superposition determines the total displacement $u_{tot} = f_1 + f_2$, with corresponding stresses $\sigma_{xx} = (\lambda + 2G)(f_1' + f_2')$ and all other $\sigma_{ij} = 0$. Corresponding particle velocity is $\dot{u} = c_p(f_2' - f_1')$. Now we can introduce these relations into equation (15), which results in the following:

$$(\lambda + 2G - d_p c_p) f_1' + (\lambda + 2G + d_p c_p) f_2' = 0 \quad (16)$$

From the previous equation, we can ensure that $f_2 = 0$ (which implies $f_2' = 0$) for any f_1 by choosing:

$$d_p = \frac{\lambda + 2G}{c_p} = \rho c_p \quad (17)$$

Similarly, for shear waves we obtain:

$$d_s = \rho c_s \quad (18)$$

These values of boundary damping are built into the infinite elements used in Abaqus. From the above discussion it is obvious that they transmit all normally impinging plane body waves exactly (provided that the material behavior close to the boundary is linear elastic). General problems involve non-plane body waves that do not approach the boundary in orthogonal direction, such as Rayleigh surface waves and Love waves. Nevertheless, these “quiet” boundaries work quite well even for such general cases, provided that they are arranged so that the dominant direction of wave propagation is orthogonal to the boundary. Since the boundaries are “quiet” rather than “silent” (perfect transmitters of all waveforms), and because the linear elastic behavior of the soil nearby the boundaries is assumed, they should be placed at some reasonable distance from the region of main interest. It has to be pointed out that the infinite elements provide “quiet” boundaries to the finite element model through the effect of a damping matrix, while the stiffness matrix of the element is suppressed. The elements do not provide any contribution to the eigenmodes of the system.

A soil model has been made in Abaqus by using the infinite elements. The solid infinite elements are based on the assumption that the far-field solution along each element edge stretches to infinity is centered about an origin, called the “pole”. For example, the solution for a point load applied to the boundary of a half-space has its pole at the point of application of the load. It is necessary to choose the position of the nodes in the infinite direction appropriately with respect to the pole. The second node along each edge pointing in the infinite direction must be positioned as twice as far from the pole as the node on the same edge at the boundary between the finite and the infinite element, Figure 3a. In addition to this

consideration, one must specify the second nodes in the infinite direction such that the element edges in the infinite direction do not intersect, Figure 3b.

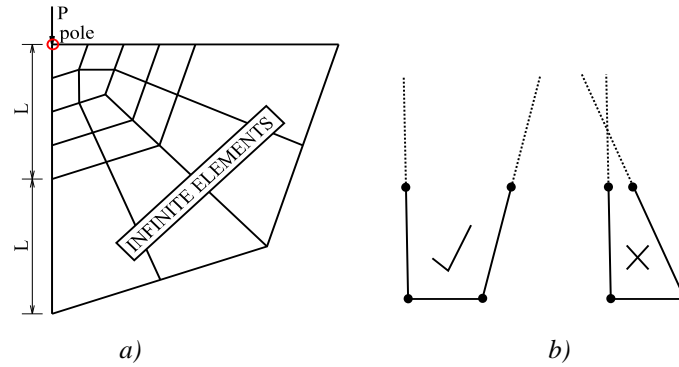


Figure 3. Appropriate disposition of infinite elements with respect to point load – pole (a); acceptable and unacceptable infinite element (b), according to [4]

The halfspace is excited by the same surface distributed harmonic load of length 1m and amplitude 10 kN/m. The finite element model in ABAQUS consists of soil part which is meshed by using plain strain finite elements (CPE4R) and quiet boundary which is modeled by using infinite elements (CINPE4), Figure 4. Approximate size of the finite elements is 0.25m, which satisfies Lysmer’s condition for the highest frequency (100 Hz) considered in this analysis [9].

On the surface ($y=0$), soil extends from -20m to 20m. Infinite elements are placed from -20m to -40m, and from 20m to 40m. Soil depth extends up to 17.5m. This model meets the requirement that the distance between the first node of the infinite element and “pole” is approximately equal to the distance between the second and the first node of the infinite element.

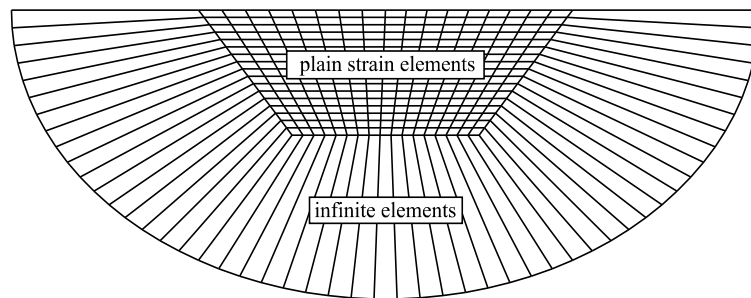


Figure 4. Plane soil model in Abaqus, meshed by using finite and infinite elements

The steady-state plain strain analysis has been performed in the frequency domain. This type of analysis is used to calculate response of the soil model to harmonic excitation at 10, 50 and 100 Hz in order to make comparison with ITM solution. Damping is introduced in the analysis in frequency domain through complex modulus:

$$E^* = E(1 + 2i\xi) \quad G^* = G(1 + 2i\xi) \quad (19)$$

where ξ is damping ratio.

3 NUMERICAL RESULTS

Figures 5 – 7 show the response of the surface of the halfspace obtained by using Abaqus and Matlab models for three different frequencies of excitation, 10, 50 and 100 Hz, respectively.

It is obvious that both models respond in a similar sense. However, the differences are visible at the contact between finite and infinite elements ($x=-20\text{m}$ and $x=20\text{m}$). The relative difference is more distinct in the case of lower frequencies. That is due to the fact that the wavelengths for lower frequencies are longer and the amount of space needed to dump the excitation amplitude is higher. These differences could be reduced by setting the boundaries further. That does not have to influence the robustness of the model necessarily, since the size of finite elements is inversely proportional to the frequency. Anyhow, this fact does not affect the response within the sensitive zone under the excitation area. Accordingly, the boundaries should be placed carefully in order to optimize the size of the model without significantly changing the accuracy of the response.

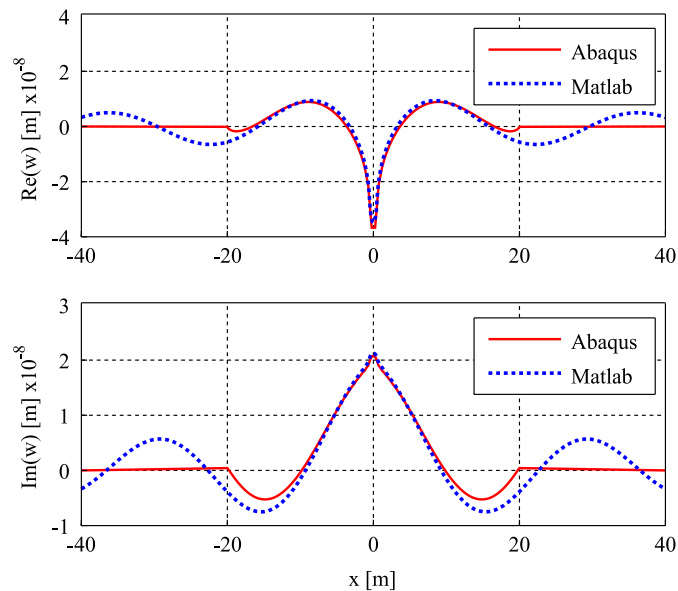


Figure 5. Displacement field of the surface of the halfspace at excitation frequency of 10 Hz

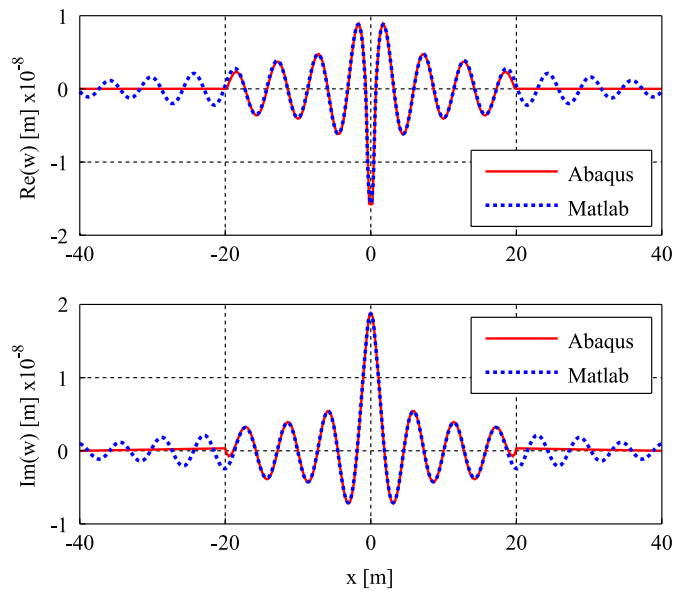


Figure 6. Displacement field of the surface of the halfspace at excitation frequency of 50 Hz

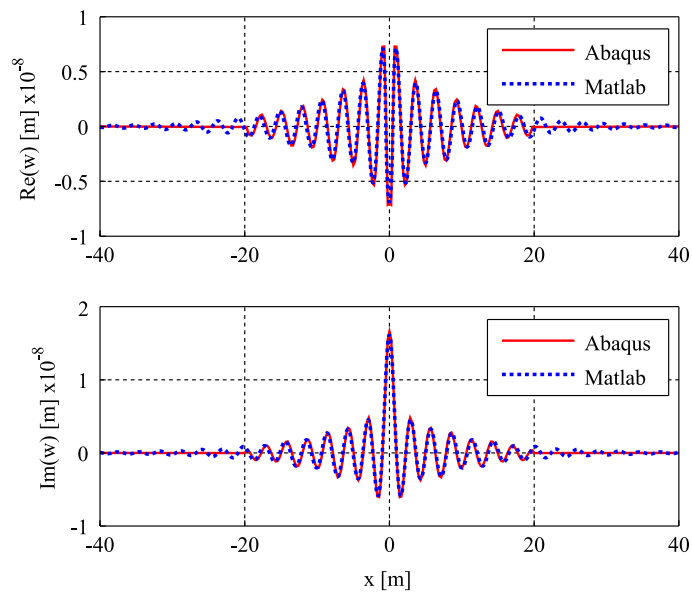


Figure 7. Displacement field of the surface of the halfspace at excitation frequency of 100 Hz

4 CONCLUSIONS

As an essential part of Soil structure interaction analysis the soil model inspired the development of various numerical methods for modeling wave propagation in semi-infinite medium – halfspace. Regarding the approach of solving the system of differential equations, a method could be analytical or discrete. This paper presents the comparison of the results of one analytical approach written in Matlab with the help of Integral transform method, and one discrete approach that uses the finite element method combined with infinite elements, implemented in the commercial software package Abaqus. It is shown that the results could be accurate regardless of the approach as long as the model respects the limitation of the approach. While discrete approach is applicable to the problems of irregular and complex geometry, analytical approach leads to better understanding of the physics of the problem. Therefore, the development of both approaches is very important, as well as the mutual comparison with regard to the verification of the response of the models solved by different methods.

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PROCENA SEIZMIČKOG KAPACITETA ZA GRANIČNA STANJA PREMA EN 1998-3 PRIMENOM N2 METODE

Rezime:

S-47

Određbe EN 1998-1 propisuju N2 metodu kao referentnu za određivanje ciljnog pomeranja koje predstavlja seizmički zahtev. Ova metoda se može upotrebiti za procenu seizmičkog kapaciteta, odnosno vrednosti intenziteta ubrzanja tla koje će uzrokovati odgovarajući nivo oštećenja. Za četiri okvirne konstrukcije iste spratnosti, koje su projektovane za dve klase duktilnosti i dva intenziteta seizmičkog dejstva, procenjene su vrednosti ubrzanja tla za dva granična stanja prema EN 1998-3. Cilj rada je da se procenjene vrednosti uporede sa projektnim i utvrdi uticaj klase duktilnosti i nivoa intenziteta seizmičkog dejstva na njihov odnos.

Ključne reči : N2 metod, granično pomeranje, granična stanja, rotacija tetive

ESTIMATION OF SEISMIC CAPACITY FOR LIMIT STATES ACCORDING TO EN 1998 USING THE N2 METHOD

Summary:

The provisions of EN 1998-1 prescribes the N2 method as a reference method for the determination of the target displacement which represents the seismic demands. This method can be used to estimate the seismic capacity, i.e. the value of ground acceleration intensity that will cause the appropriate level of damage. For four frame structures with same number of storeys, which are designed for two ductility classes and two levels of seismic actions, were estimated values of ground acceleration for two limit states according to EN 1998-3. The aim of this paper is to compare the estimated values with design ones and to determine the influence of ductility class and the level of seismic actions on their relationship.

Key words: N2 method, ultimate displacement, limit states, chord rotation

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1 INTRODUCTION

Provisions of EN 1998-1 are applied to design and construction of buildings and civil engineering works in seismic regions with demands that, in event of earthquake, ensure the protection of human lives, damages limitation and that structure important for civil protection remain operational [1]. Provisions of EN 1998-3 [2] are, among other things, related to providing criteria for evaluation of the seismic performance of existing building structures. Besides the existing structures, evaluation of the seismic performance is necessary for new designed structures too, in order to identify any potential weaknesses and deficiencies in new structures that are designed with appropriate, acceptable level of structural damage in case of design earthquake. Measure of seismic performance is expressed through the chord rotation of element, whose capacity depends on the limit states.

Nonlinear static or pushover analysis is widely used for seismic evaluation of buildings and is carried out under constant gravity loads and monotonically increasing horizontal loads until a target displacement is reached. Target displacement is defined as seismic demand, and represents a maximum displacement of control node (top displacement) that is expected during the earthquake. Seismic demand must be based on relevant seismic action for appropriate limit state. The provisions of EN 1998-1 prescribe the N2 method as a reference method for the determination of the target displacement. This method can be used to estimate the value of ground acceleration intensity that will cause the appropriate level of damage.

In this paper RC frames, previously designed according to European codes, were analyzed by method of nonlinear static analysis and pushover curves were determined. Using the N2 method were estimated values of ground acceleration for two limit states according to EN 1998-3. These values were compared with the design ground acceleration. The aim of this paper is to determine the influence of ductility class and the level of seismic actions on their relationship.

2 ULTIMATE DISPLACEMENTS FOR LIMIT STATES

In the current version of EN 1998-1 there are two performance requirements (level): no collapse requirement, for design seismic action with value of return period of 475 years; and damage limitation requirement, for seismic action with return period of 95 years. Unlike for the design, assessment of buildings is carried out for three levels of performance. Performance levels refer to the state of damage in structure and are defined through three limit states: near collapse (NC), significant damage (SD) and damage limitation (DL). Every limit state corresponds to different seismic hazard level. Return periods for the seismic action may be defined in National Annex of countries. Recommended values, for basic objective of protection for ordinary new buildings, given in EN 1998-3, are: for NC limit state - 2475 years; for SD limit state - 475 years; and for DL limit state - 225 years. It can be noticed that return period for SD limit state, for assessing of structure, is equal with the ones for no collapse requirement, in structures seismic design. NC limit state given in EN 1998-3 is closer to the actual collapse of building, than the no collapse requirement given in EN 1998-1.

Nonlinear behaviour due to earthquakes can be observed at the global level in form of pushover curve (top displacement - base shear) or at the local level through appropriate deformation of members (the chord rotation in EN 1998-3). Generally accepted definition of ultimate (near collapse) limit state at level of structure is missing, while at local level, ultimate

limit state of element is when a deformation capacity is reached. The near collapse limit state is used as a conservative approximation of the structural collapse. Possible and practical definition of global ultimate limit states is NC limit state of the most exposed important vertical element [3]. The ultimate displacement which is in relation with collapse of structures corresponds to displacement when it is reached NC limit state of the vertical element.

According to EN 1998-3 the main measure of deformation of RC structures is expressed through chord rotation on elements ends. The chord rotation is the angle between the tangent to the axis at the yielding end and the chord connecting that end with the end of the shear span. The total ultimate chord rotation, for limit state NC, of concrete members (beams, columns and walls) under cyclic loading is:

$$\theta_{NC,um} = \frac{1}{\gamma_{el}} 0.016 \cdot 0.3^v \cdot \left[\frac{\max(0.01; \omega')}{\max(0.01; \omega)} \cdot f_c \right]^{0.225} \cdot \left(\frac{L_v}{h} \right)^{0.35} \cdot 25^{\left(\alpha \cdot \rho_{sx} \cdot \frac{f_{yw}}{f_c} \right)} \cdot 1.25^{100 \cdot \rho_d} \quad (1)$$

where are:

- γ_{el} 1.5 for primary seismic members and 1.0 for secondary ones,
- h the depth of cross section,
- L_v shear span, ratio between moment and shear at the end section,
- v the normalized axial force,
- ω and ω' the mechanical reinforced ratio of the tension and compression, respectively, longitudinal reinforcement,
- f_c and f_{yw} concrete compressive strength and the stirrup yield strength,
- ρ_{sx} ratio of transverse steel parallel to loading x direction,
- ρ_d the steel ratio of diagonal reinforcement in each diagonal direction and
- α the confinement effectiveness factor.

Equation (1) can be used to determine the chord rotation capacity depending on the type of loading (monotonically or cyclic) and, although the provisions of Eurocode does not mention, in the case of monotonic loading capacity is 1.75 times higher than the corresponding values for cyclic loads. Total chord rotation capacity for significant damage limit state (SD) is taken as 3/4 of the total ultimate chord rotation capacity (for NC limit state).

Seismological survey of Serbia provides seismic hazard maps for return periods of 95, 475 and 975 years. Seismic hazard map for return period of 475 years is used for design of structures and for assessment for SD limit state. If the assessment determined to limit states NC or DL then the design seismic action should be reduced. Some authors state that seismic actions for checking the limit state NC should amount to 150% of design ones.

3 N2 METHOD

The N2 method combines the pushover analysis of structures model of multi degrees of freedom (MDOF) with the response spectrum analysis of equivalent single degree of freedom (SDOF) system [4]. This method was implemented in EN 1998-1 for the determination of the target displacement which represents the seismic demands.

The N2 method can be used to calculated seismic capacity of structures, i.e. for estimation the value of ground acceleration intensity which will cause the appropriate level of damage in the structure [5], [6]. Based on top displacement of the structures the corresponding limit states can be determined for appropriate ground motion.

4 NUMERICAL EXAMPLES

In this paper, 4 RC frame structures with four storeys, designed for two ductility classes (DCM and DCH) and two cases of seismic action ($a_g = 0.2g$ and $a_g = 0.3g$) were analyzed. RC frame structures are symmetrical in both directions, with 3 bays of 5 m. First storey is 3.5 m height, and for others storeys heights are 3.0 m. Beams of all RC frames have the same section, 30×45 cm, and a slab thickness is 15 cm. Cross section dimensions of columns of all RC frames are 45×45 cm. RC frames structures meet the criteria of regularity in plane and can be analyzed as a 2D structures. Numerical analysis of the planar model of the inner frame with a T section beam of effective width 170 cm was conducted. Concrete class C30/37 and steel S500 class C were used in the design of frames. Seismic actions were represented by the horizontal elastic response spectrum of type 1 and soil category C ($S = 1.15$, $T_B = 0.2$ s, $T_C = 0.6$ s and $T_D = 2.0$ s). Elastic analyses were carried out for two ductility classes, DCM and DCH, with the behaviour factors $q = 3.9$ and $q = 5.85$, respectively. The vertical actions, permanent and imposed loads, were also considered. Permanent load includes self weight of elements and added dead load with assumed value of 2.5 kN/m², so that the total permanent load is 8.05 kN/m². Imposed load for the building category B according to EN 1991-1-1 is taken as equally distributed of 2.5 kN/m² intensity. Frame structures were designed according to EN 1992-1-1 and EN 1998-1. Adopted reinforcements, both the longitudinal and especially transverse reinforcements, depend on fulfilment of special provisions depending on the ductility class. Same columns reinforcement with a certain ductility class and seismic action was adopted. Adopted reinforcement for beams is not constant along the element and is different for sections at the ends and in the middle. For medium ductility class frames adopted reinforcement is a same in all storeys. For high ductility class frames, in order to meet the condition of the limitation diameter of longitudinal bars which pass through beam - column joint to prevent the bond failure, adopted reinforcement is different by storeys. Reinforcement for beams ends and columns for 4 analyzed frames are shown in Table 1, where with m is marked number of legs stirrups. In this table, and the tables and figures that are below, frames are named depending on ductility class and intensity of design seismic action.

Table 1 – Reinforcement for beams ends and columns for 4-storey frames

		BEAMS			COLUMNS	
		Long. reinforcement		Stirrup	Longitudinal reinforcement	Stirrup
		Top	Bottom			
DCM-0.2g		6Ø14+6 Ø8	4Ø14	Ø8/10 $m=2$	12Ø16	Ø8/10 $m=4$
DCH-0.2g	1. - 2.	5Ø14+6 Ø8	5Ø14	Ø8/8 $m=2$	12Ø16	Ø8/7.5 $m=4$
	3. - 4.	3Ø14+6 Ø8	3Ø14	Ø8/8 $m=2$		
DCM-0.3g		6Ø16+6 Ø8	5Ø16	Ø8/10 $m=2$	12Ø16	Ø8/10 $m=4$
DCH-0.3g	1.	6Ø14+6 Ø8	6Ø14	Ø8/8 $m=2$	12Ø16	Ø8/7.5 $m=4$
	2.	5Ø14+6 Ø8	5Ø14	Ø8/8 $m=2$		
	3. - 4	4Ø14+6 Ø8	4Ø14	Ø8/8 $m=2$		

Masses of a dynamic model are lumped at floor levels and amounts 55 t for the first storey, 54.2 t for the second and third and 53.6 t for the fourth storey. The elastic 2D model has a fundamental period $T_1 = 0.69$ s (with cracked section), and first vibration mode of $\square_{\square}^{\square} = [0.33 \ 0.64 \ 0.87 \ 1.00]$. Nonlinear static analysis was carried out under constant gravity

loads and monotonically increasing lateral loads applied on the masses of the structural model. Pushover curves, in terms of top displacement and base shear, were determined for two patterns of lateral load: with linear distribution (L) and uniform distribution (U) (Figure 1 and Figure 2). Pushover analysis was conducted using the OpenSees. According to EC 8, target displacements were determined by the N2 method. The MDOF systems were transformed into equivalent SDOF systems (Table 2). Values of target displacements for two limit states and two patterns of lateral loads are shown in Table 3. The target displacements for limit state SD were determined due to seismic action which corresponding to the design seismic action, while the target displacements for limit states NC were determined by the value which is 1.5 greater than the design ones.

Table 2 – Properties of the equivalent SDOF system

		DCM-0.2g	DCH-0.2g	DCM-0.3g	DCH-0.3g
Linear	m^* (t)	138.73			
	Γ^*	1.333			
	F_v^* (kN)	426.6	412.8	499.7	444.1
	D_v^* (cm)	6.72	6.95	6.44	7.15
	T^* (s)	0.929	0.960	0.840	0.939
Uniform	m^* (t)	217.58			
	Γ^*	1.0			
	F_v^* (kN)	622.9	612.9	714.6	650.1
	D_v^* (cm)	8.06	8.24	7.64	8.23
	T^* (s)	1.054	1.075	0.958	1.043

Table 3 – Target displacement for two limit states and two patterns of lateral loads

	$D_{SD,L}$ (cm)	$D_{NC,L}$ (cm)	$D_{SD,U}$ (cm)	$D_{NC,U}$ (cm)
DCM-0.2g	10.6	15.9	9.0	13.6
DCH-0.2g	11.0	16.5	9.2	13.8
DCM-0.3g	14.4	21.6	12.3	18.5
DCH-0.3g	16.1	24.1	13.4	20.1

The ultimate displacements for limit state NC were estimated on the basis of the achieved chord rotation capacity of columns for NC limit state, and the like for estimation of the ultimate displacements for SD limit state (Table 4).

Table 4 – Ultimate displacement for two limit states and two patterns of lateral loads

	$D_{SD,L}$ (cm)	$D_{NC,L}$ (cm)	$D_{SD,U}$ (cm)	$D_{NC,U}$ (cm)
DCM-0.2g	17.0	21.0	15.0	18.5
DCH-0.2g	19.9	24.9	16.8	20.8
DCM-0.3g	15.0	18.5	13.0	16.0
DCH-0.3g	18.9	23.4	15.9	19.9

Pushover curves for all analyzed frames and for two patterns of lateral load are shown in Figure 2. On the curves are marked target displacements and ultimate displacement for two limit states (SD and NC). It can be observed, that for frames designed for lower seismic action, ultimate displacements are greater than target displacements for the corresponding limit states. In case of frames designed for greater seismic action, ultimate displacements for NC limit state are lower compared to target displacements for same limit state, significantly for frame designed for medium ductility class. In other words, the ultimate displacements where the NC limit states have been achieved in the column are less than seismic demands for 150% of design seismic action.

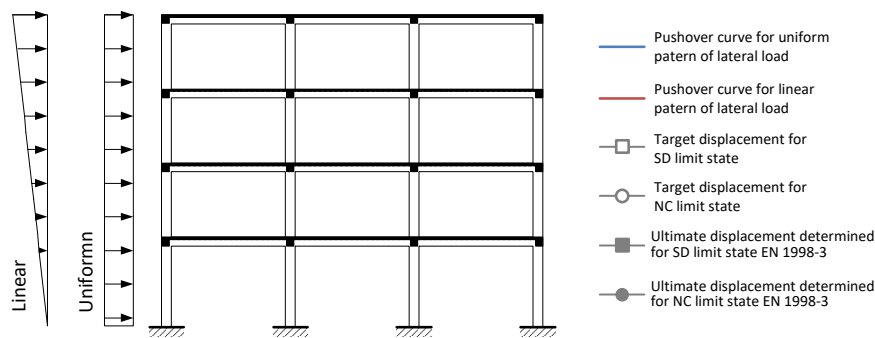


Figure 1 – Two patterns of lateral loads and keys for the pushover curves in Figure 2

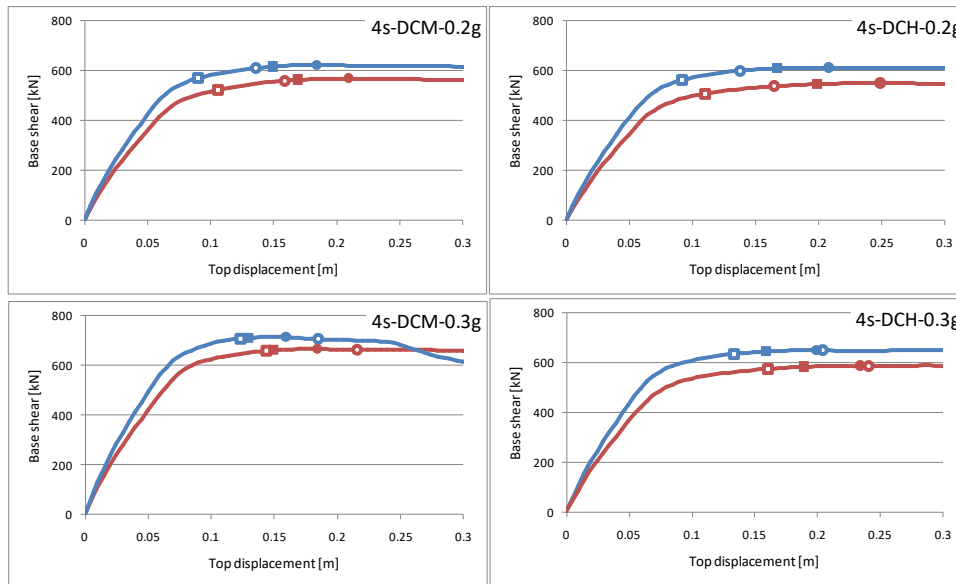


Figure 2 – Pushover curves for RC frames

In Figures 3, 4, 5 and 6 are shown estimated ground acceleration values (seismic capacity) for equivalent SDOF systems in acceleration - displacement (AD) format, for all frames, both

limit states (SD and NC) and two lateral load distributions (L and U). With a gray is shown elastic spectrum.

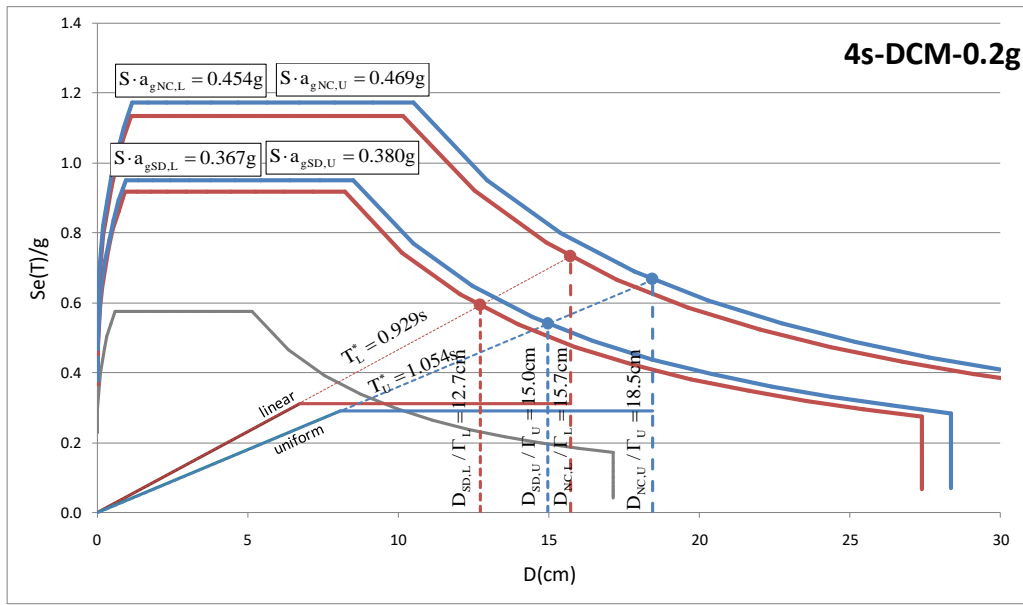


Figure 3 – Estimated ground acceleration values for SDOF system of DCM-0.2g frame

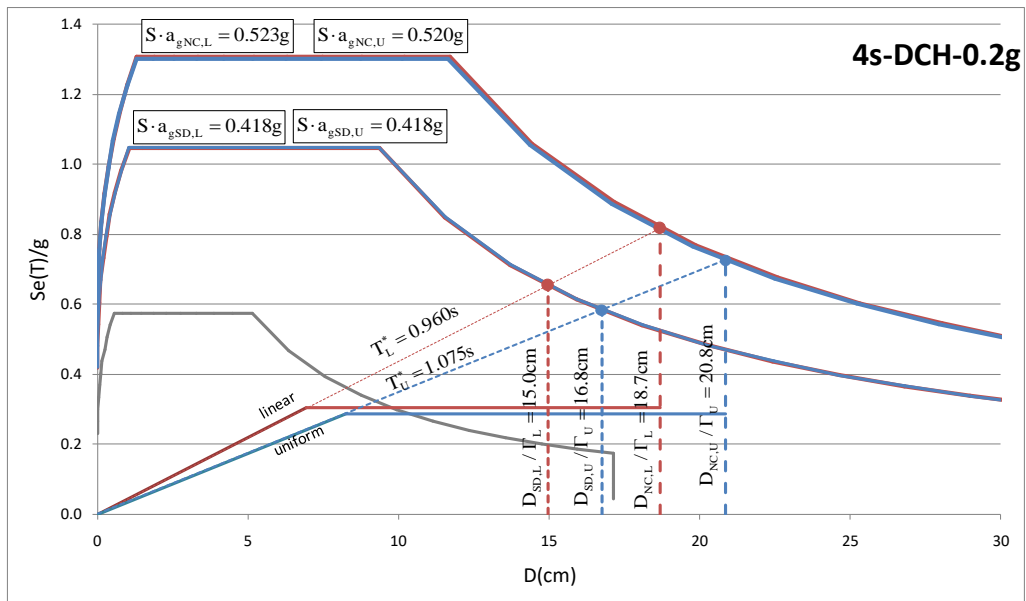


Figure 4 – Estimated ground acceleration values for SDOF system of DCH-0.2g frame

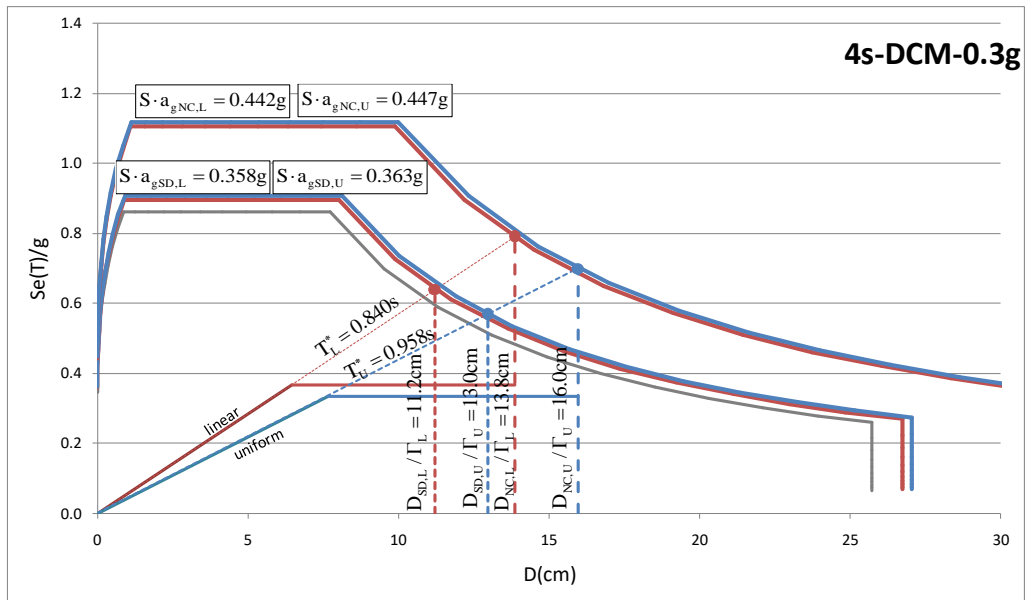


Figure 5 – Estimated ground acceleration values for SDOF system of DCM-0.3g frame

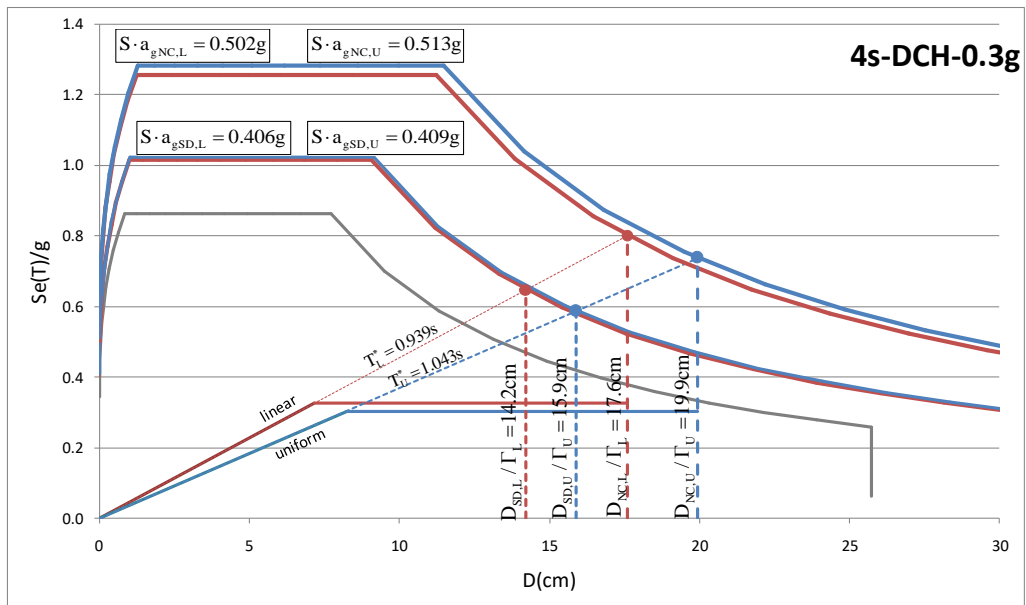


Figure 6 – Estimated ground acceleration values for SDOF system of DCH-0.3g frame

In table 5 are given the estimated values of ground acceleration all analyzed frames for two limit states and two pattern of lateral loads. These values are approximately the same for linear and uniform lateral loads and the expected higher for NC limit state.

Table 5 – The estimated values of ground acceleration for two limit states and two patterns of lateral loads

	$a_{gSD,L}$	$a_{gNC,L}$	$a_{gSD,U}$	$a_{gNC,U}$
DCM-0.2g	0.32	0.39	0.33	0.41
DCH-0.2g	0.36	0.45	0.36	0.45
DCM-0.3g	0.31	0.38	0.32	0.39
DCH-0.3g	0.35	0.44	0.36	0.46

In Figure 7 are shown ratio between the estimated and design ground acceleration values for two limit states and two patterns of lateral loads. At frames designed for same intensity of seismic action DCH frames had greater seismic capacity compared to DCM frames, for both limit states. Also, 0.2g frames had greater seismic capacity compared to 0.3g frames, which can be explained by the higher reserved strength of these frames [7].

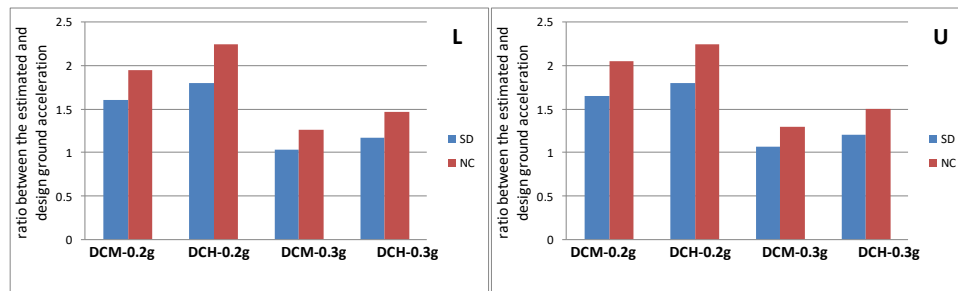


Figure 7 – Ratio between the estimated and design ground acceleration values for two limit states and two patterns of lateral loads

5 CONCLUSION

The N2 method was implemented in EN 1998-1 as a reference method for the determination of the target displacement which represents the seismic demands. This method can be used to calculate seismic capacity of structures, i.e. for estimation the value of ground acceleration intensity which will cause the appropriate level of damage the structure. The state of damage in the structure is defined through limit states. Provisions of EN 1998-1 defined two limit states, while provisions of EN 1998-3 defined three limit states, each with appropriate seismic hazard level. Return period for the seismic action for SD limit state (EN 1998-3) is equal with the ones for no collapse requirement (EN 1998-1). NC limit state given in EN 1998-3 is closer to the actual collapse of building, than the no collapse requirement given in EN 1998-1. To determine the seismic capacity of the structure, it is necessary to define and evaluate the ultimate displacements. In this paper has been adopted that the ultimate displacement corresponds to displacement when NC limit state of the vertical element is reached.

In this paper were analyzed RC frames, designed by EC 8 with different ductility classes and intensity of seismic actions. For all analyzed frames were determined pushover curves for two distributions of lateral loads, and based on them, target displacements and seismic capacity for two limit states.

Seismic capacity was approximately the same for linear and uniform lateral loads for corresponding frame. For frames designed for same intensity of seismic action, DCH frames had greater seismic capacity in comparison to DCM frames, for both limit states. Also, 0.2g frames had greater seismic capacity compared to 0.3g frames, which can be explained by the higher reserved strength of these frames.

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Vladimir Vitanov¹

SIMULACIJA FRP OJAČANJA AB ZIDOVA PRI CIKLIČNOM OPTEREĆENJU

Rezime:

U poslednje vreme nova tehnika za ojačanje AB konstrukcija je primena vlaknima ojačanih polimera (FRP) kao spoljašnjih ojačanja. Istraživanja u ovom polju se stalno potenciraju u pokušaju da se optimalno iskoriste neke od specifičnih svojstava ovih materijala. U radu je prikazan jedan takav pokušaj modeliranja materijala koji uspešno može simulirati ponašanje FRP-om ojačanih AB zidova izloženih cikličnom opterećenju kako bi se omogućio dalji razvoj u ovom polju. Rezultati inicijalnog ispitivanja predloženog modela, iako imaju neke nedostatke, ukazuju na to da se daljim razvojem može doći do modela koji će učiniti analizu takve konstrukcije lakšom i jednostavnijom, dodatno promovišući primenu tih materijala za ojačanje konstrukcija.

Кljučне речи: AB, FRP, ojačanje, MKE, numerički model, ANSYS

SIMULATION OF FRP STRENGTHENED RC WALL UNDER CYCLIC LOAD

Summary:

Fairly recent novel technique for strengthening of RC structures is the use of Fiber Reinforced Polymers (FRP) as externally bonded reinforcement. The research in this field continually intensifies in an attempt to optimally utilize some of the specific properties of these materials. The paper presents one such attempt to formulate a material model that can successfully simulate the behavior of FRP strengthened RC walls subjected to cyclic loading with a motivation to facilitate the further developments in this area. The results of the initial testing of the proposed model, while experiencing some drawbacks, indicate that its further development can lead to material model which can potentially make the analysis of such structures easier and simpler further promoting the use of these materials in structural strengthening.

Key words: RC, FRP, strengthening, FEM, numerical model, ANSYS

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1 INTRODUCTION

Recently, state-of-the-art strengthening and retrofit techniques increasingly utilize externally bonded fiber reinforced polymer (FRP) composites, which offer unique properties in terms of strength, lightness, chemical resistance, and ease of application. Such techniques are most attractive for their fast execution and low labor costs.

Only recently have researchers attempted to simulate the behavior of reinforced concrete strengthened with FRP composites using the finite element method. The majority of the studies that included numerical modeling of FRP strengthened RC members with FEM use element overlaying, where solid or layered that represent the FRP material are superimposed over the concrete elements, either with or without interface elements that represent the influence of the adhesive material or the bond between the FRP and the concrete.

The approach to numerical modeling of FRP strengthened RC concrete presented below adopts a different strategy in which instead of using separate finite elements for each of the constitutive material, a composite material model which incorporates the behavior of its constituents is firstly formulated and then employed as a material property of the finite elements in the finite element analysis. This approach could reduce the number of necessary finite elements, potentially reducing the complexity and the time needed for numerical analysis of RC members strengthened with FRP.

2 MODEL DESCRIPTION

The proposed model [1] for simulation of cyclically loaded reinforced concrete strengthened with FRP expands on the approach of the widely used inelastic model for cyclic biaxial loading of RC concrete formulated by Darwin and Pecknold in 1974 [2]. The RC model of Darwin and Pecknold uses the “equivalent uniaxial stress” approach with compressive loading curve proposed by Saenz [3] to model the biaxial material loading state, and the concrete failure surface proposed by Kupfer and Gerstle [4] based on the experimental data of Kupfer et al.[5]. Although comparably simple, this model is still capable of simulating the cycling behavior of reinforced concrete members in plane stress state. Because of its simplicity and computational efficiency it was selected as a basis for the new model proposed in this study. The proposed model further extends the RC model by introducing additional material, the FRP, as an additional third component of the composite constitutive matrix which becomes:

$$D' = D'_C + \sum_{i=1}^n D'_{S,i} + \sum_{i=1}^m D'_{F,i} \quad (1)$$

where:

D' is the constitutive matrix of the composite material in global coordinates,

D'_C is the constitutive matrix of the concrete in global coordinates,

$D'_{S,i}$ is the constitutive matrix of the steel in global coordinates,

$D'_{F,i}$ is the constitutive matrix of the FRP in global coordinates,

n is the number of different steel reinforcement,

m is the number of different FRPs.

These matrices in global coordinates are obtained by rotating their local coordinate representation using:

$$D' = T^T D T \quad (2)$$

where:

T is the transformation matrix,

$$T = \begin{bmatrix} \cos^2 \theta & \sin^2 \theta & \sin \theta \cos \theta \\ \sin^2 \theta & \cos^2 \theta & -\sin \theta \cos \theta \\ -2\sin \theta \cos \theta & 2\sin \theta \cos \theta & \cos^2 \theta - \sin^2 \theta \end{bmatrix} \quad (3)$$

where:

θ is the rotation angle between the two coordinate systems.

The concrete is considered as incrementally linear elastic material (it is assumed that it behaves elastically during each load increment). It is also considered to be isotropic before, and orthotropic after a crack occurs at a point, exhibiting different properties in two orthogonal directions, described with two different elasticity parameter sets in the two directions. Its constitutive matrix in material coordinates is [2]:

$$D_C = \frac{1}{1-\nu^2} \begin{bmatrix} E_1 & \nu\sqrt{E_1E_2} & 0 \\ \nu\sqrt{E_1E_2} & E_2 & 0 \\ 0 & 0 & (1-\nu^2)G \end{bmatrix} \quad (4)$$

where:

E_1, E_2 are the elasticity moduli at the two orthogonal directions

$\nu = \nu_1 \cdot \nu_2$ is the equivalent Poisson's ratio

$G = \frac{1}{4(1-\nu^2)}(E_1 + E_2 - 2\nu\sqrt{E_1E_2})$ is the shear modulus

If a crack occurs in direction 1 it is simulated by reducing the elasticity modulus in that direction to 0. This means that cracks in the concrete are not considered as distinct material discontinuities, but rather as an occurrence of many small cracks in the vicinity of the point considered, which is known as a "smeared" crack approach.

Much simpler material models are adopted for the simulation of the behavior of the other two constitutive materials. Bilinear elastic material model is adopted for the simulation of the reinforcing steel, whose constitutive matrix in local coordinates is:

$$D_S = p_S \begin{bmatrix} E_{Steel} & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix} \quad (5)$$

where:

p_S is the reinforcing ratio

E_{Steel} is the steel elasticity modulus, which is reduced by the strain hardening stiffness ratio δ when the stress level in the steel exceeds its yield strength

Linear elastic material model with a brittle failure point is adopted for the simulation of the FRP, with constitutive matrix in local coordinates:

$$D_F = p_F \begin{bmatrix} E_F & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix} \quad (6)$$

where:

p_F is the "strengthening" ratio (the ratio between the cross-section areas of the FRP and the concrete)

E_F is the FRP elasticity modulus

This adopted approach in the modeling of the steel and the FRP is known as “smeared” approach which means that the materials are considered as uniformly distributed, or smeared, throughout the element. This modeling approach implies that perfect bond between the constitutive materials exists.

This modeling approach is compatible with and suitable for the finite element method for structural multi-step analysis. The load on the structure can be divided in several steps and gradually applied. In each load step the constitutive matrix of Eq.1 can be updated by assembling the constitutive matrices of each of the materials (Eq.4-6) after they are all rotated to global coordinates using the Eq.2. This procedure was coded and implemented into the FEA software ANSYS [6].

3 VERIFICATION EXAMPLE

In order to verify the mathematical model, it was implemented as a custom material model into the commercial FEA software ANSYS, and used in the simulation of the behavior of cyclically loaded RC wall strengthened with FRP after which the results from the experimental investigation [7] and the numerical simulation were compared. The results of the numerical model were also compared with the results of another numerical simulation [8] of the same experiment using somewhat different, more conventional, approach in the modeling of such problems.

3.1 MODEL SETUP

Lombard conducted a testing on reinforced concrete shear wall specimens (Fig. 1). The walls were constructed using 40 MPa concrete with identical reinforcement of 400 MPa, 10 mm reinforcing bars. The height of the walls from the base of the panel to the center of the cap beam is 2 m, the length is 1.5 m and the thickness is 10 cm. The vertical reinforcement consist of five pairs of 10 mm bars, spaced at 40 cm for a reinforcement ratio of 0.8%. The horizontal steel consisted of five pairs of 10 mm bars, spaced at 40 cm for a reinforcement ratio of 0.5%. Three of the test specimens included a control wall and two strengthened walls. The control wall was tested in its original state which provided a baseline for the evaluation of the repair and strengthening techniques. The two strengthened shear walls were strengthened by applying 0.11 mm carbon fiber sheets to the walls without pre-damage. The carbon fiber sheets had an elastic tensile modulus of 230 GPa and failure strain of 1.5%. The first specimen was strengthened with one vertical layer of FRP externally bonded to each face of the wall (Wall 1). The second specimen had one horizontal and two vertical FRP layers on each face of the wall (Wall 2). Both specimens were not loaded until the strengthening was applied.

This paper presents the results of the analysis of the specimen Wall 1, i.e. the RC wall strengthened with a single layer of FRP with fibers in vertical direction on each face of the wall. For the FEM model in this case triangular as well as quadrilateral meshes were tested. The preliminary analyses showed using triangular mesh generally led to better solutions Fig. 2. A mesh of triangular, 6-node Plane183 elements with average size of 25 cm was used for the final results.

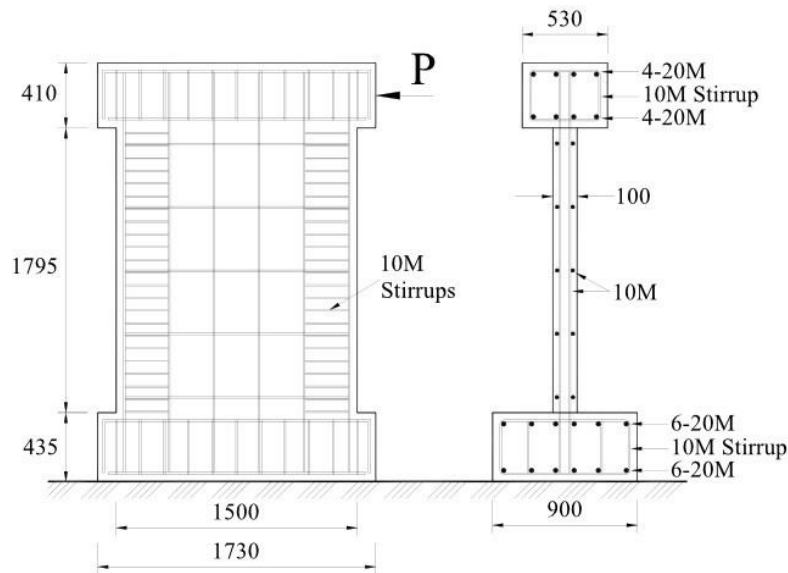


Figure 1 – Measures and Reinforcement Details of the RC Wall

Five different sections of the wall with different properties were defined: top and bottom beam, two side section ('columns') and a middle section ('panel'). Since the top and the bottom beam are significantly stiffer than the wall and their actual purpose is to provide the load transfer and anchorage for the tested wall, they were modelled as linear-elastic with very high elasticity modulus. The confining effect of the stirrups in the 'columns' was approximately accounted for by slightly increasing the concrete compressive strength in those regions, taking it to be 46 MPa in the 'columns', and 40 MPa in the 'panel'. The other concrete parameters were taken as: tensile strength of 4 MPa, initial elasticity modulus of 35 GPa, equivalent uniaxial strain of 0.35% and equivalent Poisson's ratio of 0.2. The steel material parameters were taken as: yield strength of 400 MPa, elasticity modulus of 200 GPa and strain hardening stiffness ratio of 1.8. The reinforcement ratio in vertical direction is 0.8%, and in horizontal direction 3% (in the 'columns') and 0.5% (in the 'panel'). The FRP material parameters were taken as: elasticity modulus of 230 GPa, ultimate strain at failure 1.5% and "strengthening" ratio of 0.22 for both the 'columns' and the 'panel'.

The cyclic load was applied at the middle of the top beam as a series of small displacements. The force and displacement at the same point were taken as results of the performed analyses. These were compared not only with the experimental data, but also from a numerical investigation performed by Khomwan and Foster [8]. They took a more conventional approach into modelling the same shear walls. Their models were created using 4-node concrete membrane elements with the main steel reinforcement modelled as 1D bar overlay elements. The stirrups in the 'columns' were modelled as smeared through concrete elements. The FRP elements were overlaid over the concrete elements. The connection between them was established via 2D interface elements. The top and bottom beam were also modelled as linear-elastic.

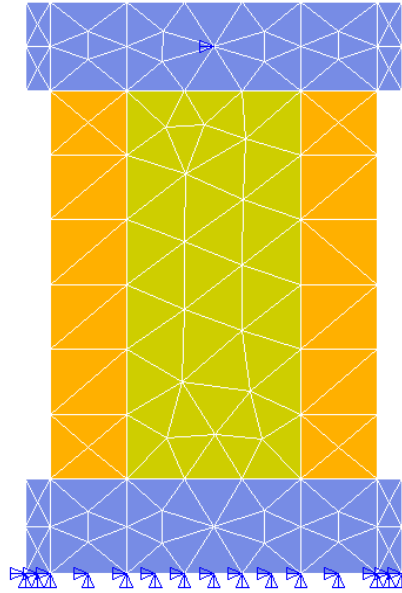


Figure 2 – Triangular Element Mesh of the FEM Model

3.2 RESULTS

The resulting load-deflection curves are shown in Fig. 3. Individual hysteretic loops developed at greater deflections where the models show distinct inelastic behavior are also shown in separately (Figs. 4). To measure how the numerical results compare to the experimental data the energy dissipated at each cycle (which corresponds to the area of the hysteretic loop) was calculated. The calculated energy dissipation is given in Table 1. The results indicate quite good correspondence with the experimentally acquired data.

It should also be noted that although for the cyclic loading analyses yielded good results, the solution showed significant sensitivity on the input parameters (element type and size, load step sizes, material data). Non-convergent load-step solutions frequently occurred leading to premature failure of the model. To obtain good and stable solution the model needed to be calibrated by performing several parametric analyses which would yield the most appropriate set of input parameters. As the final results show, once stable solution is reached, the simulation show satisfactory correspondence to the test results.

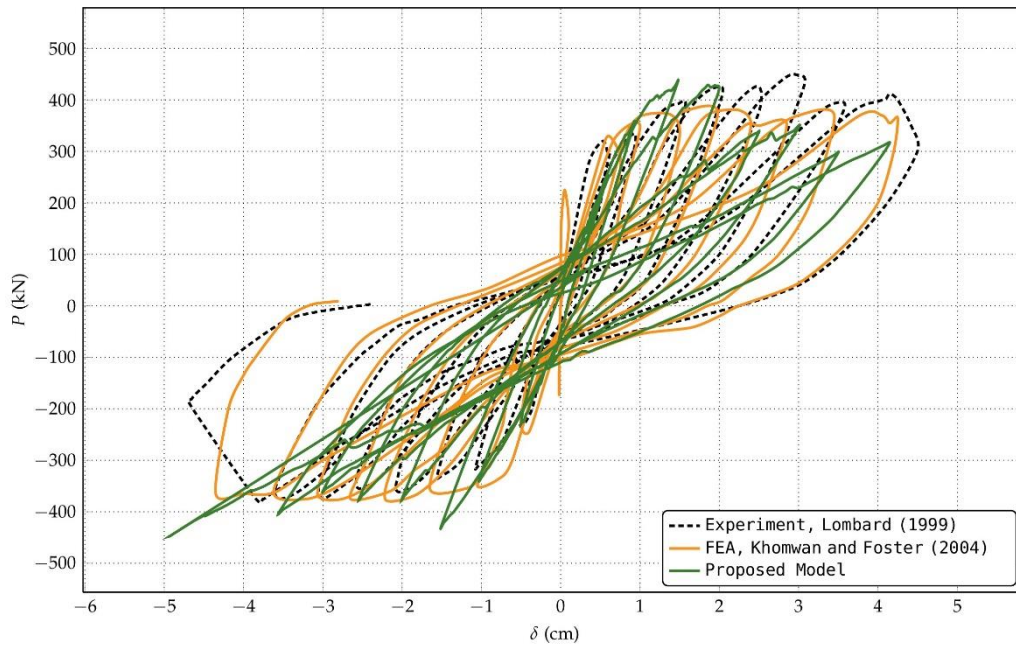


Figure 3 – Load-Deflection Curves for Test Wall 1 (RC wall strengthened with single vertical layer of FRP externally bonded to each face of the wall)

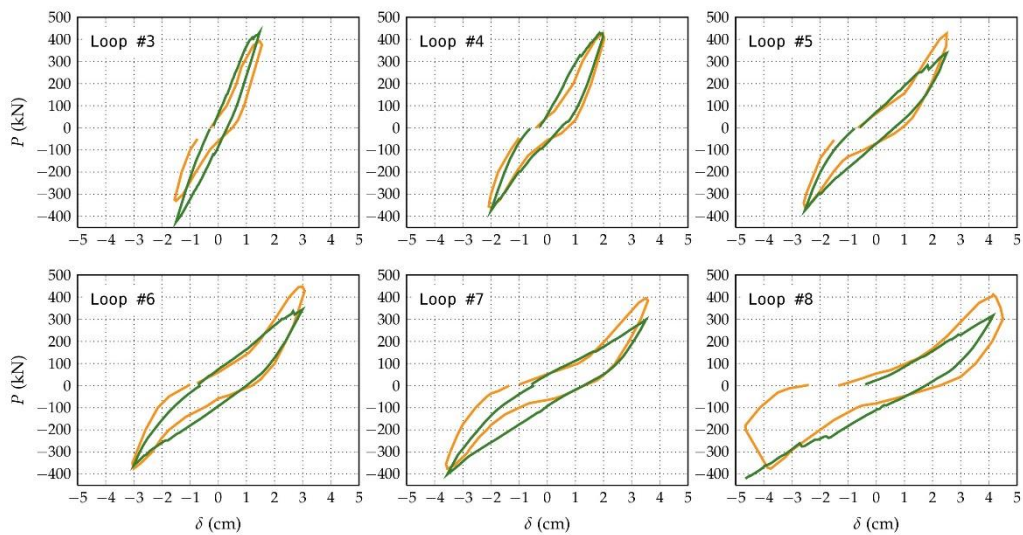


Figure 4 – Load-Deflection Curves for Test Wall 1- cycles #3 to #8, Orange Line – Experimental Results, Green Line – Numerical Results

Table 1 – Comparison of Energy Dissipation per Cycle

Loop #	Experimental	FEM	Ratio
3	62.225	49.481	0.80
4	141.225	144.294	1.02
5	226.810	196.496	0.87
6	460.300	351.148	0.76
7	696.088	579.795	0.83
8	866.070	816.154	0.94

4 CONCLUSION

The results from the performed analysis show that the presented material model can successfully simulate the behavior of cyclically loaded RC wall strengthened with FRP. The results shown in Table 2 indicate that the numerical model underestimates the energy dissipation for about 13% on average compared to the experimental results. Considering the highly inelastic nature of the simulated processes, this can be considered as a good result. The model also predicts the ultimate forces and displacement in each cycle quite favorably (Fig.4). It must also be pointed out that during the extensive testing of the proposed model, some drawbacks could be identified. Namely, the model showed significant sensitivity to the values of the input parameters rendering, and the simulation times were very high. This drawbacks render the model not suitable for practical use until those issues are solved. However, the presented results seem to indicate that the adopted approach in simulation of RC strengthened with FRP is promising and could lead to simpler and faster modeling which can facilitate the design and research activities in this area.

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OSVRT NA DVE NOVE GRUPE KARATA SEIZMIČKOG HAZARDA KOJE SU PREDLOŽENE ZA SRBIJU SAGLASNO EVROKODU 8

Rezime:

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U ovom radu diskutovane su dve nezavisne grupe karata seizmičkog hazarda za teritoriju Srbije koje su nedavno napravljene za potrebe projektovanja u skladu sa Evrokodom 8, i u kojima je hazard izražen preko vrednosti maksimalnog ubrzanja tla. Prva grupa karata je predložena od strane Republičkog seizmološkog zavoda, dok je druga grupa karata urađena korišćenjem UHS metode, tj. istovremenim uzimanjem u obzir i verovatnoće pojave zemljotresa, i prostorne distribucije zemljotresnih žarišta, i regionalnih karakteristika frekventno-zavisne atenuacije amplituda jakog kretanja, kao i karakteristika duboke geologije i lokalnog tla.

Ključne reči: karte seizmičkog hazarda, Srbija, UHS metoda

REVIEW OF THE TWO NEW GROUPS OF SEISMIC HAZARD MAPS THAT ARE PROPOSED FOR SERBIA IN COMPLIANCE WITH EC8

Summary:

In this paper we discuss two independent groups of seismic hazard maps for the territory of Serbia, which have recently been created for the needs of structural design according to Eurocode 8, and in which the hazard is expressed by the values of peak ground acceleration. The first group has been proposed by the Seismological Survey of Serbia, while the second group of maps has been created by using the UHS method, that is, by simultaneously taking into account the probability of earthquake occurrence, spatial distribution of earthquake sources, regional features of the frequently-dependent attenuation of strong motion, as well as characteristics of the deep geology and local soil.

Key words: seismic hazard maps, Serbia, UHS method

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1 UVOD

Propisi za građenje u seizmičkim područjima u Republici Srbiji, koji su i dan danas još uvek u upotrebi u Republici Srbiji, nastali su 1981. godine, sa poslednjom izmenom iz 1990. godine (*Pravilnik o tehničkim normativima za izgradnju objekata visokogradnje u seizmičkim područjima*, Sl. list SFRJ br. 31/81, 49/82, 29/83, 21/88, 52/90 [1]). Poslednjom izmenom u ovim propisima, 1990. godine (Sl. list SFRJ 52/90 [1]), uvedeno je u upotrebu 6 seizmoloških karata, koje su na bazi probabilističke analize hazarda sračunate za povratne periode od 50, 100, 200, 500, 1000 i 10,000 godina, pri čemu je u Članu 6 [1] mapa za povratni period od 500 godina usvojena za projektovanje konstrukcija II i III kategorije (stambene zgrade, hoteli, restorani, pomoćno-proizvodne zgrade, javne i industrijske zgrade koje nisu svrstane u kategoriju I).

Tokom 1990-ih, u Srbiji je otpočeo proces harmonizacije domaćih propisa za projektovanje i građenje sa Evrokodovima, koji još uvek traje. Kako Evrokod 8 [2], koji se odnosi na aseizmičko projektovanje i građenje konstrukcija, preporučuje da se u najjednostavnijem vidu seizmički hazard izrazi preko vrednosti maksimalnog ubrzanja tla, pojavila se potreba za novom kartom seizmičkog hazarda za Srbiju. U ovom radu će biti diskutovane dve nezavisne grupe karata seizmičkog hazarda za teritoriju Srbije koje su nedavno napravljene za potrebe projektovanja u skladu sa Evrokodom 8.

2 KARTE PREDLOŽENE OD STRANE RSZ-A

Seizmološki zavod Srbije je sredinom 2013. godine na svojoj zvaničnoj web prezentaciji (www.seismo.gov.rs/Seizmicnost/Karte_hazarda.html) postavio karte seizmičkog hazarda za teritoriju Srbije za povratne periode $T_r = 95, 475$ i 975 godina, odnosno, redom, za verovatnoće prevazilaženja od $p = 10\%$ u 10 godina, 10% u 50 godina i 5% u 50 godina. U tim kartama hazard je prikazan putem vrednosti maksimalnog ubrzanja na tlu tipa A (stena tj. tlo sa prosečnom brzinom prostiranja smičućih talasa u površinskih 30 metara na lokaciji većom od ili jednakom 800m/s), izraženih u delovima gravitacionog ubrzanja g .

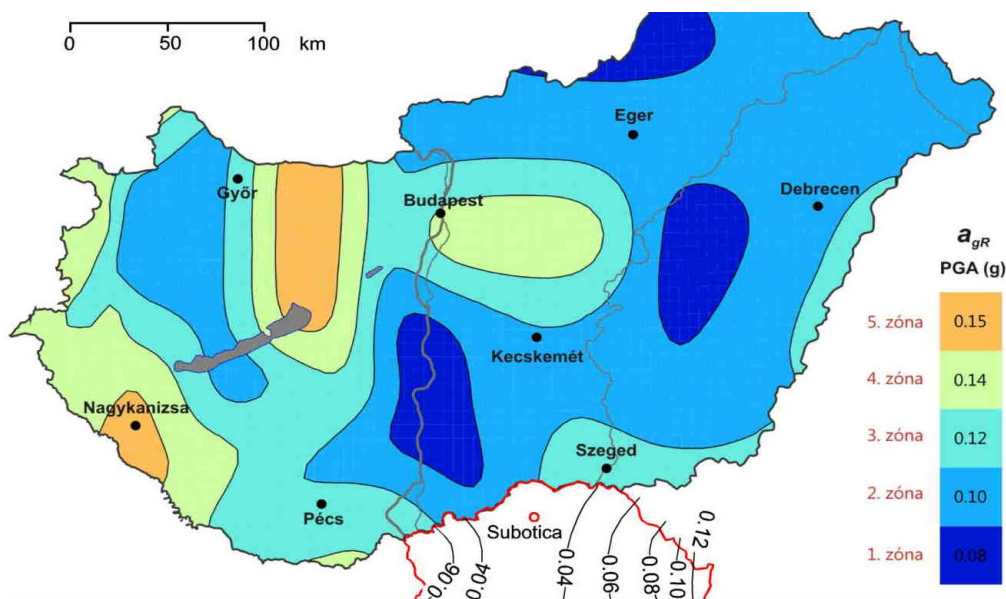
Imajući u vidu da krajnji rezultati u proračunu seizmičkog hazarda zavise od tri osnovna elementa: 1) modela žarišta potencijalnih budućih zemljotresa (tj. konfiguracije seizmički aktivnih zona), 2) statističkog opisa seizmičnosti u tim žarištima (izvršenog na bazi istorijskih podataka o dogođenim zemljotresima), i 3) atenuacione jednačine relevantne za razmatran parametar hazarda, bilo je potrebno skoro 3 godine različitih vidova istraživanja kojim bi došli do objektivnih saznanja o tome da li su karte hazarda koje su predložene za Srbiju od strane RSZ-a pouzdane ili ne [3-4].

Na osnovu detaljnog pregleda podataka o zemljotresima datim u Katalogu zemljotresa RSZ-a, i njihovog poređenja sa postojećim podacima o zemljotresima dogođenim na teritoriji Srbije, i koji su dati u relevantnim katalozima susednih zemalja i regionalnim katalozima, utvrdili smo da u pomenutom katalogu RSZ-a nema podataka za oko 80 zemljotresa koji su se dogodili u prošlosti na teritoriji Srbije i čije su magnitude bile u opsegu od 3.5 do 5.8 [3-4]. Takođe smo utvrdili da u analizi seizmičkog hazarda za Srbiju nisu uključeni podaci za oko 100 najjačih zemljotresa sa $M_w \geq 5$ koji su se dogodili u susednim zemljama na prostoru koji je razmatran u analizi hazarda ($41^\circ\text{--}47^\circ\text{N}$ i $18^\circ\text{--}24^\circ\text{E}$) [3-4]. Neuzimanje tako velikog broja zemljotresnih podataka dovelo je do neadekvatne procene parametara ponovljivosti zemljotresa

u pojedinim žarišnim zonama na razmatranom prostoru, što je pak rezultiralo u neadekvatnom seizmotektonskom modelu koji je bio korišćen u analizi seizmičkog hazarda za Srbiju.

Takođe se pokazalo [3-4] da je primena neadekvatnih atenuacionih jednačina, izvedenih za druge seizmotektonske regione, dodatno uticala na netačnost/nepouzdanost odnosno veliku potcenjenost vrednosti maksimalnih ubrzanja tla prikazanih u kartama hazarda RSZ-a za povratne periode od 95, 475 i 975 godina, što potvrđuju i poređenja ovih karata sa odgovarajućim kartama koje su date za neke susedne zemlje, Severni Balkan, Panonski region i Evro-Mediteran [3-4].

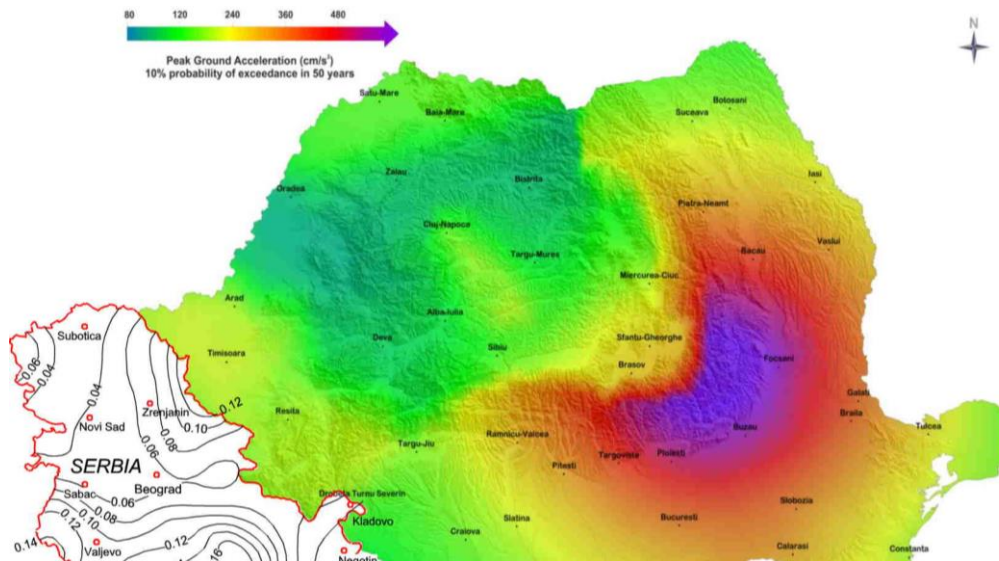
Na slikama 1-3 prikazujemo komparaciju vrednosti maksimalnih ubrzanja tla duž severne i istočne granice Srbije, datih na karti koju je predložio RSZ za povratni period od 475 godina, sa vrednostima maksimalnih ubrzanja koja su prikazana u kartama hazarda Mađarske [5], Rumunije [6] i Bugarske [7]. Na slici 1 upoređen je najpre severni deo karte seizmičkog hazarda koju je predložio RSZ za Srbiju za povratni period od 475 godina, sa kartom hazarda za Mađarsku ([5]; <http://www.foldrenges.hu/>). Na slici 2 upoređen je zatim severo-istočni deo karte seizmičkog hazarda koju je predložio RSZ za Srbiju za povratni period od 475 godina, sa kartom hazarda za Rumuniju [6]. Na slici 3 upoređen je na kraju i jugo-istočni deo karte seizmičkog hazarda koju je predložio RSZ za Srbiju za povratni period od 475 godina, sa kartom hazarda za Bugarsku [7].



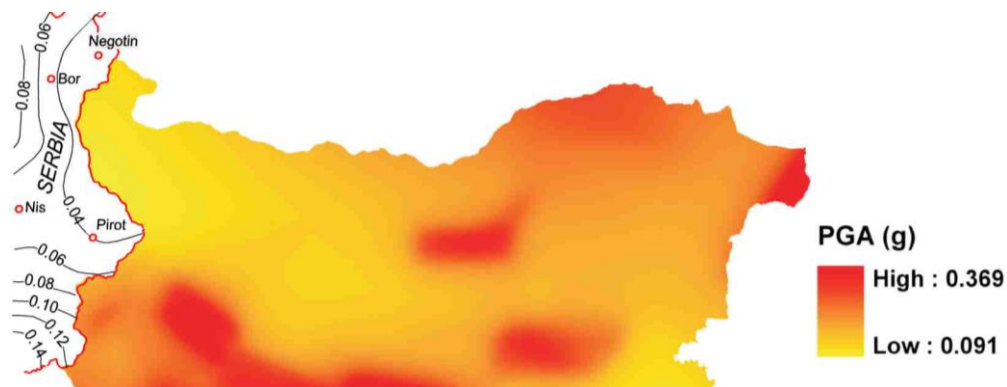
Slika 1. Poređenje vrednosti maksimalnih ubrzanja tla datih na karti seizmičkog hazarda koju je predložio RSZ za Srbiju za povratni period od 475 godina, sa vrednostima maksimalnih ubrzanja koja su prikazana u karti hazarda za Mađarsku ([5]; <http://www.foldrenges.hu/>).

Kao što se može videti sa slika 1-3, vrednosti maksimalnih horizontalnih ubrzanja tla koje su prikazane u kartama hazarda predloženim za Srbiju od strane RSZ-a su mnogo manje (od 2 pa do čak i 5 puta za neke delove) od vrednosti prikazanih u kartama za susedne zemlje duž severne i istočne granice Srbije. Ovako velike razlike u vrednostima maksimalnog ubrzanja u

karti hazarda predloženoj za Srbiju od strane RSZ-a i kartama hazarda za Mađarsku, Rumuniju i Bugarsku su s jedne strane posledica ili neadekvatnog definisanja parametara seizmičnosti za seizmičke zone u Mađarskoj, Rumuniji i Bugarskoj koje imaju uticaj na hazard u Srbiji ili pak kompletnog zanemarivanja uticaja ovih zona u analizi seizmičkog hazarda za Srbiju, a s druge strane i posledica korišćenja neadekvatnih atenuacionih jednačina u analizi hazarda za Srbiju.



Slika 2. Poređenje vrednosti maksimalnih ubrzanja tla datih na karti seizmičkog hazarda koju je predložio RSZ za Srbiju za povratni period od 475 godina, sa vrednostima maksimalnih ubrzanja koja su prikazana u karti hazarda za Rumuniju [6].



Slika 3. Poređenje vrednosti maksimalnih ubrzanja tla datih na karti seizmičkog hazarda koju je predložio RSZ za Srbiju za povratni period od 475 godina, sa vrednostima maksimalnih ubrzanja koja su prikazana u karti hazarda za Bugarsku [7].

3 KARTE IZVEDENE PRIMENOM UHS METODE

U ovom poglavlju diskutovana je nova seizmička rejonizacija Srbije, u kojoj su pored maksimalnih vrednosti ubrzanja tla sračunati kompletni spektri uniformnog hazarda – UHS spektri [8-9].

Od sredine 1970-ih, nakon što su objavljene prve neposredne empirijske jednačine za skaliranje spektralnih amplituda jakog kretanja tla, postalo je moguće formulirati seizmičku (makro-)rejonizaciju i mikrorejonizaciju pomoću pristupa koji uzimaju u obzir i verovatnoću pojave zemljotresa, i prostornu distribuciju seizmičkih žarišta, i regionalnu frekventno-zavisnu atenuaciju amplituda jakog kretanja, kao i karakteristike lokalne (duboke) geologije i lokalnog tla, čiji bi rezultat bili tzv. spektri uniformnog hazarda („uniform hazard spectra“, skraćeno kao UHS) [10]. Prednost ovog pristupa je da se istovremeno razmatra, i to na uravnotežen način, doprinos svih faktora koji utiču na krajnji rezultat.

Za sve proračune spektralnih amplituda u analizi hazarda za Srbiju korišćeni su podaci jakog kretanja registovani na teritoriji bivše SFR Jugoslavije [11], koji su obrađeni sa USC sistemom za automatsku digitalizaciju [12], i na osnovu kojih su od strane Lee-ja razvijene jednačine za skaliranje niza spektralnih amplituda [13]. Detaljan opis ovih jednačina može se naći u radovima: [14-15].

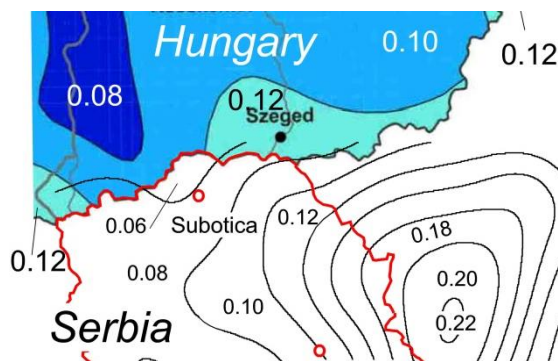
Važna karakteristika jednačina za skaliranje spektralnih amplituda korišćenih u ovom radu je da su one sa regionalnim obeležjem. Svi parametri skaliranja: magnitude ili intenziteti, duboka geologija i lokalni uslovi tla, i sve atenuacione jednačine, su svi bili formulisani za ovaj isti region za koji smo izvršili ocenu UHS spektralnih amplituda. Ovo je važna prednost pošto praktično eliminiše uvođenje grešaka u krajnji rezultat. Ove greške mogu biti prouzrokovane različitim regionalnim praksama u proračunu magnitude zemljotresa ili atenuacije amplituda jakog kretanja tla sa rastojanjem [16-18].

Osim toga, korišćene jednačine za skaliranje [13] uzimaju u obzir istovremeno i direktno i karakteristike duboke geologije i karakteristike lokalnog tla [19]. Jednačine za skaliranje koje koriste samo karakteristike lokalnog tla i ignorišu istovremene – simultane efekte geoloških uslova (duboke geologije) dovode do pogrešnih procena i ne treba da se koriste [20].

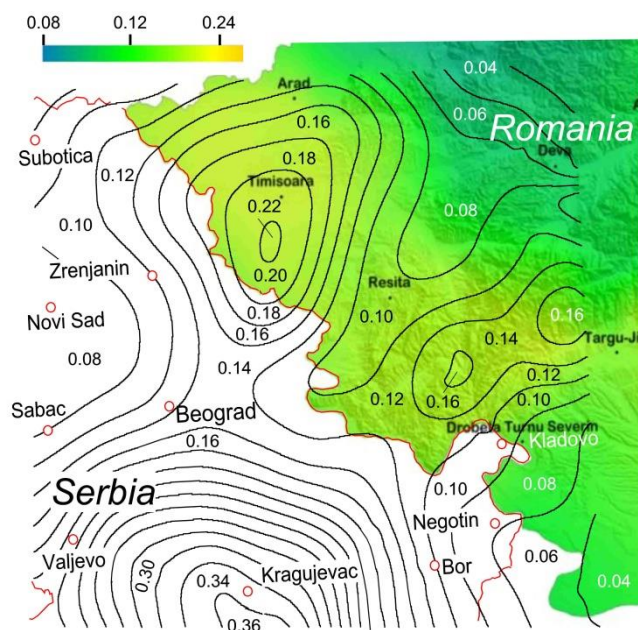
Primenom UHS metode hazard se izražava kroz amplitude UHS spektara pseudo relativne brzine, PSV. Množenjem tih amplituda sa $2\pi T$ dobiće se UHS amplitude pseudo apsolutnog ubrzanja („Pseudo Absolute Acceleration“ – PSA), dok će deljenje sa $2\pi T$ dati UHS amplitude relativnog pomeranja („Relative Displacement“ – SD). Kada period vibracija teži ka nuli, $T \rightarrow 0$, spektar akceleracije teži ka maksimalnom ubrzanju tla, tako da se karte hazarda lako mogu izraziti i u skladu sa sugestijama Evrokoda 8, odnosno putem vrednosti maksimalnog ubrzanja tla – videti slike 4-6.

Na slikama 4-6 prikazujemo komparaciju vrednosti maksimalnih ubrzanja tla duž severne i istočne granice Srbije, datih na karti izvedenoj primenom UHS metode [8] za povratni period od 475 godina, sa vrednostima maksimalnih ubrzanja tla koja su prikazana u kartama hazarda za Mađarsku [5], Rumuniju [6] i Bugarsku [7]. Na slici 4 upoređen je najpre severni deo karte seizmičkog hazarda izvedene primenom UHS metode za Srbiju za povratni period od 475 godina, sa kartom hazarda za Mađarsku ([5]; <http://www.foldrenge.hu/>). Na slici 5 upoređen je zatim severo-istočni deo karte seizmičkog hazarda izvedene primenom UHS metode za Srbiju za povratni period od 475 godina, sa kartom hazarda za Rumuniju [6]. Na slici 6 upoređen je na kraju i jugo-istočni deo karte seizmičkog hazarda izvedene primenom UHS metode za Srbiju za povratni period od 475 godina, sa kartom hazarda za Bugarsku [7].

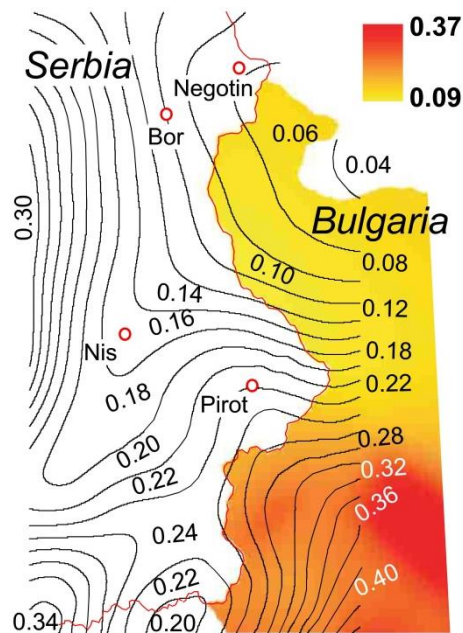
Kao što se može videti sa slika 4-6, vrednosti maksimalnih horizontalnih ubrzanja tla koje su prikazane u karti hazarda izvedenoj primenom UHS metode za Srbiju za povratni period od 475 godina su u dobroj saglasnosti sa vrednostima prikazanim u kartama za susedne zemlje duž granice sa Srbijom.



Slika 4. Poređenje vrednosti maksimalnih ubrzanja tla datih na karti seizmičkog hazarda izvedenoj primenom UHS metode za Srbiju za povratni period od 475 godina, sa vrednostima maksimalnih ubrzanja koja su prikazana u karti hazarda za Mađarsku ([5]; <http://www.foldrenge.hu/>).



Slika 5. Poređenje vrednosti maksimalnih ubrzanja tla datih na karti seizmičkog hazarda izvedenoj primenom UHS metode za Srbiju za povratni period od 475 godina, sa vrednostima maksimalnih ubrzanja koja su prikazana u karti hazarda za Rumuniju [6].



Slika 6. Poređenje vrednosti maksimalnih ubrzanja tla datih na karti seizmičkog hazarda izvedenoj primenom UHS metode za Srbiju za povratni period od 475 godina, sa vrednostima maksimalnih ubrzanja koja su prikazana u karti hazarda za Bugarsku [7].

Na ovom mestu jako je važno napomenuti i da su prve empirijske jednačine za skaliranje niza amplituda spektara pseudo-brzine koje se odnose na jako kretanje tla usled zemljotresa u bivšoj SFRJ, tj. koje su bile bazirane na regionalnim podacima jakog kretanja tla, nastale sredinom 1990-ih. Nedavno izvršena analiza atenuacija amplituda Fourier-ovih spektara za Srbiju usled umereno jakih i jakih rumunskih Vrančea zemljotresa [21], koji se događaju na epicentralnim rastojanjima od preko 300km u proseku 5 puta u jednom veku sa magnitudom većom od 7, dokazala je neophodnost uzimanja u obzir i ovih jakih Vrančea zemljotresa u analizi seizmičkog hazarda na teritoriji Srbije. Saznanja dobijena u toj analizi iskorišćena su za razvijanje novih empirijskih jednačina za skaliranje niza PSV amplituda u Srbiji [22], koje su takođe, pored ranije razvijenih regionalnih empirijskih atenuacionih jednačina [13], iskorišćene prilikom izrade karata seizmičkog hazarda za Srbiju primenom UHS metode kako bi se u proceni seizmičkog hazarda pored regionalnih umereno jakih i plitkih zemljotresa takođe uzeli u obzir i udaljeni a veoma jaki Vrančea zemljotresi. Iako se pokazuje da je u slučaju vrednosti maksimalnog ubrzanja tla uticaj Vrančea zemljotresa ograničen na istočne delove Srbije, i za ostale delove Srbije nije značajno izražen, u slučaju spektralnih amplituda na većim periodama oscilovanja koji odgovaraju sopstvenim vibracijama višespratnih zgrada, Vrančea zemljotresi dominantno utiču na jačinu kretanja tla ne samo na istočnim nego i na unutrašnjim delovima Republike Srbije. Kao ilustracija za ovo mogu da posluže primeri spektara uniformnog hazarda za Kraljevo, Beograd, i Negotin, prikazani u radu [9], u kojima se vidi da bi zanemarivanje uticaja Vrančea zemljotresa, dovelo čak i do duplo manjih spektralnih amplituda na periodama oscilovanja od preko 1.0 sekunde. Pri tome, za Beograd i Kraljevo, gde Vrančea zemljotresi

nemaju veliki uticaj na vrednosti maksimalnog ubrzanja tla, do isto tolikog potcenjivanja seizmičkog opterećenja bi dovelo i skaliranje normalizovanih spektara samo pomoću vrednosti maksimalnog ubrzanja tla, kao što to predlaže Evrokod 8 [2], umesto korišćenja spektara uniformnog hazarda.

4 DISKUSIJA I ZAKLJUČCI

U ovom radu analizirali smo dve nezavisne grupe karata seizmičkog hazarda za teritoriju Srbije koje su nedavno napravljene za potrebe projektovanja u skladu sa Evrokodom 8.

U prvoj grupi karata, predloženoj od strane Republičkog seizmološkog zavoda, seizmički hazard je izražen preko vrednosti maksimalnog ubrzanja tla, kojima bi zatim trebalo skalirati projektne spektre odgovora koje predlaže Evrokod 8 [2], a koji imaju normalizovan oblik i ne uzimaju u obzir ni razlike u rastojanju od žarišta zemljotresa niti kontinualne promene spektralnog sastava sa promenom jačine zemljotresa. U radovima [3-4], kao i u ovom radu, data je diskusija karata seizmičkog hazarda predloženih od strane RSZ-a koja ukazuje na to da ove karte iz čitavog niza razloga ne bi smelo da budu prihvaćene kao podloge za opredeljivanje vrednosti seizmičkog dejstva u projektovanju građevinskih konstrukcija u Srbiji.

Slike prikazane u ovom radu pokazuju da su vrednosti maksimalnih horizontalnih ubrzanja tla koje su prikazane u kartama seizmičkog hazarda predloženim za Srbiju od strane Republičkog seizmološkog zavoda mnogo manje (čak i do 5 puta za neke delove) od vrednosti prikazanih duž granične linije u kartama seizmičkog hazarda za Mađarsku, Rumuniju i Bugarsku. Ovako velike razlike su posledica neadekvatnog definisanja parametara seizmičnosti za seizmičke zone koje se nalaze u pomenutim susednim zemljama a koje imaju uticaj na hazard u Srbiji kao i korišćenja neadekvatnih atenuacionih jednačina u analizi hazarda za Srbiju [3-4].

Karte napravljene korišćenjem UHS metode za Srbiju su proračunate istovremenim uzimanjem u obzir i verovatnoće pojave zemljotresa, i prostorne distribucije zemljotresnih žarišta, i regionalnih karakteristika frekventno-zavisne atenuacije amplituda jakog kretanja, kao i karakteristika duboke geologije i lokalnog tla.

Ova druga grupa karata, izvedena korišćenjem UHS metode, daje vrednosti hazarda ne samo za maksimalna ubrzanja tla nego takođe i za čitav niz spektralnih amplituda, čime se omogućuje direktno definisanje spektara uniformnog hazarda (UHS spektri), kojima je moguće direktno uzeti u obzir i uticaj Vrančea zemljotresa na spektralne amplitude, pogotovo za veće periode oscilovanja za koje su nedavne analize pokazale da udaljeni a veoma jaki i relativno česti Vrančea zemljotresi dominantno utiču na skoro celoj teritoriji Republike Srbije [9].

Vrednosti maksimalnih horizontalnih ubrzanja tla koje su prikazane u kartama hazarda izvedene primenom UHS metode za Srbiju su takođe i u dobroj saglasnosti sa vrednostima maksimalnog ubrzanja tla prikazanim u kartama seizmičkog hazarda za susedne zemlje duž severne i istočne granice Srbije.

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Nenad Simić¹, Dražen Balažević², Marta Ostrogonac³

PROJEKTOVANJE I IZVOĐENJE OBJEKTA TIGAR TYRES PIROT

Rezime:

S-50 Koncept realizacije objekata u sistemu *Design and Built* je sve prisutniji i na našem tržištu. Za takav aranžman izgradnje proizvodnog kompleksa se odlučio i jedan od najvećih proizvođača u Srbiji, fabrika Tigar Tyres u Pirotu, deo grupacije Michelin. U datom momentu, to je bila jedna od najvećih investicija na teritoriji republike Srbije. Posao je dobio konzorcijum „Termoinženjering-Deneza-North Engineering“ gde je North Engineering bio angažovan na izradi projektne dokumentacije. Sistem realizacije *Design and Built* zahtevao je aktivno učešće projektanta od samog početka procesa izgradnje pa do primopredaje objekta.

Ključne reči: Design and Built, Tigar Tyres, proračun i analize konstrukcije

DESIGN AND BUILT OF TIGAR TYRES PIROT CONSTRUCTURE

Summary:

Design and Built concept for Project realisation is nowadays becoming more present even on Serbian market. That kind of arrangement was also the first choice of Tigar Tyres as part of Michelin group, in realisation of the production plant inside their complex in Pirot. In that moment that was the one of the biggest investment in Serbia. Project was appointed to consortium „Termoinženjering-Deneza-North Engineering“ inside which North Engineering's scope was elaboration of design documentation. Whole arrangement required full engagement of the designer from the very beginning until the handover of the building.

Key words: Design and Built, Tigar Tyres, Structure analysis

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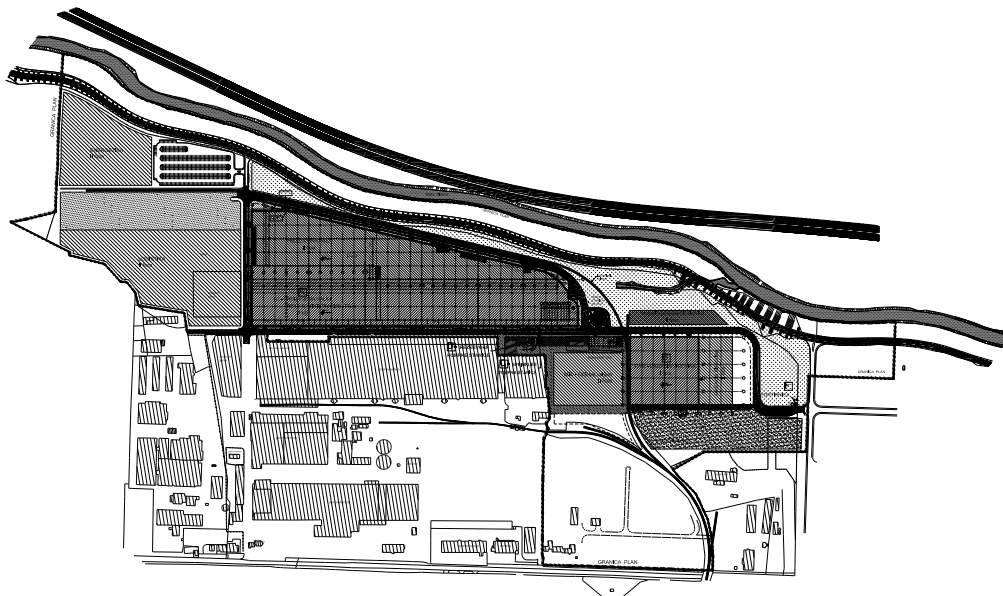
³dipl.ing.arh., NORTH Engineering d.o.o. Subotica

1 UVOD

Industrijski objekti u Srbiji zauzimaju veoma značajno mesto u građevini, obzirom na njihov broj kao i samu vrednost investicija. Otvaranjem tržišta, u Srbiju su došle mnoge inostrane kompanije koje su ovde instalirale svoju proizvodnju ako je reč o industrijskim objektima, odnosno prodaju i logistiku ako je reč o trgovinskim lancima.

U martu 2013. godine, u Srbiji je raspisan tender za izbor izvođača za izgradnju jedne od najvećih fabrika ZA PROIZVODNJU AUTOMOBILSKIH GUMA I PROIZVODA OD GUME , „TIGAR TYRES“ AD, koji posluje u sastavu grupacije „MICHELIN“, u Pirotu. Kako je ugovorom određeno, objekat je izvođen po sistemu Design And Built pa je „NORTH Engineering“ iz Subotice bio deo konzorcijuma koji je zadužen za izgradnju i projektovanje a koji su činili „TERMOINŽENJERING“ iz Beograda i „DENEZA“, takođe iz Beograda.

U sklopu postojećeg industrijskog, građevinskog kompleksa TIGAR TYRES u Pirotu. izgrađen je proizvodni pogon za proizvodnju gumenih pneumatika za automobile, aneks administracije sa svim neophodnim pratećim objektima u kojima funkcioniše proizvodnja, infrastrukturna podrška i skladištenje sirovina i proizvoda. Projektovanje je obuhvatalo dve faze objekta dok je, za sada, izgrađena samo prva faza.



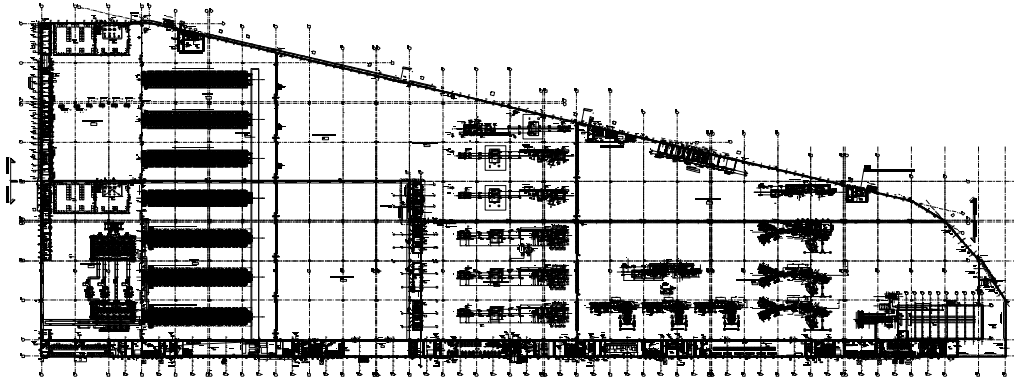
Slika 1 - Situacioni plan

Polazna osnova za projektovanje je bilo idejno rešenje tačnije koncept i projektni zadatak koje je za potrebe investitora izradila konsultantska firma „TEBODIN“ a na osnovu kog je izrađena tenderska dokumentacija prema kojoj je data ponuda za izvođenje, sistemom Design and Built. Rok za izradu tenderske dokumentacije za davanje ponude bio je više nego kratak, nepune dve nedelje. U tom vremenskom roku bilo je neophodno sagledati sve elemente konstrukcije, instalacija, naročito mašinskih, koje su do pred sam završetak objekta pretrpele mnogobrojne izmene i dopune, prema zahtevima tehnologije i investitora.

Objekti kompleksa su izvedeni za 17 meseci.

NETO POVRŠINE OBJEKATA U PRVOJ FAZI IZGRADNJE:

Magacin sirovina	14.193	m ²
Proizvodna hala sa aneksima 1 i 2	56.869	m ²
Četvorovaljak	1.063	m ²
Rashladne kule	715	m ²
Sprinkler pumpna stanica	81	m ²
Platoi za odlaganje got. Proizvoda	17.000	m ²
	Ukupno :	89.921 m ²
Saobraćajnice	68.581	m ²



Slika 2 - Osnova objekta I+II faza

U prvoj fazi izvođenja, proizvodna hala sa aneksima je u osnovi pravougaonog oblika, dimenzija 577 x 107m i visinom u najvišoj tački slemena od 11.45m. Bruto površina objekta u osnovi je oko 52.000m² odnosno 59.000m² koliko je površina svih etaža i u potpunosti je u funkciji proizvodnog procesa koji se odvija unutar i oko objekta. Ukupna površina objekta za obe faze iznosiće 93.300m². Objekat je zbog dimenzija gabarita podeljen u 8 dilatacionih celina koje su iznad kote temelja konstruktivno potpuno nezavisne. Spratnost objekta je prizemna P, u delu proizvodnje odnosno P+1 u aneksnom delu. Krov je ravan, sa nagibom krovne ravni od 3%.

Pored proizvodnog pogona i aneksa 1 uz objekat odnosno aneksa 2 unutar samog proizvodnog objekta, konstrukcija za četvorovaljak unutar objekta, kompleks novoizgrađenih objekata sačinjavaju još i magacin sirovina, objekat za pripremu vode sa sistemom za hlađenje i rashladnim kulama, sprinkler pumpna stanica sa rezervoarom i crpnom stanicom, trafostanica ts-15 kao i platoi za odlaganje gotovih proizvoda.

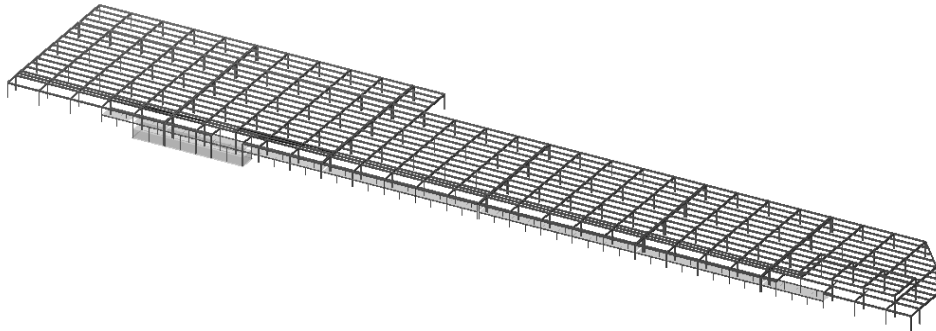
2 PROIZVODNA HALA SA ANEKSOM PROIZVODNJE

Glavnu noseću konstrukciju objekta formira skeletni sistem montažne konstrukcije od armiranog betona raspona 24.0x20.0m. Montažna konstrukcija se sastoji od konzolnih stubova na koje se montiraju glavni nosači preko kojih se u ortogonalnom pravcu montiraju sekundarni krovni nosači — rožnjače. Krovna obloga je slagani krovni pokrivač koji se sastoji od trapez lima, sloja termoizolacije i krovne PVC membrane. Fasadna obloga je izvedena od sendvič panela. Obzirom na zahtev da paneli budu postavljeni vertikalno, bila je neophodna čelična potkonstrukcija na polovini visine panela. Umesto standardnog načina izvođenja fasadne potkonstrukcije sistemom međustubova i horizontalnih nosača, projektovana je i izvedena fasadna tropojasna rešetka raspona 20m odnosno 24m.

Aneksni deo objekta koji se pruža paralelno uz objekat, je spratnosti P+1. Konstrukciju sprata čine podvlake, u podužnom pravcu, oslonjene delom na međustubove a delom na stubove osnovne konstrukcije preko kojih se postavljaju ošupljene ploče raspona 10m koje se monolitizuju nakon montaže.

Aneksni deo objekta unutar proizvodne hale je formiran uz osu 2, u širini od cca 9m i dužine 67m uz osu 2. Spratnost na delu aneksa je P+1. Konstrukciju sprata čine podvlake u dužem pravcu i šuplje ploče raspona 7.5m sa konzolnim prepustom dužine 1.5m koji se izvodi zajedno sa slojem monolitizacije preko ošupljenih ploča.





Slika 3 - Prostorni model montažne konstrukcije

2.1 MONTAŽNA KONSTRUKCIJA

2.1.1 Čašice

Armiranobetonske čašice za montažne stubove se razlikuju u zavisnosti od dimenzija stubova koje prihvataju. Čašice za stubove 70x70 su dimenzija 140x140x105cm sa debljinom zida 25cm a za stubove 50x50cm su dimenzija 120x120x105 sa debljinom zida od 25cm. Unutrašnje površine svih čašica su izvedene sa orebrenjem. Iz temeljne čašice vertikalna armatura je usidrena u monolitni deo konstrukcije temelja samaca.

2.1.2 Stubovi

Armirano betonski stubovi su poprečnog preseka 70x70cm, 50x50cm i 40x40cm su konzolnog statičkog sistema sa uklještenjem u konstrukciju temelja samaca. Stubovi imaju viljuškasto oformljenu glavu za oslanjanje glavnih I-170 nosača, na rasteru od 24.0x20.0m. Na delu gde je predviđena međuspratna šuplja ploča postavljaju se kratki međustubovi dimenzija 40x40cm koje služe za prihvatanje međuspratnih greda. Na delu objekta sa SPF bazenom, stubovi se oslanjaju na zidove bazena. Montažni nastavak stubova je izveden montažnim Peikko nastavcima ugrađenim u zidove bazena.

2.1.3 Glavni nosači

Glavni krovni nosači su adheziono prednapregnuti nosači raspona 24.0m na međusobnom razmaku od cca 20.00m. Poprečni presek nosača u sredini polja je I-170. Glavni nosači se oslanjaju u viljušku stuba preko neoprenskog ležišta. Montažna veza sa stubom se ostvaruje preko dva trna odgovarajuće dužine od armature na oba kraja nosača i trnova koji se ostavljaju iz viljuške stuba.

Krovni nosač aneksa je klasično armirani nosač raspona 10m na međusobnom razmaku od 20m. Poprečni presek nosača je T-140. Glavni nosači se oslanjaju u viljušku stuba preko neoprenskog ležišta dimenzija. Montažna veza sa stubom se ostvaruje preko trnova.

2.1.4 Rožnjače

Sekundarni krovni nosači - rožnjače su adheziono prednapregnuti nosači raspona cca 20.00m koji se oslanjaju na glavne nosače. Poprečni presek rožnjača je T-140. Na krajevima rožnjača T-140 preseka "prelazi" u kratki element pravougaonog poprečnog preseka. Rožnjače se oslanjaju na glavne nosače preko neoprenskog ležišta dimenzija . Montažna veza sa glavnim nosačem se ostvaruje preko dva trna

2.1.5 Podvlake

Na delu ispod galerije aneksa 1, na koti +4.35 u osama A i B su klasično armirani nosači nosači raspona 10.0m na međusobnom razmaku od 10.0m. Poprečni presek nosača je L100. Podvlake se oslanjaju na kratke elemente stubova preko neoprenskog ležišta. Montažna veza sa stubom se ostvaruje preko trnova

2.1.5 Ošupljene ploče

Šuplje ploče su izvedene za raspon od 10m i za maksimalno ukupno opterećenje od oko 8.00kN/m². Debljina ploča je 32cm tip PSP 4/32 od betona kvaliteta MB 60 (C50/60). Šuplje ploče se preko neoprenskih traka debljine 5mm i širine 50mm oslanjaju na međuspratne grede. Nakon postavljanja šupljih ploča izvršena je monolitizacija u prosečnoj debljini od cca 7cm.

2.2 MONOLITNA KONSTRUKCIJA

2.2.1 Temelji samci

Temeljenje objekta izvedeno je primenom plitkog fundiranja preko temelja samaca livenih na licu mesta. Temelji samci u osnovi su pravougaone osnove, dimenzija u zavisnosti od opterećenja. Montažni stubovi se na temelje samce oslanjaju preko montažnih temeljnih čašica koje su ubetonirane u temeljne stope. Na delu objekta kod aneksa i kod četvorovlajka, pojedini stubovi su na temeljne samce oslonjeni preko montažnih ugradbenih Peikko elemenata. Temeljne trake po obimu objekta su pravougaonog preseka, osnovni poprečni presek je 20x150 cm, sa termoizolacijom prema spoljašnjosti objekta.

2.2.2 SPF bazen

SPF bazen (stanica za pripremu fluida) je konstrukcija koju sačinjava sistem horizontalnih i vertikalnih ploča koji su međusobno povezani uklještenom vezom. Dimenzije bazena u osnovi su 71.0x11x5m sa dubinom 6.5m od kote poda hale. Zidovi bazena su debljine 50cm i uklješteni su u temeljnu ploču debljine 60cm. Na delu bazena ispod stubova, u monolitnoj izvedbi izvedeni su stubovi debljine 70x70cm u ravni zida koji se oslanjaju na donju ploču. Stubovi su izvedeni do gornje kote zidova i na toj koti ugrađeni su Peikko elementi za montažni nastavak stuba. Zidovi i stubovi su armirani rebrastom armaturom kvaliteta B500B i izvedeni se betonom marke MB40 vodonepropusnosti V4. Sa spoljne strane bazena, prema tlu, postavljena je hidroizolaciona zaštita obzirom da se bazen nalazi u podzemnoj vodi.



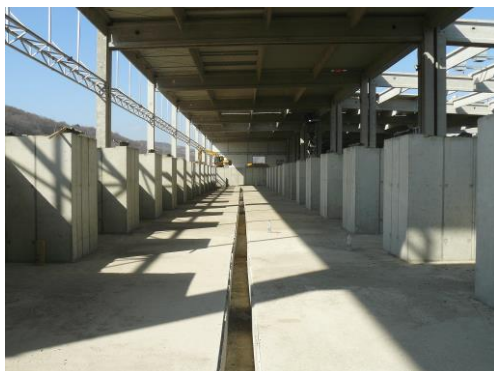
2.2.3 Konstrukcija četvorovaljka

Nezavisna jednoetažna AB monolitna konstrukcija unutar montažne hale i predviđen je za smještaj tehnološke opreme i pratećih sadržaja. Objekat je u osnovi pravougaoni dimenzija 20.6x50.6m i visine etaže na koti +5.20 od kote poda hale.

Konstruktivni sistem objekta čini prostorni, armirano betonski okvir, sa pločama, riglama, stubovima i trakastim temeljima – kontra gredama. Konstrukcija četvorovaljka je izvedena u monolitnoj izvedbi. Konstrukcija na koti +5.20 je a/b kontinualna ploča debljine $d=25\text{cm}$ promenljivih raspona $l_1=4.5-5.0-6.0\text{m}$, koja se oslanja na a/b rigle poprečnih jednobrodnih ramova. Rigle poprečnih ramova su dimenzija $b/d = 60/200\text{cm}$, sistemske dužine $L=20\text{m}$. Temeljne trake su sistema kontragrede. Temelji su izvedeni tako da pored konstrukcije četvorovaljka mogu da preuzmu temeljenje centralnih montažnih stubova.

2.2.4 Jame za vulkanizaciju

Objekat je jednokomorni armirano-betonski objekat i celom visinom je upušten u tlo. Po sistemu, objekat je sistema pločastih nosača. Temeljna ploča je debljine $d=40\text{ cm}$ sa pravougaonom osnovom gabarita $65.30 \times 10.34\text{ m}$ i konzolno je prepuštena u odnosu na zidove. Gornja kota temeljne ploče je na koti -2.50m . Zidovi su nazubljeni i monolitno su povezani sa temeljnom pločom. Svi zidovi su debljine 30 cm , a svi idu do kote -0.26m . Svi zidovi se krutom vezom vezuju za temeljnu ploču – uklješteni su.



3 MAGACIN SIROVINA

Objekat je u osnovi pravougaonog oblika sa dimenzijama cca. 120.60 m x 121.00 m , sa rasterom 24.0m x 20.0m. Spratnost objekta je "P" – prizemlje. Visina objekta u najvišoj tački (sleme) je +12.84 m' mereno od kote gotovog poda objekta. Krov je dvovodni, sa nagibom krovnih ravni (5.0%) od slemena prema podužnim osama. Objekat se sastoji iz dve dilatacione celine. Glavna konstrukcija je skeletna AB montažna konstrukcija raspona 24.0x20.0m. Montažna konstrukcija se sastoji od stubova na koje su montirani glavni krovni nosači, a na koje su oslonjeni sekundarni krovni nosači – rožnjače na koje je postavljen krovni lim i slojevi za termoizolaciju i hidroizolaciju. Stubovi su postavljeni u montažne ili monolitne temeljne čašice, koje su monolitnom vezom povezane sa monolitnim temeljima samcima. Temeljne grede su izvedene monolitno.



Slika 4 - Magacin sirovina

4 RASHLADNE KULE

Objekat za pripremu vode sa sistemom za hlađenje sa rashladnim kulama, lociran je naspram proizvodne hale sa severoistočne strane u sklopu industrijskog kompleksa. Objekat je u osnovi pravougaonog oblika sa dimenzijama cca. 55.10 m x 16.00 m i oslanja na temeljno tlo podnom-temeljnog AB monolitno izvedenom pločom debljine $d_{pl}=40$ cm. Iznad temeljne ploče, tehnički i tehnološki, objekat je podeljen na tri dela :

Bazen tople vode predstavlja prvu celinu. Konstrukcija bazena se sastoji od vertikalnih zidova koji su međusobno uklješteni, a takođe su uklješteni u podnu-temeljnu ploču. Spratnost prvog dela je prizemlje – P.

Središnji deo objekta je spratnosti P+1, izveden kao ramovska konstrukcija a čine je stubovi na koje se oslanjaju AB grede. Međuspratnu konstrukciju čine pune monolitne AB ploče. Krov je izveden kao ravan krov, sa slojevima termo i hidro izolacija.

Bazen hladne vode je treća celina. Konstrukcija bazena se sastoji od međusobno uklještenih vertikalnih zidova i temeljne ploče, debljine $d_{zid}=30$ cm. Spratnost ovog dela je prizemlje – P. Unutar bazena, u preseku osa su izvedeni AB monolitni stubovi, na koje se oslanjaju monolitne AB grede koje zajedno sa obodnim zidovima „nose“ AB monolitnu punu ploču iznad bazena debljine $d=20$ cm na koti +3.80 m. Gornja ploča bazena ima otvore cca. 345x218 cm, na koje se oslanja mašinska oprema.



Slika 5. Rashladne kule

5 PLATOI ZA ODLAGANJE GOTOVIH PROIZVODA

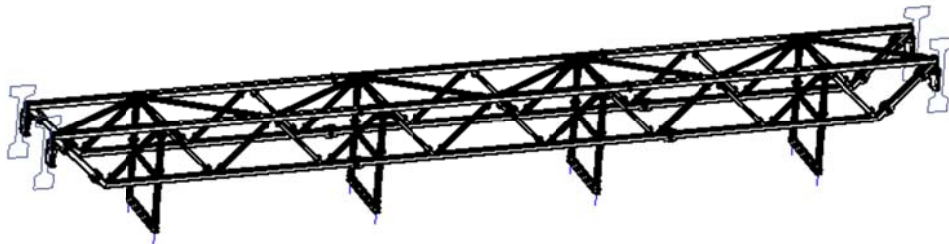
Platoi za odlaganje proizvoda su otvoreni sa mogućnošću natkrivanja montažnim šatorima. Uticaji od moguće konstrukcije za natkrivanje su uneti u proračun i konstrukcija je dimenzionisana za dobijene uticaje. Platoi su usnovi dimenzija 25.4x100.4m, 25,4x85.9m odnosno 25.4x30,4m. Ukupna površina AB platoa iznosi 17.530 m². AB ploča platoa je debljine 20cm armirana mrežastom armaturom po obimu ploče, prema detaljima armature i iglicama u količini od 20kg/m³ betona.

6 MOST ZA INSTALACIJE

Proizvodni objekat, uz sve prateće objekte i infarstukturu je bio vrlo zahtevan sa aspekta tehnologije koja se odvija unutar objekata, a što je za posledicu imalo izuzetno komplikovanu sinhronizaciju mašinskih, elektro i ostalih instalacije koja je trebalo da obezbedi nesmetano odvijanje tehnoloških procesa u periodu eksploatacije. Ono što je projektantima, naročito projektantima konstrukcije, zadavalo probleme na početku projektovanja i izrade izvođačkih detalja je bio nedostatak podataka vezanih za tehnologiju kao i obim i položaj instalacija koji prate funkcionisanje objekta. Praktično paralelno sa projektovanjem mašinskih instalacija teklo je projektovanje elemenata konstrukcije kao i izvođenje objekta. U tom momentu bilo je neophodno predvideti mogućnost dodatnog opterećenja na samu konstrukciju objekta, dovoljno da se smeste sve buduće instalacije, ali da konstrukcija i dalje bude racionalno isprojektovana. Iz razloga racionalnosti i praktičnosti, odlučeno je da svi instalacioni vodovi budu postavljeni uz podužnu osu, čitavom dužinom objekta cca 600m, sa bočnim račvanjem prema unutrašnjosti objekta.

Kako su se tokom projektovanja, ali već i izvođenja, postepeno dobijale i kompletirale informacije o zahtevima vezanim za tehnologiju i opremu tako je tekla i naša analiza dodatnog opterećenja na konstrukciju, kao i osmišljavanje 'u hodu' novih rešenja za preraspodelu opterećenja na već izvedenu konstrukciju. Most za instalacije proteže se čitavom dužinom objekta u dužini od cca 600m i sastavljen je iz segmenata dužine 24m. Svaki segment je izveden kao prostorna čelična rešetka oslonjena bočno na glavne nosače, statičkog sistema proste grede. Sve instalacije su u 2 nivoa smeštene unutar čelične konstrukcije. Primenom mosta ispod krovne ravni dobila se ušteda u materijalu, obzirom na to da su izbegnuti stubovi

platformi a ujedno je koristan prostor ispod koridora sa instalacijama ostao slobodan za komunikaciju.



7 ZAKLJUČAK

Prateći tržište kao projektanti se sve više suočavamo sa Design & Build aranžmanima, gde projektant aktivno učestvuje od početka projekta pa sve do samog kraja izvođenja objekta. Pri tome, procesna tehnologija za koju se gradi objekat se zna menjati i razvijati kako napreduje izvođenje. Ponekad to iziskuje i izmenu same tehnologije izvođenja objekta. Sve to zahteva veliku operativnost, pragmatičnost i fleksibilnost kako projektanta tako i samih rešenja koje projektant priprema - imajući pri svemu tome na umu ograničen budžet.

Da je projekat bio izuzetno obiman pokazuju i neke zanimljive činjenice koje govore o količinama. Tako je npr. tokom projektovanja ishodovano 7 dozvola za rušenje i 8 građevinskih dozvola, ukupan broj listova u zvaničnoj dokumentaciji je preko 100.000, sa površinom odštampanih tekstova i crteža od cca 2ha i težinom povezanih svezaka od oko 2T. Na samom gradilištu proizvedeno je preko 90.000t betona, ugrađeno 980t čelika, 312km kablova, 50km cevi za fluide, izvedeno 7 trafo stanica.

Na zadovoljstvo svih učesnika u izgradnji ovako kompleksnog objekta, sistemom design & built, objekat je u rekordnom roku priveden nameni.

Predrag Blagojević¹, Darko Živković², Aleksandar Šutanovac³

POSTROJENJE ZA PREČIŠĆAVANJE OTPADNIH VODA U LESKOVCU – ISKUSTVA I PREPORUKE U TOKU PROJEKTOVANJA KONSTRUKCIJSKIH SISTEMA

Rezime:

S-51

U radu su date najvažnije smernice za projektovanje konstrukcijskih sistema hidrotehničkih objekata, konkretno, postrojenja za prečišćavanje otpadnih voda, bazirana na projektantskom iskustvu. Definisana su merodavna dejstva i njihov direktni uticaj na količinu ugrađene armature. Prikazana je posebna analiza koja je sprovedena od dejstva temperature i skupljanja betona. Data je uporedna analiza varijantnih rešenja u cilju racionalizacije i smanjenja cene koštanja celog kompleksa. Prikazani su rezultati analize uticaja režima rada postrojenja na merodavne uticaje.

Ključne reči: aeracioni bazen, rezervoar, temperaturna promena, uzgon

WASTE WATER TREATMENT PLANT IN LESKOVAC – EXPERIANCE AND RECOMMENDATIONS IN THE COURSE OF STRUCTURAL SYSTEMS DESIGN

Summary:

Based on the design experiance, the paper presents the main recommendations for the design of hydraulic structures, specifically, waste water treatment plant structural systems. The design loads are defined, and their direct influence on the steel reinforcement quantity is shown. A separate analysis is given for temperature loads vs. concrete shrinkage. A comparative analysis is presented for several design solutions with an aim of rationalization and cost reduction for the entire waste water treatment plant complex. The results of the influence analysis for waste water treatment plant operation mode on the design loads is also shown.

Key words: aeration tank, storage tank, temperature change, uplift force

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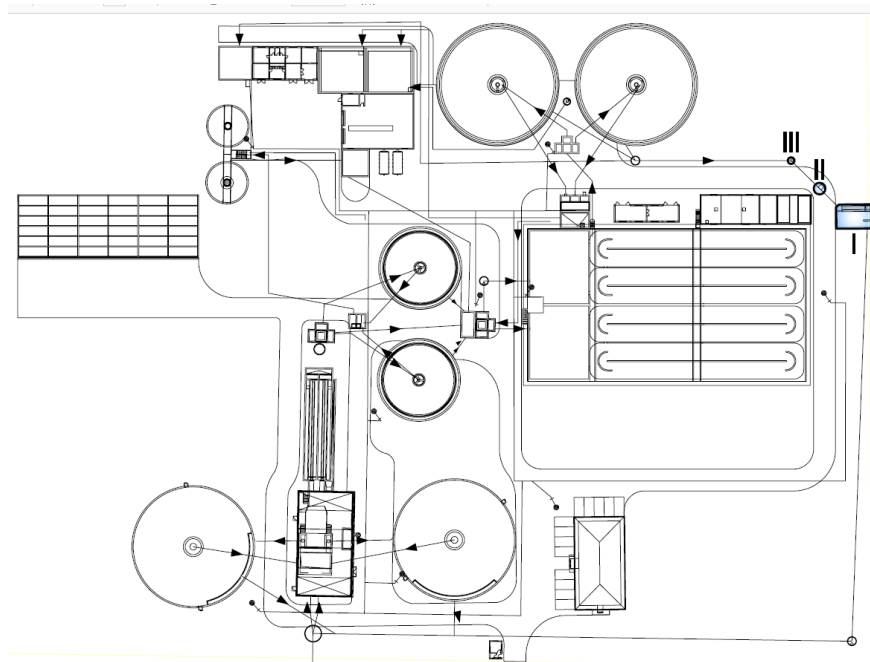
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1 UVOD

Kompleks objekata za pečišćavanje otpadnih voda u Leskovcu (PPOV) se sastoji od 38 objekata koji su prema funkciji podeljeni u 18 celina i spada među najveće na našim prostorima. Funkcionalne celine su: kompenzacija atmosverske vode, pumpna stanica za atmosversku vodu, primarno taloženje, finalno taloženje, rezervoar za aeraciju, zgušnjavanje mulja, odvodnjavanje mulja, objekat za odvodnjavanje mulja, objekat za skladištenje mulja, bajpas komora, kompresorska stanica, energetski blok, digestorska stanica, rezervoar za gas, komora za granulaciju, garaža i radionica, administrativni objekat, stražarnica.

Projektovanje i građenje kompleksa je trajalo oko tri godine.

Planirani kapacitet postrojenja u prvoj fazi izgradnje je 86.000 ES, dok je za kraj projektnog perioda predviđeno proširenje na konačnih 129.000 ES.



Slika 1 – Situacija PPOV u Leskovcu

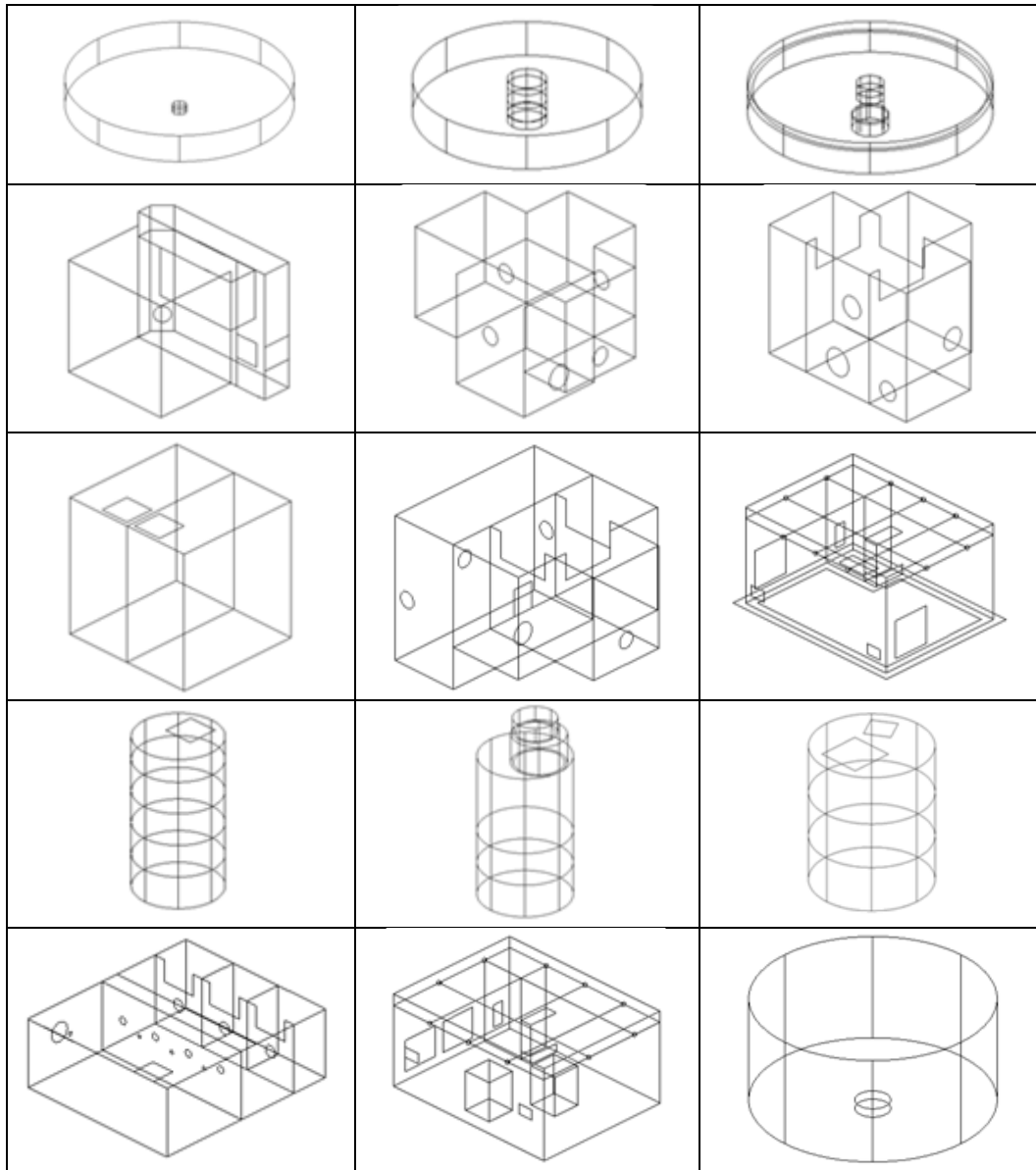
2 KONSTRUKCIJSKI SISTEMI OBJEKATA

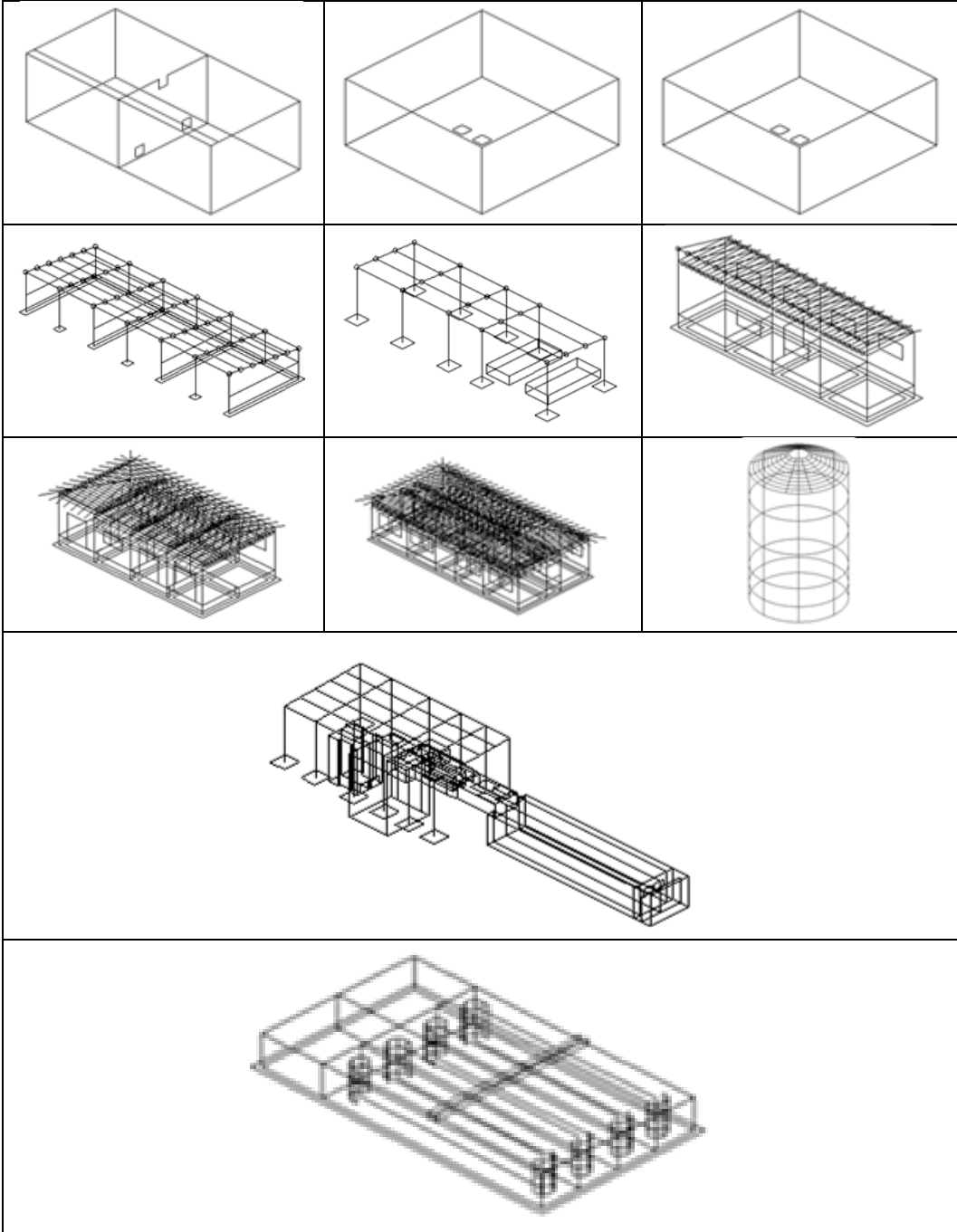
U konstrukcijskom smislu svi objekti se mogu podeliti na hidrotehničke armiranobetonske konstrukcije i prateće objekte visokogradnje (skeletni i zidani). U proračunskim modelima objekti su tretirani kao prostorni sistemi sa površinskim elementima kruto ili zglobovno povezanim između sebe. Uticaji u poprečnim presecima su dobijeni simulacijom opterećenja u skladu sa trenutno važećom regulativom za ovu vrstu objekata i lokaciju na kojoj se objekat nalazi. U toku projektovanja su ispoštovani svi zahtevi iz trenutno važeće regulative za ovu vrstu objekata. Za armiranje svih konstrukcijskih elemenata je usvojem čelik B500B i beton kvaliteta MB40 V7 M150. Proračun temeljne konstrukcije je urađen simulacijom tla kao elastične podloge preko modula reakcije tla za vertikalni pravac a u skladu sa elaboratom o geotehničkim uslovima temeljenja koji je sastavni deo projektne dokumentacije. Svi hidrotehnički objekti su dimenzionisani na dejstvo tla i dejstvo vode ili mulja. Objekti koji su zatrpani sračunati su i na vertikalno dejstvo nasipa i dejstvo snega ili vozila. Detaljno su analizirana dejstva od uzgona, temperature podzemne vode, temperature vazduha, temperature tečnosti u objektima leti i zimi. Hidrodinamički pritisak tečnosti na zidove i dno rezervoara (impulsivni i konvektivni) je sračunat u skladu sa Pravilnikom o tehničkim normativima za projektovanje i proračun inženjerskih objekata u seizmičkim područjima (članovi od 35 do 38). Analize u okviru ovog proračuna kao i iskustva sa sličnih objekata pokazuju da ova vrsta uticaja nije merodavna za dimenzionisanje preseka.

U okviru postrojenja za prečišćavanje otpadnih voda u Leskovcu nalazi se i nekoliko objekata koji u konstrukcijskom smislu predstavljaju prizemne zidane konstrukcije sa vertikalnim i horizontalnim serklažima. U toku projektovanja su ispoštovani svi zahtevi iz trenutno važeće regulative za ovu vrstu objekata a naročito odredbe iz "SI lista broj 87/91", "SI lista broj 31/81" (sa dopunama) i EN 1996 Evrokod EC7 (Proračun zidanih konstrukcija). Ispoštovan je osnovni preduslov za ovaj tip konstrukcija o potrebnoj površini zidova u odnosu na bruto površinu objekta za oba ortogonalna pravca, kao i o minimalnim procentima armiranja za vertikalne i horizontalne seklaže. Ploče iznad prizemlja su projektovane kao monolitne AB (OMNIA) konstrukcije koje se oslanjaju na zidane zidove i ivične grede. Međuspratne konstrukcije su debljine 15cm i u proračunu su tretirane kao kontinualne armiranobetonske ploče. Iznad tavanjskog prostora je projektovana klasična drvena konstrukcija uobičajenih raspona. Proračun preseka zidanih zidova je urađen prema dozvoljenim naponima.

Svi objekti kod kojih je bilo mogućnosti projektovani su kao armiranobetonski (AB) montažni objekti. Sekundarni krovni nosači su montažni, klasično armirani, u proračunu tretirani kao proste grede. Poprečni presek je tipa "T", visine 45cm. Širina flanše je 40cm, a rebra 14-16cm. Debljina flanše je 20cm. Glavni krovni nosači su klasično armirani ili atheziona prethodno napregnuti što zavisi od raspona. Poprečni presek je tipa "T" ili "I" visine u funkciji raspona u svemu prema licenciranoj tehnologiji izvođača radova ("Putinženjering" Niš). Geometrija nosača je uslovljena licenciranim kalupima. Osnovne dimenzije stubova su 50/50cm. Visine stuba su različite u zavisnosti od položaja u objektu. Svi stubovi su od MB 40 i armirani sa B500B. Temeljna konstrukcija je projektovana kao temelji samci sa temeljnim čašicama. Temeljne čašice su montažne a ploče temelja samaca su livene na licu mesta.

3 PRORAČUNSKI MODELI





4 ANALIZIRANA DEJSTVA I KOMBINACIJE DEJSTAVA

Pored standardnih kombinacija opterećenja analizirane su i sve moguće, realne i nerealne (zahtevane od strane Investitora) konstelacije opterećenja koje se odnose na dejstva od vode, zemlje i temperaturnih razlika i temperaturnih promena (od vazduha i podzemne vode) u režimu redovnog rada i remonta postrojenja:

- Sve faze u toku građenja;
- Potpuno završen objekat pre probnog punjenja (prazan i nije zatrpan) leti i zimi;
- Dejstvo vode i zemlje je analizirano za sve slučajeve redovnog režima rada (sve komore pune) i režim rada u fazi remonta (neke komore pune a neke prazne);
- Analizirana je situacija probnog punjenja objekta (sve komore pune i ne zatrpan objekat);
- Analizirana je situacija probnog punjenja objekta (ne zatrpan i pola komora prazno a pola puno);
- Dejstvo temperature u letnjem i zimskom redovnom režimu rada (sve komore pune);
- Dejstvo temperature u toku remonta usklađeno sa zahtevom tehnološkog procesa;
- Dejstvo temperature podzemne vode od 14 stepeni;
- Remont objekta, bilo koja komora puna ili prazna;
- Faza proširenja, dogradnja objekata;

U toku projektovanja objekata u okviru Postrojenja za prečišćavanje otpadnih voda u Leskovcu nije ispoštovan zahtev projektanta armiranobetonske konstrukcije da rezervoari za otpadnu vodu budu zatrpani celom svojom visinom (dubinom) ili da budu termički izolovani na nekim delovima. Delimično zatrpani rezervoari pored dejstava od opreme, vode, zemlje, skupljanja betona, dimenzionišu se i za uticaje od dejstava temperaturnih promena i temperaturnih razlika. U našoj trenutno važećoj regulativi dejstva od temperaturnih promena i temperaturnih razlika su definisana samo za drumske i železničke mostove. Za objekte u okviru Postrojenja za prečišćavanje otpadnih voda u Leskovcu ne postoji regulativa koja definiše intenzitete dejstava prirodne promene temperature. Projektantima konstrukcije je prepušteno da na osnovu podataka o temperaturama vazduha na lokacijama gde se objekti nalaze i tehnoloških procesa procenjuju intenzitete računskih graničnih temperatura.

Temperatura otpadne vode koja se nalazi u rezervoarima je u proseku $10^{\circ}C$ u zimskom periodu godine sa mogućnošću promene od $\pm 2^{\circ}C$ u zavisnosti od temperature vazduha (podatak dobijem od tehnologa). Temperatura otpadne vode koja se nalazi u rezervoarima je u proseku $20^{\circ}C$ u letnjem periodu godine sa mogućnošću promene od $\pm 2^{\circ}C$ u zavisnosti od temperature vazduha (podatak dobijem od tehnologa). Radi provere, konsultovan je Zavod za zaštitu zdravlja iz Leskovca kao i JKP Leskovac koji su prikupljali ulazne podatke i izvršili merenja temperature otpadnih voda kao ulazne parametre za projektovanje postrojenja. Podaci su provereni i upoređeni sa merenim temperaturama vode i vazduha na PPOV u Subotici za proteklu godinu. Podaci o temperaturama vazduha preuzeti sa sajta HMZ Srbije.

5 POSTROJENJE U FAZI GRADENJA I EKSPLOATACIJE



Slika 2 – Armiranje temeljne ploče kružnog rezervoara.



Slika 3 – Armatura zida kružnog rezervoara.



Slika 4 – Kružni rezervoar u fazi izgradnje.



Slika 5 – Aeracioni bazen u fazi građenja.



Slika 6 – Izgled dela kompleksa PPOV u Leskovcu.



Slika 7 – Izgled dela kompleksa PPOV u Leskovcu.

6 ZAKLJUČAK

Na osnovu rezultata u projektovanju 38 objekata u ovom postrojenju za prečišćavanje otpadnih voda i nekoliko sličnih postrojenja manjih kapaciteta iskustva su sledeća:

- Merodavni kriterijum za dimenzionisanje armiranobetonskih preseka je granično stanje upotrebljivosti a ne granično stanje nosivosti;
- Kombinacije opterećenja sa seizmičkim dejstava nisu merodavne ni u jednom slučaju;
- Posebno analizirati delove konstrukcije koji se nalaze iznad tečnosti (ne potpoljeni), potopljeni a ne zatrpani, potpoljeni i zatrpani do dobine od 100 cm i potpoljeni i zatrpani na većim dubinama od 100 cm;
- Za temperature vazduha usvojiti prosečne mesečne temperature (posebno za letnji i zimski period);
- Za temperature tečnosti koristiti vrednosti dobijene od tehnologa i potvrđene merenjem na objektu koji je u eksploataciji;
- Referentnu temperaturu usvojiti u skladu sa dinamikom građenja ili u skladu sa predlogom Pravilnika za opterećenje mostova;
- Prsline u armiranobetonskim konstrukcijskim elementima nisu smetnja ako im širina ne premašuje dopuštenu graničnu vrednost, odnosno pojava prslina je dozvoljena;
- Prsline koje su se pojavile i posle toga sanirane ne predstavljaju opasnost u daljoj fazi eksploatacije objekta. Ovo se dokazuje ne curenjem tečnosti koja se nalazi u objektu (prethodne probe). Sanirane prsline ne umanjuju kvalitet konstrukcijskih elemenata i objekta u celini;
- Na cenu koštanja direktno utiče izloženost objekta temperaturnim uticajima što navodi na zaključak da je poželjno objekte (kada je to moguće) zatrpati – zasuti zemljom ili termički izolovati.

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PROCENA STANJA AB REZERVOARA U SKLOPU POSTROJENJA ZA PREČIŠĆAVANJE VODE ZA PIĆE

Rezime:

S-52

U radu je prikazana procena stanja rezervoara čiste nehlorisane vode u sklopu postrojenja za prečišćavanje vode za piće. Konstrukcija je izvedena sa velikim brojem defekata, a u toku eksploatacije su se pojavila oštećenja koja ugrožavaju funkcionalnost i trajnost postrojenja. Kako bi se utvrdio stepen oštećenja i procenila mogućnost daljeg korišćenja, sprovedeni su detaljan vizuelni pregled konstrukcije i ispitivanje kvaliteta ugrađenih materijala. Zaključeno je da globalna stabilnost konstrukcije nije ugrožena, ali da je trajnost svih pregledanih elemenata značajno smanjena, usled defekata i fizičke i hemijske korozije betona i korozije armature.

Ključne reči: procena stanja, AB konstrukcija rezervoara, defekti, oštećenja, trajnost

ASSESSMENT OF RC RESERVOIR WITHIN DRINKING WATER TREATMENT FACILITY

Summary:

The paper presents assessment of reservoir for clean non-chlorinated water within a drinking water treatment plant. The structure was built with a large number of defects, while damages that endanger the functionality and reduce durability of facility appeared in the meantime. In order to determine type and extent of damages and assess the possibility of further use, a detailed visual inspection of structure and subsequent testing of built in materials were conducted. It was concluded that the global stability of the structure is not jeopardized, while durability of examined elements is significantly reduced, due to defects, physical and chemical corrosion of concrete and reinforcement corrosion.

Key words: assessment, RC reservoir, defects, damages, durability

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1 INTRODUCTION

The paper presents assessment of RC structure of reservoir for clean non-chlorinated water within drinking water treatment plant in Sombor. The reason for assessment was reconstruction and enlargement of the plant which was built 30 years ago.

The assessment involved the following activities:

- Visual inspection of all accessible parts of RC reservoirs structure,
- Testing of the concrete cover quality by measuring the depth of carbonation,
- Subsequent testing of concrete compressive strength by extraction of cores from foundation slab and reservoirs walls and
- Checking of disposition, type and diameter of embedded reinforcement.

Based on obtained results and classification of defects and damages [1,2,3,4,5] a conclusion on the structure state, in terms of load-bearing capacity, stability and durability, was given.

2 FACILITY DESCRIPTION

Whole plant for drinking water treatment is situated in one building with rectangular floor plan and with dimensions 26x16x13,3m. The building is partially dug into the ground to a depth of 2.3m (Figure 1). The load bearing structure is complex and consists of massive RC structure and RC skeleton-frames. Functionally, viewed from the foundation slab to the top of building, the plant consists of:

- Reservoir for clean non-chlorinated water, with volume of 1500m³,
- Retention basin, with aerator facility and revision trail above,
- Sand filters with double bottom and
- Pipe gallery.

Reservoir for clean non-chlorinated water is located in the basement and covers the whole basis of the building (Figure 2). The dimensions of reservoir are 26mx16mx5,15m.



Figure 1 – The appearance of the facility



Figure 2 – View of interior of reservoir

Basic structural elements of the reservoir are: the foundation designed as a mushroom slab with depth of 30cm, that is also a reservoir floor slab, RC columns with square cross section, 30cmx30cm and with capitals located in their down parts, upper RC slab POS 5 (A-E/1-6) with depth of 20cm, that is also a bottom slab of retention basin; upper RC slab POS 8a (E-F/1-6)

with depth of 15cm, that is also a bottom slab of pipe gallery, upper RC slab POS 12 (F-I/1-6) with depth of 20cm, that serves as a bottom of the filters facility and RC perimeter walls - POS Z4 with variable width from bottom up to 0,8m height and with constant width of 25cm from 0,8m, up to the top of reservoir. Positions of the upper slabs of the reservoir, columns and perimeter walls, are given in Figure 3.

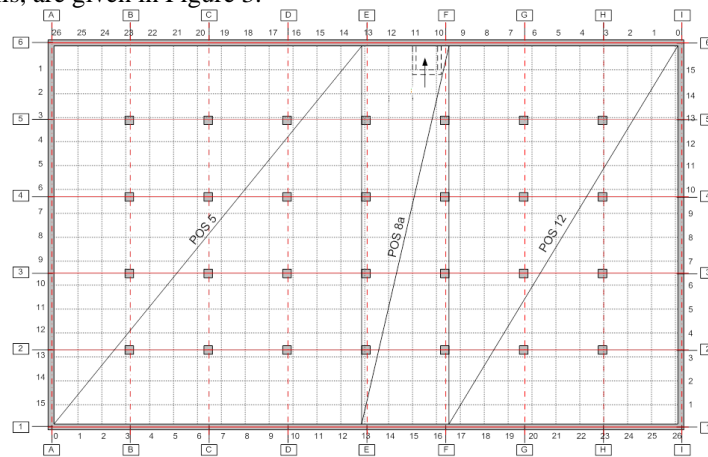


Figure 3 – Position of the structural elements of reservoir for clean water

3 DETAILED VISUAL INSPECTIONS

Detailed visual inspection was done after cleaning and washing inner surfaces of reservoir and covered: upper side of foundation-floor slab, inner and outer sides of perimeter RC walls, down side of upper RC slabs and RC columns.

All visible surfaces inside the reservoir are protected with epoxy coating, which is mostly in good condition. The exterior surfaces of the walls are left as the "natural" concrete. The characteristic defects and irregularities of the outer side of the walls are: insufficient concrete cover irregularly executed cold joints during concreting, uneven surfaces, honeycombs and linear segregations and improper repair works. It was concluded that reservoir walls were concreted in the wooden panels' formwork. Joints of adjacent panels were not sealed, hence cement mortar leaked out during the concrete compaction. Therefore, in these zones, concrete remained porous and honeycombed and was repaired by cement mortar. Also, this mortar used as a concrete cover in areas where reinforcement bars were visible or where cover was insufficient.

Typical damages of walls are vertical cracks and fissures that are almost uniformly distributed along them. By comparing the distribution and orientation of registered cracks from both sides of the walls it was concluded that most of the cracks pass through whole depth of the walls. The cracks are dominantly vertical, with width 0.1-1mm and with length 2-2.5m. Traces of leakage and dissolved salts, as well as mosses are noticed on most of the cracks on the external side of walls. Some cracks visible on the inner side of the walls are connected with cracks in upper slabs. Characteristic damages of outer and inner side of the walls are graphically illustrated on Fig. 4 and 5, while appearance of the cracks is given in Fig. 6 -8.).

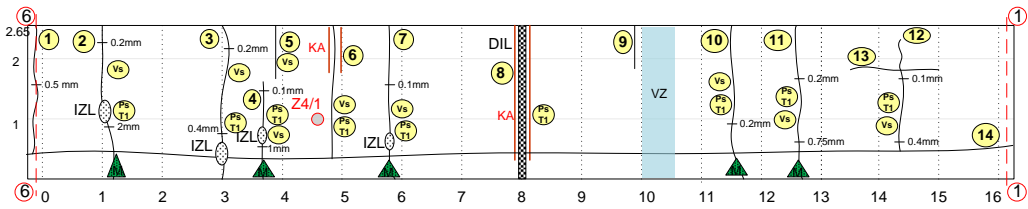


Figure 4 – Characteristic damages on the outer side of the wall Z4 A/1-6,S

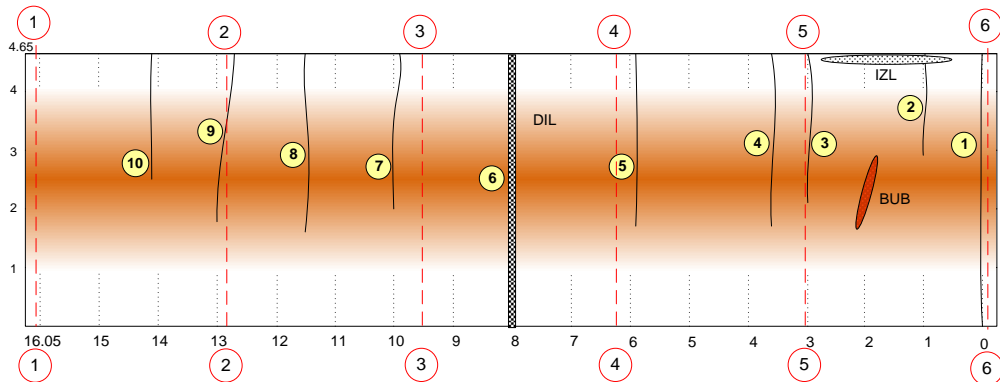


Figure 5 – Characteristic damages on the inner side of the wall Z4 A/1-6,S

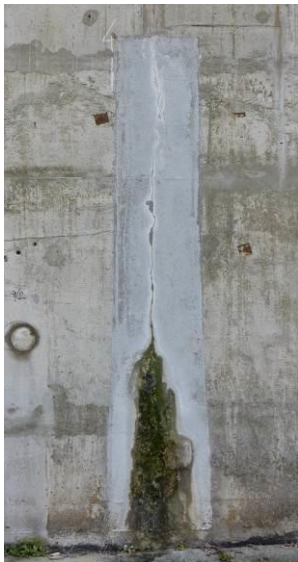


Figure 6 – Vertical crack, leakage, moss, previous repair – T1



Figure 7 – Vertical crack, leakage, moss, previous repair - T2



Figure 8 – Series of vertical cracks, linked with cracks in the upper slab

On the basis of the morphology of the registered cracks it was assumed that they appeared in the period of hardening of concrete due to hydraulic shrinkage.

In the past period repair of cracks was performed several times. Two surface repair techniques of cracks (T1 and T2) were observed:

- Application of polymer-cement coating in form of a strip (T1) – Figure 6,
- Application of the same coating which was reinforced by fiberglass mesh, (T2) – Fig. 7.

Some of the cracks were injected in previous period.

From 50 cracks registered on the reservoir walls 25 were repaired by technique T1, 3 by T2 and 3 were injected. On all cracks repaired by technique T1 the traces of leakage and dissolving salts were noticed, due to cracking, delamination and flaking of polymer cement coating. It was concluded that this technique was inappropriate. Also, traces of leakage were registered on more than 50% unrepaired cracks.

Structural defects on the external side of walls and their inadequate repairs caused quick corrosion of reinforcement bars due to concrete carbonation. Since defects encompass a large area of external side of walls, the corrosion of reinforcement is the second most important damage of the perimeter walls of the reservoir. This damage is the most evident in the lower parts of walls (Fig 9) and in cold joints areas. The reinforcement corrosion is followed by cracking, delamination and falling of the concrete and repair mortar.



Figure 9 – Insufficient thickness of the concrete cover, cracking, delamination and falling of the concrete, reinforcement corrosion in the lower part of the wall



Figure 10 – Appearance of damaged expansion joint, inner side of wall

In the thirds of longitudinal walls length, as well as in the halves of transverse walls length, expansion joints were registered. Expansion joints are fulfilled with permanently flexible material and from the inner side of reservoir additionally are covered with a protective epoxy coating. Due to the movement of the filling material, rigid epoxy coating had cracked (Fig. 10). This enabled water pass through joints. From a total of 6 expansion joints, water leakage was registered on 3 of them.

Visual inspection of down side of upper slabs POS 5, POS 8a and POS 12, was performed from the floor slab level. Characteristic damages are cracks and fissures in the protective coating and RC slabs (Fig.11). There are three types of cracks:

- Slant cracks; these cracks are usually located in the corners of the slabs (Fig. 11, 14) and pass through the whole thickness of slab; the dark liquid comes out of the cracks.
- Cracks above the expansion joints; these cracks are perpendicular to the nearest wall and also pass through the whole slab thickness; the dark liquid comes out of the cracks (Fig 13).
- Cracks parallel to main axis; they can be individual or in groups, mainly pass through the whole slab depth and have white traces of leakage and stalactites (Fig. 12).

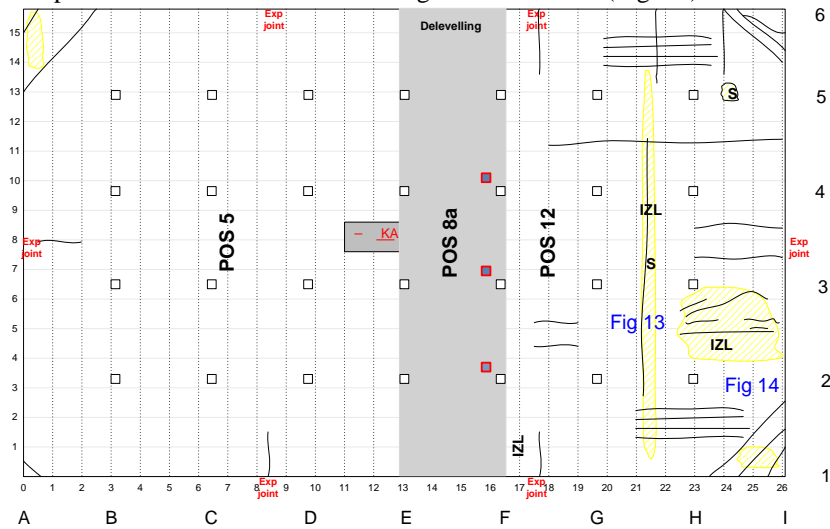


Figure 11 – Disposition of damages on down side of upper slabs POS 5, POS 8a and POS 12



Figure 12 – Slab POS 12, cracks parallel to main axis, leakage of $\text{Ca}(\text{OH})_2$, white stalactites



Figure 13 – Slab POS 5; crack above expansion joints, traces of dark liquid leakage

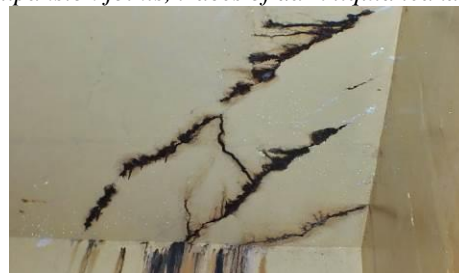


Figure 14 – Slab POS 12; slant cracks in corner, traces of dark liquid leakage, dark stalactites

Possible causes of appearance of cracks are: plastic concrete shrinkage, long-term hydraulic concrete shrinkage, connection of materials with different deformation properties, reinforcement corrosion and irregular cold joints.

By analysing location and pattern of identified cracks it was concluded:

- On down side of upper slab POS 5 only few cracks were registered; they are slant cracks and cracks above expansion joints;
- No damages were identified on upper slab POS 8a and
- Upper slab POS 12 is the most damaged. All three types of cracks were noticed on its down surface. Consequently, bearing capacity of the slab is reduced.

RC floor slab is also protected with epoxy coating. During the visual inspection, a large surface area was covered with water and some dark sludge, which made visual inspection and registration of damages difficult. It was concluded that there are no damages on the visible parts of floor slab (Figure 15).

By visual inspection of RC columns only local delamination and flaking of protecting epoxy cover were noticed (Figure 16).



Figure 15 – The appearance of foundation slab



Figure 16 - RC column, local peeling of the protective coating

4 IN-SITU TESTING

4.1 CONCRETE COMPRESSIVE STRENGTH

In order to assess of in-situ compressive strength concrete cores with diameter 100mm were taken, examined, prepared and tested in accordance with SRPS U.M1.048 [6] and SRPS U.M1.049 [7]. Three cores were taken from the floor slab and three from the perimeter reservoir walls. The appearance of extracted concrete cores is given in Fig 17 and 18. By analysing the results of the examination and by evaluation of the concrete compressive strength, it was concluded that concrete built into floor slab and perimeter walls of reservoir meet the requirements for class C25/30 and fulfil requirements for designed compressive strength.



Fig. 17 –Concrete core taken from floor slab

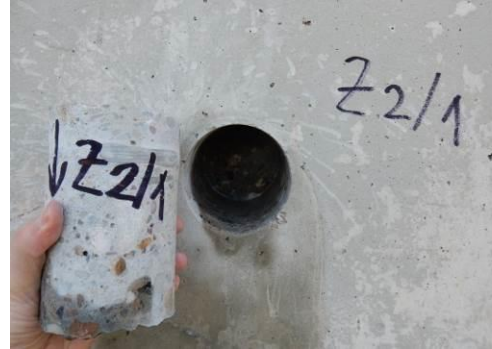


Fig 18 –Concrete core taken from RC wall

4.2 DEPTH OF CARBONATION

In order to determine the condition of the concrete cover in terms of corrosion protection of reinforcement, colorimetric method using phenolphthalein was chosen. The depth of carbonated concrete was measured immediately after extraction of concrete cores (Fig. 19, 20).



Figure 19 – Measuring of carbonised concrete layer depth – RC wall (10mm)



Figure 20 – Measuring of carbonised concrete layer depth – RC floor slab (0mm)

It was concluded that the carbonation has affected 0 cm to 20 cm of concrete cover in perimeter walls, but only from external side. Since the typical defects of the RC walls are linear segregation, honeycombs, as well as insufficient thickness of the concrete cover, it can be concluded that carbonation front reached the reinforcement. Therefore, the concrete cover has lost its protective role on the greater part of the walls surface and reinforcement bars are exposed to corrosion. Floor slab concrete did not carbonize thanks to protective coating that prevents the penetration of CO₂ into the concrete (Figure 20).

4.3 REINFORCEMENT BAR DISPOSITION

The arrangement and type of reinforcement bars in upper zone of foundation slab were done by combination of destructive and non-destructive methods. Using pick hammer the cover was removed and bars from both directions were bared. Profometer PROCEQ was used for non-destructive testing on three places. Water and sludge made these measurements very difficult. The disposition of reinforcement is shown in Fig 20 and 21.



Figure 21 – Upper side of foundation slab, destructive method, reinforcement arrangement

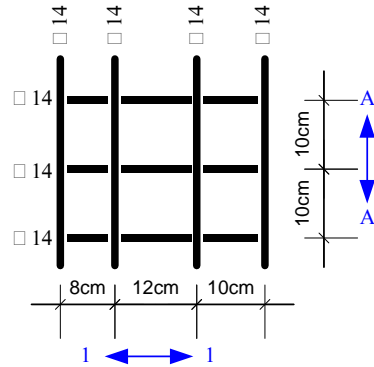


Figure 22 – Disposition and dimensions of built in reinforcement

5 CONCLUSIONS

Based on the analysis of the results obtained through detailed visual inspection, as well as the analysis of the results obtained by in-situ and laboratory tests of concrete and reinforcement, the following was concluded:

- Concrete built into foundation slab and perimeter walls meet the requirements for class C25/30 and for designed compressive strength.
- The carbonation affected 0 cm to 20 cm of concrete cover in perimeter walls from external side. Due to that reinforcement bars are exposed to corrosion.
- Structural elements of reservoir were built with numerous of defects and irregularities. They have been manifested as honeycombs, segregation zones and insufficient concrete cover, irregular cold joints and inadequate repairs.
- The characteristic damages of RC walls and slabs are cracks and fissures that pass over the whole thickness of walls or slabs. Most of cracks and fissures have traces of water leakage and leaching.
- Corrosion of reinforcement, followed by cracking, delamination and falling of concrete cover and cement mortar is characteristic for outer side of perimeter walls.
- Protective epoxy coating is in relatively good condition. Characteristic damages of coating are "copied" cracks from RC walls and slabs.

Based on the assessment of load-bearing RC structure of reservoir for clean non-chlorinated water, the following was concluded:

- Load-bearing capacity of RC perimeter walls is not compromised,
- Load-bearing capacities of RC slabs POS 5 and POS 8a are not compromised,
- Load-bearing capacity of RC slab POS 12 is reduced due to the existence of a large number of cracks that passing through whole slab thickness,
- Durability of all inspected RC walls and RC slabs is significantly reduced, primarily due to the large number of defects, physical and chemical corrosion of concrete and reinforcement corrosion,
- RC columns, as well as foundation slab, are free from visible damages, that could reduce their bearing capacity and durability,

- Functionality of reservoir is compromised due to existence of cracks and fissures with leaking water.

In order to ensure the required load-bearing capacity of slab POS12, it is necessary to perform structural repair of that element.

In order to ensure water impermeability of reservoir for clean non-chlorinated water, it is necessary to repair all cracks and fissures with traces of leakage, regardless whether or not they have been repaired in the previous period. Also, for the same reason, it is necessary to repair the expansion joints. Repair measures should encompass corroded reinforcement rods, as well as damaged and insufficient concrete cover.

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Vladan Stamenić¹

ISKUSTVA PRI IZVOĐENJU AMIRANOBETONSKIH DIJAFRAGMI

Rezime:

S-53

U radu su prikazana iskustva stečena tokom izvođenja armiranobetonskih dijafragmi na objektima u Ruskoj Federaciji, grad Soči. Rad je nastao adaptacijom priručnika koji sam namenio mlađim kolegama u firmi PUTEVI Užice (početnicima) s ciljem da im omogućim lakše savladavanje problematike vezane za ovu vrstu poslova. Posebna pažnja je posvećena slikovitom prikazu ključnih faza radova (redosled operacija zaključno sa betoniranjem).

Ključne reči: dijafragma, bentonit, top & down, stopsol

EXPERIENCE IN PERFORMING REINFORCED CONCRETE DIAPHRAGM

Abstract:

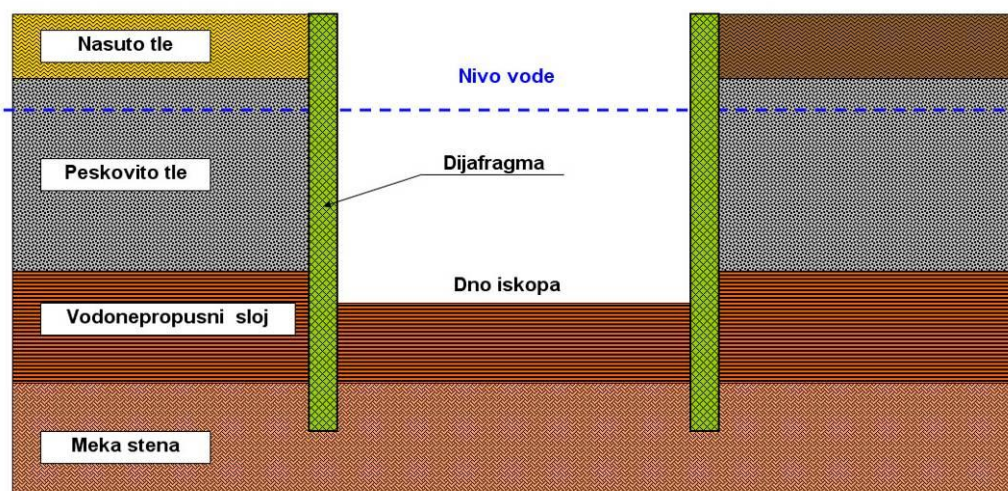
The views expressed in this paper represent the experience gained during the execution of reinforced concrete diaphragms on projects in Russian Federation, the city of Sochi. This paper represents an adaptation of the manual that I wrote for my younger colleagues (beginners), employees in construction company PUTEVI Užice, in order to help them overcome challenges related to this kind of work. Special attention was paid to the picturesque view of key phases of work (the order of operations ending with pouring concrete).

Key words: diaphragm walls, bentonite, top & down, stopsol

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1 UVOD

Dijafragme se primenjuju pri izgradnji podzemnih objekata u stesjenim gradskim uslovima gde u neposrednoj blizini postoje susedni objekti. Dijafragme mogu biti od monolitnog betona, ili od montažnih panela. Ovde će mo razmatrati monolitne dijafragme. One zaštićuju iskop od obrušavanja okolnog tla i daju stabilnost susednim objektima u neposrednoj blizini. Najbolji rezultati se postižu kada dijafragma prolazi kroz sve vodonosne slojeve i završava se u vodonepropusnom sloju (slika 1).



Slika 1.

U tom slučaju moguće je u iskopu izvoditi radove bez spuštavanja nivoa podzemne vode. Dakle dijafragma s obzirom na svoju vodonepropustivost može obavljati i ulogu vertikalne protivfiltracione zavese. Dubina dijafragme zavisi od projektnih rešenja objekta koji zaštićuje i od mogućnosti opreme za njenu izradu koja stoji na raspolaganju. Jedana od najpoznatijih firmi u svetu koja se specijalizovala za izradu dijafragmi je **Soletanche Bachy** (Francuska), koja izvodi radove u 80 zemalja sveta.

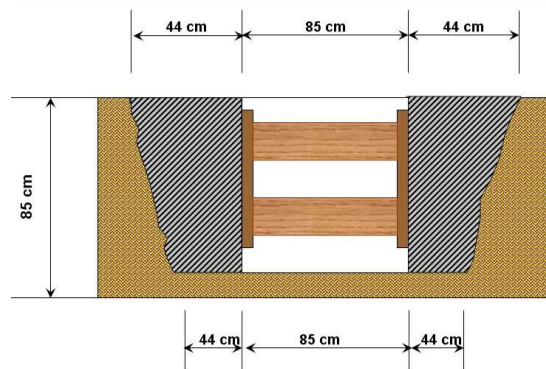
Kod izvođenja dijafragmi prisutni su sledeći tehnološki procesi:

- Montaža opreme (mašina za iskop i montaža centrale za spravljanje, prečišćavanje i distribuciju rastvora bentonita)
- Izrada vođice (foršahte)
- Iskop tla pod rastvorom bentonita koji se vrši po posebnim sekcijama (panelima) pomoću mašine sa hidrauličkim grejferom ili frezom
- Armiranje i betoniranje posebnih panela dijafragme

Centrala za bentonit je veoma važan element u procesu izvođenja radova na dijafragmi i o njoj će biti reči kasnije u posebnom poglavlju.

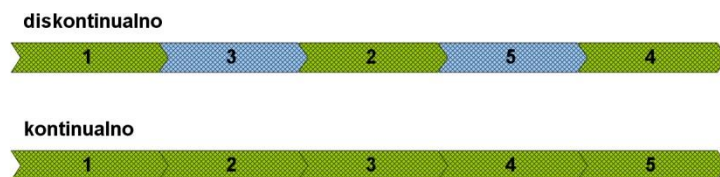
2 ISKOP ROVA ZA DIJAFRAGMU

Pri iskopu za dijafragme se može koristiti mašina sa hidrauličkim grejferom ako su u pitanju disperziona tla (pesak, šljunak i glina) ili mašina sa hidrauličkom frezom ako su u pitanju stenovita tla niske čvrstoće (argiliti, peščari). Pored hidrauličnih, grejferi mogu biti i mehanički. Debljina dijafragme najčešće se kreće od 0,60 do 1,00 m, pa se u ovom rasponi kreću i debljine čeljusti grejfera. Širina čeljusti grejfera u otvorenom stanju su najčešće od 2,5 do 3 metra. Najpoznatiji proizvođači tehnike za iskop dijafragmi su **Casagrande i Bauer**. Neke firme, kao naprimer **Soletanche Bachy** imaju razrađenu konstrukciju grejfera i sopstvenu proizvodnju. Pre početka iskopa za dijafragmu potrebno je po osi izraditi betonsku vođicu-foršahtu (slika 3), koja može biti montažna ili livena na licu mesta. Foršahta obezbeđuje da dijafragma ima potrebnu geometrijsku formu, a istovremeno odezbeđuje stabilnost gornjeg dela zidova rova za dijafragmu.



Slika 3. Primer vođice za dijafragmu debljine 80 cm

Front rada pri iskopu rova za dijafragmu se deli na posebne sekcije (panele) koje su najčešće širine od 2,5 do 6 metara. Širina panela dijafragme zavisi od geoloških uslova na lokaciji, blizine susednih objekata i raspoložive opreme firme koja izvodi radove. Obično se u slučaju postojanja susednih objekata projektuju paneli širine do 3 metra. Iskop rova za sekcije i njihovo betoniranje može se obavljati diskontinualno ili kontinualno (slika 4).

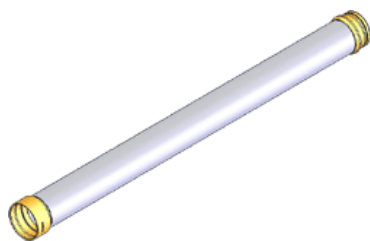


Slika 4.

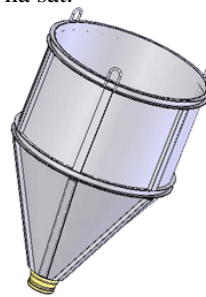
Sekcije dijafragme se armiraju prostornim armaturnim koševima sa vertikalnom nosećom armaturom. U koševima se ostavljaju otvori za prolazak cevi za betoniranje, a na njih se stavljaju graničnici za fiksiranje njihovog položaja i obezbeđenje zaštitnog sloja. Armaturni koševi se obično izrađuju na celu dubinu dijafragme.

3 BETONIRANJE DIJAFRAGMI

Oprema za betoniranje dijafragmi se sastoji od metalnih cevi (slika 5-a) koje su opremljene vezama koje obezbeđuju brzu demontažu, bunkera za prijem betona sa klapnom (slika 5-b) i pumpe za crpljenje bentonita. Redosled betoniranja je sledeći: Posle iskopa rova za dijafragmu i montaže armaturnog koša postavljaju se cevi za betoniranje na dubinu do dna iskopanog rova. Na početnu cev se postavlja bunker za betoniranje. Kad se bunker napuni betonom otvara se razdelna klapna i beton kroz metalnu cev pada na dno rova. Istovremeno se iz rova ispumpava rastvor bentonita u specijalnu posudu. Betoniranje se prekida kada se na vrhu foršajte pojavi čist beton. Intenzitet betoniranja osnovnih sekcija dijafragme treba da bude najmanje 8 m³ na sat, a sekcija između osnovnih najmanje 20 m³ na sat.



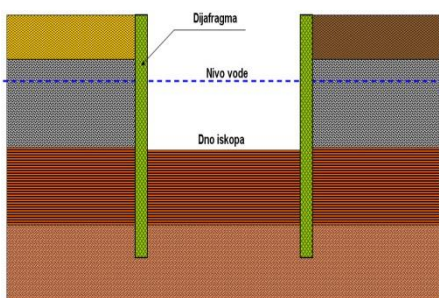
Slika 5-a



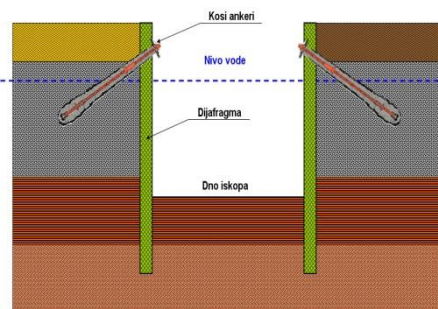
Slika 5-b

4 STABILNOST DIJAFRAGMI

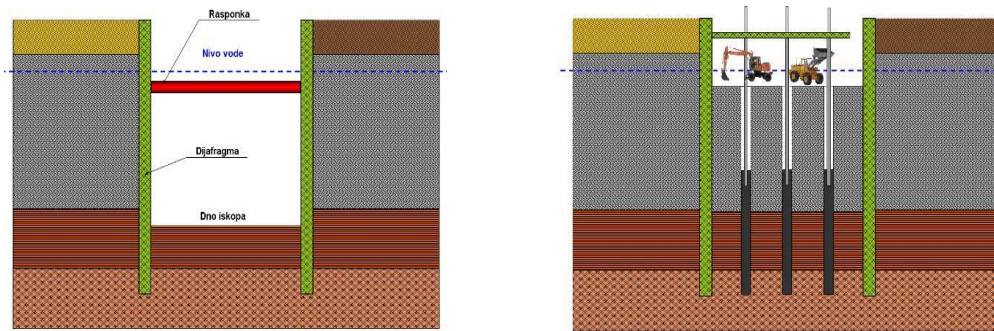
U pogledu stabilnost dijafragme mogu biti konzolne, sa jednim redom kosih ankera, sa više nivoa kosih ankera i sa rasponkama. Često se za razupiranje koriste podzemni delovi konstrukcije objekta, kao što su međuspratne ploče. U tom slučaju se najpre po obimu objekta izvodi dijafragma, a zatim vertikalni elementi podzemnog dela objekta. Oni se betoniraju na isti način kao i dijafragma. Posle toga se vrši iskop tla na kotu međuspratne ploče prve ispod nivoa terena, pa se betonira sama međuspratna ploča. Dalje se vrši iskop do kote sledeće međuspratne ploče i tako redom do temelja. Ploče služe kao rasponke dijafragme.



Konzolna dijafragma



Dijafragma sa jednim redom kosih ankera



Dijafragma sa rasponkama

Dijafragma koja se razupire podzemnim delovima objekta

Konzolne dijafragme se primenjuju u slušaju kada dubina iskopa nije velika. Stabilnost se obezbeđuju uklještenjem dijafragme u tlo ispod dna iskopa. Ovakvi slučajevi se u praksi javljaju ređe, odnosno u tim slučajevima se obično i ne izvodi dijafragma, već neka druga jeftinija konstrukcija

Ankerne konstrukcije se primenjuju da bi se obezbedila stabilnost dijafragme, smanjile deformacije i u unutarnja naprezanja u njoj. Primena ankera je u većini slučajeva najekonomičnije rešenje, a dobija se i slobodan prostor unutar dijafragme za nesmetane izvođenje drugih radova. Ankeri se mogu izvoditi u jednom ili više nivoa. Broj nivoa ankera, sila u njima, kao i osovinsko rastojanje se određuju statičkim proračunom dijafragme i zavise od dubine iskopa, geoloških uslova i konstrukcije samih ankera. Primena ankera je dozvoljena u svim vrstama tla izuzev glina tečne konzistencije i treseta. Ankeri mogu biti stalni i privremeni. Pri izradi ankera moraju se vršiti kontrolna ispitivanja i merenja, na osnovu kojih se mogu korigovati broj i dužina ankera. U slučaju postojanja više nivoa ankera iskop unutar dijafragmi se obavlja fazno, svaki put do kote konkretnog nivoa ankera. Primena ankera je ograničena u slučaju postojanja dubokih temelja susednih objekata, podzemnih komunikacija i sl. Mogu se javiti i problemi prodiranja vode u iskop na mestu prolaska ankera kroz dijafragmu.

Rasporne konstrukcije se takođe primenjuju da bi se obezbedila stabilnost dijafragme, smanjile deformacije i u unutarnja naprezanja u njoj. Kao i ankeri rasporne konstrukcije mogu biti u jednom ili više nivoa. Kod objekata gde dijafragma u osnovi ima izduženu formu, odnosno iskop nije širok (širina od 15-18 m) rasponke se rade bez privremenih oslonaca. U suprotnom se moraju formirati privremeni oslonci. Rasponke se najčešće izvode od metalnih cevi, kao i od valjanih čeličnih profila. Pored rasponki, koje moraju biti horizontalne dijafragma se može razupirati i kosnicima. U tom slučaju se najpre vrši iskop u centralnom delu, odnosno duž dijafragme se ostavlja zemljana berma. Zatim se na iskopanom delu izvodi temeljna ploča na koju se oslanjaju kosnici. Nedostatak rasponki i kosnika je što prostor unutar dijafragme nije potpuno slobodan, čime je znatno otežan iskop.

Jedan od najekonomičnijih načina da se obezbedi stabilnost dijafragme je da se za to iskoriste podzemni delovi objekta koji se gradi, odnosno međuspratne ploče podzemnih spratova. Ovakav način gradnje može se izvoditi metodom "**top & down**" (u prevodu gore i dole) ili "**up-down**" (samo dole). Izuzev ove dve metode može se koristiti i metoda "**semi**

up-down". Kod gradnje metodom "**top & down**", posle završetka dijafragme istovremeno se izvode podzemni i nadzemni delovi konstrukcije objekta, dok se kod gradnje metodom "**up-down**", najpre izvodi podzemni deo objekta, a po njegovom potpunom završetku nadzemni. I u jednom i drugom slučaju po završetku dijafragme se na mestu stubova objekta izrađuju bušeni šipovi sa postojeće kote terena, a betoniraju se samo do kote dna temeljne ploče. Odmah po završetku betoniranja se u šip ugrađuje privremeni stub koji je najčešće čelična cev ili valjani profil i koji zalazi u beton na potrebnu dubinu, naprimer 2 m. Neizbetonirani deo bušotine se zatim zasipa šljunkom. Kad su izvedeni svi šipovi i postavljeni svi privremeni stubovi, vrši se iskop na kotu donje ivice međuspratne ploče nad poslednjim podzemnim spratom. Preko dna iskopa se zatim postavlja polietilenska folija ili radi sloj mršavog betona, zatim se vrši armiranje i betoniranje međuspratne ploče. Ploča se po obimu oslanja na dijafragmu u kojoj se pri betoniranju moraju ostaviti gnezda, a po centralnom delu se oslanja na privremene čelične stubove. U ploči se moraju ostaviti privremeni otvori kroz koje će se vaditi zemlja pri sledećoj fazi iskopa. Kad je beton ploče dostigao potrebnu marku specijalnom tehnikom se vrši iskop ispod nje do kote dna međuspratne ploče prve ispod. Zatim se ta ploča armira i betonira i postupak ponavlja do se sa iskopom ne dođe na kotu dna temeljne ploče. Po završetku izvođenja temeljne ploče se odozdo na gore izvode vertikalni elementi konstrukcije, zidovi stubovi, stepeništa i sl. Privremeni stubovi se ne demontiraju već ostaju zabetonirani u unuteajnosti stalnih stubova objekta. Kod metode "**top & down**" se odmah po betoniranju prve međuspratne ploče, paralelno sa iskopom i izradom podzemnih delova konstrukcije nastavlja sa izvođenjem nadzemnog dela konstrukcije, a kod metode "**semi up-down**" se izvodi samo prvi red privremenih stubova do dijafragme dok se središnji deo objekta izvodi klasičnim načinom.

5 RASTVOR BENTONITA

Pri iskopu za dijafragmu u rov se dodaje rastvor bentonita koji obezbeđuje stabilnost zidova kako u samom procesu iskopa, tako i u fazama koje slede kasnije (spuštanje armaturnog koša i betoniranje). Rastvor bentonita kojim se zapunjava rov treba da obezbedi potrebni hidrostatički protivpritisak, koji u svakoj tački zida iskopa mora biti veći od aktivnog pritiska okolnog tla i podzemne vode

Relativna viskoznost rastvora bentonita predstavlja vreme (u sekundama) za koje istekne 500 ml rastvora kroz otvor viskozimetra. Zahtevi u pogledu viskoznosti rastvora su protivrečni. Za smanjenje otpora na grejfer mašine za iskop bolje je da je viskoznost manja, a da bi se sprečilo sleganje čestica iskopanog tla na dno rova bolje je da je veća. U praksi se pokazalo da je najoptimalnija relativna viskoznost rastvora bentonita od 20-25 sekundi.

Gubitak vode je veoma važna osobina rastvora bentonita I predstavlja sposobnost predavanja vode vodopropustljivom tlu oko zidova rova. Gubitak vode se meri pod pritiskom od 0,1 Mpa, na površini od 100 cm², u toku 30 minuta. Gubitak vode ne bi trebalo da je veći od 25-30 ml.

Važna je i stabilnost rastvora bentonita, a to je njegova osobina da ostane u mirnom stanju bez raslojavanja. Određuje se pomoću posude zapremine 800 ml koja se puni rastvorom bentonita, pa se 24 sata posle punjenja određuje razlika zapremine težine gornjih i donjih slojeva. Ova veličina ne sme biti veća od 0,02 grama/cm³.

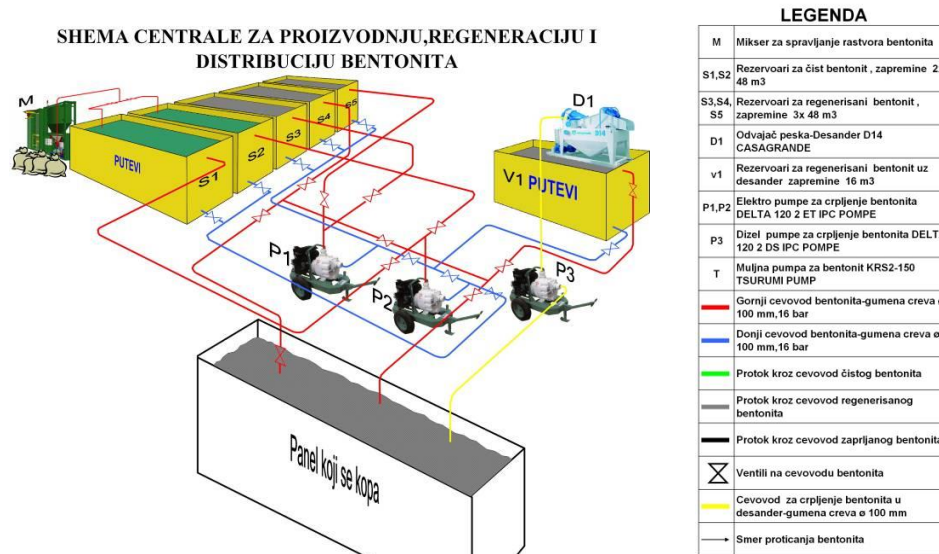
Sadržaj peska izražava se u procentima zapremine peska u odnosu na zapreminu rastvora bentonita i dozvoljeno je max. 4%.

Za spravljanje rastvora bentonita se koristi glineni prašak koji treba da ima sledeće karakteristike:

- Zapreminska težina od 2,7-2,75 g/cm³
- Bubenje, najmanje 15-20%
- Granulometrijski sastav zrna u intervalu od 1,0-0,05 mm

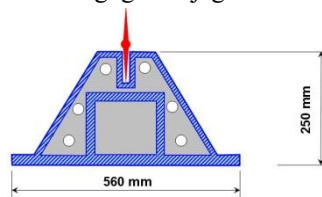
PH faktor rastvora bentonita treba da bude od 8-11,5.

Za spravljanje, regeneraciju(prečišćavanje) i distribuciju rastvora bentonita na gradilištu je potrebno imati centralu za bentonit.



6 FORMIRANJE SPOJEVA PANELO DIJAFRAGME METODOM SOLETANCHE BACHY

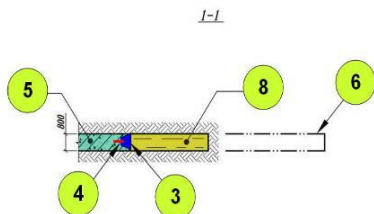
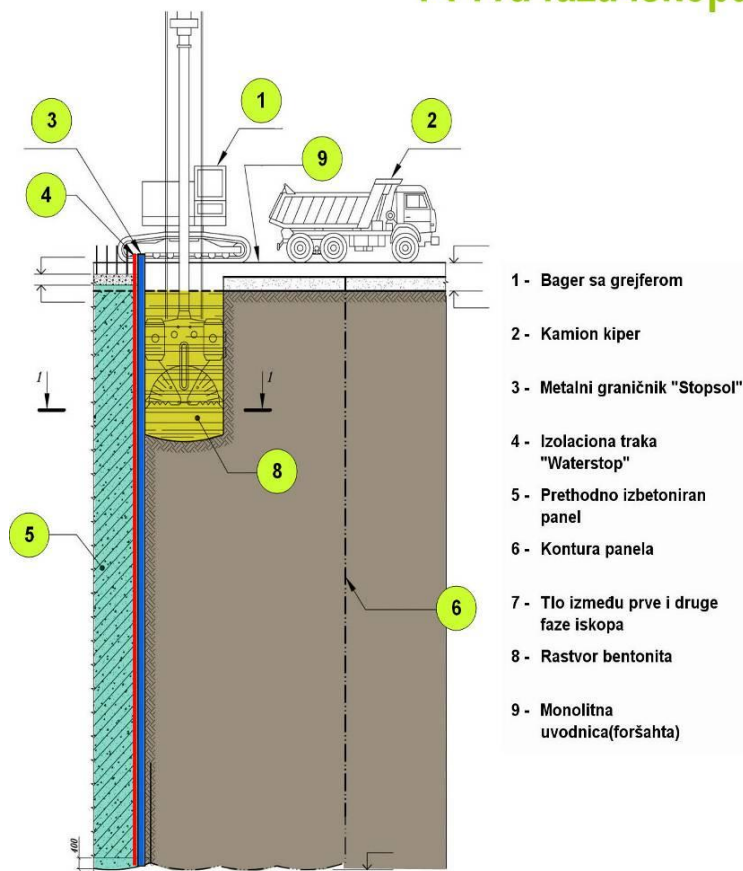
Ova metoda je nastala kao rezultat dugogodišnjeg iskustva Francuske firme Solitanche.



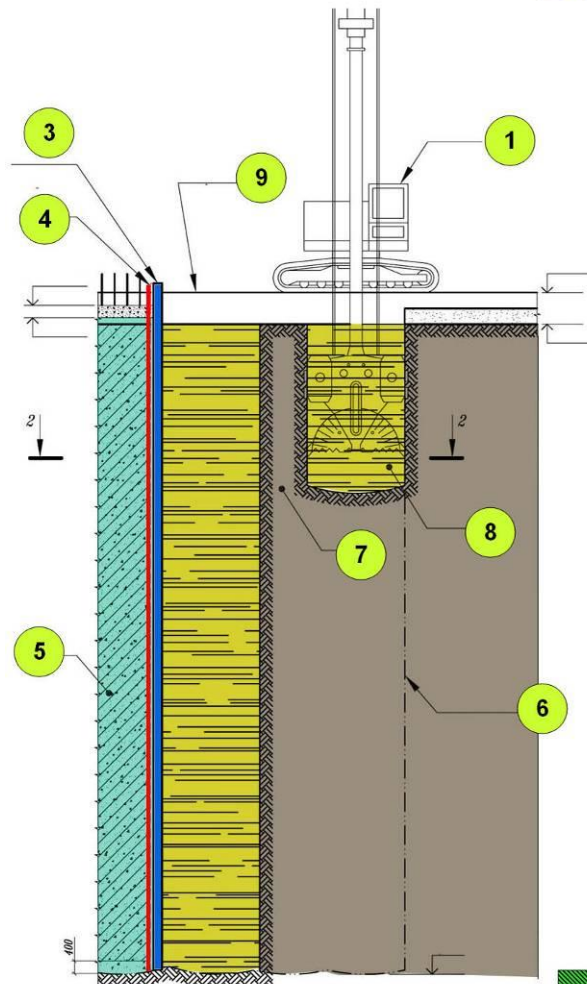
Spoj se izvodi pomoću metalnog graničnika koji je prikazan na skici iznad. U graničnik se postavlja WATERSTOP traka koja je na skici obojena u crveno. U priložima koji slede na listu koji nosi naslov Prva faza iskopa, waterstop traka je jednim delom zabetonirana u prethodno izvedenom panelu a drugim delom je obuhvaćena metalnim graničnikom. I metalni graničnik (" STOPSOL " traka) i " WATERSTOP " traka ostaju u tom položaju sve do potpunog završetka iskopa za razmatrani panel dijafragme. Stopsol graničnik se od betona prethodno

izvedenog panela odvaja pomoću specijalnog alata koji se montira na grejfer bagera za iskop. Odvajanje se obavlja postupno, odozgo na dole da se ne ošteti waterstop traka. Zatim se graničnik podiže pomoću dizalice, stavlja se u njega nova waterstop traka i vrši spuštanje na ivicu iskopa uz tle. Tek posle se ubacuje armaturni koš, a zatim vrši betoniranje.

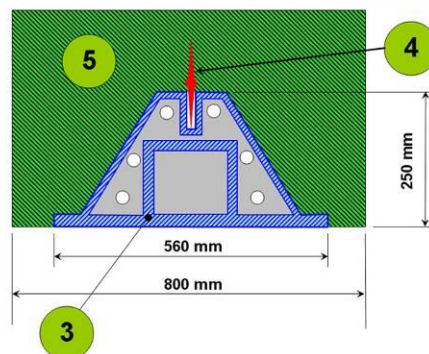
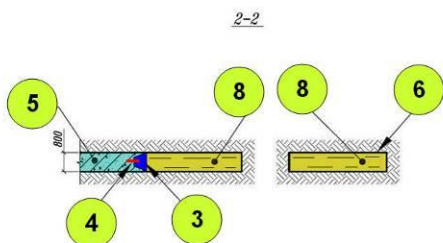
1-Prva faza iskopa



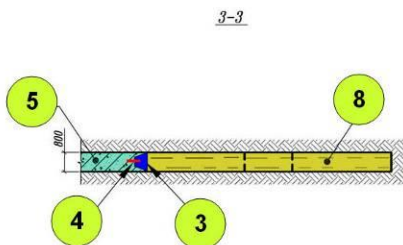
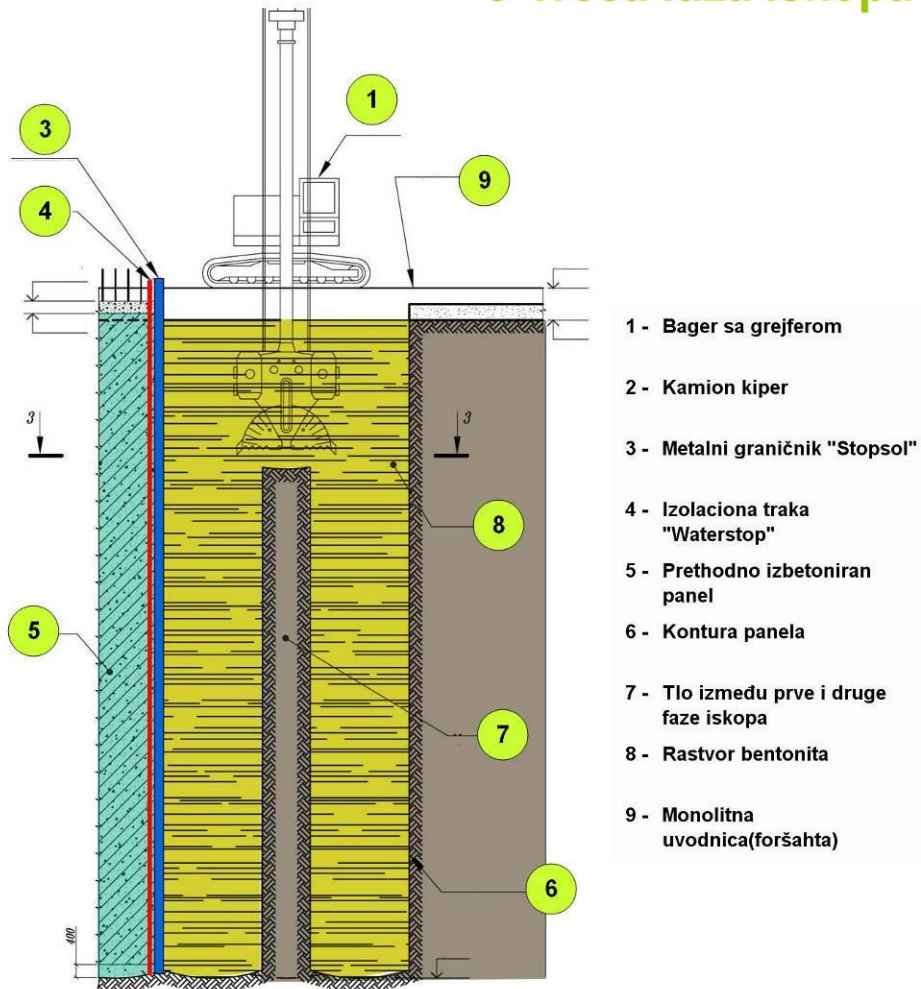
2-Druga faza iskopa



- 1 - Bager sa grejferom
- 2 - Kamion kiper
- 3 - Metalni graničnik "Stopsol"
- 4 - Izolaciona traka "Waterstop"
- 5 - Prethodno izbetoniran panel
- 6 - Kontura panela
- 7 - Tlo između prve i druge faze iskopa
- 8 - Rastvor bentonita
- 9 - Monolitna uvodnica(foršahta)

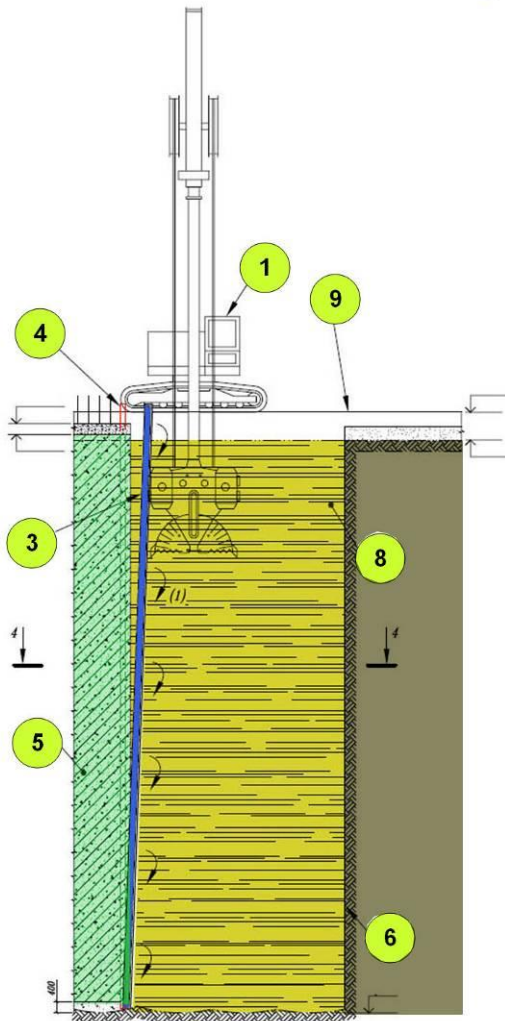


3-Treća faza iskopa

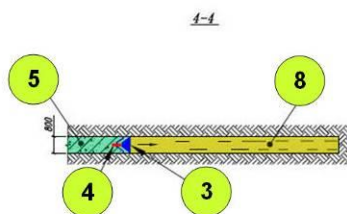


Fotografija 1: Izolaciona traka WATERSTOP

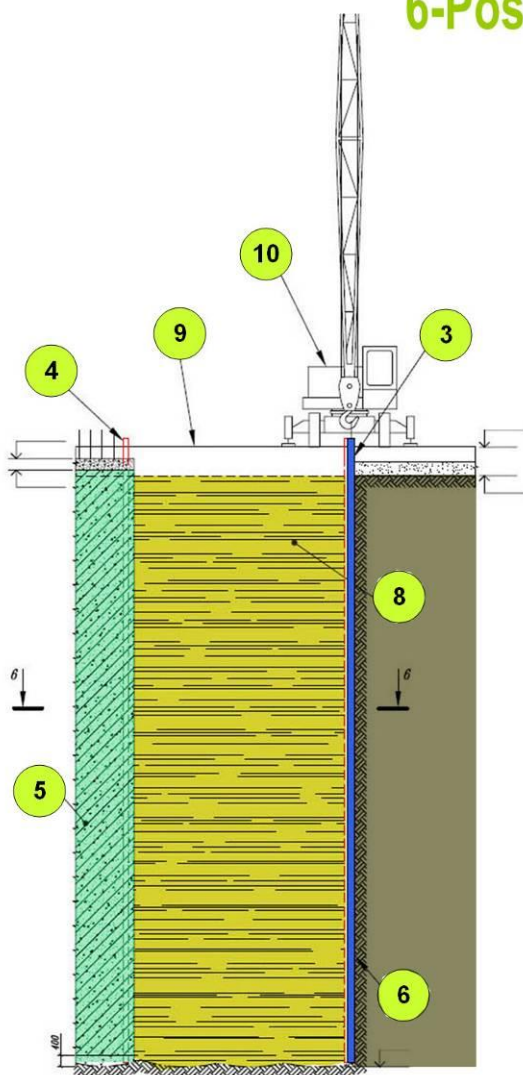
4-Odvajanje graničnika



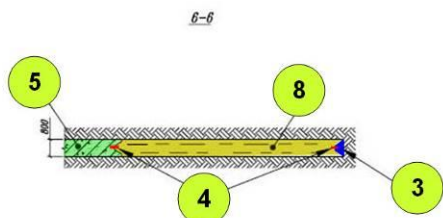
- 1 - Bager sa grejferom
- 3 - Metalni graničnik "Stopsol"
- 4 - Izolaciona traka "Waterstop"
- 5 - Prethodno izbetoniran panel
- 6 - Kontura panela
- 7 - Tlo između prve i druge faze iskopa
- 8 - Rastvor bentonita
- 9 - Monolitna uvodnica (foršahta)
- 10 - Autodizalica



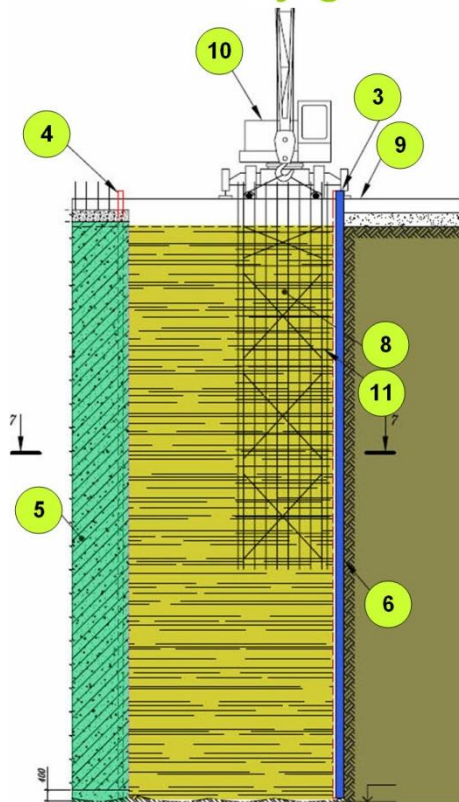
6-Postavljanje graničnika



- 1 - Bager sa grejferom
- 3 - Metalni graničnik "Stopsol"
- 4 - Izolaciona traka "Waterstop"
- 5 - Prethodno izbetoniran panel
- 6 - Kontura panela
- 7 - Tlo između prve i druge faze iskopa
- 8 - Rastvor bentonita
- 9 - Monolitna uvodnica (foršahta)
- 10 - Autodizalica



7-Montaža donjeg dela armaturnog koša i njegovo kačenje na uvodnicu



3 - Metalni graničnik "Stopsol"

4 - Izolaciona traka "Waterstop"

5 - Prethodno izbetoniran panel

6 - Kontura panela

8 - Rastvor bentonita

9 - Monolitna uvodnica(foršahta)

10 - Autodizalica

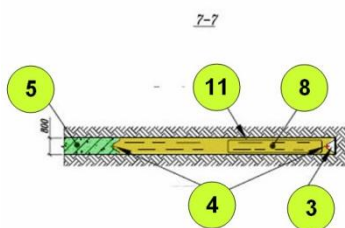
11 - Donji deo armaturnog koša

12 - Gornji deo armaturnog koša

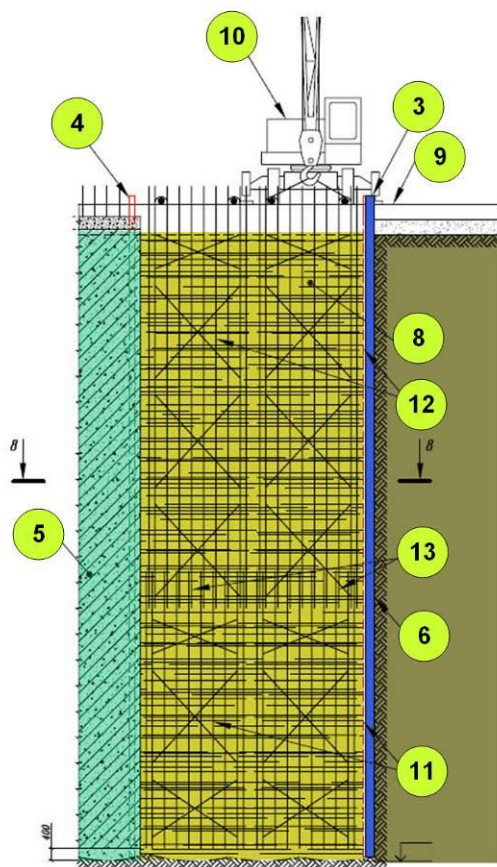
13 - Spoj armaturnih koseva

14 - Mikser

15 - Cev za kontinualno betoniranje



8-Montaža gornjeg dela armaturnog koša i njegovo kačenje na uvodnicu



3 - Metalni graničnik "Stopsol"

4 - Izolaciona traka "Waterstop"

5 - Prethodno izbetoniran panel

6 - Kontura panela

8 - Rastvor bentonita

9 - Monolitna uvodnica(foršahta)

10 - Autodizalica

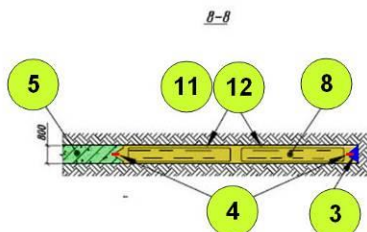
11 - Dornji deo armaturnog koša

12 - Gornji deo armaturnog koša

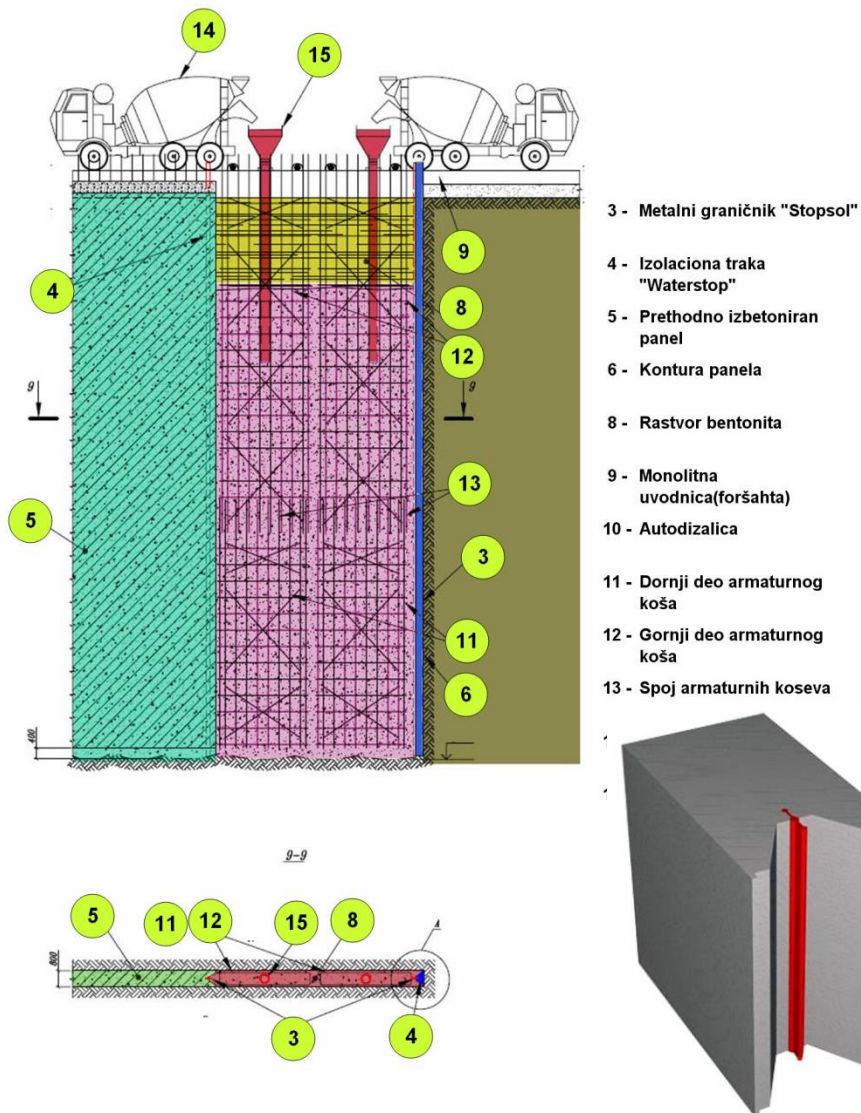
13 - Spoj armaturnih koseva

14 - Mikser

15 - Cev za kontinualno betoniranje



9-Betoniramje panela



Na gornjoj skici je prikazan izgled prethodno izbetoniranog panela dijafragme posle vađenja metalnog graničnika STOPSOL.



Miloš Debeljković¹

PROJEKAT POSTROJENJA ZA REGENERACIJU ISTROŠENE SUMPORNE KISELINE U RN PANČEVO (NIS)

Rezime:

S-54

U rafineriji nafte u Pančevu 2012. godine pušteno je u rad postrojenje za regeneraciju istrošene sumporne kiseline (SARU) koje predstavlja deo novih postrojenja za „Blagi hidrokreking“ i „Hidro doradu“ pravljenih u okviru projekta modernizacije rafinerije. Postrojenje SARU predstavlja "know-how" češke kompanije "TKB Kovoprojekta" iz Brna. Građevinske projekte su izradili srpski projektanti. Čitavo postrojenje su, uz nadzor čeških inženjera, izgradila srpska preduzeća. U ovom izlaganju biće prikazana noseća čelična konstrukcija objekta SK1 i pripadajući temelji.

Ključne reči: NIS, Rafinerija, modernizacija, sumporna kiselina, SARU

DESIGN PROJECT OF SPENT ACID REGENERATION UNIT (SARU) IN THE OIL REFINERY PANČEVO (NIS)

Summary:

In 2012, within the Oil Refineru Pancevo, the Spent Acid Regeneration Unit (SARU) was commissioned. This Unit is the part of the new MHC/DHT complex, which is the integral part of the modernisation project within the OR Pancevo. Namely, SARU unit is the "know-how" project of the Czech Company "TKB Kovoprojekta" from Brno. Civil and Structural Design was done by Serbian Designers. The whole Unit was built also by Serbian Companies under supervision of Czech Engineers. This paper will present the suporting Steel Structure for the Building SK1, as well as, the concrete foundations.

Key words: NIS, Refinery, modernization, Sulphuric Acid, SARU

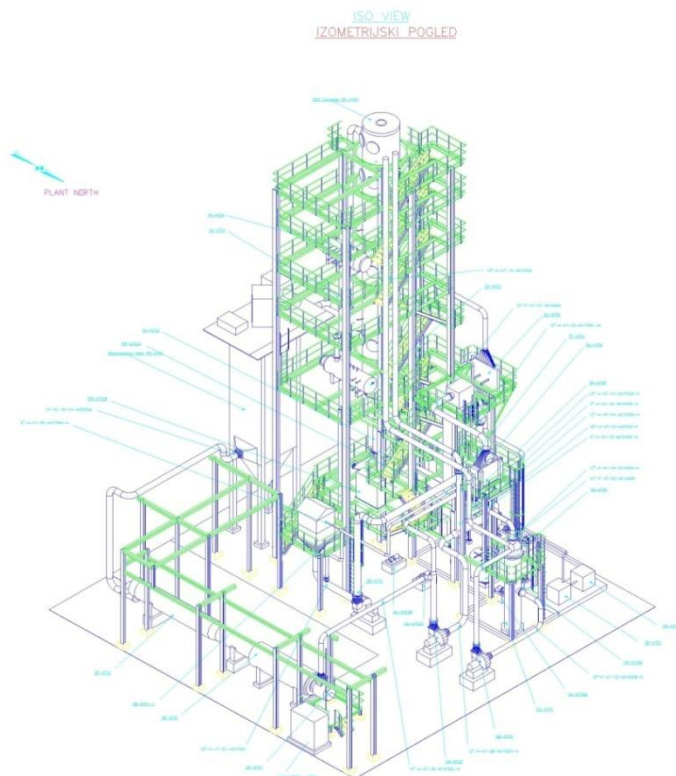
¹ dipl. inž. građ, savetnik direktora, Energoprojekt Industrija a.d.

1 UVOD

Tokom 2008 godine, pokrenut je proces modernizacije Rafinerije nafte u Pančevu. Kao deo kompleksa za blagi hidrokreking i hidro doradu (Mild Hydrocracking and Hydrotreatment – MHC/DHT), izgrađeno je postrojenje za regeneraciju istrošene sumporne kiseline (S 4700 - SARU). Vrednost ovog postrojenja iznosila je oko **14,3 miliona evra**.

Namena kompleksa MHC/DHT je povećanje dubine prerade nafte, odstranjivanje maksimalne količine sumpora, azota i ostalih hemijskih jedinjenja koja negativno utiču na kvalitet gotovog proizvoda – goriva.

Postrojenje SARU ima veliki značaj u zaštiti životne sredine. Ovo postrojenje koristi tehnologiju "bez otpada" (waste-free technology). Štaviše, potrošena sumporna kiselina, koja se smatra za opasan otpad, kompletno se rafiniše i ponovo koristi u procesima kao koncentrovana kiselina. Ono predstavlja "know-how" projekat češke kompanije "TKB Kovoprojekta" iz Brna. Tehnološki proces je patent danske firme "Haldor Topsoe", sve ostalo je češko. Građevinske projekte su izradili projektanti Ljubica Čupić i Dragan Mikić, dipl. inž. građ. Uz nadzor i učešće čeških inženjera, postrojenje je završeno i svečano pušteno u rad 2012. godine.



Slika 1 – Dispozicija postrojenja SARU

2 DISPOZICIJA POSTOJENJA - LAYOUT

Od građevinskih objekata na lokaciji postrojenja najznačajnija i najkompleksnija je zgrada izrađena u čeličnoj konstrukciji (oznaka SK1), na kojoj se nalazi oprema od koje je najvažniji SO₂ konverter (oznaka DC-4701). Ispod zgrade je na ploči formirana tankvana sa izdignutim zidom od 20 cm, popločana kiselo otpornim pločicama radi prikupljanja tečnosti pri akcidentima.

Postoji i nekolicina cevnih mostova, visine do 5 metara za nošenje cevi sa fluidima. Pored elektrostatičkog filtera PB-4701 (ESP) koji je postavljen na sopstvenoj nosećoj konstrukciji, nalazi se ESP zidana prizemna zgrada malih dimenzija (3,75*7,75 m), koja služi za skladištenje pomoćne opreme za ESP. Uređaj za odlaganje čvrstog otpada za ESP nalazi se ispod samog filtera.

Građevinske konstrukcije kao i samostojeca oprema oslonjeni su na temelje samce koji su ili plitko fundirani ili su fundirani na šipovima na terenu koji nije prirodan, već je novonasut i nabijen do zahtevane stišljivosti. Proračun temelja je urađen prema "Izveštaju o geotehničkim ispitivanjima tla u RN Pančevo", izrađenom od strane IMS-a, decembra 2005. godine.

Plato, koji je formiran od dvostruko armirane betonske ploče, debljine 20 cm, kvaliteta MB40 V8, ima padirane površine sa slivnicima za odvođenje atmosferskih padavina u novoprognovanu kanalizacionu mrežu prema daljoj (postojećoj) mreži rafinerije.

3 ČELIČNA KONSTRUKCIJA ZGRADE SK1

Noseća čelična konstrukcija projektovana je kao ramovska konstrukcija sa vertikalnim spregovima za ukrućenje. Gabarit zgrade je 8,700*5,700 + 3,475 m sa aneksom 4,000*2,400 m. Visina objekta je 26,250 m (kota završne platforme).

Od kote temelja podiže se 10 glavnih stubova i 4 stuba aneksa. Objekat je horizontalno podeljen sa platformama u nivoima rigli ramovskih konstrukcija. Platforme sa gazištima od "giter rosta" i čeličnim nosačima u oba pravca, tretirane su kao krute figure u svojoj ravni za horizontalne sile. Na platformama +3 m, +8 m, +17,9 m i +26,65 m projektovani su horizontalni spregovi koji obezbeđuju krutost konstrukcije u montaži i kasnije u eksploataciji. Na koti 8,0 m van granice platformi montirane se rigle koje povezuju stubove izvan platformi radi smanjenja dužine izvijanja tih stubova.

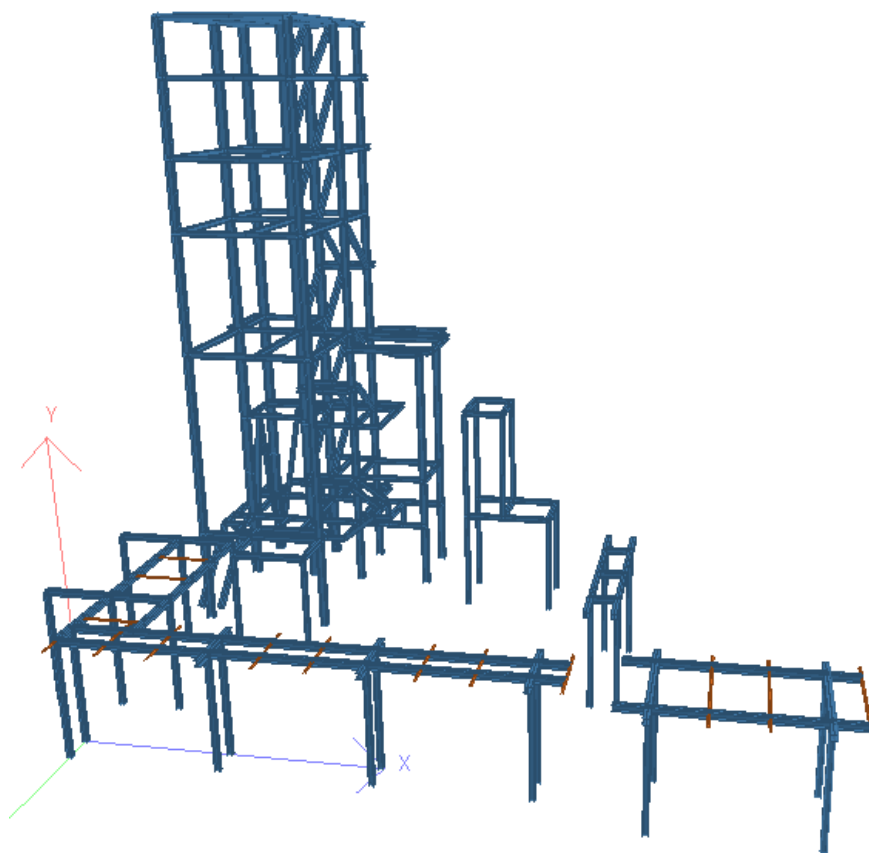
Stubovi konstrukcije su uglavnom HEA320 valjani profili, u dve ose su HEA240. Ragle ramova su uglavnom HEA160. Sekundarni elementi konstrukcije projektovani su sa profilima HEA240 (elementi ispod opreme sa značajnom težinom), HEA160 i ivični nosači na konzolnim prepustima IPE180. Stepence su projektovane sa U180 profilima koji su međusobno spojeni elementima gazišta te su ukrućeni u horizontalnoj ravni.

Računska analiza konstrukcije izvršena je u prostornom modelu (program STAAD). Statičkim proračunom obuhvaćena su stalna i povremena dejstva na konstrukciju:

- sopstvena težina konstrukcije,
- težina opreme instalirane na konstrukciji,
- korisno opterećenje na platformama 3,0 kN/m²
- korisno opterećenje na platformama od cevi i instalacija 2,50 kN/m²
- sneg 1,14 kN/m² (prema projektnom zadatku),
- promena temperature ±35 °C,
- dejstvo vetra,

- seizmičko dejstvo.

Dejstvo vetra razmatrano je za osnovnu brzinu vetra , dok je za seizmičke uticaje tretirana VIII zona za povratni period $T=500$ godina. Proračun seizmičkog dejstva sproveden je na istom modelu konstrukcije kao i za dejstvo vetra. Proračun je sproveden za mase od stalnog, eksploatacionog i polovinu korisnog opterećenja. Participacija masa po stubovima izvršena je prema zadatim gravitacionim uticajima. Razmatrano je seizmičko dejstvo u dva ortogonalna pravca.

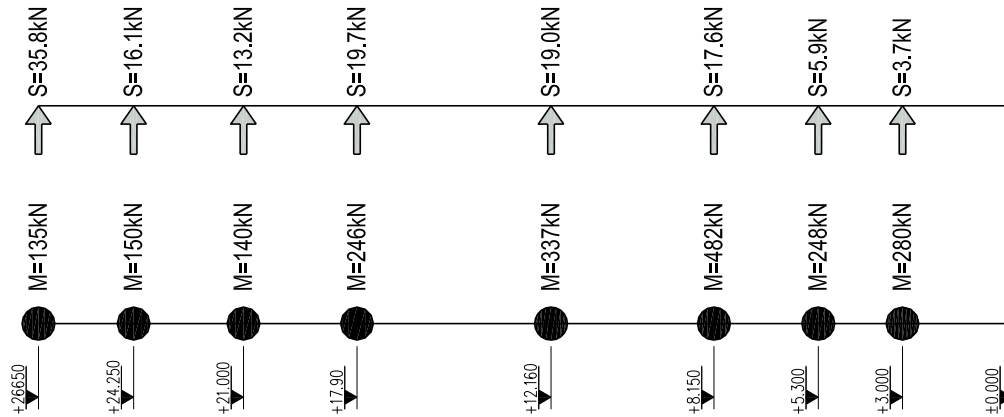


Slika 2 – prvobitni prostorni model konstrukcije (SK1 i ostali objekti)

Prvobitna zamisao mašinaca (tehnologa) je bila da kompletna konstrukcija bude ramovska (vertikalni ramovi u oba pravca), bez vertikalnih spregova zbog lakšeg prolaska mnogobrojnih cevi i kanala kroz konstrukciju.

Tokom izrade projekta, zbog konstantnog dodavanja novih elemenata opreme i novih opterećenja na horizontalne platforme, konstrukcija je morala biti ukrućena sa vertikalnim spregovima u oba pravca, čime je zakomplikovan posao projektantima mašinskog projekta, ali je i to uspešno rešeno. Postignut je optimalan odnos između količine potrebnog čelika za konstrukciju i upotrebljivosti objekta. Maksimalna proračunska horizontalna pomeranja

konstrukcije na vrhu stuba iznose oko **20 mm**. Količina upotrebljenog čelika za zgradu (sa aneksom) iznosi **85,50 tona**, što daje potrošnju od **64 kg/m³**, što je u granicama prihvatljivog.



Slika 3 – raspored masa i horizontalnih seizmičkih sila po visini objekta

Elementi konstrukcije izrađeni su od čelika S235JRG2, vijčane veza su ostvarene vijcima klase 8.8, bez sile prednaprežanja. Antikorozivna zaštita konstrukcije je, prema eksplicitnom zahtevu Investitora, za razliku od svih drugih čeličnih konstrukcija u Rafineriji, bila izvedena sistemima boja za kategoriju korozivnosti C5 - M (zbog mogućih akcidentnih situacija), prema SRPS ISO 12944.

4 TEMELJENJE

Na osnovu pomenutog geomehaničkog elaborata IMS-a, donet je zaključak da čitav plato za postrojenje SARU treba iskopati do minimalne dubine zamene tla (-2,0 m), pa potom čitavu površinu nasuti peskovito/šljunkovitim materijalom ukupne debljine 50 cm. Tako se dobila kota platoa na koti -1,50 m sa koje je vršeno fundiranje svih objekata, bili oni plitko ili duboko fundirani.

Zgrada SK1 se preko betonskih postolja oslanja na armirano betonsku ploču. Dimenzije ploče su 5,20*6,70 m, debljine 0,65 m. Ploča je formirana na koti pripremljenog temeljnog dna -1,45 m, na sloju mršavog betona od 5 cm. Postolja glavnih stubova u osama 1,2, A, B i C, sa već ugrađenim potrebnim ankerima, završavaju se na koti +0,20 m, pri čemu je urađeno podlivanja ležišnih ploča sa 3,0 cm podlivke.

AB ploča je fundirana na bušenim CFA Ø400 mm šipovima, dužine 10,0 metara. Šipova ima ukupno 16 (ispod ploče).

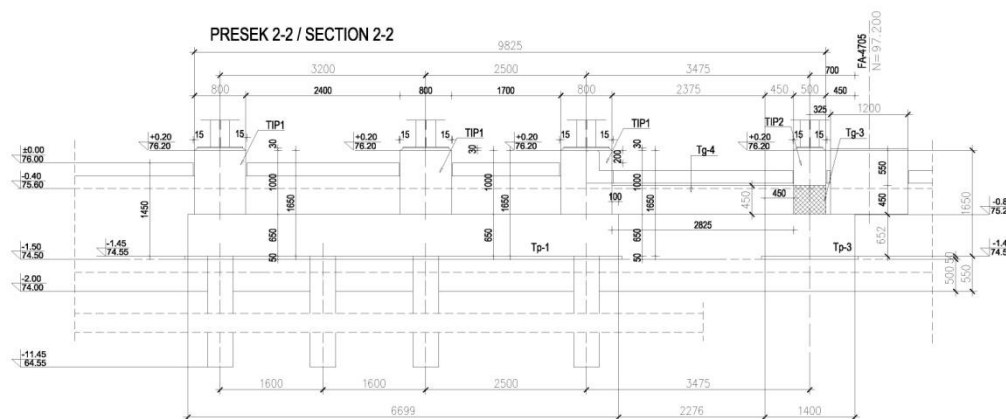
Za bušene CFA šipove (continuous flight augering) koji su opterećeni horizontalnim opterećenjem, nosivost zavisi od vrste zemljišta i njegovih geotehničkih osobina, kao i od vrste betona i procenta armiranja šipa. Za geotehničke uslove zemljišta u kom je izvršeno fundiranje, kao i unapred definisanom armaturom (6 komada ø22 mm, zatežući napon 450 MPa (RA 400/500-2)), uz maksimalnu deformaciju vrha šipa od 10-15 mm, maksimalno horizontalno opterećenje je sračunato da bude:

Tip i prečnik šipa	pritisak		zatezanje	
	Maks. hor. opt. - faktorisano [kN]	Maks. hor. opt. - nefaktorisano [kN]	Maks. hor. opt. - faktorisano [kN]	Maks. hor. opt. - nefaktorisano [kN]
CFA- 400 mm	80	65	74	62

Statičkim proračunom dokazano je da su i vertikalne i horizontalne sile u šipovima manje od dozvoljenih.

Za oslanjanje ostale čelične konstrukcije - stubova u osama 3 i D, usvojeni su temelji samci, dimenzija 1,50*2,20 m i 1,20*1,40 m, povezani sa temeljnom pločom i međusobno temeljnim gredama b/d=50/45 cm.

Za temelje i šipove korišćen je beton MB 30 i armatura RA 400/500-2.



Slika 4 – poprečni presek kroz temeljnu konstrukciju

5 IZVOĐENJE RADOVA I MONTAŽA

Građevinski radovi su započeti u proleće 2011. godine. Posle izrade prvog (probnog) šipa, šip je testiran na ispitno (vertikalno) opterećenje, po dokazivanju nosivosti nastavljena je izrada ostalih šipova. Čelična konstrukcija je izrađena u Žitorađi, gde je i AK zaštićena. Nažalost, kvalitet AK zaštite nije bio zadovoljavajući, tako da je nova AK zaštita morala biti nanošena na licu mesta. Paralelno se montirala noseća konstrukcija i oprema. Radovi su završeni u roku. Odgovorni izvođač građevinskih radova je bio Miroslav Kopunović, dipl. inž. građ.

Donco Partov¹, Radan Ivanov², Dobromir Dinev³

RAZVOЈ ČELIČNIH ORTOTROPNIH MOSTOVA U BUGARSKOЈ

Rezime:

U ovom radu autori prikazuju koncept projektovanja i tehnologiju izgradnje sedam čeličnih ortotropnih mostova u Bugarskoј. Predložena konstrukcija je opisana do detalja, uključujući opterećenja, detaljan projekat ortotropnih ploča, sve glavne i sekundarne nosače.

Ključne reči: čelični nosači, ortotropne ploče, sandučasti preseци

DEVELOPMENT OF STEEL ORTHOTROPIC BRIDGES IN BULGARIA

Summary:

In this paper the authors present the conceptual design and technology of construction of seven steel orthotropic bridge in Bulgaria. The particular structure proposed is described in detail, including structural action and detailed design of the orthotropic bridge deck, all principal and secondary members.

Key words: steel bridges, orthotropic plate, box girder

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1 INTRODUCTION

Bulgarian bridge construction enters a new phase of development as a result of the decision of the expert council of the Ministry of Transport of 22 May 1968, which gives the green light for the use of steel structures in bridge construction in the country. This reasonable act gives the opportunity to announce several competitions for bridging large obstacles, which in that period of development of reinforced concrete technology could not be realized. Some of these steel bridges were built with orthotropic steel deck, whose design, research and development will be the subject of this article. The article will be described seven steel bridges with orthotropic steel deck, arranged in chronological order, depending on the year of their construction and opening for traffic.

2 DESCRIPTION OF THE DESIGN DECISION OF 7 STEEL BRIDGES WITH ORTHOTROPIC DECK

2.1 “ASPARUHOV” BRIDGE

With the growth of the industrial importance of the city Varna in the early 70s and the construction of the port of Varna - west there is a need to create a more - convenient waterway there. The old canal Black See - Lake can not take no expected ship traffic, which requires the digging of a new -Big waterway. However, this in turn creates problems to solve conflict-free crossing of the shipping channel in the future with transport - the communication system of the city in the direction Asparuhovo - Burgas. In 1967-68, the competition was organized by the technical solution of the transport problem of high trestle crossing over the canal was taken after studies of all proposed options: a tunnel under the canal bridge Clapps, crawl Varna Lake and ferry. The competition at the conceptual stage to a jury of prominent Bulgarian public figures and experts, under the leading of world famous Bridge engineers from Faculty in Prague prof. F. Faltus and prof. J. Klimeš, evaluated as best decision the proposal for large steel bridge construction with a total length of 2200 meters and spans ranging between 100 and 200 meters, developed by a team with tutor Dr. Eng. B Bankov (1912-1992). At that time such a decision in acute deficit of large quantities of high-strength steel construction was almost impossible to realize. For this reason, working phase bridge is designed as a combination of four different structural and compositional relation parts: reinforced lower part, called elongation at North Node consisting of 15 sections with 21.50 meters; two concrete trestle parts: from Varna -25 sections with 40,20 m and in direction to Burgas 10 section with 40,20 meters, and a central part from steel structure(Fig.1). The reinforced concrete part is designed by “Patproject” company with a head designer B. Stoyanov; and the new steel part is designed by M. Braynov, D. Dimitrov, P. Staykov, and K. Tsekov. The bridge owner is the Agency “Road Infrastructure”. The bridge owner is the Agency “Road Infrastructure”. The “Asparuhov” bridge was opened to traffic in 1976. The bridge consists of two parallel parts which rise up to 50.00 m above the canal. The total length of the bridge is 2050.00 m and includes 39 spans of 40.20 m of precast prestressed concrete T-beams of 2.4 m deep and 3 spans of 80.50 + 160.00 + 80.50 m steel structure. The 18 tendons of high strength steel St 150/170 with a diameter of 24Ø5 mm are used for the prestressing. The top flange of the beams is 2.20 m wide; the web thickness is 0.20 m. Each of the spans has three main beams spaced at

3.50 m. The total width of the bridge deck is 21.00 m. The expansion joints are placed at every 5 spans. The concrete used for main beams is C40/45. The steel part of the bridge passes over the ship canal. It is a 3-span continuous welded orthotropic box beam. The box section is 5.50 m wide and has a varying depth from 2.80 m to 6.60 m. The deck plate has a varying thickness from 12 mm to 20 mm. The longitudinal ribs have cross-section dimensions of 12x200 mm and a distance between them of 300 mm; the transversal beams are “T”-shaped fabricated sections with web dimensions of 500x8 mm and flange dimensions of 160x10 mm. The stability problems of the box-section are solved by adding of “V”-shaped braces placed at every 4.00 m. The connections between the assembly units are by high strength bolts. The pier bearings are steel rollers produced by “Creutz” company. The steel used for fabrication of the superstructure is St52 ($f_y=330$ MPa). The bridge piers are two-column hammerhead bents. The pier foundation is set on very complex soil conditions. The columns are supported on pile foundation which consists of 6 cast-in-place concrete piles with a diameter of 1.20 m and up to 53.00 m deep. The bridge superstructure was constructed using a semi-cantilever launching method [1].



Fig. 1. “Asparuhov” bridge.

2.2 “ELIN PELIN” BRIDGE

The “ElinPelin” bridge is an overpass structure located in Elin Pelin as shown in Figure 2. It carries the traffic of a second class road 165 from Yordankino to Novi Han and crosses over the railway of the local railway station. The bridge was designed and built by “Gosha” company and supervised by B. Bankov. The bridge owner is the Municipality of ElinPelin. The overpass was opened to traffic in 1981. The overpass is a steel orthotropic plate girder structure, Gerber system, with a total length of 300 m and includes 15 spans of different lengths. The largest span has a length of 30.00 m; the rest of the spans vary from 18.766 to 18.793 m. The superstructure is 7.00 m wide and consists of four main girders. The deck plate is 12 mm thick and stiffened with a longitudinal ribs and transversal beams. The longitudinal stiffeners are cold formed trapezoidal sections fabricated by a 6 mm thick metal sheet. The main girders of the central span are 1.10 m deep. The bottom flange has dimensions of 28x350 mm and is reinforced at the middle of the span by an additional 28x320 mm cover plate. The

girders web is 10 mm thick. The rest of the spans have girders with same dimensions as above, except of the bottom flange which dimensions are 20x350 mm. At 400 mm away from the bridge piers there are 100 mm diameter openings for the hinges of the Gerber system. The transversal beams with overhangs are 2.85 m long, 600 mm deep and are spaced at 3.00 m. The flanges have dimensions of 10x100 mm. The overhangs have a varying depth. The road part of the deck in an each span consists of two symmetrical assembly units each 3.00 m wide. They are connected to each other by 24 mm diameter high strength bolts. Two pedestrian lanes (2.35 and 1.50 m wide) are attached to the road parts. Each pedestrian lane is designed as an orthotropic slab with 10 mm thick top plate, and stiffeners with cross-section of 6x150 mm, spaced at 435 mm. The pedestrian units are connected to the road units by bolts. The piers are steel single-column hammerhead bents. The columns for the part of bridge with spans 18.766 m are designed as steel "I"-sections. The columns for the span 30.00 m are designed as steel box-sections. The steel base plates of the columns have dimensions of 20x1600x2600 mm for the box-section columns and 20x740x2500 mm for the "I"-section columns. 18 anchor bolts BM 36 are used to connect the base plates to the footings. The cap beam is a double-armed cantilever with a varying depth which is connected to the column by bolts. The transversal beams and the stiffeners of the orthotropic deck are welded to the steel plate. The orthotropic deck units and secondary beams are made from steel St37-2 and the main girders and piers are made from steel St52-3[1].



Fig.2 "ElinPelin" bridge.

2.3 “BRUSSELS BLVD” STEEL BRIDGE

The steel bridge is a part of the “Brussels blvd” overpass and serves as a connection between the Sofia Airport and the entrance highway of the city (Fig.). The bridge is placed over the railway Sofia-Istanbul. The designer is B. Bankov, the owner is the Municipality of Sofia. The overpass was opened to traffic in 1983. The overpass consists of two parallel structures separated with a gap of 1.56 m. The superstructure is a steel orthotropic 3-cell box girder of, 90.00 m long and includes two spans of 45.00 m. The bridge is placed in a horizontal curve with a radius of 1000.00 m. The box girder is 1.30 m deep and the total width of 21.75 m. These dimensions were required and limited by aesthetic considerations, because the steel part should fully follow the silhouette of the concrete part with the same depth. In longitudinal direction the box walls are connected to each other by upper and lower transversal steel beams at intervals of 1.493 m. The upper transversal steel beams are connected at the top by a 12 mm thick steel plate. The lower transversal beams are connected at the bottom by a 10 mm thick steel plate. The steel orthotropic deck plate is stiffened with longitudinal flat ribs: - 12x150 mm with a distance between them of 313 mm. The upper transversal beams have “T” cross-sections with web dimensions of 8x300 mm and flange dimensions of 10x120 mm. The lower beams have “T” cross-sections with web dimensions of 8x200 mm and flange dimensions of 10x100 mm. The box walls are 12 mm thick and stiffened by vertical ribs of 10 x 100 mm spaced at 1.493 m. The lateral bracings are placed at intervals of 7.465 m and at supports regions. The bridge end piers are “V”-shaped bents with inclined columns. The columns are triple-box cross-sections with wall thickness of 12 mm. The columns are rigidly connected to the deck and have hinged connections to the footings. The central pier is a concrete hammerhead bent [1].



Fig. 3 “Brussels blvd” steel bridge.

2.4 VIADUCT 1 “HEMUS” HIGHWAY, 26.884 KM

A viaduct 1 of the “Hemus” highway is a steel continuous orthotropic plate girder structure (Fig. 4.). The bridge designers are D. Dimitrov, P. Staykov, and K. Tsekov. It was produced in “Kremikovtsy” company and constructed by a company “Steel structures” using an incremental launching method by roller chains. It was opened to traffic in 1986. The bridge is 410 m long including 6 spans of 60 m + 4x72.5 m + 60 m. The superstructure consists of two parallel structures with a gap between them of 0.80 m. Each of the structures carries a 13.7 m roadway and consists of two steel main girders 3.6 m high spaced at 9.2 m. The web is stiffened by hot-rolled channel sections 160 mm deep. The main girders are joined together by braces and transversal beams with overhangs, places at distance of 3.15 m from each other. An additional longitudinal beam placed at center of span connects the transversal beams. The bottom flanges of the main girders are connected by lateral X-braces. The orthotropic deck of 12 mm is stiffened by longitudinal trapezoidal ribs. The viaduct is placed at a horizontal curve with a radius of 850 m and a longitudinal slope of 4.61 %. The transversal slopes of each of the roadways are 4.5 %. The intermediate supports of the structure are reinforced concrete piers with a maximum height of 28.0 m. The piers have a rectangular cross-section and individual footings. The steel used for the producing of the bridge superstructure is St09 G2B according to BDS with yielding strength equal to 330 MPa. The total weight of the superstructure is 21850 kN. Segments of 12.85 m of orthotropic plate girders were welded prefabricated and assembled each other by using high strength bolts. The bridge structure is supported on the abutments by steel roller supports, type “Creutz” and the pier supports – by the elastomeric bearing pads produced by “Gumba” GmbH [1].



Fig.4 Viaduct 1 “Hemus” highway, 26.884 km. (Courtesy of prof. M. Minev and prof. P. Staykov. With permission.)

2.5 VIADUCT 2 “HEMUS” HIGHWAY, 27.500KM

A viaduct 2 of the “Hemus” highway is a steel continuous orthotropic box girder structure, crossing the valley 60 m above the ground as shown in Figure 3.6. The bridge designers are D. Dimitrov, P. Staykov, S. and K. Tsekov. It was constructed by a company “Steel structures” using an incremental launching method with hydraulic jacks and opened to traffic in 1986. The bridge is 362 m long including spans of 100 m + 162 m + 100 m. The main span of 162 m is the largest span in Bulgaria. The superstructure consists of two parallel steel boxes with a height of 5.0 m and a width of 6.50 m. The gap between the structures is 0.8 m. The viaduct is placed at a horizontal curve with a radius of 1407.25 m and has 4.61 % of a longitudinal slope. The total width each of the roadway is 13.70 m. The top and the bottom plates of the boxes are orthotropic structures stiffened by flat ribs. The thicknesses of the top plate vary from 12 to 16 mm; the webs – from 10 to 14 mm and the bottom plate – from 10 to 30 mm. The distances between the transversal beams of the orthotropic decks are 1.85 and 2.00 m. The intermediate supports of the structure are two piers with a maximum height of 42.50 m. The reinforced concrete piers have a rectangular cross-section and individual footings. The steel used for the producing of the bridge superstructure is St52.3 according to DIN. The total weight of the superstructure is 22000 kN. The steel consumption for the superstructure is 4.20 kN/m². Segments of the orthotropic box girders were prefabricated by welding. The top and the bottom plates of the segments were assembled by welding. The remaining joints were high strength bolt connections. The bridge structure is supported on the abutments by steel roller supports, type “Creutz” and the pier supports – by the neoprene bearing pads produced by “Gumba” GmbH [1].



Fig.5 Viaduct 2 “Hemus” highway, 27.500km. (Courtesy of prof. M. Minev and prof. P. Staykov. With permission.)

2.6 “VARTOPA” BRIDGE

The “Vartopa” bridge is a 5-span continuous steel orthotropic trapezoidal-box girder structure connecting “Mladost”, a large Sofia district with the downtown (Figure 3.8). The bridge passes over the Vartopa River and carries the city metropolitan railway and the highway traffic. The structure was designed by P. Staykov and R. Mladjov. The bridge was built by “Metal Structures “Company and was opened to traffic in 1999. The owner is the agency “Road Infrastructure”. The bridge has a total length of 276.00 m and includes spans of 48.00 m + 60.00 m +60.00 m + 60.00 m + 48.00 m. It is placed in a horizontal curve with a radius of 1000.00 m and the longitudinal slope of 2.578 %. The superstructure consists of two parallel steel boxes of 2.80 m high and they are spaced at 13.60 m. The top plates of the boxes are 6.40 m wide and the bottom plates are 4.00 m wide. The orthotropic deck is formed by longitudinal and transversal stiffening beams. The transversal beams are 645 mm deep and they are spaced at 3.00 m. The longitudinal beams are also 645mm deep and they are placed at 1.50 m. The deck is additionally strengthened by longitudinal stiffeners with a trapezoidal cross-section 250 mm deep. The thicknesses of the deck slab vary from 12 to 16 mm; the box walls vary from 10 to 14 mm; the bottom of the box varies from 10 to 20 mm. The total width of the deck is 27.50 m. The expansion joints are placed on the bridge abutments. The box girders are spliced by welding. The bridge piers are two concrete filled steel tubular column bents with a maximum height of 15.20 m, a diameter of 1.20 m. The distance between columns is 13.60 m. The cap beams have a “I”-shaped cross-section. The abutment supports are steel rollers produced by the “Creutz” company; the pier supports are elastomeric bearing pads produced by the “Gumba” GmbH. The bridge piers have concrete footings. The welded structure of the bridge is manufactured by steel grade St 52.3 according to DIN standard. The superstructure was constructed by a cantilever launching [1].



Fig.6 “Vartopa” bridge. (Courtesy of P. Staykov. With permission.)

2.7 “VARNA LAKE” MOVABLE BRIDGE

The “Varna Lake” movable bridge is a steel structure near Varna (Fig.7). The bridge carries the railway and road traffic of the industrial part of Varna over the ship canal which connects the Varna Lake with the Black Sea. The bridge was designed and built by the “MAN” company. The bridge owner is the Municipality of Varna. It was opened to traffic in 1939. The bridge is a 3-span riveted steel structure with a total length of 80.34 m and consists of two stationary parts with spans of 24.36 m and a movable central span of 31.62 m. In 1975 the movable part of the bridge was completely destroyed by a ship accident and the stationary parts were seriously damaged. The new “Varna Lake” movable bridge was designed by B. Bankov. The Varna side stationary part was designed as simply supported bridge with an orthotropic deck. The superstructure consists of two main girders are 2.80 m deep with a distance between them of 8.00 m. The deck slab is 12 mm thick and 13.86 m wide. The girder has bottom flanges with dimensions of 12x400 mm and a web 12 mm thick. The floor beams are 1.55 m deep with a distance between them of 1.975 m and have a bottom flanges with dimensions of 20x300 mm and a webs 8 mm thick. The centreline of the railroad has an offset from the centreline of the bridge by 1.55 m, and is supported by two longitudinal girders spaced at 1.50 m, with a span of 1.975 m and depth equal to the depth of the secondary beams. Their web has dimensions of 8x540mm. The longitudinal girders also serve as deck stiffeners, and are designed as continuous beams supported by the secondary beams (at 1.975 m centers). The lateral bracings of the deck are “K”-braces placed at bottom flanges of the main girders. The central movable part is a trough truss structure which consists of two identical Warren trusses with spans of 31.62 m which include 6 panels of 3.162 m long. The members of the upper, the lower chords and compression diagonals have box cross-sections; the tension diagonals have “I”-shaped cross-sections. The truss deck is also orthotropic plate structure and consists of bottom chord members, a 12 mm thick steel plate. The deck is stiffened by longitudinal ribs of 10x160 mm spaced at 300 mm. The floor beams are placed at the lower joints of the trusses and have a span of 8.40 m.



Figure 7 “Varna Lake” movable bridge.

The distances between the stringers are 1.757 m. The portal bracing of the trusses is formed by box-sections members placed at upper joints of the trusses. The Asparuhovo side stationary part remains as the original riveted structure. The old bridge piers are used for a new movable part. The bearings are elastomeric bearing pads. Two type steel grades were used for steel bridge: steel grade M16C and steel grade 10G2CF [1].

3 CONCLUSION

The construction of the steel orthotropic bridge structures in Bulgaria, should be considered as a significant achievement of the Bulgarian bridge construction. The steel orthotropic bridges mentioned above are examples of an effective structural method for large span bridges. It is applicable for spans of 80-160m, considering that the prefabricated steel components are produced by the mounting site. To achieve of design versatility with the use of steel as a building material, a visually attractive structure could also be created. Since the region of Bulgaria is an active earthquake area, seismic codes had to be considered. However, the structure was proven to be ideal for earthquake regions. A high level of precision was required during the process of construction, since much of the work had to be carried out in advance under the factory conditions. The technical tests proved the high quality of the structure. This important and complex bridges has been operating successfully more than about 20-30 and 40 years. (see fig. 1-7) [1].

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SPECIFIČNOSTI PROJEKTOVANJA KONSTRUKCIJE CENTRALNE BANKE AZERBEJDŽANA

Rezime:

956

Zgrada Centralne Banke Azerbejdžana, u glavnom gradu Bakuu, sastoji se od dve kule, severne - visine 141m i južne - visine 123m, koje su međusobno povezane na 4 sprata pločama, kao i kosnicima na vrhu tornjeva. Dimenzije kula u osnovi su po 17x48m. Projekat je rađen prema američkim propisima, koji za ovu visinu objekta zahtevaju primenu dualnog sistema za obezbeđenje bočne stabilnosti, koji se sastoji od betonskih jezgara i specijalnih momentnih ramova. Analiza konstrukcije obuhvata i nelinearnu dinamičku analizu, po metodi Projektovanje Konstrukcije Prema Performansama, koja se danas može sprovesti programom Etabs Ultimate.

Ključne reči: momentni ramovi, nelinearna dinamička analiza, Etabs Ultimate

DESIGN OF STRUCTURE OF CENTRAL BANKA OF AZERBAIJAN – SPECIFIC TOPICS

Summary:

Building of Central Bank of Azerbaijan is located in capital city Baku. It consists of two towers, north one 141m tall and south one 123m tall, which are connected on 4 floors by slabs, as well as bracings on the top. Dimensions of towers in plan are 17x48m. Design is conducted in accordance with USA codes. For the given height, codes are prescribing dual system as a lateral force resisting system, composed of concrete cores and special moment frames. Structural analysis includes nonlinear dynamic analysis based on Performance Based Design methodology, which can be conducted today in Etabs Ultimate software.

Key words: moment frames, nonlinear dynamic analysis, Etabs Ultimate

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1 UVOD

Originalni projekat zgrade Centralne Banke Azerbejdžana (CBA u daljem tekstu), u glavnom gradu države Bakuu, autorsko je delo arhitektonske firme Coop Himmelblau iz Austrije, dok je projekat konstrukcije poveren turskoj firmi Ural. Projekat je završen 2012. god., a 2015. god., belgijska građevinska kompanija Besix, ugovorila je izvođenje objekta. Sastavni deo izvođenja obuhvatao je i preprojektovanje. U domenu konstrukcije, Besix je pored sopstvenog biroa u Briselu, angažovao i kompaniju DNEC d.o.o., čiji je ugovoreni obim posla obuhvatao:

- Izradu zajedničkog DNEC-Besix prostornog FEM modela i odgovarajuće standardne analize,
 - Definisanje parametara za seizmičku analizu, uključujući i akceleroگرامe,
 - Projekat čelične konstrukcije, čiji su osnovni elementi: spregnute tavanice, specijalni momentni ramovi, spregnute grede, elementi koji povezuju dve kule,
 - Definisanje kriterijuma analize po metodi Projektovanje Konstrukcije Prema Performansama (engl. Performance Based Design, PBD u daljem tekstu),
 - Sprovođenje nelinearnih dinamičkih analiza (NDA u daljem tekstu).
- Preprojektovanje je kao i osnovni projekat rađeno u skladu sa paketom američkih propisa.

2 OPIS OBJEKTA

Zgrada CBA, sastoji se od dve kule, severne - visine 140.9m, sa 33 sprata i južne - visine 123.3m i 29 spratova, koje su međusobno povezane na 6., 16., 25. i 27. spratu spregnutim pločama, kao i kosnicima na vrhu tornjeva (Slika 1). Dimenzija kula u osnovi su približno 17x48m, pri čemu se od prizemlja do 18. sprata, osnove rotiraju za po 4 stepena na svakom spratu, a zatim do vrha zadržavaju paralelni položaj na razmaku od 12m, čime je dobijen amorfnu oblik objekta. Spratna visina je 4.2m do 27. sprata, a zatim 4.4m do vrha zgrade. Kule su pri dnu povezane zajedničkim podzemnim objektom dimenzija 91x81m, koji se sastoji od jedne etaže i mezanina (Slika 1 desno).

Fasada je usvojena tako da objedinjuje obe kule u jedinstveni volumen (Slika 1, levo i sredina), čiji je sastavni deo i atrijum, formiran u prostoru između kula i fasade.

3 OPIS KONSTRUKCIJE

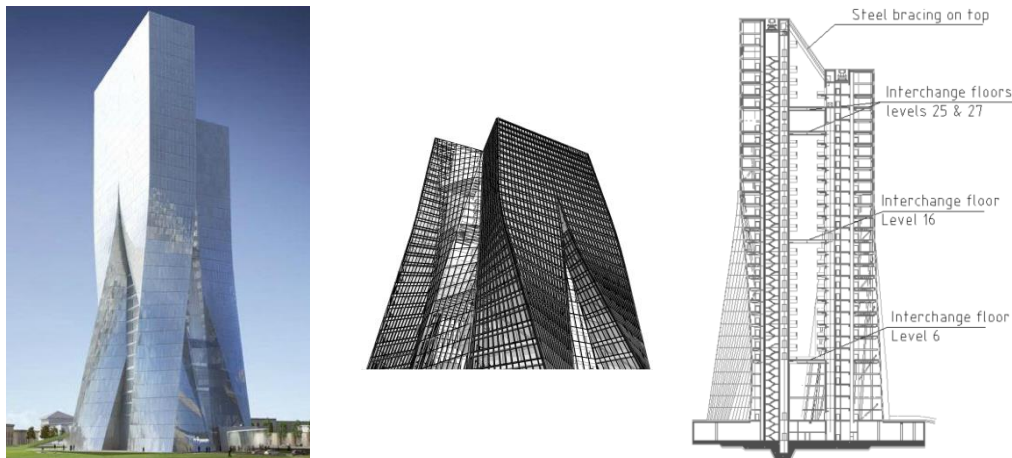
Zgrada je po značaju svrstana u III kategoriju prema američkom propisu ASCE7-10, što podrazumeva da bi kolaps konstrukcije nosio značajan rizik po ljudske živote. Locirana je u seizmički aktivnom području, na obali Kaspijskog mora. Projektna seizmička ubrzanja, u skladu sa ASCE7-10 (Ref. [1]), iznose 0.79g i 0.46g na platou spektra i pri periodu od 1sec, respektivno, čime je konstrukcija svrstana u seizmičku projektну kategoriju D. Tlo je kategorije D – koje se opisuje kao tvrdo tlo sa brojem udaraca u SPT metodi između 15 i 50.

Zbog svoje visine i specifičnog oblika, vršena su i tunnelska ispitivanja vetrom, od strane nemačke firme Wacker Ingenieure. Brzina vetra sa povratnim periodom od 50 godina, osrednjena na 3 sec (engl. 3 sec gust), iznosi 55m/sec.

Konstruktivni sistem za prenos vertikalnih opterećenja usvojen je u vidu spregnutih tavanica, formiranih od spregnutih ploča i spregnutih greda, koje su na jednom kraju oslonjene na fasadni ram, a na drugoj na betonsko jezgro. Fasadni ram se sastoji od spregnutih stubova –

čeličnih cevi ispunjenih betonom (CFT u daljem tekstu), i čeličnih greda usvojenih od I profila. Stubovi su postavljeni na konstantnom rastojanju od ivice ploče na svim spratovima, tako da su, usled rotacije tavanica, kosi od prizemlja do 18. sprata, a zatim vertikalni do vrha. Jezgra su vertikalna, dimenzija 30x10m. Postavljena su paralelno, jedno u odnosu na drugo, na razmaku od 12m i smaknuta za 18m u podužnom pravcu. Sastoje se od većeg broja zidova, debljine 300,400 i 600mm, koji formiraju složeni poprečni presek (Slika 2).

Podzemna konstrukcija usvojena je od armiranog betona. Zgrada je fundirana na šipovima.

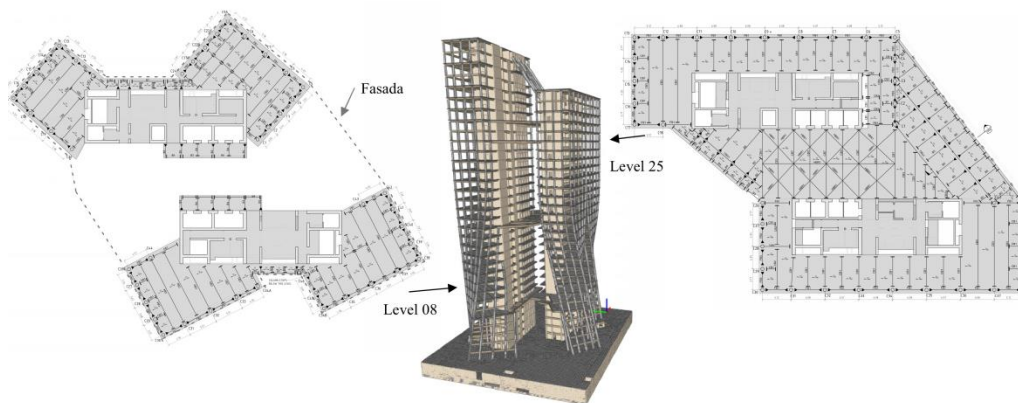


Slika 1 – Izometrijski prikaz objekta (sredina i levo) i poprečni presek (desno)

Sistem za obezbeđenje bočne stabilnosti, je prema zahtevima američkog propisa ASCE 7-10, za seizmičku projektnu kategoriju D i visinu zgrade preko 50m, usvojen kao dvojni sistem, koji se sastoji od sistema AB jezgara, koja prenose više od 75% seizmičkih sila i specijalnih momentnih ramova u fasadi (SMR u daljem tekstu), koji redukuju spratna pomeranja na gornjim etažama i obezbeđuju rezervu sigurnosti pri dejstvu zemljotresa, a treba da budu projektovani tako da mogu da prenesu najmanje 25% ukupnih seizmičkih sila. Spregnute tavanice, koje u ovom sistemu imaju ulogu dijafragmi, su zbog velikog broja otvora za instalacije postavljenih po obodu jezgara, sa njima povezane na manje od 40% od bruto kontaktnog obima (Slika 2).

Elementi koji povezuju kule, na nivoima 6, 16, 25 (Slika 2) i 27 sastoje se od horizontalnih čeličnih spregova usvojenih od čeličnih cevastih profila $\Phi 1150 \times 30$ mm, u okviru spregnutih ploča, dok su vrhovi kula povezani kosnicima, usvojenim od čeličnih cevastih profila $\Phi 300 \times 16$ mm. Vezni elementi, a naročito kosnici na vrhu značajno utiču na ponašanje konstrukcije, a pre svega su efikasni u kontroli pomeranja.

Prema prvobitnom, Uralovom rešenju, zgrada je imala i čelične dijagonale u fasadi, u svrhu povećanja torzione krutosti konstrukcije. U novoprojektovanom rešenju ovi elementi su uklonjeni, jer kompromituju rad momentnih ramova, remete predviđeno ponašanje konstrukcije tokom zemljotresa i formalno nisu u skladu sa pravilnikom.



Slika 2 – Tipične osnove CBA konstrukcije – Nivo 8 (Levo), nivo 25 (desno)

4 KONCEPT ANALIZE KONSTRUKCIJE

Projekat konstrukcije rađen je prema američkim propisima. Za analizu opterećenja i zahteve konstrukcije u celini, korišćen je ASCE7-10. U cilju definisanja seizmičkog dejstva urađena je i mikro seizmička rejonizacija (engl. Site Specific Spectra Report), na osnovu koje je utvrđeno da je spektralna kriva, korišćena u originalnom projektu, bila previše konzervativna, pa je korigovana usvajanjem realnih vrednosti parametara koji je definišu, čime je faktički smanjen nivo seizmičkih sila. Originalni Projektant Ural je, u vezi sa tim, zahtevao da se ponašanje konstrukcije, pored provera po odredbama osnovnog pravilnika, potvrdi i po metodologiji PBD.

Za razliku od ASCE7-10, koji u slučaju seizmičkog proračuna kao projektni cilj ima očuvanje života ljudi (engl. Life Safety – LS u daljem tekstu), pri zemljotresu sa povratnim periodom od 475 god., PBD podrazumeva da zgrada ostane potpuno operativna pri čestim zemljotresima sa povratnim periodom od 43 god. (engl. Immediate Occupancy – IO u daljem tekstu), a da pri maksimalno razmatranom zemljotresu, sa povratnim periodom od 2475 god. ne dođe do rušenja (engl. Collapse Prevention – CP u daljem tekstu). Provera ovih zahteva u određenim slučajevima podrazumeva sprovođenje nelinearnih dinamičkih analiza (NDA u daljem tekstu). Do nedavno je standardni alat za NDA bio softver Perform 3D, razvijen od strane kompanije “Computers and Structures” iz SAD, ali je tokom 2013. god. ista kompanija unapredila svoj znatno popularniji softver Etabs, u Etabs Ultimate, tako da je pored opcija za nelinearno modeliranje linijskih elemenata, u ovoj verziji to omogućeno i za zidove. Time je postignuto da se i linearne i nelinearne analize sprovode u istom softverskom paketu, što znatno ubrzava rad i smanjuje mogućnost za pravljenje grešaka.

Sa druge strane, provera svih usvojenih kriterijuma za projektovanje, uslovila je razvijanje više nezavisnih Etabs Ultimate modela. Tako su napravljeni i korišćeni u analizi:

- osnovni model za proveru nosivosti elemenata i seizmičkih pomeranja, u kojem su krutosti vertikalnih elemenata, prema ACI 318 (Ref. [3]) redukovane na 70% bruto krutosti,

- model za proveru pomeranja na vetru, u kome nisu redukovane krutosti vertikalnih elemenata,
- model za proveru pomeranja pri čestom zemljotresu, sa krutostima elemenata zadatim u skladu sa dokumentom TBI (Ref. [10]), koji globalno definiše metodologiju PBD,
- model za proveru SMR, u kojem su oslabljena jezgra, tako da se 100% seizmičke sile nanosi na ramove, a zatim se uticaji skaliraju na potrebnih 25%,
- model za analizu gravitacionih uticaja pri dugotrajnim sleganjima, koji jedini od svih pomenutih modela, umesto idealizovanih oslonaca ima opruge,
- model za proveru elemenata koji povezuju zgrade, u kome je smanjena aksijalna, kao i krutost na savijanje spregnutih ploča mostova, tako se da celokupni prenos sila vrši preko horizontalnih čeličnih spregova i kosnika na vrhu,
- model za nelinearne dinamičke analize.

Dimenzionisanje betonskih elemenata sprovodi se u skladu sa ACI 318-14 (Ref. [3]).

Što se tiče SMR, oni treba da pored odredbi generalnog pravilnika za projektovanje čeličnih konstrukcija AISC 360-10 (Ref. [4]), ispune i uslove posebnog pravilnika za proračun čeličnih zgrada u seizmičkim područjima, AISC 341-10 (Ref. [5]). Naročita pažnja se poklanja vezama greda i stubova, u skladu sa pravilnikom AISC 358-10 (Ref. [6]), koje pri seizmičkom dejstvu moraju da obezbede duktilno ponašanje.

Spregnute tavanice su dimenzionisanje u skladu sa AISC 360-10. Po pitanju upotrebljivosti, merodavna je bila provera vibracija tavanica u skladu sa dokumentom CCIP-016 (Ref. [7]). U pogledu seizmičkog proračuna, kako je već pomenuto, zbog velikog broja otvora na vezi ploče i jezgra, posebno je analiziran prenos sila u ovoj zoni.

Sastavni deo američke regulative za projektovanje je i dokument UFC 4-023-03 (Ref. [11]), koji daje smernice za projektovanje konstrukcije protiv progresivnog loma. Zgrada CBA je prema UFC-u, svrstana u III kategoriju po značaju (kat. IV ima najstrožije uslove), za koju se zahteva analiza po metodama Alternativnog Puta Sila i Povećane Lokalne Otpornosti, pri čemu obe podrazumevaju uklanjanje pojedinačnih stubova konstrukcije.

Kao što je najavljeno u uvodu ovog poglavlja, pored prethodno opisanih standardnih analiza, u okviru preprojektovanja CBA konstrukcije bila je predviđena i provera ponašanja po PBD metodologiji. Ona je nastala kao proizvod rada grupe konstruktivaca i geotehničara okupljenih u udruženju Pacific Earthquake Engineering Research Center, koji su 2010. god. izdali dokument TBI (Ref. [10]), koji sadrži smernice za projektovanje visokih zgrada, koje mogu da posluže kao alternativa trenutno obavezujućim procedurama važećih američkih pravilnika. Osnovna ideja je da se analizom konstrukcije, detaljnijom od standardne, potvrdi da konstrukcija u globalu ima dovoljnu krutost (kontrolom pomeranja), a da na nivou komponenata konstrukcije ispunjava definisane projektne ciljeve IO i CP. U određenim slučajevima, pre svega vezanim za maksimalni razmatrani zemljotres to podrazumeva sprovođenje NDA. One omogućuju realnije određivanje odgovora konstrukcije izložene dejstvu zemljotresa tako što uzimaju u obzir preraspodelu sila do koje dolazi pri nelinearnom ponašanju. Nelinearno ponašanje konstrukcije se uvodi preko geometrijske (P- Δ efekti) i materijalne nelinearnosti (definisani plastični zglobovi). Za primenu ove analize zahtevano je minimum 3 dvo-komponentalna akcelrograma čestih zemljotresa i 7 dvo-komponentalnih akcelrograma maksimalno razmatranih zemljotresa.

U nastavku je dat kratak pregled nekih kriterijuma za ove dve analize:

1. Kriterijumi vezani za česte zemljotrese (T=43god):

- Relativno spratno pomeranje manje od 0.5%, odnosno 1/200.

- Za zidove, nadvratne grede, stubove i grede odnos zahtevane nosivosti i nosivosti, treba da bude manji od 1.5, ali se prekoračenje dopušta za duktilne tipove loma, uz sprovođenje NDA, a kada se primenjuje kriterijum prihvatljivosti vezan za projektni cilj IO.

2. Kriterijumi vezani za maksimalno razmatrane zemljotrese (T=2475god):

- Srednja vrednost spratnih pomeranja (apsolutne vrednosti) dobijenih pri analizi pojedinačnih zapisa je ograničena na 3%, a maksimalna apsolutna vrednost u svim analizama je ograničena na 4.5%.

- Srednja vrednost spratnih pomeranja zaostalih na kraju zapisa (apsolutne vrednosti) dobijenih pri puštanju pojedinačnih zapisa je ograničena na 1%, a maksimalna apsolutna vrednost u svim analizama je ograničena na 1.5%.

- na nivou komponenata konstrukcije, za neduktilne tipove loma kontroliše se nosivost po procedurama TBI (Ref. [10]), za kritična i nekritična dejstva, gde se kao kritična definišu kao ona koja mogu da izazovu lom sa značajnim posledicama po stabilnost konstrukcije.

- na nivou komponenata konstrukcije, za duktilne tipove loma definisane su vrednosti graničnih deformacija, poput plastičnih rotacija ili dilatacija. Neke od ovih vrednosti date su u samom TBI dokumentu, poput graničnih vrednosti dilatacija za zidove, dok za kriterijume za linijske elemente TBI upućuje na druge dokumente poput ASCE 41-13 (Ref. [2]), ili ATC 72-1 (Ref. [9]) u kojima su definisane granične vrednosti povezane sa projektnim ciljem CP.

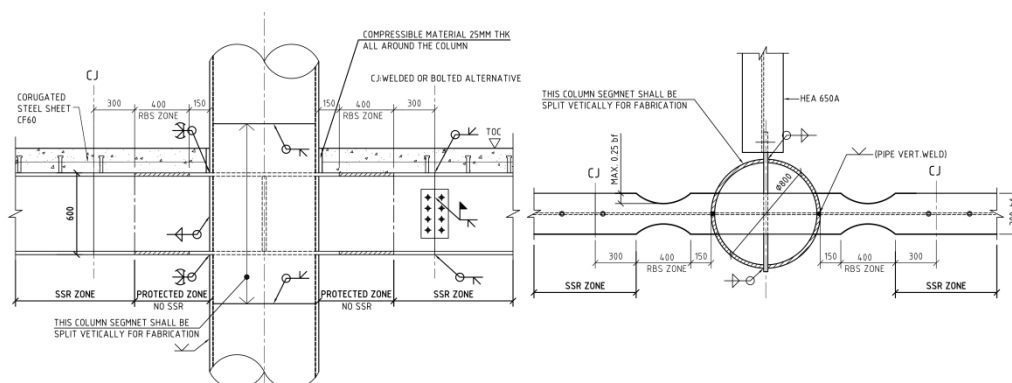
5 SPECIJALNI MOMENTNI RAMOVI

Specijalni momentni ramovi na CBA konstrukciji sastoje se od CFT stubova i čeličnih greda od valjanih profila. Osnovni zahtev pravilnika AISC 341 (Ref. [5]) je da ram bude projektovan po principu Jak Stub - Slaba Greda. U konkretnom slučaju ovo je postignuto slabljenjem greda u zonama oko stuba, putem zasecanja flanši, u iznosu od maksimalno 50% širine flanše, prema AISC 358 (Ref. [6]), čime su direktno diktirana mesta pojave plastičnih zglobova usled dejstva zemljotresa (Slika 4).

Pomenuti pravilnik definiše tipove veza, koje su odobrene za primenu u seizmički aktivnim zonama, pri čemu se u slučaju primene veze, koja nije navedena u dokumentu, zahteva testiranje u laboratoriji, u cilju dokaza adekvatnog (duktilnog) ponašanja pri prinudnim pomeranjima.

U samom pravilniku je detaljno opisana veza čeličnog stuba i grede I preseka, dok je za vezu CFT stuba i I grede, u delu sa komentarima, preporučena studija Azizinaminija i Snajdera (Ref. [8]), koji su testiranjem pokazali da je od 6 pod-tipova ove veze, najbolje rezultate po pitanju postelastičnih kapaciteta rotacija imala veza u kojoj je greda (i flanše i rebro) propuštena kroz stub. Shodno tome ovaj tip veze je usvojen i za CBA kulu.

Kako je pokazano na tipskom detalju veze (Slika 3), zona između zaseka na gredi i lica stuba se tretira kao zaštićena zona, u kojoj nije dozvoljeno bušenje grede, pa je samim tim montažni nastavak grede izmešten van ove zone. U zaštićenoj zoni nema moždanika za sprezanje ploče i grede, a sam stub je od betona odvojen zazorom širine 25mm u cilju smanjenja nosivosti grede.



Slika 3 – Tipična veza momentnog rama - greda I preseka propuštena kroz CFT stub

Sa stanovišta izvođenja, najosetljiviji deo, predstavlja betoniranje čvora, koje se izvodi kontraktorskim cevima, provučenim kroz slobodan prostor formiran između flanše grede i unutrašnje ivice cevi stuba. U dogovoru sa Besix-ovim izvođačima, kao minimalna dimenzija koja obezbeđuje neometano betoniranje čvora, usvojena je širina ovog dela od 200mm, na osnovu čega je sračunato da minimalna moguća dimenzija stuba iznosi 711mm.

Sa stanovišta projektovanja, ključne provere, koje se odnose na momentni ram su:

- provera smicanja grede u centru plastičnog zgloba, na silu koja odgovara momentu plastifikacije,
- provera čelične grede na spoju sa stubom, koja pored momenta plastifikacije treba da prenese i deo momenta koji je posledica ekcentriciteta smičuće sile, a koja deluje u sredini zgloba,
- dokaz da je nosivost stuba, uzimajući u obzir aksijalnu silu u stubu, 1.5 puta veća od nosivosti greada,
- dokaz da je nosivost elemenata na smicanje, koji obezbeđuju rebro grede i cev stuba veća od zbira aksijalnih nosivosti flanši, koje unose smicanje u čvor, pri čemu se zbog pomenutih potencijalnih problema sa betoniranjem čvora, nosivost betonske dijagonale u čvoru svesno zanemaruje.

Dok su prethodni zahtevi vezani za dejstvo seizmike, definisali “upper bound”, odnosno gornju granicu nosivosti greada, zahtevi vezani za dejstva gravitacije i vetra su definisali “lower bound”, odnosno donju granicu.

Kao rezultat prethodnih provera usvojen je specijalni momentni ram sa gredama H600A na rasponima dužim od 4.2m i gredama IPE400 na kraćim rasponima. Usvojeni prečnici CFT stubova su 711, 762, 813 i 1016mm.

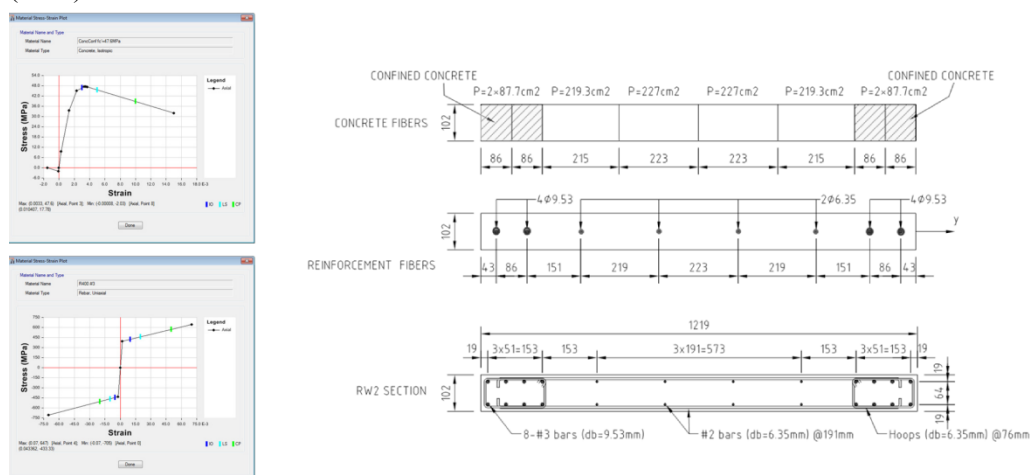
6 PBD UZ POMOĆ PROGRAMA ETABS ULTIMATE

Kako je već pomenuto, u okviru preprojektovanja CBA konstrukcije, kao alat za sprovođenje NDA izabran je softver Etabs Ultimate.

Uzimajući u obzir složenost i osetljivost nelinearnih analiza na ulazne parametre, kao i činjenicu da se Etabs Ultimate još uvek ne tretira kao standardni softver za ovu vrstu analiza, pre sprovođenja rigorozne NDA na konstrukciji u celini, sprovedeno je testiranje mogućnosti

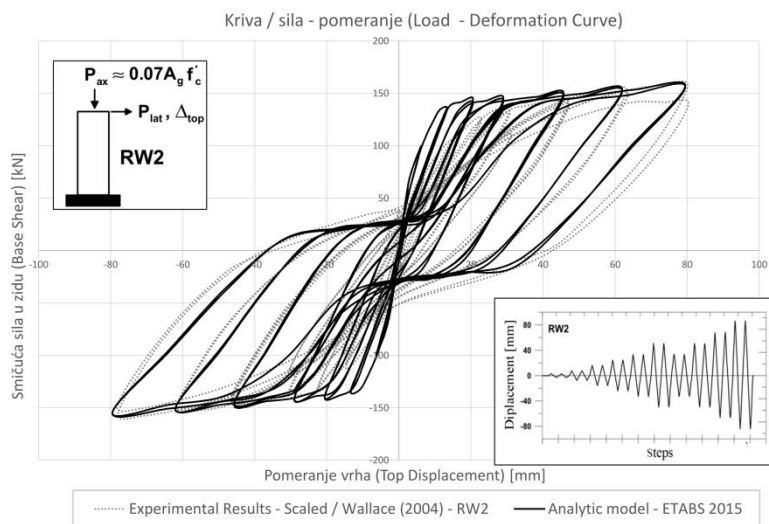
softvera kao i kalibracija ulaznih parametara za odgovarajuće nelinearne elemente prema rezultatima referentnih, verifikovanih, i široko prihvaćenih eksperimenata. Pored lokalnih modela sa definisanim plastičnim zglobovima (pravougaoni zid, zid T preseka, nadvratna greda, spregnuti stub, čelična greda sa zasečenim flanšama), razmatrane su i specifičnosti povezanih zidova sa nadvratnim gredama, kao i mogućnosti redukcije smičuće krutosti plastičnih zglobova zidova jezgra u nelinearnoj analizi. Na osnovu DNEC-ovog izveštaja sa prethodno opisanim testovima, Etabs Ultimate je prihvaćen i od strane Originalnog Projektanta Urala kao alat za NDA na ovom projektu.

Kao primer prikazana su u nastavku poređenja rezultata eksperimenata i definisanih analitičkih modela za pravougaoni zid i nadvratnu gredu. Nelinearnost u zidovima definisana je preko vlaknastih (“fiber”) plastičnih zglobova (Slika 4). Za vezu napon-dilatacija čelika, utegnuto i neutegnuto betona zadati su modeli definisani u referentnom eksperimentu, Wallace (2004) (Slika 5), dok je za histerezisno ponašanje zadat Etabsov “degradirajući” histerezis za oba materijala. Kao opterećenje je zadato ciklično pomeranje sa velikim vremenskim intervalima da bi dinamički efekat bio zanemarljiv. Prvo je naneto vertikalno opterećenje a zatim je stanje takvog deformisanog sistema uzeto kao početak nelinearne dinamičke analize koja se rešava direktnom integracijom, primenom „Hilber-Hughes-Taylor“ (HHT) numeričke metode.

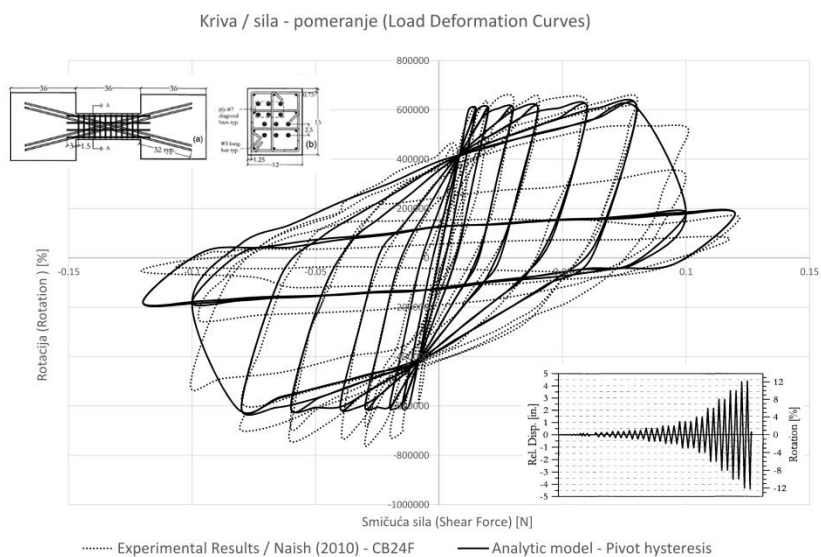


Slika 4 – σ - ϵ dijagrami utegnuto betona i čelika i odgovarajući vlaknasti elementi u preseku

Nelinearno ponašanje nadvratnih greda sa dijagonalnom armaturom je definisano preko smičućeg plastičnog zgloba koji je zadat na sredini elastične grede. Parametri ovog plastičnog zgloba zadati su u skladu sa preporukama za modeliranje ATC 72-1(Ref. [9]), a zajedno sa opterećenjem i ostalim parametrima prema referentnom eksperimentu, Naish 2010 (Slika 6).



Slika 5 – Funkcija sile u osloncu u odnosu na pomeranje vrha zida RW2 (pravougaoni zid) poređenje / eksperiment(Wallace-2004) u odnosu na analitički model(ETABS) – Ref [12]



Slika 6 – funkcija smičuće sile u odnosu na rotaciju (nad vratna greda), poređenje / eksperiment (Naish-2010) u odnosu na usvojen analitički model(ETABS) – Ref [13]

Kalibracijom parametara analitičkih modela plastičnih zglobova formirana je baza nelinearnih elemenata.

Plastični zglobovi se inicijalno dodeljuju samo elementima na kojima se očekuje nelinearno ponašanje (npr. na zidovima jezgara u nivou prizemlja). Ukoliko bi se, na osnovu rezultata

linearne analize, recimo na zidovima jezgra, uočila tendencija ka plastifikaciji van očekivane zone, usvajanjem veće količine armature bi se presek vratio u elastično područje, a u slučaju da to nije moguće, na takvom mestu bi bio zadat plastični zglobovi.

7 TRENUTNO STANJE PROJEKTA

Dispozicija konstrukcije je usvojena i potvrđena na osnovu analiza sprovedenih po odredbama osnovnog pravilnika ASCE7. Svi kriterijumi za projektovanje su zadovoljeni.

U pogledu proveru incidentnih slučajeva gubitka pojedinačnih stubova prema UFC-u, neke od greda momentnih ramova nisu imale potrebnu nosivost. Za rešavanje ovog problema potreban je dogovor sa Revidentom i Klijentom, a kao moguće rešenje razmatrana je mogućnost uvođenja moćnih rešetki na vrhu zgrade, o koje bi bili ovesani stubovi.

Što se tiče analiza po Metodologiji Programiranog Ponašanja, definisani su akceleroگرامи, kao i kriterijumi, urađene su neke od linearnih analiza, poput kontrole spratnih pomeranja pri čestom zemljotresu, dok je vezano za NDA, testiran i potvrđen Etabs softver kao alat za rad.

Projekat je zamrznut u trenutku neposredno pre sprovođenja nelinearnih dinamičkih analiza, tako da će ovaj deo posla biti urađen u očekivanom nastavku projekta.

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Milan Petrović¹, Darko Popović², Vanja Alendar³*

PROJEKTOVANJE KONSTRUKCIJE OBJEKTA CENTRA ZA PROMOCIJU NAUKE U BEOGRADU

Rezime:

Objekat Centra za Promociju Nauke, nalazi se na Novom Beogradu, u bloku 39. Sastoji se iz 3 glavna dela: Nadzemnog, Srednjeg i Podzemnog dela. Nadzemni deo sadrži glavni izložbeni prostor. Zamišljen je kao lebdeća kutija, dimenzija 70x70m u osnovi, koja je 16-20m izdignuta iznad terena i oslonjena na četiri tronošca. Srednji deo sadrži elemente za vertikalnu komunikaciju. Konstrukcija oba ova dela usvojena je kao čelična konstrukcija. Podzemni deo je jedno etažni objekat dimenzija 150x100m. Sadrži izložbeni prostor, servise, parking, perone za autobus. Konstrukcija podzemnog dela usvojena je kao betonska konstrukcija.

Ključne reči: naučni centar, čelična konstrukcija, betonska konstrukcija

DESIGN OF STRUCTURE FOR CENTER FOR PROMOTION OF SCIENCE

Summary:

Center for Promotion of Science is located in New Belgrade in Block 39. It consists of 3 main parts: Above ground part, Middle part and Basement part. Above ground part comprises main exhibition area. It is anticipated as a floating box with 70x70m dimensions in plan, elevated 16-20m, above terrain and supported by 4 tripods. Middle part comprises elements for vertical transportation. Structure for both parts is adopted as steel structure. Basement part has one floor. Its dimensions are 150x100m. It comprises exhibition areas, services, parking and bus drop off area. Structure of this part is adopted as reinforced concrete structure.

Ključne reči: science center, steel structure, reinforced concrete structure

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1 UVOD

Objekat Centra za promociju nauke (CPN u daljem tekstu), nalazi se na Novom Beogradu, u bloku 39. Investitor je JUP Istraživanje i razvoj iz Beograda, a korisnik Centar za promociju nauke. CPN je deo šireg naučnog kompleksa, koji se sastoji od CPN-a, kule „Naučni Institut“ i podzemnog „Nano Centra“, kao i podzemne garaže, koja povezuje različite delove kompleksa i sadrži dodatne sadržaje, kao što su prijemni hol, kontrolne sobe itd.

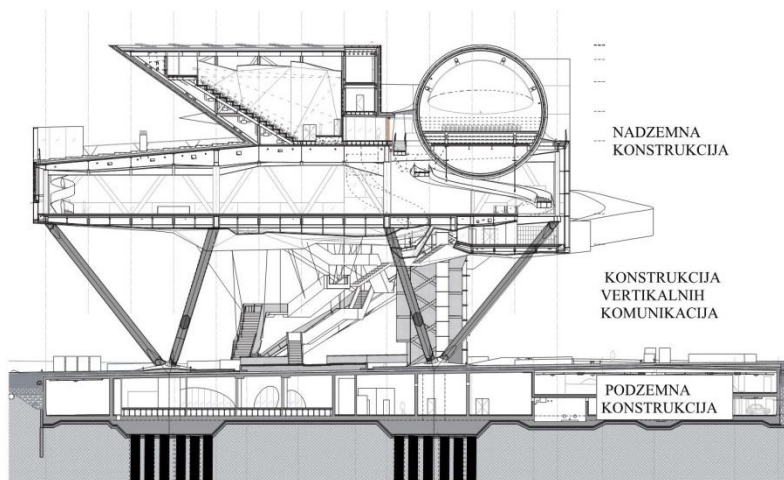
Na međunarodnom konkursu raspisanom 2010. god., koji je sa preko 200 prijavljenih radova bio konkurs sa najviše učesnika na svetu iste godine - za prvonagrađeno, izabrano je rešenje autora arhitekta Wolfganga Tschappelera – biro TWA iz Beča.

Nosilac projekta je projektni biro TWA iz Beča, a lokalni nosilac posla na projektima svih drugih struka je BWK Engineers iz Beograda, koji je kao podizvođača za projekat konstrukcije izabrao DNEC d.o.o iz Beograda. Projekat fundiranja je radila firma Fundament MB iz Beograda.

2 OPIS OBJEKTA

Objekat CPN-a sastoji se iz tri glavna dela (Slika 1):

- Nadzemna konstrukcija, koja sadrži glavni izložbeni prostor zamišljena je kao izdignuta čelična konstrukcija („lebdeća kutija“ – „floating box“), koja se nalazi na 16-20m iznad terena, oslonjena na četiri grupe od po tri nagnuta čelična stuba – četiri 'tripoda' ili tronošca.
- 'Vertikalne komunikacije', koje čine stepeništa, liftovi, eskalatori koji povezuju podzemni i nadzemni deo konstrukcije –
- Podzemna konstrukcija, armirano-betonska konstrukcija ('podzemna kutija'), dimenzija u osnovi 150x100m, sadrži izložbene prostore, servise, parking, peron za autobuse...



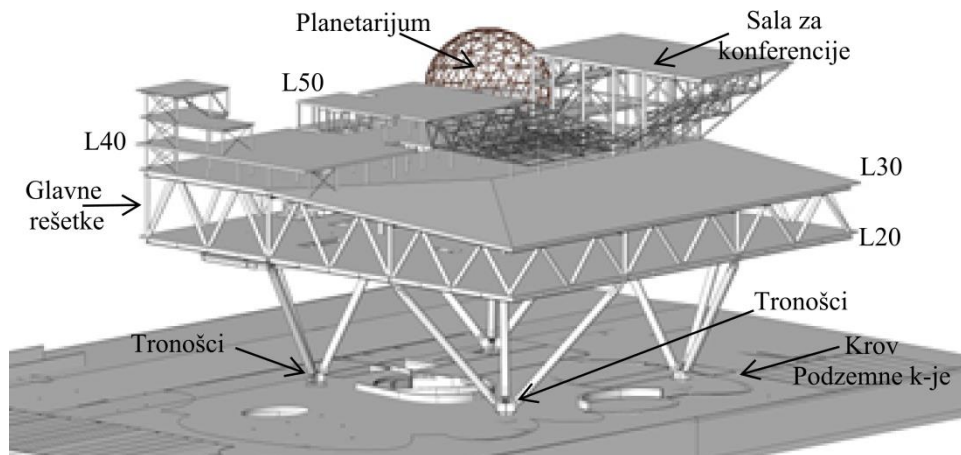
Slika 1 - Izgled konstrukcije CPN-a

Na nadzemnoj konstrukciji nalazi se nekoliko manjih dodatnih konstrukcija, kao što su: pravougaona „Konferencijska sala“, sferični „Planetarijum“, restoran i kancelarijski prostor, a o nju su okačeni: ulazni hol i lokalna infrastruktura, poput stepenica, liftova i eskalatora. Nadzemna konstrukcija CPN-a, se preko tronožaca oslanja na četiri masivna betonska stuba – pijedestala, koji prodiru kroz podzemnu kutijastu konstrukciju, i konačno se oslanjaju na četiri baterije bušenih šipova.

2.1 NADZEMNA KONSTRUKCIJA

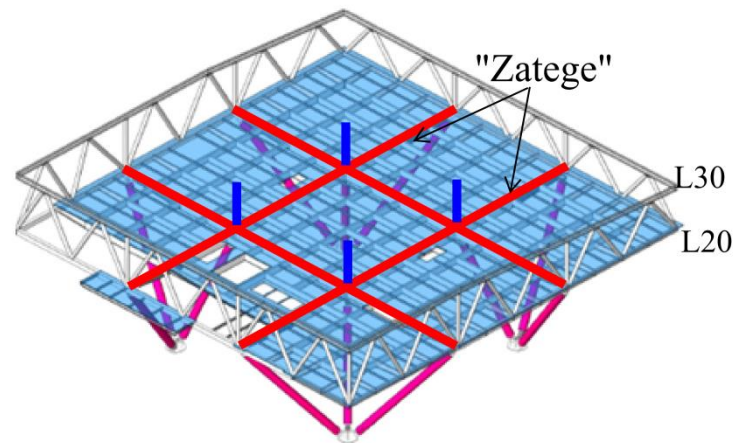
Konstrukcija izložbenog prostora je kutijastog oblika, dimenzija u osnovi 70x70m i visine 8-10m. Vertikalni obodni zidovi kutije su formirani od rešetkastih nosača pune spratne visine. Pod (L20) i krov (L30) konstrukcije izložbenog prostora je formiran kao roštiljni sistem od čeličnih nosača.

Dva od ukupno tri kosa stuba svakog tronošca podupiru obodne rešetke kutijaste konstrukcije, dok treći kosi stub iz svake grupe pridržava ploču L20 u trećinama raspona i vertikalnim stubom u nastavku pridržava gornju ploču konstrukcije L30. Na nivou kote tla četiri oslonca tronošca su horizontalno povezana AB pločom koja je ujedno i krovna ploča podzemnog dela konstrukcije, a vertikalno oslonjeni na masivne AB stubove fundirane na grupama bušenih šipova.



Slika 2 – 3D prikaz Nadzemne konstrukcije

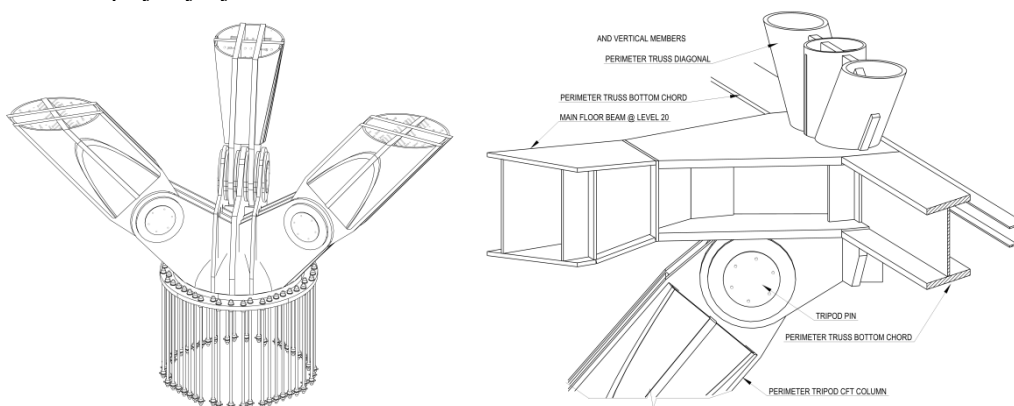
Nivo L20 predstavlja glavni izložbeni nivo i projektovan je kao otvoreni prostor (Slika 3). Na južnoj strani je „otvoren“ da bi se dobio pogled na ulazni hol na nivou L10. Konstrukcija nivoa L20 služi kao oslonac za tačke vešanja stepenica i spratova ispod nivoa L20.



Slika 3 - Nivo L20 _ Prikaz „Zatega“

Roštiljni sistem glavnih greda na ortogonalnom rastojanju od 23.2m označenih kao “zatege”, služi kao glavni sistem za primanje gravitacionih opterećenja, kao i horizontalnih sila iz kosih stubova tronožaca. Zatege su promenljive visine 1600-800mm. Ploča na nivou L30 je roštiljni sistem ponovljen sa nivoa L20, i služi da primi gravitaciono opterećenje. Glavne grede imaju promenljivu visinu, od 800 do 2240mm. Ploče iznad nivoa 30 su projektovane kao 300 mm debele ploče livene na licu mesta

Tronošci, kao glavni stubovi konstrukcije, predstavljaju glavne oslonce izdignute konstrukcije. Projektovani su kao spregnuti stubovi prečnika 1200mm sa zglobnim vezama sa trnom prečnika 500mm na bazama i vrhovima stubova (Slika 4). Dužina unutrašnjih stubova je 22.4m, a spoljašnjih je 27.0m.

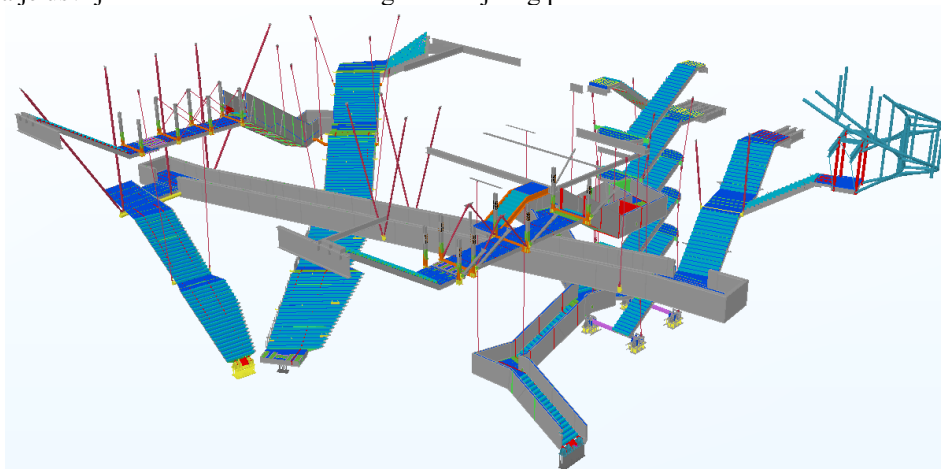


Slika 4 - Donja i gornja veza stuba tronošca

Paralelno sa detaljnim analizama u glavnom projektu formiran je i model u programu Tekla sa svim detaljima, koji služi za dobijanje kompletne radioničke dokumentacije.

2.2 KONSTRUKCIJA SREDNJEG DELA – VERTIKALNE KOMUNIKACIJE

Vertikalne komunikacije obezbeđene su pomoću stepeništa, dva lifta (teretni i putnički) i eskalatora. Stepeništa središnjeg dela su na gornjem kraju oslonjena na konstrukciju glavne zgrade, dok su na donjem kraju oslonjene na ploču prizemlja. U međurasponu stepeništa su pomoću vešaljki ovešane o podne grede glavne konstrukcije. Vešaljke su povezane sa konstrukcijom stepenica preko poprečnih čeličnih greda. Za konstrukciju stepeništa središnjeg dela je usvojen zavareni nosač složenog višecelijskog preseka.

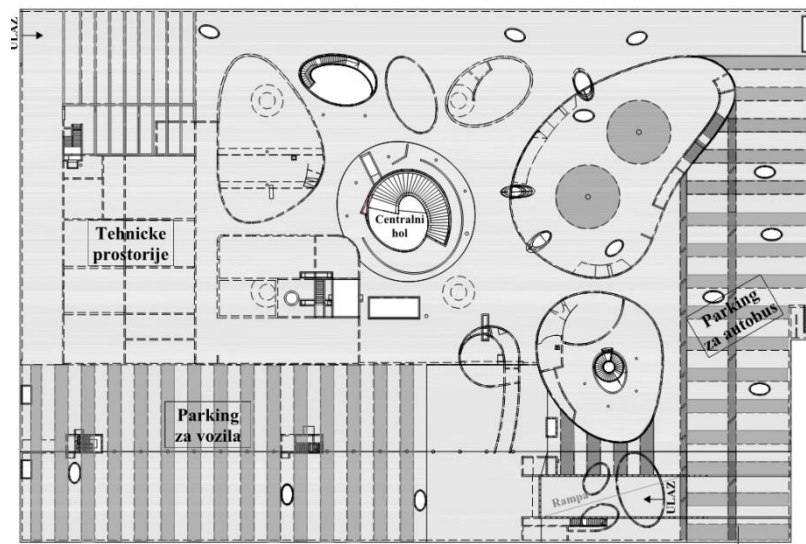


Slika 5 - 3D prikaz stepeništa srednjeg dela

2.3 PODZEMNA KONSTRUKCIJA

Podzemna konstrukcija je kutijastog oblika, oko 150x100m u osnovi, čiste visine oko 6m. Konstrukcija je projektovana kao integralna, bez stalnih dilatacija. Elementi konstrukcije su: temeljna ploča na koti -6.65; među ploča parkinga na koti -3.58; ploča na koti terena \approx -0.60, unutrašnji zidovi, stubovi i obodni potporni zidovi, kao i rampe za komunikaciju.

Funkcionalno, objekat je podeljen na dve glavne zone: Zona garaža i saobraćajnica za autobuse i vozila po obodu i centralna zona koja je pretvorena u izložbeni prostor. Gornja ploča na koti tla je pretežno debljine 400mm i prekrivena je slojem od oko 0.6m zemlje (kota 77.50/0.00). Da bi se obezbedio potreban broj parking mesta usvojen je koncept garaže povećanog raspona od 16.5m. Saobraćajni zahtevi u zoni ulaza i parkinga za autobuse rezultirali su tavanicom velikih raspona od 17.3 i 15m. Sve konstrukcije tavanice povećanih raspona projektovane su kao AB orebrane tavanice.



Slika 6 - Dispozicija Podzemne konstrukcije

Na ulazu i izlazu iz garaže za konstrukciju podzemlja vezane su konstrukcije tunela i rampi. Sa druge strane centralni ulaz za pešake omogućavaju spiralne stepenice širine 5m i raspona 20m, koje se oslanjaju na temeljnu ploču i krovnu ploču podzemlja. Pored njih komunikaciju omogućavaju još dva lifta, eskalator, kao i šest manjih stepeništa.

Pored uobičajenih zidova izdvajaju se jajoliki zidovi koji okružuju ulazni hol. Svi zidovi su debljine 250mm i 400mm.

2.4 FUNDIRANJE OBJEKTA

Podzemna konstrukcija je fundirana direktno na tlu uz poboljšanje terena na dubini od oko 0,5m. Koncentrisano opterećenje od nadzemne konstrukcije ima tendenciju da izazove nejednaka sleganja pa zahteva formiranje četiri baterije šipova da bi se ublažili ovi efekti. Ispod svakog od glavnih masivnih stubova na koje se oslanja nadzemna konstrukcija usvojena je grupa od po 24 bušena šipa prečnika 1200mm. Šipovi su dužine 19-23m, i zalaze u šljunkovit pesak.

Za koncept konstrukcije usvojena je integralna konstrukcija bez permanentnih dilatacija, osim privremenih prekida betoniranja u fazi izgradnje, što bitno pojeftinjuje cenu konstrukcije.

3 ANALIZA KONSTRUKCIJE

Idejno rešenje konstrukcije izradio je austrijski biro Werkraum, još u fazi konkursa, a zatim je faze projekta počev od revidovanog idejnog rešenja, pa sve do glavnog projekta uradila firma DNEC

Za konstrukciju objekta CFPOS karakteristično je da je nadzemna konstrukcija malo osetljiva na sleganje svojih oslonaca, da krutost spoljašnjih stepeništa malo utiče na globalno ponašanje konstrukcije i da je interakcija tla i konstrukcije eventualno značajna za naprezanje

samo donje, podzemne konstrukcije. Shodno tome, analiza konstrukcija izvršena je na tri nezavisna FE modela: nadzemna konstrukcija zglobno oslonjena na nivou terena, opterećena reakcijama stepeništa; stepeništa obešena o gornju konstrukciju i oslonjena na podzemnu; i podzemna konstrukcija opterećena reakcijama gornje konstrukcije i stepenica srednjeg dela.

3.1 NADZEMNA KONSTRUKCIJA

Globalni odgovor nadzemne konstrukcije je analiziran u prostornom modelu koji je urađen u programu Sap2000. Kao posledica nejednakog gravitacionog opterećenja konstrukcija ima globalnu tendenciju da se pomera ka jugozapadu, i da blago rotira suprotno od kretanja kazaljke na satu. Izvršena je detaljna analiza deformacija – ugiba. Relativni ugib usled korisnog opterećenja je u granicama dozvoljenog $L/500$. Iako je ukupan ugib za kvazi stalnu kombinaciju opterećenja unutar granice od $L/250$, neophodno je kontrolisati ukupan ugib sa nadvišenjem.

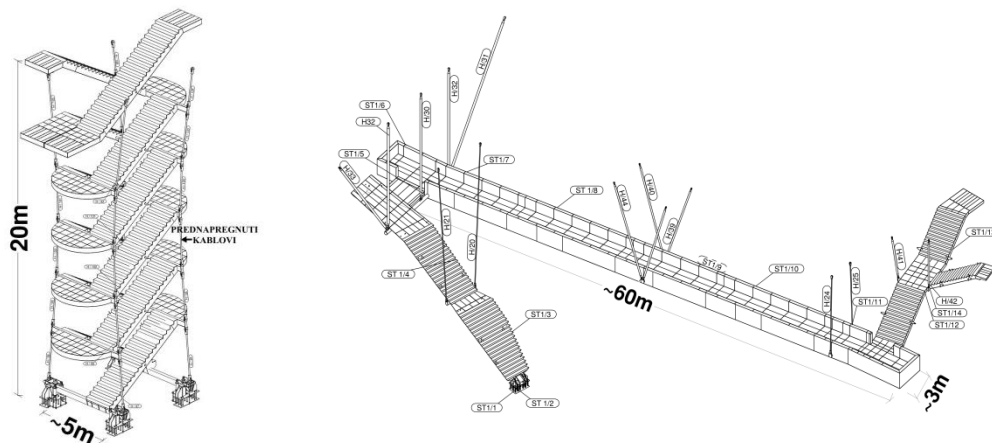
Prema podacima dobijenim od RHMZ-a, maksimalna osrednjena desetominutna brzina vetra, u skladu sa Evrokodom, na visini od 10m, za povratni period od 50 godina je $V_{m,50,10}=17.2\text{m/s}$. Prema internim preporukama komisije za uvođenje Evrokodova u domaću praksu, za projektnu brzinu vetra usvojeno je 20m/s . Projektom je predviđeno da se pre izvođenja konstrukcije uradi ispitivanje vetra u tunelu.

Seizmičko opterećenje je dominantno horizontalno opterećenje, i ono daje znatno veće globalne uticaje od vetra. Već sa prva tri tona dostiže se više od 90% angažovane modalne mase, što pokazuje da se sistem dominantno ponaša kao sistem sa jednom masom. ($T_1=1.72\text{s}$ - translacija X pravac, $T_2=1.65\text{s}$ - translacija Y pravac, $T_3=0.88\text{s}$ - torzija). Maksimalna pomeranja usled seizmičkog opterećenja prema Evrokod 8 su okvirno 105mm , što preneseno na otklon konstrukcije daje okvirno $105 / 40000 \approx 1/380$. Usvojen je faktor ponašanja $q=1.5$ (EC8), što odgovara elastičnom ponašanju konstrukcije.

Analiza efekata vibracija urađena je u skladu sa propisima EN 1990 i ISO 10137. Studija je urađena za unutrašnjost nivoa L20, sa sopstvenim frekvencama 2-3.5Hz. Odgovor konstrukcije je proračunat na kontinualnu aktivnost hodanja jedne osobe i analiza je pokazala da su vibracije ploče u dozvoljenim granicama, a da je faktor odgovora manji od 4.

3.2 KONSTRUKCIJA SREDNJEG DELA – VERTIKALNE KOMUNIKACIJE

Konstrukcija vertikalnih komunikacija je analizirana na: gravitaciono opterećenje, opterećenje vetrom, seizmiku i vibracije. Merodavno opterećenje za ovakve konstrukcije je vetar. Na sva stepeništa ispod nivoa zgrade apliciran je pritisak vetra od 4kN/m^2 . U nedostatku rezultata iz tunelskog testa za vetar ova vrednost je usvojena kako bi se uzeli u obzir efekti povećanja intenziteta brzine vetra ispod zgrade. Pokazano je da su neka od stepeništa, kao i konstrukcija lifta osetljiva na vibracije usred vetra. Za očekivati je da servisna horizontalna ubrzanja budu iznad limita 1 do 2 puta godišnje. Dalja analiza je moguća nakon detaljnog ispitivanja konstrukcije u tunelu za vetar. Finalne efekte indukovanih oscilacija usled vetra i ljudi, kao i zaključke o potrebnim dodatnim merama (npr. damperima) glavni Izvođač treba da potvrdi merenjem.

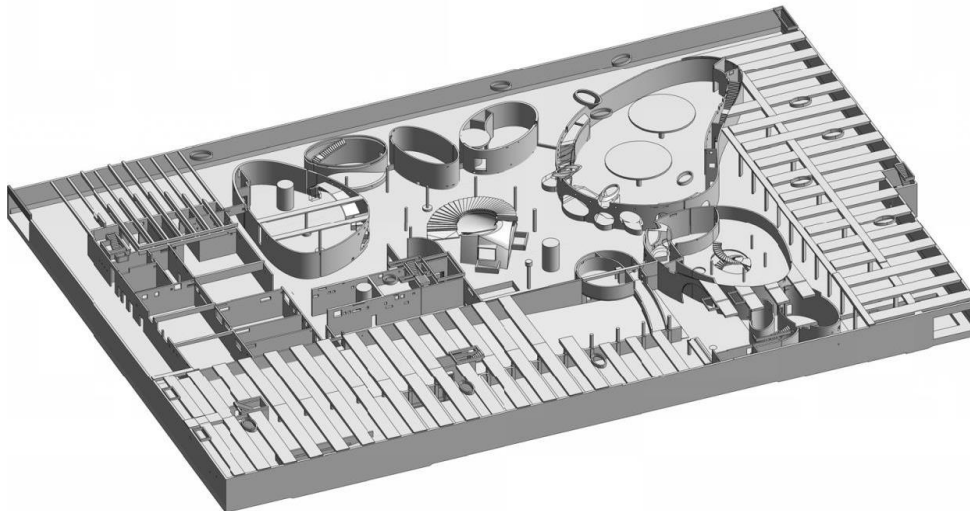


Slika 7 - 3D Prikaz stepeništa

3.3 PODZEMNA KONSTRUKCIJA

Podzemna konstrukcija je analizirana na: gravitaciono opterećenje, sva opterećenja koja dolaze od Nadzemne konstrukcije (vetar, seizmika...), kran za unošenje eksponata u glavni izložbeni prostor, uzgon i incidentno opterećenje.

Imajući u vidu da je krutost na savijanje kutije mala (odnos visine i dužine je $\approx 1:20$), ona se ne ponaša kao kruti temelj, već se najveći deo opterećenja sa gornje konstrukcije prenosi direktno kroz 4 oslonačke tačke, pa su na ovim mestima usvojene baterije šipova kako bi se smanjila kako ukupna, tako i diferencijalna sleganja i samim tim naprezanja ploča i zidova podzemne konstrukcije.



Slika 8 - 3D Prikaz Podzemne konstrukcije

Sa druge strane krutost podzemne konstrukcije za prenos horizontalnih opterećenja sa gornje konstrukcije je značajna i u ovom slučaju se ona ponaša kao kruta kutija. Tako se horizontalne reakcije nadzemne konstrukcije od vetra i seizmike prenose preko ploče 000 na obodne zidove. Ploča 000, vrši i ulogu kolektora, – razupirača, budući da prima i uravnotežuje horizontalne reakcije nadzemne konstrukcije preko oslonaca četiri tronošca.

Od vertikalnih elemenata u podzemnoj konstrukciji posebno se izdvajaju jajoliki zidovi. Njihova bitna karakteristika jesu otvori, koji sa unutrašnjim, zidanim zidovima formiraju efekat pećina, tunela (Slika 9). Pored standardne analize prenosa vertikalnog opterećenja kroz zidove do temelja, akcentat je stavljen na uticaj prostornih deformacije konstrukcije. Deformacije temeljne ploče u zoni baterija šipova – temelja gornje konstrukcije deformišu i zidove podzemlja, izazivajući velika smicanja u njima. Da bi se ovaj efekat smanjio, napravljene su vertikalne dilatacije u zidovima, u zonama u kojima sedaju na naglavne ploče. S obzirom da jajoliki zidovi okružuju prostor koji je izložbeni, a završni sloj zidova je „natur beton“ posebna pažnja je posvećena kontroli prslina i umesto uobičajenih 0.3mm usvojen je kriterijum od 0.15mm.

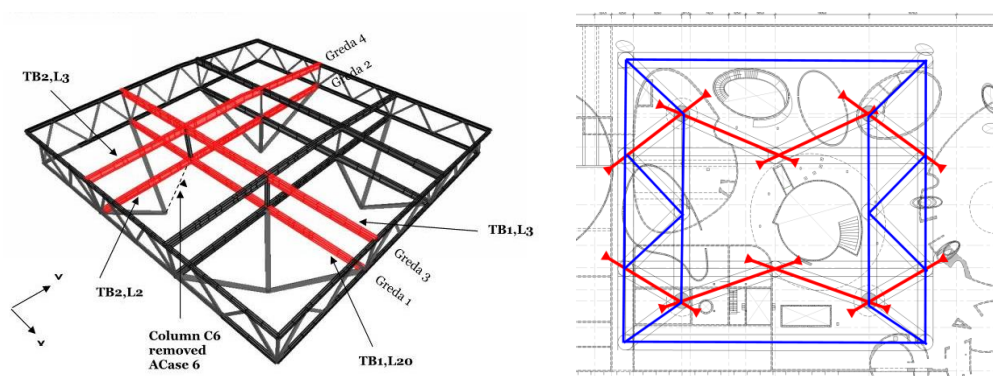


Slika 9 - Izgled jajolikih zidova Podzemlja

Usvojeni projektni nivo podzemne vode je 74,00mnm. Tokom istražnih radova zabeleženi nivo podzemne vode se kretao između 69,00mnm i 71,00mnm. Zbog postojanja velikih raspona u zonama parking prostora, posebno na istočnoj strani, pri projektnom nivou podzemne vode dolazi do odizanja dela podzemne konstrukcije. Iz tog razloga za dejstvo podzemnih voda usvojeno je da sopstvena težina konstrukcije drži vodu do nivoa 73.00mnm, a pri pojavi vode iznad ovog nivoa dolazi do aktiviranja bunara koji su predvođeni projektom. Bunari će biti aktivni sve dok voda ne počne da opada prirodnim putem.

4 SPECIJALNE ANALIZE

Da bi se sagledala kompletna slika potencijalnih opasnosti po ceo objekat analizirana su dva tipa incidentnih dejstava: 1. *Identifikovana incidentna dejstava* kao što su: a. Udarno dejstvo (udar vozila, viljuškara u stub i slično), b. Eksplozije u zatvorenom (eksplozija gasa u kuhinji, laboratoriji...); 2. *Neidentifikovana incidentna dejstva*, dejstva usled lokalnih otkaza konstrukcije obično prouzrokovanih eksplozivnom napravom. U slučaju neidentifikovanih incidentnih dejstava, nivo rizika je u Evrokodu definisan Klasama (Consequences Classes). U slučaju CPN objekta, klasa je procenjena kao 2b. Za nadzemni deo objekta koncept nesrazmernog kolapsa konstrukcije je analiziran za slučaj gubitka bilo kog kosog stuba Tronožaca. Za gravitaciono opterećenje reakcije su balansirane kroz mehanizam unutrašnjih zatega u ploči na koti tla ili kombinacijom pritisnutih i zategnutih dijagonala unutar ploče. Najracionalnije rešenje bi bilo povezati oslonce Tronožaca pravim zategnutim ili pritisnutim štapovima ali postojanje velikih otvora o ploči u zonama između oslonaca Tronožaca uslovljava formiranje trajektorija pritisnutih i zategnutih štapova oko otvora. Principijelno, svi štapovi u ploči su usvojeni širine 3m. Za slučaj gravitacionog opterećenja je potreban procenat zategnute armature oko 1.0%. Za incidentne slučajeve opterećenja pri gubitku stuba Tronošca ovaj procenat naraste, pa se na određenim pravcima umesto obične armature predviđaju DYWIDAG zatege.



Slika 10 - Gubitak jednog stuba tronošca i sistem zatega u ploči krova Podzemlja

Pored dokaza da konstrukcija ima kapacitet da primi opterećenje u slučaju otkaza jednog od glavnih nosećih elemenata, predviđen je i niz drugih mera prevencije od terorističkoj napada. Između ostalog glavni stubovi podzemne konstrukcije su zaštićeni zakrivljenim zidovima od direktne izloženosti parking prostoru i potencijalne opasnosti od dovoženja veće količine eksploziva. Ovi zidovi su kontrolisani na pritisak od eksplozije kao impulsno opterećenje. Krov podzemne konstrukcije sadrži otvore koji predstavljaju odušak pri potencijalnoj eksploziji, a uz to omogućavaju da podzemna konstrukcija ima prirodno svetlo. Predviđeni su kontrolni punktovi na ulasku na parcelu, kao i skrivene barijere u okviru zelenih površina koje služe da spreče kretanje vozila van obeleženih ruta kao i pokušaj direktnog udara u konstrukciju.



Miroslav Bešević¹, Aleksandar Prokić¹, Dragana Tabaković², Ilija Miličić³

NADOGRADNJA OBJEKTA U ULICI VOJVODE STEPE BR. 282 U BEOGRADU

Rezime:

S-58

Novoprojektovana noseća konstrukcija dogradnje je u vidu čelične konstrukcije sa pokrivačem od sendvič samonosećih Fe- plastificiranih panela (veći deo krovne površi) i u kombinaciji sa drvenim rogovima na strmijim ravnima krova, gde je predviđen krovni pokrivač u obliku eternit ploča. Glavni noseći sistem čelične konstrukcije je u statičkom smislu dvozglojni ram oslonjen na postojeće grede-stubove u fasadnim ravnima. Postojeći objekat je izveden kao zidana konstrukcija sa vertikalnim armiranobetonskim stubovima i vertikalnim serklažima, dok je međuspratna konstrukcija izvedena kao sitnorebrasta konstrukcija oslonjena na sistem greda i serklaža. Fundiranje objekta je izvedeno na armirano betonskim trakama-roštilju, različitih poprečnih preseka u skladu sa statičkim uticajima.

Ključne reči: sanacija, bet. konstrukcija, čelič.konstrukcija, dogradnja.

ROOFTOP ADDITION IN THE VOJVODE STEPE STREET NO. 282 BELGRADE

Summary:

Designed supporting structure for addition is in the form of a steel structure. Addition has a gambrel roof form with self-supporting sandwich Fe plasticized panels on the upper slope and wooden rafters with eternit covering. In static terms main supporting steel structure system is double-hinged frame supported by existing beam-columns in facade planes. The existing building was constructed as a masonry structure with vertical reinforced concrete columns and vertical ring girder, while floor construction is designed as ribbed construction supported by the system of beams and vertical ring girders. Foundation of the structure was carried out on reinforced concrete two-way grid, with different cross-sections in accordance with the static actions.

Keywords: reconstruction, reinforced concrete, steel structures, rooftop addition

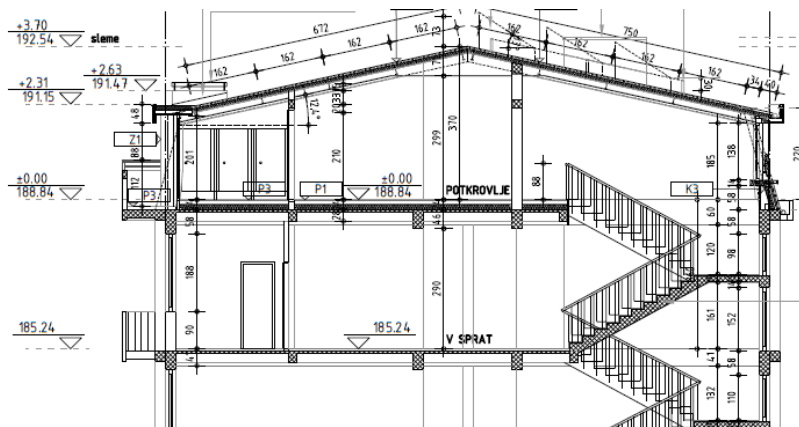
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1. UVOD

Postojeći objekat se nalazi u Beogradu u ulici Vojvode Stepe br.238. Zbog dugogodišnjeg prokišnjavanja ravnog krova i većih stalnih troškova na sanaciji izolacije a i potrebama za novim korisnim prostorom, pristupilo se izradi projekta nadogradnje postojećeg objekta spratnosti P+5. Korisnik objekta je Visoka škole elektrotehnike i računarstva strukovnih studija. Urađena je projektna dokumentacija (Projektant ZINCO DOO-Beograd) i dobijena građevinska dozvola za izvođenje. Projektant je sveobuhvatno analizirao projekat konstrukcije i način izvođenja radova u otežanim gradskim uslovima izvođenja. Usvojen je poprečni noseći konstruktivni sitem dvozglonog rama izrađenog od čelika. Na slici br.1 prikazan je poprečni presek kroz dograđeni de objekta-potkrovlja.

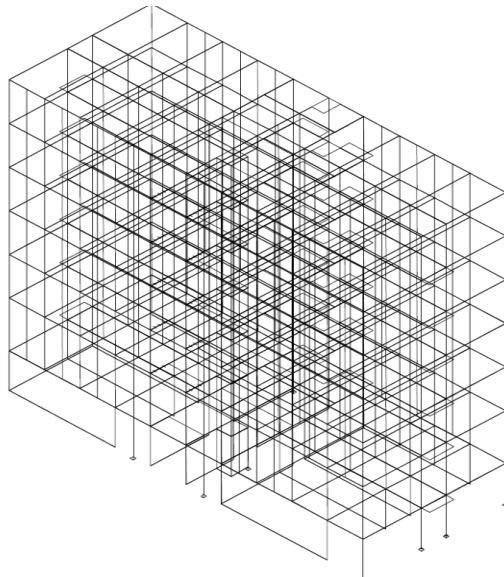


Slika 1- Poprečni presek dogradnje postojećeg objekta

Za potrebe vertikalne komunikacije vrši se delimično rušenje postojeće krovne konstrukcije Pos 600+ iznad stepenišnih krakova i izvodi se novo armiranobetonsko stepenište od V sprata do potkrovlja. Novoprojektovani pregradni zidovi se zidaju iznad postojećih zidova oko stepeništa i za sanitarne čvorove dok su preostali novi zidovi suvomontažni u vidu gipskartonskih pregrada.

2. KONTROLNI STATIČKI PRORAČUN KONSTRUKCIJE OBJEKTA

Postojeća konstrukcija objekta je analizirana za uticaje od dogradnje. Sprovedena je sveobuhvatna analiza uticaja od postojećeg i novododatnog opterećenja. Proračun konstrukcije postojećeg objekta je sproveden korišćenjem programskog paketa Tower. Kontrolnim proračunom je dokazana stabilnost postojećeg objekta t.j. konstrukcije, a izvršeno je i poređenje sa uticajima iz arhivskog projekta. Na slici br.2 prikazan je prostorni računski model konstrukcije, a na slici br.3 je prikazan kontrolni statički proračun trmelja objekta sa dogradnjom. U analizi napona i sleganja temelja korišćen je i geomehanički elaborat.



POS T5		Temeljna traka	b(m)= 2,00	d(cm)= 40
Analiza opt.:	od postojećeg opt.	iz statičkog proračuna		
	Stubovi $S^2_{k2} \times S^2_{k3}$			22800 dNm
	zid skloništa d=60 cm	0.60*3.2*24=		4608 -II-
	od sop. težine trake			2784 -II-
	od postojećeg opt.		q=	30192 dNm
Skida se				
	krovni pokr. Bet ploče	0.03*2400*5.92/2=		213 dNm
	pesak	0.03*1800*5.92/2=		160 -II-
	asfalt	0.03*1800*5.92/2=		160 -II-
	hidro izolacija	30*5.92/2=		89 -II-
	cementna košuljica	0.015*2100*5.92/2=		93 -II-
	sloj za pad šljako beton	0.04*1600*5.92/2=		189 -II-
	Šuplja opeka u pesku	124*5.92/2=		367 -II-
	Sneg i vjeter	10*5.92/2=		296 -II-
			q _s =	1567 dNm
od dodatnog opterećenja opt.				
	od stubova	5178/3.6=		1438 dNm
	zvučna izolacija	20*5.92/2=		59 -II-
	cementna košuljica	0.04*2100*5.92/2=		249 -II-
	pod parket	0.02*700*5.92/2=		41 -II-
	od pregradnih zidova	50*5.92/2=		148 -II-
	od fasadnog zida			208 -II-
	od korisnog opterećenja	250*5.92/2=		740 -II-
			q _s =	2884 dNm
dodato opterećenje 1316 dNm/m ~ 4.4%				
	σ_{opt} =	150 KN/m ²		
Napon u temeljnoj spojnici od postojećeg opterećenja:				σ_{av} = 151.0 KN/m ²
Napon u temeljnoj spojnici sa dodatnim opterećenjem:				
	σ_{av} =	157.5 KN/m ²	prekorac. napona:	5%
	q =	31508 dNm	q = q-sop.tez. =	28724.3 kNm

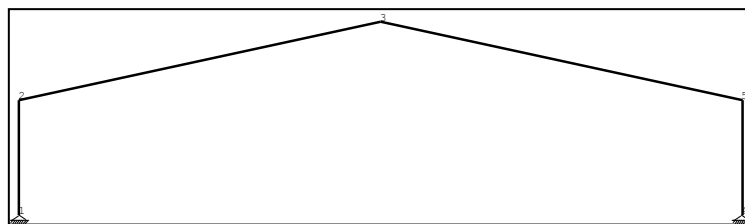
Slika 2-Prostorni model konstrukcije

Slika 3-Kontrolni proračun temelja

Izvršena je analiza opterećenja na postojeću konstrukciju iznad petog sprata za novoprojektovano stanje-potkrovlje kako bi se zaključilo da li je potrebna sanacija. Rezultati analize su pokazali da je novoprojektovano opterećenje približne vrednosti projektovanom opterećenju (manje od postojećih), ($q_{pk} = 5.30 \text{ kn/m}^2 \cong q_{npr} = 4.7 \text{ kn/m}^2$), što znači da postojeća međuspratna konstrukcija može sa prihvatiti opterećenje od dogradnje bez ojačanja-sanacije.

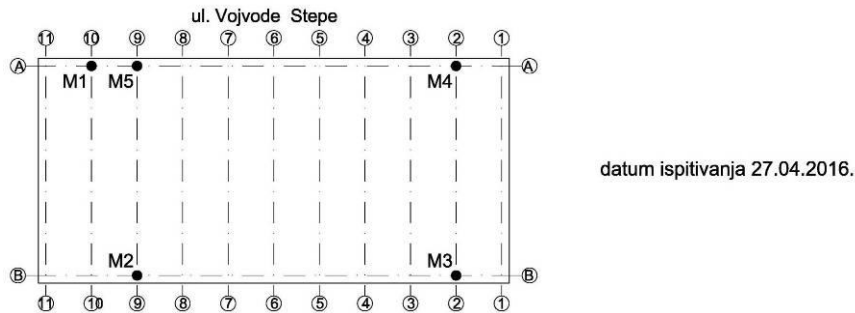
3. IZVOĐENJE DOGRADNJE OBJEKTA

Izvođenje objekta je sprovedeno tako sto su prvo utvđene dimenzije- gabarit postojećeg ravnog krova objekta geodetskim snimanjem. Ustanovljeno da nema velikih odstupanja od projekta. Prvo su obeležene tačke oslanjanja (ose) za postavljanje glanog poprečnog čeličnog nosača statičkog sistema dvozglonog rama(slika.br4).



Slika 4 -Statički sistem čeličnog poprečnog nosača

Zbog nepoznavanja kvaliteta ugrađenog betona u grede i stubove postojeće konstrukcije objekta, doneta je odluka da se ispituju ankeri koji će se montirati na mestima oslanjanja stopa čeličnih stubova. Ispitivane su dve vrste ankera (sa zalivanjem epoksidom 4 uzorka i jedan bez). Na sl. 5 prikazana su mesta ispitivanja ankera.



Slika 5-Merna mesta ispitivanih ankera

Ispitivanje je sprovedeno na četiri merna mesta (M1, M2, M3 i M4) i to zalivanjem epoksidom navojne šipke M16 ...5.8., dubine l)20cm. Čupanje ankera je vršeno sve do trenutka pada pritiska u presi, sa maksimalno m silom čupanja od $Z=39,21\text{KN}$ (100bara), što je znatno manje od maksimalnih računskih uticaja. Na slici br.6 su prikazane fotografije ispitivanja. Ispitan je i jedan anker Wurth bez zalivanja merno mesto M5.



Slika 6- Ispitivanje nosivosti ankera

Glavna noseća konstrukcija je od čelika i proizvedena je u radionici uz vođenje računa o načinu transporta i montaže na postojećem objektu. Iz tog razloga su izvedeni montažni nastavci u riglama ramova na tri približna dela. Na taj način je olakšan način transporta i ugradnje. Na sl. br. 7 prikazane su fotografije izrade čelične konstrukcije u radionici.



Slika 7- Izrada čelične konstrukcije

3.1 MONTAŽA ČELIČNE KONSTRUKCIJE

Pre početka montaže su prvo ugrađeni svi potrebni ankeri i ležišne ploče na mestima oslanjanja stubova poprečnih nosećih ramova, koji su prvo zaliveni epoksidem. Prvi montažni komadi koji se montiraju su stubovi sa delom rigle do montažnog nastavka, a zatim je izvršeno montiranje nosača rigli. Veza u montažnom nastavku je ostvarena primenom običnih zavrtneja i odgovarajućih podvezica (slika br.8).



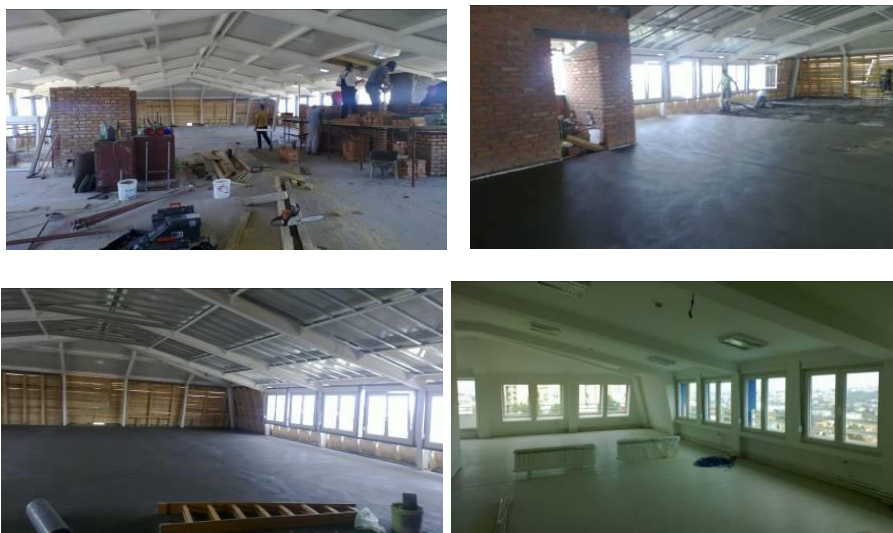
Slika 8- Poprečni presek čelične konstrukcije i montažni nastavak

Po završetku montaže glavnih poprečnih ramova montirani su kalkanski nosači, a zatim su montirane rožnjače. Po montiranju rožnjača pristupilo se montaži al-sendvič krovnog pokrivača. Na sl.9 su prikazane fotografije montaže rožnjača i krovnog pokrivača.



Slika 9- Montaža rožnjača i krovnog pokrivača

Po završetku radova na montaži čelične konstrukcije i postavljanje krovnog pokrivača pristupilo se završnim radovima prema funkciji i nameni prostora. Većina radova je sprovedena u suvom postupku osim izvođenja sanitarnih čvorova i podloge u vidu cementnog estriha za polaganje podova. Veza između V sprata i potkrovlja(V sprata) je ostvarena izvođenjem armirano betonskog stepeništa. Prilikom izvođenja radova na čeličnoj konstrukciji je sprovedene i mere PPZ u skladu sa projektom. Na slici br.10 su prikazani završni radovi.



Slika 10- Završni radovi

Osim radova na dogradnji objekta izvedeni su i radovi na rekonstrukciji celokupne fasade objekta.(slika br. 11).



Slika 11- Rekonstrukcija i sanacija fasade objekta

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ISPITIVANJE TIPSKE ČELIČNE REŠETKASTE SKELE ZA MONTAŽU HALA SA FEROCEMENTNIM ELEMENTIMA

Rezime:

U radu je izvršeno ispitivanje tipske čelične rešetkaste skele za montažu lučnih hala raspona 23 m od ferocementnih elemenata, koju je tipizirala firma Milinković Company. Prikazani su rezultati globalnih i lokalnih deformacija koje nastaju na rešetkastoj čeličnoj skeli pri izradi montažnih hala od armiranobetonskih lukova sa ispunom od ferocementnih ploča (koruba). Probno opterećenje bila je realna težina ferocementnih ploča (koruba) i zamenjujući teret za uticaj težine armiranobetonskih lučnih nosača za monolitizaciju. Merena su globalna pomeranja i sile u najopterećenijim štapovima rešetke. Urađena je računaska kontrola pomeranja i presečnih sila, i izvršeno poređenje rezultata.

Ključne reči: ispitivanje, čelična skela, ferocementne hale

TESTING OF TYPICAL STEEL TRUSS SCAFFOLD FOR ERECTING OF HALLS WITH FERROCEMENT ELEMENTS

Summary:

In the paper is presented testing of typical steel truss scaffold for erecting of arch structure halls spanning 23 m, made of ferrocement elements, which was standardized by Milinković Company. Results of global and local deformations of the steel truss scaffold during erection of assembly halls made of reinforced concrete (RC) arches with filling of ferrocement plates (channel units) are presented. Testing load was the real weight of the ferrocement plates, and for the weight of the RC arched girders that monolithize the structure a supplant load was used. Global displacements and forces in the most heavily loaded truss members are measured. The calculation check of the displacements and member forces is done, and comparison of results is performed.

Key words: testing, steel scaffold, ferrocement halls

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1 INTRODUCTION

Technical description of the structure: the tested scaffold represents a tool for erecting of ferrocement elements E23 and monolithization of the RC elements during constructing of arched structure halls with span of 23 m. The scaffold is an arched girder with an approx. span of 22 m, and minimal length of approx. 8 m. The height of the scaffold is 11 m. The basic structure of the scaffold are truss arches with square cross-section, made of corrugated sheets 3 mm thick and tubes with diameter of $\text{Ø}48$ mm. the weight of one arch is approx. 2000 kg. The arches are placed side by side, with axis distance of 2 m (minimum 4 pieces) and connect for each other by $\text{Ø}48$ mm tubes. Between the arches are also placed steel platforms for workers. On the edges of the arched girders are set adjusting „P“ girders on which the ferrocement (FC) elements are rested during the construction.

Position description:

1 – truss arches; 2 – connecting elements; 3 - adjusting „P“ girders

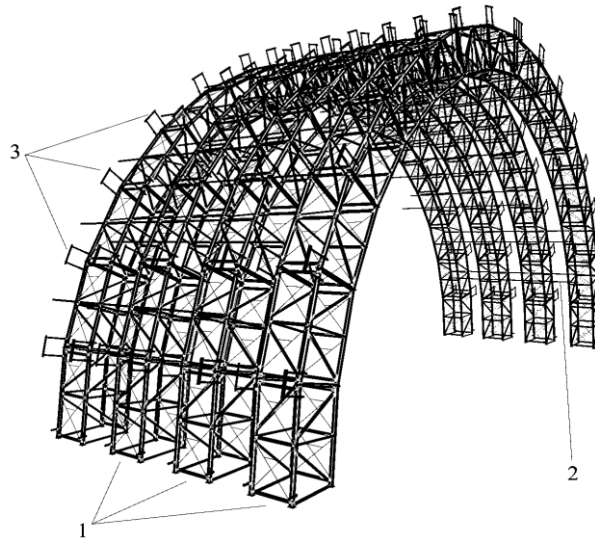


Fig. 1 – Steel typical scaffold S 23-26

The arched girders are composed of 16 segments (Fig. 2). Every segment represents a space truss formed by 4 two-dimensional trusses. Top (1) and bottom (2) truss have rectangular shape, and the two side trusses (3) have trapezoidal shape. The trusses are made of corrugated sheets 3 mm thick. The side trusses have also one tube (6) in the middle, with diameter of 48 mm which connects longitudinally several truss arched girders. The top and bottom truss have diagonally placed reinforcement bars (7) with a cross bar (8) which serve as ladders for workers, and at the same time they stiffen the truss. On the top truss are also welded tubes (9) for erecting the "P" girder. Joint of the top and bottom truss with the side trusses is made by bolts through the holes (10) on the free end of the U-profile. At the ends of every two-dimensional truss are flanges (4) which connect the segments. Between the flanges of two nearby segments are placed joint plates (5) and join by bolts into a compact unit.

Connecting of the arched girder is performed in downcast position on wooden planks rested on concrete slab. After assembling of the whole arch, wheels are mounted at specific places, so the scaffold can be moved along the slab and set into a suitable position for erection by crane.

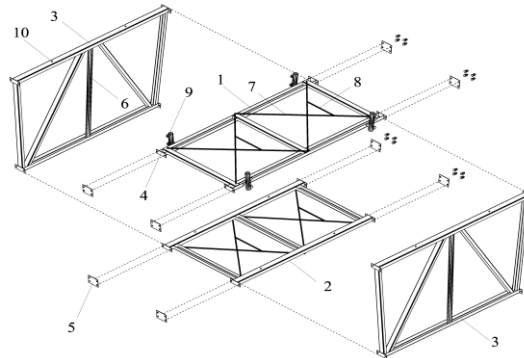


Fig. 2 – Elements of the truss arched girder

2 RESULTS OF THE EXPERIMENTAL ANALYSIS

2.1 SELECTION OF A SAMPLE (STEEL TRUSS ARCH) FOR TESTING

A total of four truss arches were erected, and for the testing was chosen one of the inner arches, which, by static analysis, withstands higher influences regarding the peripheral arches.



Fig. 3 – General view of the tested scaffold and applying of load (FC channel units).

2.2 EXPERIMENTAL PROCEDURE

The testing was conducted according to the regulations for this type of operation, i.e., by standard SRPS U. M1. 047. Measuring was done for influences due to static load, namely in two basic phases: 1) influence of load by ferrocement elements, 2) influence of symmetrically applied supplant (testing) load from RC arches, and from the longitudinal beam (ridge beam) in the crown of arch. Loading of the first phase was applied by erection of ferrocement elements symmetrically, beginning from the ends towards the middle. Loading of the second phase was

realized by supplant load using concrete blocks, and platforms hung in the P-girder location, and it was done in increments until the maximum load.

2.3 SELECTION OF MEASURING SECTIONS

The selection of measuring sections was done according with maximal expected influences, based on static analysis of the structure. Selected characteristic sections with maximal expected forces (stresses) were section in the crown of arch (section α - α), then the side section on the third segment of the scaffold starting from ground (section β - β), and section in the support (section γ - γ). Global deformations were defined based on the measuring of vertical displacements (deflections) in the midspan and in $\frac{1}{4}$ of span. Testing of the ferrocement channel units was a subject of a separate report No. IK 41/14 .

2.4 EQUIPMENT USED

Setting of the measuring instruments was done according to the principle of section encircling by instruments. Adequate equipment for tracking of global and local deformations (stresses) in characteristic sections in which maximal influences obtained by calculation analysis was used. For the tracking of global displacements (deflections) displacement gauges – LVDT were used, and for dilatations in the members strain gauges. The transducers were connected to the measuring station using multi-channel measuring-acquisition system MGCplus and SPIDER8. Recording and data processing was performed by software package CATMAN.

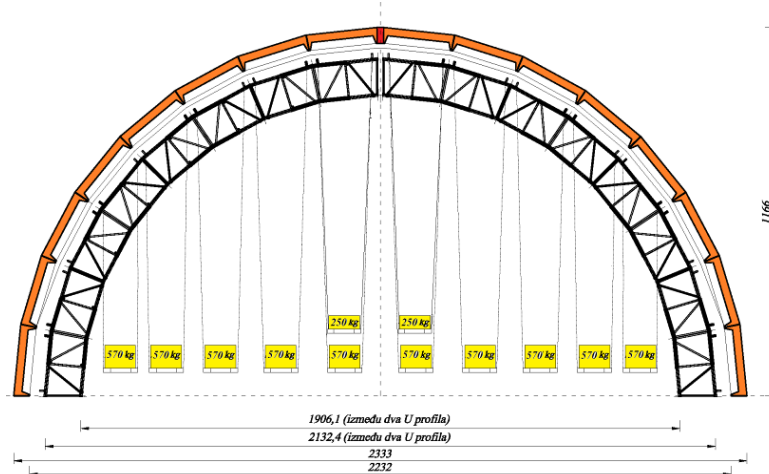


Fig. 4 – Scheme of load application in phases

I PHASE – Load from ferrocement elements, applied by symmetrical erection, starting from supports towards the middle (total of 16 elements of 1850 kg).

II-a PHASE – Load from RC arches symmetrically place starting from supports towards the middle (arch weight $g=5,0$ kN/m')

II-b PHASE – Load from RC longitudinal beam (ridge beam) placed in the middle (ridge) segment ($g=2,5 \text{ kN/m}'$)

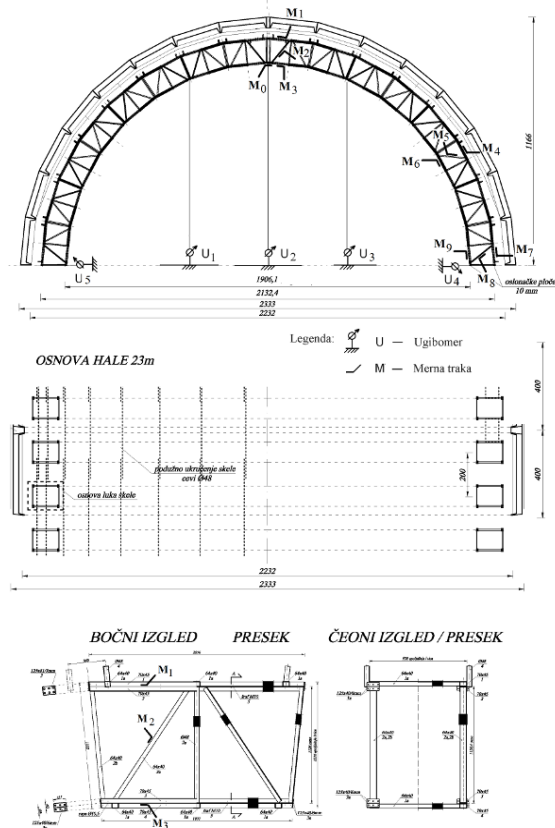


Fig. 5 – Setting of instruments at static load

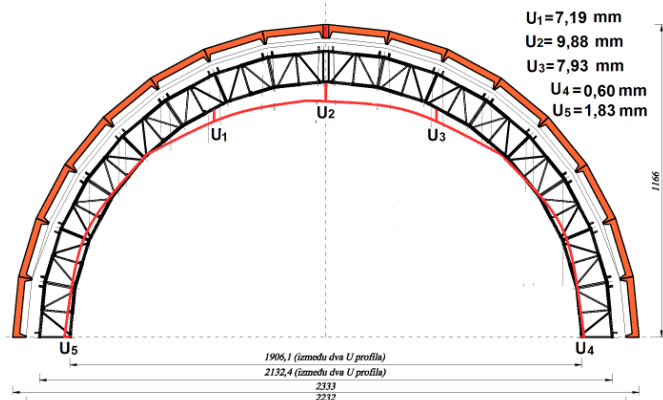


Fig. 6 – Diagram of measured deflections in the phase I; U [mm].

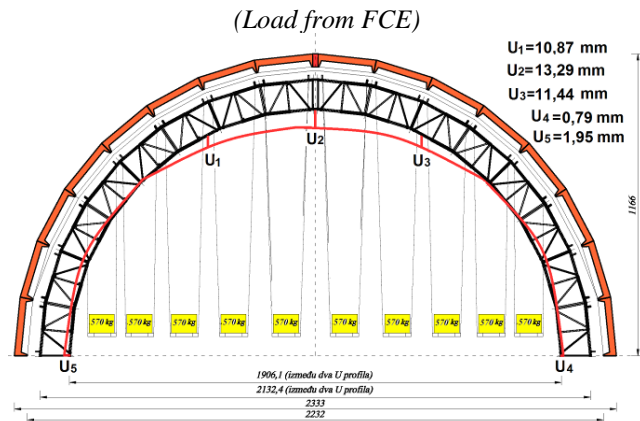


Figure 7 – Diagram of measured deflections in the phase II-a; U [mm]
(Load from FCE + RC arch)

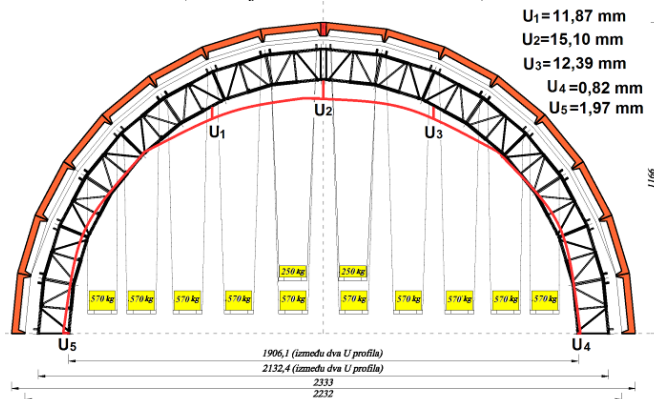


Fig. 8 – Diagram of measured deflections in the phase II-b – Symmetry; U [mm]
(Load from FCE+RC arch+RC ridge beam)

Graphical presentation of the measured dilatations (stresses)

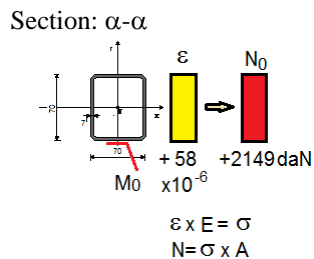


Fig. 9 – Diagrams of measured total dilatations and axial force N_0 in the section $\alpha-\alpha$
(I+II-a+II-b phase)

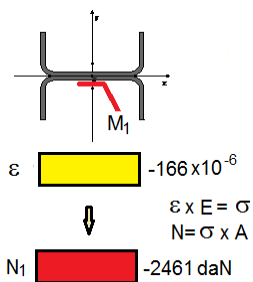
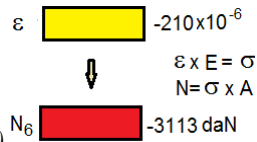
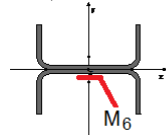
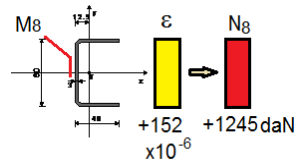


Fig. 10 – Diagrams of measured total dilatations and axial force N_1 in the section $\alpha-\alpha$

(I+II-a+II-b phase) section $\alpha\text{-}\alpha$ (I+II-



a+II-b phase)
 Fig. 11 – Diagrams of measured total dilatations and axial force N_6 in the section $\beta\text{-}\beta$ (I+ II-a+II-b phase)



$$\epsilon \times E = \sigma$$

$$N = \sigma \times A$$

Fig. 12 – Diagrams of measured total dilatations and axial force N_8 in the section $\gamma\text{-}\gamma$ (I+ II-a+II-b phase)

3 RESULTS OF THE NUMERICAL ANALYSIS

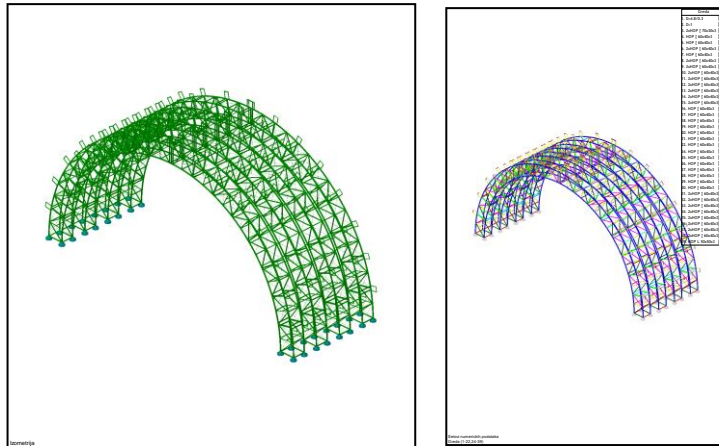


Fig. 13 - Calculation model of the tested scaffold

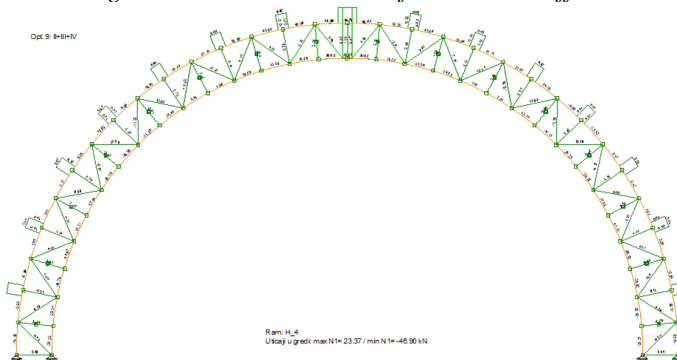


Fig. 14 – Member forces in the truss from the load FCE+RC arch+RC ridge beam

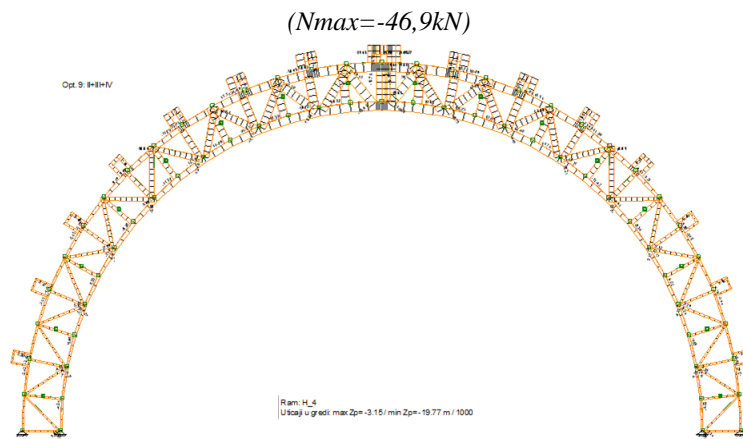


Fig. 15 – Deformation of the truss from the load FCE + RC arch+RC ridge beam
($Z_{max}=19,77$ mm)

4 CONCLUSION

Results of the testing of the typical scaffold for erection of typical halls made of RC arches with ferrocement filling (channel units) show that this type of construction is technically feasible and economically justified. Assembling elements of the scaffold are simply erected and easily transported, thus justifying its use. Required bearing capacity and deformability of the structure was proved by testing, and a satisfying safety factor was achieved. Measured displacements and member forces were lower than calculated, by which a condition for technically correct structure was realized. Behaviour of the structure was elastic, i.e., residual deformations after unloading were in the allowed limits for steel structures. Maximal measured stresses were allowed limits for the correspondent material.

Significance of such innovative solution is that a similar type of structure, with adequate adjusting, can be applied for other concrete assembling objects which are not self-supporting, i.e., they are monolithized on the site.

Acknowledgement:

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Dragan Majkic¹

WALTERDALE MOST U EDMONTONU, KANADA

Rezime:

S-60

Kada bude završen, novi Walterdale most će sačinjavati dva čelična luka koja će premostiti Saskatchewan reku u centralnom delu Edmontona, AB, Kanada. Za montažu lukova je izabrana veoma kompleksna operacija. Centralni deo lukova, približno 1/3 ukupne dužine lukova, je montiran na zemlji u samoj blizini mosta. Po montiranju, ovaj deo, težine 1000t, klizan je do reke i posebnom operacijom teret je prebačen na dve barže. Segment je sa barža podignut putem hidrauličkih džekova u poziciju za spajane sa prethodno pripremljenom konstrukcijom luka u dve faze. Allnorth je takodže razvio tehnologiju izvođenja kolovozne konstrukcije i konačnog podešavanja kablova zatega. I ovaj deo projekta kao i prethodni dao je interesantna tehnička rešenja.

Ključne reči: lučni most, tehnologija izvođenja, barže

WALTERDALE BRIDGE, EDMONTON, AB, CANADA

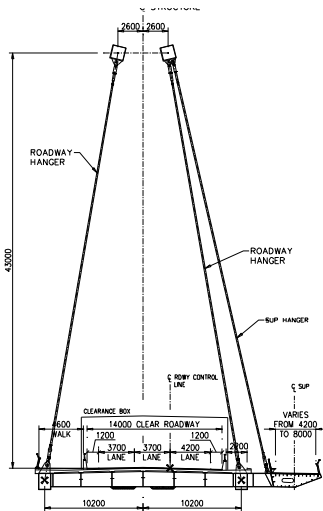
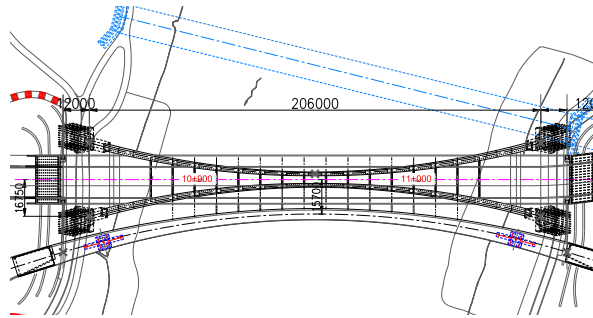
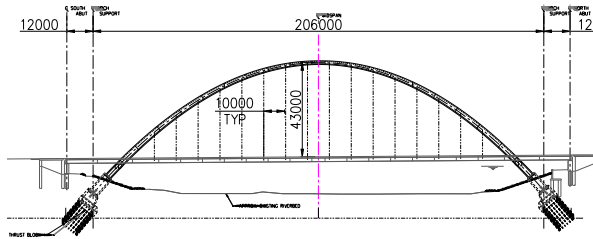
Summary:

The New Walterdale Bridge, when complete, will consist of a pair of steel arches across the Saskatchewan River in downtown Edmonton. From these arches, a steel deck will be suspended from 16 pairs of steel hanger cables. A crescent shaped pedestrian crossing will then hang adjacent to the bridge deck. This has been a very complex operation. Initially, the midspan of the arch, approximately one third of the total arch length, was installed adjacent to the location. This portion, weighing approximately 1000 tonnes was then skidded down Teflon rails onto barges on the icy Saskatchewan River. After arriving at this location, the midspan was lifted approximately 16 meters into a second intermediate location. Second heavy lift of approximately 2000 tonnes was carried out, bringing the arch to its final configuration. Allnorth has also designed the procedure for deck erect. There are interesting technical challenges ahead as well.

Key words: construction technology, arch bridge, scaffolding

¹ P.Eng., Allnorth Bridge Engineer Specialist, Vancouver, BC, Canada

1 OPIS KONSTRUKCIJE MOSTA



Slika 1 Dispozicija mosta

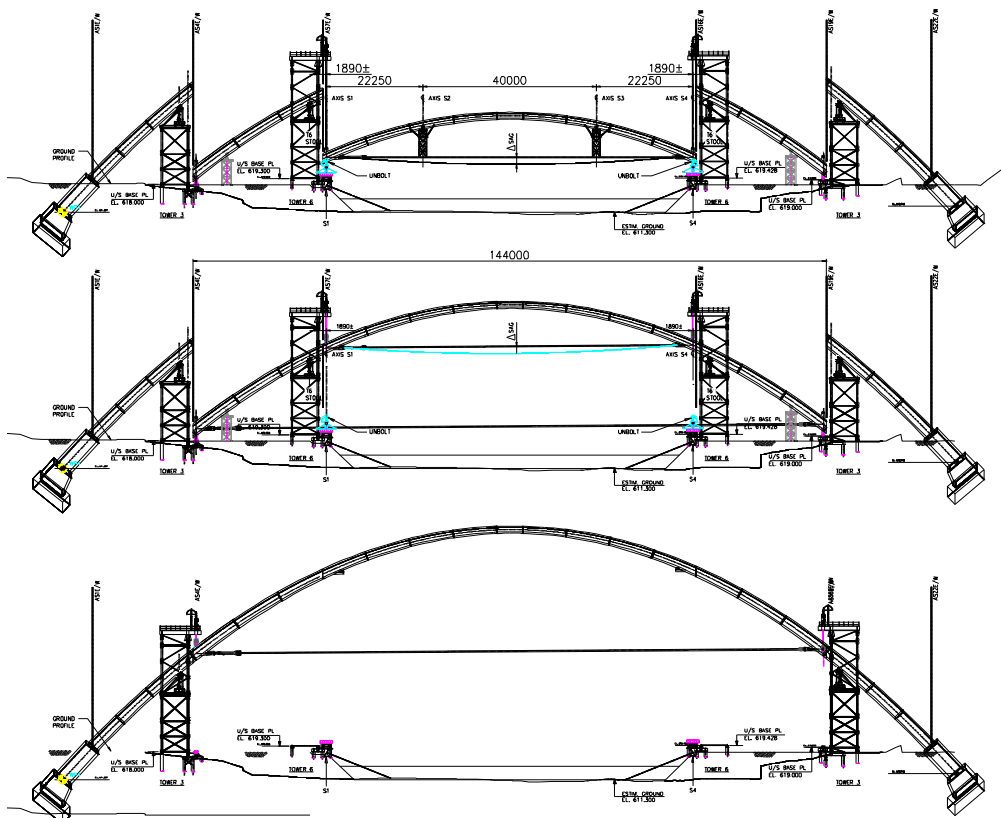
Osnovni konstruktivni sistem Walterdale mosta predstajaju uparena dva čelčna luka raspona 206 m, ukljestena u obalnim oporcima. Projektovana strela lukova je 43.0 m. Radi horizontalne stabilnosti lukovi su međusobno povezani celicnim rasponkama na svakih 10 m. Spregnuta kolovozna konstrukcija mosta je okacena putem celicnih zatega (kablova) u ritmu na svakih 10.0 m. Sa istočne strane mosta je projektovan horizontalno zakrivljen pesacki most okacen o istočni luk putem celicnih kablova i prislonjen na mostovsku kolovoznu konstrukciju putem poprecnih nosaca (Sl. 1). Kada bude završen, novi walterdale most ce imati tri saobracajne trake, jednom pesackom stazom na zapadnoj strani i pesackim mostom na istočnoj strani (Sl. 1 i Sl. 9)

2 TEHNOLOGIJA IZVODJENJA LUKOVA MOSTA

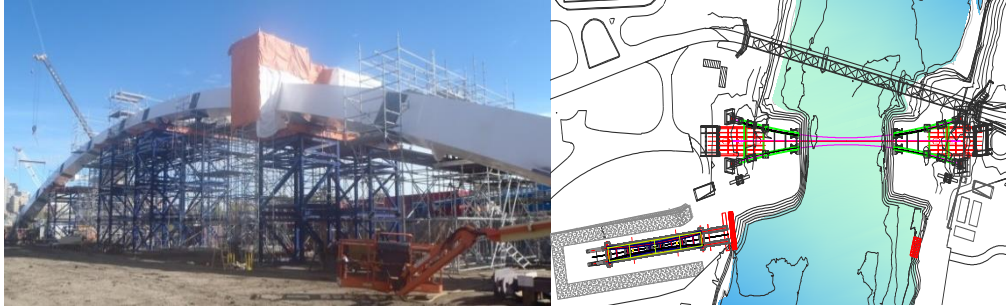
Walterdale most projekt sadrži izvodjenje novog lucnog mosta i demontazu postojećeg resetkastog mosta, izvedenog 1913g. Predmet ovog izlaganja bice tehtologija izvodjenja novog lucnog mosta.

Tehnologija izvodjenja mosta moze da se podeli u cetiri celine:

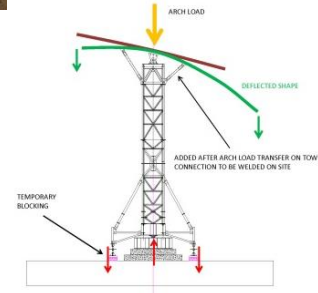
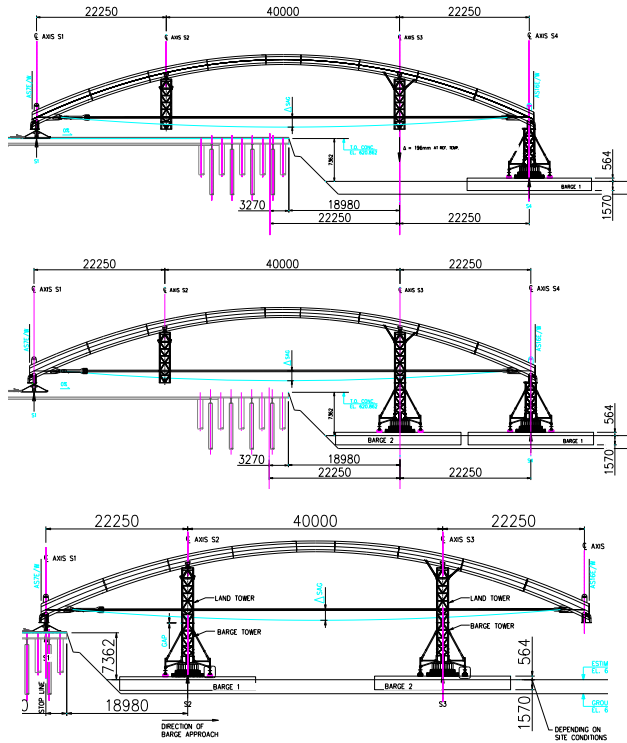
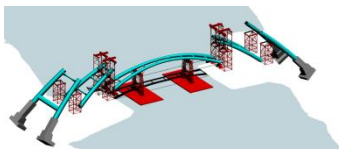
- Izvodjenje lukova mosta
- Izvodjenje kolovozne konstrukcije mosta
- Izvodjenje pesackog mosta
- Finalno podesavanje zatega



Slika 2 Shema tehnologije izvodjenja lukova mosta



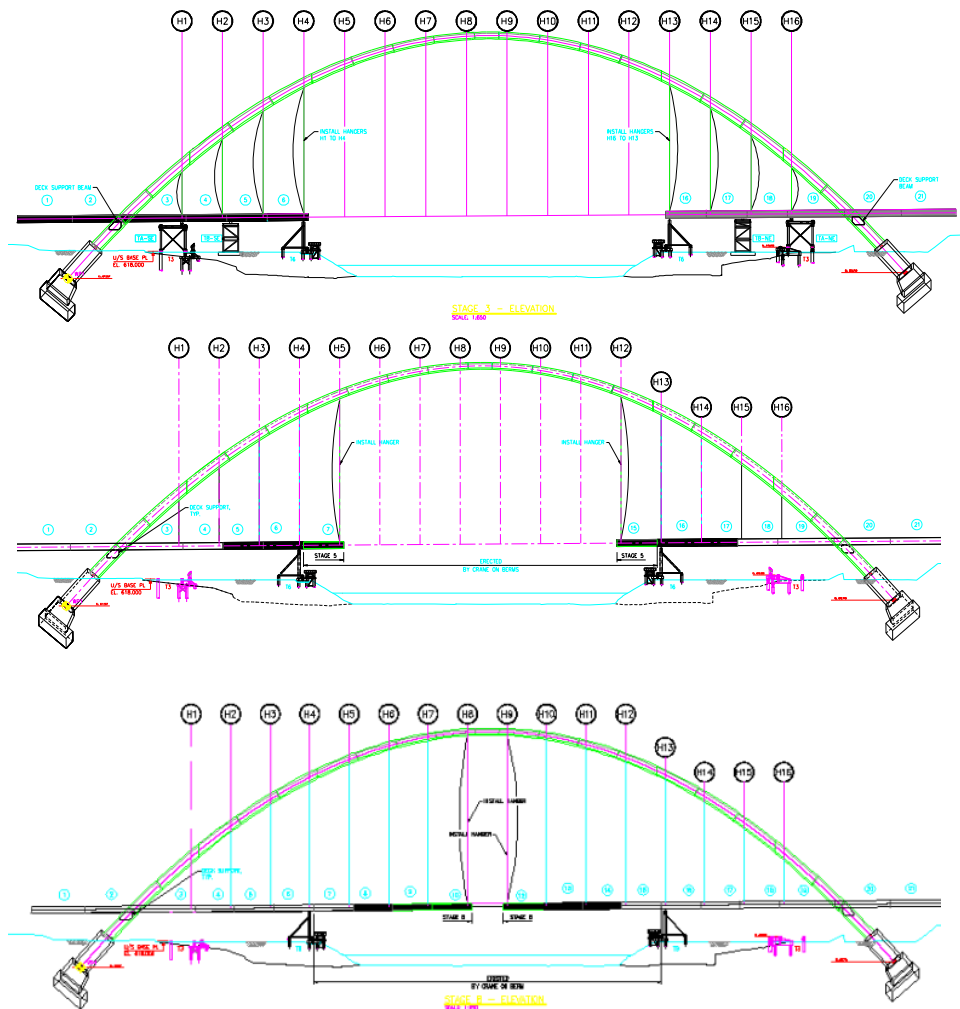
Slika 3 Montaza centralnog dela lukova na zemlji



Sl. 4 Prenos centralnog dela lukova sa obale na barze

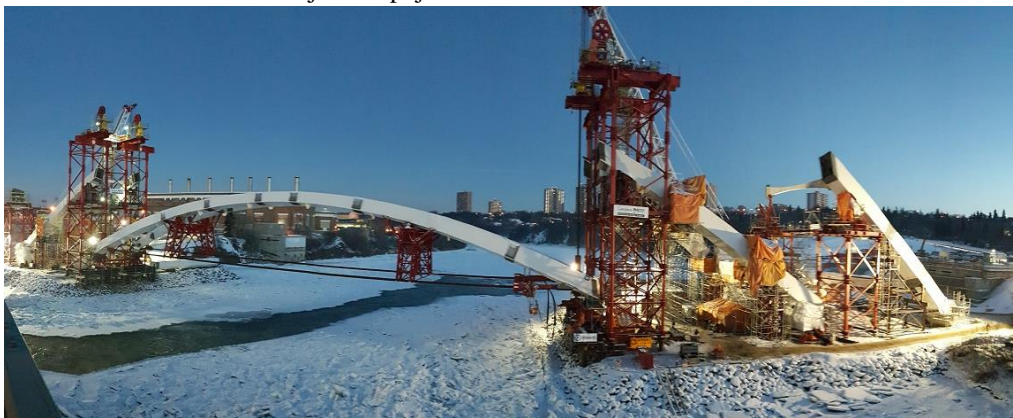
Izvodjenje lukova je uradjeno u cetiri faze (Sl 2):

- Izvodjenje centralnog dela lukova (88.0 m)
- Klizanje centralnog dela lukova
- Prenos tereta na barze.
- Podizanje prvog segmenta lukova
- Podizanje drugog segmenta lukova (144m)



Sl.5 Shema tehnologije montaze kolovozne konstrukcije

U prvoj fazi, centralni deo lukova, (88m) približno 1/3 ukupne duzine lukova, je montiran na obali u samoj blizini mosta. Montaza je obavljena tradicionalnom metodom koriscenjem skele. Po montiranju ovaj deo, tezine 1000t, je klizan do reke i posebnom operacijom teret je prebacen na dve barze. Klizanje i prenos na barze je omoguceno pomocu dodatih konstruktivnih elemenata: zatega i dva privremena stuba (Sl 4). Stubovi za klizanje su bili projektovani iz dva dela: obalni stub i stub na barzi. U toku klizanja i prenosa tereta na barze stubovi su menjali konstruktivni sistem radi stabilnosti lukova i samih barzi, i iz razloga da se izbegnu zarobljeni naponi u vaznim konstruktivnim elementima. Sila u ztegama je bila promenljiva tokom prenosa tereta na barze 4400 kN do 220 kN. Izuzetan inzenjerski izazov je bio ekstremno nizak vodostaj reke i pojava leda.



Sl.6 Priprema za podizanje srednjeg dela mostovskih lukova (1000t)



Sl.7 Montaza prvog 88.0 m lucnog segmenta (1000t)

Koriscenjem 15 kablova i 15 fiksnih tacaka, na obali i na postojećim mostovskim stubovima u vodi, barze su dovedene u poziciju za montazu mosta. Tu su već pripremljeni segmenti lukova, oslonjeni na obalne stubove, cekali spremni za montazu sa centralnim delom. Operacija montaze lukova u jednu celinu je obavljena u dve faze dizanja (Sl. 2) pomoću hidrauličkih kablovskih dizalica kapaciteta 8000 kN. Prvo je oformljen luk od 144m sa novom zategom, a potom luk od 206m. Zbog izuzetno visoke geometriske tolerancije (25mm), za svaku od ovih operacija izvodjenja radjena je temperaturna i tezinska geometrijska korekcija uzimajući u obzir faze izvodjenja konstrukcije. Maksimalna sila u zategi za podizanje segmenta od 144m i tezine 2000t je bila 5200 kN. Projektom montaze izracunata teoretska krutost luka je kasnije proverena na licu mesta: Promena distance krajeva luka od 144m je 120mm za promenu sile u zategi za 45kN.

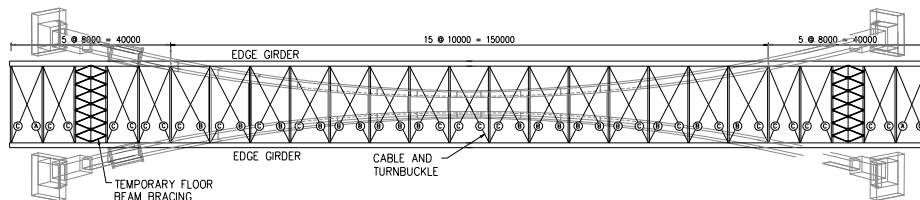
3 TEHNOLOGIJA IZVODJENJA KOLOVOZNE KONSTRUKCIJE

Za izvodjenja kolovozne konstrukcije primenjene su dve tehnologije (Sl. 5):

- Deo iznad nasutog dela sa izvodjenjem pomocu skele
- Deo iznad reke konzolna montaza koriscenjem kрана sa pozicijom na nasutom delu

Deo kolovozne konstrukcije u zoni nasutog dela je montiran sa nadvisenjem radi lakse montaze kablova (zatega), montaza bez uredjaja za prednaprezanje. Zatege su aktivirane u kasnijoj fazi pomocu spustanja kolovozne konstrukcije, spustanja skele koriscenjem hidrauličkih dizalica. U zoni iznad reke, gde je primenjena konzolna metoda izvodjenja, koriscen je uredjaj za prednaprezanje (Sl. 6)

Horizontalna stabilnost kolovozne konstrukcije u toku izvodjenja je postignuta pomocu dodatnih privremenih kablovskih spregova



Sl.8 Shema privremenog horizontalnog sprega za vreme montaze

Nakon montaze konstruktivnog celika kolovozne konstrukcije betonirana je kolovozna ploca. Iz razloga smanjenja uticaja od skupljanja betona betoniranje kolovozne ploce je predvidjeno projektom u cetiri faze.



Sl.9 Skela za kolovoznu konstrukciju na nasutom delu



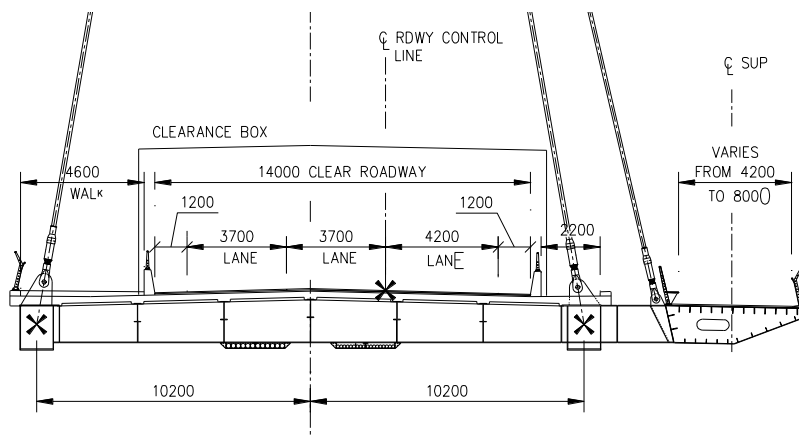
Sl.10 Montaza celicne kolovozne konstrukcije

Pesacki most je zakrivljeni celicni sanducasti presek okacen putem celicnih zatega o glavni istocni luk mostovske konstrukcije i prislonjen na kolovoznu konstrukciju mosta. Montaza pesackog mosta je predvidjena po zavrsetku betoniranja kolovozne ploce.

Isto kao za montazu kolovozne konstrukcije, za izvodjenja pesackog mosta primenjene su dve tehnologije (Sl. 5):

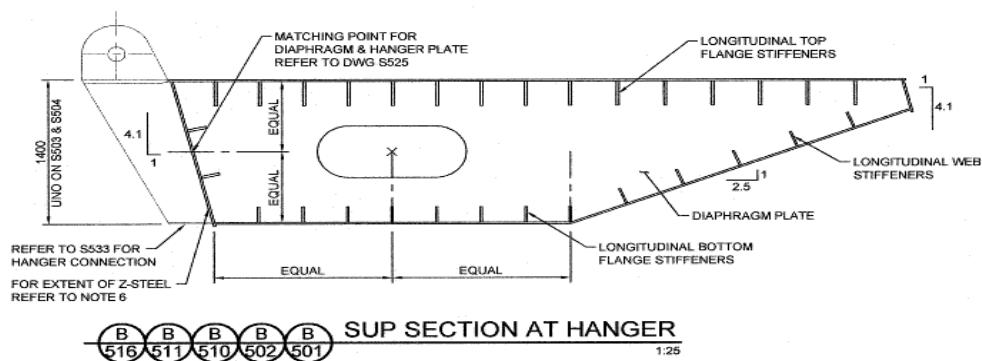
- Deo iznad nasutog dela sa izvodjenjem pomocu skele
- Deo iznad reke konzolna montaza koriscenjem kрана sa pozicijom na nasutom delu

Deo iznad reke je predvidjen da se montira pomocu kранаova (500t) lociranih na nasutom terenu.

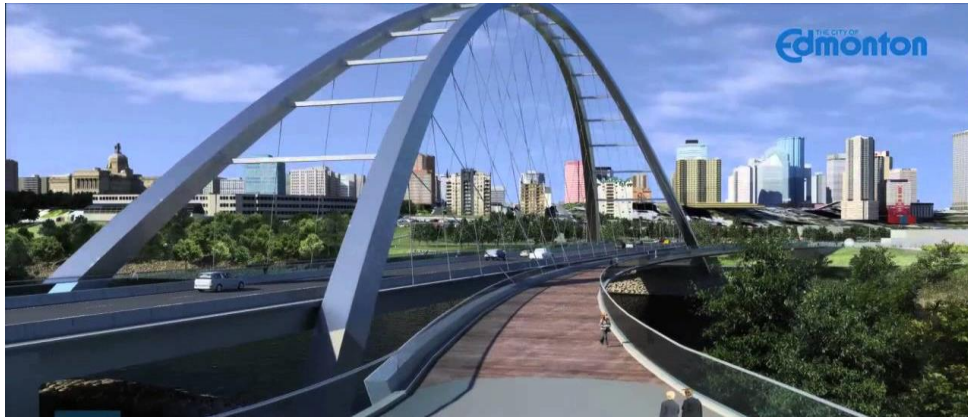


Sl.11 Tipican presek sa pesakim mostom

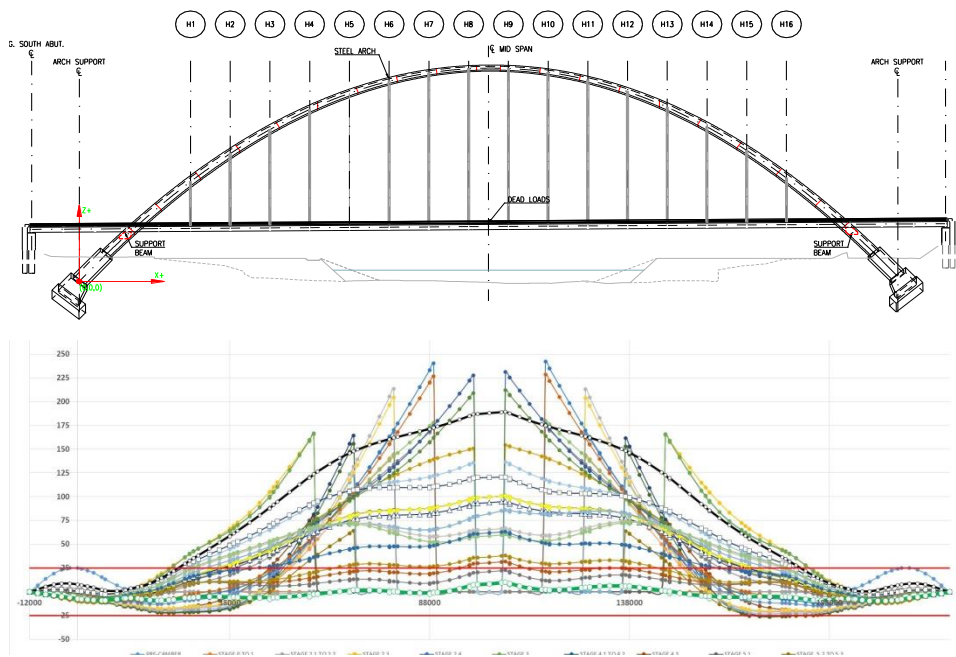
Zakrivljeni pesacki most je trapeziodni celicni sanducasti presek okacen u svom sredisnjem rasponu o istocni luk putem celicnih zatega. Sanducasti presek pesackog mosta konstantne visine 1400mm i promenljive sirine 4776 mm u centru raspona i 8899mm na krajevima. Pesacki most je oslonjen na dva stuba “delta” oblika, lociranim u cetvrtinama raspona i oslonjen na drumski most putem 8 propustenih poprecnih kolovoznih greda



Sl.12 Tipican presek pesackog mosta



Sl.13 Pogled na pesacki most sa juzne strane



Sl.14 Deformacija kolovozne konstrukcije u toku faza izvodjenja

Reference

Investitor: The City of Edmonton

Main Contractor : Acciona Infrastructure & Pacer Corporation

Projekt tehnologije izvodjenja: Allnorth

Projekat mosta: Dialog & COWI (Buckland & Taylor)

Saša Plisnić¹, Goran Radeka², Vladimir Vukobratović³

MODELIRANJE INDUSTRIJSKE HALE, KONSTRUKTIVNI SISTEM I IZVOĐENJE OBJEKTA BEZ TEMPERATURNIH DILATACIJA

Rezime:

S-61 Rad prikazuje izvod iz projekta konstrukcije proizvodne hale „LEAR“ koja se nalazi u radnoj zoni Sever IV o Novom Sadu. Pored standardno apliciranih opterećenja koja su uobičajena za industrijske objekte, u radu je prikazan uticaj temperaturnog opterećenja na noseću konstrukciju objekta. Projektovanje objekata sa savremenim zahtevima po pitanju energetske efikasnosti, mogu da budu od ključnog značaja za nosivost i trajnost konstrukcije i saveznik u prevazilaženju temperaturnih kolebanja koja zahtevaju jedan ozbiljan inženjerski pristup. Takođe, tehnologija izvođenja i fazna montaža čelične krovne konstrukcije, značajno doprinosi smanjenju presečnih sila i pomeranju vertikalne noseće konstrukcije.

Ključne reči: proračunski model, opterećenje, temperaturni uticaji i dilatacije,

DESIGN MODEL OF INDUSTRIAL HALL, STRUCTURAL SYSTEM AND CONSTRUCTION WITHOUT TEMPERATURE DILATATIONS

Summary:

This publication shows excerpt from the Structural Design of the production “LEAR” hall, located in Novi Sad industrial zone North IV. In addition to standard loads that are common for industrial buildings, this article shows the influence of temperature loads on the building supporting structure. Designing of buildings with modern requirements in terms of energy efficiency, could be of key importance for load capacity and durability, and an ally in overcoming of temperature fluctuations which require a serious engineering approach. In addition, construction technology and construction phases of roof steel structure can significantly reduce the cross-sectional forces and deformations the vertical supporting structure.

Key words: calculation model, load, temperature influences and dilatations

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1 UVOD

Za potrebe američke kompanije "LEAR" kao budućeg korisnika objekta, u Novom Sadu je planirano infrastrukturno uređenje radne zone Sever IV i gradnja proizvodnog pogona u kojoj će ova kompanija zaposliti oko 2500 radnika. Osnovna delatnost kompanije je proizvodnja automobilskih sedišta, grejača sedišta kao i električnih kablova sa primenom u automobilskoj industriji. U veoma kratkom roku je trebalo izraditi svu neophodnu tehničku dokumentaciju uključujući Idejno rešenje, Projekat za građevinsku dozvolu i Projekat za izvođenje. Izgradnja ovog značajnog privrednog objekta je trenutno u toku, a stepen izgrađenosti je cca 90%. U ovom radu želimo da prezentujemo konstruktivne pojedinosti, nesvakidašnje iskustvo i specifičnosti tokom projektovanja, kao i neke aspekte u fazi izvođenja proizvodne hale.



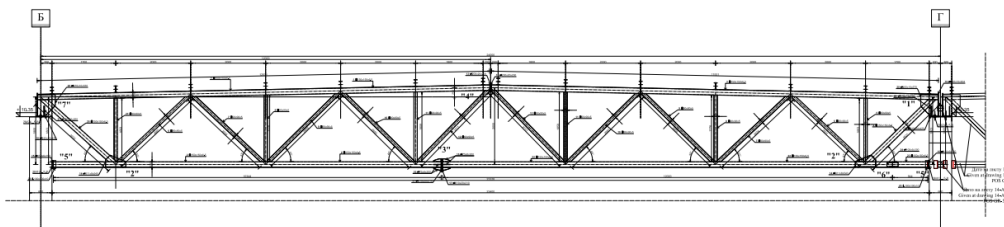
Slika 1 – Renderovani 3D prikaz proizvodne hale "Lear"

2 OPIS KONSTRUKCIJE OBJEKTA

Proizvodna hala ima ukupnu površinu od 28000 m² i sastoji se od centralnog proizvodnog pogona sa dimenzijama osnove 156x144 m, dva aneksa na fasadi objekta sa dimenzijama 12x96 m i 12x66 m i administrativnog dela na sprat čije su dimenzije 12x144 m. Za vertikalnu noseću konstrukciju u unutrašnjosti objekta usvojen je pravougaoni raster prefabrikovanih AB stubova od 12x24 m. AB stubovi poprečnog preseka 60/60 cm su svojim donjim delom uklješteni u prefabrikovane AB temeljne čašice i zglobno vezani u svojim vrhovima sa čeličnim rešetkastim krovim nosačima u obo ortogonalna pravca. Na delovima objekta gde je projektovan sprat, pored prefabrikovanih AB stubova, predviđeni su i čelični stubovi kroz jednu etažu kao međuoslonci za podvlake spregnute tavanice sprata raspona L=12 m.

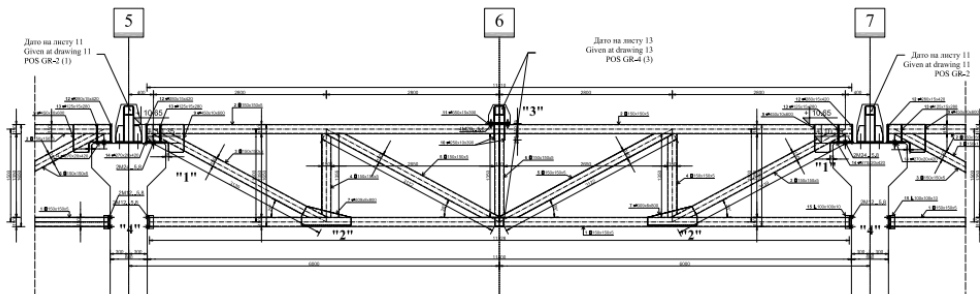
Na fasadi objekta zbog zahtevane vatrootpornosti i smanjenja raspona fasadnih rigli, predviđeni su prefabrikovani AB stubovi na međuosovinskom rastojanju od 6,0 m sa dimenzijama poprečnog preseka 40/60 cm. Fasadni vertikalni paneli debljine 15 cm oslanjaju se na fasadne rigle cevastog poprečnog preseka i zglobno povezuju vijčanim vezama za čelične stolice formirane na AB fasadnim stubovima. Stabilnost objekta u podužnim i poprečnim fasadnim ravnima, obezbeđena je vertikalnim čeličnim spregovima sa ukrštenim dijagonalama od UNP-100 profila.

Za krovnu konstrukciju objekta izbran je sistem sa rešetkastim čeličnim nosačima postavljenim u dva ortogonalna pravca. Rešetkasti nosači POS GR-1, GR-2, GR-3 i GR4 imaju raspon od $L=24$ m i postavljaju se u pravcu brojnih osa objekta. Na ovaj način, zajedno ove rešetke formiraju pravougaoni raster od 6×24 m koji je idealan za montažu čeličnih rožnjača od gotovih kutijastih profila sa rasponom od $L=6$ m. Gornji i donji pojasevi ovih rešetkastih nosača uključujući i krajnje zategnute dijagonale u blizini oslonaca, predviđeni su od čelične cevi kvadratnog poprečnog preseka $\square 150 \times 150 \times 6$. Ispuna rešetkastih nosača (dijagonale i vertikale) projektovana je od čeličnih cevi kvadratnog poprečnog preseka $\square 90 \times 90 \times 5$. Rešetkasti nosači POS GR-1 i POS GR-2 direktno se oslanjaju na vrhove AB stubova u kojima su prilikom prefabrikacije ugrađene šablon ploče sa ankerima od 2M24...5.8 za formiranje zlobne veze rešetke i stuba. Zbog dužine rešetke od 24 m i nemogućnosti transporta iz radionice do gradilišta u jednom komadu, predviđeni su montažni nastavci gornjeg i donjeg pojasa u vijčanoj izvedbi sa vijcima M24... 10.9 klase čvrstoće sa momentom pritezanja vijka od 250 Nm. Donji pojas rešetke je ravan i postavljen je na visini 9,0 m u odnosu na pod prizemlja hale. Gornji pojas rešetke ima oblik dvovodnog krova sa padovima od 2% usmerenim od slemena rešetke prema uvalama krova. Rešetkasti nosač na svom oslonačkom kraju ima visinu od 1800 mm, a njegova visina na sredini raspona (u slemenu) je 2050 mm.



Slika 2 – Rešetkasti nosač POS GR-1 raspona $L=24$ m

Rešetkasta podvlaka POS GR-5 sa rasponom od $L=12$ m je jedina koja se postavlja u pravcu slovnih osa objekta i na taj način formira pravougaoni raster 12×24 m. U nivou gornjeg pojasa rešetkaste podvlake POS GR-5, tačno na mestu iznad vertikale smeštene na sredini raspona ove rešetke, formiran je oslonac za naleganje krajeva rešetkastih nosača POS GR-3 i POS GR-4. Oslonci rešetkaste podvlake POS GR-5 su AB kratki elementi formirani na vrhovima unutrašnjih i fasadnih AB stubova preseka 60/60. S obzirom na velike reakcije koje podvlaka prihvata, svi štapovi ispune i svi štapovi gornjeg i donjeg pojasa POS GR-5 su predviđeni od čelične cevi kvadratnog poprečnog preseka $\square 150 \times 150 \times 5$. Krajevi donjeg pojasa svih pozicija rešetkastih nosača se približavaju AB stubovima (kao i donjem pojasu rešetkastih podvlaka POS GR-5) na udaljenost od 30 mm, a zatim se posebno konstruisanom vezom sprečava horizontalno pomeranje upravno na ravan rešetke.

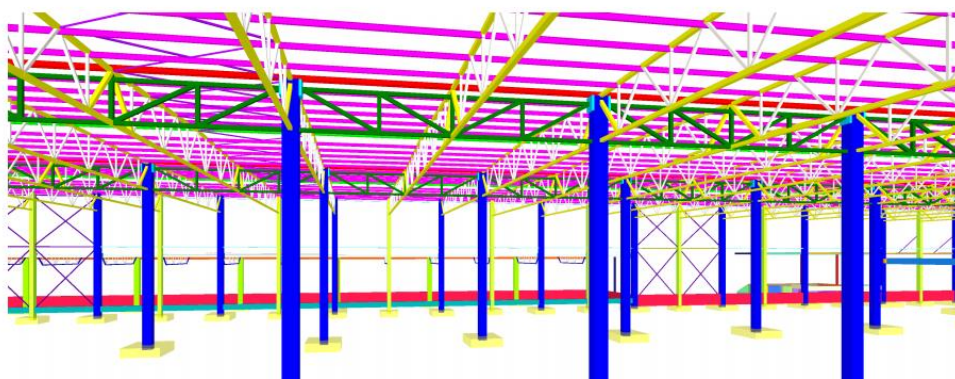


Slika 3 – Rešetkasta podvlaka POS GR-5 raspona $L=12$ m

Upravno na ravan rešetkastih nosača POS GR-1 do GR-4 postavljene su čelične rožnjače od [200x100x5 cevastog profila oslanjaju na mestima čvorova u kojima se sučeljavaju štapovi ispunje i gornji pojas rešetke. Veza rožnjača sa rešetkastim nosačima urađena je u vijčanoj izvedbi sa 2M12...10.9 vijcima bez sile pritezanja. Upravno na pravac pružanja rožnjača montira se trapezni čelični plastificirani lim sa visinom grebena od 60 mm. Preko trapeznog lima postavlja se kamena vuna konstantne debljine od 24 cm, a preko nje kao završni krovni pokrivač polaže se krovna membrana Sika plan 15G. Uvale koje se formiraju u osama u kojima su oslonjeni rešetkasti nosači raspona 24 m, nemaju svoj podužni pad kojim se odvodi kiša sa krova, već je odmah ispod uvale u unutrašnjosti objekta projektovan cevasti razvod "pluvija" vakumskog sistema. Međuspratna tavanica u administrativnom delu objekta predviđena je kao spregnuta tavanica sa rasponom od 12 m. Spregnut presek čine podni čelični nosači od IPE 500 profila i AB ploča konstantne debljine. Čelični podni nosači postavljeni su na međusovinskom rastojanju od $\lambda=3,0$ m. Sprezanje čeličnog nosača i AB ploče izvršeno je sa cilindričnim moždanicima $\Phi 19/100$ mm (čepovi sa glavom od čelika čvrstoće 450 N/mm²). AB ploča spregnute tavanice ima debljinu 12 cm i u statičkom smislu predstavlja kontinualnu ploču raspona 3,0 m koja nosi u jednom pravcu. Prilikom izvođenja spregnute tavanice u fazi betoniranja ploče svežom betonom kvaliteta MB 30, nije potrebno podupiranje čeličnih nosača, a oplatu ploče treba osloniti preko donjih flanši čeličnog spregnutih nosača. Fundiranje objekta je predviđeno da se izvede na AB temeljima samcima čije su dimenzije u osnovi 250x350 cm, 350x350 cm i 200x250 cm. Uklještenje AB stubova u temelje samce obavlja se pomoću prefabrikovanih čašica koje se postavljaju i spuštaju u svoj projektovani položaj, a zatim se na licu mesta obavlja armiranje i betoniranje stopa temelja samaca. U skladu sa zahtevima Investitora i budućeg korisnika objekta (američka kompanija LEAR), potrebno je pod prizemlja hale visinski postaviti na relativnu kotu od -1,20 m u odnosu na teren i saobraćajnicu koja se nalazi neposredno uz fasade objekta. Zbog ovog izdizanja poda hale u unutrašnjosti objekta, AB parapetne grede pored uloge oslonca fasadnih panela, moraju da imaju i ulogu potpornog zida kojim se prihvataju horizontalni pritisci od nasipa ispod podne ploče objekta. Oslonac parapetnih greda je vrh čašice stuba, a prihvatanje horizontalnih reakcija od pritisaka tla obavlja se zahvaljujući čeličnim sidrima ugrađenim u prefabrikovane parapetne grede i AB prefabrikovane fasadne stubove.

3 PRORAČUNSKI MODEL

Modeliranje konstrukcije objekta urađeno je primenom programskog paketa Tower 7 u kome je formiran prostorni model sastavljen pretežno od linijskih elementa koji imaju svoju geometriju poprečnog preseka i odgovarjuće mehaničke karakteristike. Model konstrukcije je obuhvatio AB stubove kao dominantnu vertikalnu noseću konstrukciju, čeličnu krovnu konstrukciju sastavljenu od rešetkastih nosača u dva ortogonalna pravca, rožnjače i spregove u nivou krovne ravni, fasadne rigle i vertikalne spregove za ukrućenje. Za realnije ponašanje i sagledavanje konstrukcije u celosti, vertikalna noseća konstrukcija je oslonjena na temeljnu konstrukciju sa elastičnom Winkler-ovom podlogom krutosti $15000 \text{ kN/m}^2/\text{m}$.



Slika 4 – 3D pogled na segment konstrukcije iz proračunskog modela

Kao osnovni slučajevi opterećenja, na modelu su aplicirana sledeća opterećenja:

- sopstvena težina konstruktivnih elemenata koja se automatski generiše,
- stalna opterećenja na krovnoj i fasadnoj konstrukciji ,
- sneg na krovu objekta
- četiri slučaja opterećenja vetrom na konstrukciju fasade i krova (+X, -X, +Y i -Y pravac)
- seizmika u X i Y pravcu kao alternativno opterećenje
- temperaturne promene u osama linijskih elemenata proračunskog modela

4 ANALIZA TEMPERATURNOG OPTEREĆENJA

Za objekat sa nestandardnim dužinama osnove, uticaj temperature na konstrukciju objekta je od presudnog značaja kako u izboru statičkog sistema, tako i u dispoziciji vertikalne noseće konstrukcije sa svim neophodnim elementima za prostornu stabilnost objekta. Od početka je bilo evidentno, da bi postojanje dilatacija uticalo na funkcionalnost objekta, značajnije izdvajanje finansijskih sredstva za izradu dopunskih pozicija konstrukcije, dodatne troškove u održavanju objekta i konačno na vremenske rokove u izgradnji objekta. Kod određivanja temperaturnog opterećenja, razlikuju se dva slučaja:

- potpuno funkcionalan i zatvoren objekat (eksploatacija),
- faza montaže noseće armiranobetonske i čelične konstrukcije

Posmatrajući objekat u toku njegove eksploatacije za vreme merodavnog letnjeg perioda, potrebno je odrediti temperaturnu promenu koja se uniformno raspoređuje kroz poprečni presek konstruktivnog elementa. Ona je data sa sledećim izrazom:

$$\Delta T_u = T - T_0 \quad (1)$$

gde je:

ΔT_u temperaturna promena u osi štapa,

T srednja temperatura konstruktivnog elementa,

T_0 početna temperatura u toku izvođenja konstrukcije,

Srednja vrednost temperature T se proračunava kao vrednost letnje ili zimske temperature posmatranog elementa konstrukcije i data je pomoću izraza:

$$T = \frac{T_{out} + T_{in}}{2} \quad (2)$$

gde je:

T_{out} temperatura spoljašnje okoline,

T_{in} temperatura unutrašnje okoline,

Kao ulazni podatak za analizu temperaturnog opterećenja, poslužio nam je podatak definisan projektnim zadatkom u kome je temperatura u unutrašnjosti objekta $T_{in} = 25^{\circ}C$ (letnji i zimski period). Početna temperatura (srednja temperatura) u toku izvođenja konstrukcije je pretpostavljena u vrednosti $T_0 = 17^{\circ}C$. Temperatura spoljašnje okoline u letnjem periodu u funkciji je refleksije izložene površine sa jugozapadnom orijentacijom i data je sa izrazom:

$$T_{out} = T_{max} + T_4 = T_{max} + 30^{\circ}C \quad (3)$$

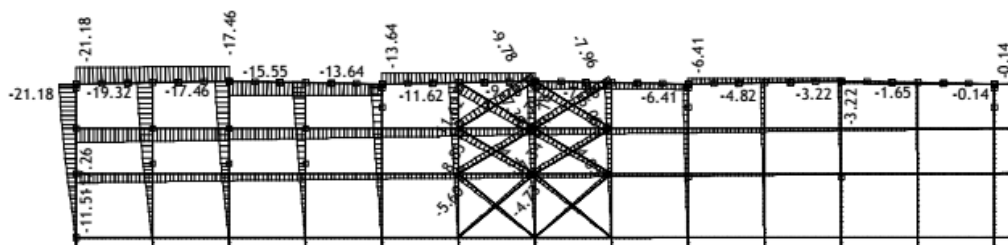
gde je:

$T_{max} = 39^{\circ}C$ maksimalna temperaturna na lokaciji objekta (usvojena vrednost) ,

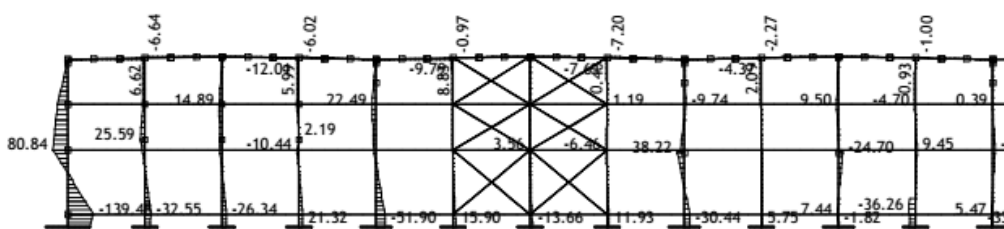
Prema izrazu (3) dobija se $T_{out} = 39 + 30 = 69^{\circ}C$ pa je prema izrazima (2) i (1)

$$T = \frac{69 + 25}{2} = 47^{\circ}C, \quad \Delta T_u = 47 - 17 = 30^{\circ}C$$

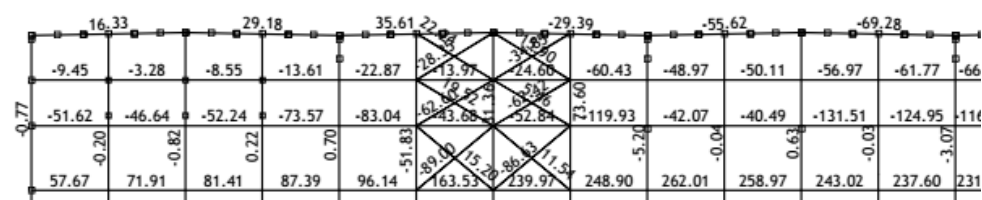
Aplicirajući dobijenu vrednost temperaturne promene na linijske elemente iz proračunskog modela, dobijaju se dijagrami presečnih sila i deformacija na fasadnom ramu (slike 5,6, i 7), i karakterističnom unutrašnjem ramu (slike 8 i 9) koji su izdvojeni iz prostornog modela konstrukcije. Zbog velike dužine ramova u odnosu na visinu objekta i čitkosti vrednosti ordinata presečnih sila i deformacije, posmatrana je samo jedna polovina fasadnog i unutrašnjeg rama, jer su presečne sile i deformacije skoro simetrične u odnosu na srednji stub rama.



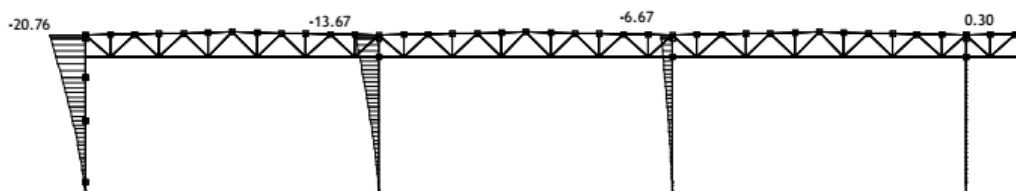
Slika 5 – Dijagram pomeranje u Y pravcu na fasadnom ramu usled $T=30^{\circ}\text{C}$ (min $Y=21,18\text{ mm}$)



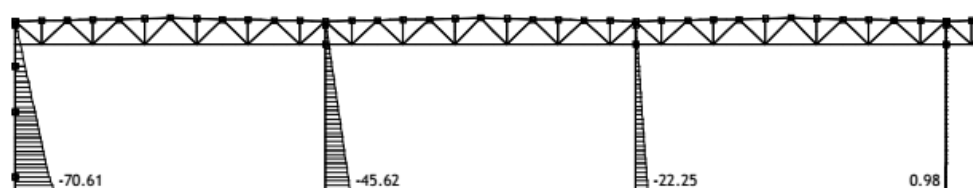
Slika 6 – Dijagram momenata savijanja AB stubova u ravni fasadnog rama, usled $T=30^{\circ}\text{C}$ (min $M=139,48\text{kNm}$) na ugaonom stubu u uklještenju u temelj



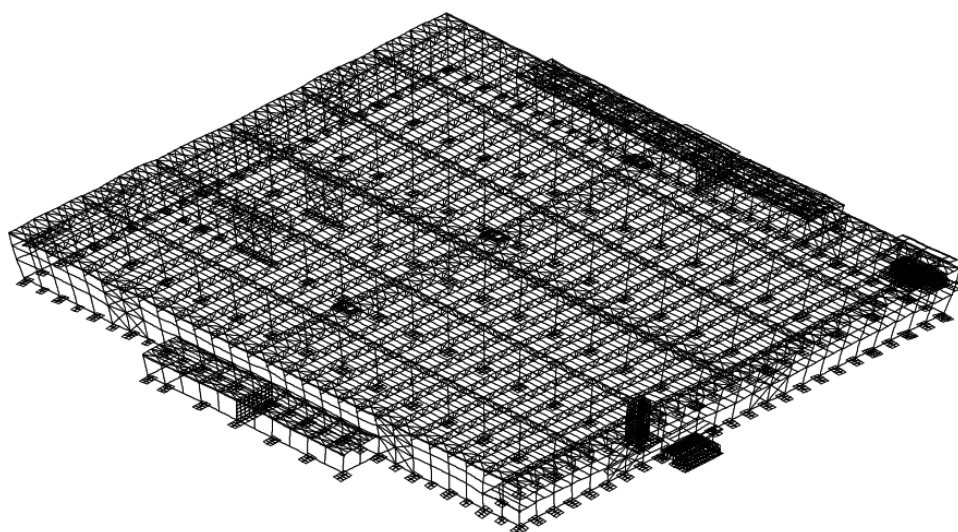
Slika 7 – Dijagram normalnih sila u fasadnom ramu, usled $T=30^{\circ}\text{C}$



Slika 8 – Dijagram pomeranja u Y pravcu unutrašnjeg rama usled $T=30^{\circ}\text{C}$ (min $Y=20,76\text{mm}$)



Slika 9 – Dijagram momenata savijanja u unutrašnjem rama, usled $T=30^{\circ}\text{C}$
min $M=70.61\text{kNm}$ na ivičnom AB stubu



Slika 10 – Deformacija kompletne konstrukcije usled $T=30^{\circ}\text{C}$

5 KONSTRUKCIJA OBJEKTA U FAZI IZVOĐENJA



Slika 11 –Parcijalna montaža čelične krovne konstrukcije, trapeznog lima i svetlosnih kupola



Slika 12 –Krovne rešetke raspona 24 m, rešetkaste podvlake raspona 12 m i spregnuta tavanica sprata na administrativnom delu objekta

6 ZAKLJUČCI

Analizirajući dobijene dijagrame presečnih sila i deformacija, možemo zaključiti sledeće:

- pomeranje konstruktivnih elemenata u nivou vrha objekta je malo i reda veličine $\delta=H/527=11500/527=21,82$ mm, gde je H visina od mesta uklještenja AB stuba do ravni krovne konstrukcije.
- raspored konstruktivnih elemenata u osnovi je pravilno raspoređen. Aksijalna krutost krovne konstrukcije zajedno sa krovnim spregovima omogućuje distribuciju i prenošenje horizontalnih sila na dovoljan broj AB stubova (fleksiona krutost) i vertikalnih spregova postavljenih na fasadama i u unutrašnjosti objekta (velika aksijalna krutost). Ne treba zaboraviti i dodatnu krutost krovne ravni od trapeznog lima postavljenog upravno na pravac krovnih rožnjača, kao i krutost fasadnih vertikalnih panela debljine 15 cm.
- vrednosti pomeranja u vrhu objekta i momenata uklještenja AB stubova u temeljnu konstrukciju, postepeno rastu u smeru od središnjih stubova objekta do fasadnih stubova na kojima su ove vrednosti i najveće.
- usled temperaturnih uticaja, najvećim momentima savijanja su opterećeni ugaoni stubovi i to u obe ravni fasade. Prema rezultatima proračuna iz našeg proračunskog modela, vrednosti momenta savijanja M2 i M3 imaju prihvatljive vrednosti sa kojima nije bilo poteškoća u dimenzionisanju kako ugaonog stuba, tako i njegovog temelja.

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- [4] Pravilnik o tehničkim normativima za beton i armirani beton, Službeni list SFRJ, br. 11/87.
- [5] Pravilnik o tehničkim normativima za noseće čelične konstrukcije, Sl. list SFRJ, br. 61/86.
- [6] Pravilnik o tehničkim merama i uslovima za spregnute konstrukcije, Sl. list SFRJ br.35/70.



Dragan Šuput¹, Vladimir Vujčić²

MONTAŽA ČELIČNE KONSTRUKCIJE LUKOVA NOVOG ŽEŽELJEVOG MOSTA PREKO DUNAVA U NOVOM SADU

Rezime:

S-62 U radu je prikazana tehnologija i organizacija ukрупnjavanja i montaže čelične konstrukcije lukova novog Žeželjevog mosta preko Dunava u Novom Sadu. Kroz kratak rezime dat je uvid u osnovnu podjelu radova i njihovu organizaciju, kao i redosled izvođenja glavnih aktivnosti u procesu montaže, uzimajući u obzir osnovne karakteristike i specifičnosti konstrukcije mosta.

Ključne reči: tehnologija i organizacija, pomoćna konstrukcija, konstrukcija lukova

CONSTRUCTION OF STEEL ARCHES OF NEW ZEŽELJ BRIDGE OVER THE DANUBE IN NOVI SAD

Summary:

This paper presents the technology and organization of enlarging and installation of steel structures arches of New Žeželj bridge over the Danube in Novi Sad. Through a brief summary an insight into the basic division of work and their organization is given, as well as the order of execution of the main activities in the editing process, while taking into account the basic characteristics and the specific construction of the bridge.

Key words: technology and organization, auxiliary constructions, arches

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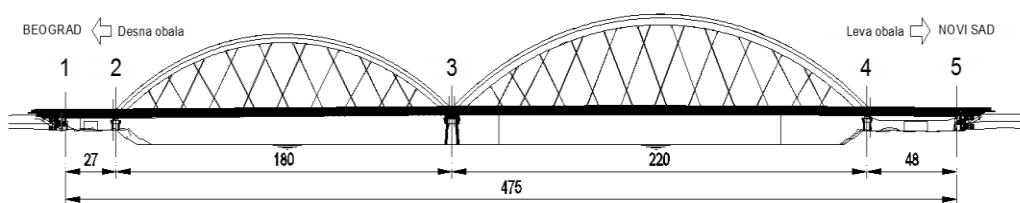
1 UVOD

Montaža čelične konstrukcije lukova je deo Projekta izgradnje novog železničko-drumskog mosta preko reke Dunav u Novom sadu – „Žeželjev most”, koju je pored radova na antikoroziivnoj zaštiti, čišćenju dna Dunava u zoni srednjeg stuba P3 i njegovoj izradi kao Podizvođač realizovala GP „MOSTOGRADNJA“AD iz Beograda.



1.1 OSNOVNE KARAKTERISTIKE MOSTA

Novi Žeželjev most projektovan je kao konstrukcija od dva lučna mosta raspona 180m i 220m sa prilaznim konstrukcijama na petrovaradinskoj, odnosno novosadskoj strani Dunava, raspona 27m i 48m. Visina Luka raspona 180m je 34m, a visina Luka raspona 220m je 42m. Ukupna širina mosta je 31.6m. (Slika 1)



Slika 1 – Dispozicija drumsko-železničkog mosta preko Dunava u Novom sadu

1.2 TEHNOLOGIJA I ORGANIZACIJA RADOVA

Tehnologijom predviđeno je da se kompletni radovi na montaži čelične konstrukcije lukova, što uključuje i nanošenje antikorozijske zaštite i montažu vešaljki, izvode paralelno na obe obale Dunava. Nakon kompletno završenih radova na montaži lukova, pristupa se prevlačenju lukova na reku gde bi se oslonili na srednji stub P3. Takva tehnologija izvođenja radova na montaži čelične konstrukcije mosta, uslovljena pre svega statičkim sistemom konstrukcije mosta i lokacijskim uslovima, zasnovana je i na ekonomski najracionalnijem rešenju proizašlom na osnovu sveobuhvatne i detaljne tehno-ekonomske analize.

S obzirom da Žeželjev most formiraju dva luka raspona 180m i 220m radovi na montaži organizovani su u skladu sa tim na:

- radovi na montaži luka 180m na desnoj obali Dunava (petrovaradinska strana)
- radovi na montaži luka 220m na levoj obali Dunava (novosadska strana)

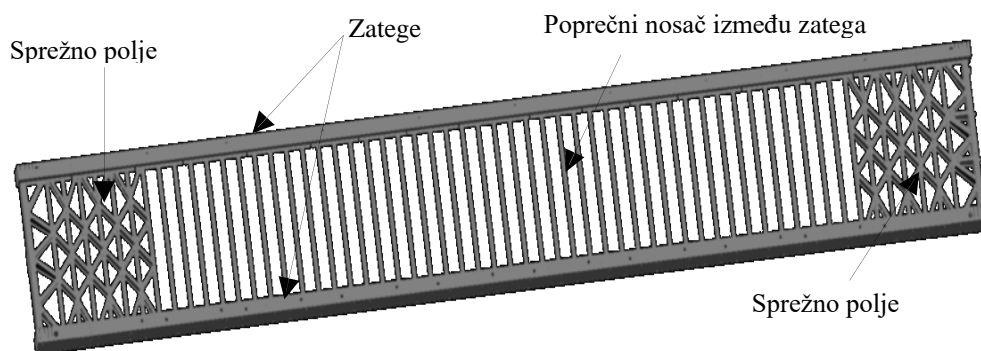
Dalja podela radova i njihova organizacija definisana je u skladu sa statičkim sistemom konstrukcije:

- radovi na montaži donjeg dela konstrukcije (montaža zatega, poprečnih nosača i spregova)
- radovi na gornjem delu konstrukcije (montaža lukova i transverzalnih nosača)

Tehnologijom montaže lukova predviđeno je da se prvo montira i zavari donji deo konstrukcije mosta, nakon čega sledi montaža i zavarivanje gornjeg dela konstrukcije.

1.2.1 Radovi na montaži donjeg dela konstrukcije

Glavni noseći sistem Lukova Žeželjevog mosta jeste čelična konstrukcija statičkog sistema luka sa zategom. Zatege su međusobno povezane poprečnim nosačima i zajedno sa dva sprežna polja na susednim krajevima formiraju donji deo konstrukcije (Slike 2 i 3). Veze između ovih elemenata konstrukcije ostvaruju se zavarivanjem.



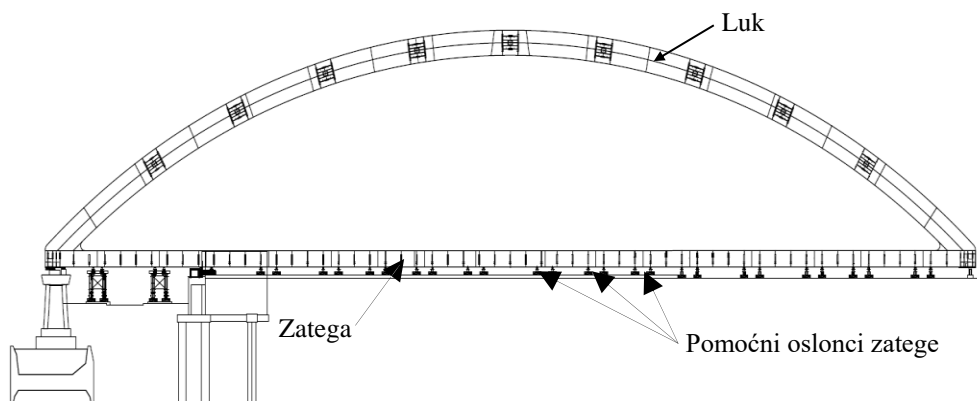
Slika 2 – Donji deo čelične konstrukcije Lukova

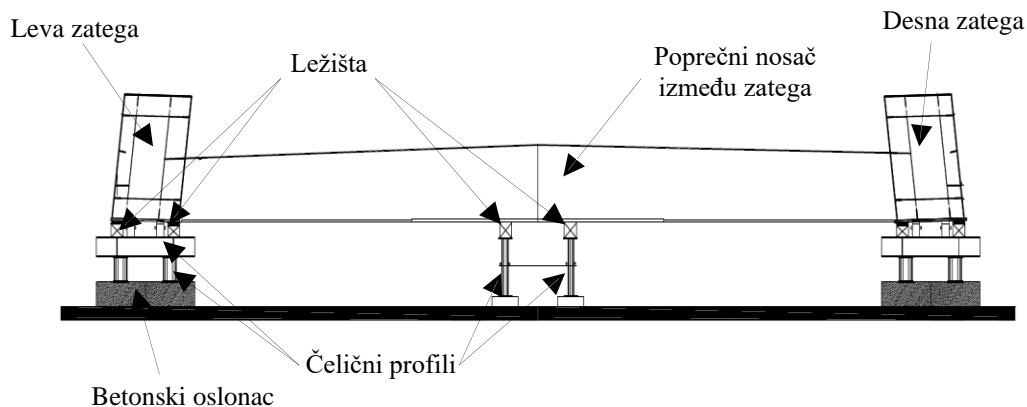


Slika 3 – Donji deo čelične konstrukcije lukova u fazi montaže

Za radove na montaži lukova na petrovaradinskoj i novosadskoj strani gradilišta, nasipanjem materijala formirane su radne platforme. Nosivost radne platforme projektovana je da prihvati opterećenja konstrukcije lukova sve do njihovog prevlačenja, kao i različite mehanizacije koja je korišćena za montažu lukova. Na novosadskoj strani gradilišta zbog nedovoljno prostora, izvršeno je pobijanje šipova za oslanjanje konstrukcije nad rekam. Na taj način se deo konstrukcije luka 220m oslanja na nasutu radnu platformu, a deo na šipove u reci.

Na radne platforme se u skladu sa predviđenim geodetskim položajem postavljaju privremeni pomoćni oslonci, na koje će se tokom montaže oslanjati elementi donjeg dela konstrukcije lukova (Slika 4). Pomoćni oslonci, koji su formirani od čeličnih profile, limova i betonskih oslonaca, projektovani su i izrađeni u fabrici čelične konstrukcije GP „Mostogradnja“ AD. Ukupna težina pomoćne konstrukcije za oslanjanje donjeg dela konstrukcije iznosi 800t.





Slika 4 – Pomoćni oslonci za oslanjanje elemenata zatege i poprečnih nosača

Dimenzije čeličnih profila i betonskih oslonaca određene su u skladu sa uticajima koji se javljaju u konstrukciji tokom montaže. Potrebno je naglasiti da tokom montaže sama konstrukcija menja statički sistem, pa se prilikom dimenzionisanja oslonačkih elemenata mora uzeti u obzir najnepovoljnija kombinacija uticaja.

Kako bi se obezbedio nesmetan “rad” tj. pomeranje konstrukcije u toku montaže, između elemenata donjeg dela konstrukcije i oslonačkih elemenata postavljaju se ležišta različitih vrsta i dimenzija. Krajnji elementi zatege oslanjanju se na Nga, Nge i NAL ležišta dok se ostali elementi zatega i poprečnih nosača oslanjanju na prohrom-teflonska ležišta. Raspored pokretnih odnosno nepokretnih ležišta određuje se prema statičkom sistemu konstrukcije i prilagođava se različitim fazama kroz koje prolazi konstrukcija tokom montaže. Sva privremena ležišta za oslanjanje konstrukcije tokom montaže usvojena se iz proizvodnog programa GP “Mostogradnje” AD u čijem pogonu su i izrađena.

Prvi elementi donjeg dela konstrukcije koji se postavljaju na pomoćne oslonce su elementi zatege. Zatege se montiraju paralelno leva i desna strana u nizu od najmanje dva elementa kako bi se istovremeno uz potvrdu njihovog geodetskog položaja pripremio i spoj između dva elementa za zavarivanje. Montažu zatega paralelno prati montaža poprečnih nosača. Poprečni nosači montiraju se iz polovina – leve i desne. Nakon što se u zoni spregova završi montaža i zavarivanje poprečnih nosača, počinje montaža spregova koji se tokom montaže oslanjanju direktno na poprečne nosače (Slika 5).

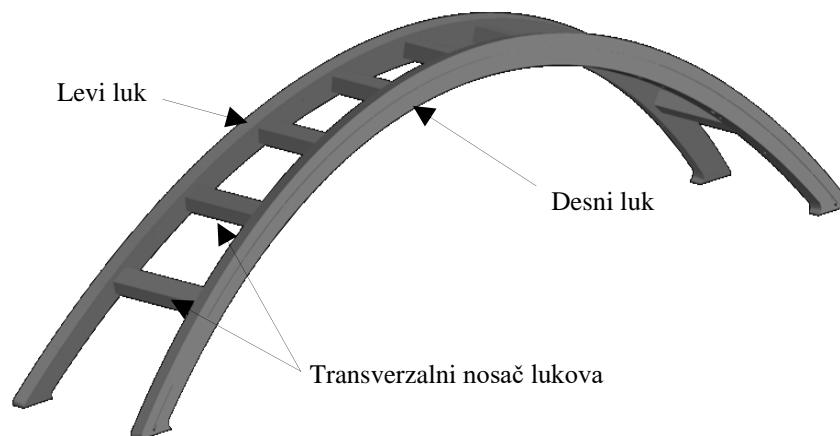
Tehnologijom montaže predviđeno je da se montaža elemenata donjeg dela konstrukcije vrši autodizalicama različitih nosivosti. Prosečna težina elemenata zatega je 30t, poprečnih nosača 4t i spregova 2.5t, pa su za njihovu montažu korišćene autodizalice nosivosti 40t, 80t i 120t. S obzirom da je zahtevana dozvoljena tolerancija za odstupanja od projektovanog visinskog položaja $\pm 3\text{mm}$, fina podešavanja kota oslonačkih tačaka elemenata vrše se hidrauličnim presama nosivosti 50t.



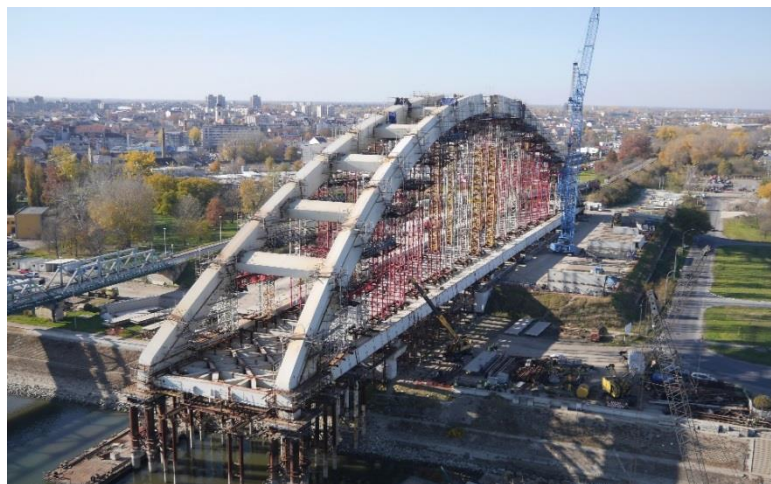
Slika 5 – Montaža elemenata donjeg dela konstrukcije

1.2.2 Radovi na montaži gornjeg dela konstrukcije

Gornji deo konstrukcije formiraju levi i desni luk međusobno povezani transverzalnim nosačima. Tranverzalni nosači lukova su projektovani kao sandučasti nosači, oslonjeni na susedne elemente lukova, pri čemu se veza sa lukovima ostvaruje sučeonim zavarivanjem. Karakteristika lukova je da se nalaze pod nagibom prema osi mosta (Slike 6 i 7).



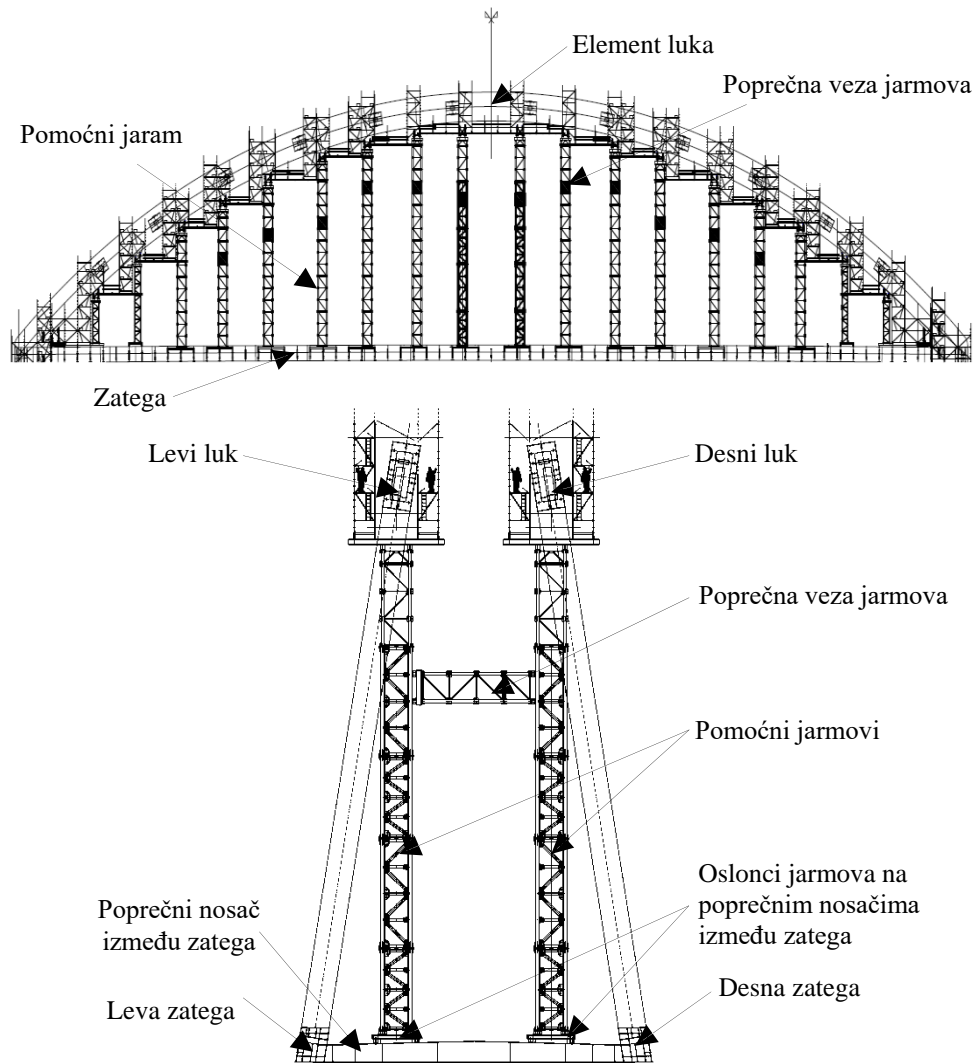
Slika 6 – Gornji deo čelične konstrukcije Lukova



Slika 7 – Gornji deo čelične konstrukcije lukova u fazi montaže

Elementi lukova se u fazi montaže oslanjaju na pomoćne čelične jarmove (Slike 8 i 9). Montaža jarmova, kao i njihova kasnija demontaža jedna je od specifičnosti montaže lukova Žeželjevog mosta. Ta specifičnost se ogleda pre svega u visini samih jarmova, koja se kreće od

12m do 36m. Osim toga na luku 180m montirano je 28 jarmova, dok je na luku 220m montirano 36 jarmova, Ukupna težina jarmova za oslanjanje na oba luka iznosi 1400t. Pomoćni jarmovi za oslanjanje elemenata lukova oslanjanju je se na zatege i poprečne nosače, na koje se postavljaju tek nakon završene montaže i zavarivanja donjeg dela konstrukcije.



Slika 8 – Pomoćni jarmovi za oslanjanje elemenata lukova

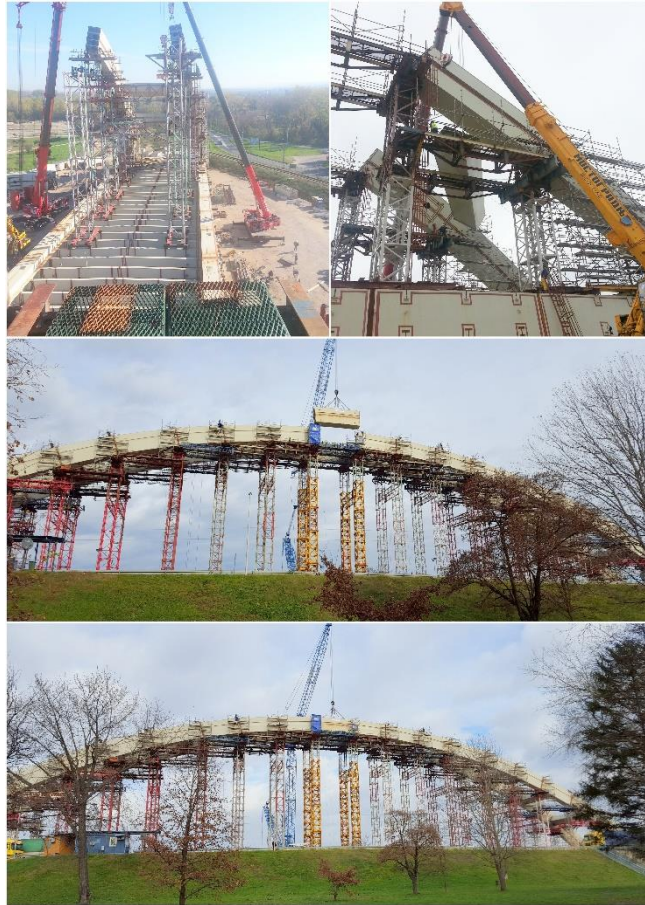


Slika 9 – Pomoćni jarmovi za oslanjanje elemenata lukova u fazi montaže

Nakon što se odgovarajući jarmovi za oslanjanje lukova montiraju i zavare, sledi montaža elemenata luka. Lukovi se montiraju paralelno leva i desna strana u nizu od najmanje dva elementa kako bi se istovremeno uz potvrdu njihovog geodetskog položaja pripremio i spoj između dva elementa za zavarivanje. Nakon montaže para lukova između kojih se nalazi transverzalni nosač, pristupa se geodetskom merenju rastojanja oslonačkih tačaka transverzalnog nosača kao bi se na osnovu njih odredile dužine stranica transverzalnih nosača. Na osnovu tako definisanih dužina, vrši se opsecanje i priprema stranica transverzalnog nosača nakon čega sledi njihova montaža. Montaža elemenata lukova sa dalje nastavlja sve do montaže poslednjih, središnjih i najviših elemenata lukova – pas elemenata.

Osnovna specifičnost montaže lukova ogleda se u težini pojedinih elemenata, projektovanoj visini na kojoj se nalaze i nagibu prema osi mosta. Težine pojedinačnih elemenata luka iznose od 40t do 73t, pri čemu su ovi elementi montirani na oslonce sa kotama i do 42m. Ovakve karakteristike uslovile su da se montaža elemenata luka izvodi dizalicom nosivosti 500t. Za montažu pomoćnih jarmova koriste se dizalice nosivosti 80t, 100t i 120t sa dohvatom od 50m. S obzirom da je zahtevana dozvoljena tolerancija za odstupanja od projektovanog visinskog položaja $\pm 3\text{mm}$, fina podešavanja kota oslonačkih tačaka elemenata lukova vrši se hidrauličnim presama nosivosti 50t i 100t.

Na slici 11 prikazane su karakteristične faze montaže gornjeg dela konstrukcije lukova.



Slika 11 – Pomoćni jarmovi za oslanjanje elemenata lukova u fazi montaže

2 ZAKLJUČAK

U grupi železničko-drumskih mostova sa dva koloseka, Žeželjev most sa svojim lukovima raspona 180m i 220m, predstavlja most sa rekordnim rasponima lukova u svetu čija ukupna težina čelične konstrukcije iznosi 10500t. Kao takav organizacija radova na njegovoj izgradnji predstavlja jedinstven i kompleksan posao, koji zahteva angažovanje velikog broja resursa specijalne opreme i mehanizacije

LITERATURA

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ZAVARIVANJE VUTA BAZNIH ELEMENATA IZRAĐENIH OD ČELIKA KVALITETA S 460N

Rezime:

S-63

U radu je prikazan metodološki pristup zavarivanja fino-zrnih mikrolegiranih čelika kvaliteta S 460N na primeru čeličnih vuta baznih komada Žeželjevog mosta preko reke Dunav u Novom Sadu.

Ključne reči: Žeželjev most, zavarivanje, čelik kvaliteta S 460N.

WELDING OF STEEL HAUNCHES ON BASE ELEMENTS MADE WITH STEEL GRADE S460N

Summary:

The paper describes methodological approach on welding of fine grain micro alloy steels grade S 460N used on steel haunches of base elements of Žeželj Bridge across Danube River in Novi Sad.

Key words: Žeželj bridge, welding, steel grade S460N.

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1 UVOD

Predmet ovog rada je zavarivanje čeličnih vuta na početnim segmentima lukova mosta.

Na osnovu analize naponskog stanja u lukovima mosta, Projektant je došao do zaključka da bazni elementi luka treba da budu izrađeni od materijala višeg kvaliteta u odnosu na ostalu noseću čeličnu konstrukciju mosta i projektom je za ovu poziciju usvojen materijal S 460N.

Ovaj materijal spada u grupu finoznih mikrolegiranih konstrukcionih čelika. Zajednička karakteristika finoznih konstrukcionih čelika je nizak procenat ugljenika i legirajućih elemenata (mikrolegiranje i niskolegiranje), visok stepen čistoće (nizak procenat fosfora i sumpora) i sitnozrna struktura, što sve ukupno obezbeđuje dobru žilavost.



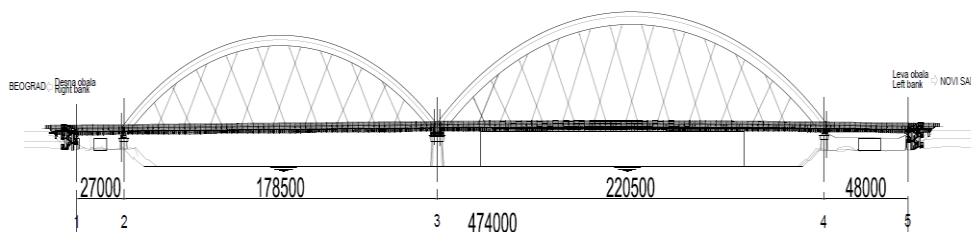
Slika 1. Železničko-drumski most na Dunavu u Novom Sadu. Žeželjev most

1.1. TEHNIČKI OPIS I KARAKTERISTIKE MOSTA

Žeželjev most je železničko-drumski most koji je u prvobitnoj izvedbi bio izgrađen kao betonski lučni most. Izgrađen je 1961. godine i porušen je u NATO bombardovanju SR Jugoslavije 1999. godine. Gradjen je kao jedinstveni primer gradnje uz korišćenje prednapregnutog betona.

Novi Žeželjev most čine dva lučna mosta raspona 180m i visine 34m, odnosno 220m i visine 42m i dve prilazne konstrukcije, dužine 48m i 27m. Na mostu su predviđene dve pešačko-biciklističke staze, dve trake za drumski saobraćaj i dva koloseka za železnički saobraćaj.

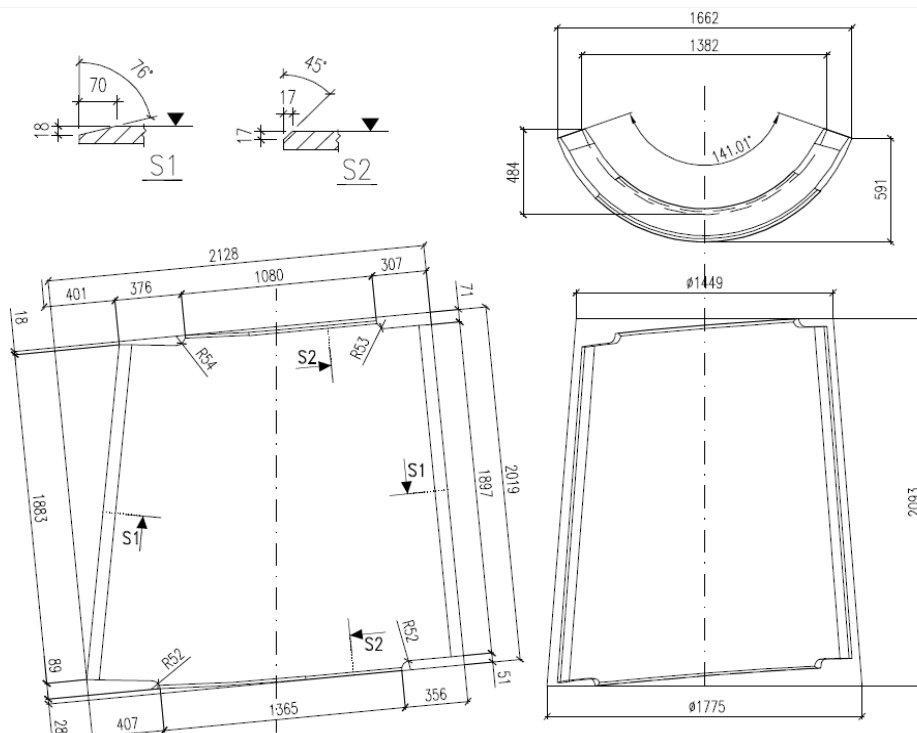
Glavni noseći sistem mosta jeste čelična konstrukcija statičkog sistema luka sa zategom. Levi i desni luk međusobno su povezani transverzalnim nosačima pri čemu se veza ostvaruje sučeonim zavarivanjem, dok su zatege međusobno spojene poprečnim nosačima zajedno sa dva sprežna polja na krajevima. Na međusobnim vezama zatega i lukova nalaze se vute.



Slika 2. Dimenzije železničko-drumski most na Dunavu u Novom Sadu. Železjevi most

1.2. VUTE

Čelična konstrukcija elemenata vuta je izrađena od finoznog, mikrolegiranog čelika S 460N, pa je iz tog razloga njihovo zavarivanje trebalo da se izvede prema posebnoj tehnologiji zavarivanja. Prema svojim dimenzijama, konstrukcija vuta pripada konstrukcijama izrađenim od čelika povećanih debljina, što dodatno otežava njihovo zavarivanje.



Slika 3. Dimenzije čeličnih vuta baznih elemenata kvaliteta S 460N na luku 220m sa pripremom ivica za zavarivanje

2 KVALITET I ZAVARLJIVOST ČELIKA KVALITETA S 460N

U oznaci stanja isporuke finoznih čelika razlikuju se dve oznake: oznaka N - za normalizaciono žareno i normalizaciono valjano stanje i oznaka M - za termomehanički valjano stanje.

Normalizaciono žareno stanje je postupak konvencionalnog valjanja sa naknadnom termičkom obradom za usitnjenje zrna normalizacionim žarenjem. Normalizaciono valjanje je postupak toplog valjanja finoznih (mikrolegiranih) čelika u kome se predvaljanje i završno valjanje izvode u austenitnoj oblasti u kojoj je rekristalizacija potpuna, a hlađenje do sobne temperature se vrši na vazduhu-normalizacijom. Završna struktura se dobija otpuštanjem. Kod normalizovanih konstrukcionih finoznih čelika predviđenih za noseće čelične konstrukcije, pored propisanih vrednosti mehaničkih osobina, važna konstruktivna karakteristika je i najmanja vrednost žilavosti na određenoj temperaturi kao i na sniženim temperaturama.

Tabela 1: Hemijski sastav osnovnog materijala

Oznaka po EN 10027-1	C	Mn	Si	P	S	Nb	V	Al	Ti	Cr	Ni	Mo	Cu	N	CE
S460N	0,20	1,00-1,70	0,60	0,030	0,025	0,05	0,20	0,02	0,05	0,30	0,80	0,10	0,55	0,025	0,53

Tabela 2: Mehaničke karakteristike osnovnog materijala

Red. br.	Oznaka po EN 10027-1 i CR 10260	Gornji napon tečenja R_{EH} [N/mm ²]	Zatezna čvrstoća R_m [N/mm ²]	Izduženje A_5 min %	Žilavost KV [J]
1.	S460N	430	550÷720	17	40 (-20°C)

Finozrni čelici, generalno zahtevaju pažljiviji pristup prilikom zavarivanja u poređenju sa običnim konstrukcionim čelicima obzirom da pri zavarivanju ovih čelika može doći do sledećih grešaka:

- nastanak hladnih prslina izazvanih vodonikom u metalu šava i u osnovnom materijalu u zoni uticaja toplote usled zavarivanja (ZUT) i prslina usled zakaljivanja u ZUT-u;
- sniženje čvrstoće u široj zoni ZUT-a usled otpuštanja, kod čelika kaljenog i otpuštenog stanja.
- sniženje žilavosti i povećanje prelazne temperature krtosti usled nastanka grubog zrna, zakaljivanja ili izlučivanja u ZUT;

Uzroci i sprečavanje grešaka prilikom zavarivanja:

- vodonik u metalu šava, unet preko potrošnog materijala i okolne atmosfere, difunduje iz metala šava u ZUT, i znatno smanjuje deformabilnost martenzita nastalog usled zakaljivanja te povećava sklonost nastajanju hladnih prslina, posebno kod N finoznih čelika. Za sprečavanje nastanka hladnih prslina usled prisustva vodonika koriste se potrošni materijali sa niskim sadržajem vodonika, a u zavisnosti od sklonosti čelika

zakaljivanju primenjuje se predgrevanje i/ili zavarivanje sa povišenom pogonskom energijom.

- nedovoljan unos toplote prilikom zavarivanja može dovesti do pojave zakaljivanja. Kod čelika N stanja zakaljivanje se može smanjiti primenom predgrevanja i povišenom pogonskom energijom zavarivanja. Takođe, u znatnom stepenu zakaljivanje se može smanjiti tehnikom polaganja pravih tankih preklopljenih zavara. Otpuštanje ZUT-a prethodno nanetog zavara moguće je polaganjem dodatnih tehnoloških zavara koji preklapaju zavare temena šava sa osnovnim materijalom. Termička obrada otpuštanja/žarenja za smanjenje napona posle zavarivanja je prihvatljiva, uz određena ograničenja kod tipova čelika koji su skloni pojavi prslina usled ponovnog zagrevanja.
- u uskoj zoni ZUT sa porastom zrna smanjuje se žilavost i povećava prelazna temperatura. Širina grubozrne zone može se smanjiti sniženjem pogonske energije i temperature predgrevanja. U izvesnoj meri, kod višeslojnih spojeva grubo zrno se može izbeći normalizacionim efektom narednih slojeva.

Finozrni čelici se zavaruju svim elektrolučnim postupcima. U odnosu na ručno elektrolučno zavarivanje obloženom elektrodom prednost imaju MIG/MAG i EPP postupak, usled jednostavnijeg upravljanja procesom i kontrolisanja količine unete toplote preko brzine zavarivanja.

Tehnološka rešenja koja se koriste prilikom zavarivanja usmerena su na:

- sprečavanje nastanka hladnih prslina i
 - određivanje uslova zavarivanja za dobijanje zahtevanih osobina (tvrdoće i žilavosti) ZUT koje obezbeđuju sigurnost zavarenog spoja u eksploataciji.
- i zavise od:
- vrste čelika - primenjenog postupka valjanja (N ili M), hemijskog sastava (uključujući i mikrolegiranje), primenjenog postupka hlađenja i otpuštanja po završetku valjanja;
 - mehaničko-tehnoloških osobina čelika i zahteva za mehaničko-tehnološke osobine ZUT zavarenog spoja.

2.1. IZBOR DODATNOG MATERIJALA KOD NORMALIZOVANIH ČELIKA

Izbor dodatnog materijala za zavarivanje se bazira na vrednostima čvrstoće osnovnog materijala i njegovoj osetljivosti za nastanak hladnih prslina izazvanih vodonikom iz metala šava.

Sa aspekta sadržaja vodonika, kod čelika povišene čvrstoće preporuke su za primenu dodatnog materijala sa nižim sadržajem difundovanog vodonika. Takođe, kod većeg mešanja osnovnog i dodatnog materijala, kao što je to kod korenih zavara sučeonog spoja, preporučuje se primena dodatnog materijala nižeg, odnosno niskog sadržaja difundovanog vodonika, zavisno od sklonosti osnovnog materijala zakaljivanju.

Izbor dodatnog materijala se vrši u odnosu na čvrstoću osnovnog materijala i to po mogućstvu tako da čist metal šava ima veću vrednost čvrstoće od osnovnog materijala.

Na osnovu podataka o hemijskim i mehaničkim karakteristikama osnovnog materijala, dimenzijama i vrsti žlebova, kao i položaju zavarivanja za izvođenje zavarivanja korišćiće se sledeći postupci (oznake prema EN ISO 4063) za čelike definisane po standardu EN 10025-3:2004 :

136– elektrolučno zavarivanje u zaštiti aktivnog gasa punjenom žicom.

Tabela 3: Klasifikacija žičanih elektroda za postupak 136

Standard	EN 758	EN ISO 17632-A	AWS A 5.29
žičana elektroda	T 46 6 1,5 Ni P M 1 H5	T 46 6 1,5 Ni P M 1 H5	E 81T1-K2M

Tabela 4: Hemijski sastav metala čistog šava

Žič. elektroda	C [%]	Si [%]	Mn [%]	Ni [%]
T 46 6 1,5 Ni P M 1 H5	0,04	0,30	1,40	1,50

Tabela 5: Mehaničke karakteristike metala čistog šava

Žičana elektroda	Gornji napon tečenja R_{EH} [N/mm ²]	Zatezna čvrstoća R_m [N/mm ²]	Izduženje A_5 min %	Žilavost KV [J]
T 46 6 1,5 Ni P M 1 H5	550	620	27	75 (-60°C)

Tabela 6: Komercijalne oznake potrošnog materijala za postupak

Proizvođač	ELGA	KOBELCO
žič. elektr.	Elgacore DWA 55L	DW-A55L

2.2. PREDGREVANJE

Predgrevanje i održavanje temperature predgrevanja u toku zavarivanja je mera za sprečavanje nastanka hladnih prslina usled zakaljivanja i uticaja difundovanog vodonika u metalu šava. Primena predgrevanja i temperatura predgrevanja zavise od sledećih faktora:

- ugljeničnog ekvivalenta (CE - %),
- sadržaja vodonika u metalu šava (HD – ml/100g);
- količine unete toplote pri zavarivanju (Q– kJ/mm), i
- debljine materijala (d - mm).

U slučaju niske ambijentalne temperature preporučuje se predgrevanje na 50°C (kada zbog ostalih razloga nije potrebno predgrevanje) ukoliko je temperatura okoline ispod 5°C. Za konkretnu vrstu konstrukcije preporuke su date u standardima, zavisno od vrste materijala i konstrukcije. Za određivanje temperature predgrevanja (Tp) zbog uticaja hemijskog sastava, izraženog veličinom ugljeničnog ekvivalenta, predložene su dve metode :

$$CE = C + \frac{Mn}{6} + \frac{Cr+Mo+V}{5} + \frac{Ni+Cu}{15} \quad (1)$$

$$CET = C + Mn + \frac{Mo}{10} + Cr + \frac{Cu}{20} + \frac{Ni}{40} \quad (2)$$

Koja će formula da bude primenjena zavisi od opsega hemijskog sastava čelika utvrđenog analizom topioničke šarže (sertifikat isporuke), ili date gornje nominalne vrednosti u standardu za vrstu čelika.

Najveće vrednosti ugljeničnog ekvivalenta (CE), zasnovane na analizi šarže, u zavisnosti od vrste i debljine čelika, date su prema standardu EN10025-3:2004 u Tabeli 1.

Na osnovu preporuka iz standarda SRPS EN 1011-1:2007 i SRPS EN 1011-2:2007 ovi zavareni spojevi se predgrevaaju. Vrednost ekvivalentnog ugljenika za osnovni material se određuje po formuli [1]. Temperature predgrevanja za konkretne zavarene spojeve date su u tehnološkim listama (slike 6, 7 i 8). Merenje dostignutih temperatura vršiti u skladu sa SRPS ISO 13916.

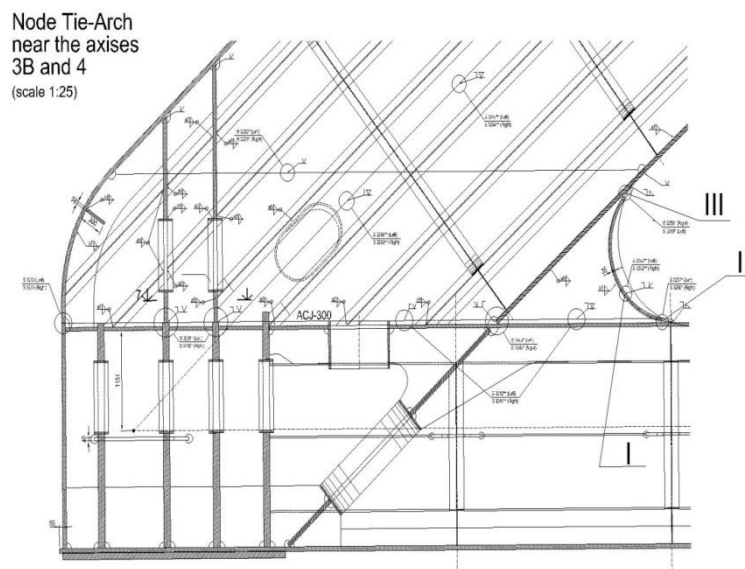
3 NAČIN I REDOSLED IZVOĐENJA ZAVARIVANJA

Pre pristupanja zavarivanju, delovi konstrukcije koja se zavaruju moraju se dovesti u određeni međusobni položaj, u skladu sa oblikom i dimenzijama žljeba, datim u WPS listama, bez obzira da li se zavaruju u slobodnom stanju, u alatima ili u pozicionerima. Nakon što se odobri položaj namontiranih elemenata treba da se osigura da je površina koja se zavaruje očišćena i obrušena do metalnog sjaja.


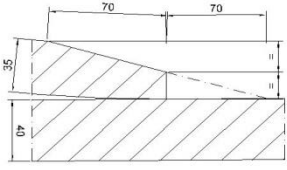
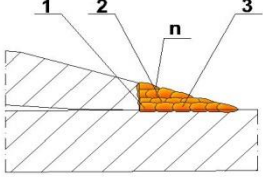


Slika 4. Priprema žleba za zavarivanje


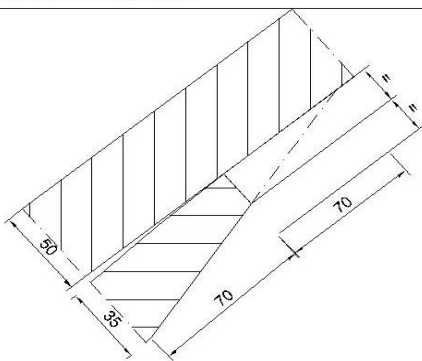
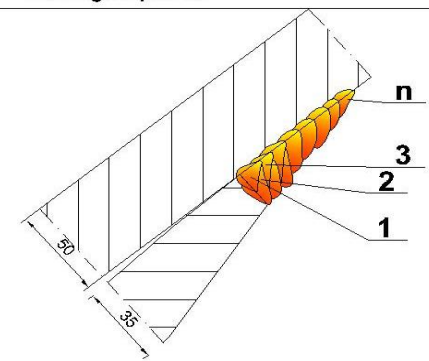
Kada se oblast u kojoj se izvodi zavarivanje obezbedi od nepovoljnih uticaja atmosferskih uslova onda se pristupa zavarivanju po sledećem redosledu: najpre se zavaruje vuta za vertikalni lim baznog komada luka, obe vertikale se zavaruju istovremeno sa dva zavarivača (I); Nakon zavarenih vertikalnih spojeva pristupa se zavarivanju spoja vute sa gornjom flanšom zatege, ova pozicija se zavaruje od sredine levo i desno tehnikom povratnog koraka sa dva zavarivača (II); na kraju se izvodi zavarivanje spoja donje flanše luka za vutu na isti način i istom tehnikom kao spoj gornje flanše zatege i vute (III). Na početku i kraju svakog sučeonog zavarenog spoja postavljaju se ulazno-izlazne (“startne”, “tehnološke”) pločice istog oblika i dimenzija kao i žleb zavarenog spoja. Početak i kraj zavarivanja spoja mora se izvesti na ulazno-izlaznim pločicama.



Slika 5. Način i redosled izvođenja zavarivanja

 Specification of welding technology (WPS) TECHNOLOGY LIST		WPS No: 163 Welding technology No: ST 03/12 WPAR No: IN PROGRESS								
CONSTRUCTION: PC "BATAJNICA" - WELDING DEPARTMENT Construction project: Railway road bridge across the Danube in Novi Sad Big Arch (3B - 4); Drawing No. AP 19c.B										
Welded joint mark: S 3256		Quality class of joint: B+								
Identification of basic material: Standard: EN 10025-3/EN 10025-2 Quality: S 460 N / S 355 J2+N Thickness (mm): 40 with 35										
Welding procedure: MAG Classification: 136										
Commercial mark of consumables: A: Eltacore DWA 55L B: Gas Mixture EN 439 : M21 C:		Classification: S 460N Depozit: $\varnothing 1.2$ 2,1-4,8 kgh								
Preparing og basic material and consumables for welding:										
Welding position: PB Method of preparing edges: Oxygen cutting Type of joint: Lap joint Additional treatment: Grinding		Backing: One-side or two-sided welding: One-side Drying of electrode: Cleaning: Steel brush								
Groove dimensions:		Welding sequence:								
										
Welding parameters:										
Weld No.	Filling with consumables material	Diam. of additional material	Free end (mm)	Shielding gas (l/min.)	Amper range(A)	Volt range(V)	Polarity	Wire speed (cm/min)	Welding speed (cm/min)	Input energy (kJ/cm)
1	A	1,2	15-20	10-15	220	27	DC+	375(950)	30,0	10,0
2-n	A	1,2	15-20	10-15	220-230	27	DC+	392(980)	30,8-40,8	6,2-10,3
Heat treatment:										
Heating method: gas/induction/resistance heating Preheating temperature (°C): 130 Post weld heat temperature (°C): Maintaining at temperature (°C):			Heating rate (°C/h): 100 Interpass temperature (°C): 220 Cooling rate (°C/h): free on air Insulation type and thickness:							
Approvals:										
Date: 17.09.2012.		Performed by: R. Savić, dipl.maš.inž.		Date: 17.09.2012.		Approved by: G. Ristić, dipl.maš.inž.				

Slika 6. Tehnološka lista zavarivanja položaj PB

 MOSTOGRADNJA	Specification of welding technology (WPS) TECHNOLOGY LIST		WPS No: 165 Welding technology No: ST 03/12 WPAR No: IN PROGRESS							
	CONSTRUCTION: PC "BATAJNICA" - WELDING DEPARTMENT Construction project: Railway road bridge across the Danube in Novi Sad Big Arch (3B - 4); Drawing No. AP 19c.B									
Welded joint mark : S 3258		Quality class of joint : B+								
Identification of basic material :										
Standard: EN 10025-3		Quality: S 460 N		Thickness (mm): 50 with 35						
Welding procedure : MAG		Classification : 136								
Commercial mark of consumables :		Classification :		Depozit:						
A: Elgacore DWA 55L		S 460N		Ø 1.2 : 2,1- 4,8 kg/h						
B: Gas Mixture EN 439 : M21										
C:										
Preparing og basic material and consumables for welding:										
Welding position : PD		Backing :		One-side or two-sided welding : One-side						
Method of preparing edges : Oxigen cutting		Drying of electrode :		Steel brush						
Type of joint : Lap joint		Cleaning :								
Additional treatment : Grindling										
Groove dimensions:		Welding sequence:								
										
Welding parameters:										
Weld No.	Filling with consumable material	Diam. of additional material	Free end (mm)	Shielding gas (l/min.)	Amper range(A)	Volt range(V)	Polarity	Wire speed (inch/min cm./min)	Welding speed (cm./min)	Input energy (kJ/cm.)
1	A	1,2	15-20	10-15	200	27	DC+	337(850)	31,6	8,7
2-n	A	1,2	15-20	10-15	175-185	25	DC+	295(750)	29,5-46,5	4,8-8,0
Heat treatment:										
Heating method:.....gas/induction/resistance heating					Heating rate (°C/h):.....100					
Preheating temperature(°C):.....130					Interpass temperature (°C):.....220					
Post weld heat temperature (°C) :...../					Cooling rate (°C/h):.....free-on air					
Maintaining at temperature (°C) :...../					Insulation type and thickness :...../					
Approvals:										
Date:		Performed by:		Date:		Approved by:				
17.09.2012.		R. Savić, dipl.maš.inž		17.09.2012.		G. Ristić, dipl.maš.inž				

Slika 7. Tehnološka lista zavarivanja položaj PD

 MOSTOGRADNJA	Specification of welding tehnology (WPS) TEHNOLOGY LIST	WPS No: 159
		Welding tehnology No: ST 03/12
		WPAR No: IN PROGRESS

CONSTRUCTION: PC "BATAJNICA" - WELDING DEPARTMENT

Construction project: Railway road bridge across the Danube in Novi Sad
Big Arch (3B - 4); Drawing No. AP 19c.B

Welded joint mark : **S 3252** Quality class of joint : **B+**

Identification of basic material :

Standard: **EN 10025-3** Quality: **S 460N** Thickness (mm): **40 with 35**

Welding procedure : **MAG** Classification: **136**

Commercial mark of consumables :

A: **Elgacore DWA 55L**

B: **Gas Mixture EN 439 : M21**

C:

Classification:

S 460N

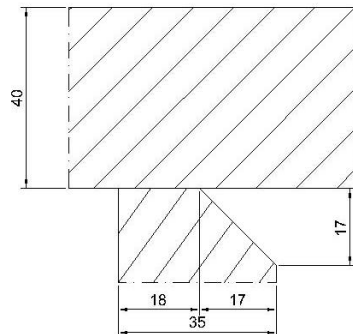
Depozit:

Ø 1.2: **2,1-4,8 kg/h**

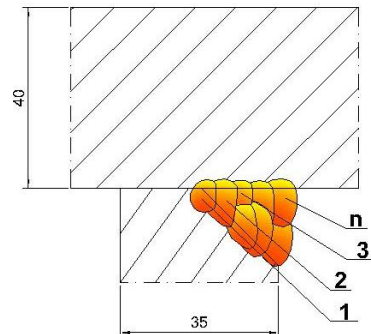
Preparing og basic material and consumables for welding:

Welding position : PF	Backing :
Method of preparing edges : Oxygen cutting	One-side or two-sided welding : One-side
Type of joint : TW partial penetration	Drying of electrode :
Additional treatment : Grindling	Cleaning : Steel brush

Groove dimensions:



Welding sequence:



Welding parameters:

Weld No.	Filling with consumable material	Diam. of additional material	Free end (mm)	Shielding gas (l/min.)	Amper range(A)	Volt range(V)	Polarity	Wire speed (inch/min cm./min)	Welding speed (cm./min)	Input energy (kJ/cm.)
1	A	1,2	15-20	10-15	220	27	DC+	375(950)	30,0	10,0
2-n	A	1,2	15-20	10-15	220-230	27	DC+	392(980)	30,6-48,8	6,2-10,3

Heat treatment:

Heating method:.....gas/induction/resistence heating

Preheating temperature(°C).....150

Post weld heat temperature (°C)...../

Maintaining at temperature (°C)...../



Heating rate (°C/h).....100

Interpass temperature (°C).....220

Cooling rate (°C/h).....free-on air

Insulation type and thickness...../

Approvals:

Date:	Performed by:	Date:	Approved by:
17.09.2012.	R. Savić, dipl.maš.inž 	17.09.2012.	G. Ristić, dipl.maš.inž 

Slika 8. Tehnološka lista zavarivanja položaj PF

4 ZAKLJUČAK

Primena finoznih čelika u nosećim čeličnim konstrukcijama ima svojih prednosti ali i mana. Međutim, ukoliko se izvođenje zavarivanja vrši kontrolisano, uz sprovođenje propisane tehnologije zavarivanja i poštovanje redosleda zavarivanja upotreba ovih materijala za noseće čelične konstrukcije ima veliku opravdanost sa aspekta ekonomičnosti i kvaliteta u eksploataciji.

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Dašić Branislav¹, Vujčić Vladimir²

TEHNOLOGIJA MONTAŽE ČELIČNOG MOSTA PREKO REKE VELIKE MORAVE U ĆUPRIJI

Rezime:

S-64

U radu je opisan način montaže čelične konstrukcije mosta primenom tehnologije “naguravanja”, uz korišćenje specijalne hidraulične opreme, sa osvrtom na određene probleme koji se javljaju prilikom korišćenja ovakve tehnologije i načinom njihovog rešavanja.

Ključne reči: most u Ćupriji, montaža, čelična konstrukcija mosta, naponi, naguravanje konstrukcije

TECHNOLOGY OF CONSTRUCTION OF THE STEEL BRIDGE OVER THE VELIKA MORAVA IN ĆUPRIJA

Summary:

This paper describes a method of assembling the steel structure of the bridge by using technology "pushing", with the use of special hydraulic equipment. The emphasis is on the specific problems that occur when using such technology and the way of solving them.

Key words: bridge in Ćuprija, assembling, steel structure of the bridge, voltage, "pushing" the structure

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1 UVOD

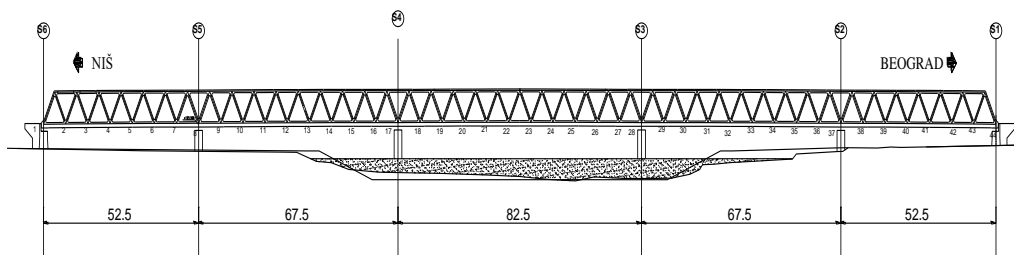
Železnički most preko Velike Morave se nalazi na magistralnoj pruzi Beograd-Niš, deonica Gilje-Ćuprija-Paraćin.

Most je dvokolosečni, sa kolosekom položenim u tucaničkom zastoru. Osovinski razmak koloseka na mostu je 4,20 m. Osovina koloseka na mostu je u pravcu, a niveleta ima vertikalnu krivinu prečnika 30.000 m. Kota GIŠ-a je na obalnim stubovima 125,000 m, a na sredini mosta je 125,431 m. Kota donje ivice konstrukcije je u sredini mosta 123,274m. Sa obe strane mosta postoji službena pešačka staza širine 1,0m. Na levoj pešačkoj stazi, sa spoljne strane ograde je na konzoli postavljen kanal za smeštaj SS i TT kablova. Pruga je elektrificirana.

Projektovana brzina je 120 km/čas.

1.1 OPIS MOSTA

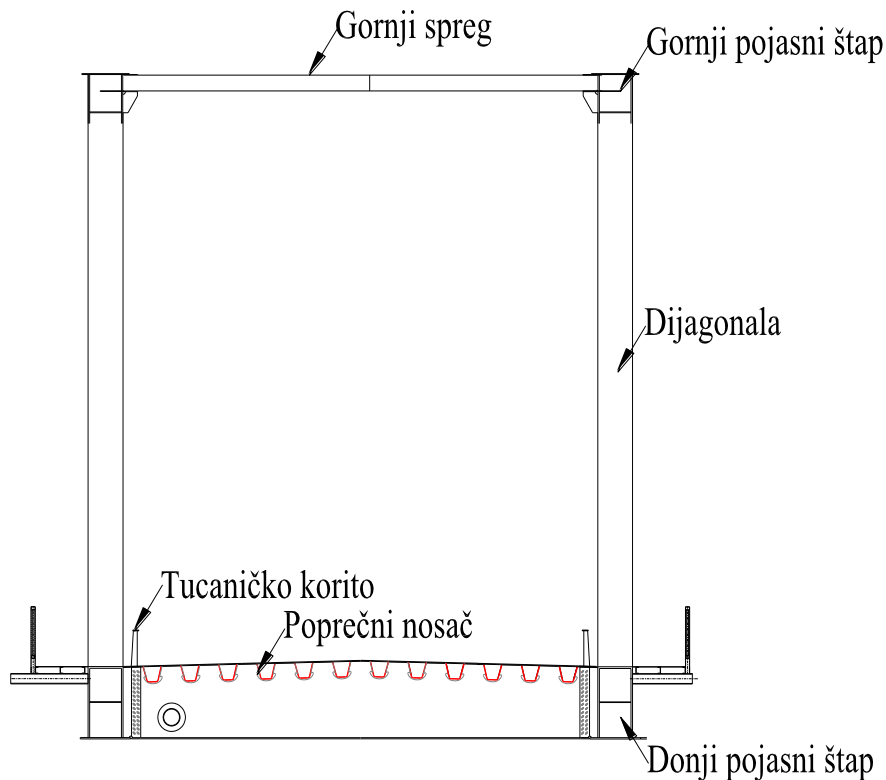
Dvokolosečni železnički most u Ćupriji premošćuje reku Veliku Moravu sa pet otvora. Konstrukcija mosta je čelična rešetkasta kontinualna greda ukupne dužine $L = 322,5$ m sa rasponima: 52,5 m, 67,5 m, 82,5 m, 67,5 m i 52,5 m. Konstrukcija je konstantne visine, $H = 8,0$ m. i osovinske širine $B = 10,0$ m, a dužina jednog polja je 7,5m.



Slika 1 – Dispozicija železničkog mosta preko Velike Morave u Ćupriji

Rešetkasta ispuna mosta je sa dijagonalama, bez vertikalama. U ravni gornjeg i donjeg pojasa postoji rešetkasti spreg. Tucanički zastor se nalazi u čeličnom koritu sa dvostranim padom od 2,5% od sredine ka krajevima.

U sredini svakog polja, odnosno na razmaku od 3.75 m, uz ivice korita se nalazi čelični rešetkasti slivnik 356x160mm sa dvostrukom rešetkom radi boljeg proceđivanja vode. Za odvođenje vode iz korita ugrađene su ispod kolovoza dve poliesterske cevi prečnika $\varnothing 250$, debljine zida 6 mm, koje su u nagibu od 0,4% od sredine mosta ka krajevima. Cevi prolaze kroz otvor prečnika $\varnothing 300$ na rebru poprečnog nosača. Ispred obalnih stubova cevi se spuštaju ispod konstrukcije i odvođe do taložnika.



Slika 2 – Poprečni presek železničkog mosta preko Velike Morave u Čupriji

1.2 OPIS KONSTRUKCIJE

Rešetkasti kontinualni glavni nosač je urađen kompletno u zavarenoj izradi. Gornji pojas je sandučastog poprečnog preseka, visine 700 mm i širine 600 mm. Gornja lamela je proširena u širini od 900mm Čelični limovi za manje opterećene elemente su u kvalitetu S235J2G3, sa debljinama od 12 do 24 mm, a za elemente u zoni oslonaca i u sredini mosta su u kvalitetu S355J2G3. Donji pojas je sandučastog preseka, visine 1300 mm i širine 600mm. Donja lamela je proširena u širini od 960mm. Debljine limova su od 14 do 26mm. Presek je formiran zavarivanjem. Zbog stabilnosti pritisnutih štapova u eksploataciji i montaži, u sredini sanduka celom dužinom je zavareno podužno ukrucenje od lima širine 600 mm i debljina 12 i 16mm. Na svakih 3,75m, (na mestima gde se priključuje poprečni nosač) u donjem pojasu postoji dijafragma.

Čelični limovi od kojih je izrađena konstrukcija su različitog kvaliteta, koji zavisi od veličine naprezanja. U najvećem delu, čelični limovi su u kvalitetu S235J2G3, a u zoni oslonaca kao i u presecima sa većim napreznjem su u kvalitetu S355J2G3. Čvorni limovi su izrađeni od čeličnih limova u najvećem broju sa debljinom od 26mm, a ostali su debljine 30 mm, osim iznad stubova S2, S3, S4 i S5 gde su izrađeni od lima debljine 36 mm. Kvalitet materijala je različit i zavisi od veličine naprezanja. Usled smanjenja dimenzija i koncentracije

napona, mesta prelaska vertikalnog lima pojasa u čvorni lim su zaobljena po elipsi. Dijagonale su sandučastog preseka formiranog zavarivanjem. Visina svih dijagonala (upravno na ravan glavnog nosača) je 600mm, a širina (u ravni glavnog nosača) je promenljiva. Najveći broj dijagonala je širine 500mm, u zoni blizu oslonaca je 600mm, a dijagonale koje se susstiču u oslonačkom čvoru na stubovima S3 i S4 su širine 650mm. Debljine limova su od 10 do 30 mm.

Kvalitet limova od koga su izrađene dijagonale je uglavnom u kvalitetu S235J2G3, osim najopterećenijih dijagonala u zoni oslonaca S3 i S4 koje su izrađene od čelika S355J2G3. Obzirom na specifičan način vezivanja za čvorni lim (sučeonim zavarivanjem koje je opisano u odeljku o tehnologiji izrade), ispred čvornog lima se, na dužini koja je jednaka širini lamele, lim dijagonale u ravni čvornog lima podebljava na debljinu čvornog lima. Limovi dijagonala, upravno na ravan čvornog lima su maksimalne debljine 16mm i zavaruju se za olučasti umetak između čvornih limova, takođe sučeono.

U ravni gornjeg pojasa glavnog nosača se nalazi spreg sa rombičnom ispunom bez vertikalala, čije su dijagonale napravljene od valjanih nosača IPB1 300, visine 290 mm. Na početku i na kraju sprega se nalazi portalna rigla sandučastog preseka, dimenzija 700x600mm. Donji spreg je takođe sa rombičnom ispunom, ali sa poprečnim nosačima kao vertikalama. Dijagonale sprega su cevi Ø168 x 71 mm.

Kolovoznu konstrukciju, u vidu korita koje nosi tucanički zastor, formiraju: kolovozni lim debljine 14mm sa podužnim trapeznim ukrucenjima - korubama, poprečni nosači na međusobnom razmaku od 3,75m, i vertikalne stranice korita, visine 700mm i debljine 14mm, koje ne učestvuju u preseku. Korube se nalaze na osovinskom razmaku od 715mm, visine su 285mm, širine 350mm i debljine 8mm, formirane su hladnim mehaničkim savijanjem i podužno su zavarene za kolovozni lim. Kolovozni lim je sa dvostranim nagibom od 2.5% i podužno je kontinualno zavaren za donji pojas glavnog nosača. Korube prolaze kontinualno kroz poprečni nosač. Zbog smanjenja koncentracije naprezanja otvori na rebro poprečnog nosača su posebno konstruisani, a koruba je zavarena uz rebro poprečnog nosačana sa 2/3 svoje visine. Kolovozni lim i korube su od čelika S235J2G3, osim u zoni oslonaca S2, S3, S4 i S5 gde su od čelika S355J2G3 kao i donji pojas glavnog nosača. Da ne bi sadejstvovala sa glavnim nosačima i na taj način bile izložene deformaciji, vertikalne stranice korita sa lamelom na vrhu se prekidaju na svakih 6m. Razmak između dva segmenta od 15mm se u donjoj zoni u visini od 100mm zavaruje kako bi se sprečilo oticanje vode iz korita, a iznad toga se prekriva uzanom čeličnom trakom zavarenom sa unutrašnje strane samo uz jedan segment od 6m. Stabilnost vertikalnog lima obezbeđuju ukrucenja zavarena sa spoljne strane korita na svakih 725 mm.

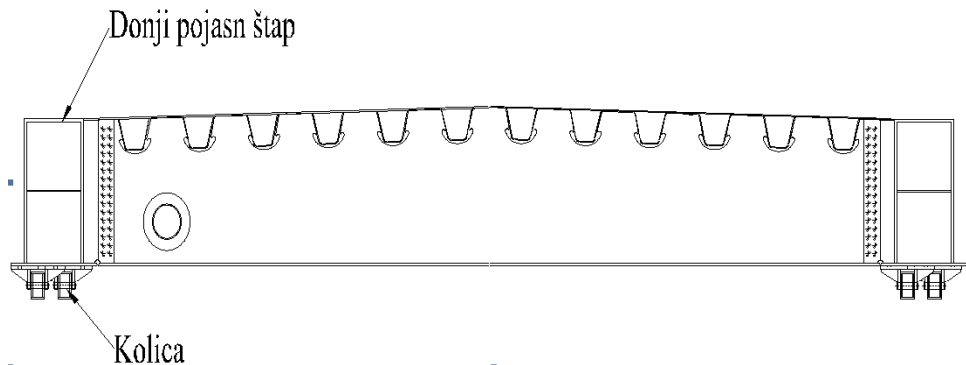
Kako je razmak čvorova glavnog nosača 7,5m to se poprečni nosači nalaze ne samo u čvorovima već i u sredinama polja donjeg pojasa, odnosno na svakih 3,75m. Visina poprečnog nosača se linearno menja od 1300mm uz donji pojas, do 1403mm u sredini raspona. Debljina vertikalnog lima je 16mm. Donja lamela je dimenzija 300x20mm. Na rebro poprečnog nosača postoje otvori Ø300 za prolaz cevi za kišnu kanalizaciju. Iznad obalnih stubova na mestima prelaska sa čelične konstrukcije na betonski parapet nalaze se sekundarni poprečni nosači manje visine, koji ujedno i zatvaraju tucaničko korito. Poprečni nosači su od čelika S355J2G3, osim sekundarnih poprečnih nosača iznad obalnih stubova koji su od čelika S235J2G3. Porečni nosači su vezani za glavni nosač zavarivanjem: donje lamele sučeono na donju lamelu donjeg pojasa glavnog nosača, a rebro za vertikalni lim donjeg pojasa. Na mestu priključka rebra poprečnog nosača se u donjem pojasu glavnog nosača nalazi vertikalna dijafragma u ravni rebra poprečnog nosača. Poprečni nosač je ujedno i vertikalala donjeg sprega.

Pešačku stazu sačinjavaju konzole IPB1 200 zavarene za glavni nosač na svakih 3,75m, podužni nosači U120 i gazeći lim $\neq 1000 \times 5$ mm. Ograda je od šupljih hladno oblikovanih profila. Sa spoljne strane leve pešačke staze se montiraju dva ugaonika 80x80x8, koji nose kanal dimenzija 300x200mm za nošenje SS i TT kablova. Kanal je izrađen od čeličnog lima debljine 3mm i sa unutrašnje strane je termoizolovan sa oblogom debljine 10mm. Kanal sa gornje strane ima poklopac. Izrađuje se u segmentima od 2m a potom se pomoću objimice nastavlja.

1.3 TEHNOLOGIJA MONTAŽE

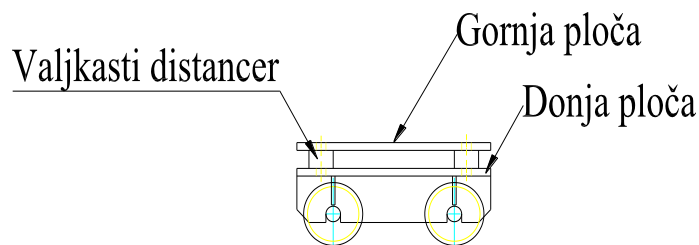
Montaža – ukрупnjavanje čelične konstrukcije mosta vršena je na obali reke Velike Morave iz sa početkom montaže elemenata u osi S6). Plato za ukрупnjavanje konstrukcije je izrađen u dimenzijama 15 m širine i 80m dužine i bio je opremljen sa mobilnom zaštitnom kućicom za izvođenje zavarivačkih radova i radova na antikorozijskoj zaštiti u dužini 50m. Betonska osnova za lansirnu rampu urađena je u dužini od 80m. Mobilna zaštitna kućica je bila široka 15m i 25 m duga sa visinom od 10m. Zaštitna kućica se prevlačila preko šina po platou za sastavljanje uzdužno po osi mosta. Ukрупnjavanje konstrukcije je vršeno prvo montiranjem levog i desnog kutijastog nosača donjeg pojasa u dužini 2m od kraja radnog platoa sa strane Beograda u pravcu Niša. Kutijasti nosači donjeg pojasa spajani su poprečnim nosačima formirajući potrebnu geometriju čelične konstrukcije.

Ispod svakog čvornog lima nosača (na svakih 7,5m) postavljana su po dva kompleta kolica, koja su prilikom naguravanja preuzimala opterećenje namontirane čelične konstrukcije, omogućavajući naguravanje po lansirnoj rampi.



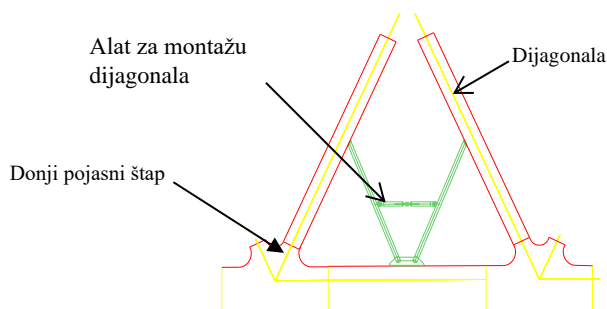
Slika 3 – Položaj kolica ispod donjeg pojasa

Gornja i donja ploča kolica međusobno su nezavisne i njihovo rastojanje je regulisano valjkastim distancerima, koji su postavljeni između njih u cilju praćenja nivelete konstrukcije.



Slika 4 – Izgled kolica ispod donjeg pojasa

Nakon montaže podužnih nosača donjeg pojasa, poprečnih nosača i kolovozne ploče, vršeno je postavljanje dijagonala. Dijagonale su pozicionirane uz korišćenje posebnih alata, prikazanih na slici 5. Alat se postavlja na donji pojasni nosač i učvršćuje se vijcima sa obe strane donjeg kutijastog nosača i to na polovini razmaka između čvornih limova što je omogućavalo podešavanje uglova dijagonale. Dijagonale se prilikom nastavljanja na donji kutijasti nosač sa donje strane osiguravaju pripojnim elementima.

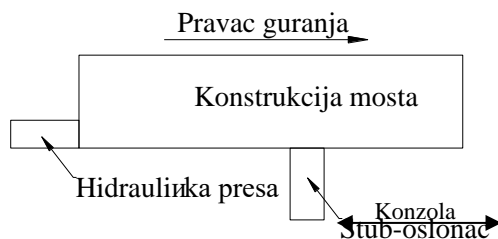


Slika 5 – Izgled alata za montažu dijagonala

Nakon završene montaže dijagonala i njihovog osiguravanja od promene položaja tokom montaže, vrši se montaža gornjih pojasnih nosača, koji se poprečno osigurava postavljanjem gornjih spregova.

Nakon zavarivanja svih spojeva na jednom segmentu mosta, formira se jedno montažno polje čije se pomeranje vrši uz pomoć hidrauličkih presa u smeru ka Nišu. Podužnim pomeranjem, naguravanjem ukрупnjenog i zavarenog polja mostovske konstrukcije, na platou za ukрупnjavanje bi se oslobađalo mesto za ukрупnjavanje sledećeg polja.

Na blok dijagramu prikazani su delovi sistema za guranje konstrukcije.





Hidraulička presa od 300 bar-a za pomeranje konstrukcije u pravcu naguravanja. Hod prese je bio 1050 mm, nakon čega je bilo potrebno presu prepakovati kako bi se moglo izvršiti naguravanje u sledećem ciklusu. Klip hidraulične prese gura konstrukciju ispred sebe, ona kliza preko stubova i tako se ostvaruje pomeranje konstrukcije.

Slika 6 – Izgled hidrauličke prese

Na stubovima se nalaze nosač i klizač koji omogućavaju klizanje-pomeranje konstrukcije.



Slika 7 – Izgled konstrukcije za klizanje na stubu-osloncu

Sa gornje strane nosača nalazi se lim od nerđajućeg čelika, a sa donje strane klizača teflon (na slici se vidi ko beli krug). Klizač se postavlja ispod konstrukcije (ostvaruje se kontakt prohrom - teflon) na početak nosača i na sebi nosi konstrukciju do kraja nosača. Tada se prekida sa guranjem konstrukcije i sa hidrauličkom presom na stubu podiže konstrukcija i klizač se oslobađa i vraća na početak nosača, nakon toga konstrukcija se spušta na klizač i vrši se ponovno klizanje u smeru guranja.

Ovo predstavlja jedan ciklus guranja. Ciklus guranja zavisi od dva parametra, prvi je dužina klipa hidrauličke prese koja gura konstrukciju, a drugi je dužina klizanja klizača

preko nosača koji je uslovljen naponima u konstrukciji i vertikalnom limu iznad oslonca u fazi guranja.

2 SPECIFIČNOSTI MONTAŽE

Niveleta železničkog mosta u Čupriji je u vertikalnoj krivini čiji je radijus 30.000 m, tako da je bilo potrebno da se svaki valjkasti distancer na kolicima posebno podešava kako bi se zadovoljila projektovana zakrivljenost nivelete mosta. Obzirom da lansirna rampa nije bila radijalnog već poligonalnog oblika, posebna pažnja je bila posvećena permanentnom geodetskom podešavanju visina kolica ispod svakog čvornog lima i ovo predstavlja prvu specifičnost montaže ove konstrukcije.

Drugi problem koji se javljao tokom montaže bila je pojava napona upravnih na glavni pravac napona tj. lokalni naponi. Ovo složeno naponsko stanje javljalo se u zonama oslonaca kada se vršilo pomeranje klizača na početak nosača radi obezbeđivanja sledećeg ciklusa guranja ili u slučaju nepredviđenih okolnosti nastalih tokom guranja konstrukcije kada je bilo potrebno zadignuti konstrukciju zbog intervencije. Naime u tim slučajevima se vršilo podizanje konstrukcije na određenu visinu iznad privremenih ležišta. U tim fazama bili su opterećeni vertikalni limovi donjeg pojasa rešetke pogotovo u sredini raspona između dva susedna čvora gde je sa presom bilo potrebno anulirati reakciju na osloncu, gde već postoje određeni naponi od gravitacionog opterećenja.

Ovaj problem je rešavan tako što se za svaku fazu guranja i zadizanja konstrukcije vršila kontrola uporednog napona i računala potrebna dužina naleganja konstrukcije preko klizača na nosač na stubu.

3 ZAKLJUČAK

U ovom radu prikazan je inženjerski problem koji se javljao na gradilištu u fazi montaže čelične konstrukcije za slučaj montaže primenom tehnologije naguravanja. Izvođenje radova na montaži mosta je zahtevalo složen inženjersko tehnički pristup svim pozicijama rada uz stalno sadejstvo projekatara i inženjera odgovornih za tehnologije i operativno rukovođenje radovima.

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Duška Nedeljković¹, Nemanja Filipović², Ivana Todorović³, Vladimir Vujčić⁴, Jelena Bojić⁵

NOVI ŽELEZNIČKI MOST PREKO REKE TAMIŠ U PANČEVU

Rezime:

S-65

Novi železnički most preko reke Tamiš izgrađen je u okviru izgradnje drugog koloseka pruge, na deonici Pančevački most- Pančevo Glavna stanica. Most je jednokolosečni sa kolosekom postavljenim u tucaničkom zastoru. Projektovana brzina vozova je 120 km/h. Most se sastoji od glavne čelične konstrukcije raspona 82,4 m iznad reke i od pet spregnutih prilaznih konstrukcija raspona 32 m. Kolovozna ploča je od armiranog betona. Kompletne čelične konstrukcije proizvedene su u fabrici preduzeća GP "Mostogradnja". Most se oslanja na sedam armirano betonskih stubova, fundiranih na bušenim HW šipovima (130 kom). Ukupna masa čelične konstrukcije iznosi približno 670 tona, a količina ugrađenog betona je približno 6.500 m³. Sve radove je izvodila GP "Mostogradnja".

Ključne reči: Tamiš, Most, Bušeni šipovi, AB stubovi, Čelična rešetka, Spregnuta konst.

NEW RAILWAY BRIDGE OVER RIVER TAMIŠ IN PANČEVO

Summary:

New railway bridge over river Tamiš was built within construction of the second railway track on Section Pančevo Bridge – Pančevo Main Station. Bridge is one tracked, with track laid in a road bed. Designed speed of trains is 120 km/h. Bridge consists of Main steel structure with span of 82,4 m above river and of five composite structures with span 32 m. Bridge deck is made of reinforced concrete. Complete steel structure was made in GP "Mostogradnja's" factory. Bridge is supported on seven reinforced concrete piers, with foundation on bored HW piles (130 pieces). Total mass of steel structure is approximately 670 tones, and amount of laid concrete is approximately 6.500 m³. All works has been done by GP "Mostogradnja"

Key words: Tamiš, Bored Piles, Reinforced concrete columns, Steel truss, Composite structure

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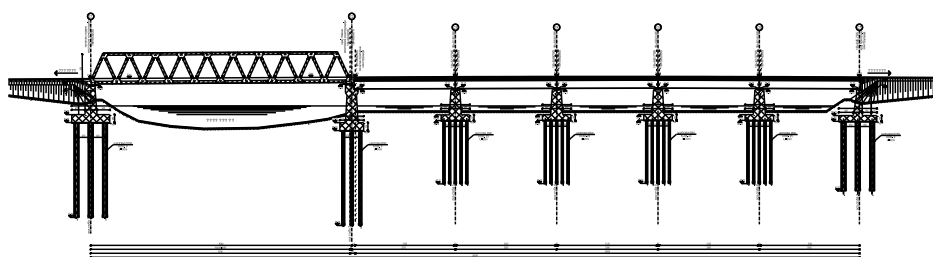
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1 TEHNIČKI OPIS I KARAKTERISTIKE MOSTA

Novi železnički most preko reke Tamiš nalazi se na 18. km pruge Beograd-Pančevo. Most je projektovan paralelno sa postojećim železničkim mostom na osnom rastojanju od 13m. Raspored stubova novog mosta je identičan rasporedu stubova postojećeg mosta, tako da su i rasponi novog mosta jednaki rasponima starog pa je $L = 81,9 + 5 \times 32 = 241,9$ m. Za prilaznu konstrukciju usvojeno je rešenje sa spregnutim glavnim nosačem statičkog sistema proste grede raspona 31,2 m. Za statički sistem mosta preko same reke Tamiš usvojena je čelična rešetkasta prosta greda raspona $L=81,9$ m, osne visine $H=9,0$ m.



Dispozicija mosta

Izgradnja Novog železničkog mosta preko reke Tamiš je počela u julu 2014. godine i trajala je do avgusta 2015. godine. U toku izgradnje mosta u nekoliko navrata dolazilo je do prekida radova usled izlivanja reke Tamiš. Uprkos prekidima radova, većih kašnjenja u izgradnji mosta nije bilo.

Svi radovi na izgradnji mosta bili su podeljeni na betonske radove – stubovi i temelji, radove na spregnutoj konstrukciji inundacije i radove na montaži glavnog čeličnog raspona.

2.1 IZRADA ŠIPOVA

2.1.1 Pripremni radovi

Usled izrazito močvarnog tla, a da bi se omogućio pristup mehanizaciji i obezbedili uslovi za rad, početna faza realizacije projekta podrazumevala je pripremne zemljane radove, koji su se sastojali iz izrade pristupnog puta i radnih platoa.

Položaj stuba S2 je bio u rečnom koritu pa je za njega bilo potrebno izraditi nasip sa dodatnom stabilizacijom tj. kamenom oblogom.

Radna platforma za izradu krajnjih stubova S1 i S7 je ulazila u keglu postojećeg mosta. Visina platforme je određena na osnovu maksimalne moguće dužine kolone mašine za bušenje šipova (maksimalno 40m).

Visina radne platforme za stubove S3-S6 određena je na osnovu srednjih nivoa vodostaja reke Tamiš.



Izrada prostupnog puta i radnog platoa

2.1.2 Potrebna mehanizacija

Za izradu ukupno 130 kom bušenih zacevljenih šipova prečnika Ø600 (100 kom), Ø900 (12 kom), Ø1500 (18 kom) korišćena je sledeća mehanizacija:

- Mašina za iskop (bušenje) Delmag RH32,
- HW oprema,
- Kombinovana mašina i utovarivač za razastiranje materijala,
- Autodizalica P&H 670.

2.1.3 Fundiranje

Stubovi S3, S4, S5, S6 su fundirani na po 25 šipova prečnika Ø600 mm dužine 20 m . Stubovi S1 i S7 su fundirani na 9 šipova prečnika Ø1500 mm dužine 30m, odnosno 22m. Stub S2 je fundiran sa 12 šipova prečnika Ø900 dužine 30 m.

Materijal koji je iskopavan prilikom izrade šipova je u prvih 0-1m šljunak pomešan sa glinom a ispod tog sloja nalazi se 7-8m dubok sloj sitnog peska. Na dubini oko 10m nailazili smo na sloj gline debljine 0-2m. Ispod gline nalazi se sloj vrlo sitnog peska koji je izrazito nepovoljno deluje na opremu za iskop. Dolazi do oštećenja grajfera kod HW tehnologije i svrdla (alata) kod Delmag mašine za bušenje.

Za betoniranje šipova korišćen je podvodni beton sa povećanom količinom cementa (400 kg/m³ betona).



Izrada bušenog šipa mašinom za iskop Delmag RH 32

2.2 IZRADA STUBOVA

2.2.1 Prethodni radovi

Za potrebe izvođenja stubova projektovana je zaštita temeljnih jama u vidu priboja. Za izradu priboja korišćene su talpe tipa Larssen 604n, dužine 8 m i Larsen 607n dužine 16 m. Kao razupirači projektovani su čelični ramovi.

Zbog prisustva gline i sitnog peska izrada zidova priboja vršena je jako sporo. Čelične talpe su se pobijale pomoću PTC mašine i Delmag čekića D12 i D22 (težine klipova 1200 kg odnosno 2200 kg).

Iskop materijal iz priboja se vršio podvodno kako ne bi došlo do proboja dna tj. punjenja priboja novim materijalom.

Da bi se obezbedio rad u suvim uslovima betonirani su tampon slojevi. Glavnim projektom debljine tampon slojeva su 20 cm što je bilo nedovoljno da obezbedi rad u suvom. Izvođačkim projektom priboja bilo je neophodno definisati nove debljine tampon slojeva koje su se kretale od 80 cm do 110 cm.

Nakon očvršćavanja betona tampon sloja pristupano je crpljenju vode iz priboja. Veliki problem su predstavljali ostaci drvenih šipova koji su korišćeni prilikom izgradnje starog mosta, a koji su se nalazili delimično u tampon sloju. Na tim mestima tampon sloj nije imao dovoljnu debljinu betona i dolazilo je do proboja odnosno prodiranja vode i sitnog peska (muljevitog materijala) u priboj. Sanacija takvih oštećenja zahtevala je podvodno razbijanje zone oko drvenih šipova, ponovni iskop „mamut“ pumpom i dodatno podvodno betoniranje.

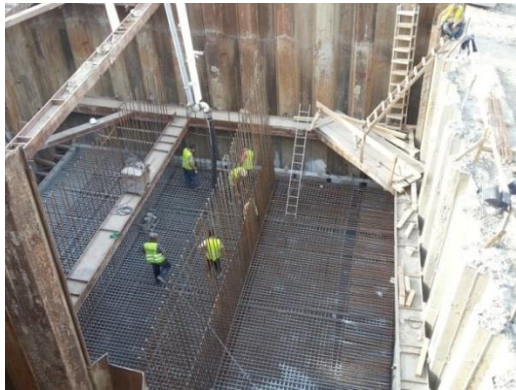
Priboji na stubovima S1 i S7 su delimično ušli u nasip stare pruge koja je bila u funkciji. Zbog opterećenja od vozova neophodno je bilo projektovati zaštitu postojeće konstrukcije mosta a unutar priboja postaviti dva rama I poluram. Dubina iskopa unutar priboja je ~10 m. Nakon izrade prve faze stubova vršilo se zatrpavanje i sukcesivno uklanjanje ramova.



Betoniranje tampon sloja i pobijanje talpi priboja

2.2.2 Izrada naglavnih greda

Kao oplata naglavnih greda korišćeni su specijalno izrađeni AB elementi dimenzija 120x12x250 (oplatne ploče sa gredom na vrhu) koji su ostali zabetonirani kao sastavni deo naglavne grede. Ploče su premazivane hidroizolacionim premazom tako da su služile i kao hidroizolacija naglavnih greda.



Betoniranje naglavne grede stuba S1

2.2.3 IZRADA STUBOVA

Donji stroj mosta čine armirano-betonski stubovi fundirani na šipovima sa naglavnim gredama.

Stub S2 je najviši sa visinom od 12,29 m dok je stub stub S1 najširi sa širinom od 12 m.

Stubovi su betonirani na licu mesta u projektovanoj oplati u dve faze (stubovi S2-S6) odnosno tri faze (stubovi S1 i S7 – krajnji stubovi).

Kao oplata za stubove na ravnim delovima korišćene su tipske metalne table a na lučnim delovima koristila se posebno izrađena drvena oplata u kombinaciji sa čeličnim gredama.



Betonaža II faze stuba S3

2.3. MONTAŽA KONSTRUKCIJE

Radovi na izradi elemenata čelične konstrukcije obavljali su se u proizvodnom centru Batajnica GP “Mostogradnja”. Na gradilištu je vršena samo njihova montaža.

2.3.1. Montaža prilazne konstrukcije (Inundacija)

Prilaznu konstrukciju (inundaciju) čini 5 sistema proste grede raspona $L=31,2$ m.

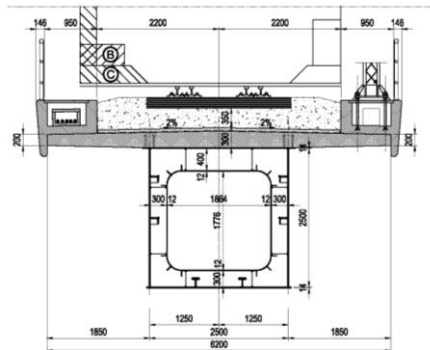
Kolovozna ploča je nominalne debljine 30 cm. Gornja ivica kolovozne ploče je u obostranom padu od 2%. Na mestu stubova postavljene su dilatacione spojnice koje omogućuju slobodno dilatiranje posebnih konstrukcija.

Čelični deo konstrukcije je sanduk u zavarenoj izradi koji se sastoji od glavnih nosača i poprečnih ukrčenja.

Veza tj. sprezanje između betonske ploče i čeličnog sanduka ostvarena je pomoću moždanika.

Način montaže: Na sredini raspona pravljen je privremeni oslonac – Jaram koji je služio da prihvati oba montažna elementa koji su nakon toga spajana zavarivanjem na licu mesta.

Težine čeličnih montažnih komada iznosila je približno 21 t.



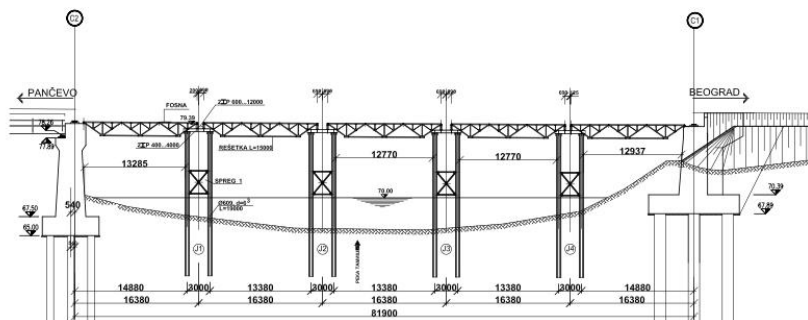
Jaram i montažni element - Poprečni presek

2.3.2. Montaža glavne konstrukcije

Za statički sistem konstrukcije na delu preko reke Tamiš usvojena je rešetkasta prosta greda raspona $L=81,9$ m.

Kolovoznu konstrukciju sačinjavaju spregnuti nosači i AB ploča debljine $d=20$ cm.

Za potrebe montaže glavne čelične konstrukcije preko reke Tamiš, neophodno je bilo montirati privremenu konstrukciju na vodi. Formirana su četiri jarma međusobno spojena tipskim rešetkama dužine 25 m (ukupno 4 po polju). Pobijanje šipova ($\varnothing 609$) za izradu jarmova vršeno je sa autodizalicom + PTC vibro mašinom koji su se nalazili na plovnom objektu.



Dispozicija jarmova na vodi

Nakon izrade jarmova na vodi počela je montažna glavne konstrukcije. Prvo je montirano polje od stuba S2 do jarma J1 polaganjem donjeg pojasa rešetke (uzvodnog i nizvodnog). Posle toga montirani su poprečni nosači i spregovi. Na ovako formiranu krutu figuru montirane su dijagonale uz pomoć alata koji dovodi dijagonale u projektovani položaj. Preko dijagonala su montirani gornji pojasevi koji su ukrućeni gornjim spregom. Ovim je završena montaža prvog polja. Ovakav postupak ponovljen je između ostalih jarmova sve do stuba S1: Na jarmove su postavljeni privremeni oslonci tj. lonci sa peskom, koji su nakon montaže mosta otpušteni. U toku izrade glavnog čeličnog nosača formiran je kontinualni statički sistem a po otpuštanju privremenih oslonaca prelazi u prostu gredu.

Ukupna težina glavne konstrukcije iznosi 459,5 t.



Izrada jarmova na vodi - Pobijanje šipova Ø609

Posle završetka montaže i zavarivanja glavne konstrukcije pristupilo se demontaži jarmova na vodi. Šipovi sa uzvodne strane mosta su se izvlačili uz pomoć autodizalice i vibro mašine postavljenih na plovnim objektima. Šipovi na nizvodnoj strani mosta (prema starom mostu) su isečeni do dna reke uz angažovanje ronioca (nije ih bilo moguće izvući zbog položaja starog mosta).



Montaža glavne konstrukcije



Demontaža jarmova na vodi - Sečenje šipova Ø609

3. ISPITIVANJE MOSTA

Ispitivanje mosta preko reke Tamiš izvršio je Saobraćajni institut CIP .

3.1 PROBNO OPTEREĆENJE

Ispitivanje mosta je vršeno pomoću dva tipa probnog opterećenja Statičkog i dinamičkog.

- Statičko opterećenje: angažovana je lokomotiva serije 661 i osam vagona tipa VPM - 770. Osovinski pritisak vagona VPM-770 iznosio je $4 \times 200 \text{ kN} = 800 \text{ kN}$. Položaj kompozicije je pomeran tako da karakteristični preseći budu maksimalno opterećeni.
- Dinamičko opterećenje :Vršeno je pri kretanju lokomotive različitim brzinama $V=10\text{km/h}$, $V=40\text{km/h}$, $V=80 \text{ km/h}$, $V=100 \text{ km/h}$. Most je projektovan za brzine do $V=120\text{km/h}$ ali zbog nedostatka pruge (još uvek nije bila završena celokupna deonica) u smeru ka Beogradu u trenutku ispitivanja mosta nije bilo moguće dostignuti projektovanu brzinu.

3.2 REZULTATI PROBNOG OPTEREĆENJA

3.2.1. Ponašanje konstrukcije pod statičkim probnim opterećenjem

Veličine vertikalnih pomeranja – ugiba konstrukcije, izložene dejstvu statičkog probnog opterećenja, za svaki njegov položaj (fazu), izmerene su sa uzvodne i nizvodne strane.

Sva merenja izvršena su sa geodetskom tačnošću 0,1 mm.

Maksimalna izmerena deformacija – ugib na glavnoj rešetkastoj konstrukciji iznosi: $\max \delta_{mer} = 38,7 \text{ mm}$, što predstavlja 1/2130 raspona.

Odgovarajuća računsa vrednost maksimalnog ugiba za primenjeno probno opterećenje iznosi : $\max \delta_{rač} = 42,0 \text{ mm}$.

Maksimalni izmerena ugib na prilaznim konstrukcijama iznosi: $\max \delta_{mer} = 12,4 \text{ mm}$, što predstavlja 1/2580 raspona.

Odgovarajuća maksimalna računsa vrednost maksimalnog ugiba za primenjeno probno opterećenje iznosi : $\max \delta_{rač} = 12,0 \text{ mm}$.

3.2.1. Ponašanje konstrukcije pod uticajem dinamičkog probnog opterećenja

Dinamičko ispitivanje mosta izloženog uticaju dinamičkog probnog opterećenja obuhvatilo je registrovanje:

- Dinamičkog koeficijenta izazvanog kretanjem lokomotive,
- Vertikalnih vibracija u preseku u sredini raspona konstrukcije
- Koeficijenta prigušenja konstrukcije.

Dobijeni dinamički dijagrami su pravilni, imaju miran i ustaljen tok i male amplitude oscilovanja. Amplitude vibracija se relativno brzo prigušuju i ukazuju na dovoljnu krutost konstrukcije.

3.3 ZAKLJUČAK SA ISPITIVANJA MOSTA

Na osnovu rezultata pregleda konstrukcije pod dejstvom statičkog i dinamičkog probnog opterećenja, na osnovu opšteg elastičnog ponašanja konstrukcije i na osnovu sprovedene teorijske analize zaključeno je da je novi železnički most preko reke Tamiš sposoban da primi i prenese projektovano opterećenje.



Ispitivanje mosta



Pogled sa reke

Krstan Laketić¹

LEPLJENO LAMELIRANO DRVO RASPONA 72.5 m NIŠ, MEDIJANA, ARHEOLOŠKO NALAZIŠTE

Rezime:

Prostorna konstrukcija od lepljenog lameliranog drveta je napravljena sa rasponom od 72,5m kao zaštitna konstrukcija arheološkog nalazišta "Vila sa peristilom" na Medijani u Nišu. Pokrivanje konstrukcije je izvršeno PVC membranom. Prostorni sistem čine centralni brod sa nosačima statičkog sistema luka na dva zgloba, na koji su priključene dve polukalote u kojima su zrakasto raspoređeni nosači sistema luka na dva zgloba raspona 36,25m. Radove na LLD konstrukciji je izvršila firma Piramida doo iz Sremske Mitrovice.

Ključne reči: lepljeno, lamelirano, drvo, medijana, čelik, okov, luk

GLUED LAMINATED TIMBER SPANNING 72.5 m NIŠ, MEDIJANA, ARCHAEOLOGICAL SITE

Summary:

Space structure in glued laminated timber spanning 72,5m is made as a protective structure for archaeological site "Vila sa peristilom" in Medijana, Niš. Covering of structure is done with PVC membrane. Space structure is composed of central part of the structure where the main frame is two hinged arch, on which are attached two semi domes in which are two hinged arches radially spaced across the span of 36,25m. Works on glulam construction were done by company Piramida doo, Sremska Mitrovica.

Key words: lepljeno, lamelirano, drvo, medijana, čelik, okov, luk

¹ Piramida doo, Sremska Mitrovica

1 MODELIRANJE KONSTRUKCIJE

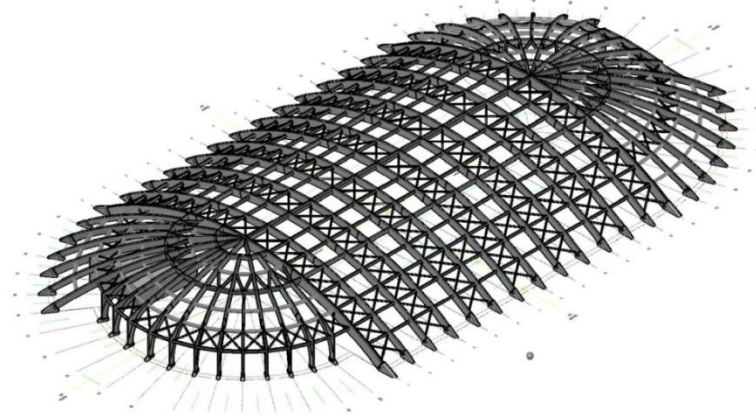
Analiza konstrukcije od lepljenog lameliranog drveta je urađena prostornim modeliranjem konstrukcije u softveru Tower. Sistem je modeliran kao prostorni sistem. Konstrukciju čini centralni brod gde je zastupljen niz ravanskih nosača statičkog sistema luka na dva zgloba raspona 72,5m raspoređenih na međusobnom osovinskom rastojanju od 5,85m. Svi nosači od lepljenog lameliranog drveta su oslonjeni na armirano betonske oporce koji su u proračunu modelirani kao nepokretni oslonci. Na centralni brod se nastavljaju polukalote sa oba čela objekta u kojima su radialno raspoređeni nosači statičkog sistema luka na dva zgloba. Radijus polukalote je 36,25m. Primarni glavni nosači u polukalotama imaju jedan oslonac na betonskim oporcima, a drugi oslonac je u slemenu centralnog broda. Sekundarni i tercijarni glavni nosači u polukalotama su jednako lukovi na dva zgloba, bez obzira na oblik nosača, iako su prekinuti podvlakama između nosača višeg ranga u polju polukalote prema projektovanoj geometriji. Aksijalni rad glavnih nosača u polukalotama nameće pojavu horizontalne sile u slemenu centralnog broda koja se uravnotežava sa silom iz naspramne polukalote. Arhitektonskim oblikovanjem objekta je uvedena nesimetrija u formi konstrukcije, polukalote nisu jednake. Ova nesimetrija remeti pravilan rad konstrukcije, dovodi do preopterećivanja jedne strane konstrukcije, a rasterećivanja druge. Narušena je idealna raspodela sila u konstrukciji. Ovo dalje znači da možemo očekivati nepravilan odgovor konstrukcije usled horizontalnih pobuda, te i nesimetrično deformisanje u slučaju vertikalnih tereta. Maksimalno angažovanje nosača u polukalotama na prijem aksijalnih uticaja, i na taj način eliminisanja dela savijanja, je omogućeno uvođenjem pritisnutog prstena u sistem konstrukcije. Pritisnuti prsten kao i rešetkasta ispuna dela kalote pospešuje krutost sistema i omogućavaju povoljno preraspodeljivanje uticaja u konstrukciji u smislu anuliranja negativnog efekta nesimetrične forme konstrukcije u cilju kontrolisanja rada objekta i izbegavanja, odnosno smanjivanja efekta, nesimetričnog deformisanja konstrukcije.

Praktično centralni brod je ravanski sistem. Nosačima u polukalotama je omogućeno da prihvataju aksijalne uticaje pomoću pritisnutog niza štapova u slemenu konstrukcije kao i formiranog pritisnutog prstena u konstrukciji. Ovakvo oblikovanje zapravo i čini ovu konstrukciju prostornim sistemom. Za projektovanu raspodelu uticaja je neophodno da kompletna konstrukcija bude završena i da se svi naočigled skriveni statički sistemi u konstrukciji angažuju na predviđeni rad putem pravilnog vođenja grednih elemenata u konstrukciji, te odgovarajućim oblikovanjem čeličnih veza, odnosno prenosom sila kako je to predviđeno pretpostavkama statičkog proračuna.

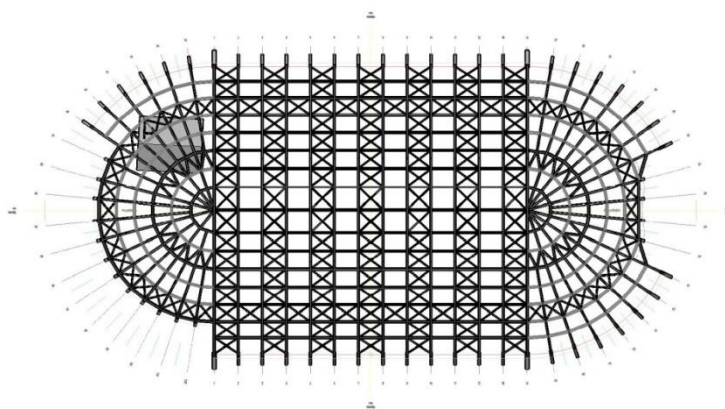
Modeliranje konstrukcije za potrebe formiranja grafičke dokumentacije je urađeno softverskim paketom AutoCAD. Korišćen je pristup modeliranja u tri dimenzije uz pomoć oblikovanja elemenata konstrukcije pomoću punih tela - solida. Svi nosači se oblikuju naspram svojih realnih mera u prostoru kao puna tela koja sadrže informacije o volumenu i okupiranju prostora. Pomoću ovakvog modeliranja se omogućava sagledavanje realnog utroška materijala, lako utvrđivanje tačnih mera i rastojanja između elemenata konstrukcije te pronalaženje eventualnih zanemarenih prodora među elementima konstrukcije koji ravanskim crtežima možda ne bi bili sagledani. Na osnovu modelirane forme drvene konstrukcije na 3D modelu se zatim radi razrada čeličnih okova. Okovi se dimenzionišu prema uticajima na mestu okova, a oblikuju u tri dimenzije prema tačnom prilazu svih pripadajućih grednih elemenata drvene konstrukcije. Moguće je sagledati prilaz svakog štapa, način ugradnje svakog drvenog štapa,

svakog dela sklopa čeličnog okova te svakog spojnog sredstva u sklopu uz eventualno nametanje redosleda montaže kako bi ugradnja bila moguća. Na osnovu konačnog crteža, kada su ucrtani svi elementi, usvojeni svi položaji štapova u sklopu, položaji svih spojnih sredstava te izvršena kontrola i eliminisani svi prodori između drvenih štapova i/ili čeličnih elemenata, i kao krajnje - kada su uvedena eventualna krojenja nosača i limova koja imaju samo estetski efekat, iz grafičkog modela se brzo dobijaju ortogonalne projekcije objekta, radionički crteži i specifikacija materijala na osnovu kojih se zatim dalje radi proizvodnja i krojenje nosača, sečenje limova i sklapanje okova, planovi za montažu kao i detaljni crteži za projektnu dokumentaciju objekta.

Zaštitnom konstrukcijom je natkriveno 9200m^2 . Za izvođenje kompletne zaštitne konstrukcije je potrebno 176m^3 ravnog i 618m^3 lučnog lepljenog lameliranog drveta kao i 68500kg čeličnih okova i spojnih sredstava.



Slika 1 - Grafički model konstrukcije



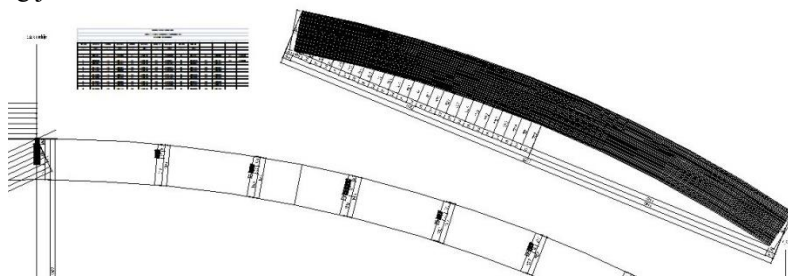
Slika 2 - Grafički model konstrukcije

2 PROIZVODNJA KONSTRUKCIJE

Na osnovu razrađene grafičke dokumentacije vrši se proizvodnja elemenata konstrukcije. Pravi se detaljna radionička dokumentacija za nosače od lepljenog lameliranog drveta, radionička dokumentacija i planovi ugradnje čeličnih okova neophodnih za povezivanje konstrukcije te planovi za montažu i sklapanje elemenata konstrukcije na gradilištu. Tokom razrade se vrši fino doradivanje osnovne projektantske ideje te uvode tačni položaji montažnih nastavaka u proračun i crteže, te tačni položaji greda u sistemu u cilju očuvanja usvojenog statičkog sistema konstrukcije. U toku razrade projekta vodilo se računa o tehnološkim ograničenjima fabrike te ograničenjima putnim pravcima kojima će konstrukcija biti vožena do gradilišta. Plan montaže je uslovljen stanjem na terenu i rešavan je kao presek problema na terenu sa već oformljenim gredama koje mogu biti proizvedene i dopremljene na gradilište.

2.1 PROIZVODNJA DRVENIH ELEMENATA KONSTRUKCIJE

Prema projektovanom obliku nosača, na osnovu usvojenog rasporeda montažnih nastavaka izrađuje se radionički crtež glavnog nosača u konstrukciji. Poprečni presek glavnih nosača u polju konstrukcije, u centralnom brodu je $b/h = 20/150\text{cm}$. Karakteristični su krajnji nosači centralnog broda preko kojih se ostvaruje veza sa polukalotama na čelima objekta. Ovi nosači su zbog povećanog opterećenja koja prihvataju u slemenu od dodatih polukalota morali biti moćnijeg poprečnog preseka. Usvojen je presek nosača $b/h = 40/200\text{cm}$. U okviru radioničke dokumentacije za nosače se obrađuje oblik trase stegača na kojoj će nosač biti lepljen, te broj i dužina svake lamele koja čini slog poprečnog preseka nosača. Na osnovu ovih crteža se zatim u proizvodnom pogonu prave lamele dovoljne dužine koje se zatim u presi oblikovanoj prema trasi stegača lepe u projektovani nosač. Nosači su napravljeni od četinarske građe, jele i smreke. Lepljenje se vršilo dvokomponentnim melaminskim lepilom, specijanim lepilom za upotrebu u konstrukcijama. Svi nosači su završno hoblani i premazani lazurnim premazom za zaštitu od gljivica i crvotočine.



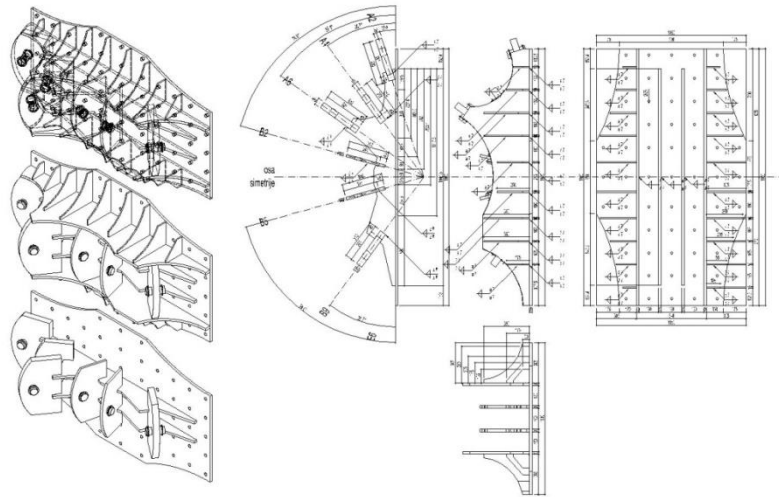
Slika 3 - Radionički crtež LLD nosača

2.2 PROIZVODNJA ČELIČNIH OKOVA

Projektovana forma nosača i usvojeni prostorni sistem konstrukcije određuju tok sila kroz drvenu konstrukciju koji je neophodno obezbediti odgovarajućim oblikovanjem drvenih nosača i prevashodno odgovarajućim oblikovanjem čeličnih okova. Svaki čelični okov koji se ugrađuje u konstrukciju mora da omogući pravilan projektovani prenos sila. U isto vreme okov mora biti ugradljiv, radnik mora da ugradi okov na osnovnu gredu sklopa, te da kasnije

nesmetano prinese i ugradi druge gredne elemente sklopa u okov te da bezproblema ugradi sva predviđena spojna sredstva.

Svi okovi koji su ugrađeni u konstrukciju su napravljeni od čelika S235 i S355. Sklapanje sklopova je urađeno ugaonim varovima odgovarajuće debljine. Zaštita sklopova je izvršena toplim cinkovanjem za sklopove i galvanizacijom za spojna sredstva.



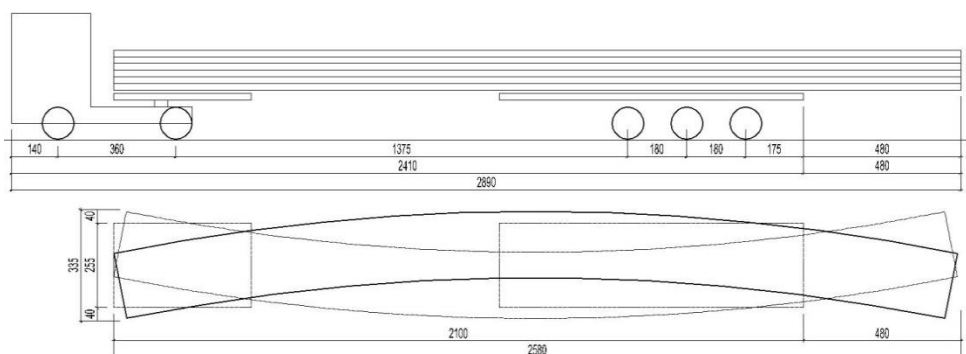
Slika 4 - Radionička dokumentacija - Slemeni okov



Slika 5 - Slemeni okov

3 TRANSPORT I MONTAŽA

Transport konstrukcije je izvršen kao specijalni, vanredni transport sa pratnjom. Delovi nosača koji su transportovani su bili dužine 25,80m. Odgovarajućim vučnim vozom, u smislu tegljača sa prikolicom na razvlačenje je omogućen transport ovako dugačkih greda. Ukupna dužina vučnog voza sa teretom je 28,90m. Prilaz objektu je izabran iz uslova mogućnosti skretanja sa magistralnog puta bez opasnosti po vučni voz i teret. U ovom slučaju je korišćen prilaz lokalitetu iz pravca Niške banje jer ta saobraćajnica dopušta najveću propusnost za vanredni transport.



Slika 6 - Šema vanrednog transporta

Zaštitna konstrukcija raspona 72,5m je predviđena na lokalitetu Vile sa peristilom kako bi se natkrio jedan od najvećih očuvanih mozaika u ovom delu Evrope. Mozaik se nalazi na podu nekadašnje vile, u polju centralnog broda objekta. Ni u kom slučaju se tokom montaže nisu smeli koristiti privremeni oslonci u polju objekta. Kompletna montaža objekta je morala biti izvršena spolja. Raspon od 72,5m koji je premošten nosačem statičkog sistema luka na dva zgloba sa dva montažna nastavka, greda je iz tri dela, bi zahtevao angažovanje jako velike tri dizalice ukoliko bi se pokušalo sa montažom van polja. Ovakav plan montaže bi značio da jedna dizalica sa jedne strane oporca drži prvi segment nosača, druga dizalica kod drugog oporca drži drugi krajnji segment nosača, a treća vrši ubacivanje srednjeg dela nosača u projektovani položaj. Pristup okovima za ugradnju spojnih sredstava bi se morao rešiti putem laganih skela koje bi bile montirane u polju. Odustali smo od ovako komplikovanog plana montaže.

Lokalitet se nalazi na vodozahvatu Medijane, na poljani koja daje ogroman slobodan prostor za rad van lokaliteta. Kako bi se uspešno izvršila montaža usvojen je metod prevlačenja gotovog polja broda sa placa za predmontažu u projektovani položaj. Plac za predmontažu je postavljen sa čela centralnog broda u prostoru buduće polukalote. Za potrebe prevlačenja su u okolini oporaca formirane dve betonske staze. Vršena je predmontaža jednog polja centralnog broda na placu za predmontažu. Polje je kompletno završeno, ugrađeni su svi okovi i sekundarni nosači, te je izvršeno sprezanje polja čeličnim dijagonalnim štapovima - spregovima, kako bi se obezbedila krutost figure tokom transporta do projektovanog položaja. Kako statički sistem luka na dva zgloba uslovljava postojanje horizontalnog oslonca, razmicanje konstrukcije na osloncima je onemogućeno uvođenjem zatege u oslonačkoj zoni

nosača. Kako bi se omogućio transport konstrukcije u projektovani položaj preko očuvanih zidova Vile koji postoje na lokalitetu, zatega je morala biti postavljena dovoljno visoko iznad oslonaca da ne zakači i ne poruši zidove u toku transporta. Praktično je sila iz zatege uvedena ekscentrično u glavni nosač. Ovo su mali uticaji spram sila koje nosači prihvataju u eksploataciji. Premeštanje nosača je izvršeno putem posebno oblikovanog vučnog voza napravljenog namenski za ove potrebe. Voz se kretao betoniranom trasom od placa za predmontažu do projektovanog položaja transportovanog polja. Voz se kretao preko cevi koje su polagane na betonsku trasu. Postavljanje predmontiranog segmenta na vučni voz, kao i skidanje sa vučnog voza te postavljanje u oslonačke okove je izvršeno putem vešanja segmenta u četiri tačke. Polje od dva nosača je hvatano u svakoj oslonačkoj tački kako bi se sigurno izvršilo translatorno pomeranje konstrukcije. Sa jedne strane je konstrukcija dizana sa dve dizalice, a sa druge pomoću improvizovanog namenski oblikovanog portalnog kрана.

Montaža kompletnog centralnog broda je izvršena polje po polje. Nosači između montiranih polja su naknadno umetnuti ubacivanjem spolja.

Montaža kalota je vršena ubacivanjem nosača spolja ili iz prostora kalote, jer ovi položaji ne ugrožavaju mozaik na lokalitetu. Tokom montaže je rešen i problem postojanja zgrade muzeja na lokalitetu koja je zadirala u prostor glavnih nosača kalote. Problem je prevaziđen finim prerađivanjem postojeće krovne konstrukcije kako bi se propustio glavni nosač nove konstrukcije. Zgrada muzeja nije oštećena niti ugrožena.



Slika 7 - LLD konstrukcija preko zgrade muzeja

Predviđeni obim radova ne obuhvata izvođenje kompletnog objekta Zaštitne konstrukcije te je jedan deo konstrukcije na kraju ostao otvoren do raspisivanja sledeće faze radova, kada će konstrukcija biti kompletirana i završena.



Slika 8 - Predmontaža jednog polja



Slika 9 - Spregnuto polje na vučnom vozu

Prilikom transporta nosača na vučnom vozu, kompletno polje je tokom transporta bilo osigurano pomoću užadi zavezanih za okolne betonske oporce za slučaj preturanja konstrukcije usled dejstva vetra. Kako bi se osiguralo razmicanje oslonaca nosača, u privremenu zategu je, pomoću prese integrisane u sklop zatege, uvedena sila prednaprezanja - inicijalna sila zatezanja. Praktično je nosač blago nategnut zategom kako bi zadržao projektovani položaj i po spuštanju u oporce i otpuštanju prese, drugačije se zatega ne bi mogla ni skinuti, uneo H silu u oporac.



Slika 10 - Zaštitna konstrukcija od LLDa



Slika 11- Zaštitna konstrukcija od LLDa



Slika 12- Zaštitna konstrukcija od LLDa



Slika 13- Zaštitna konstrukcija od LLDa



Predrag Mihajlović¹, Marija Mihajlović²

UTICAJ GRAĐENJA NA ŽIVOTNU SREDINU I DRUŠTVO

Rezime:

Uporedo sa porastom potreba za stambenim prostorom i infrastrukturom raste i uticaj gradnje na životnu sredinu ali i na samo društvo. Građevine troše ogromnu količinu energije, u svim fazama njihovog životnog ciklusa: od izgradnje, preko upotrebe i održavanja i sve do rušenja. U ovom radu biće reči o identifikovanju uticaja pocesa građenja na životnu sredinu i na društvo, kako bi se očuvao skladan odnos između velikih i globalnih izazova, to su uticaji građevinske industrije na ekološku, ekonomsku i društvenu korist.

Ključne reči: građenje, održiva gradnja, životna sredina, društvo

IMPACT OF CONSTRUCTION ON THE ENVIRONMENT AND SOCIETY

Summary:

Along with increasing needs for housing and infrastructure is growing and the impact of construction on the environment but also for society. Buildings consume a tremendous amount of energy at all stages of their life cycle from construction through operation and maintenance and everything collapsed. This paper will discuss the identification of the impact began to build on the environment and on society, in order to preserve the harmonious relationship between large and global challenges to the construction industry impacts on the ecological, economic and social benefits.

Key words: construction, sustainable construction, environment, society

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1 UVOD

Svaka ljudska aktivnost, bez izuzetka, utiče na životnu sredinu i na društveno okruženje. Ista je situacija i sa aktivnostima na izgradnji objekata. Danas, više nego ikada pre, zgradarstvu se posećuje posebna pažnja kada se govori o potrebi da se ljudske aktivnosti dovedu u sklad sa životnom sredinom i društvenom okolinom.

U opštem slučaju, uticaj ljudskih aktivnosti na životnu sredinu svih živih bića i biosferu ogleda se u kontinualnom ugrožavanju donjih slojeva atmosfere, hidrosfere i gornjih slojeva litosfere. To ima direktne implikacije na ugrožavanje života i opstanka čoveka i svih živih bića. Degradiranjem prirodnih ekosistema nastaju svi danas poznati osnovni problemi: zagađenje tla, vode i vazduha. Ovi problem dovode do neslučenih štetnih posledica na klimatske poremećaje. Sa druge strane, rast stanovništva i uvećane potrebe za hranom, vodom, mineralnim i energetskim resursima, postojeće tehnologije, itd., stvaraju eneormni pritisak na ekosistem, koji je, danas je to notorna činjenica doveden do granice izdržljivosti, tako da rečenica iz uvoda dobija svoj puni smisao: "Počinje borba za opstanak."

Lepeza zainteresovanih struka je vrlo široka, od ekonomista, lekara, psihologa i ekologa do građevinskih inženjera, urbanista, arhitekata i ostalih inženjera.

Svi procesi unutar složenog odnosa zgradarstvo-životna sredina se odvijaju na osnovu međusobne zavisnosti, odnosno sadejstva mnogobrojnih faktora, pri čemu se kao rezultat dešavaju i mnogobrojne promene - promene se kreću od sasvim neznatnih do tako drastičnih da - pojedini elementi gube svoja osnovna obeležja.

"Prema majci zemlji i bratu nebu beli čovek se odnosi kao prema stvarima koje se mogu kupiti ili opljačkati i prodati poput stoke ili sjajnog nakita. Njegova pohlepa će uništiti zemlju i za sobom ostaviti samo pustoš... U gradovima belog čoveka nema mirnog kutka, nema mesta na kome bi se čulo otvaranje lišća i treptaj krila mušice... Divljak sam i ne razumem kako gvozdeni konj iz koga suklja dim može biti važniji od živog bizona... Što god zadesi životinje, ubrzo snađe i čoveka... Zemlja ne pripada čoveku, čovek pripada zemlji. Čovek nije tkanica života već samo jedno vlakno u njoj...

Pravom življenju je kraj. Počinje borba za opstanak."

- Reči poglavice indijanskog plemena Sietl, odgovarajući na ponudu o kupovini njihove zemlje predsedniku SAD, Abrahamu Linkolnu (Abraham Lincoln), 1854. G

2 KONCEPT UTICAJA GRADNJE NA ŽIVOTNU SREDINU

Kada se govori o konceptu uticaja gradnje na životnu okolinu i društveno sredinu, obavezno je da se razmotre mogući vidovi uticaja. Obično se razlikuju tri vida uticaja, to su:

1. Uticaji koji se nastaju usled građenja objekta i koji su po prirodi privremenog karaktera – posledica su prisustva ljudi i mašina, kao i primene određene tehnologije i organizacije izvođenja radova; po pravilu, negativne posledice se javljaju kao rezultat iskopa i deponovanja materijala, transporta i ugrađivanja velikih količina materijala, kao i trajnog ili privremenog zauzimanja prostora i svih aktivnosti koje su vezane sa navedenim procedurama.

2. Uticaji koji se javljaju kao posledica postojanja objekta u prostoru i njegove eksploatacije - imaju trajni karakter; posebno interesantni sa stanovišta odnosa infrastruktura-životna sredina; u većini slučajeva, imaju karakter prostornog i vremenskog širenja ili

uvećanja, što upućuje na činjenicu da je potrebno pravovremeno obratiti pažnju na njihovu prirodu.

3. Aakcidenti ili incidenti – obično su malo verovatni događaji nastali nepažnjom ili uticajem “viših sila”; obično mogu imati, i najčešće imaju, značajne posledice po životnu sredinu i društvenu okolinu.

Na osnovu napred sistematizovanih vidova uticaja možemo napraviti razliku između tri kategorija uticaja, sve u nameri da se što detaljnije razmotri uticaj gradnje na životnu sredinu i društvo. Tako razlikujemo:

- Direktni uticaj - potiče od samog infrastrukturnog objekta ili sistema. Manifestuje se zauzimanjem zemljišta, uklanjanjem vegetacije ili ugrožavanje poljoprivrednog zemljišta, itd. (Lakše se uočavaju, procenjuju i kontrolišu nego posredni, pošto je uzročnopsledična veza obično očigledna i jasno uočljiva)

- Sekundarni uticaji – obično su povezani sa projektom, ali mogu prouzrokovati mnogo veće posledice na životnu sredinu nego direktni uticaji. Njihovo uočavanje je teže prilikom početnog sagledavanja projekta i kasnija kvantifikacija; Vremenom mogu ugroziti mnogo šire geografsko područje nego što je to u startu procenjeno i često mogu biti nekontrolisani-erozija, povećana eksploatacija šuma; zona razmatranja mogućih uticaja treba da obuhvati ne samo “uzani” pojas, već i oblasti povezane sa određenim projektomdeponije i pozajmišta, zone za preradu materijala, kamenolome, pristupne puteve i objekte za smeštaj radnikanajčešće zone u kojima se dešavaju posredni uticaji; (najčešće dolazi do izražaja i veza između prirodne i društvene sredine)

- Zbirni uticaji na životnu sredinu - proističu iz različitih događaja (pojedinačan veliki događaj, odnosno veliki projekat, nekoliko povezanih događaja, odnosno više projekata u regionu, katastrofalni iznenadni događaji, npr. veliko klizanje zemljišta ili odron u rečni tok i sl., široko rasprostranjena, spora promena, kao što je loše projektovan drenažni sistem duž puta koji ulazi u vodonosne slojeve; mogu uticati na razvoj dodatnih, višestrukih efekata, koji mogu prouzrokovati ugrožavanje funkcije jednog ili više ekosistema ili strukture ekosistema; (Procena kumulativnih efekata je kompleksan proces koji zahteva veoma dobro poznavanje ekoloških principa i mehanizama odgovora ekosistema).

Ako se kao kriterijum za klasifikovanje uticaja gradnje na životnu sredinu i društveno okruženje uzme podela prema prirodni uticaja, onda razlikujemo:

- Pozitivni i negativni uticaji,
- Verovatni i malo verovatni,
- Lokalni i rasprostranjeni
- Privremeni i trajni,
- Kratkotrajni i dugotrajni.

Sama klasifikacija i podela uticaja prema vrsti efekta na životnu sredinu nije dovoljna. Uticaji se moraju klasifikovati i prema njihovoj ozbiljnosti, odnosno veličini (najozbiljniji i najtrajniji uticaji zahtevaju prioritarno i neposredno obraćanje i oni će biti prvi koje treba ublažiti ili izbeći, ako je to moguće).

2.1 ANALIZA PROCENE GRADNJE NA ŽIVOTNU I DRUŠTVENU SREDINU

Najvažniji koraci analize procene gradnje su preliminarna procena uticaja i analiza varijantnih rešenja.

Preliminarna procena obezbeđuje uvid u moguće uticaje i njihovu veličinu, uz određivanje neophodnosti za detaljnijom analizom i njenim obuhvatom.

Analiza varijantnih rešenja dovodi do dokumentovane odluke o rešenjima i optimalnog projekta, i u ovom koraku se pružaju i jedine realne šanse za izbor varijante sa najmanjim mogućim uticajima na životnu sredinu

Analiza uticaja na životnu sredinu ne obuhvata samo procenu ili predviđanje mogućih uticaja i eventualnih mera za ublažavanje ili otklanjanje negativnih dejstava, kako se to često smatra, radi zadovoljenja forme, samo sa planerske ili projektantske pozicije već podrazumeva da je neophodno pratiti i analizirati sve moguće aspekte uticaja i parametre tokom kasnijih faza životnog ciklusa kako bi se odgovarajućim aktivnostima sprečili neželjeni uticaji ili, u krajnjem slučaju, u pravovremenom trenutku moglo reagovati na ugrožavanje životne sredine

Akcenat je na činjenici da se procene uticaja ne mogu raditi po završetku procesa planiranja i projektovanja kao ocena ispravnosti rešenja sa još jednog aspekta, već moraju biti sastavni deo tih procesa i u pojedinim koracima ići ispred njih. Samo na taj način će svako rešenje u polaznoj, kreativnoj fazi imati već ugrađena saznanja iz oblasti zaštite životne sredine kao ravnopravan element u procesu vrednovanja varijantnih rešenja

2.2 STRATEŠKA STUDIJA UTICAJA NA ŽIVOTNU SREDINU

Najšira prostorna analiza problematike životne sredine koja svoj puni smisao ima u polaznim planerskim i projektantskim koracima, a često i na političkom nivou usmeravanje razvoja zasnovanog na konceptu i principima održivog razvoja, odnosno obezbeđivanju uravnoteženog razvoja i zaštite prirodnih, ekonomskih i društvenih faktora životne sredine, stvarajući tako uslove za inventar ekoloških potencijala, njihovu kategorizaciju i procenu nivoa eventualnih ekoloških rizika. Razlikujemo nekoliko važnih koraka u proceni uticaja gradnje na životnu sredinu i društvo. To su:

- Preliminarna procena uticaja na životnu sredinu - podrazumeva utvrđivanje potreba za izradom procene uticaja na životnu sredinu, kao i obima analize, odnosno aspekata životne sredine koje bi trebalo razmatrati u daljim fazama procene omogućava se pravilno usmeravanje uvek ograničenih sredstava ka odgovarajućim stavkama i odgovarajuća raspodela vremena za izradu daljih faza istraživanja, a ujedno se i dalji izveštaji ograničavaju na razumnu meru utvrđuje se šta je moguće i šta je nemoguće postići tokom procene uticaja na životnu sredinu i dalje analize i resursi se fokusiraju na konkretne aspekte;

- Studija uticaja na životnu sredinu - procena uticaja koja se radi u toku izrade generalnog projekta i predstavlja ključni korak u pozitivnom pristupu ovoj problematici cilj je definisanje mogućih uticaja projekta na okolinu pri čemu se postupak procene svodi na globalnu procenu intenziteta i smera uticaja, i to svakog uticaja posebno u odnosu na svaku pojedinačnu posledicu s obzirom da se radi o prostornom razmeštaju budućih zagađivača od posebnog je značaja da se sistemski definišu svi uticajni kriterijumi i izvrši njihova kvantifikacija kroz odgovarajuće pokazatelje

- Projekat ekološkog uređenja gradilišta – kojim su obuhvaćeni svi radovi koji se izvode na objektu, a opisani su u tehničkim uslovima za izvođenje radova u okviru glavnog projekta projektant daje predlog potrebnih mera zaštite, koje bi obuhvatile organizacione, građevinske, tehničko-tehnološke i druge mere obuhvatiti i rekonstrukciju gradilišta nakon završetka izgradnje

- Ugovorne obaveze vezane za realizaciju zahteva zaštite i unapređenja životne sredine - moraju se ostvariti kroz uključivanje odgovarajuće dokumentacije, kao zakonskog zahteva, u

sva ugovorna dokumenta ili pripremom grupe ekoloških klauzula koje bi bile direktno uključene u ugovornu dokumentaciju praćenje stanja životne sredine omogućava povratnu informaciju u pogledu kvaliteta donete odluke, kao i sticanje iskustava radi realnijeg vrednovanja pojedinih faktora

– Monitoring - podrazumeva uzimanje uzoraka, merenja i analizu rezultata da bi se utvrdile promene u kvalitetu životne sredine koje su nastale kao rezultat projekta. Nadzor omogućava kontrolu aktivnosti i njihovo usmeravanje kako bi se postigli zacrtani ciljevi, kako kvaliteta i količine izvedenih radova, tako i sprečavanja ili ublažavanja uticaja na životnu sredinu u toku izvršenja radova. Revizija se sprovodi sistematski i od strane nezavisnih revizor ili kontrolora, a radi provere da li provedene aktivnosti i njihovi rezultati odgovaraju postavljenim ciljevima.

Kako bi se obezbedila zaštita društva od aktivnosti vezanih za procese gradnje, neophodno je u sva istraživanja uključiti i javno mnjenje učešće javnosti je od vitalnog značaja za uspešan proces planiranja i realizacije infrastrukture javne konsultacije pretpostavljaju dijalog između investitora i grupa ili pojedinaca koje mogu biti "ugrožene" projektom ili nekom aktivnošću, a da bi se prilikom definisanja i razrade projekta obuhvatili stavovi potencijalno ugroženih o mogućem uticaju na njihov život i životnu sredinu.

Neosporno je da informacije u vezi projekata brzo stižu do javnosti, ali su u većini slučajeva opšte i nepouzdana, osim ako nisu došle putem nekog vida zvaničnog obaveštavanja (obaveštenje, javne konsultacije, odgovori na komentare) obaveštavanje nije učešće javnosti, već samo informisanje učešće javnosti je proces dvosmerne komunikacije otvoren proces u kome investitor traži povećan nivo interakcije sa zainteresovanima. Da bi se postigao odgovarajući efekat javnosti, pre samog uključivanja je potrebno proučiti profil, odnosno strukturu zajednica i stanovnika, planirati način informisanja i prikupljanja informacija o nekoliko ciljnih grupa javnosti, kao što su: opšta javnost, političke strukture, mediji, agencije. Svaki od pomenutih stejkoldera ima svoje ciljeve koje tokom gradnje i kasnije tokom životnog veka objekta treba uzeti u ozbiljnu analizu, razmatranje uz puno uvažavanje zahteva javnosti i konkretnih ciljnih grupa.

3 ODRŽIVA GRADNJA

"Kuća je kao biljka koja se gradi i razvija, a ne betonski stub koji se u zemlju zabije."

- (Frank Lloyd Wright)

Pojam održive gradnje pre svega podrazumeva skladan odnos između ekologije i privrede kako bi se prirodno bogatstvo sačuvalo i za buduće naraštaje. Održiva izgradnja se okreće ka dva velika globalna izazova: ekološkom uticaju građevinske industrije, i ekonomskoj i društvenoj koristi.

Održiva gradnja pokušava da reši oba izazova ograničavanjem ekološkog i ljudskog uticaja gradnje i istovremeno garantujući najviši kvalitet estetike, snage i izdržljivosti. Uzima u obzir celokupan životni vek građevine, od izbora materijala do rušenja i reciklaže.

U savremenoj građevinskoj praksi, održiva gradnja predstavlja je usko povezana, i to toliko da se ne može odvojeno posmatrati od održivog razvoja. Pod pojmom održive gradnje najčešće se podrazumeva:

- smanjenje negativnog uticaja gradilišta na životnu sredinu i društvo (buka, prašina, repetitivni zadaci),
 - integraciju obnovljivih izvora energije u fazi izgradnje,
 - upotreba sekundarnih sirovina u procesu gradnje kako bi se očuvali prirodni resursi,
 - poboljšanje toplotne inercije građevina kako bi se smanjili troškovi grejanja i klimatizacije (kao i emisije CO₂),
 - kontrola starenja građevina, održavanje i pravovremena restauracija,
 - recikliranje materijala i građevina posle rušenja,
 - dizajn stambenih objekata kako bi se poboljšali uslovi života ljudi.
- Održiva gradnja se realizuje odgovarajućim merama, najvažnije su:
1. Prioritetno smanjenje potrošnje energije u svim fazama životnog ciklusa građevine.
 2. Efikasno zadovoljiti energetske potrebe, opet u svim fazama životnog ciklusa zgrade.
 3. Aktivno uključiti obnovljive izvore energije i mere unapređenja energetske efikasnosti u zgradarstvu.
- Svaka od navedenih mera realizuje se odgovarajućim pravilima u cilju unapređenja odnosa između građenja i životne sredine i sruštva.
 - Prioritetno smanjenje potrošnje energije u svim fazama životnog ciklusa građevine realizuje se poštovanjem sledećih pravila:
 - Izgradnja zgrade takve optimalne kompaktnosti koja obezbeđuje smanjenje gubitaka preko spoljašnjih površina
 - Efikasna toplotna izolacija u cilju smanjenja potrošnje energije za zagrevanje i hlađenje
 - Poboljšana zaptivenost spoljne površine u cilju smanjenja gubitaka toplote usled strujanja vazduha
 - Standardizovana zaštita od sunca da bi se izbeglo pregrevanje
 - Različite debljine staklenih površina u zavisnosti od položaja zidova u cilju smanjenja gubitaka energije
 - Svetlo obojen krov u cilju sprečavanja pregrevanja odgovarajuća konstrukcija krova radi prilagođavanja meteorološkim prilikama.
- Efikasno zadovoljenje energetske potrebe u svim fazama životnog ciklusa zgrade podrazumeva sledeće aktivnosti kako bi se obezbedili uslovi vezani za:
- Efikasnu i kontrolisanu ventilaciju za unutrašnju cirkulaciju vazduha
 - Napredni ventilacioni sistemi za regulisanje toplote
 - Konstrukcija koje je projektovana tako da zadovoljava potrebe zagrevanja i hlađenja i ekonomičnosti u pogledu obnovljive energije kao savremenog zahteva u zgradarstvu
- Aktivno uključivanje obnovljivih izvora energije i mera unapređenja energetske efikasnosti u zgradarstvu podrazumeva:
- Položaj zgrade treba da je takav da se najbolje iskoristi energija Sunca
 - Optimizacija neprozirnih površina radi najvećeg stepena iskorišćenja sunčeve energije
 - Sistemi za iskorišćenje sunčeve energije kroz ne-staklene površine
 - Izložene betonske površine radi iskorišćenja sunčeve energije ili hladnog vazduha tokom noći
 - Reflektovanje svetlosti radi iskorišćenja dnevnog osvetljenja
 - Optimalni prozori radi maksimalnog iskorišćenja dnevne svetlosti

- Makimalno korišćenje lokalno dostupnih obnovljivih izvora energije u cilju zamene fosilnih goriva
- Efikasna akustična izolacija u cilju većeg komfora stanara
- Ekonomično korišćenje vode radi očuvanja prirodnih resursa
- Materijali i konstrukcije odabrani u cilju smanjenja uticaja na životnu sredinu u celokupnom procesu gradnje
- Krovovi zasađeni zelenilom da bi se poboljšala integracija u okruženje (i iskorišćenost kišnice) radi pozitivnog uticaja na društveno okruženje i konkretno na stanare zgrada.

U današnje vreme se susrećemo sa brojnim ekološkim problemima, od zagađenja vode, vazduha, zemlje, smanjenjem šuma, gubitkom životne sredine za ugrožene vrste, izumiranjem flore i faune, klimatskim promenama i prebrzim širenjem gradova.

Postoje dve činjenice koje je potrebno navesti, prva jeste da u prirodi ne postoji otpad, već je sve korisno i ima svrhu za nešto drugo u ekosistemu. Proizvodnja otpada koji ne može da se reciklira je isključivo ljudska aktivnost i ti materijali su većinom napravljeni u laboratoriji, bez prirodnih neprijatelja koji bi ih razgradili. Ukoliko pogledamo svoju urbanu životnu sredinu, verovatno uočićemo mnoštvo stvari koje su napravljene od plastike, zbog njene male cene, vodonepropusnih svojstva i fleksibilnosti da se formira u bilo kojem obliku, veličini i boji. Ovo znači da mi koristimo ogromne količine nafte za proizvodnju plastike, koja je zaista jako praktičan materijal, ne misleći pri tom konačnosti ovog resursa sa kojom ćemo se neminovno suočiti u skorjoj budućnosti.

U održivoj gradnji, treba u što većoj meri primenjivati novu proizvođačku logiku „from cradle to cradle“, gde je otpad iz jednog procesa repromaterijal za drugi proces, umesto prevaziđene stare logike „from cradle to grave“, koja je dovela do nepovratnog narušavanja ravnoteže globalnog ekosistema. Drugo, održivi gradovi treba da budu dizajnirani sa jednakim poštovanjem za sve ljude koji će ih naseljavati, treba da podržavaju održivu ekonomiju, ekološku ravnotežu i visok nivo kulture stanovanja. Ako je zgrada izgrađena po principima održivosti, ona će trajati duže, obezbeđujući bolje uslove za život i rad, što će se prirodno odraziti na veću motivisanost i efikasnost ljudi. Održiva zgrada teži da izbegne „sindrom bolesne zgrade“ gde su ljudi u većini slučajeva više bolesni od onih koji žive u održivim objektima.

Održivi gradovi treba da promovišu koncept zdrave urbane ekologije, ekološke filantropije, društvene odgo - vnosti i održivog razvoja. Isto tako, prilikom implementacije inovativnih tehnoloških rešenja neophodno je voditi računa o celokupnom životnom ciklusu proizvoda i tehnologija kako bi uvede - na rešenja mogla biti dugoročno isplativa i ekološki prihvatljiva.

4 ZAKLJUČAK

Sa rastom globalizacije i industrijalizacije porasli su i zahtevi za čistom okolinom i zdravom životnom sredinom koje je moguće ispuniti unapređenjem tehnologija gradnje pretvarajući ih u održive i ekološki prihvatljive. Svaka građevinska konstrukcija je izgrađena za neku određenu namenu i pri tome je izložena raznim opterećenjima ili kombinacijama opterećenja. Potrebno je poznavati uticaj građevine u svim fazama životnog ciklusa.

Održivost je način življenja, pristup ukupnoj privrednoj delatnosti u skladu sa eko sistemom, za šta je potrebno imati viziju razvoja. Osnovni princip održivog razvoja u graditeljstvu je korišćenje što manje prirodnih resursa i stvaranje što manje, po životnu sredinu i društveno okruženje štetnog otpada. Zaštita okoline i ušteda energije postaju svetski problemi

u svim poljima tehnologije. Pri teorijskom razmatranju održivosti postoje četiri aspekta: ekološki, tehnološki, ekonomski, sociološki, a svaki od navedenih aspekata sadrži još niz podgrupa ili skupova.

Pronalaženje rešenja za smanjenje negativnog uticaja koji ljudske delatnosti, među njima i graditeljstvo, imaju na okolinu, zadatak je svih grana industrije. Građevinarstvu kao jednom od najvećih potrošača prirodnih resursa i zagađivača okoline to treba da bude primarni zadatak. U ovom radu predstavljene su glavne postavke za stvaranje održive industrije gradnje: očuvanje prirodnih resursa, smanjenje emisije CO₂ i gradnja trajnijih konstrukcija.

Jedna od osnovnih postavki je promena u načinu razmišljanja i delovanja ljudi. Jedino u slučaju sagledavanja problematike zagađenja okoline iz svih aspekata (ekoloških, ekonomskih, tehnoloških i socioloških), moguće je stvoriti tehnologiju gradnje koja je ekološki održiva.

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PRIMJENA PREDNAPREZANJA PRI REKONSTRUKCIJI I SANACIJI BETONSKIH MOSTOVA

Sažetak:

U novije vrijeme su sve češće potrebe za rekonstrukciju ili sanaciju odnosno ojačanjem betonskih mostova. Rezlozi mogu biti različiti, bilo da je ugrožena nosivost oštećenjima ili je potrebno povećati nosivost ili i jedno i drugo. Ojačanje se može postići na razne načine. Jedan od mogućih načina ojačanja je prednapreznjem. Ovaj način je pogodan za jače oštećene mostovske konstrukcije i konstrukcije gdje se zahtijeva značajnije povećanje nosivosti. Ojačanje na ovaj način je posebno pogodno na gredne sisteme. U ovom radu opisan je način sanacije prethodnim napreznjem. Prikazane su linije kablova za karakteristične gredne sisteme kao i detalji ankerovanja kablova i vođenja kablova u polju preko devijatora i srednjih oslonaca.

Ključne riječi: betonski mostovi, sanacija, rekonstrukcija, prednapreznje

APPLICATION OF PRESTRESSING DURING RECONSTRUCTION AND REPAIR OF CONCRETE BRIDGES

Summary:

Recently there is more necessity for reconstruction and repair that is for strengthening of concrete bridges. The reasons can be different, either the bearing capacity is at risk because of great damage or it is necessary to increase bearing capacity or the both. Strengthening can be made in various ways. One of possible ways of strengthening is prestressing. This way is convenient for bigger damages of the bridge structure and for the structures where it needs importantly improvement of the bearing capacity. Strengthening in this way is specially convenient for beam systems. This paper describes the way of repair by prestressing. It is presented the lines of cables for characteristic beam systems as well as details of anchoring of cables and taking cables in the field by deviators and middle supporters.

Key words: concrete bridges, repair, reconstruction, prestressing

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1 UVOD

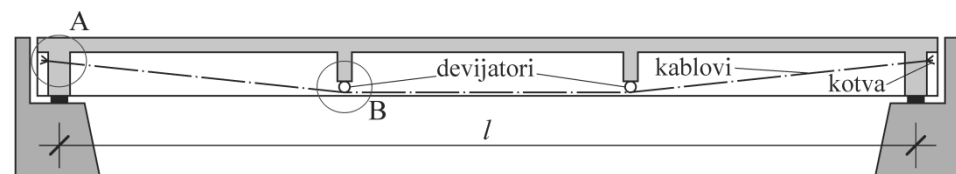
Potrebe za rekonstrukcijom i/ili sanacijom armirano-betonskih mostova mogu biti različite. U oba slučaja je potrebno ojačati konstrukciju mosta, bilo da je ugrožena njena nosivost oštećenjima, bilo da je potrebno povećati nosivost ili i jedno i drugo zajedno. Ojačanje konstruktivnih elemenata može se postići na razne načine. Najčešće korišćeni načini su povećanjem dimenzija betonskog elementa uz dodatnu armaturu, zatim korišćenjem čeličnih i karbonskih traka kojima se nadomješćuje nedostatak armature. Vrlo efikasan način rekonstrukcije i sanacije može se postići prednaprezanjem. Koji će se od ovih načina primijeniti zavisi od karaktera i stepena oštećenja odnosno potrebnog stepena ojačanja konstrukcije.

Ojačanje konstrukcije mosta povećanjem poprečnog presjeka je vrlo efikasan i jeftin način. Najčešće primjenjivan metod je povećanje debljine kolovozne ploče. Ovim načinom se povećava statička visina ploče, uzdužnih i poprečnih nosača u polju, a samim tim i nosivost. Na osloncima u dodatnom betonu može se dodati nedostajuća armatura. Na ovaj način se formira spregnuti poprečni presjek. Veza osnovnog i dodatnog betona se postiže odgovarajućim premazima i ankerima. Nedostatak ovog načina ojačanja je povećanje sopstvene težine kao i činjenica da se dodatni beton aktivira za dodatne stalne uticaje i pokretno opterećenje. Stepenn ojačanja konstrukcija na ovaj način je ograničen.

Ojačanje konstrukcije mosta čeličnim limovima i karbonskim trakama se primjenjuje kada imamo nedostatak armature, u novije vrijeme se uglavnom koriste karbonske trake. I u ovom slučaju stepen ojačanja je ograničen.

Značajnije povećanje nosivosti konstrukcije mosta se može postići prednaprezanjem. Ovaj način ojačanja je pogodan za jače oštećenje mostovske konstrukcije i mostovske konstrukcije gdje se zahtijeva značajnije povećanje nosivosti. Ovaj način je vrlo pogodan za gredne sisteme.

Za konstrukciju sistema proste grede uobičajeni položaj kablova je prikazan na slici 1. Kablovi se kotve u kranje poprečne nosače u gornjoj nozi, dok se u polju kablovi vode kroz i preko poprečnih nosača. U zavisnosti od broja i rasporeda poprečnih nosača. Kablovi se postavljaju simetrično u odnosu na glavne nosače i oni su poligonalnog oblika.



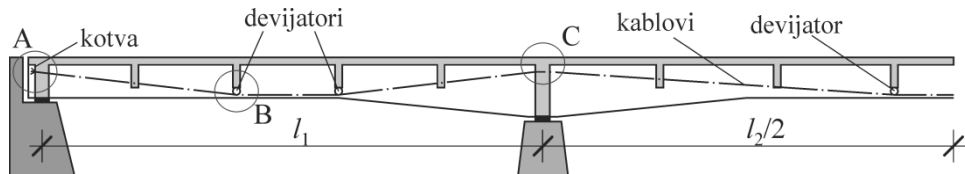
Slika 1. Uobičajeni položaj kablova za prostu gredu

Kod kontinualnih sistema linija kablova je takođe poligonalna i ona se oblikuje tako da izazove suprotne uticaje od eksploatacionih opterećenja. Na osloncima je linija kablova u gornjoj zoni, a u poljima u donjoj zoni konstrukcije. I u ovom slučaju kablovi se postavljaju simetrično u odnosu na glavne nosače. Tipičan oblik trase kablova kontinualnog nosača na tri polja prikazan je na slici 2.

Na krajnjim osloncima kablovi se sidre u poprečne nosače na tom mjestu. Pošto su ovi preprečni nosači uglavnom konstruktivno armirani neophodno je njihovo ojačanje podebljanjem i dodatnim armiranjem posebno u zoni kotvljenja kabla i veze sa podužnim nosačima i kolovoznom pločom. Na tim mjestima se u postojećem poprečnom nosaču izbuši

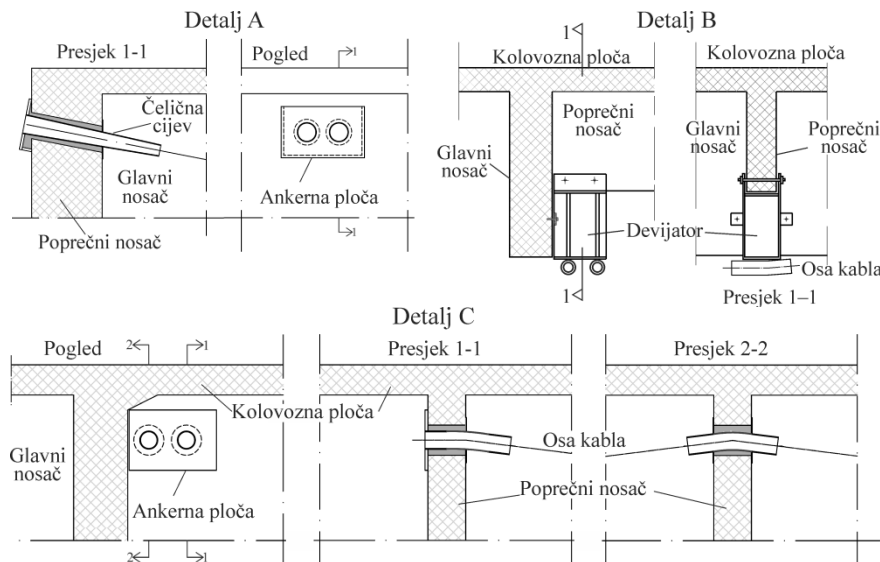
odgovarajuća rupa kroz koju se provlači kabal. U ovu rupu se prethodno ugrađuje čelična cijev sa obodnom čeličnom pločom na mjestu kotvljenja kabla.

Na srednjim osloncima kablovi se provlače kroz prethodno izbušene otvore u poprečnim nosačima u njihovoj gornjoj zoni čime se ostvaruje njihov kontinuitet ili se dio kablova na tom mjestu kotvi u zavisnosti od konstruktivnih potreba.



Slika 2. Uobičajeni položaj kablova za gredu na tri polja

U slučaju kotvljenja i ove nosače treba računski provjeriti da li ih je potrebno ojačavati. Na središnjim osloncima kablovi se provlače kroz prethodno ugrađene čelične cijevi koje su oblikovane prema liniji kablova u zavisnosti da li kabal na tom mjestu ima kontinuitet ili se kotvi. U slučaju većeg broja kablova za svaki kabal se radi poseban otvor. Ukoliko su ostali poprečni nosači na liniji kablova kroz njih se rade otvori za provlačenje kablova. Prečnik cijevi za provlačenje i kotvljenje kablova zavisi od prečnika kablova. Otvori u poprečnim nosačima se rade većeg prečnika od prečnika cijevi da bi se cijevi mogle normalno ugraditi. Zazor između betona i cijevi zalijeva se visokovrijednim malterima ili epoksidnim smolama, da bi se postigao odgovarajući kontakt između cijevi i betona, tj. da bi se izbjegla koncentracija naprezanja i lokalno mrvljenje betona. Kablovi se postavljaju u odgovarajuće cijevi i antikorozijski se štite prema upustvima proizvođača. Detalji kotvljenja, provlačenja kablova preko srednjih oslonaca i devijatora dati su na sl. 3.



Slika 3. Detalji fiksiranja kablova

Rekonstrukciju i sanaciju treba da prati detaljan statički proračun koji treba da obuhvati sve faze i stanja konstrukcije.

U ovom radu prikazan je primjer sanacije znatno oštećenog Nika Strugara preko rijeke Lim, koji se nalazi u Beranama, Crna Gora.

2 MOST NIKA STRUGARA U BERANAMA

2.1 OPŠTE

Most premošćuje rijeku Lim i spaja centar grada sa naseljima Hareme i Aluge, tj. radi se o gradskom mostu. Izgrađen je od armiranog betona i stavljen u upotrebu 1965. godine. Za ovaj most nije sačuvana projektna i izvođačka dokumentacija.

Krajem 2010. godine usled velikih padavina i poplava došlo je do podlokavanja prvog rječnog stuba S3 iz pravca naselja Hareme i Aluge, a samim tim i do njegovog ravnomyjernog slijeganja za 64 cm. Ovo je izazvalo ozbiljna oštećenja rasponske konstrukcije, zbog čega se moralo pristupiti sanaciji.

2.2 OSNOVNI PODACI O MOSTU

Most je armiranobetonski, sistema kontinualne grede sa četiri raspona. Krajnji rasponi su po 24,60 m, a dva srednja po 27,60 m. Ukupna dužina mosta je 104,40 m. Širina mosta je 10,46 m, i on ima dvije saobraćajne trake širine po 3,57 m i dvije pješačke staze čija je širina zajedno sa ogradom 1,66 m.

Glavni nosivi konstruktivni sistem mosta čine po dva blizna gredna nosača širine 36 cm postavljenih na osovinskom odstojanju od 114 cm. Nosači su promjenljive visine. U polju imaju visinu (uključujući i ploču) 160 cm, a na osloncima 220 cm. Povećanje visine uz srednje oslonce mijenja se pravolinijskim vutama u dužini od 590 cm, dok je između vuta u polju konstantna visina.

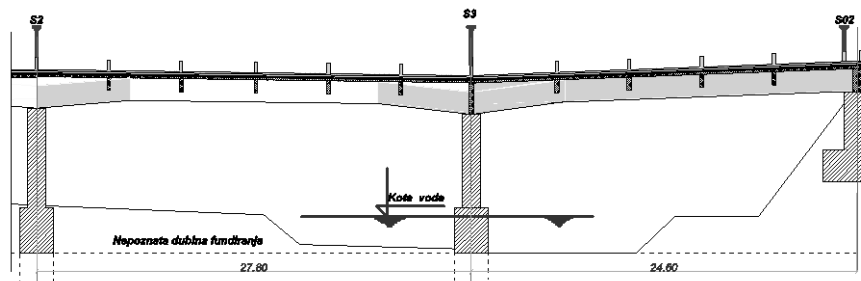
Parovi bliznih nosača su postavljeni na osovinskom odstojanju od 558 cm. Na dužini vute nosači su povezani donjom flanšom deljine 15 cm. Most ima poprečne nosače na osloncima i u polju. Dimenzije poprečnih nosača na krajnjim osloncima su 50x160 cm, a na srednjim 25x220 cm. Poprečni nosači u polju su debljine 25, a visine 85 cm.

Kolovozna ploča je debljine 20 cm ojačana vutama širine 20 cm i debljine 20 cm uz glavne i poprečne nosače.

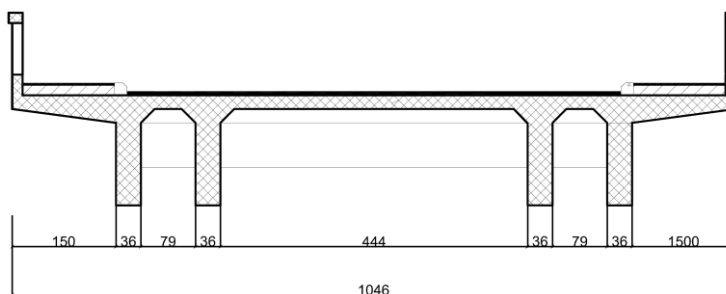
Rječni obalni stubovi su masivne betonske konstrukcije. Na krajnjem lijevom obalnom oporcju je nepokretni oslonac, dok su na ostalim stubovima urađeni pokretni oslonci sistema betonskih klatana.

Pošto za ovaj most nije postojala projektna dokumentacija izvršeno je snimanje geometrije mosta. Utvrđen je kvalitet i količina armature. (GA 240/360) Vađenjem kernova je utvrđen kvalitet betona MB40.

Na slikama 4 i 5 prikazani su uzdužni i poprečni presjek mosta.



Slika 4. Uzdužni presjek mosta



Slika 5. Poprečni presjek mosta

2.3 OŠTEĆENJA MOSTA

Usled podlokavanja i slijeganja rječnog oporca S3 za 64 cm došlo je do ozbiljnih oštećenja konstrukcije mosta. U prvom polju na osovinskom odstojanju 3,3 m od potonulog stuba glavna rasponska konstrukcija praktično je pretrpjela lom. U glavnim nosačima su se otvorile u donjoj zoni pukotine širine oko 1,5 - 2 cm, dok je beton u gornjoj zoni na vijencima pješačkih staza i ograde mosta zdrobljen. Na ovom presjeku i u kolovoznoj ploči je došlo do drobljenja betona. Pri ovolikim ugibima došlo je i do tečenja armature i formiranja plastičnog zgloba.

U drugom polju uz drugi rječni S2 oslonac na osovinskom odstojanju od 2,35 m, od oslonca takođe je došlo do oštećenja. Na ovom mjestu su se pojavile pukotine u gornjoj zoni, dok u donjoj zoni u glavnim nosačima nije primijećeno drobljenje betona.

Takođe je došlo do otvaranja spojnice na krajnjem oporcju prema Alugama usled povlačenja konstrukcije zbog slijeganja stuba.

2.4 PRORAČUN KONSTRUKCIJE

Proračun rasponske konstrukcije izvršen je prema važećim propisima. Usvojeno je vozilo V600. Svi konstruktivni elementi kontrolisani su za odgovarajuće kombinacije opterećenja.

Konstrukcija mosta je prvo kontrolisana za projektovano stanje prema važećim propisima. Ova kontrola je pokazala da pojedini konstruktivni elementi ne zadovoljavaju u pogledu nosivosti. Zatim je urađen proračun konstrukcije od uticaja slijeganja potonulog rječnog stuba za utvrđeno slijeganje od 64 cm. Dobijeni rezultati su pokazali da je u ovom slučaju u dva presjeka došlo do loma konstrukcije jer dobijeni uticaji višestruko premašuju granične uticaje, što se i desilo.

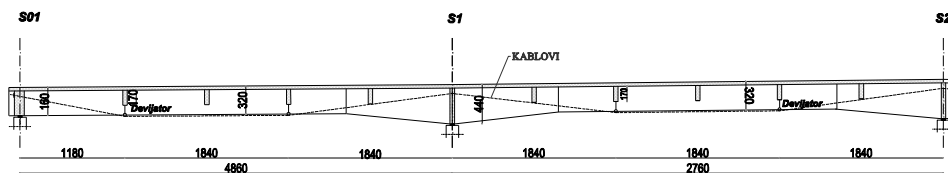
U sljedećoj fazi konstrukcija je kontrolisana u postojećem polomljenom deformisanom stanju. U presjecima gdje je došlo do loma u konstrukciji ubačeni su zglobovi tako da je formiran novi konstruktivni sistem sa zglobovima. Ni u ovom stanju konstrukcija nije zadovoljila nosivost zbog čega se pristupilo prethodnom naprezanju. Za izabrane kablove i sile prethodnog naprezanja konstrukcija mosta je u potpunosti zadovoljila u pogledu nosivosti i sigurnosti. Dobijeni koeficijenti sigurnosti su veći od zahtijevanih za sve kombinacije opterećenja i sve presjeke.

2.5 SANACIONO RJEŠENJE

Rječni potonuli stub. Riječni potonuli stub je saniran podbetoniranjem i proširivanjem temelja. Prije pristupanja sanaciji ovog stuba vodotok je skrenut iz ove zone.

Rasponska konstrukcija. Imajući u vidu oštećenja koja je pretrpjela rasponska konstrukcija opredijelili smo se za sanaciono rješenje prethodnim naprezanjem kablovima. Ovaj način sanacije, u ovom slučaju, ima višestruke prednosti. Rasponska konstrukcija mosta i prije oštećenja nije posjedovala odgovarajuću sigurnost prema važećim propisima za opterećenje mostova, jer je računata prema starim propisima. Pored toga, formiranjem dva plastična zgloba u rasponskoj konstrukciji mosta, nakon potonoća rječnog stuba, došlo je do preraspodjele uticaja u rasponskoj konstrukciji, koji prekoračuju graničnu nosivost u pojedinim presjecima.

Imajući u vidu prednje izabran je takav raspored kablova koji povoljno utiče na ukupne uticaje, uravnotežuje sistem mosta i obezbjeđuje njegovu odgovarajuću statičku sigurnost. Za prethodno naprezanje usvojenaj su dva kabla $15 \times 150 \text{ mm}^2$ koji su formirani od užadi od čelika, kvaliteta 1670/1860 MPa. Kablovi se postavljaju uz unutrašnju stranu glavnog nosača stim što se na osloncima postavljaju u gornjoj, a u poljima u donjoj zoni. Trasa kablova je pravolinijska. Po jedan kabl se vuče uz glavne nosače duž cijelog mosta, a u dva oštećena polja dodaje se još po jedan kabl. Plan kablova prikazan je na slici 6.



Slika 6. Plan kablova

Prethodno naprezanje se vrši sa oba kraja. Na mjestima gdje se lomi trasa kabla postavljaju se čelične cijevi, kao devijatori, za provlačenje kablova. Kablovi se ugrađuju u cijevi i injektiraju cementnom emulzijom. Na krajnim osloncima se rade ankerni blokovi na koje se ugrađuju kotve.

Rasponska konstrukcija se na mjestim gdje su formirani plastični zglobovi ojačava karbonskim platnima kojima se obuhvataju glavni nosači i kolovozna ploča. Karbonske trake su proračunate tako da prime smičuću silu na tom mjestu. Da bi se formirala odgovarajuća niveleta kolovoza na mjestu ulegnuća, projektovana je posebna konstrukcija u vidu AB roštilja sa ispunom od tvrdog stiropora (slika 3). Na ovom mjestu je potrebno prethodno skinuti asfaltni zastor i hidroizolaciju i dobro očistiti betonsku površinu, pjeskarenjem radi boljeg povezivanja starog i novog betona.

Na slici 7 je prikazan most nakon oštećenja, a na slici 8 nakon sanacije. Slika 9 prikazuje konstrukciju mosta sa donje strane sa kablovima. Na slici 10 prikazan je devijator formiran na mjestu spoja glavnog i poprečnog nosača. Kotvljenja kablova na krajnjem osloncu prikazna je na slici 11, dok je na slici 12 prikazano kotvljenje kablova na srednjem osloncu i prevođenje kablova preko srednjeg oslonca.

Lokalna oštećenja konstrukcije mosta su sanirana na uobičajen način. Ograda mosta je zamijenjena sasvim novom.



Slika 7. Izgled oštećenog mosta



Slika 8. Izgled rekonstruisanog mosta



Slika 9. Pogled na most odozdo



Slika 10. Devijator



Slika 11. Kotva na krajnjem osloncu



Slika 12. Kotvljenje i prevođenje kablova na srednjem osloncu

3 ZAKLJUČAK

Rekonstrukcija i/ili sanacija betonskih mostova se vrlo efikasno može izvesti prednaprežanjem. Prednaprežanjem se može znatno povećati nosivost mostovske konstrukcije, a i trajnost jer se odgovarajućim prednaprežanjem pukotine mogu ograničiti ili sasvim izbjeći. Ovaj način sanacije je vrlo pogodan za gredne sisteme mostova i mostove koji su pretrpjeli određena oštećenja. Primjer uspješne sanacije znatno oštećenog mosta čiji su glavni nosači pretrpjeli klasičan lom je most Nika Strugara u Beranama čija je sanacija prezentirana u ovom radu. Za sanaciju je primijenjeno rješenje sa prednapregnutim kablovima kao optimum u pogledu sigurnosti i cijene koštanja. Most je ujedno rekonstruisan. Urađena je nova ograda, pješačke staze, osvjetljenje i anitkorozivna zaštita betonskih površina.

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SANACIJA KULTURNO – HISTORIJSKOG SPOMENIKA SAHAT-KULE U DONJEM VAKUFU

Rezime:

S-69

Sahat-kula u Donjem Vakufu (Bosna i Hercegovina) izgrađena je u XVIII vijeku. Od 2008. godine sahat-kula je proglašena nacionalnim spomenikom kulturno-historijskog nasljeđa Bosne i Hercegovine. Zbog dotrajalosti i neadekvatnih opravki, pristupilo se izradi projekta sanacije ovog nacionalnog spomenika. U okviru ovog rada data je ocjena stanja postojeće konstrukcije i predložene su mjere sanacije.

Кljučne reči: sahat-kula, kulturno-historijsko nasljeđe, dijagnostika stanja

REHABILITATION OF THE CULTURAL - HISTORICAL MONUMENT CLOCK TOWER IN DONJI VAKUF

Summary:

Clock Tower in Donji Vakuf (Bosnia and Herzegovina) was built in XVIII century. Since 2008, the clock tower was declared a national monument of cultural and historical heritage of Bosnia and Herzegovina. Due to deterioration and inadequate repair, the project of rehabilitation of the national monument was prepared. Within this paper assessment of the existing structure and proposed rehabilitation measures was presented.

Key words: clock tower, cultural and historical heritage, diagnostics

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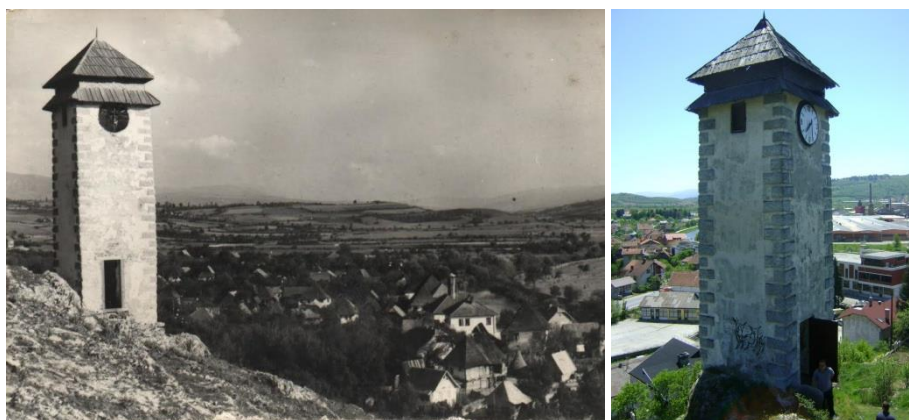
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1 UVOD

Nacionalni spomenik - Sahat-kula u Donjem Vakufu nalazi se u općini Donji Vakuf, Bosna i Hercegovina. Donji Vakuf je gradić u središnjoj Bosni smješten u Uskopljanskoj dolini. Nalazi se oko 35 km južnije od Jajca.

Sahat-kula u Donjem Vakufu je, sudeći prema načinu zidanja i vanjskom izgledu, sagrađena u XVIII vijeku. Nalazi se na padinama brda u središtu grada (slika 1), na lokalitetu poznatom kao Bagina strana. U neposrednoj blizini objekta prolazi lokalni put koji vodi prema naselju Čolaci.



Slika 1 - Sahat kula u Donjem Vakufu nekad i sad

Sahat-kule su visoke i vitke građevine sa satom, kvadratne, rjeđe osmougone osnove. Građene su najčešće u središnjim zonama naselja i to uz džamije. Pored toga nekada se sahat-kule izgrađuju i na drugim mjestima, poput sahat-kule u Maglaju koja je podignuta u oboru maglajske utvrde, ili onih u Tešnju, Gradačcu ili Počiteiju, odnosno u Donjem Vakufu. Javljaju se u našim krajevima tek krajem XVI vijeka u Banja Luci, a nešto kasnije, tokom XVII i XVIII vijeka, i u mnogim drugim naseljima.

Sačuvan je podatak da sahat-kula u Donjem Vakufu nije radila od 1917. godine. Jedno duže vrijeme stajala je i bez krova, te bila izložena nepovoljnim atmosferskim uticajima, ali je uz angažman Zavoda za zaštitu spomenika obnovljena 1952. godine. Ovom prilikom nije vraćen sat, niti zvono koje se čuva u srednjovjekovnoj zbirci Zemaljskog muzeja u Sarajevu. Novi satni mehanizam sa digitalnom baznom stanicom i satelitskim signalom tačnog vremena je postavljen u martu 2004. godine.

Od 2008. godine Sahat-kula je proglašena nacionalnim spomenikom Bosne i Hercegovine. Na nacionalni spomenik primjenjuju se posebne mjere zaštite. U cilju trajne zaštite nacionalnog spomenika dopušteni su isključivo konzervatorsko - resturatorski radovi, radovi na tekućem održavanju i radovi koji imaju za cilj prezentaciju spomenika.

U ovom radu dati će se detaljan opis postojećeg stanja sahat-kule sa prikazom svih oštećenja, kao i prijedlog mjera sanacije. U okviru predloženih mjera sanacije, predviđa se upotreba izvornih i tradicionalno primjenjivanih materijala, u skladu sa odobrenjem nadležnih službi za očuvanje i zaštitu kulturno-historijskog nasljeđa.

2 DIJAGNOSTIKA STANJA SAHAT-KULE

Pravilna i tačna ocjena stanja ključna je osnova za uspješan popravak konstrukcije koja je izložena degradacijskim procesima. U okviru ocjene stanja postojeće konstrukcije potrebno je poštovati opću metodologiju koja uključuje:

- Prikupljanje postojeće dokumentacije,
- Pregled ispitivanja i proračuni,
- Odluka o daljnjem postupanju [1].

Sahat-kula je u prošlosti rekonstruisana u nekoliko navrata. 1952. godine je izvršena rekonstrukcija krova Sahat-kule. Nakon perioda 1992.-95. godine rađeni su radovi na fasadi, krovnoj konstrukciji i drvenom stubištu.

Za građevine čije je stanje potrebno utvrditi u nekom trenutku njihovog postojanja, najjednostavniji način prikupljanja podataka o konstrukciji jeste proučavanje dokumentacije o projektovanju i građenju, te izvještaja o prethodnim pregledima i održavanju. No, podaci o izvornom projektu, kao i kod brojnih drugih objekata kulturno-historijskog nasljeđa u Bosni i Hercegovini, ne postoje. Stoga se pristupilo izradi nacрта na osnovu izmjere presjeka i geometrije konstrukcije, i proučavanju parametara koji opisuju svojstva materijala od kojih su izgrađeni pojedini konstrukcijski elementi.

Sljedeći korak pri ocjeni stanja konstrukcije je vizuelni pregled konstrukcije, kojim se utvrđuju oštećenja na pojedinim konstrukcijskim elementima, njihovi uzroci, raširenost, te uticaj na nosivost i upotrebljivost tog elementa i konstrukcije u cjelini.

Tokom vizuelnog pregleda posebna pažnja se treba posvetiti:

- Geometriji i izmjeri presjeka,
- Izgledu i razlici u boji površine konstrukcije,
- Pojavi pukotina, njihovoj veličini i rasporedu,
- Vlažne površine, odnosno mjesta procurivanja vode,
- Deformacije konstrukcije i sl. [1]

U okviru pregleda konstrukcije označena su kritična mjesta na konstrukciji, sa kojih su uzeti uzorci za laboratorijska ispitivanja osnovnog konstruktivnog materijala objekta - kamena.

U okviru ovih ispitivanja određena su osnovna fizičko – mehanička svojstva postojećeg kamena. Sljedeći korak u dijagnosticiranju i ocjeni stanja konstrukcije je izrada proračunskog modela konstrukcije sa realnim parametrima dobivenim laboratorijskim ispitivanjima u svrhu provjere stabilnosti konstrukcije.

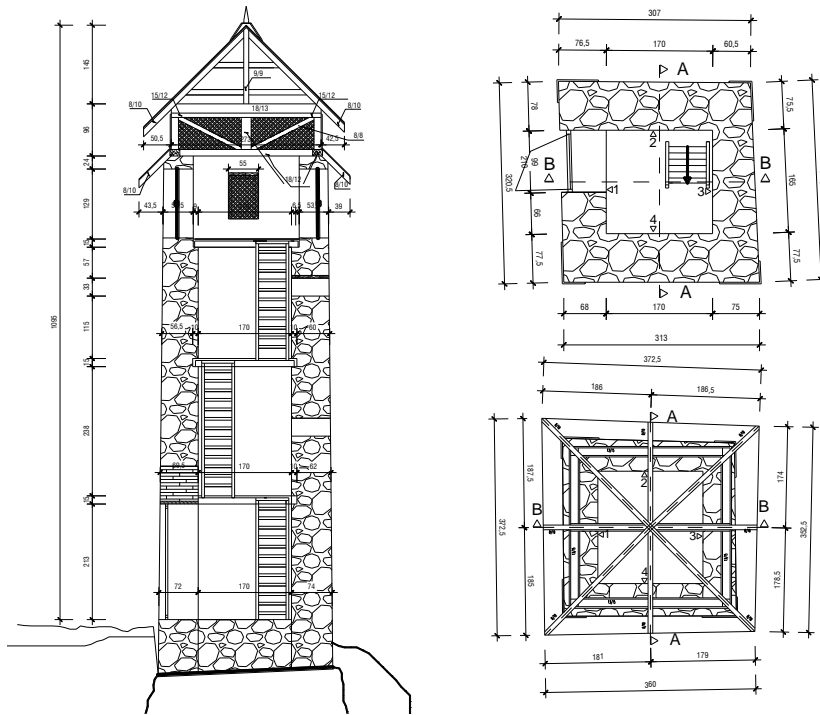
Kao završni korak data je ocjena postojeće konstrukcije i predočene su mjere sanacije i rekonstrukcije postojećeg kulturno-historijskog objekta.

2.1 VIZUELNI PREGLED KONSTRUKCIJE

2.1.1 Geometrija i izmjere presjeka konstrukcije

Sahat-kula u Donjem Vakufu je jednodimenzionalni objekat spratnosti P+3 kvadratne osnove dimenzija cca 3,20x3,20 m. Visina objekta iznosi oko 10 metara i ima oblik tornja koji se postepeno sužava prema vrhu (slika 2). Nosivu konstrukciju objekta čine vanjski zidovi debljine cca 70 cm, zidani od poluobrađenih krečnjačkih kamenih blokova sa ispunom od lomljenog kamena krečnjaka u krečnom malteru. Debljina zida u prizemlju iznosi cca 0,70 m i postupno se smanjuje prema vrhu objekta, tako da na posljednjoj etaži debljina zidova iznosi cca 0,60 m. Struktura zida i vrsta veziva je vidljiva u unutrašnjosti objekta, koja, kao i kod

većine objekata ovog tipa, nije omalterisana. Uglovi su naglašeni pravilno odrezanim naizmjenično složenim kamenim blokovima sličnih dimenzija sa vrlo debelim spojnicama (čošnjaci).



Slika 2 – Presjek i karakteristične osnove sahat-kule

Do posljednje etaže objekta, na kojem se nalazi satni mehanizam pristupa se pomoću strmog, drvenog stubišta koje je postavljeno uz zidove sahat-kule. Stubište ima tri podesta ($h_1=2,30$ m; $h_2=4,80$ m; $h_3=7,00$ m), izrađenih od drvenih greda dimenzija 10/10 cm sa daščanim pokovom $d=5,0$ cm.

Svjetlost u unutrašnjost objekta ulazi kroz posebne prozorske otvore. Na sjevernoj fasadi kule se nalazi jedan prozorski otvor pravugaonog oblika, dimenzija $0,52 \times 0,82$ m. Prozor se nalazi na posljednjoj etaži. Na istočnoj fasadi se nalaze dva prozorska otvora, dimenzija $0,45 \times 0,45$ m u unutrašnjosti objekta, dok je na fasadi širina ovih otvoraje manja i iznosi približno $0,10$ m. Smješteni su jedan ispod drugog, a po svom izgledu podsjećaju na puškarnice.

Ulaz u objekat se nalazi sa zapadne strane i ostvaren je kroz željezna vrata dimenzija $0,98 \times 2,13$ m. Ova vrata svakako nisu izvorna. Podataka o izgledu i materijalizaciji izvornih vrata nema, ali su najvjerovatnije bila drvena i ne na vanjskoj strani zida zbog izloženosti atmosferijama. Vrata imaju kameni prag koji je od razine terena uzdignut za 50 cm. Najvjerovatnije zbog uticaja erozije teren je ispred vrata skinut.

Na sahat-kuli nalaze se tri okrugla sata, orijentirana na tri strane prema gradu: istok, zapad i jug.

Krovnna konstrukcija je drvena četverovodna sa nagibom 45°. Krov sahat-kule je pokriven je šindrom. Krov je bio uništen u razdoblju 1992.-1995. godine i nova krovnna konstrukcija je postavljena 2000. godine. Prostor ispod četverovodnog krova zatvoren je drvenim mušepcima visine 0,85 m. Ispod ovog dijela je izvedena streha također pokrivena šindrom.

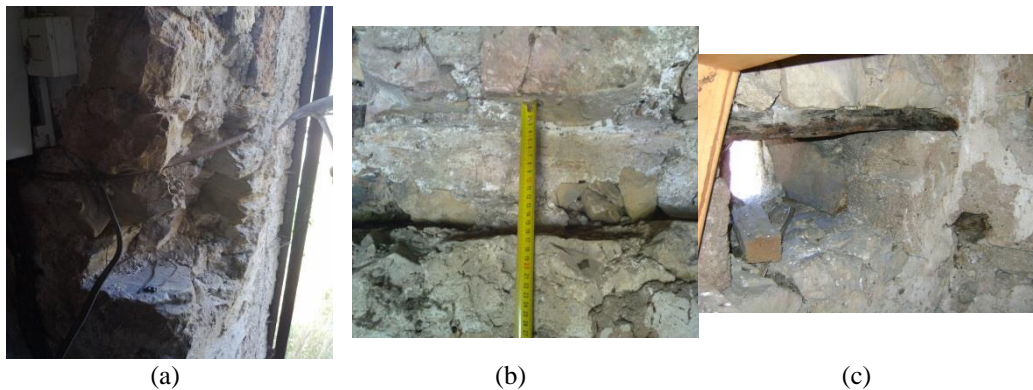
2.1.2 Opis oštećenja konstrukcije

Objekat je temeljen direktno na stijenski masiv. Vizuelno je uočeno ispucavanje i degradacija stijenskog masiva uslijed dejstva atmosferilija (slika 3).



Slika 3 – Prikaz oštećenja temeljnog stijenskog masiva

Na zidovima nisu uočena značajnija oštećenja izuzev mjestimičnog nedostatka maltera u fugama i uklonjenog kamena (slika 4). Fasada je djelimično oštećena metcima ili pucanjem maltera zbog prevelike debljine nanesenog sloja prilikom zadnje sanacije.



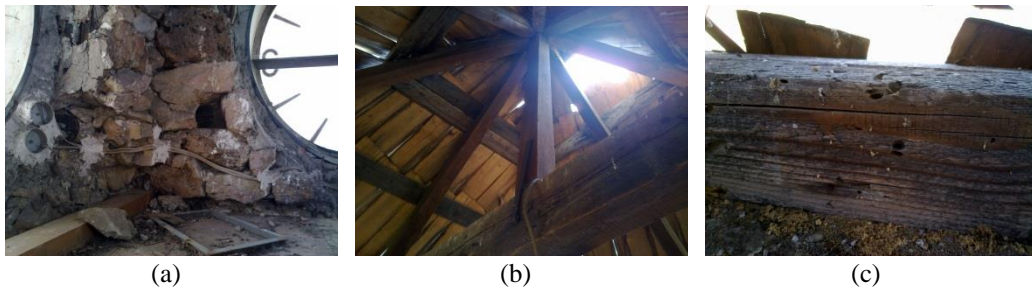
Slika 4 – Prikaz oštećenja na: (a) prizemlju, (b) prvoj i (c) drugoj etaži

Međuspratne tavanice - podesti su obnovljeni i nalaze se u dobrom stanju, međutim isti nisu adekvatno izvedeni jer su korištene grede znatno manjeg presjeka od originalnih, te nije izvedeno pravilno povezivanje greda za kamene zidove.

Na krovnoj konstrukciji primjećena su oštećenja nosivih greda u vidu truljenja drveta (slika 5). Pored konstrukcije primjećeno je oštećenje pokrova od šindre, koji je mjestimično oštećen i

kroz njega prodire voda. Uslijed prodora vode došlo je do oštećenja i drvenih mušebaka krova, kao i mušebaka prozora i samog prozora (slika 6). Drvene mušebke krova potrebno je zamjeniti.

Ulazna vrata uraditi od kvalitetne drvene građe i ugraditi ih sa unutrašnje strane vanjskog zida sa otvaranjem vani (slika 6).



Slika 5 – Prikaz oštećenja na: (a) trećoj etaži i (b), (c) krovu



Slika 6 – Prikaz postojećeg stanja ulaznih vrata i oštećenja mušebaka

2.2 LABORATORIJSKA ISPITIVANJA I PRORAČUN KONSTRUKCIJE

U svrhu ocjene svojstva kamena, materijala od koga je izgrađena nosiva konstrukcija objekta, izvršeno je ispitivanje pritisne čvrstoće i zapreminske mase kamena. Određivanje pritisne čvrstoće je izvršeno u skladu sa JUS B.B8.012 [2]. Ispitivanje je vršeno na tri serije po 5 uzoraka i dobiveni su sljedeći rezultati:

- Srednja vrijednost čvrstoće pri pristisku je 36,60 MPa
- Srednja vrijednost zapreminske mase kamena je 1925 kg/m³

Rezultati laboratorijskih ispitivanja su korišteni kao ulazni podaci pri provjeri stabilnosti konstrukcije, programskim paketom Tower, pri čemu je konstrukcija zadovoljila zahtjevane kriterije stabilnosti.

2.3 MJERE SANACIJE

Uvidom u postojeće stanje objekta, utvrđeno je da postoji opasnost od daljnje degradacije strukture objekta sahat-kule usljed djelovanja atmosferilija, starosti i neadekvatnih radova. Zbog toga je neophodno izvršiti konzervatorsko - resturatorske radove na objektu. Radovi bi se trebali odvijati po sljedećem redosljedu:

- Sanacija temelja
- Postavljanje skele
- Sanacija krova
- Injektiranje zidova i sanacija fasade
- Sanacija podesta i stepeništa
- Izmjena ulaznih vrata
- Elektro i gromobranska instalacija.

2.3.1 Sanacija temelja

Kod temelja na stijenskom masivu se predviđa nanošenje učvršćivača za kamen, koji ima za cilj spriječavanje prodora površinske vode u stijenski masiv, te površinsku stabilizaciju manjih kamenih blokova.



Slika 7 – Detalj zida u podnožju kule

Za veće pukotine u stijenskom masivu predviđeno je injektiranje (zapunjavanje) hidrauličkim krečnim malterom, uz prethodno postavljanje PVC odvodnih cijevi Ø20 unutar pukotine. Na južnoj strani objekta vidljivi su tragovi popravki na samom podnožju zida kule, koji ustvari predstavlja temelj objekta, obzirom da je isti oslonjen direktno na stijenski masiv (slika 3). Za trajnu stabilizaciju ovog dijela zida predviđena je izrada kamenog nabačaja u krečnom malteru, koja ima za cilj da obezbjedi bočnu stabilnost ovog dijela zida, odnosno da obezbjedi njegovo djelovanje kao temelja objekta. Kameni nabačaj se izvodi do visine označene na slici 6, a radi se u nagibu prema obližnjoj stijeni na koju se oslanja čime se stabilizuje i sam nabačaj. U cilju smanjenja hidrostatskog pritiska u stijenskoj masi, predviđena je izrada barbakanama Ø20, bušenjem do dubine cca 60cm.

2.3.2 Postavljanje skele

Oko čitavog objekta neophodno je postaviti cijevnu skelu. Ona služi kako za izmjenu krovnog pokrivača tako i za sanaciju fasade. Skelu je neophodno izvesti u skladu sa važećim propisima.

2.3.3 Sanacija krova

Postojeću krovnu konstrukciju i krovni pokrivač je neophodno zamjeniti novim zbog navedenih oštećenja krovne konstrukcije i mogućnosti daljnje degradacije. Nova krovna konstrukcija bi se radila od ručno tesane hrastove građe istih dimenzija kao i postojeća, a pokrivanje bi se izvršilo kvalitetnom šindrom dužine 60 cm i širine 12 cm. Kako je fasada objekta nepravilnog oblika treba voditi računa da se prepusti strehe urade paralelno sa zidovima. U okviru izmjene krovne konstrukcije i pokrova, izvršiti izmjenu i ugradnju drvenih mušebaka krova, mušebaka prozora i sam prozor kvalitetnom hrastovom građom.

2.3.4 Injektiranje zidova i sanacija fasade

Za potpunu obnovu i zaštitu starih kamenih zidova od uticaja atmosferilija i visoke vlažnosti, često se injektiranje obavlja po različitim fazama, različitom opremom i materijalima. Pošto je objekat Sahat-kule godinama izložen agresivnom djelovanju vlage iz atmosfere, pogotovo oborinskih voda, injektiranjem možemo postićemo slijedeće:

- Popunjavanje zidova, zatvaranje vidljivih pukotina na zidovima (unutrašnjost i na fasadi)
- Injektiranje zidova materijalima i smjesama koje su hidrofobne u cilju zaštite od kapilarnih ili hidrauličnih prodora vode.

U okviru postupka injektiranja predviđa se čišćenje fuga od starog maltera, vegetacije i ostalih nečistoća nakupljenih u otvorima, zatim zapunjavanje fuga sa hidrauličnim krečnim malterom za fuge, sa istovremenom ugradnjom PVC cijevi Ø25 do 35mm u fugama na razmaku 50 cm. Nakon završenog injektiranja potrebno je ukloniti cijevi iz fuga, a zid očistiti od injekcione. Po završetku svih radova izvršiti završno fugiranje kompletnih površina unutrašnjih zidova.

Nakon izvršene sanacije zidova slijedi čišćenje fasade metalnim i PVC četkama u svrhu otklanjanja oštećenja nastala od djelovanja atmosferilija, metaka i neadekvatne debljine maltera nanesenog prilikom zadnje sanacije.

2.3.4 Elektro i gromobranska instalacija

Postojeća elektrinstalacija u objektu je veoma dotrajala što zbog svog očitog stihijskog nastanka, što zbog velike količine vlage unutar objekta. Kablovi su vidljivi a utičnice i rasvjetna tijela su na lošim mjestima. U objektu je montirana i sirena za uzbunjivanje koju treba ukloniti. Predloženi princip rješenja elektroinstalacija je kroz fuge jugozapadnog ugla objekta.

3 ZAKLJUČAK

Objekat Sahat Kule u Donjem Vakufu iziskuje veoma ozbiljnu rekonstrukciju nosivih kamenih zidova. Sadašnje stanje ukazuje na veliku količinu oborinskih voda koje djeluju na unutrašnje zidove i prodiru u njih. Zbog toga je najvažnije uraditi injektažu na najkvalitetniji

moćući naćin (zajedno sa ćišćenjem i konsolidacijom fasade). Zidovi se moraju detaljno oćistiti od lošeg krećnog maltera i fugirati. Ponovo fugirati zidove po završenoj injektaži.

Krovna konstrukcija i krovni pokrivać moraju se zamjenuti sa novim. Prozori, vrata, mušebaci, podesti, serklaži i stepeništa zamjenuti kvalitetnom hrastovom graćom.

Da bi objekat funkcionisao dugo i bez problema potrebno je osigurati kvalitetan i neprestan sistem ventilacije.

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REKONSTRUKCIJA I SANACIJA ŽELEZNIČKOG MOSTA „TREBALJEVO“ NA PRUZI VRBNICA-BAR

Rezime:

S-70

U radu je prikazan istorijat izgradnje, međufaze sanacije, uzroci nastanka eksploatacionih problema i završna faza rekonstrukcije i sanacije Železničkog mosta „Trebalevo“ u Crnoj Gori. Radovi na rekonstrukciji i sanaciji mosta, kako se u skladu sa izvršenim radovima ovaj projekat može nazvati, za osnovu su imali i izvedeni su prema dokumentaciji „Glavni projekat rehabilitacije (sanacije i antikorozijske zaštite) mosta "Trebalevo" na km 334+673.68 (51+192.50) pruge Vrbnica - Bar u Crnoj Gori“. Radovi su završeni 15.08.2016. Izvođač radova je bio STRABAG AG Glavna podružnica u Podgorici.

Ključne reči: Rekonstrukcija, Žel. most, Bunari, Beton, Čelik, Gornji stroj.

RECONSTRUCTION AND REMEDY OF THE RAIL BRIDGE "TREBALJEVO" OF THE VRBNICA – BAR RAILWAY LINE

Summary:

The paper presents the history of construction, interphase rehabilitation, the causes of the problem of exploitation and final phase of reconstruction and remedy of Railway Bridge "Trebalevo" in Montenegro. Works on the reconstruction and remedy of the bridge, as in accordance with the works executed this project may call, for the basis they had been carried out according to the "Design for rehabilitation (rehabilitation and corrosion protection) bridge "Trebalevo "at km 334 + 673.68 (51+ 192.50) railway Vrbnica - Bar in Montenegro". The works were completed 15.08.2016. The contractor was STRABAG AG, main branch Podgorica.

Key words: Reconstruction, Rail bridge, Wells, Concrete, Steel, Permanent way.

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1 UVODNE NAPOMENE

Na lokaciji sadašnjeg mosta „Trebiljevo“, a po Glavnom Projektu iz 1970. godine bila je predviđena izgradnja mosta sistema tipskih lako armiranih betonskih svodova, raspona $L=15+8*18m$. Tokom izgradnje temelja za stubove br. 6 i 7 došlo je do veoma izraženog pomeranja tla. Nakon konstatovanja drugačijeg geološkog profila od projektom predviđenog, urađen je inovirani Glavni projekat mosta kojim je predviđena izgradnja dve rešetkaste čelične konstrukcije raspona $L=2*61.60m$, koje se oslanjaju na stubove br. 3, 6 i 9. Rešetkaste konstrukcije su dvopojasne, statičkog sistema prostih greda, sistemna visina rešetke, pojasnog glavnog nosača je 7.5m, sa razmakom glavnih nosača od 5,3m. Stubovi br. 3, 6 i 9 su adaptirani, a stubovi br 4, 5, 7 i 8 nisu ni izvedeni. Izgradnja dva betonska svoda između stubova S1, S2 i S3 na strani prema Vrbnici i jednog betonskog svoda između stubova S9 i S10 na strani prema Podgorici (Baru) već je bila okončana, kada je utvrđeno odstupanje geologije tla u odnosu na bazni projekat. Konačna dispozicija mosta (slika 1.) je i zatečeno stanje.



Slika 1 – Most „Trebiljevo“ izgled pre rekonstrukcije (04.12.2014.)

Pri pobnom ispitivanju mosta (februar 1976. god.) na betonskim svodovima mosta uočene su prsline, a osmatranjima do 1990. god. utvrđene su deformacije koloseka po osovini i niveleti, povećanje prsline i pojava pukotina u betonskim svodovima lukova kao i oštećenja ankeri nepokretnih ležišta na stubu S3. Za sve navedeno je utvrđeno da je posledica pomeranja tla, koje je prouzrokovalo značajno pomeranja stubova S2, S3 i S9, u najvećoj meri upravno na most i vertikalno u pravcu osovine mosta, a ne znatno manje u pravcu mosta.

Tokom 1995., pošto su se povećavala pomeranja, deformacije i oštećenja betonskog dela mosta, izvršena je delimična sanacija mosta ugradnjom čeličnih podužnih zatega sa ciljem „sprečavanja podužnog razmicanja oslonaca, nejednake rotacije i bočnog pomeranja stubova u odnosu na trasu pruge“. Dubina temelja stubova nije menjana, niti su temelji na bilo koji način ojačavani, nije sanirana kolovozna ploča sa izolacijom, niti je zaustavljeno bočno pomeranje konstrukcije u celini, a nije ništa urađeno ni na smanjenju sleganja stubova.

Od 1995. pa do izrade Glavnog projekta rehabilitacije (kakav nesrećan naziv), na mostu nisu izvršene nikakve sanacione mere, a deformacije, pomeranja i oštećenja, utvrđena nakon geodetskih snimaka 2012. i 2015. i pregleda stanja konstrukcije 2012. i 2015. su pokazala da se most nalazi u stanju labilne ravnoteže i pred mogućim kolapsom.

2 PROJEKTNJA DOKUMENTACIJA I UGOVARANJE

„Glavni projekat rehabilitacije (sanacije i antikorozijske zaštite) mosta "Trebiljevo" na km 334+673.68 (51+192.50) pruge Vrbnica - Bar u Crnoj Gori“ je puno ime projektne

dokumentacije koju je za potrebe Investitora „Željezničke Infrastrukture Crne Gore“ (ŽICG) uradio SI CIP iz Beograda, a koji je revidovao „Institut za građevinarstvo“ Podgorica.

Glavni Projekat rehabilitacije finansiran je iz kredita Evropske investicione Banke (EIB), prema uslovima Red Fidic book edicija 1999, a na tenderskoj proceduri koja je obavljena decembra 2014 izvođenje radova je dobio JV STRABAG AG Austria i Crnogoraput AD Podgorica, sa nominovanim podizvođačem GP Mostogradnja AD Beograd predstavljen kroz dio stranog društva Podgorica, sa kojim je tokom izvođenja radova zbog ne izvršavanja ugovorenih obaveza, a po zahtevu Inženjera i Investitora raskinut podizvođački ugovor. Iz tenderske procedure bili su povučeni, Knjiga 1 Utvrđivanje postojećeg stanja mosta, Knjiga 2 Idejni projekat mosta i Knjiga 9-Idejni projekat privremenih saobraćajnica.

Ugovor sa Investitorom ŽICG, potpisan je 05.02.2015., rešenje da je projekat oslobođen izdavanja građevinske dozvole dobijeno je 15.04.2015., a radovi su zvanično počeli 30.04.2015. Predviđeni rok za izvršenje radova je bio godinu dana, ali je produžen zbog velikog obima naknadno utvrđenih dodatnih i ne predviđenih radova prvo do 11.07.2016., a zatim i zbog ne predviđenih radova na injektiranju temeljnog podtla stubova do 15.08.2016. Inženjer na izgradnji objekta je bila firma iC consulenten GesmbH, Austria.

Budžet za izvršenje radova zajedno sa 10% Contingencies je iznosio 2.952.066Evra, ali je premašen iz razloga ne predviđenih i dodatnih radova i dodatnih troškova na trajnom zbrinjavanju opasnog otpada koji je posledica peskarenja čelične konstrukcije mosta, a sadržavao je u sebi otpad od olovnog pigmenta u procentu manjem od 3%. Zbog ne dostajućih sredstava u budžetu i premašene vrednosti radova, od ukupno ugovorenih radova, po odluci Investitora, a uz saglasnost Projektanta, Inženjera i Izvođača radova, izdvojeni su radovi koji ne utiču na stabilnost konstrukcije. Radovi na mostu su završeni, budžet prekoračen, ali dodatna sredstva nisu do danas nadoknađena Izvođaču. Odluka o daljim koracima još nije doneta.

3 OPIS PROJEKTOG REŠENJA

Glavnim projektom rehabilitacije mosta predviđeno je:

1. Promena sistema fundiranja mosta sa plitkog temeljenja na „Bunare“ - šipove
2. Sanacija betonskih površina i prslina lukova i zidova mosta
3. Izrada novih AB obložnih lukova i zidova mosta i pešačkih konzola mosta
4. Podužno sprezanje nove i stare betonske konstrukcije, utezanjem oslonačkih delova lukova kod stubova S1 i S10
5. Popoprečno sprezanje novih naglavnih greda šipova sa postojećim temeljima
6. Sanacija prslina, oštećenja i zamena spojnih sredstava čeličnih rešetkastih nosača
7. Kompletna antikorozivna zaštita čelične konstrukcije mosta i svih čeličnih elemenata na mostu (zatege, ograde, pešačke i revizione staze)
8. Kompletna rekonstrukcija gornjeg stroja mosta (zamena tucaničkog zastora, šina, šinskog pribora i pragova, sanacija betonske ploče zastora i izrada nove MMA hidroizolacije, zamena konzola elektromreže napojnog kabla);
9. Pomeranje jedne čelične rešetke mosta za 32cm sa kompletnom revizijom ležišta
10. Rektifikacija koloseka i uklapanje novo-izgrađenog sa postojećim
11. Rekonstrukcija sistema spoljašnjeg odvodnjavanja
12. Izrada pristupnog puta i radnih platoa oko mosta i do mosta
13. Rekonstrukcija geodetske mreže uz stalno geodetsko praćenje stanja mosta

3.1 PROJEKTOVANI SANACIONI RADOVI BETONSKIH KONSTRUKCIJA

Obzirom da su temelji stubova S1, S2, S3, S9 i S10 plitko fundirani, projektnom dokumentacijom je predviđeno da se celokupna sopstvena težina konstrukcije kao i celokupno saobraćajno opterećenje prenose na novoprojektovane "bunare"- šipove, koji se izrađuju pored temelja stubova i ukupno opterećenje prenose na dobro nosivi sloj četiri (prema geološkom profilu tla u osi mosta). Prenos opterećenja projektovan je preko „rigle“ krutog rama formirane od naglavnih greda armirano betonskih "bunara" prečnika Ø2000mm sa nosivim jezgrom- AB šipom prečnika Ø1700mm i temelja/tela stuba, međusobno utegnutih kablovima za prednaprezanje. Tako formirana kruta „rigla“, odnosno zajednički rad naglavnih greda i temelja/tela stuba, ostvaruje se preko kontaktne površine beton-beton, nosivošću na trenje. Koeficijent trenja beton – beton usvojen je u iznosu od $\mu=0,8$.

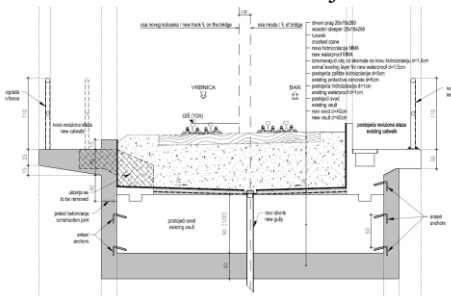
Projektom je predviđeno da se uz bočne strane temelja izrade: kod stuba S1 2x1 armirano betonski "bunar", dubine oko 13,00m, kod stuba S2, izvedu 2x2 AB "bunara", kod stuba S3, izvedu 2x2 AB "bunara", dubine 21,70m., stuba S9, izvedu 2x2 AB "bunara" i kod stuba S10 2x1 armirano betonski "bunar", dubine oko 16,00m

Horizontalna dejstva kao posledica pomeranja tla primaju se geotehničkim sidrima 7Ø16 koja se preko naglavnica šipova pod uglom od približno 45°, sidre u čvrste stenovite slojeve tip "4", sidrene dužine oko 10m. Slobodna dužina sidra koja se uteže se kreće od 17m do 30m.

Projektom je predviđena izgradnja nove lučne mostovske konstrukcija koja se formira od AB lukova d=40cm, koji su locirani ispod postojećih svodova i izgradnja nove AB "maske" – zidova d=25cm, tako da se formira kontura mosta kakva već postoji. Na taj način noseći AB lukovi i bočne "maske" formiraju neku vrstu "kade" - preseka u kojoj se nalazi postojeća

konstrukcija (slika 2.).

Takođe je na postojećim ne armiranim lučnim betonskim konstrukcijama projektovano izvođenje sanacionih radova (zalivanje i injektiranje prslina izrada izravnavajućeg sloja za pad, izrada nove hidroizolacije MMA, reparature slivnika i slično), kao i oblaganje reparaturnim malterom spoljnih površina stubova i izrada novih konzolnih pešačkih staza.



Slika 2 – AB obloga postojećih svodova,

3.2 PROJEKTOVANI SANACIONI RADOVI ČELIČNIH KONSTRUKCIJA

Statičkim proračunom čelične konstrukcije utvrđeno je da lokalna i globalna stabilnost čeličnih rešetkastih konstrukcija nije ugrožena za šemu pokretnog opterećenja D4, ali su pregledom konstrukcije uočena oštećenja čeličnih elemenata, prslina na varovima, nedostaci vijaka i zakivaka i deformacije vertikalne sprega za BU u polju XVIII-XIX. Projektovana je za svaki od naznačenih elemenata metoda sanacija. Pored navedenog, zahtevano je pomeranje čelične konstrukcije rešetke na delu od stuba S3 do stuba S6, zbog korekcija na stubu S3 koja je uslovljena pomeranjem trase pruge u odnosu na projektovani položaj, pa je u odnosu na čeličnu konstrukciju bilo potrebno izvršiti korekciju osovine čelične konstrukcije K1 (između stubova S3 i S6) tako da se konstrukcija K1 zarotira oko sredine oslonake poprečne rigle (XI) na stubu S6 da bi pomeranje u osloncu na S3 ka brdu iznosilo 320mm.

Na konstrukciji K2 nije predviđeno pomeranje konstrukcije, već samo privremeno podizanje presama radi čišćenja i podmazivanja (pokretna) ležišta i obnavljanje podlivki ispod donjih ležišnih ploča.

Detaljnim pregledom čelične konstrukcije mosta utvrđeno je da je antikoroziona zaštita degradirana, da je prilikom izgradnje kao premaz upotrebljen ftalatni minijum koji sadrži olovo i da je prionjivost premaza mala (klasa 4), pa je predviđena kompletna obnova AKZ premaza sa sledećim karakteristikama: AKZ zaštita za kategoriju korozivnosti C4 sa vekom trajanja od min 20 godina, sistemom na bazi epoksidnih smola, debljine: osnovni premaz cink epoksid, $d=60\mu\text{m}$ i dva međupremaza, svaki $d=80\mu\text{m}$. Završni pokrivni premaz na bazi poliuretana $d=60\mu\text{m}$, što daje ukupnu debljinu od $280\mu\text{m}$. Unutrašnje strane zaštititi na isti način, samo bez završnog poliuretanskog premaza. Ukupna debljina 3 premaza na tim mestima je $220\mu\text{m}$.

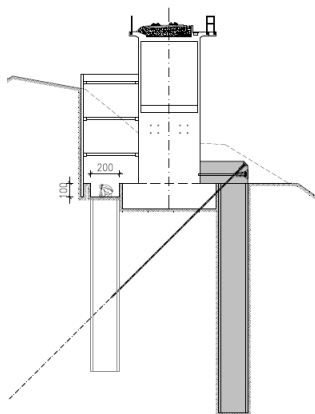
3.3 PROJEKTOVANI SANACIONI RADOVI GORNJEG STROJA

Glavnim projektom je predviđeno uređenje koloseka na delu pruge od km 334+437.90 do km 335+052.46 sa mostovskim konstrukcijama dužine 182.80m, i delovima otvorene pruge 112.05m ispred mosta i 256.06m iza mosta. Za deo koloseka na mostovskim konstrukcijama je izvršena provera njegove stabilnosti, a rezultati su pokazali da kolosek ne bi bio stabilan ukoliko bi se zavarilo u dugi šinski trak (DTŠ).

Gornji stroj na čeličnim rešetkastim konstrukcijama je projektovan kao i postojeći bez zastora samo sa novim šinama tipa 49E1, sigurnosnim ugaonicima, oslonačkim papučama, tirfonima pragovima i kolosečnim pričvrstnim priborom tipa K za mostovske drvene pragove. Između sigurnosnih ugaonika nalazi se traka rebrastog lima koja ostaje, ali se reparira i sa donje strane antikoroziono zaštićuje. Čelična konstrukcija K1 se nalazi u prelaznici $L=120\text{m}$, a čelična konstrukcija K2 u kružnoj krivini radijusa $R=300\text{m}$. Gornji stroj na betonskim lučnim konstrukcijama je predviđen sa novim šinama tipa 49E1, novim pričvrstnim kolosečnim priborom tipa K za drvene pragove i tucaničkim zastorom eruptivnog porekla. Prve dve betonske lučne konstrukcije su u prelaznici $L=120\text{m}$, a treća u kružnoj krivini radijusa $R=300\text{m}$.

3.4 OSTALI PROJEKTOVANI RADOVI

Ostali projektovani radovi odnosili su se na izgradnju pristupne saobraćajnice i radnih platoa pored stubova, na radove izrade novog dekorativnog osvetljenja mosta i radove izgradnje zatvorenog sistema prikupljanja otpadnih voda. S obzirom da je glavni projekat pristupne saobraćajnice i radnih platoa povučen iz tenderske procedure i da Izvođač radova nije imao uvid u isti i da nije ni ugovoren, projekat odvodnjavanja atmosferskih voda je izmenjen uz saglasnost Inženjera i Investitora. Iz razloga povlačenja projekta pristupne saobraćajnice, Izvođač radova je morao da u fazi tenderske procedure donese i odluke vezane za tehnologiju izvođenja radova i način dopreme materijala, opreme i ljudstva, pa je Izvođač ipak odlučio da izgradi pristupnu saobraćajnicu i radne platoe po sopstvenim planovima. To je uključivalo obavezu da plati troškove privremenog zakupa zemljišta, rekonstruiše most na reci Tari i promeni idejni projekat tehnologije građenja. Investitor nije promenio predmer radova iako je povukao projekat privremene saobraćajnice, što je direktno uticalo na tehnologiju rada. Kao primer u radu se navodi izrada podgrada iskopa iz predložene tehnologije izgradnje slika 3. Izvođač nije mogao da prihvati da, prema projektu rehabilitacije koji je oslobođen građevinske dozvole, garantuje stabilnost mostovske konstrukcije kada je predviđeno da se iskop bunara



vrši prema sa slike prikazanom detalju, a sve pozicije radova iskopa su u predmeru radova tretirane na prikazani način. Razmimoilaženje u stavovima da se ne sme primeniti tehnologija iskopa prikazana na slici već da se iskop mora izvoditi sa novo formiranih platoa prethodno obezbeđenih sa padinske strane gabionskim zidovima(što nije bila stavka Iskopa) je stvorio razliku u obračunu iskopa koju Investitor nije hteo da prihvati do kraja projekta, a Nadzor nije mogao da verifikuje jer je izmenama člana 3.1.u partikularima osnovnog ugovora bilo navedeno da konačnu odluku u odobravanju varijacija ima Investitor.

Slika 3 – Projektom predložena tehnologija iskopa,

4 OPIS IZVOĐENJA RADOVA

Pre otpočinjanja sa izvršenjem radova Izvođač radova je pregledao konstrukciju izvršio geodetska osmatranja, uradio Plan i program radova i uporedio stanje konstrukcije sa stanjem koje je prikazano u projektnoj dokumentaciji. Konstatovao je ne poklapanje sa projektnom dokumentacijom i predložio Inženjeru određene izmene i poboljšanja uz angažovanje projektantskog nadzora. Pre zvaničnog početka radova u očekivanju izdavanja rešenja da je projekat oslobođen građevinske dozvole otpočeli su radovi na izgradnji pristupne saobraćajnice, sanaciji postojećeg mosta preko Tare i zakupa zemljišta za pristupnu saobraćajnicu. Na osnovu sprovedene analize projektne dokumentacije Izvođač radova je izneo zahteve za izmenu i dopunu projektovanog stanja:

1. Izmenu projektovane tehnologije zaštite iskopa. Predlog izmene se prihvata od strane Projektanta i Inženjera, ali bez izmena u obračunu količina iskopa od strane Inženjera
2. Preprojektovanje ramovskih konstrukcija na način da se: kablovi za prednaprezanje rasporede u dva reda, da se dodatno ugrade anker sidra za vezu stub-naglavnica i da se dodaju water stop trake i betonski holker da spreče prodor vode u spoj naglavnica – stub. Predlog je prihvaćen od strane Projektanta, Inženjera i Investitora.
3. Zahtev za tehnološku promenu izrade konstrukcije Bunara-šipova pomenom sistema ručno kopanih bunara na sistem bušenih šipova. Predlog Izvođača prihvata Projektant koji je i uradio dopunu Glavnog projekta, ali ne i Inženjer, pa isti nije sproveden.
4. Zahtev da se radovi na sanaciji ploče tucaničkog zastora i izradi hidroizolacije zbog sumnji da su oštećenja značajna i da se ne mogu izvesti za ugovoreni zatvor saobraćaja od 10 dana, izvedu uz upotrebu „šinskog provizorijuma“, kojim bi se omogućio železnički saobraćaj, a rizik da radovi ne budu izvedeni u predviđenom roku izbegnut. Predlog je posle višemesečnog pregovaranja i odlaganja radova delimično sproveden uz saglasnost Investitora i Inženjera.

U kasnijim fazama radova naknadno su utvrđena i mnogobrojna dodatna oštećenja i nedostaci na betonskoj konstrukciji, na čeličnoj konstrukciji, a i u pod tlu postojećih temelja.

4.1 IZVEDENI SANACIONI RADOVI BETONSKIH KONSTRUKCIJA

U tekstu koji sledi prikazane su najkarakterističnije slike sa pratećim opisima radova koji su izvedeni u cilju izvršenja rekonstrukcije, sanacije i rehabilitacije konstrukcije mosta.

Na slici 4 prikazan je izgled jednog od 16 izvedenih bunara. Vidi se obloga bunara koja je



izvedena od AB prstenova prečnika 2000mm, debljine stenke od 150mm. Između obložnih prstenova i okolnog tla na zahtev Inženjera izvršeno je injektiranje spoja cementnom emulzijom. Dubine bunara su od 13m do 24m, a sam iskop je izveden ručno, primenom samo pneumatskog alata- čekića.

Slika 4 – Izgled skopanog „Bunara“



U unutrašnjost bunara smešteni su armaturni koševi i izvršeno je betoniranje šipa prečnika 1700mm. Pre armiranja i betoniranja naglavnih greda izvršena su bušenja postojećih stubova/temelja za montažu kablova za prednaprezanje kojima su utegnute naglavne grede sa obe strane stuba i sam stub, montirane su vodeće cevi za geotehnička sidra i ugrađeni su ankeri za vezu postojećih stubova/temelja sa novim naglavnim gredama.

Slika 5 – Montaža armaturnih koševa



Slika 6 –Izgled naglavnica sa i bez instaliranih kablova za prednaprezanje



Projektom predviđena sanacija oštećenja betonskih obložnih zidova i lukova pokazala se kao nedovoljna, s obzirom da su nakon peskarenja i odstranjivanja oštećenog betona utvrđene ne samo prsline već i pukotine, kaverne i segregacije betona. Novim projektom sanacije je predloženo i izvršeno injektiranje i obrada svih oštećenja visokovrednim reparaturnim malterima klase IV sa završnim zaštitnim reparatur premazom.

Slika 7– Oštećenja betonskih lukova

Nakon izvršenih injektiranja i obrade betonskih površina, ugrađeni su ankeri za vezu stari novi beton, montirana armatura obložnih zidova i luka i sproveden postupak betoniranja nove betonske konstrukcije. Nova konstrukcija lukova je kablovima utegnuta za obalne stubove S1(lukovi S1-S2-S3) i S10 (luk S9-S10), a na stubovima S2, S3 i S9 je direktno spojena sa postojećim temeljima/stubovima i novim naglavnim gredama. Tako formiranom betonskom konstrukcijom opterećenja predviđena projektom prenose se na novo formirani kruti ram i šipove. Na slikama su prikazane faze sa montiranom armaturom postavljenom oplatom zidova i lukova. Betoniranje je sprovedeno u tri faze. I faza: betoniranje betonskih obložnih zidova za vezu novoformiranih naglavnih greda i postojećih stubova; II faza: betoniranje novih lukova i bočnih obložnih zidova i III faza: betoniranje gornjih delova obložnih zidova i konzolnih kratkih elemenata novih pešačkih staza.



Slika 8–Oplata, skela i armatura novih betonskih lukova i zidova

Sanacija oštećenja ploča i zidova sanduka u kojima je smešten tucanički zastor koloseka sprovedena su u toku četvorodnevno totalnog zatvora saobraćaja primenom visokovrednih i brzovezujućih specijalnih injekcionih maltera i smola koji su morali biti kompatibilni sa novo projektovanom hidroizolacijom, sistema na bazi Meta-Methyl akrilata (MMA). Na slikama su prikazane faze i izgledi sprovedenih postupaka. Primenjen je sanacioni materijal kompanije Stirling Loyd -Metaset kompatibilan sa MMA sistemom istog proizvođača



Slika 9–Sanacija oštećenja reparatur malterima i smolama

4.2 IZVEDENI SANACIONI RADOVI GORNJEG STROJA

Sanacioni radovi na gornjem stroju sprovedeni su u dve faze. I faza u toku totalnog zatvora saobraćaja kada je izvršena demontaža postojećeg koloseka na betonskim delovima mosta, sanacija oštećenja betonskog sanduka za smeštaj tucaničkog zastora koloseka, izrada hidroizolacionog sloja sanduka, izrada novog tucaničkog zastora, montaža novih šina, pragova, kolosečnog pribora na betonskim konstrukcijama mosta, zamena oštećenih delova i ugradnja novih elemenata visokonaponske mreže i uklapanje starog i novog koloseka kako bi se saobraćaj odvijao u režimu smanjene brzine. II faza, za vreme dnevnih zatvora saobraćaja u trajanju od 4 do 5 časova dnevno, kada je izvršena zamena kolosečnog pribora, pragova i šina na čeličnoj konstrukciji mosta, finalne rektifikacije, podbijanje pragova i završno uklapanje postojećeg i novog koloseka. Tokom prve i druge faze radova sprovedeni su i radovi na sanaciji ograde mosta, izrada hidroizolacije pešačkih staza, sanaciji kanala za smeštaj optičkog kabela, samog optičkog kabela, zameni ostalih signalnih uređaja i uzemljenja objekata mosta i opreme na njemu. Na prikazanim slikama mogu se videti neke od opisanih faza radava.



Slika 10 Faze radova na sanaciji i rekonstrukciji gornjeg stroja

4.3 IZVEDENI SANACIONI RADOVI ČELIČNIH KONSTRUKCIJA

Radovi na obnovi AKZ mosta izvršeni su prema zahtevima glavnog projekta, u potpunosti. Izvršenje radova pratili su složeni uslovi s obzirom na atmosferske prilike i činjenicu da se



most nalazi na 1000m nadmorske visine. Visoka vlažnost i vetar uslovili su posebne zahteve u pogledu primenjenog sistema AKZ zaštite. Most je na celokupnoj dužini bio pod radnom skelom, po konturi zaštićenom mrežom i folijama. Peskarenje je obavljeno gritom pod visokim pritiskom, a otpadni materijal je skupljan vakum usisavanjem i deponovan na gradilištnu deponiju kao kontaminirani otpad. Po izvršenju radova otpad je zatvoren u dva objekata potpuno zaštićena od spoljašnjih uticaja. Urađena je kompletna toksikološka provera zemljišta i vode pre, posle i za vreme izvršenja radova. Pored AKZ zaštite konstrukcije na mostovskim konstrukcijama, K1 i K2, urađena je revizija i podmazivanje ležišta, a sama konstrukcija K1 je horizontalno pomerena, upravno na podužnu osu za 340mm. Zahvat sa pomeranjem konstrukcije izveden je tokom totalnog zatvora saobraćaja uz izradu novih ležišnih kvadera i nivelaciju mostovske konstrukcije po visini i pravcu.

Ključni problem tokom izvršenja radova na sanaciji konstrukcije je stanje nosećih rešetkastih konstrukcija na kojima su utvrđena, još tokom peskarenja značajna i brojna oštećenja. Analiza tih oštećenja pokazala je da su ista nastala tokom eksploatacije zbog ne održavanja konstrukcije i u najčešćem slučaju je uzrok ne dostajućim/ ne pritegnuti zakivci/vijci koji su pod uticajem dinamičkih uticaja ispali iz funkcije i prouzrokovali prsline u šavovima, oštećenja na limovima i ukrućenjima i deformacije flanši i delova spregova. Iz razloga sanacije, bilo je potrebno, kao i na betonskim delovima mosta, izvršiti pregled svih elemenata mosta, uraditi novi projekat sanacije i obezbediti saglasnost Investitora jer su radovi zajedno sa radovima na sanaciji betonskih površina prekoračivali vrednost od 10% ukupne vrednosti radova. Slike uz tekst pokazuju faze pojedinih radova.



Slika 11 Izgled mosta nakon sprovedenih sanacionih radova (kao zaključak prezentacije)

Dorđe Martinović¹

TRANSFORMATORSKE STANICE 132 / 11kV U DUBAIU

Rezime:

U saopštenju se daje prikaz Transformatorskih stanica izvedenih u Dubaiu kao deo prenosne mreže enegretskog sistema uklpoljenog u jedinstveni sistem Ujedinjenih Arapskih Emirata -UAE. To je deo konsultanskih usluga urađenih za klijenta Dubai Electricity and Water Authority -DEWA od strane Energoprojekt Entel.

Ključne reči: Energoprojekt , DEWA, Dubai, Transformatorske stanice

GRID SUBSTATIONS 132 / 11 kV IN DUBAI

Summary:

The paper presents Grid Substations constructed in Dubai as the part of Power Transmission Network connected to the Power System of the United Arab Emirates - UAE . It is the part of the Consultancy Services performed for the Client Dubai Electricity and Water Authority - DEWA from the Energoprojekt Entel as Consultant.

KeyWords: Energoprojekt, DEWA, Dubai, Grid, Substation

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1 UVOD

Energoprojekt - Entel je angazovan od 2004 godine u Dubaiu na pružanju konsultantskih usluga za Dubai Electricity and Water Authority - DEWA. Predmet ovog referata je prikaz izgradjenih 12 Transformatorskih stanica napona 132 / 11 kV od 2004-2006 godine.

Prvih pet Trafostanica je realizovano ugovorom o vršenju elektro - građevinskog nadzora na kojima je generalni izvodjac radova i isporucioc opreme bio SIEMENS kako sledi :

- Emirates Mall
- Ruwayyah
- Burj Khalifa (Burj Dubai)
- Meadows
- Up Town

Sledeci ugovor od tri Trafostanice na kojima je Energoprojekt- Entel vrsio projektantske usluge kao i elektro - građevinski nadzor gde je generalni izvodjac radova bio Emirates Electrical Engineering - EEE su sledece :

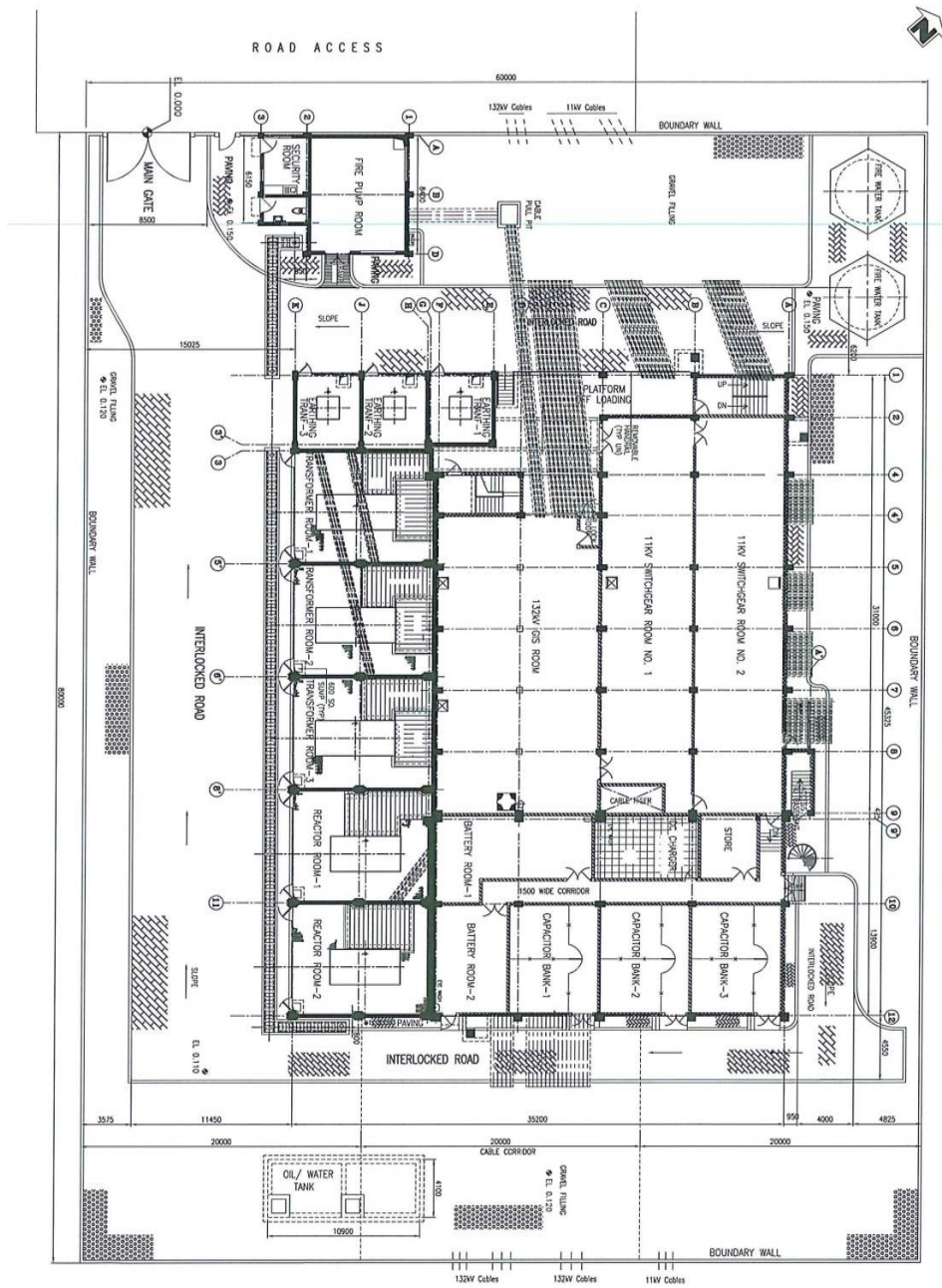
- Techno Park
- Diamond Centre
- Gardens of Discovery

Takodje jos cetiri Trafostanice sa elektro-građevinskim nadzorom od strane Energoprojekt - Entela na kojima su generalni izvodjaci bili Siemens , Emirates Electrical Engineering - EEE i Emirates Trading Agency - ETA su :

- Ahmadia (Ras Deira) - SIEMENS
- Dubai Investments Park - EEE
- Dugas - EEE
- Gazelle - ETA

2 GENERALNA DISPOZICIJA TRAFOSTANICE

Dispozicija svake Trafostanice je definisana na poseban nacin u zavisnosti od raspolozivog placa predvidjenog generalnim urbanistickim planovima svake od lokacija kao i tehnoloskih zahteva proisteklih iz plana elektro- energetske prenosne mreze. Karakteristicna Trafostanica Emirates Mall 132/11 kV (vidi Sliku 1) ima dimenzije placa 60 x 80 m. ogradjena klasicnim zidom od betonskih blokova stabilizovanim armirano betonskim stubovima i temeljima Centralno mesto zauzima Glavna zgrada trafostanice. Takodje kao pomocni objekat izvedena je Pumpna stanica sa portirnicom neposredno kod glavnog ulaza. Dva protiv pozarna celicna rezervoara za vodu kao i standardni ukopani armirano betonski uljno - vodeni rezervoar / separator. Prvenstveno prema zahtevima opreme funkcionalno su postavljeni unutranji putevi i trotoari oko glavnog objekta od interlok prefabrikovanih ploca postavljenih na standardno pripremljenoj podlozi. Ostali delovi placa koji nisu obuhvaceni objektima i putevima /trotoarima su pokriveni slojem sljunka odredjene granulacije.Takodje ispred glavnih Tranformatora izveden je podzemni armirano betonski kanal sa protiv pozarnim cevovodom od Pumpne stanice celom duzinom Glavnog objekta.

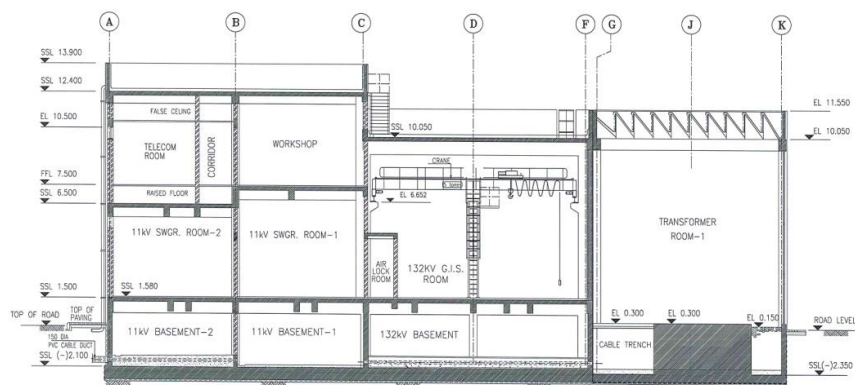


Slika - 1

3 ARHITEKTONSKO - GRADJEVINSKA KONCEPCIJA

Glavna zgrada velicine u osnovi 45x35 m je jednospratna sa podrumom gde se vrši prijem ulaznih kablova kroz spoljne zidove njihov smestaj i veza u kompletan sistem Trafostanice kao i obezbedjenje izlaza potrebnih kablova kroz zidove posle elektricne transformacije . Prizemlje služi za tri Razvodna postrojenja jedno napona od 132 kV i dva napona od 11 kV. Takodje u prizemlju su glavni Transformatori 132/11 kV kao i pomocni Transformatori 11/0.4 kV sa ostalom opremom. Prvi sprat iznad razvodnih postrojenja 11 kV služi za smestaj Kontrolne sobe , Telekomunikacione sobe , i drugih prostorija sa odgovarajucom elektro opremom. Oprema za klimatizaciju je smestena na delu krova . Glavni kao i pomocni transformatori su smesteni u posebne sobe/boksove odvojene betonskim protivpozarnim zidovima, Svaki Transformator ima zasebnu kadu za razvod kablova i prijem ulja u slucaju moguće havarije. Trafo boksovi su sa prednje fasadne strane zatvoreni sigurnosnom metalnom ogradom a odozgo su zatvoreni testerastim / sed krovom sa mogucnoscu ventilacije preko vertikalnih strana sto je zahtevano funkcijom Transformatora. (vidi Slike 1 i 2).

Pumpna stanica sa portirnicom velicine u osnovi 10.6 x 8.4 m je prizemna zgrada za smestaj Pumpi za protiv pozarnu i servisnu vodu kao i dizel generatoraza za vanredne situacije Velicina Pumpne stanice u osnovi je 8.4x7.3 m a Portirnice 3.3x 6.15 m. (vidi Sliku 1).



Slika – 2

4 ZAVRSNI RADOVI NA ZGRADAMA

Krovovi su izvedeni kao ravani sa bitumenskom vodonepropusnom membranom debljine 4 mm preko koje je postavljena termoizolacija a kao završni sloj su korišćene nearmirane betonske ploče 500x500x50 mm. Vodonepropusnost je dokazivana testiranjem uz zadržavanje vode na krovu u toku 24 casa sa naknadnom inspekcijom. Pregradni zidovi su izvedeni od betonskih osupljenih blokova a spoljni od betonskih blokova takodje osupljenih sa dodatkom termoizolacije i spolnjeg punog betonskog bloka kao sendvic sistem. Zid između Razvodnog postrojenja i Transformatora je takodje termoizolacioni tipa sendvic s tim sto je umesto supljih blokova na strani prema Trafo boksovima izveden armirano betonski zid do vrha objekta radi zaštite od moguće havarijske eksplozije. Podovi su od sitnozrnog betona sa završnim

epoksidnim premazom, debljine 50-80 mm zavisno od funkcije prostorije i potrebe za ostavljanjem odgovarajucih profila za fiksiranje opreme (podrumski delovi , razvodna postrojenja i transformatorski prostori) Razvodna postrojenja 11 kV imaju dodatni sloj pre završnog premaza od epoksidnog maltera 6 mm protiv habanja zbog zahteva opreme. Keramicki podovi su predvidjeni za prostorije sa baterijama od kiselo otpornog materijala na prethodnoj podlozi od sitnozrnog betona debljine 50 mm. Podovi stepenista su od terazzo -a. U Kontrolnim i Telekomunikacionim sobama podovi su izvedeni kao dupli sa donjim prostorom za uvođenja raznih elektro i komandno signalnih kablova. Plafoni su ili ravni na obradenoj betonskoj podlozi sa odgovarajucim premazima ili spušteni plafoni prema potrebi smestaja skrivenih instalacija. Spoljasnji i unutrašnji zidovi su malterisani . Unutrašnji zidovi imaju odgovarajuće ravne premaze a na fasadi sa posebnom teksturom otporne prema zahtevima spoljne sredine. Ukrasni elementi na fasadama su izradjeni od prefabrikovanog betona (vidi Sliku 3). Klima uredjaji na krovu zaklanjaju se aluminijumskim panelima zbog estetskog razloga . Takodje iz estetskih zahteva trafo boksovi su na fasadi su dodatno oblagani perforiranim livenim aluminijumom. Sva unutrašnja vrata su celicna a spoljasnja aluminijumska prema funkcionalnim zahtevima, posebno u zavisnosti od vrste opreme koja se unosi. Prozori su takodje aluminijumski.

5 STATICKO - KONSTRUKTIVNI DEO

Armirano betonska konstrukcija livena na licu mesta Glavne zgrade je podeljena u tri nezavisna dela (vidi Slike 1 i 2). Prvi i najveći deo je konstrukcija koja zahvata razvodna postrojenja izmedju osa A-F/1-9 dimanzija u osnovi sa dve razlicite visine krovne konstrukcije od 13.90/11.55 m. Konstrukcija je u statickom smislu prostorni ramovski sistem sa tri glavna broda raspona $6.50+6.50+12.50 = 25.00\text{m}$. U rasponu od 12,50 m predvidjen je mostni kran nosivosti 50 kN za montazu i održavanje opreme 132 kV Razvodnog postrojenja.

Fundiranje je izvršeno na ravnoj masivnoj ploči bez greda debljine 650 mm koja po obodu ima kontinualne armirano betonske zidove koji zajedno formiraju centralni kablovski podrumski prostor. Zidovi su projektovani sa velikim brojem perforacija/cilindricnih otvorara rasporedjenih u blokovima prema zahtevu ulaska/izlaska kablova precniku 250-300mm. Iz tog razloga zidovi debljine 250 mm su ojacavani iznad serije otvora skrivenim gredama sa poebnom dodatnom armaturom.

Armirano- betonska ploca prizemlja iznad kablovskog podruma debljine 200-250mm je takodje visoko perforirana za potrebe veze kablova sa opremom. Ojacanja ploce su vrsena dodatnim sistemom vidnih i skrivenih sekundarnih greda pored primarnih greda

Ploca prvog sprata iznad Razvodnih postrojenja 11 kV oslonjena preko primarnih greda je karakteristicno izvedena u dve visine sa razlikom od 1.00 m za potrebe projektovanog dvostrukog poda u Kontrolnoj i Telekomunikacionoj sobi kao i drugim slicnim sobama. Postoje karakteristicne grede za prenosenje opterećenja ploca na gornjoj ,srednjoj i donjoj ivici koje imaju visinu od 1.00 - 2.00m. Najviše grede od 2.00 m u pojedinim projektnim resenjima mogu imati u sredini raspona skokovitu promenu visine i do 1.00m. Usled razlicite visine i znatnog broja kruznih kablovskih otvora potrebna je posebna konstruktivna obrada sa vrlo slozenim armiranjem.

Krovna ploca je debljine 200 mm koja ima veće otvore uglavnom za potrebe uvođenja kanala za klimatizaciju do odgovarajuće opreme smestene na krovu, a oslanjaju se na sistemske grede prostornih ramova. Po celom obimu krova je zastitni armirano - betonski parapet visine

1,40m a gde je potrebno iz estetskih razloga dodatno sakriti rezervoare vode ili opremu za klimatizaciju visina se može po potrebi povećati.

Drugi dilatacioni blok je zona Transformatora i Reaktora velicine u osnovi 45.0x10.0m zmedju osa E-K/ 1-12. Transformatori su oslonjeni na nivou prizemlja preko pojedinačnih masivnih temelja i masivne ravne ploče bez greda debljine 450mm. Po celom obimu ploče su podzemni betonski zidovi debljine 250-300mm koji sa temeljno plocom formiraju Kadu transformatora sa dodatnim međuzidovima za svaki Trafo boks. Zidovi izmedju svakog Trafo boksa se takodje podizu do visine od 10.05 m i služe kao protivpozarni. Trafo boksovi su pokriveni celicom resetkastom konstrukcijom

Treci poslednji konstruktivni blok Glavne zgrade dimenzija u osnovi 14.0x25.0m izmedju osa A-F/9-12 je prizemna prostorna armirano- betonska ramovska konstrukcija sa betonskom krovnom plocom debljine 200 mm. i plocom prizemlja oslonjenom na kompaktirano tlo. Fundiranje je izvedeno na trakastim temeljima / kontra-gredama debljine 650 mm. Temelji sva tri konstruktivna bloka Glavne zgrade su fundirani na istoj dubini od 2.75 m, a sto je iznad nivoa podzemne vode.

Konstruktivni (Ready Mix) beton je kvaliteta MB 40 MPa sa maksimalnom velicinom agregata od 20 mm. Ordinary Portland Cement -OPC je upotrebljavan iznad zemlje a Moderate Sulphate Resistant Cement - MSRC u zemlji obzirom na značajne vrednosti sulfata i hlorida u građevinskom tlu. Sekundarni betoni za podove su MB20 sa maksimalnom velicinom agregata od 10 mm. Sve podzemne betonske konstrukcije se dodatno štite od agresivnog dejstva tla prema standardima i CIRIA preporukama za područje Bliskog Istoka u zavisnosti od sadržaja agrevnih materija i značaja konstrukcije (bitumensko - epoksidni premazi, plasticne folije, samolepljive 1.5 mm bitumenske membrane kao i 4 mm vodonepropusna bitumenska membrana). Armatura je rebrasta sa minimalnom granicom razvlačenja 460 MPa.



Slika – 3

Gojko Grbić¹

PROJEKAT SANACIJE ČELIČNE KONSTRUKCIJE MOSTA "KRALJ ALEKSANDAR I KARADORĐEVIĆ" U ZVORNIKU

Rezime:

S-72

U radu se daje prikaz Glavnog i izvođačkog projekta sanacije čelične konstrukcije i kolovozne konstrukcije drumskog mosta Kralj Aleksandar I Karađorđević na Drini u Zvorniku. Konstrukcija mosta je prostorna čelična rešetka raspona 64+80+64 m. Štapovi rešetke su složenog poprečnog preseka od valjanih profila, sve veze na mostu su izvedene zakivcima. Statička analiza za opterećenje od šeme V300 je pokazala da čelična konstrukcija mosta kao i stubovi imaju potrebnu nosivost. Projektom sanacije je predviđena temeljna antikorozivna zaštita čelične konstrukcije mosta i zamena kompletnog postojeće kolovoza sa novim kolovozom od montažnih armirano betonskih ploča.

Ključne reči: most, sanacija, antikorozivna zaštita, montažne armirano betonske ploče

DESIGN OF REHABILITATION OF STEEL BRIDGE "KING ALEXANDER I KARADJORDJEVIC" IN ZVORNIK

The paper gives an overview of the Main and detailed design of the rehabilitation steel structure and pavement of the road bridge King Aleksandar I Karadjordjevic on the Drina in Zvornik. Construction of the bridge is a spacious steel grate range 64 + 80 + 64 m. Truss members are complex cross-section of the rolled profiles, all the connections to the bridge were carried out rivets. Static analysis for the load of the scheme V300 showed that the steel structure of the bridge and pillars have the necessary capacity. The rehabilitation project predicts basic corrosion protection of steel structure of the bridge and the complete replacement of the existing road with a new pavement of prefabricated reinforced concrete slabs.

Keywords: bridge, rehabilitation, corrosion protection, prefabricated reinforced concrete slab

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1 UVOD

Most „Kralja Aleksandra I Karađorđevića“ (slika 1) se nalazi na reci Drini u Zvorniku. Most je projektovan i građen u periodu od 1926 do 1930 godine. Stubove, prilaze mostu i kolovoz su izradile firme iz zemlje a čeličnu konstrukciju mosta je isporučila Nemačka, prema mirovnom ugovoru nakon I svetkog rata, kao ratnu reparaciju. Konstrukcija mosta je izrađena u Šćećinu u Poljskoj. Most je pušten u saobraćaj 1930 godine. Most je projektovan kao drumski i do pre 20 godina odvijao se drumski saobraćaj.

Most je prostorna čelično rešetkasta konstrukcija na tri polja. Rasponi su 64,0m+80,0m+64,0m ukupni raspon je $L=208,0m$ (slika 1). Statički sistem rešetkaste konstrukcije na tri polja je Gerberov nosač sa prosotm gredom u srednjem polju raspona 48 m i konzolnim prepustima od 16 m rešetki iz krajnjih polja. Oslonci rešetkaste konstrukcije na srednjim stubovima su fiksni dok su oslonci na obalnim stubovima klizni.



Slika 1

U toku izrade projektne dokumentacije pronašao sam deo tehničke dokumentacije mosta u arhivu Jugoslavije u Beogradu i to: projekte stubova mosta i ostalu gradilišnu dokumentaciju vezanu za

U međuvremenu nakon formiranja granice ovaj most je pretvoren u pešački granični prelaz između Republike Srbije i Republike Srpske. Zahtev Investitora je da se konstrukcija mosta i novi kolovoz proračunaju i na saobraćajno opterećenje V300, zbog toga što je moguć ograničen drumski saobraćaj preko mosta u vanrednim situacijama. Prema značaju puta na kojem se mosta nalazi on spada u III kategoriju – mostovi na svim ostalim putevima. U toku izrade projektne dokumentacije konstrukcija je snimana na licu mesta sa raznim metodama (merenjem, fotografisanjem, uzimanjem uzoraka za ispitivanje kvaliteta čelika ...).

2 STANJE MOSTA PRE SANACIJE

Kolovozna konstrukcija je sa jasno definisanim kolovozom i trotoarima. Kolovoz je širok 5,35 m a trotoari sa obe strane kolovoza su široki 80 cm. Svetli saobraćajni profil mosta je širine 6,95 m i visine 6,80 m.

Kolovoz je od granitne kocke a trotoari su od a.b. ploča presvučenih asvaltom. Odvodnjavanje kolovoza je preko slivnika i kratkih vertikala ispod kolovoza direktno u reku.

Na mostu je postavljena ulična rasveta. Preko mosta prelaze instalacije osvetljenja, telefonske veze graničnih prelaza. Konstrukcija mosta je neoštećena. Most je star preko 80 godina i zbog nemara ista nije antikorozivno šticećena u ovom periodu. Na kolovozu mosta su se pojavile rupe koje predstavljaju opasnost za bezbedno korišćenje mosta.

3 STATIČKI KONTROLNI PRORAČUN

U statičkoj analizi konstrukcije proračunati su uticaje za sledeće opterećenje:

IZVEDENA KONSTRUKCIJA

- Sopstvena težina
- Težina kolovozne konstrukcije
- Pokretno opterećenje – pešački most

Za ova opterećenja i njihovu kombinaciju izvršen je statički proračun uticaja u konstrukciji, koji će služiti za poređenje sa uticajima i opterećenjima sanirane konstrukcije.

SANIRANA KONSTRUKCIJA

- Sopstvena težina
- Težina kolovozne konstrukcije
- Pokretno opterećenje – pešački most
- Pokretno opterećenje za drumske mostove – šema V300
- Uticaj vetra za opterećen i ne opterećen most
- Uticaj temperature
- Uticaj sile kočenja

Na pobrojana opterećenja i njihove kombinacije urađen je statički proračun uticaja u elementima konstrukcije mosta i na osnovu statičkih uticaja izvršena je kontrola napona u konstrukciji.

Da bi se mogli prekontrolisati naponi u elementima konstrukcije mosta sa mosta su isečeni uzorci i ispitani u INSTITUTU ZA ISPITIVANJE MATERIJALA (IMS) Republike Srbije u Beogradu, dobijeni rezultati su pokazali da se može uzeti da je kvalitet čelika korišćen za izradu konstrukcije S275J2G3 (Č0453).

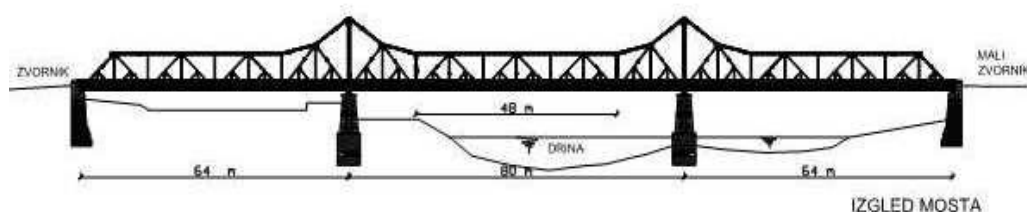
4 REZULTATI STATIČKOG PRORAČUNA I ANALIZA SU SLEDEĆI

Novoprojektovana kolovozna konstrukcija je 10% lakša od postojeće kolovozne konstrukcije pa su uticaji u konstrukciji za pešački most manji od postojećih pre sanacije. Statički proračun konstrukcije za pokretno opterećenje šema V300 je izvršen za varijantu da se vozilo kreće pored ivičnjaka i za istovremeni uticaj vetra. Izvršena je računaska kontrola napona i veza u čvorovima rešetke i proračun je pokazao da su i za ovu vrstu i kombinacije opterećenja naponi u elementima konstrukcije i vezama u dozvoljenim granicama. Kolovozna ploča je takođe proračunata i dimenzionisana na kombinacije opterećenja od šeme V300.

Uticaji na osloncima konstrukcije mosta su aplicirani na stubove i proračunati su naponi u temeljnim spojnica . Račun je pokazao da se za moguće kombinacije opterećenja u temeljnim spojnica ne pojavljuje zatezanje, dakle naponi su jednoznačni i to naponi pritiska. S obzirom da je most u periodu od 60 godina po izgradnji korišćen kao drumski most kao i u vremenu ratnih godina kompletna konstrukcija mosta je izdržala sva moguća opterećenja.

5 SANACIJA MOSTA

Most je izveden kao prostorna rešetkasta konstrukcija (slika 2). Rasponi mosta su 64+80+64 m. Most je postavljen na dva obalna i dva srednja – rečna stuba. Stubovi – donji deo u tlu su izrađeni od nearmiranog betona dok je gornji deo izrađen takođe od betona koji je obložen tesanim kamenom i u toku oblaganja – zidanja unutrašnji deo je ispunjavan nearmiranim betonom. Most je izveden sa dva pokretna ležišta na obalnim stubovima i dva nepokretna na srednjim stubovima. Srednji oslonci su nepokretni za translatorna pomeranja ali imaju ne sprečena obrtanja oni su “sferni” nepokretni oslonci. Pokretna ležišta su valjkasta i imaju omogućena pomeranja samo u pravcu ose mosta.

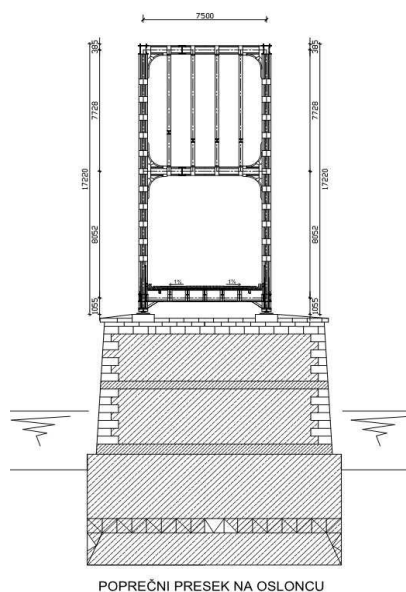


Slika 2

Čelična rešetkasta konstrukcija mosta je izrađena od profila i limova koji su formirali različite preseke zakivanjem i na licu mesta sve čvorne veze odrađene su zakivcima. Složenost pojedinih preseka štapova rešetke zahtevalo je dosta složen postupak izrade zakivanjem u radionici.

Rešetkasta konstrukcija mosta je izvedena u statičkom sistemu sa Gerberovim nosačem u srednjem polju. Čelična rešetka sa obalnog stuba preko srednjeg stuba formira gredu sa prepustom raspona u polju 64 m i prepustom od 16 m. Slika u ogledalu je i druga rešetka. Gerberov rešetkasti nosač je raspona 48 m u sredini i on je zglobnom vezom naslonjen-povezan sa rešetkastim gredama sa prepustom sa leve i desne obale.

Glavne rešetke su koncipirane sa donjim i gornjim pojasom povezani vertikalama i dijagonalama koje se pružaju u cik-cak rasporedu od čvora do čvora. Osnovni raspon polja je 8,00 m kao i visina rešetke s tim što je rešetka nad osloncima visine 16,70 m, pre oslonca dva polja i posle oslonca dva polja do “gerberovog” zgloba gornji pojas menja visinu sa 8 na 16,7 m (slika 3).

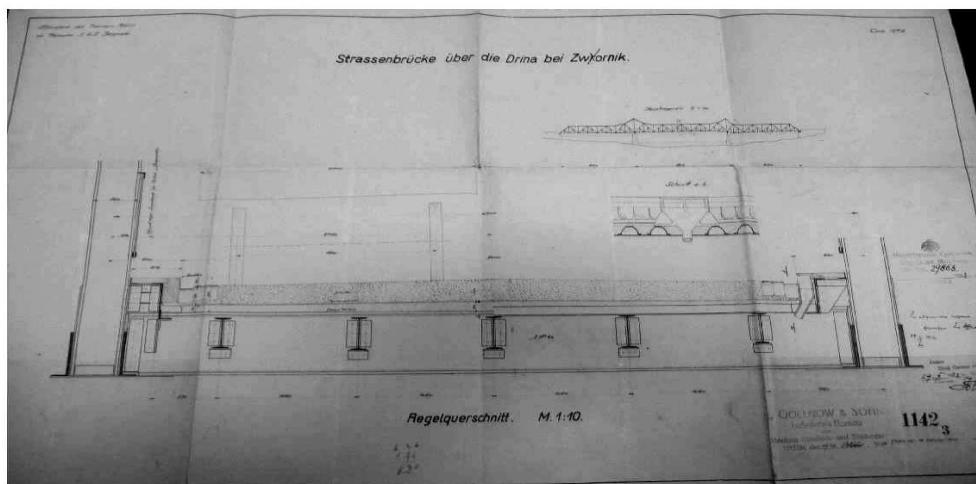


Slika 3

Kompletna rešetkasta konstrukcija zbog raspona polja od 8 m ima sekundarnu ispunu od vertikale i dijagonale tako da se u donjem pojasu raspon sa 8 m smanjuje na 4 m, ovim je

omogućeno da nosači kolovozne konstrukcije imaju raster od 4 m. Raspon polja u gornjem pojasu rešetke je ostao 8 m.

Noseća konstrukcija kolovoza (slika 4) je koncipirana tako da na raster od 4m su postavljeni poprečni nosači kolovozne konstrukcije koji su vezani za rešetke. Između poprečnih nosača postavljeni su podužni nosači kolovozne konstrukcije. Poprečno u odnosu na osu mosta postavljeni su "zores" profili na rastojanju od 40 mm a preko njih je postavljen kolovoz (drobljeni kamen + sloj mršavog betona + pesak + granitna kocka). U ravni donje ivice poprečnih nosača i donjih pojaseva rešetke postavljeni su spregovi za ukrućenje. Na slici 3. je original crtež na kojem se vidi poprečni presek mosta sa nosećom čeličnom konstrukcijom i kolovozom.



Slika 4

5.1 METALNA KONSTRUKCIJA MOSTA

Konstrukcija je zadržala svoju geometriju, na mostu nisu uočene deformacije noseće konstrukcije. Stanje konstrukcije iznad kolovoza je dobro sem što je korozija i dugogodišnje neodržavanje dovelo do toga da je sa velikog dela površine spao završni premaz i ostao vidljiv osnovni premaz a na dosta konstruktivnih delova mosta je korozija već osvojila površinu. Gornja strana gornjeg pojasa rešetke i spregova u ravni gornjeg pojasa su bez zaštite, korodirali po celoj površini.

Podužni nosači kolovoza kao i poprečni su mestimično nagriženi korozijom ali je generalno očuvan presek sa malo uticaja korozije. Spregovi u donjoj zoni noseće kolovozne konstrukcije i donjeg pojasa rešetke su delimično oštećeni korozijom. Čvorovi i čvorni limovi kao i zakivci u vezama su korodirali manje ili više ali još uvek u funkciji.

Spojna sredstva – zakivci u svim spojevima kako u složenom preseku štapa tako i u čvornim vezama stoje, bez ijednog zakivka da nedostaje ili da se pomera u vezi.

Projektom sanacije je konstatovano da nema potrebe za popravkom niti sanacijom čelične konstrukcije mosta osim antikorozijske zaštite. Most spada u inženjerske konstrukcije koje su

jako izložene uticaju korozije pa je antikorozivna zaštita predviđena projektom saglasna važećim propisima.

Prema tabeli A.4.- SRPS ISO 12944-5:2002 prelažem, sistem boja antikorozivne zaštite za kategoriju korozivnosti 4 i redosled pripreme i nanošenja boja:

- **priprema**, priprema površine (peskarenje) stepen Sa 2^{1/2}
- **osnovna prevlaka** DVOKOMPONENTNI EPOKSI OSNOVNI PREMAZ SA CINKOM , debljina suvog filma 1x40-60µm, nanošenje – vazdušno špricanje, četka , valjak.
- **međusloj**, DVOKOMPONENTNI EPOKSI OSNOVNI PREMAZ (DBS), debljina suvog filma 2x60µm, nanošenje – vazdušno špricanje, četka, valjak.
- **završni**, DVOKOMPONENTNI POLIURETANSKI PREMAZ, debljina suvog filma 1x40-50µm,

UKUPNO PREMAZI – DEBLJINA SUVOG FILMA min 200 µm.

PREMA SRPS ISO 12944, ovaj način zaštite čelične konstrukcije obezbeđuje po klasifikaciji C4 DUG vek trajanja zaštite preko 15 godina.

5.2 POSTOJEĆA KOLOVOZNA KONSTRUKCIJA

Na podužne nosače kolovoza postavljeni su popreko “zores” profili (slika 5) na razmaku tako da između nožica istih ostaje razmak od 40 mm. Ovi profili formiraju sekundarnu noseću konstrukciju kolovoza. Preko “zores” profila postavljen je drobljeni kamen ukupne debljine do 10 cm a preko kamena je sloj mršavog betona debljine do 5 cm kao izravnavajući sloj.



Slika 5

Na izravnavajućem sloju postavljen je granitna kocka u pesku u sloju debljine do 10 cm. Izvedena je posebna konstrukcija u sklopu kolovoza koja formira trotoare izdignute 15 cm u donosu na kolovoz.

Kolovozna konstrukcija trotoara su a.b. ploče debljine 10 cm sa slojem livenog asvalta od 2 cm. Kolovoz je u dvostranom nagibu poprečnom prema trotoarima i dvostranom podužnom nagibu od sredine mosta. Noseća konstrukcija kolovoza je vidljiva sa donje strane mosta koja direktno iznad reke je u dosta lošem stanju. Kolovozna table - “zores” profili su na slobodnim ivicama jako korodirali i vidljiv je drobljeni kameni agregat iznad profila.

Odvodnjavanje je rešeno sa slivnicima koji se nalaze sa obe strane kolovoza uz trotoare. Na mostu je postavljena zaštitna ograda koja je okačena na rešetke. Sve veze na mostu su izvedene zakivcima sem mesta gde je "gerberov" zglobov gde su veze na zavrtanj.

Granitna kocka je jako neravna iz razloga što je pesak u podlozi ispran pa se ista slegala i uvrtala a godinama nije popravljena. Na pojedinim mestima su se otvorile rupe u kolovozu kroz koje vidi reka. Razlog je što je zbog dotrajalosti kolovozne table "zores" profila koje je na tim mestima "pojela" korozija, propao drobljeni kamen, beton za izravnanje a za njim pesak i kocka (slika 6).



Slika 6

Ploče na trotoaru na nekim mestima su polomljene na nekim zamenjen a na nekim nedostaje asfalt. Na nekim mestima nedostaju metalni profile na ivičnjacima kao i sami ivičnjaci. Klizne prelaznice su u funkciji.

Projektom sanacije je predviđeno kompletno uklanjanje postojećeg kolovoza zajedno sa "zores profilima.

5.3 NOVA KONSTRUKCIJA KOLOVOZA

Nova kolovozna konstrukcija su montažne armirano betonske ploče (slika 7) kompletne širine saobraćajnog profila i dužine u pravcu ose mosta ~2m, debljina ploča je usvojena 18 cm. Montažna ploča je predviđena sa ivičnjakom.

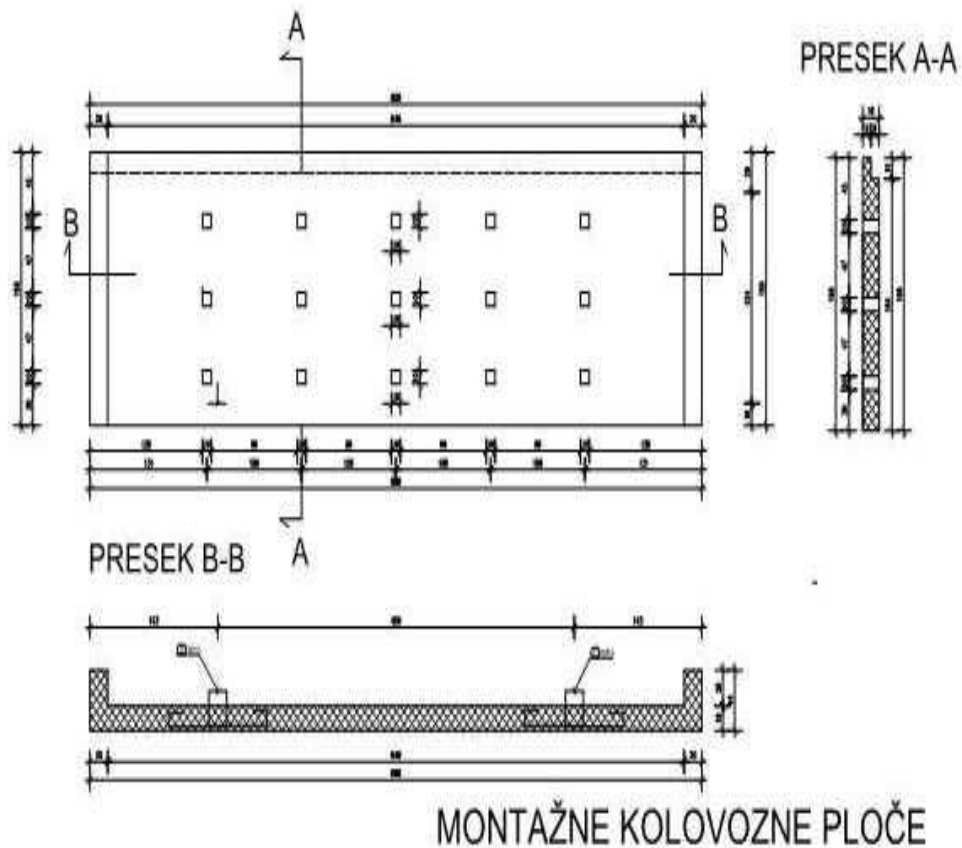
Oslanjanje montažnih ploča je predviđeno na podužne sekundarne nosače kolovoza (I 300 i I 340).

Na sekundarnim nosačima su zavareni moždanici u rasporedu kakav je raspored rupa na montažnoj kolovoznoj ploči. Nakon montiranja postavljajući ploče, rupe sa moždanicima se

zalivaju sitnozrnim betonom. Dimenzije moždanika su sračunate da mogu da prime samo silu kočenja na mostu.

Na mestu gde je “gerberov” zglob i na obalnim stubovima gde se završava i počinje mostovski kolovoz predviđene su vodonepropusne dilatacione prelaznice tip MA 50. Tip dilatacione prelaznice je usvojen prema proračunu dilatacija mosta za temperaturnu razliku predviđenu propisima.

Završna obrada montažnih ploča sa gornje strane je predviđena izrada hidroizolacije kao i izrada habajućeg sloja od tvrdo livenog asvalta min debljine 5 cm i debljine u sredini kolovoza 12 cm, čime je obezbeđen dvostrani nagib kolovoza od 2,5% i time odvodnjavanje atmosferske vode. Kompletna konstrukcija mosta je i u dvostranom podužnom nagibu od 1% od sredine ka obalama. Na ivičnjake kolovoza predviđena je ugradnja okapnice od lima $t=5$ mm, zavarena za konstrukciju mosta čime je osigurana dodatna bezbednost uz ogradu a i ocedjivanje vode sa ivičnjaka na kolovoz a ne na konstrukciju mosta.



Slika 7

Skidanje postojeće kolovozne konstrukcije je predviđeno u jednom pravcu a montaža kolovoznih ploča u suprotnom pravcu. Montažu kolovoznih ploča izvodi laka auto dizalica

koja ih montira i namontirane ploče koristi za kretanje i montiranje sledećih ploča. Ovakav način izrade i montaže kolovoznih ploča, pojednostavljuje i skraćuje rok sanacije mosta. Projektom sanacije je predviđena zamena kompletne kolovozne konstrukcije.

5.4 STUBOVI I LEŽIŠTA

Stubovi mosta su kontrolisani geodetski i na istim nema rezultata nejednakih sleganja niti rotacija. Vizuelno gledano stubovi su u dobrom stanju sem na srednjem stubu pri vrhu treba kameni ozid malo presložiti i zaliti cementnim malterom.

Klizna ležišta na obalnim stubovima obavljaju svoju funkciju al i na njima su vidljivi tragovi korozije. Nepomerljiva – fiksna ležišta su u dobrom stanju, obavljaju svoju funkciju i na njima ima tragova korozije.

5.5 ZAŠTITNA OGRADA

Ograda je na nekim mestima iskrivljena i delimično nedostaje verovatno još od vremena kad su preko mosta prelazila i vozila.

Projektom je predviđena popravka oštećenih delova i farbanje.

6 ODVODNJAVANJE

Odvodnjavanje atmosverske vode je rešen preko slivnika na kolovoznoj ploči i kišnom kanalizacijom ispod kolovozne ploče mosta. Most je po dužini u dvostranom nagibu, pa je i kanalizacija sa dva kraka jedan vodi u ulična kanalizaciju na levoj obali a drugi u uličnu kanalizaciju na desnoj obali reke Drine.

7 ZAKLJUČAK

Glavni projekat sanacije mosta „Kralja Aleksandra I Karađorđevića“ u Zvorniku je urađen u projektom birou "PIN" doo Loznica. projekat je obuhvatio prethodne preglede, snimanja i ispitivanja konstrukcije, statičke kontrolne proračune postojeće konstrukcije i opredeljenje načina sanacije, glavni i izvođački projekat sanacije, projekte organizacije i tehnologije izvođenja radova, priloge za izvršenje redovnih radova, odnosno redovnog održavanja konstrukcije mosta i projekte odvodnjavanja, kako je bilo i definisano projektom zadatkom.



*Zsolt Kokrehel*¹

NOSIVOST I TRAJNOST OFORMLJENIH DILATACIONIH SPOJEVA KOD INDUSTRIJSKIH PODOVA

Rezime:

S-73

Kod projektovanja industrijskih podova vrlo mala pažnja se obraća na nosivost i trajnost dilatacije. Uglavnom se prepušta odluka izvođaču podne konstrukcije koji će sistem da primeni. Naknadni trošak primene savremenih dilatacionih profila kod izgradnje industrijskih podova vrlo se teško opravda kod investitora. Izvođač se dovodi u situaciju gde mora da garantuje kvalitet betona, ravnost i trajnost industrijskog poda a bez odgovarajućeg rešenja je praktično nemoguće, naročito kod intenzivnog teškog saobraćajnog opterećenja.

Ključne reči: industrijski pod, čelični dilatacioni profil, trajnost, nosivost

FORMED EXPANSION JOINTS STRENGTH AND DURABILITY IN INDUSTRIAL FLOORING

Summary:

In design stage very little attention is paid to the capacity and durability of industrial floors expansion joints. Basically the decision which system will be used is left to contractor of the floor construction. To justify with investor this additional expense of application of modern expansion profile in the construction of industrial floor is very difficult. The contractor is brought into a situation where he has to guarantee the quality of concrete, flatness and durability of industrial floor without an appropriate solution is practically impossible, especially in case of intensive heavy traffic load.

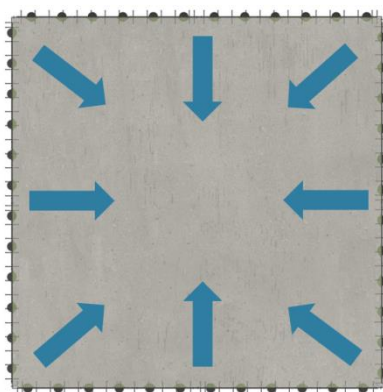
Key words: industrial floor, steel joint profile, durability, capacity

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1 DILATACIJE KOD INDUSTRIJSKIH PODOVA

Dilatacije u industrijskim podovima se ne mogu izbeći. Broj, raspored i tip dilatacije zavisi od načina konstruisanja podne ploče. Razlikujemo radne dilatacije i dilatacije za sprečavanje ili kontrolisanje pojave nepoželjnih prlina - pukotina.

Nakon što je jedan segment podne ploče izbetonirana dolazi do procesa vezivanja betona. Ovaj proces izaziva zagrevanje betona i skupljanje betona. Skupljanje betona se vrši skraćivanjem betona od ivica prema sredini površine (slika 1).



Slika 1 – Pravci skraćivanja sveže izlivenne betonske ploče pri skupljanju usled sušenja

Pukotine se pojavljuju u momentu kad naponi zatezanja usled skupljanja prekorače zateznu čvrstoću betona. Da bi se ovi naponi reducirali na što manju moguću veličinu potrebno je obezbediti slobodno klizanje betonske ploče na nosivoj podlozi primenom dvostruke PE folije ispod betonske ploče i pravilnim projektovanjem rasporeda radnih i sečenih dilatacija.

Raspored dilatacija treba da bude takav da odnos stranica jednog polja bude 1:1-1:1.5.

Kod planiranja dilatacija industrijskih podova uticaj na raspored u osnovi imaju sledeći faktori:

- raspored konstrukcije – stubova, zidova, šaftova itd.,
- primenjena tehnologija izvođenja
 - metoda dugačkih traka
 - metoda velikih površina (sa ili bez dilatacija)
 - metoda naknadnog prednapreznja
 - metoda u dva sloja

1.1. TIPOVI DILATACIJA

Dilatacije se mogu po tipu podelite na osnovu pomeranja koji dozvoljavaju i na osnovu metode kako su kreirani.

1. Dilatacije koje omogućuju slobodno pomeranje ploče
 - naknadno sečene dilatacije
 - oformljene – radne dilatacije

2. Dilatacije koje sprečavaju slobodno pomeranje ploče
 - naknadno sečene dilatacije
 - oformljene – radne dilatacije
3. Povezane dilatacije
4. Dilatacije koje omogućavaju širenje

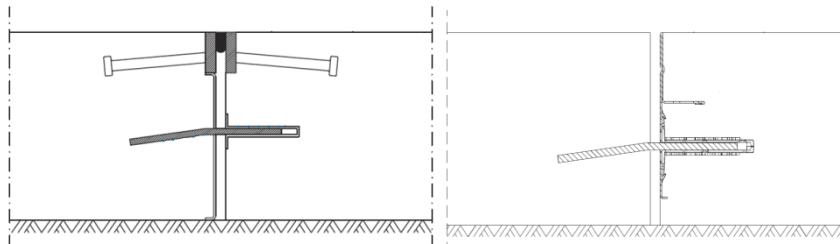
2 OFORMLJENE RADNE DILATACIJE

Oformljene radne, tehnološke dilatacije u praksi se najčešće nazivaju prekidima betoniranja i moraju se primeniti iz tehnoloških razloga mogućnosti betoniranja u jednoj celini.

Dilatacije koje omogućuju slobodno pomeranje ploče se oformljuju na obimu jedne celine i po pravilu postavljaju se na maksimalnim rastojanjima od 35mx35m (cca. 1200m²) sa otvaranjem radne fuge do 20mm i kod specijalnih tehnologija izvođenja i do 50mx50m (2500m²) sa otvaranjem radne fuge većem od 20mm. Za sprečavanje pojave pukotina primenjuje se po principu čelična vlaknasta armatura (20-30kg/m³) uz mogućnost kombinacije sa klasičnom mrežastom armaturom. Umesto čeličnih vlakna moguće je primeniti i sintetička vlakna na bazi polipropilena.

Radna dilatacija se formira postavljanjem oplata. Oplata može da bude:

1. po tipu materijala
 - čelična oplata – najčešće se primenjuje
 - drvena oplata – slabo se primenjuje i kod nezahtevnih podova
2. po broju mogućnosti korišćenja
 - oplata za višekratnu upotrebu
 - oplata za jednokratnu upotrebu - zarobljena
3. po tipu zaštite ivice betona na dilataciji
 - bez zaštite ivice betona
 - sa zaštitom ivice betona



Slika 2a i 2b – Čelična zarobljena oplata sa zaštitom ivice beton i bez zaštite ivice

Na tržištu se može naći više dobavljača sličnih proizvoda sa različitim rešenjima. Na obe slike je prikazana čelična zarobljena oplata.

Slika 2a prikazuje integrisanu čeličnu opatu pod komercijanim nazivom TERAJOINT proizvođača PEIKKO. Čelična oplata se sastoji od:

- vertikalnog čeličnog lima - oplata
- pravougaoni čelični profili – zaštita ivice betona
- moždanici u sredini visine profila – prenos opterećenja

Slika 2b prikazuje integrisanu čeličnu opatu pod komercijanim nazivom UNIFORM proizvođača PEIKKO. Ova oplata nema sistem za saštitu ivice betona.

3 SISTEM PRENOSA OPTEREĆENJA NA DILATACIJI

Podna ploča je po pravilu opterećena statičkim opterećenjem i dinamičkim opterećenjem.

Pod statičkim opterećenjem podrazumevamo površinsko opterećenje i koncentrisano opterećenje. Koncentrisano statičko opterećenje je uglavnom od nogara regala skladišta koji se prenose preko čelone ploče relativno male površine – 100x100mm. Ova opterećenja kod visokoregalnog skladišta mogu dostići i vrednost od 120kN po jednoj nozi regala, ekstremno i do 200kN.

Dinamička opterećenja se javljaju po pravilu od viljuškara za manipulaciju robe u objektima. Prema EN 1991.1.1:2002 viljuškari se klasifikuju od FL1 do FL6 sa osovinskim opterećenjem $Q_k = 26 - 170$ kN.

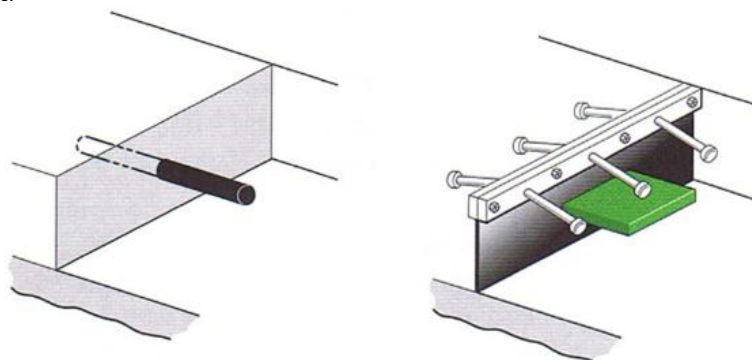
Gore navedena opterećenja na radnim dilatacijama se moraju preneti sa jednog polja na susedno polje. Potrebno je obezbediti:

- siguran prenos opterećenja moždanicima
- da se minimizuje vertikalna pomeranje dva susedna povezan polja
- da se minimizuje zakrivljenost površine (izbočenost)



Slika 3 – Ponašanje spoja podne ploče pri opterećenju – bez i sa moždanicom

Prenos opterećenja se može obezbediti ugradnjom čeličnih šipki kružnog ili pravougaonog poprečnog preseka ili čeličnih ploča određene debljine kružnog ili pravougaonog oblika. Ovi elementi prenosa opterećenja se ugrađuju u sredini debljine podne ploče pi čemu jedna strana je ubetonirana u ploču dok se strana u susednoj ploči slobodno kliže u odgovarajućoj uglavnom plastičnoj čauri.



Slika 4 – Sistemi za prenos opterećenja – čelične šipke u drvenoj oplati i integrisani sistem

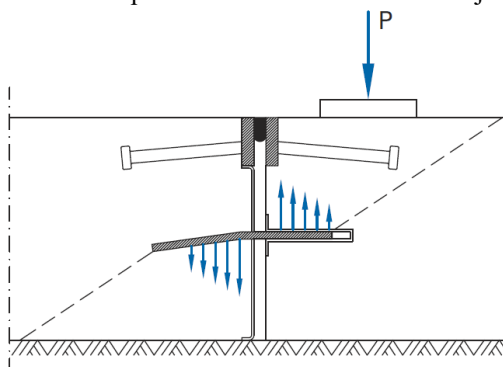
4 NOSIVOST MOŽDANIKA U PODNIM PLOČAMA

Nosivost moždanika je u funkciji:

- kvaliteta materijala i dimenzija poprečnog preseka moždanika
- debljine i kvaliteta betonske podne ploče
- veličine otvora, zazora između dve susedne ploče

Moždanici i podna ploča opterećeni su na:

- smicanje u moždaniku – zavisi od delujuće vertikalne sile i kvaliteta podloge
- savijanje u moždaniku – zavisi od delujuće vertikalne sile i širine otvora dilatacije
- probojna otpornost betonske ploče – zavisi od kvaliteta i debljine podne ploče



Slika 5 – Prenos opterećenja pločastog moždanika usled opterećenja na ploči

Nosivost moždanika se proračunava na osnovu Technical Report 34 četvrto izdanje publikovano u Avgustu 2013 od strane Društva za beton engleske.

Nosivost moždanika na smicanje prema jednačini 9.23:

$$P_{sh} = 0.6 f_y A_v / \gamma_s$$

Nosivost moždanika prema jednačini 9.24:

$$P_{bear} = 0.5 f_{cu} b_l d_d / \gamma_c$$

Nosivost moždanika na savijanje prema jednačini 9.25:

$$P_{bend} = 2 f_y Z_p / \gamma_s$$

Nosivost betona na proboj od moždanika:

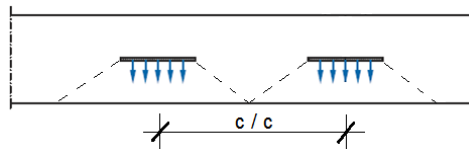
$$P_p = 0.75 v_{Rd,c} u_2 d$$

Tabela 1 - Uporedni prikaz nosivosti čeličnih pločastih moždanika iz kataloga proizvođača PEIKKO – debljina podne ploče 200mm

Tip moždanika	Otvor dilatacije x [mm]	Nosivost na smicanje P_{sh} [kN]	Nosivost na savijanje $P_{bear,bend}$ [kN]	Nosivost na proboj C30/37 P_p [kN]
TDC 6	15	145.0	41.4	24.7
UDR 8	20	381.3	93.8	27.2

Nosivost moždanika je proračunat za nearmirani beton i na osnovu preporuka TR 34.4 i ona se koristi i u slučaju betona armiran sa vlaknastom armaturom (čeličnim ili sintetičkim).

Merodavna nosivost moždanika je uvek probojna nosivost. probojna nosivost je proračunata za osno rastojanje moždanika od 500mm. Manji međusobni razmak prouzrukuje preklapanje površina proboja i smanjuje se nosivost.



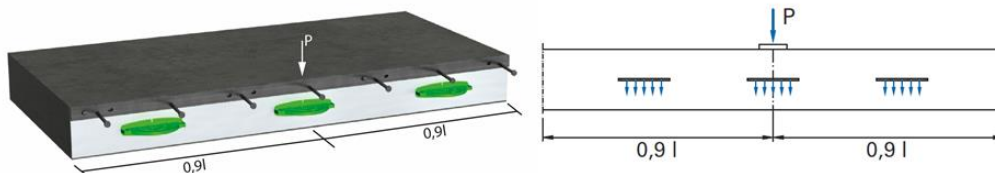
Slika 6 – Šematski prikaz probojne površine moždanika

5 NOSIVOST DILATACIONOG SPOJA

Usled opterećenja koja deluje na podnoj ploči dolazi do deformacije podne ploče i aktivacije niza susednih moždanika. Određivanje broja moždanika koji se mogu uračunati u nosivost kod dokaza nosivosti zavisi od radiusa relativne krutosti (l) koja je zavisna od Westergaard-ovog koeficijenta posteljice (k), debljine podne ploče (h) i kvaliteta betona (E_{cm}). Prema tački 9.4.6. TR 43.4 jednačina 9.4 za radius relativne krutosti glasi:

$$l = \left(\frac{E_{cm} h^3}{12(1 - \nu^2)k} \right)^{0.25} \quad (1)$$

Ispod koncentrisnog opterećenja koja deluje na dilataciju sa leve i desne strane mogu se uzeti u obzir moždanici koji su na rastojanju od 0.9 radiusa relativne krutosti.

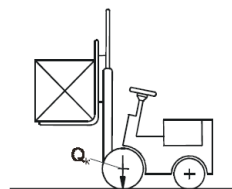


Slika 7 – Dužina sudelovanja moždanika ispod koncentrisane sile

Tabela 2 – Radius rel. krutosti u zavisnosti od kvaliteta podloge i debljine podne ploče – za kvalitet betona $f_{cu} = 40\text{N/mm}^2$ prema TR34

Debljina podne ploče [mm]	Vrednosti l [mm] za $k=0.02$ do 0.10			
	0.02	0.04	0.07	0.10
150	834	701	610	558
200	1035	870	756	692
250	1223	1029	894	818
300	1402	1179	1025	938

Class of Forklift	Net weight [kN]	Hoisting load [kN]	Width of axle a [m]	Overall width b [m]	Overall length l [m]
FL 1	21	10	0.85	1.00	2.60
FL 2	31	15	0.95	1.10	3.00
FL 3	44	25	1.00	1.20	3.30
FL 4	60	40	1.20	1.40	4.00
FL 5	90	60	1.50	1.90	4.60
FL 6	110	80	1.80	2.30	5.10



Slika 8 – Standardni viljuškari prema EN 1991-1-1:2002, tačka 6.3.2.3.

Primeru radi ako usvojimo standardizovani viljuškar nosivosti 4.0 tone, tipa FL4 prema standardu EN 1991-1-1:2002, tačka 6.3.2.3. osovinsko opterećenje od viljuškara je $Q_k = 90\text{kN}$. Razmak točkova je $a = 1.2\text{m}$. Projektujemo podnu ploču debljine 200mm sa radijusom relativne krutosti iz gornje tabele 756mm za $k=0.07\text{ N/mm}^3$. Sa ovim ulaznim parametrima možemo da uzmemo u obzir moždanike na dužini od $2x1 + a = 2712\text{mm}$.

Moždanici su na međusobnom razmaku od $c/c = 500\text{mm}$ to znači da broj komada moždanika koji se može uzeti u obzir:

$$n = 1 / 500\text{mm} = 2712/500 = 5.42 \text{ komada} \quad \text{usvojeno 5 komada}$$

Nosivost jednog moždanika iz kataloga Peikko za TDC6, debljinu ploče 200mm i otvor dilatacije 15mm je $P_p = 24.7\text{kN}$. Ukupna nosivost moždanika na merodavnoj dužini:

$$P = n \times P_p = 123.50 \text{ kN}$$

Uzimanjem u obzir dinamičkog faktora $\phi=2.0$ za viljuškare sa tvrdim gumama:

$$0.5Q_{k,dyn} = 0.5 \times 2 \times 90 = 90 \text{ kN} < P = n \times P_p = 123.50 \text{ kN}$$

S obzirom da u prenosu opterećenja sudeluje i podloga, nasip ispod podne ploče i moždanicama se proverava samo 50% od ukupnog opterećenja možemo zaključiti da odabrana debljina podne ploče, zbijenost podloge i tip moždanika odgovara predviđenom saobraćajnom opterećenju od viljuškara.

6 TRAJNOST DILATACIONOG SPOJA

U slučaju nezaštićene ivice betona na dilataciji tokom eksploatacije usled saobraćajnog opterećenja dolazi do krnjenja, ljušćenja ivice betona koji pored neestetskog izgleda i utiče na komforost korišćenja objekta. Sanacija ovog problema je moguć ali vrlo često je rešenje male trajnosti i mora se ponoviti postupak sa širom zonom sanacije.



Slika 9 – Primeri ostećenja podne ploče na dilataciji - primer iz Srbije

Primenom savremene čelične oplatne tehnike industrijskog standarda sa integrisaim sistemom zaštite ivice betona i prenosa opterećenja mogu se izvesti podne konstrukcije visoke klase ravnosti do FM3 prema standardu TR34.4.



Slika 10 – Primeri instalacije oplatnog sistema i zvedene radne dilatacije na objektu Delhaize DC u Staroj Pazovi, 2014. god.

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Goran Tadić¹

PUTNE KONSTRUKCIJE: MOSTOVI, POTPORNİ ZIDOVI, STABILNOST KOSINA I ZAŠTITA PUTA OD PADA STENA

Rezime:

S-74

U 2009 godini krenula je izrada Glavnih projekata nedovršenih deonica autoputa na evropskom Koridoru 10 kroz Srbiju a do danas traje i gradnja preostalih. U okviru trase pojavljuju se različite vrste konstrukcija od kojih će biti razmatrani mostovi, potporne konstrukcije, konstrukcije za stabilizaciju kosina i konstrukcije za zaštitu puta od pada stena. Analiziraće se koncepcije konstrukcija iz projekata, njihova usklađenost sa realnim geološkim uslovima, uobičajenim tehnologijama gradnje različitih izvođača, zahtevima sigurnosti saobraćaja i zaštite prirodnog okruženja i savremenim propisima. Komentarišaće se predlozi izvođača za uštede i poboljšanja.

Ključne reči: Autoput, Koridor 10, konstrukcije, mostovi, propusti, kosine, stene

HIGHWAY STRUCTURES: BRIDGES, RETAINING WALLS, SLOPE STABILITY AND ROAD ROCKFALL PROTECTION

Summary:

In 2009 started the preparation of Final designs of unfinished highway sections on Serbian Corridor 10 and the construction of the remaining runs to date. The different types of construction appear as a part of the route of which the bridges, retaining walls, structures for slope stabilization and road rockfall protection will be reviewed. The design concept of structures and their compliance with the real geological conditions, usual work technologies from different contractors, the requirements of traffic safety and environment protection and modern standards will be analysed. The contractors' proposals for savings and improvements will be commented.

Key words: Motorway, Corridor 10, structures, bridges, culverts, slopes, rocks

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1 AUTOPUTNI MOSTOVI, POTPORNE KONSTRUKCIJE, ZAŠTITA KOSINA I ZAŠTITA OD PADA STENA

1.1 MOSTOVSKJE KONSTRUKCIJE

1.1.1 Saobraćajni profil i oprema na autoputnim mostovima

Primena sistema za zadržavanje vozila na putevima u skladu sa EN 1317 je kod nas počela sa uvođenjem odgovarajućih SRPS EN standarda 2008 godine, sa sada važećim verzijama iz 2013 godine. Kako su Idejni projekti mostova za preostale deonice autoputeva postojali i ranije, jedan deo projekatnata je nastavio da razvija Glavne projekte mostova prema njima a deo se preorijentisao na novu regulativu. Tako su se pojavila različita rešenja saobraćajnih profila na autoputnim mostovima.

Pojavila su se i Tehnička uputstva sa obaveznom primenom: Primena sistema za zadržavanje vozila na državnim putevima Republike Srbije (Javno preduzeće "Putevi Srbije", Beograd, 2010), Pravilnik o uslovima koje sa aspekta bezbednosti saobraćaja moraju da ispunjavaju putni objekti i drugi elementi javnog puta ("Službeni Glasnik republike Srbije" br. 50 od 8. jula 2011).

U starim mostovskim propisima visina ivičnjaka od 20 cm nad asfaltom u kombinaciji sa pešačkom ogradom na spoljnoj ivici radne (pešačke, biciklističke) staze se smatrala dovoljnom zaštitom od pada vozila sa mosta. Brojne nesreće sa izletanjem vozila (posebno teških) sa puteva i njihovog pada sa mostova ubrzale su razvoj sistema za zadržavanje vozila, potkrepljen eksperimentima i verifikovan donetim standardima. Pri velikim brzinama koje se razvijaju na autoputevima, ivičnjaci ove visine doveli su do odskoka vozila pri udaru u ivičnjak i do izletanja na radnu stazu ili preskakanja zaštitne ograde ukoliko se ona postavlja iza ovakvih ivičnjaka. Rešenje sa ivičnjakom visine 7 cm u kombinaciji sa licem metalne zaštitne ograde (elastične barijere H2-W4 type) postavljenim minimum 50 cm iza ivičnjaka pokazalo se kao kvalitetno rešenje koje je sada propisano za brzine veće od 50 km/h. Kako ova kombinacija zahteva dosta prostora za ugib elastične barijere pri udaru, kod užih mostova kao varijanta se uzima betonska zaštitna ograda (kruta barijera tipa „New Jersey“) kao prihvatljivo rešenje, s tim da su oštećenja vozila i povrede putnika ovde veće, posebno kod starijih putničkih vozila.

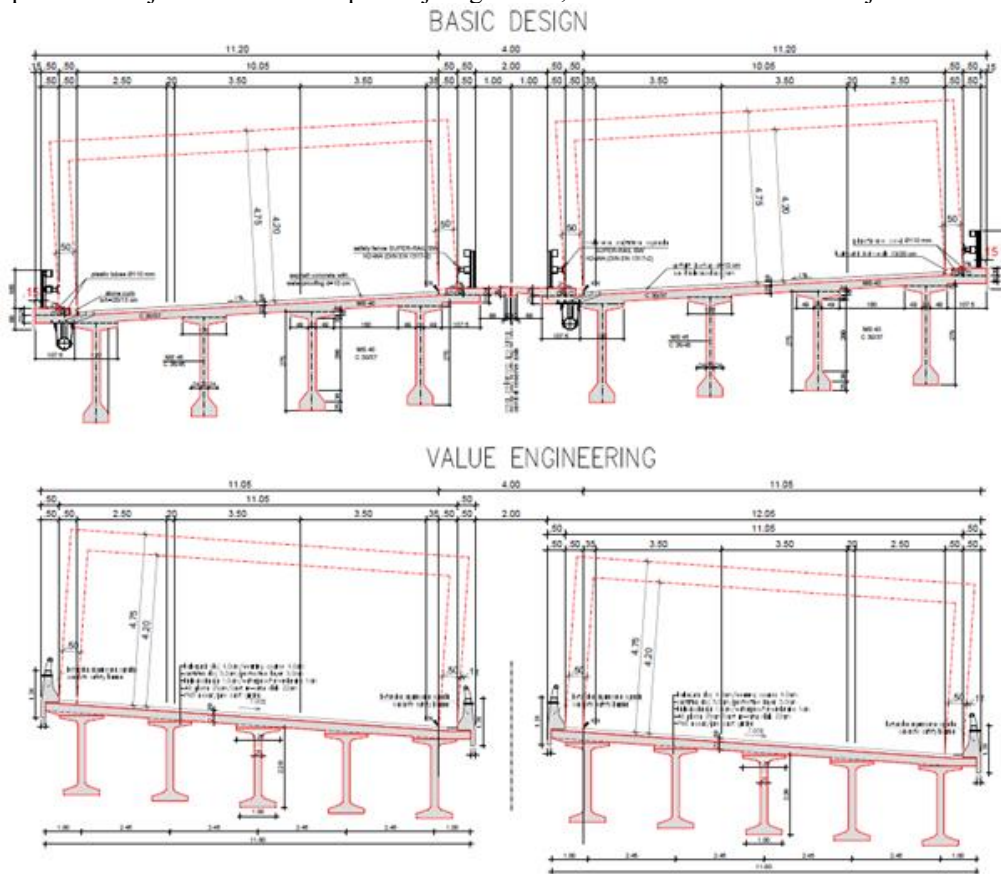
U ranije projektovanim deonicama pojavili su se projekti autoputnih mostova sa ivičnjacima visine 15 i 20 cm od kojih su neki i izvedeni. Predlozi za njihovo prilagođavanje važećim propisima nailaze na otpor izvođača zbog dodatnog vremena potrebnog za popravke.

Položaj i postojanje radnih staza je druga neusklađenost koja se pojavila u nekim projektima mostova. Ne može se smatrati radnom stazom prostor u razdelnom pojasu na mostovima, jer mu je nemoguće prići. Radne staze, ako se predviđaju, mogu biti smeštene samo na spoljnim ivicama mosta jer, uz odgovarajuću mehanizaciju, omogućavaju pristup i pregled donjih površina nosača i stubova ispod mosta.

Na autoputnim pravcima E75 i E80 u okruženju primenjivana su različiti poprečni preseci i putna geometrija na otvorenoj trasi i na mostovima. Bitno je da oni zadovolje standarde i preporučenu praksu Sveevropskog autoputnog projekta Sever-Jug – Trans-European North-South Motorway Project (TEM).

1.1.2 Tipovi mostovskih nosača predloženi za zamenu od strane nekih izvođača

Tipovi mostovskih prednapregnutih montažnih nosača koji su češće bili predmet predloga vrednosnog poboljšanja su bili T nosači (npr. težine po komadu 126 tona za raspon 45 m) kojih je u poprečnom preseku mosta za jedan autoputni pravac bilo po 4 (Slika 1). Njihovom zamenu sa 5 ili 6 lakših nosača (npr. 5 težine po 85 tona) Izvođač je pružio mogućnost uštede za sebe i Investitora podeljenu odnosom 50% i 50%. Ušteta se ostvarivala ne samo upotrebom lako raspoloživih auto dizalica umesto manje dostupne lansirne portalne rešetke, već i ukidanjem poprečnih nosača na trećinama raspona koje je trebalo liti na licu mesta, često na velikim visinama. Takođe je izbegnuto naknadno prednaprezanje monolitizovanog preseka, opet na velikoj visini. Količine upotrebljenog betona, kablova i armature su smanjene.



Slika 1 – Tipični mostovski poprečni presezi u Glavnom projektu^[3] i u izvođačkom VEP-u^[4]

1.1.3 Projekti vrednosnog poboljšanja skraćivanjem mostovskih konstrukcija

Neki izvođači su nudili skraćenje vijadukata i zamenu tih delova visokim nasipima kao vrednosno poboljšanje. Obilje mehanizacije za zemljane radove i višak kvalitetnog materijala iz iskopa za useke i tunele učinili su ovu vrstu radova jeftinom u odnosu na konstrukciju mosta.

Problemi sa deponovanjem ovog viška materijala nisu sagledani u fazi projektovanja i lokacija većine deponija u dolinama Južne Morave i Nišave je naknadno odobrena.

Ušteda skraćivanjem vijadukata i zamenom nasipima donosila je i probleme:

- Proširenje ekspropisanog pojasa (ponekad izbegnuto nasipima od armirane zemlje),
- Suženje rečnih dolina, često dragocenih pri ekstremno visokim vodama,
- Pregrađivanje dolina visokim nasipima najčešće nije ekološki prihvatljivo,
- Dodatno vreme za različita preprojektovanja, revizije i saglasnosti nadležnih institucija,
- Visoki nasipi na prirodnim kosinama se uglavnom deformišu kroz vreme^[1],
- Visoki nasipi izazivaju sleganja podtla koja zahtevaju dodatna geotehnička ispitivanja,
- Prvobitno predložena ušteda se, po pravilu, razradom Glavnog projekta značajno smanji.

Dolazilo je i do čudnih zahteva da se vijadukti zamene propustima i veoma visokim nasipima od armirane zemlje – pravim zemljanim branama koje ugrožavaju naseljena mesta u neposrednoj blizini i za koje nije trebalo mnogo razmišljanja da bi se odbili.

1.1.4 Projekti regulacija vodotokova u području mostova

Tradicionalna dugogodišnja praksa u projektovanju regulacija vodotokova uzvodno i nizvodno od mostova kao dominantne uslove je imala trajnost i mogućnost lakog održavanja. Tako su kod nas dominantna korita obložena poluobrađenim kamenom u cementnom malteru ili armiranim betonom. Poprečni betonski pragovi za umirivanje vodotokova su deo ovog rešenja. Ovakav pristup je dominantan u oblikovanju regulacija vodotokova na ukrštaju autoputeva E75 i E80 sa vodotoka i neka rešenja ne zadovoljavaju svetske ekološke standarde.

Savremen pristup oblikovanja vodotokova u Evropi teži približavanju prirodnim oblicima korita i gore navedeni načini oblaganja postali su nepoželjni a upotreba betona praktično zabranjena. Umesto toga koristi se krupan prirodno oblikovan ili lomljen kamen, a u slučaju prostornih ograničenja i gabioni. Betonski pragovi su počeli da se smatraju preprekama za kretanje riba pa se u slučaju neophodnosti upotrebe pragova koriste drveni, bar kod manjih vodotokova. Ispod otvora mostova i propusta potrebno je ostaviti staze dovoljnih gabarita paralelne vodotokovima za prolaze u suvom životinja koje nastanjuju ta područja (Slika 2).

Važno je da se i tokom gradnje ne ugrozi prirodni habitat biljaka i životinja zamuljivanjem vode, pripremanjem sužavanjem vodotokova, sprečavanje prolaza životinja i slično.

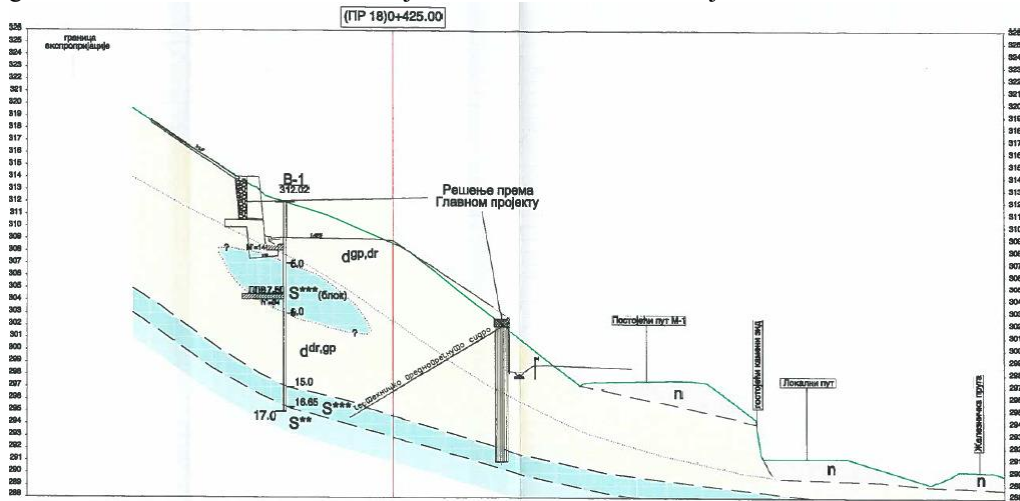


Slika 2 – Primer prirodno oblikovanog korita u području mosta sa stubovima van reke

1.2 POTPORNE KONSTRUKCIJE

1.2.1 Najviše korišćene vrste potpornih konstrukcija

Na jednoj deonici autoputa pojavljivalo se obilje tipova potpornih konstrukcija; zidovi od kamena u cementnom malteru, gabioni, plitko fundirani zidovi obrnutog T preseka, plitko fundirani zidovi sa stolicama, zidovi od niza bušenih šipova spojenih armirano betonskom gredom na vrhu sa ankerima ili bez njih, zidova od armirane zemlje i sl. što se vidi na Slici 3.



Slika 3 – Geološki poprečni profil na E75 km 883+725 (km 0+425 obilaznice M1) [3]

1.2.2 Način izvođenja potpornih konstrukcija

U Tehničkim izveštajima^[3] i Tehničkim specifikacijama^[5] propisan je način izvođenja plitko armiranih potpornih zidova po kampadama sa preskokom. Želja za brzim napretkom radova upotrebom mehanizacije dovela je do otvaranja širokih iskopa za zidove na velikim dužinama. Postavljanje oplata, armature, betoniranje, skidanje oplata, izrada hidroizolacionih premaza i zasipanje drenažnim materijalom prostora iza zidova uz upotrebu manuelnog rada nije pratilo ovaj tempo. Useci sa nagibima strmijim od prirodnih, stabilnih nagiba, mesecima su na nekim deonicama čekali zaštitu. Padavine su raskvasile uslovno stabilne kosine i došlo je do površinskog spiranja materijala i, u težim slučajevima, do stvaranja dubinskih kliznih površina.

Na E75 dominantni materijali u niveleti i kosinama su raspadnuti i drobinski izlomljeni škriljci. Ako je njihovo pružanje niz kosinu, to je nepovoljno za njenu stabilnost i vremenom se ovi sedimenti odronjavaju i dovode do klizanja gornjeg dela kosina. Gornje delove kosina u škriljcima treba štiti kraćim sidrima i torketom a u donjem nivou obično je kompaktnija stena kojoj je potrebno samo lokalno ojačanje i sidrenje stenske mase dužim sidrima povezanim betonskim gredama. Na E80 najveći obim nestabilnosti odvija se na kontaktu glina ili glina sa drobinom sa raspadnutim peščarima ili krečnjacima^[7]. Ovde su se na nekim delovima pojavili i problemi sa već zaštićenim kosinama koje su se „zarozale“. Najveći problem su delovi gde su se pojavile dubinske klizne površine koje ugrožavaju već podmaklo izgrađen autoput.

1.4 ZAŠTITA PUTEVA OD PADA STENA

1.4.1 Problem i definicija

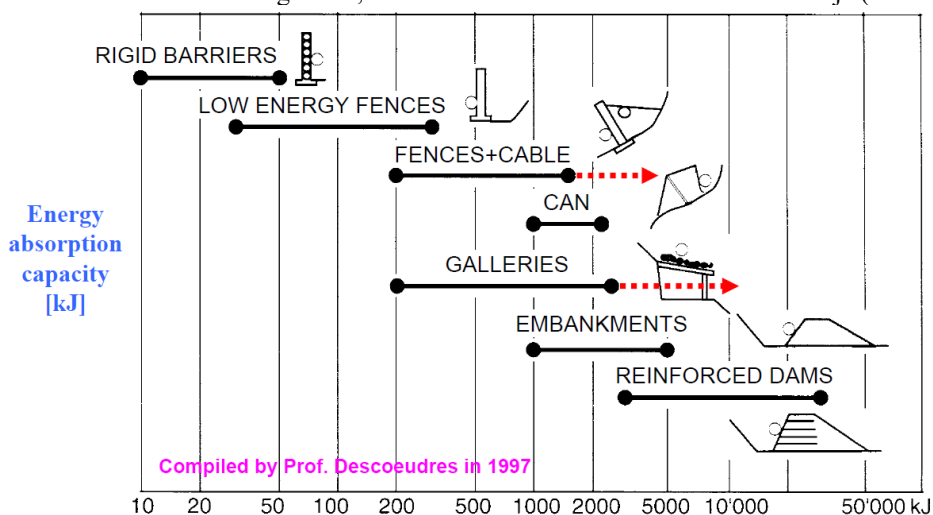
Na E75 se pojavila dilema oko potrebe, uticaja i vrste konstrukcija za zaštitu puteva izloženih rizicima od pada stena. Rizik, važnost saobraćajnice i tip zaštite su predmet detaljnih prethodnih studija. U poređenju sa drugim pokretima klizišta, pad stena je izuzetno brz proces sa velikim dužinama putovanja. Kada se pad desi, sposobnost pojedinca da izbegne posledice gotovo je svedena na nulu a rizik od povreda ili gubitka života je visoka. Oštećenja putne infrastrukture su verovatna. Stoga su u mnogim slučajevima potrebne prikladne mere zaštite.

1.4.2 Kriterijumi za preduzimanje mera zaštite od pada stena

- Prema geološkom atlasu Srbije – inženjersko-geološkoj karti, M. Lazić i D. Božović, 1995, 25% teritorija Republike Srbije je potencijalno u opasnosti od odrona zemlje i pada stena. Izgradnja useka i zaseka u područjima Koridora X (E-75 i E-80), zajedno s nekontrolisanom eksploatacijom šuma mogu narušiti uspostavljenu stabilnost prirodnog nagiba, kako u dubini, tako i na površini.
- Modeliranje pada stena teži izračunavanju putanja koje su veće nego u stvarnosti. To često rezultira projektovanju većih mreža i ograda nego što je potrebno.
- Pri proceni kvaliteta postojećeg održavanja i smanjivanja troškova starih kosina poželjno je pribaviti mišljenje Upravljača postojećih saobraćajnica. Treba težiti poboljšanju postojećeg ili uspostavljanju novog sistema upravljanja rizikom od pada stena.
- Standarde za zaštitu od pada stena treba temeljiti na važećem evropskom zakonodavstvu.

1.4.3 Načini zaštite od pada stena

Pored manje intenzivnih mera zaštite – zatravljivanjem, pošumljavanjem, oblaganjem kosina visećim mrežama sa tegovima, za zaštitu se koriste i razne vrste konstrukcija (Slika 5).



Slika 5 – Izbor konstrukcija za zaštitu u skladu s kapacitetom apsorpcije energije pada stena^[6]

2 ZAKLJUČAK

Do zahteva za izmenom projektovanih rešenja konstrukcija dolazilo je iz više razloga:

- Kratki rokovi za projektovanje u koje je bilo uključeno i vreme za geodetske i inženjersko-geološke istražne radove, često u zimskim uslovima,
- Pisanje projektnih zadataka prepušteno projektantima i neusaglašeno po deonicama,
- Prelaz sa starih jugoslovenskih standarda na evropske u vreme i posle izrade projekata,
- Nepostojanje važećih savremenih smernica za projektovanje u vreme izrade projekata,
- Revizije projekta radili su projektanti deonica jedni drugima, ne nezavisni revidenti,
- Deonice iz projekta nisu uvek bile deonice za izvođenje,
- Prilikom projektovanja autoputa nije uvek vođeno računa o paralelnom putu,
- Niske ugovorene jedinične cene gradnje kao glavni uzrok neprestanog traženja ušteta,
- Želja za ubrzanjem radova korišćenjem prikladnijih tehnologija gradnje,
- Strani izvođači koriste ranija iskustva u primeni crvenog FIDIC-a i prebacuju rešavanje problema na nadzornu službu i investitora.

U slučajevima kada predložena vrednosna poboljšanja nisu značajno uticala na produženje roka a donosila su poboljšanja i uštete, ona su prihvatana i realizovana na gradilištima.

Na sporije rešavanje zahteva izvođača uticali su i ugovorne odredbe:

- Nadzorna služba je samostalna u donošenju odluka po otvorenim pitanjima do prilično nisko određene vrednosti radova, pa se sporni zahtevi relativno sporo rešavaju,
- Investitor (Koridori Srbije d.o.o) i Upravljač (J.P. Putevi Srbije) nisu isto lice.

Ovo saopštenje treba shvatiti kao dobronamerno podsećanje za buduće projekte: resornog ministarstva, investitora, upravljača, davaoca uslova i saglasnosti, projekatnata, revidenata, izvođača i nadzornih službi, da se sve procedure, vreme potrebno za njih i pravila tehničke struke moraju ispoštovati da bi se krajnji proizvod – autoput i konstrukcije na njemu izgradili ekonomično, u razumnom roku i sigurno, i dugo služile svojoj svrsi – sigurnom i udobnom prevozu robe i putnika, bez ugrožavanja ljudi, biljnog i životinjskog sveta i okoline^[2].

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- [6] V. Labiouse: *Intenzivni kurs „Planinski rizici – Mere zaštite“*, Lozana, 2008
- [7] K. Batalović: *Komentari eksperta za geologiju u okviru Tehničke asistencije, 2012-2016*
- [8] G. Tadić: *Komentari eksperta za mostove i konstrukcije u okviru Tehničke asistencije Koridorima Srbije za deonice Koridora 10 – E75 Grabovnica-Levosojje, dužine 74,1 km, i E80 Prosek-Dimitrovgrad, dužine 84 km, 2012-2016*



Predrag Popovic¹, Dunja Vla²

RESTAURACIJA KRIVOG TORANJA U NAJLSU, ILINOIS, SAD

Rezime:

S-75

Krivi toranj u Najlsu je kopija italijanskog krivog tornja u Pizi u Italiji. Toranj je sagradjen od armiranog betona 1934-te godine i upola je manji od tornja u Pizi. Opseg i ozbiljnost betonskih oštećenja na tornju su bili toliki da je toranj morao biti zatvoren. Ispitivanje tornja je bilo uradjeno 2014. godine, i radovi na restauraciji su u toku. Izazov je bio da se prefabriquju i instaliraju montažni paneli sa vrlo malim tolerancijama, da boja i tekstura novih panela i betonskih popravki budu iste kao što su i postojeći. Opisana su oštećenja na konstrukciji i metode koje su korišćene za sanaciju.

Ključne reči: toranj, beton, oštećenja, popravke

RESTORATION OF THE LEANING TOWER OF NILES, ILLINOIS, USA

Summary:

The Leaning Tower of Niles is a concrete structure, half-size replica of the leaning tower in Pisa, Italy. The tower was constructed in 1934. Due to the severity of distress in the precast concrete panels and cast-in-place structure, the tower had to be closed. The condition of the tower was evaluated in 2014, and restoration is currently ongoing. The challenge was to prefabricate and install precast panels with close tolerances, color, and texture of the new panels and other concrete repairs to match the existing material. Described are the types of distress observed and the repair methods used to restore the tower.

Key words: tower, concrete, distress, repair

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² M. Sc., Senior Associate

1 INTRODUCTION

Construction of the Leaning Tower of Niles was started in 1931 and was completed in 1934 as part of a recreational park. The tower is a half-size replica of the original tower in Pisa, Italy. An overall view of the tower is shown in Figure 1. The original functional purpose of the tower was to store water for outdoor recreational swimming pools. The Leaning Tower plaza area has four fountains and a 30-foot long pool, and the area has been used as a tourist attraction and to host open-air music concerts since 1990.

Repairs of the tower were performed between 1995 and 1996, improving the structure, the façade, and the plaza area. The repair of the tower structure included repair of delaminated concrete at both cast-in-place (top and underside of floor slabs) and precast concrete panels and coping edges, tuckpointing and/or sealant replacement in joints between precast elements, closing openings in floor slabs at the building core, replacing doors and windows at each floor, repainting of metal stairs and handrails, replacement of cracked tiles at stair treads, application of concrete sealer, placement of a statue above the main door, and installation of new light fixtures and electric conduits at each floor.

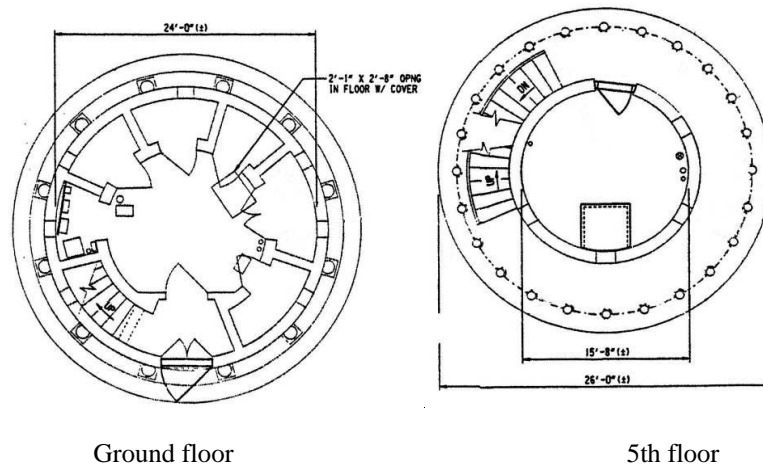
Over time the structure has deteriorated, resulting in the tower's closure to the public in 2012 due to unsafe conditions. In 2014 an investigation of the tower was conducted by Wiss, Janney, Elstner Associates, Inc. (WJE), and the tower is currently being renovated. Extensive repairs of the structure are required to restore its structural integrity.

2 DESCRIPTION OF THE STRUCTURES

The Leaning Tower of Niles is an eight story structure with an overall height of 94 ft. high, 28 ft. in diameter, and is leaning (7.4 ft.) in a south direction. Plan views of the ground and fifth floors are shown in Figure 2.



Figure 1 - Overall view of the Leaning Tower in Niles



Ground floor

5th floor

Figure 2 - Plan views of the ground and 5th floor

The core of the tower has a cylindrical shape and is constructed out of cast-in-place concrete with a 10 in. thick concrete wall, and 7 in. thick floor slabs. Floor slabs project approximately 4 ft. as cantilevers at the exterior of the tower, forming a balcony at each floor level. The core enclosure has four small windows and a metal door at each typical floor.

A total of twenty-four ornamental precast concrete columns at each level of the second through seventh floor are evenly spaced around the perimeter of the cantilever balcony slabs. Each column consists of a 10 in. wide square base, 7 in. diameter (bottom) to 6 in. diameter (top) round shaped middle section, and a decorative capital. Precast concrete arch elements span between columns at each typical floor.

Each floor balcony has a 36 in. high metal railing anchored into precast columns. The second through seventh floors are connected with curved exterior metal stairs, which are constructed of two metal stringers, metal risers, and metal treads covered with tiles set in mortar. Metal spiral stairs are located between the seventh and eighth floors, running through the center of the main core.

The ground level of the tower consists of two concentric concrete walls. The inside concrete wall is part of the main cylinder core wall. The 16 in. thick exterior wall is located directly below the cantilever balcony slab edge of the second floor, supporting columns above it. The wall is clad by twelve half circle ornamental precast concrete columns, precast arch panels spanning between columns, and precast coping at the top of precast arch panels around the perimeter of the tower. The entrance of the tower is located at its south portion and consists of a metal door set between two ornamental glazed precast columns supporting a decorative glazed precast door lintel with an arch at the top. A statue is placed above the door in an area framed with glazed lintel and an arch.

The eighth floor of the tower houses five bells. The floor slab and core walls are constructed of cast-in-place concrete. The core wall is clad on its exterior face with eight half-circle ornamental precast concrete columns, precast arch panels spanning between columns, and precast coping at the top of the precast arch panels around the perimeter of the tower. Steel frames supporting the bells are anchored into the concrete core wall.

The roof of the tower is a flat cast-in-place concrete slab with a decorative slab edge. The roof is accessible and has a metal round pipe railing.

3 TYPE OF DISTRESS

The exterior walls of the tower, including the enclosed spaces within the core of the tower at each floor, were inspected visually and hands-on from each floor level and from a boom lift. Typical and significant observations were as follow.

3.1 GROUND LEVEL

Main entrance. Cracks were observed at all precast elements framing the main entrance, two columns, door lintel panels, and arch. Freeze/thaw damage was observed at the base as well as on the side of the column to the south of the main door. Sealant in joints around the columns is cracked and separated from the substrate. Mortar joints between precast glazed elements are deteriorated. Door lintel segments have shifted outward leaving a gap in the joint at the back of the panels.

Columns. Freeze thaw damage and surface cracking were observed at two columns mostly concentrated at the bases and ornamental capitals of columns, as shown in Figure 3. Efflorescence staining is present at both columns.



Figure 3 - Freeze thaw damage observed at precast concrete column capital

Precast concrete arch panels. Freeze/thaw damage, surface cracking and efflorescence staining were observed on almost all precast arched segments. At several locations damage is very extensive, as shown in Figure 4. Some of the connections supporting arch panels to the concrete back up wall have failed due to corrosion.



Figure 4 - Extensive Freeze thaw damage observed at precast concrete arches

Precast coping. Precast coping segments located directly above arch panels at the level of the second floor were in a very poor condition. Delamination, cracking and efflorescence staining were observed around the entire perimeter of the floor. An open and deteriorated sealant joint between the precast coping and cast-in-place edge of the second floor cantilever slab was allowing water to collect within the cavity and penetrate behind the coping and travel down behind precast concrete arches, columns, and cast-in-place concrete wall, causing extensive damage, as shown in Figure 5.



Figure 5 - Distressed precast coping at the 2nd floor

Cast-in-place concrete wall. Surface cracking, efflorescence staining and freeze thaw damage were observed at the exterior surface of the wall at the ground level. The worst deteriorated areas are located directly below the precast arch segments and columns, which are also deteriorated the most. Spalling of concrete was observed at the bottom edge of window openings. Several areas were observed where reinforcing bars were originally placed too close to the concrete surface. It appears that the mortar skim coat was originally applied over the cast-in-place concrete walls in order to achieve a better finish of the wall. This finish coat has cracked, has efflorescence staining and is delaminated in some areas.

3.2 TYPICAL FLOORS

Columns. Cracks in precast columns were observed at every floor. Out of twenty-four columns per floor, on average half of them had cracks. Most of the cracking was observed at their bases. Cracking was also observed at their top and middle sections but to a much lesser extent. An exploratory opening in the middle portion of a column showed that the column was reinforced with 1/4 in. diameter vertical wires with only a 1/8 inch concrete cover over them. Reinforcing wire placed at the base of columns also has a very shallow concrete cover.

Floor slabs. Cracks were observed at approximately every 2 to 3 ft at the top surface of the balcony slabs. The undersides of the slabs were repaired with shotcrete during the 1996 repair project. Although the shotcrete repairs have cracked, repaired patches are in most areas still sound. Delaminated patches were observed on the underside of the sixth floor slab. Spalled and delaminated areas on the underside were observed mostly along the perimeter edge between columns. The exterior slab has spalled off at numerous locations. Observed spalls are small in size and are caused by corrosion of embedded steel reinforcing bars.

Precast concrete arch panels. Delamination of precast concrete arches was observed at each floor level on both interior and exterior faces of precast panels. Previously repaired sections have failed in many areas, as shown in Figure 6.



Figure 6 - Typical distress observed at precast concrete arch panels at typical floors

3.3 CONCRETE CORE OF THE TOWER

Floor slabs. Floor slabs at enclosed sections of the tower between the second and eighth floors were repaired in the past. Underside repairs are generally cracked but sound at most areas. Efflorescence stains observed on the underside of repaired areas indicate that the water is entering the enclosed areas and traveling through the slab. Delamination at the top of the slab was observed on the fifth floor.

Concrete walls. Spalled concrete was observed on the inside of the enclosure due to embedded corroded electric conduits that are not in service. They were replaced by new ones mounted on the walls during the repair project in 1996. Horizontal cracks around the exterior perimeter of the tower core were observed on each floor. Insufficient concrete cover over reinforcing bars was observed mostly around window openings in the wall. Delaminated concrete was also observed at door frames. The mortar skim coat that was applied on the exterior of the wall is spalling off at the top floor.

4 REPAIR OPTIONS

Based on the extent of the concrete deterioration, the following repairs were specified and implemented to restore the structural integrity of the tower.

4.1 GROUND LEVEL

Replace all precast arch sections and the coping edge with new precast sections. Replace precast columns and column capitals that are beyond repair due to extensive freeze/thaw damage. Repair any delaminated and spalled concrete at walls and column bases. Coat the exterior surfaces of the cast-in-place walls with a breathable concrete sealer to prevent water infiltration into the walls. Views of the new prefabricated precast panels and the condition of the tower core before and after installation of the new precast panels are shown in Figures 7a, 7b and 7c.



Figure 7a - Close up view of removed precast arch panels at the 2nd floor



Figure 7b - Close up view of the new precast arch panels during the installation



Figure 7c - Close up view of the installed new precast arch panels column capital, and coping above it

4.2 TYPICAL FLOORS

Inject cracks in the columns with epoxy. Repair any delaminated concrete along the precast arch panels, cast-in-place slab edges on both exterior and interior sides, and the underside of the balcony slabs. Apply a pedestrian traffic bearing waterproofing membrane over the top surfaces of the second and eighth floor balcony slabs. Upgrade the metal railing to comply with the current code. Completed repairs of precast concrete panel arches with mortar patches are shown in Figure 8. Repair of cast in place concrete arches on the eighth floor is shown in Figure 10. Reset displaced precast arch panel on the eighth floor if practical, if not install pin

anchors to secure the panel. The color of the repair mortar must match the existing concrete. The work in progress on repairing the underside of the structural slabs is shown in Figure 9.



Figure 8 - Overall view of repaired spalls in precast concrete arches at a typical floor



Figure 9 - Repairs in progress on the underside of structural slab



Figure 10 - Repair of delaminated cast in place concrete arch on the 8th floor

4.3 CONCRETE CORE OF THE TOWER

Epoxy inject horizontal cracks on the exterior of the core wall at each floor. Repair any delaminated and spalled concrete on the exterior and interior of the core wall, at windows, doors, and on the top and underside of slabs. Apply breathable concrete sealer over the exterior side of the cast-in-place concrete wall. An ongoing repair of the concrete core wall is shown in Figure 11.



Figure 11 - Partial and full depth wall repairs of concrete core at 8th floor

5 SUMMARY

Portions of the tower have exhibited significant levels of deterioration and dangerous conditions that resulted in closing the tower for public use. The tower had to be repaired and all of the arched precast panels at the second floor replaced. The challenge during the restoration was to prefabricate and install precast panels with close tolerances, color, and texture of the new panels to match the existing panels, as well as to match the color of the patching material. By implementing the recommended repairs, the existing distress and deterioration will be repaired and the useful life of this structure extended by many years.



Yuliya Ilieva¹, Borislav Daalov²

ARHITEKTONSKI INTEGRACIJA FOTONAPONSKIH MODULA NA FASADAMA

Rezime:

S-76

Zgrade nulte energije će postati obavezan evropski standard do 2020. One se odlikuju izuzetno visokim energetsom efikasnošću i upotrebom obnovljivih izvora energije. Danas je upotreba fotonaponskih elemenata znatno porasla. Problemi koji se odnose na njihovu arhitektonsku integraciju su u centru pažnje zbog svoje važnosti. Cilj ovog rada je istraživanje arhitektonske integracije fotonaponskih modula u fasade. Glavne grupe fotonaponskih sistema su klasifikovane prema građevinskim principima vezivanja za nosače i povezanosti sa fasadom zgrade. Prikazane su prednosti i nedostaci definisanih sklopova.

Ključne riječi: fotonaponski sistemi, integracija zgrada, fasade, održiva arhitektura

ARCHITECTURAL INTEGRATION OF PHOTOVOLTAIC MODULES INTO FACADES

Summary:

Nearly zero-energy buildings will become a mandatory European standard after 2020. They are characterized by extremely high energy efficiency and use of renewable energy. Nowadays the use of photovoltaic elements has substantially grown up. The problems concerning their architectural integration have become more important and have been a question of present interest. The aim of the current paper is to explore the architectural integration of photovoltaic modules into facades. The main groups of photovoltaic systems were systematized according to the constructional principle of their attachment to the bearing base and their connection with the structure of the building envelope. The advantages and disadvantages of the defined assemblies were studied.

Key words: Photovoltaics, building integration, facades, sustainable architecture

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1 INTRODUCTION

The word “photovoltaic” derives from two words: the Greek word “photo”, meaning light, and “voltaic”, meaning electricity. Photovoltaics are active systems that convert the absorbed solar energy into electricity through the photovoltaic effect using semiconductor materials. This process was discovered by Edmund Becquerel in 1839. Thus, the generation of electricity is independent of fossil or nuclear fuels. Photovoltaics are a form of clean renewable energy. They have a significant contribution to the protection of the environment and the reduction of the impact of climate change. Solar energy, along with that of the wind, rain, tides and geothermal, is considered practically inexhaustible and renewable. Nowadays the use of PV modules has considerably grown up. This is due to their efficiency and competitiveness and to the attitudes of society for sustainable development and use of energy from renewable sources. Photovoltaics have great potential for development. “Almost zero energy building” will become a mandatory European standard after 2020. Buildings with low energy consumption or nearly zero energy buildings are characterized by extremely high energy efficiency and with the use of renewable energy. Photovoltaic modules can be installed on or at a building, or even be integrated in the building envelope, giving a new dimension to environmentally responsive, energy-efficient and sustainable design. They can also provide an improved aesthetic appearance in an innovative way. Today building-integrated photovoltaics (BIPVs) can be used as semi-transparent or opaque glass facades and skylights, rain-screen claddings, sun shading systems, roofing elements, parapet units and canopies. The problems concerning their architectural integration became more important and turned into a question of present interest.

The paper begins with an overview of the current PV technologies. In order to reduce the volume of the current research, the application of PV modules was set for facades. Selected examples of implementations of this kind with integrated photovoltaics were presented and analyzed in the paper. The main types of photovoltaic systems were systematized according to the constructional principle of their attachment to the bearing base and their connection with the structure of the building envelope. The advantages and disadvantages of the proposed groups were studied.

2 PV TECHNOLOGIES

PV panels can be categorized into two groups: conventional crystalline solar cells (200-500 μm thick); and thin-films of semiconductors (1-10 μm thick). The most commonly used cell material is silicon. PV cells made of single-crystal silicon or often called monocrystalline cells (Fig. 1 a) are available on the market today with efficiencies close to 20%. Polycrystalline silicon (Fig. 1 b) is easier to produce and therefore cheaper. It is widely used since its efficiency (i.e., 11-15%) is only a little lower than the single crystal cell efficiency. Crystalline modules require to be embedded in a frame (usually of aluminum). They are protected by tempered glass on their front side. This leads to higher prices of the models offered. Monocrystalline silicon modules normally appear as a solid colour, ranging from blue to black. A wider variety of colours is available but at a cost of lower efficiency since their colour comes from reflection of some of the incident light which would otherwise be absorbed. Polycrystalline modules are normally blue (but again other colours are available) and have a multifaceted appearance which has a certain “shimmer”. Single crystal modules are usually recognizable as modules with polka dots or octagons. In both types, the current gathering grid

lines are well visible as a silver or black metallic color. For semitransparent modules the space between the single cells is enlarged to let light pass through.

Amorphous (thin-film) cells (Fig. 1 c) are obtained by deposition of ultra thin layer of photo-sensitive material of amorphous silicon (a-Si), copper - indium diselenide (CIS) or cadmium telluride (CdTe) on a substrate made of glass, aluminum, stainless steel or plastic. The process to manufacture this module is simple and cheap, but efficiency is so low (7–12%) that a very large area is required to produce the same kind of power made by crystalline systems. The thinness and semi-transparency of thin-film PV panels make them relatively appropriate for BIPV applications. They can be manufactured as flexible modules. Thin-film cells are resistant to high temperatures. PV laminates, compared to the crystalline solar cells, work better in cloudy weather, fog and shading, capturing different wavelengths of light [1]. These modules usually have a dark brown color. For semitransparent modules of amorphous silicon the cells themselves are pervious to light.

Dye solar cells (DSC) that are shown in fig. 1 d and organic solar cells (OSC) are new technologies with a potential of low cost production methods. Their solar efficiency is 7 % and 2.5 % and has been reached on test cells respectively.

Most PV modules have a 10 years warranty period. There are also such with 20 to 25 years. Their service period is over 30-40 years. The weight of PV modules is minimal. Depending on the type of cells and their capsulation it reaches to 18 – 23 kg/m².

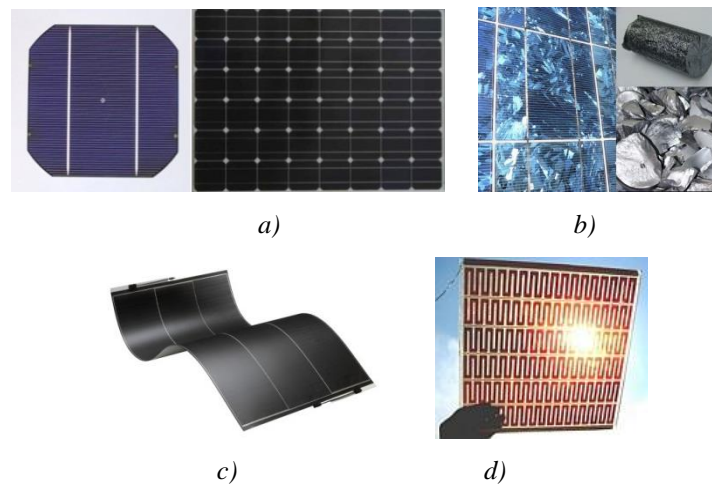


Figure 1 – Solar technologies used in BIPV: a) - single monocrystalline PV cell and assembled solar panel; b) - polycrystalline solar panel; c) - thin-film photovoltaic solar panel; d) - dye solar cells [10, 6]

3 DESIGN CASES

The building - Xicui Entertainment Complex (Fig. 2) was built in 2005 to house movie theatre and high-quality restaurant in the western part of Beijing, close to some of the 2008 Olympic Games sport facilities. In 2008 the old metal cladding of the east facade was replaced with a 60 x 33 m curtain wall. This wall is called GreenPix- Zero Energy Media Wall. It was designed by Simone Giostra & Partners Architects. The groundbreaking project applies sustainable and digital media technology to a curtain wall. The largest color LED display worldwide and the first photovoltaic system are integrated into the facade. Greenpix behaves like an organic system, absorbing solar energy during the day and then generating light from the same power that evening. The polycrystalline photovoltaic cells are laminated within the glass of the curtain wall and placed with changing density on the entire building's skin. The density pattern increases building's performance, allowing natural light when required by interior program, while reducing heat gain and transforming excessive solar radiation into energy for the media wall. The modules have a 5° tilt outward to left or right to increase the power output. Their dimensions are 890 x 890 mm. [5]

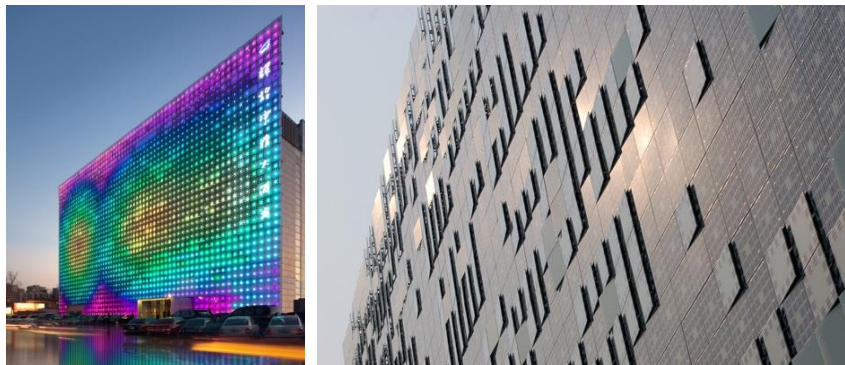


Figure 2 – GreenPix- Zero Energy Media Wall, Beijing, China [5]

The Solar Office (Fig. 3) is a new office building designed by Studio E Architects on the Doxford International Business Park, located near Sunderland in the northeast of England. It was completed in 1998. The solar facade is aligned to face due south and sloped at 60° to the ground. This configuration was chosen in order to provide good solar radiation at this northerly latitude. The facade incorporates over 400,000 poly-crystalline photovoltaic cells [8]. It is structural glazing curtain wall system, a proprietary product of Schüco International. The facade encloses an atrium which gets an interesting light and shadow effect from the PV facade. Bands of clear glazing were introduced into the facade to allow views out and ensure good internal light levels (Figure 3b). The risk of glare was minimised by the introduction of semi-transparent modules, immediately above the clear glazed panels, and by the provision of locally controlled roller blinds capable of covering both the clear and semi-transparent modules. The curtain wall structure also incorporates opaque PV modules.

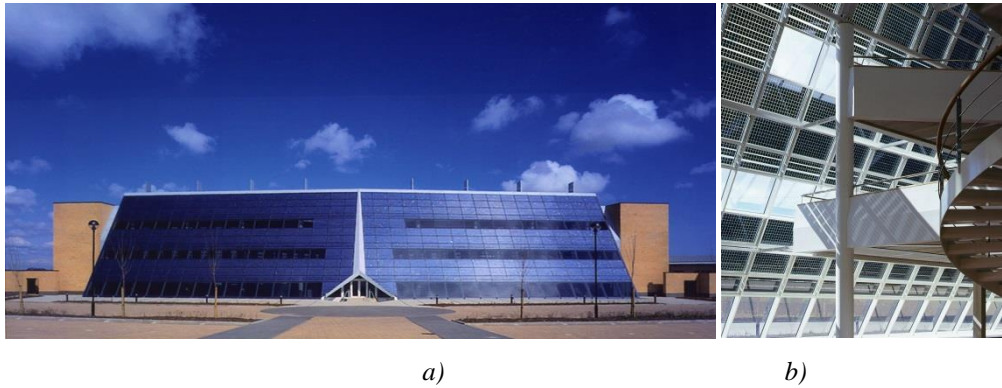


Figure 3 – Solar Office, Doxford (Sunderland), England: a) - Southern facade; b) - View of the interior [7]

The Environmental Building (Fig. 4) is sited 300 m from a major motorway in Garston and about 15 km north-west of London. It combines the highest architectural standards for offices with the latest innovations in energy efficient design. At an early stage of the project photovoltaic (PV) panels were examined but budget restrictions ruled out their inclusion [3]. Later thin film amorphous silicon panels were added to the west side of the south facade (Fig. 4). They were mounted to the external brick walls.



Figure 4 – Environmental Building, Garston, England [3]

The renovation of laboratory building 31 and the new office- and laboratory building 42 of the Netherlands Energy Research Foundation ECN in Petten (Fig. 5) are good examples of energy-efficient and sustainable architecture and they demonstrate the use of renewables in the built environment. BEAR Architecten are the authors of the project. For the south facade of the laboratory 31 (Fig. 5b) a PV-integrated sun shading system was designed. The Danish manufacturer Dasolas/Alco is involved in its production. A PV roofing system was also designed and installed. The office building 42 consists of three building units. The construction of the first unit is finished in March 2001. It is distinguished by its curved PV-glass roof.

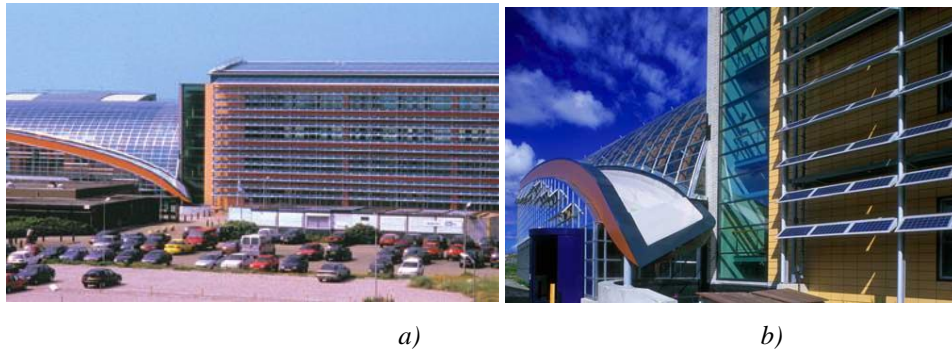


Figure 5 – ECN buildings 42 (left) and 31 (right), Petten, the Netherlands: a) - Southern facade of the buildings; b) - Solar shading system of Building 31 [2]

In 1994 the University of Northumbria in Newcastle upon Tyne (Fig. 6) was reclad with PV rain screen cladding. The arrays that consist of monocrystalline silicon modules were tilted at 65° from the horizontal [9].



Figure 6 – University of Northumbria, Newcastle upon Tyne, England

For the Großhadern Clinic in Munich the photovoltaic modules were integrated in the parapets of the balconies (Fig. 6). Thus they ensure safety of the occupants and energy generation.



Figure 7 – Großhadern Clinic, Munich, Germany [9]

4 ANALYSIS

The main types of architectural integration of PV modules into facades are shown in fig. 8.

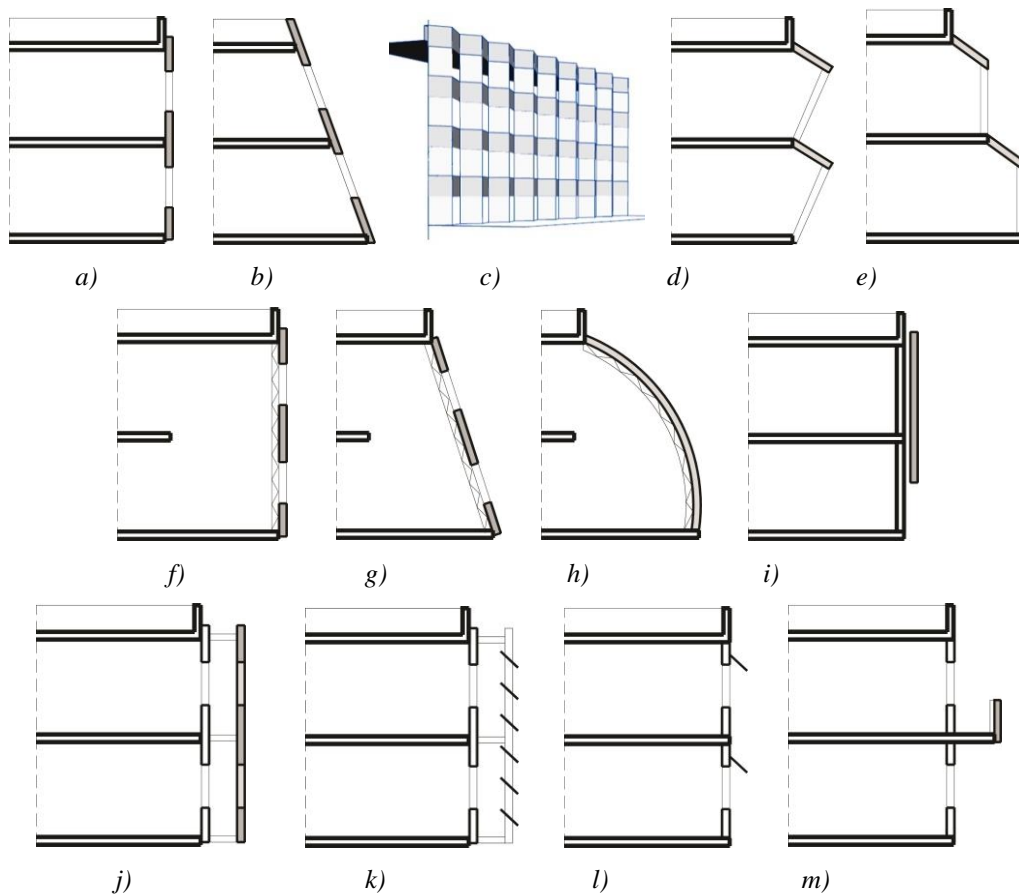


Figure 8 – Main types of architectural integration of PV modules into facades: a) – PV vertical curtain wall; b) - PV sloping curtain wall; c) - PV sawtooth vertical curtain wall; d) - PV vertical curtain wall with horizontal folding; e) - PV sloping curtain wall with horizontal folding; f) - PV vertical structural glazing; g) - PV sloping structural glazing; h) - PV curved structural glazing; i) – PV rainscreen cladding system; j) – PV double-skin facade system; k), l) – PV-integrated sun shading systems; m) – PV parapet units

The efficiency of the PV system is influenced by orientation and inclination of its modules. A favorable orientation is south. Deviation to southeast and southwest up to 30° is suitable [4]. PV modules can be placed in horizontal position, vertical position or inclined. Choice of inclination is influenced by latitude value and orientation. Maximum available irradiation is received by a south-facing unobstructed PV panels oriented at a tilt equivalent to the local latitude, while east and west-facing PV panels perform relatively well at steep angles or vertical orientation and still yield 60% of optimally inclined south oriented PV systems [4].

The favorable range of angle of inclination for the latitude of Bulgaria is $45^\circ \pm 5^\circ$. The optimal value is considered to be 42° .

PV curtain wall systems incorporate PV modules that are factory-assembled double-glazed units. A facade could consist of a combination of glazed areas, opaque PV panels and semi-transparent PV panels. Semi-transparent PV panels with medium optical quality might be used in parts of curtain wall glazing, where daylighting is the main design objective. They reduce the risk of glare.

Rainscreen cladding systems consist of panels set slightly off from the building to provide space for drainage and ventilation, as well as for running of mechanical and electrical services. The adequate ventilation gap is 100 mm or more wide.

In double-skin facade systems the photovoltaic panels are integrated in the outer skin of the facade to obtain highest performance. The ventilated cavity provides airflow to the PV modules, thereby preventing their overheating. Building mechanical and electrical services can be placed in it. The cavity also facilitates the maintenance of the solar system.

Photovoltaics can be used as sun shading systems. There are two main kinds – movable and fixed. In the case of movable sunshades both the tilt and orientation of the PV panels can be adjusted mechanically. Thus the building envelope becomes a smart structure which is responsible to day and season changes.

5 CONCLUSION

A wide variety of BIPV systems are available today. The most widespread ones for facades are the curtain wall systems, the PV structural glazing, the rainscreen cladding systems, the double-skin system and the PV-integrated sun shading systems. Their efficiency depends on the weather conditions and solar irradiation, the type of the cell, tilt and orientation, overheating and over shadowing. The PVs have to be considered as part of the overall environmental design of the building.

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Dunja Vla¹, Predrag Popovic²

SANACIJA FASADA OD KREČNJAČKIH PANELA

Rezime:

S-77 Zgrade sa krečnjačkim oblogama sagrađene u 1920-tim i 1950-tim godinama, potiču iz različitih arhitektonskih perioda, i imaju različite veličine i debljine krečnjačkih panela. Sistemi veza krečnjačkih fasadnih panela za konstruktivni sistem kod tih zgrada su različiti. U oba slučaja metalne veze su napravljene od korodirajućeg čelika. Oštećenja krečnjačkih panela kao posledica korozije njihovih veza usled izloženosti vlazi su vrlo slična kod svih tih zgrada. Opisana su oštećenja krečnjačkih obloga zbog korozije čeličnih veza i metode korišćene za njihovu sanaciju.

Кljučне речи: krečnjak, fasada, čelične veze, oštećenja, sanacija

RESTORATION OF LIMESTONE FACADES

Summary:

Buildings with limestone cladding built in the 1920s and 1950s, date from different architectural eras, and have different size and thickness of limestone panels. The system of connections used to attach the limestone blocks to the structural system of the buildings is different. In both cases, metal connections are made of corrodible steel. Distress in limestone panels as a result of deterioration of steel connections caused by moisture is similar. Distress caused in limestone blocks due to this deterioration and repair methods used to restore damaged limestone are presented.

Key words: limestone, facade, steel, attachments, distress, repair, restoration

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1 INTRODUCTION

The investigation and restoration of buildings with exterior walls clad with limestone was completed on eight buildings located at Northwestern University campuses in Evanston and Chicago over a period of 15 years. Northwestern University is a private research university established in 1851. The main campus is based in Evanston, Illinois, USA. The Chicago campus is only 12 miles to the south along Lake Michigan from the Evanston campus. The university has approximately 21,000 students and is composed of twelve schools and colleges. There is a large number of buildings located at both the Evanston and Chicago campuses.

Two buildings are selected to present typical conditions of the limestone facades and the restoration methods used on these projects. Buildings presented in this report were built in 1925 and 1953, making those 91 and 63 years old, respectively. Exterior walls of these buildings are clad with limestone, as shown in Figures 1 and 3. Moisture intrusion behind the limestone cladding has caused the corrosion of concealed metal straps and shelf angles. Cracking, spalling, and displacement of limestone panels were typical forms of distress observed.

The building built in 1925 had numerous very visible spalled, cracked, and displaced limestone blocks. Some of them, due to the possibility of falling off the building, presented hazardous conditions for pedestrians.

The building built in 1953 at first appeared to be in very good condition with some areas of minor distress until several slightly displaced stones above windows were removed. It was revealed that certain sections of the window lintels were severely corroded and had to be replaced.

2 DESCRIPTION OF BUILDINGS

2.1 1925 BUILDING

The overall size and plan views of these buildings at Northwestern University varied significantly. The largest building from this era that we worked on was the Montgomery Ward Building. The building measures 266 feet long and is 97 feet wide at the east end, 125 feet wide at the center, and 76 feet wide at the west end. The main portion of the building is 15 stories tall with a roof approximately 176 feet above ground level. At the center of the building there is an additional five-story tower. The top of the tower is 277 feet above ground level. An overall view of this building is shown in Figure 1. Limestone blocks used to clad the exterior walls of these buildings vary in size, with average height and width of 6 to 24 inches. Typical thickness of limestone blocks is 4 inches. Every fourth row of blocks is 8 inches thick, and serves to anchor the stone cladding into the brick masonry behind it. Decorative limestone pieces are present above building entrances, around windows, and at the top of the tower. The limestone facade is anchored to a masonry backup layer and is supported by a series of steel shelf angles and straps. Sections of window mullions are connected with steel dowels. The masonry, steel shelf angles, and the building's concrete floors are supported, in turn, by concrete-encased steel beams and columns. A typical cross section of the building is shown in Figure 2.



Figure 1 - Overall view of the Montgomery Ward Building

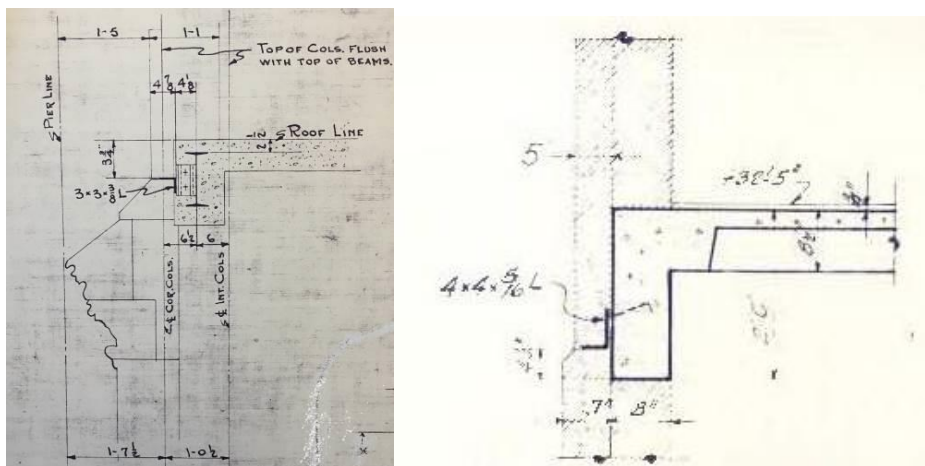


Figure 2 - Typical cross section of exterior wall, 1925 building

2.2 1953 BUILDING

The 1953 building is two stories tall and was constructed in two phases. The first phase was completed in 1953, and the second phase was completed in 1963. The building plan has a rectangular shape with two wings at its east elevation. The overall dimension of the building is 133 ft by 196 ft. The building's structural system consists of cast-in-place concrete beams, columns, and ribbed floor slabs. Exterior walls, 12 in. thick, are composed of 4 in. thick lannon stone with 8 in. brick masonry back up, and limestone bands directly below and above windows, and slate stone tiles between some of the windows. Parapet walls have limestone coping and a 14 in. thick limestone band panel located directly below it. Copper flashing was placed below coping stones. The canopy in front of the main entrance on the south elevation is

clad with limestone. An overall view of the building is shown in Figure 3, and the cross section through the exterior wall in Figure 4.

The limestone blocks above the window lintels are supported by two steel angles. The top angle provides lateral support, and the bottom angle provides a gravity support. Those limestone blocks are hung from the top shelf with two steel anchors set in lead. The bottom portion of stone is notched in order to be supported by a lower shelf angle and for the shelf angle to remain concealed.



Figure 3 - Overall view of the 1953 building

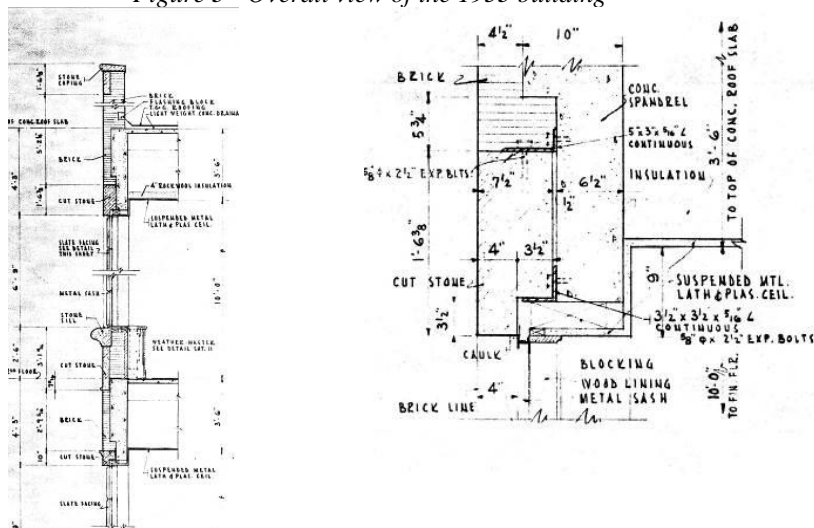


Figure 4 - Cross section through exterior wall, 1953 building

3 TYPE OF DISTRESS

3.1 1925 BUILDING

Distress observed at exterior walls, as shown in Figure 5, consists of deteriorated mortar joints between limestone panels; spalled limestone due to corrosion of embedded strap anchors; spalled limestone due to corrosion of embedded shelf angles; cracks observed along the building corners; displaced wall sections with spalled limestone; cracked and displaced window mullions; and cracked and displaced window rosettes.

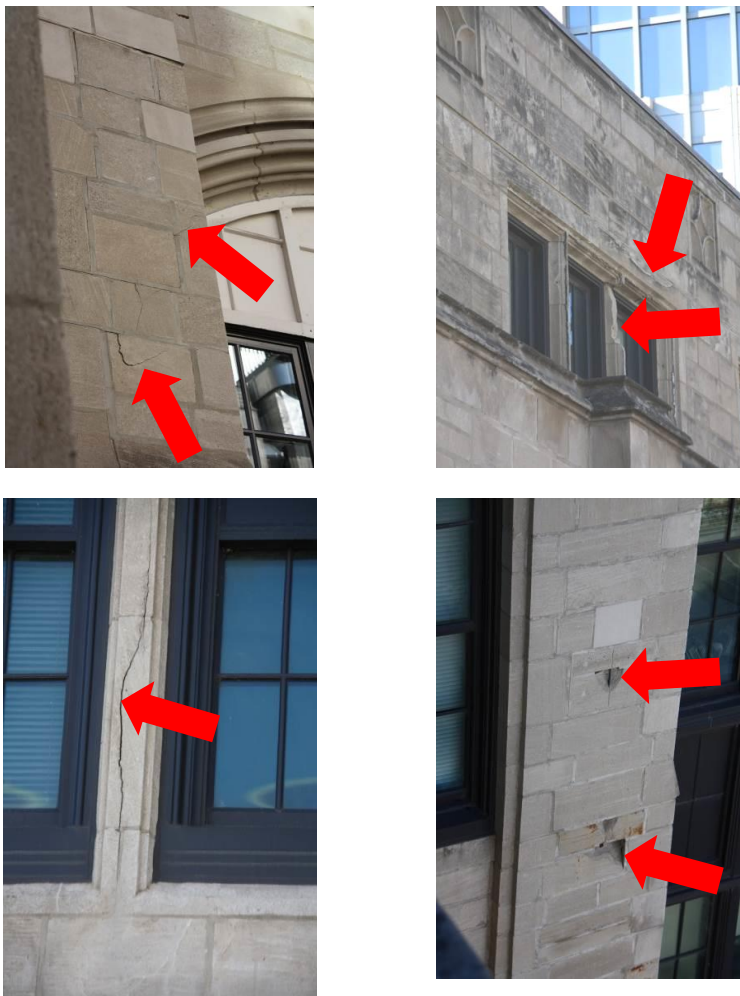


Figure 5 - Spalled, cracked limestone due to corrosion of embedded steel strap anchors and dowels

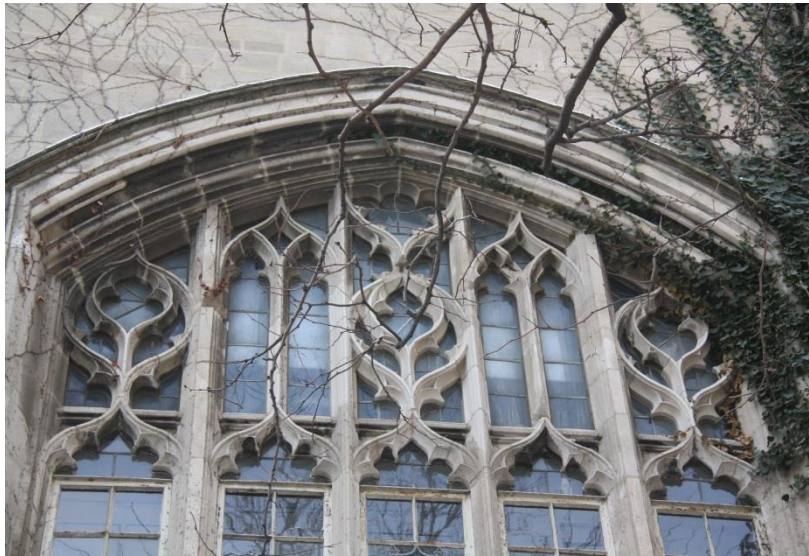


Figure 6 - Cracked and spalled window mullions and rosettes

3.2 1953 BUILDING

Initial observed distress at the building was minor and consisted mostly of deteriorated mortar joints, several small spalls in limestone blocks, and several slightly displaced limestone panels above the third floor windows and soiled exterior walls.

After removal of displaced limestone panels, it was discovered that sections of steel lintels behind the limestone panels above windows were severely corroded. The expansion of corrosion build-up has created large expansion forces causing pressure on adjacent stone, which caused displacement and cracking of the limestone blocks at their back portion. In

addition, it was revealed that portions of some of the steel shelf angles were never attached to the concrete beam of the building's structural system, had an insufficient bearing width, and at several locations the attachment anchors were severely corroded. Typical observed distress is show in Figures 7 and 8.



Figure 7 - Severely corroded steel shelf angle and cracked and spaled limestone block due to corrosion



Figure 8 - Overall and close up view of detached shelf angle from structural beam and cracked limestone panel at the back side due to corrosion of shelf angle

4 TYPE OF REPAIRS

Depending on the size of the limestone blocks, extent of their spalling and cracking, their location and the condition of the steel shelf angles supporting them, the following repair types were implemented to restore their condition and structural integrity:

- Repair 1 - Tuckpointing of mortar joints: includes grinding out deteriorated or cracked mortar joints to a depth of 3/4 in. minimum and installation of new mortar in order to prevent water infiltration into the building. One hundred percent of exterior wall joints were tuckpointed.

- Repair 2 - Patching of spalled limestone: includes removal of sections of deteriorated or damaged limestone and repairing the removed area with a hand placed mortar patch. Some patches may require stainless steel threaded rods as reinforcement, as shown in Figure 9.

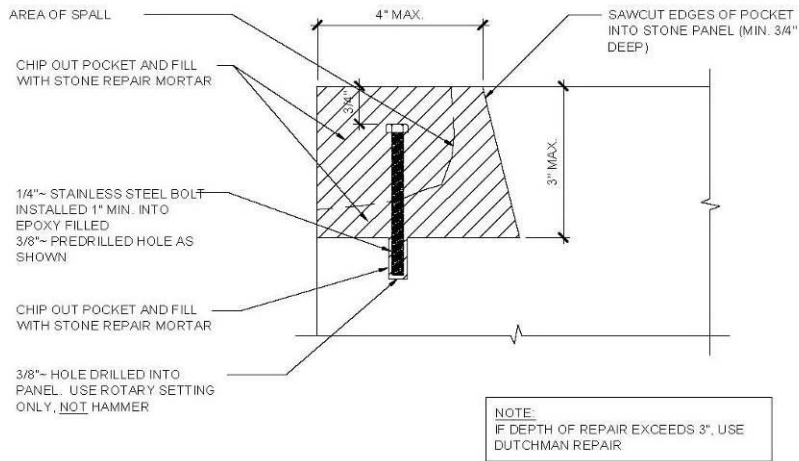


Figure 9 - Limestone patch

- Repair 3: Dutchman repair: includes removal of sections of deteriorated or damaged limestone and repairing the removed area with limestone dutchman held in place with strap anchors and/or stainless steel pins, as shown in Figure 10.

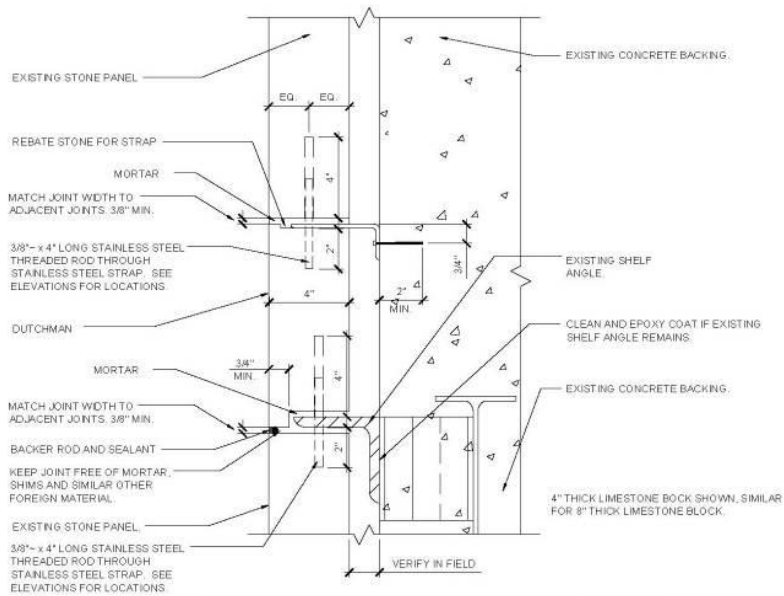


Figure 10 - Dutchman limestone repair

- Repair 4: Replacement of damaged limestone with new: includes removal of deteriorated or damaged limestone blocks and installation of new ones to match in size and texture the original position while providing supplemental stainless steel anchors where necessary.

- Repair 5: Installation of repair anchor to stabilize/secure the movement of displaced stones: includes drilling the hole through mortar joints, masonry, or concrete backing, installing the anchor, and tuckpointing the joint with mortar so that the installation of the anchor is not visible.

- Repair 6: Repair of shelf angle/lintel: includes removal of stones above the shelf angle/lintel as needed to expose the corroded portion of the embedded steel, as shown in Figure 11. The engineer who reviewed the condition of the existing steel and percentage of section loss, based on its condition, determined if the shelf angle/steel lintel would be repaired or replaced. If the level of corrosion was acceptable, the corroded steel was sandblasted clean, its surfaces properly prepared, and two coats of epoxy paint applied. Bytil flashing with a stainless steel drip edge was installed over it. Weep tubes with ropes were installed in vertical joints between stones, and the entire section of the wall was rebuilt with limestone to restore its original profile. New limestone work was installed flush with the existing limestone. If the level of corrosion was severe, the steel lintel/shelf angle was replaced with new material, and flashing with a drip edge was installed over it prior to rebuilding the section with limestone.



Figure 12 - Repair 6, repairing of steel shelf angles / window lintels at building built in 1953.

5 CONCLUSION

Distress observed in the limestone cladding of the exterior walls was caused by corrosion of embedded steel strap anchors, pins, and steel shelf angles and lintels. Spalled, cracked, and displaced limestone panels had to be repaired. Certain portions of facades had to be rebuilt in areas where corroded concealed steel supports had significant section loss, insufficient bearing width, or improper anchorage to the structural system of the building. Repairs were necessary in order to remove hazardous conditions dangerous for pedestrians, to prevent water infiltration into the building, and to extend the serviceability of the buildings.



Boško Stevanović¹, Mihailo Muravljev², Silvestar Horvat³

REKONSTRUKCIJA KROVNE KONSTRUKCIJE CRKVE KARMELSKOG MANASTIRA U SOMBORU

Rezime:

S-78

Crkva Karmelskog manastira nalazi se u centru Sombora. Njene dimenzije u osnovi su 58x25m. Detaljnim pregledom drvene krovne konstrukcije konstatovana su značajna oštećenja, čiji je stepen i karakter takav da je ugrožena njena nosivost. Analizirana je mogućnost sanacije postojeće konstrukcije ili njena rekonstrukcija. Konstatovano je, sa svih aspekata, da je opravdano uraditi rekonstrukciju. Uradjen je projekat potpuno nove krovne konstrukcije od lepljenog lameliranog drveta. Radovi na celokupnoj krovnoj konstrukciji i krovnom pokrivaču izvedeni su rekordno kratkom roku od tri meseca.

Ključne reči: crkva, manastir, krovna konstrukcija, rekonstrukcija

RECONSTRUCTION OF THE ROOF STRUCTURE OF THE CARMEL MONASTERY CHURCH IN SOMBOR

Summary:

Church of the Carmel monastery is located in the centre of Sombor. Dimensions of the church base are 58x25m. Detailed examination of the timber roof structure has shown significant damage, whose extent and nature compromised load carrying capacity of the structure. The possibilities of rehabilitation of the existing structure or its reconstruction were both analysed. It was concluded, from all aspects, that it was justified to do the reconstruction. The project of completely new roof structure made from glued laminated timber was developed. Works on the entire roof structure and roof cladding were done in the record time of three months.

Key words: church, monastery, roof structure, reconstruction

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1 OPIS OBJEKTA I NJEGOVOG KROVA PRE REKONSTRUKCIJE

Crkva Karmelskog manastira u centru Sombora poznata je još i kao *crkva sa dva tornja* (slika 1). Njene globalne dimenzije u osnovi su 58x25m, pri čemu u konstrukcijskom smislu crkva predstavlja objekat zidane konstrukcije sa drvenom krovnom konstrukcijom. Računato od površine terena, zidana konstrukcija je visine cca 19m, pri čemu se zidovi koji formiraju tu konstrukciju završavaju takođe zidanom "tavanskom konstrukcijom" u vidu sistema svodova i lukova.



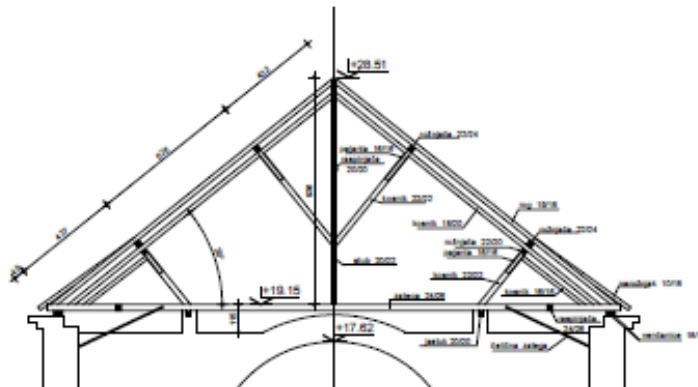
Slika 1 - Izgled crkve Karmelskog manastira

Osim obimnih zidova koji imaju debljine do skoro 2m, zidanu konstrukciju objekta čine i dva niza srednjih stubova sa preseccima približno 160/160cm, koji su izvedeni podužno u odnosu na objekat na osovinskim rastojanjima cca 7,60m. Ovi stubovi smanjuju "čist" razmak između zidova u unutrašnjosti objekta (taj razmak "od zida do zida" iznosi skoro 20m), tako da se u poprečnom preseku objekta dobija šema raspona zidane konstrukcije približno 5,40m + 12,20m + 5,40m.

Iznad opisane zidane konstrukcije, do visine cca 28,5m računato od površine okolnog terena, a sa osloncima na obimnim zidovima i na podužno postavljenim srednjim stubovima preseka 60/100cm (koji predstavljaju nastavke donjih stubova iz prostora crkve), izvedena je drvena krovna konstrukcija. Radi se o vrlo složenoj konstrukciji za čije se glavne konstrukcijske elemente - udvojene vezače - može reći da predstavlja kombinaciju krovne konstrukcije sa kosim stolicama i krova sa vešaljkom u sredini raspona. Parovi ovih vezača su postavljeni u rasteru cca 7,60m, od čega ima samo odstupanja na toranjskom i oltarskom delu crkve. Na tim mestima, koje karakteriše prisustvo masivnih poprečnih zidova i manjih razmaka između podužnih zidova, primenjena su drugačija, ali isto tako vrlo složena konstrukcijska rešenja, a što je uslovljeno kako oblicima, tako i geometrijskim merama prostora koje je trebalo pokriti.

Posmatrano u poprečnom preseku, predmetna krovna konstrukcija (njen glavni deo) sa udvojenim vezačima u rasteru 7,60m, a na mestima kosih stolica i kosnika, ima na dva nivoa

ukupno 2x2 rožnjače. Na ove rožnjače se oslanjaju rogovi, pri čemu usvojen raspored rožnjača po kosini krova nije zahtevao i izvođenje rožnjače-slemenjače. S obzirom na veliki raspon od cca 7m, niže rožnjače su izvedene u vidu složenog preseka koji čine tri gredna elementa, pri čemu su u ravnima kosih stolica na tom nivou primenjena i po dva para pajanti. Što se, pak, tiče više (gornje) rožnjače, ona je izvedena od samo jednog grednog elementa.



Slika 2 - Glavni deo ranije krovne konstrukcije

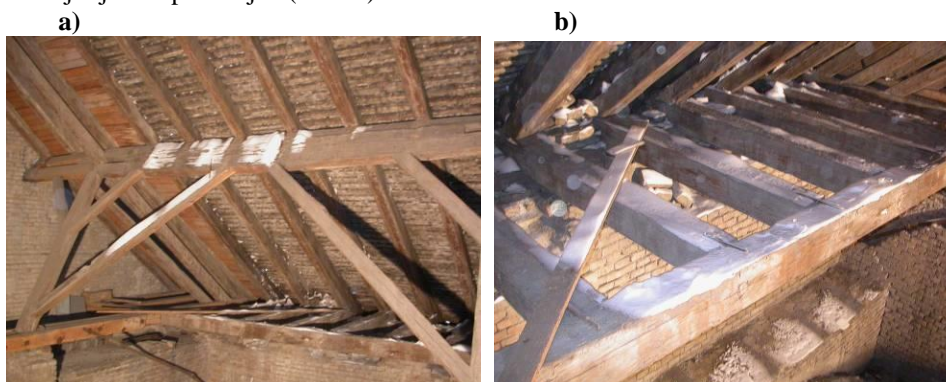
Niže kose stolice oslanjaju se na horizontalnu gredu-zategu na mestima kratkih zidanih stubova preseka 60/100cm, dok se kosnici izvedeni iznad ovih stolica (koji u neku ruku takođe predstavljaju kose stolice) oslanjaju na centralni stub - vešaljku - koja je povezana sa gredom-zategom. Opisana korovna konstrukcija prikazana je na slici 2.

Rogovi su, kao što se može zaključiti iz napred rečenog, u slemenu direktno međusobno povezani (bez slemenjače), dok su oni idući od slemena naniže, na dva nivoa oslonjeni na rožnjače. Na svojim donjim krajevima, pak, rogovi nisu neposredno oslonjeni na venčanice, već se oni oslanjaju ili na grede-zatege ili na kratke gredne elemente izvedene u ravnima greda-zatega. Na taj način je dobijeno rešenje po kome su kratki elementi o kojima je reč na jednom kraju, kao i same grede-zatege, oslonjeni na venčanice, dok su na suprotnim krajevima "učepljeni" u posebne grede koje se pružaju paralelno podužnim zidovima objekta, a sa dužinama od vezača do vezača. Na taj način u horizontalnoj ravni dobijen je konstrukcijski sistem koji se sastoji isključivo od pojasnih štapova i "vertikala" izvedenih na mestima svakog roga.

U zoni tornjeva (između njih) i u oltarskom delu crkve, koje karakteriše prisustvo masivnih poprečnih zidova i manjih razmaka između podužnih zidova, primenjena su slična konstrukcijska rešenja kao na glavnom - napred opisanom delu objekta. U tim delovima konstrukcijska rešenja krovova su takođe vrlo složena (posebno u oltarskom delu), međutim, ona su potpuno logična s obzirom na geometriju krovnih površina, odnosno na oblike prostora koje je trebalo pokriti. Ovde se takođe napominje da su predmetni delovi krovne konstrukcije - toranjski i oltarski - na adekvatne načine povezani sa glavnim delovima krova, tako da ukupno posmatrano celokupna krovna konstrukcija predstavlja jedinstven konstrukcijski sistem.

2 NALAZI VIZUELNO-MAKROSKOPSKOG PREGLEDA

Vizuelno-makroskopski pregled krovne konstrukcije objekta izvršen je u decembru mesecu 2009. godine. pri čemu je na samom početku pregleda uočeno da krovni pokrivač - biber crep - na velikom broju mesta "propušta" svetlost, što ukazuje da je taj krovni pokrivač značajno oštećen. Pored toga, a s obzirom da je pregled objekta izvršen u vreme snežnih padavina, na značajnom broju mesta uočeno da u tavanskom prostoru crkve postoje snežni nanosi. Sasvim logično, i to govori u prilog iznetoj činjenici o oštećenjima krovnog pokrivača, kao i letava preko kojih je ona postavljen (slika 3).

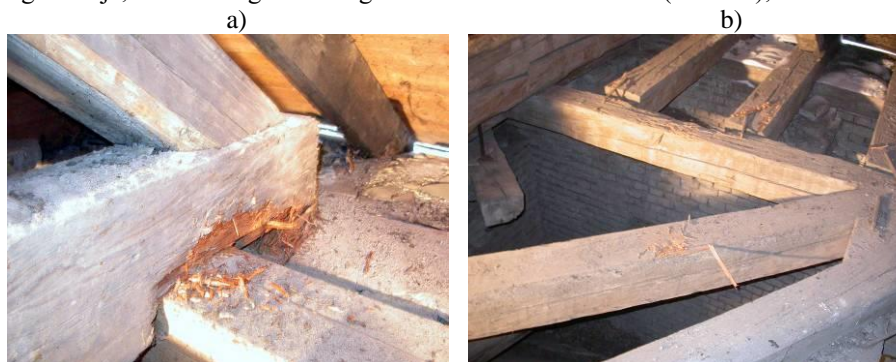


Slika 3 -Nanosi snega na krovnoj konstrukciji; a) - kosa stolica, kosnici i pajante na koje se oslanjaju rožnjače sastavljene od tri elementa; b) - nanos snega u zoni donjih krajeva rogova

Osim oštećenja krovnog pokrivača - biber crepa i letava, vizuelno-makroskopskim pregledom su konstatovani i sledeće oštećenja na samoj drvenoj krovnoj konstrukciji:

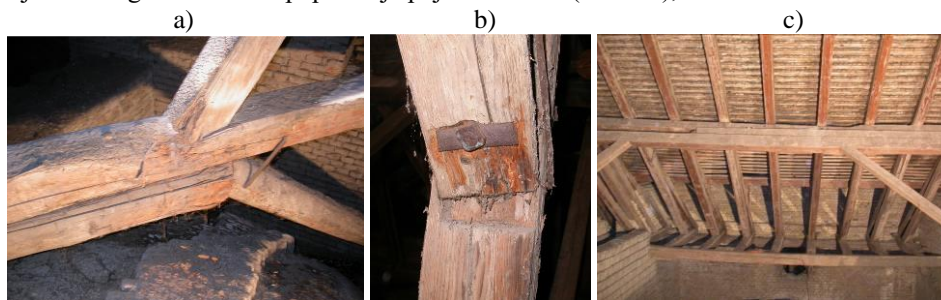
- trulež, koja je prisutna na velikom broju mesta, odnosno na mestima na kojima je dolazilo do vlaženje drvene građe, a što je direktno povezano sa oštećenjima krovnog pokrivača i sa neispravnošću oluka za odvođenje vode sa krovnih površina (slika 4a);

- crvotočina, koja je zahvatila vrlo velik broj drvenih elemenata, uključujući i elemente od vitalnog značaja, kao što su grede-zatege u okviru krovnih vezača (slika 4b);



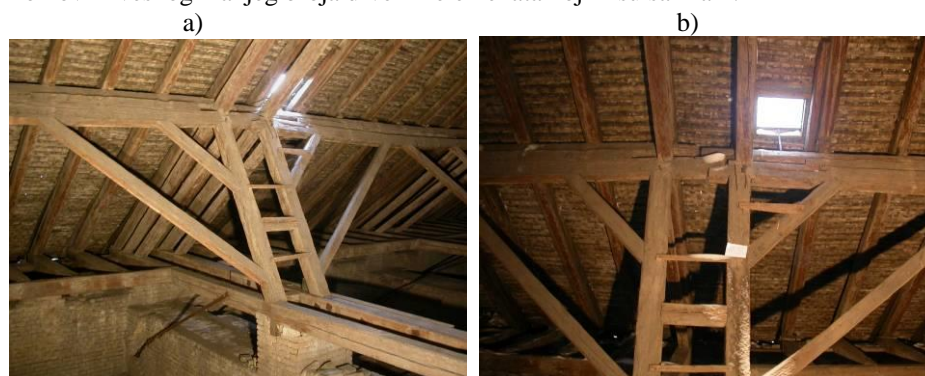
Slika 4 - Detalj truleži (a) i crvotočine (b) drvene građe

- popuštanje pojedinih veza, koje je uslovljeno ili efektima truljenja i "rada" crvotočine, ili neadekvatnim konstrukcijskim rešenjima primenjenim pri izvođenju radova (slike 5a i 5b);
- deformacije (ugibi) konstrukcijskih elemenata, do kojih je došlo ili usled fenomena tečenja drvene građe ili usled popuštanja pojedinih veza (slika 5c);



Slika 5 - Dva slučaja oštećenja, odnosno popuštanja veza elemenata konstrukcije (slike a) i b) i vrlo izražen ugib jedne rožnjače (slika c)

- pukotine u drvenoj građi, kao posledica termohigrometrijskih faktora, odnosno promene (smanjenja) vlažnosti drveta tokom vremena eksploatacije;
- lomovi izvesnog manjeg broja drvenih elemenata koji nisu sanirani.



Slika 6 - Pukotina u horizontalnoj gredi (a) i lom elementa između udvojenih vezača (b)

Ovde se napominje da je na pojedinim mestima u okviru konstrukcije krova uočeno prisustvo sanacionih intervencija izvršenih u prethodnom periodu; ove intervencije, međutim, predstavljale su samo palijativna rešenja primenjivana od slučaja do slučaja, a sa ciljem da se preduprede moguće ozbiljnije posledice.

3 ANALIZA STANJA KONSTRUKCIJE I PREDLOG INTERVENCIJE

Na osnovu prikupljenih nalaza vezanih za stanje predmetne krovne konstrukcije i analize istih zaključeno je da predmetna konstrukcija u kratkom roku zahteva vrlo radikalne sanacione intervencije. Naime, karakter i stepen oštećenja velikog broja vitalnih konstrukcijskih elemenata u okviru krova je takav da se slobodno može reći da je sigurnost predmetne konstrukcije u narednom vremenu ozbiljno dovedena u pitanje.

U vezi sa navedenim stavom pristupilo se izradi odgovarajućeg projekta sanacije, pri čemu se praktično na samom kraju izrade predmetnog projekta, došlo do zaključka da u sklopu sanacionih radova u postojećoj krovnoj konstrukciji treba izvršiti zamenu adekvatnom novom građom bar 50% "stare" građe. Pored toga, a s obzirom na registrovana oštećenja i složenost krovne konstrukcije o kojoj je reč, u vidu ojačanja treba da bude primenjena i veoma značajna količina čeličnih elemenata - blizu 9 tona. Takođe, izvođenje predviđenog sanacionog zahvata se pokazalo kao složeno i delikatno (opet iz razloga oštećenja i složenosti sistema postojeće krovne konstrukcije), pri čemu je zaključeno da bi pri izvođenju projektovanih sanacionih radova, mnogi detalji tih radova bili "otkriveni" tek u samom postupku izvođenja. A to bi u principu uslovljavalo kako određene korekcije projektnih rešenja, tako i potrebu izrade potpuno novih, a sve to povezano i sa odgovarajućim korekcijama količina i cena radova.

S obzirom na sve probleme vezane za *s a n a c i j u* postojeće krovne konstrukcije, u okviru izrade Glavnog projekta sanacije, a koji se projekat može smatrati za *I varijantu*, razmatrano je i jedno drugo varijantno rešenje. Naime, radi se o *II varijanti* koja podrazumeva da se umesto sanacije izvede potpuno nova, savremeno koncipirana krovna konstrukcija izrađena od tzv. lameliranog lepljenog drveta, koja bi ukupno zahtevala upotrebu znatno manje drvene građe nego što je to količina od oko 150m³ koja je danas prisutna u okviru krovne konstrukcije. Prema proceni, ta količina bi iznosila oko 105m³, pri čemu bi jedinična cena lamelirane drvene konstrukcije bila nešto veća od cene klasične drvene građe koja bi se primenila pri sanaciji postojeće krovne konstrukcije. Međutim, bez obzira na cenu radova koja bi u slučaju navedene *II varijante* bila za oko 20% veća od cene radova po *I varijanti*, smatralo se da ta razlika nije toliko značajna da bi se *II varijanta* odbacila. Pri ovome svakako treba uzeti u obzir i činjenicu da bi se realizacijom *II varijante*, između ostalog, dobilo i rešenje koje bi garantovalo funkcionalnost, upotrebljivost i trajnost tog dela objekta u vremenskom periodu koji bi se mogao meriti stotinom godina.

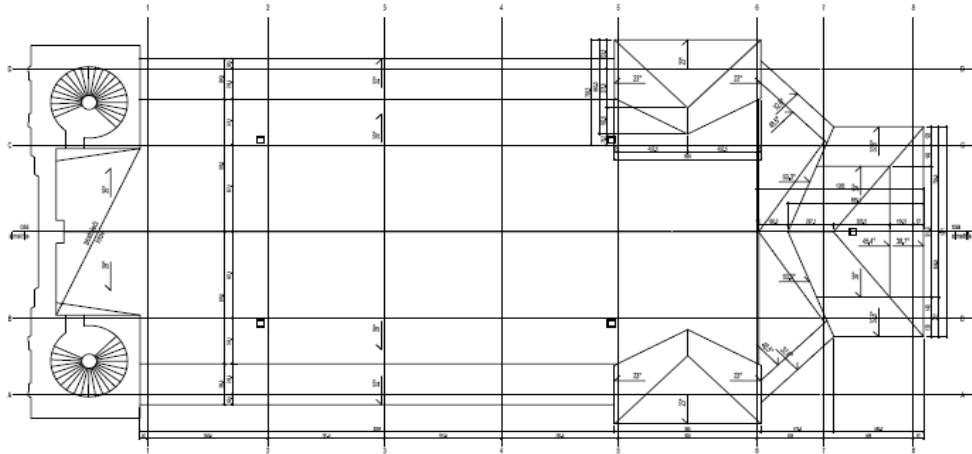
Sve napred navedeno detaljno je obrazloženo u dopisu upućenom maručiocu projekta, koji se, nakon razmatranja svih predloženih činjenica, a na bazi predloga projektanta da se radovi na predmetnoj krovnoj konstrukciji izvedu po *II varijanti*, složio sa tim predlogom, odnosno složio da se predmetna krovna konstrukcija reši u vidu potpuno nove konstrukcije od lameliranog lepljenog drveta.

4 PROJEKAT I IZVOĐENJE NOVE KROVNE KONSTRUKCIJE

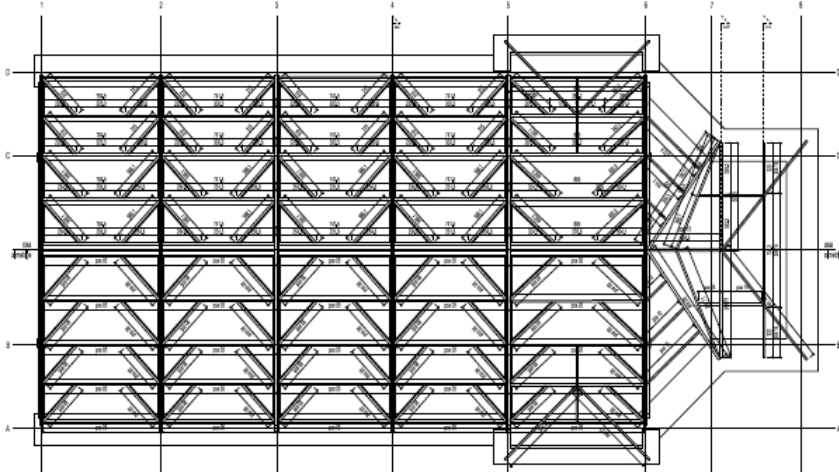
Izvođenje nove krovne konstrukcije od lameliranog lepljenog drveta, što s obzirom na karakter radova predstavlja *r e k o n s t r u k c i j u* postojećeg krova, podrazumevalo je da se prethodno u potpunosti ukloni celokupna postojeća krovna konstrukcija, uključujući tu i krovni pokrivač od biber crepa, kao i sve prateće elemente koji su bili prisutni u okviru postojećeg sistema pokrivanja krova (slemenjaci, limovi, letve i dr.), kako bi moglo da se pristupi postavljanju novoprotjektovane krovne konstrukcije.

Prilikom projektovanja nove krovne konstrukcije od lameliranog lepljenog drveta pošlo se od oblika postojeće krovne konstrukcije, odnosno od nagiba glavnih krovnih ravni te konstrukcije, od njenih visinskih kota i od samog načina oblikovanja krova. Time je u sklopu nove krovne konstrukcije definisan jedan dominantan glavni deo sa krovom na dve vode između poprečnih osa **1** i **6**, na koji se, iza ose **6**, a na delu iznad oltara, nadovezuje jedan drugačiji i složeniji viševodni krov (slika 7).

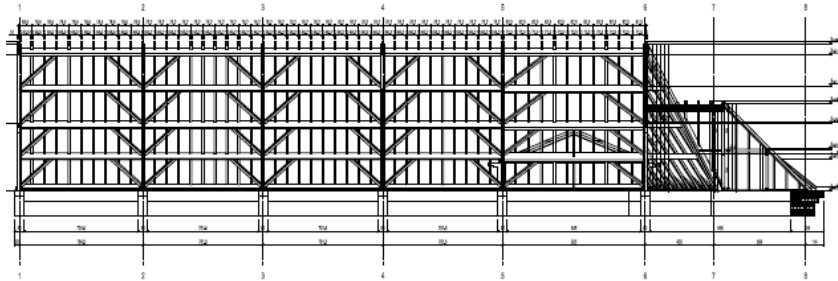
Glavne krovne ravni su konstantnog nagiba duž najvećeg dela svog pružanja, ali se zbog očuvanja postojeće arhitekture krova, pri dnu svake ravni vrši promena nagiba, odnosno ublažavanje "pada" krovne ravni, čime se krov na izvestan način "lomi". Nagib ovog "izlomljenog" dela krova dobija se upotrebom narožnjaka - dodatnih elemenata od masivnog drveta - koji se ne pružaju predaleko po kosini krova.



Slika 7 - Izgled krovnih ravni sa glavnim delom krova između osa **1** i **6**

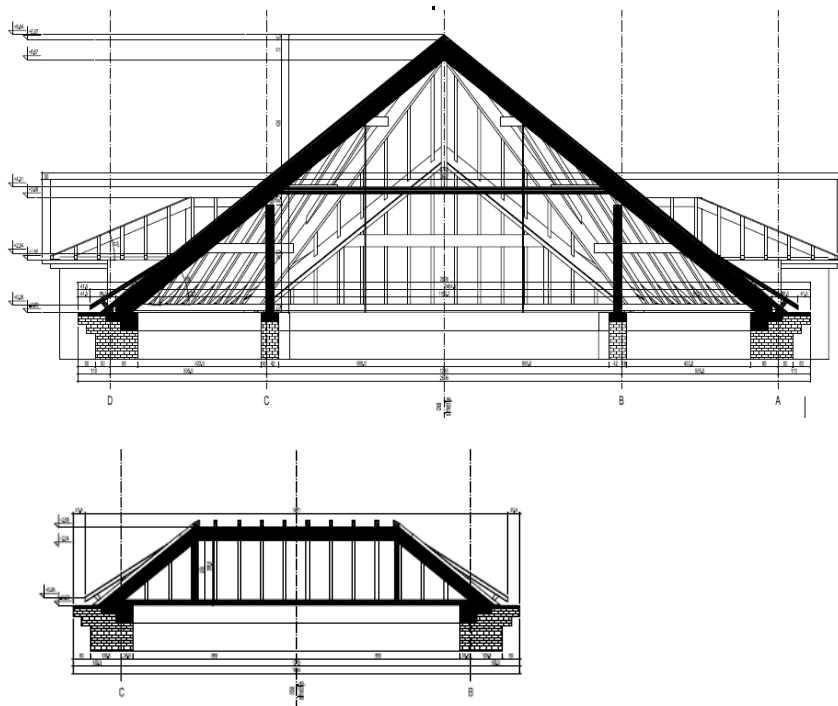


Slika 8 - Izgled u osnovi glavnog dela krovne konstrukcije između osa **1** i **6**, kao i deo iza ose **6** (na slici nije prikazan vrlo mali deo krova "na dve vode" između tornjeva)



Slika 9 - Podužni presek glavnog dela krovne konstrukcije uključujući i deo iznad oltara

Na slici 8 prikazan je izgled u osnovi glavnog dela krovne konstrukcije između osa **1** i **6**, kao i deo iza ose **6** (na slici nije prikazan vrlo mali deo krova "na dve vode" između tornjeva), dok je na slici 9 prikazan podužni presek glavnog dela krovne konstrukcije zajedno sa delom iznad oltara. Na slici 10, pak, prikazana su dva poprečna preseka novog krova.



Slika 10 - Poprečni preseki: presek kroz glavni deo konstrukcije krova (gornji presek) i presek kroz oltarski deo konstrukcije (donji presek)

Krovna konstrukcija prikazana na slikama 8, 9 i 10, a to se odnosi i na sekundarni deo krova između tornjeva, predviđena je za pokrivanje daskom, preko koje se prvo postavlja

hidroizolacija, da bi preko toga došle kontraletve, letve i biber crep - u obliku gustog - dvostrukog pokrivanja. U okviru navedene krovne konstrukcije nije predviđeno postavljanje termoizolacije.

Dispozicija oslonaca nove krovne konstrukcije bila je uslovljena postojećim zidovima, pri čemu je predviđeno da se preko gornjih površina zidova - po čitavoj dužini - izvede armiranobetonski serklaž MB 30. Ovaj serklaž, ili bolje rečeno armiranobetonska greda sa konzolnom pločom od armiranog betona, ima ulogu da poveže erkere koji "izlaze" iz zidova crkve sa unutrašnjim delovima tih zidova, pri čemu će on biti i u funkciji "raspoređivača" koncentrisanih sila (prvenstveno vertikalnih, ali i horizontalnih) koje se na zidove tačkasto prenose sa krovne konstrukcije.

Krovna konstrukcija prikazana na slikama 8, 9 i 10 oblikovana je kao sistem primarnih glavnih vezača povezanih sekundarnim nosačima - rožnjačama sa odgovarajućim kosnicima, preko kojih se postavljaju tercijarni nosači - rogovi. Svi elementi krovne konstrukcije su projektovani u savremenom sistemu lameliranog lepljenog drveta. Samo su rogovi, venčanice i narožnjaci predviđeni od masivnog drveta.

Glavni vezač je luk na tri zgloba ojačan donjom primarnom zategom, pri čemu je on u polju dodatno ojačan i sa dva srednja stuba, kao i sa sekundarnom zategom postavljenom u visini gornjih krajeva tih stubova; ovi stubovi se svojim donjim krajevima oslanjaju na odgovarajući način "doterane" postojeće kratke zidane stubove.

Sve veze između drvenih elemenata u okviru vezača izvode se primenom odgovarajućih čeličnih elemenata, pri čemu su sve takve veze dimenzionisane prema uticajima koji su dobijeni statičkim proračunom.

Gornji pojas glavnog vezača ima poprečni presek dimenzija $b/h = 20/54,4\text{cm}$, dok je gornja zatega preseka $b/h = 20/22,4\text{cm}$. Oblikovanje veza uslovalo je, pak, da donja zatega bude dvostrukog preseka $2xb/h = 2x10/22,4\text{cm}$. Kako bi se umanjio uticaj sopstvene težine na donju zategu, ta zatega je pomoću metalnih vešaljki u polju "okačena" za gornji pojas vezača. Noseći stubovi u okviru glavnog vezača su dimenzija $b/h = 20/28,8\text{cm}$. Glavnim vezačem oblikovanim na opisani način uspešno se premošćava raspon od 22,86m, koji se sastoji iz tri polja $5,355 + 12,15 + 5,355\text{m}$.

Na glavni vezač oslanjaju se rožnjače koje su upuštene u odnosu na gornju ivicu glavne kose grede u sklopu vezača, pri čemu su one postavljene tako da su im gornje ivice izravnate sa gornjom ivicom te grede. Rožnjače su oko podužne ose zarotirane prema nagibu krovne konstrukcije, a dimenzije njihovih poprečnih preseka variraju u funkciji uticajnih površina i raspona koje premošćavaju. Karakteristično je polje konstrukcije između osa **5** i **6** - mesto gde se u glavni sistem krova uključuju "badže" - gde se ugrađuju rožnjače većeg poprečnog preseka i to $b/h = 16/54,4\text{cm}$ i $b/h = 20/57,6\text{cm}$. U ostalim poljima konstrukcije, pak, rožnjače su preseka $b/h = 16/44,8\text{cm}$, pri čemu je u tim poljima prva rožnjača do oslonca na zidu prema tornjevima manjeg preseka, odnosno preseka $b/h = 16/32\text{cm}$.

S obzirom da su rožnjače zbog velikih raspona vrlo nepovoljno opterećene na savijanje oko "slabije" ose inercije, u krovnu ravan su uvedeni kosnici. Kosnici imaju funkciju smanjivanja raspona, pri čemu one istovremeno obezbeđuju potrebnu podužnu krutost konstrukcijskog sistema za slučaj delovanja horizontalnih uticaja. One praktično preuzimaju ulogu spregova u krovnoj ravni, te stoga u krovnoj ravni nije predviđeno formiranje "spregnutih" polja.

Preko sistema rožnjača postavljaju se rogovi od masivnog drveta preseka $b/h = 10/16\text{cm}$ koji nose krovni pokrivač, pri čemu je njihova donja ivica upuštena 3cm u odnosu na gornju

ivicu rožnjače; na taj način je omogućen efikasniji prenos sila sa rogova na rožnjače i kvalitetnije "utezanje" krovne konstrukcije.

Na delu iznad oltara, na delu sekundarnih krovnih ravni, takođe se formira konstrukcija od nosača sa zategom, od rožnjača i od rogova kao tercijarnih nosećih sistema.

Svi elementi krovne konstrukcije izvode se od lameliranog lepljenog drveta, gde se kao osnovni materijal koristi jela ili smreka, a veze između pojedinih drvenih elemenata, kao i veze sa zidovima postojećeg objekta, izvode se pomoću namenski oblikovanih čeličnih okova.

U daljem će biti prikazano nekoliko snimaka načinjenih tokom izvođenja radova na rekonstrukciji krova crkve *Karmelskog manastira* u Somboru, uz napomenu da je pri izvođenju tih radova u potpunosti potvrđena pretpostavka o opravdanosti prihvatanja rešenja rekonstrukcije predmetnog krova, umesto njegove prvobitno planirane sanacije. Naime, pri izvođenju radova o kojima je reč u potpunosti je potvrđen karakter i visok stepen oštećenja velikog broja vitalnih konstrukcijskih elemenata u okviru postojećeg krova (trulež, crvotočina i dr.) konstatovan tokom izvršenog vizuelno-makroskopskog pregleda. Kao ilustracija toga, na slici 11 prikazani su detalji a) i b) koja govore o izuzetno visokom stepenu "ispučnosti" drvene građe u okviru starog krova.

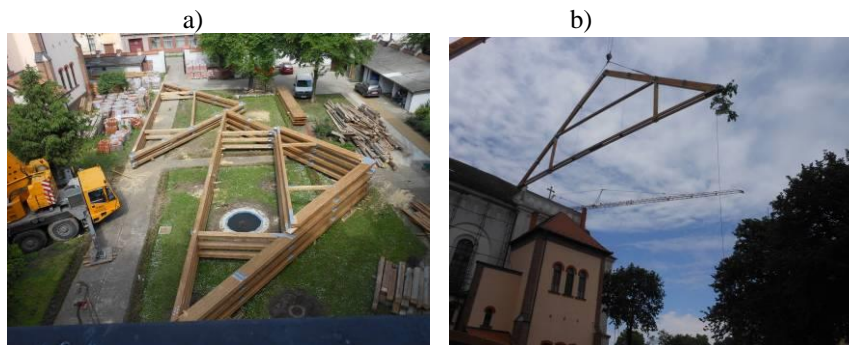


Slika 11 - Ilustracija visokog stepena "ispučnosti" drvene građe u okviru postojećeg krova

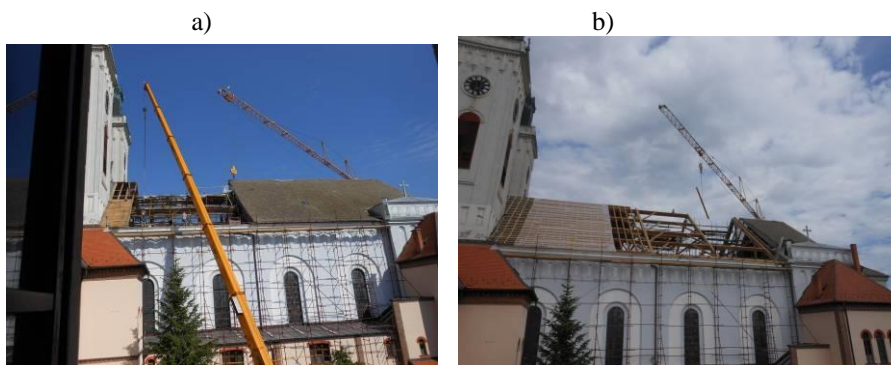
Na slici 12a prikazani su gotovi glavni vezači krovne konstrukcije pre njihove montaže, dok je na slici 12b prikazano "nošenje" jednog od tih vezača na mesto ugrađivanja.

Slika 13 prikazuje dve faze izvođenja radova na novoj krovnoj konstrukciji. Na slici 13a vidi se sam početak radova, kada je samo sa jednog dela krova uklonjena postojeća konstrukcija (drugi deo je još uvek pod "starim" krovom). Slikom 13b, pak, ilustruje se jedna od narednih faza izvođenja radova, odnosno vidi se sukcesivno napredovanje radova - uklanjanje stare i montaža nove konstrukcije.

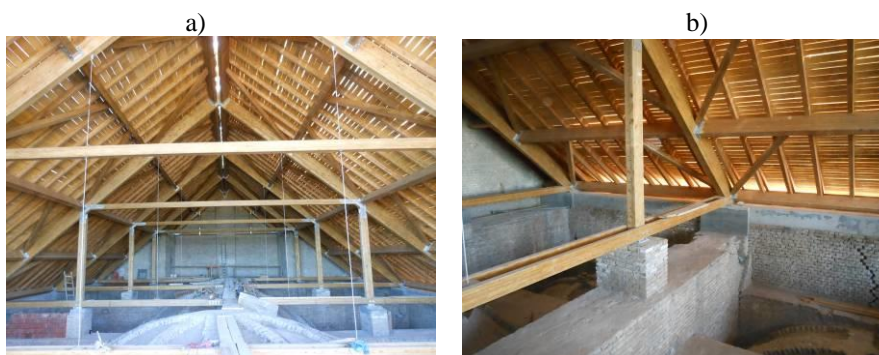
Na slici 14a prikazan je poprečni izgled dela završene krovne konstrukcije, dok slika 14b prikazuje detalj oslanjanja drvenog stuba primenjenog u okviru glavnog krovnog vezača na kratak zidani stubić.



Slika 12 - Gotovi glavni vezači krovne konstrukcije pre montaže (a) i "nošenje" jednog od vezača putem kрана na mesto ugrađivanja (b)



Slika 13 - Dve faze izvođenja radova na rekonstrukciji krova: a) - jedna od početnih faza, kada je samo sa dela krova uklonjena postojeća konstrukcija; b) - jedna od narednih faza sukcesivnog uklanjanja delova starog i montaže novog krova

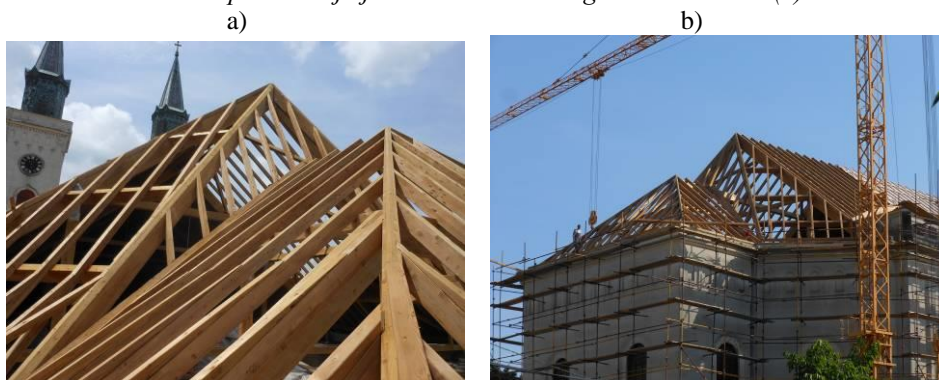


Slika 14 - Poprečni izgled dela završene krovne konstrukcije (a) i detalj drvenog stuba koji se oslanja na kratak zidani stubić (b)

Na slici 15a prikazan je izgled dela završene krovne konstrukcije snimljen iz tavanškog prostora, dok je na slici 15b dat detalj povezivanja jedne od rožnača sa glavnim vezačem.



Slika 15 - Izgled dela završene krovne konstrukcije snimljen iz tavanjskog prostora (a) i detalj povezivanja jedne od rožnača sa glavnim vezačem (b)



Slika 16 - Radovi na izvođenju dela krova iznad oltarskog dela crkve



Slika 17 - Snimak završnih radova na rekonstrukciji krova snimljen putem drona



Slika 18 - Snimak gotovog krova snimljen putem drona

Na slici 16a i 16b prikazani su radovi na izvođenju dela krova iznad oltarskog dela crkve, dok je na slikama 17 i 18 prikazan snimak završnih radova na rekonstrukciji krova, kao i izgled gotovog krova, snimljen putem drona.

5 UČESNICI NA REALIZACIJI POSLA NA REKONSTRUKCIJI KROVNE KONSTRUKCIJE KARMELSKOG MANASTIRA U SOMBORU

Investitor: Crkva Svetog Stjepana kralja, Karmelski samostan, Sombor, putem donatora:

- Renovabis E.V. (Nemačka)
- Karmelska provincija Sv. Josipa (Hrvatska)
- Kirche in Not (Nemačka)
- „Potisje“ a.d. Kanjiža (većina crepa i slemenjaci)
- Budžet AP Vojvodine (Stručno mišljenje o stanju krovšta, Glavni projekat sanacije krovšta)
- Budžet Sombora (za nabavku dela crepa i slemenjaka)
- Konkurs AP Vojvodine za verske zajednice (za nabavku dela crepa)
- „Panonija“ d.o.o. Sombor
- „Panonija pharm“ d.o.o. Sombor
- JKP Sombora „Prostor“, „Čistoća“, „Elektrodistribucija“, „Parking servis“

Projekat: IMK Građevinskog fakulteta Univerziteta u Beogradu

- odgovorni projektant prof. dr Boško Stevanović, dipl.inž.građ.
- projektant prof. dr. Mihailo Muravljev, dipl.inž.građ.
- saradnici: Srđan Kosmač, građ.tehn, Miodrag Savić, građ.tehn.

Glavni izvođač radova: „SMB-GRADNJA“ d.o.o. Subotica sa kooperantima:

- „AS GRADNJA“ SZR, Apatin,
- „KONC“ SZR, Sombor,

-„MARKO LIMAR“ d.o.o. Subotica
-„SMB-MEHANIZACIJA I TRANSPORT“a.d., Subotica

Odgovorni izvođač radova: Zdravko Dragović, dipl.inž.građ.

Rukovodilac radova: Goran Bogić, dipl.inž.građ.

Izvođač radova lepljene lamelirane krovne konstrukcije:
„HOJA“ lepljene konstrukcije in žaga d.d., Škofljica, Slovenija
sa kooperantom:
Tesarstvo in krovstvo „Marko Rus“s.p. Polhov Gradec, Slovenija

Odgovorni izvođač radova: Jurij Žejn, dipl.ing.građ.

Rukovodilac radova: Polde Zavodnik, građ. tehn.

Projektanski nadzor: Univerzitet u Beogradu, Građevinski fakultet - IMK

Direktivni nadzor na gradilištu: Horvat Silvestar, dipl.ing.građ.

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PROJEKTOVANJE I IZGRADNJA LUKE NAUTIČKOG TURIZMA U OKVIRU KOMPLEKSA PORTONOVI

Rezime:

S-79

U Herceg Novom je u toku izgradnja luksuznog turističkog kompleksa Portonovi u Kumboru, koji će se, između ostalog, sastojati i od Luke nautičkog turizma. Luka obuhvata marinski basen i formiraju je dva gata fiksne dužine 200 m, mjereno od postojeće obalne linije. Projektovanu konstrukciju gata čine montažni elemenati koji se oslanjaju na bušene šipove. Prilikom projektovanja uzeti su u obzir geotehnički uslovi, seizmičnost lokacije, trajnost u morskoj sredini, itd.

Ključne reči: radovi u marini, luka nautičkog turizma, likvefakcija

DESIGN APPROACH AND CONSTRUCTION OF A NEW PORT OF NAUTICAL TOURISM IN PORTONOVI

Summary:

The municipality of Herceg Novi projected a wide development known as Portonovi in Kumbor, which is a world class resort with a Port for nautical tourism. The port is formed through the construction of two main fixed jetties which extend 200 m in length from the existing coast line and encapsulate the marina basin. The new jetties are designed as pre-cast decks supported by concrete bored pile; the design takes into account problems related to geotechnical conditions, seismicity, durability in the sea water.

Key words: Marina works, port of nautical tourism, liquefaction

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1 INTRODUCTION

As part of the restoration of degraded areas of Montenegro and their integration into the urban system, the municipality of Herceg Novi projected a wide development known as Portonovi in Kumbor, which is a world class resort located along the coast in Kotor bay with a Port for nautical tourism.

Portonovi Marina has been designed in order to accommodate boats from 8m up to 110 m in length, and to be fully functioning in all weather conditions. The port is formed through the construction of two main fixed jetties which extend 200 m in length from the existing coast line and encapsulate the marina basin.

Both jetties have a width ranging from 12 to 20 m and will act as a breakwater structures. They are conceived with a sufficient width to allow vehicular access and to accommodate the following specific facilities: support buildings for port operation management, electrical and mechanical plants, refueling.

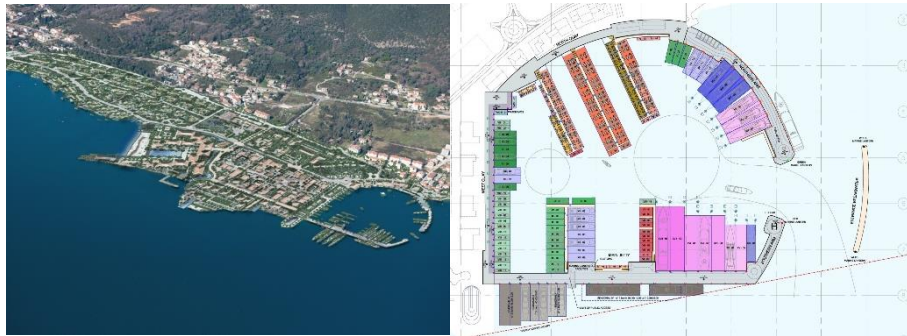


Figure 1. Portonovi resort: Masterplan and Berth arrangement

2 STRUCTURAL LAY-OUT OF JETTIES AND QUAYS OF THE NEW PORT

The new jetties and the quays are designed as opened concrete bored pile-supported decks, taking into account all problems related to geotechnical conditions, seismic activity in the area, durability of structures in the sea water.

Decks are constituted by a grid of precast beams completed by a cast in place concrete slab 30 cm thick.

Concrete structures are suspended on cast in situ concrete bored piles, executed with the aid of a drag bit and a permanent hollow-section steel casing; diameter of concrete piles varies from 600 mm to 800 mm in diameter to support the berth structure. Piles length varies roughly from 15 to 35 m.

Typical transversal dimensions of jetties and quays vary between 12 and 20 m; the rows of piles are spaced 6-8 m and are composed by 3 up to 6 piles each row.

Jetties are provided with transversal joints every 80/100 m; typical sections for jetties and Quays are shown in the following figures.

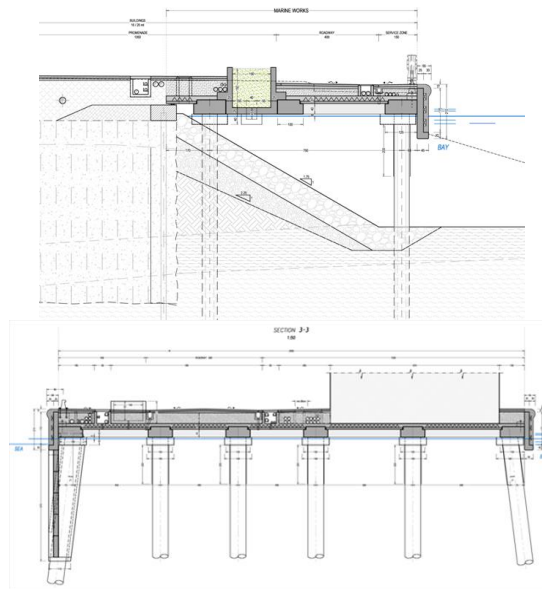


Figure 2. Quays and Jetties: typical section

3 SITE WIDE GEOTECHNICAL AND GEOLOGICAL CHARACTERISTICS

3.1 STRATIGRAHY

The ground conditions at Portonovi site consist of a bedrock overlain by sequences of Quaternary, or recent soil deposits having different origins. Site-wide ground investigation has been performed both into land and marine areas.

The bedrock consists of Flysch, a “weak rock”, which comprises inter-bedded layers of mudstones and sandstones. The Flysch bedrock has weathered over time to produce a residual clayey soil (HWF) which occurs as a “blanket” over the Flysch bedrock.

The uniaxial compressive strength of Flysch is within the range 3.6 to 10.8 MPa, The most frequent values of the uniaxial compressive strengths are in the range 4.5÷6.0 MPa. The calculated RQD values obtained from marine investigation in the first 15 m of the geotechnical formation vary mainly in the ranges from 0 to 40% with most frequent value (48%) in range 20-40%.

Quaternary Deposits, often referred to as “Terra Rossa”, are eluvial soils derived by the weathering of the limestone present higher up the mountainside above the site. Marine Deposits are alluvial material laid down over parts of the site by the effect of fluctuation of the sea. Man-made soils are present in more superficial stratum because of the reclamation efforts. Not cohesive strata have a friction angle varying in the range 28°-35°; the maximum value c' is equal to 20-30 kPa in the cohesive strata.

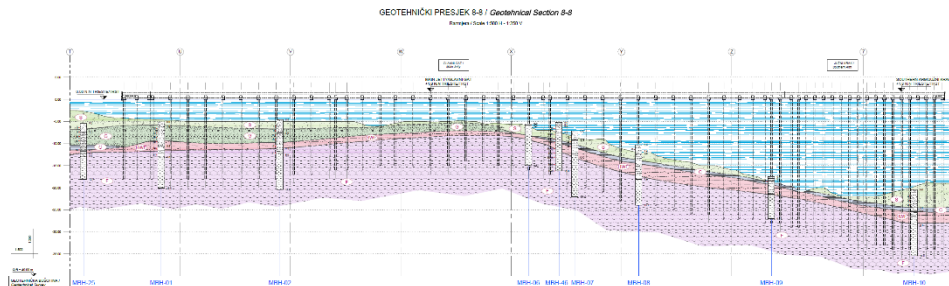


Figure 3. Geotechnical section along the Main Jetty

3.2 SEISMICITY AND LIQUEFACTION PROBLEMS

The site is located in the part of Montenegro with the highest seismicity. The most recent major earthquake took place in 1979 in Budva located approximately 56 km from the site and measured a magnitude of 7.0 on the Richter scale. Three further earthquakes with magnitudes between 4.0 and 7.0 have been recorded in 1979, 1984 and 1992 within 100 km radius from the site. In the Marina area the expected peak ground accelerations is 0.28g. In the zones where marine deposits have a thickness greater than 4÷5 m this value was amplified to take in account a local amplification of a seismic event.

Due to the high seismicity and the presence of fine loose materials under the sea water level, liquefaction analyses have been performed in order to verify the effective risk in the Marina area and to quantify the extent of the liquefiable layers.

The factor of safety (FS) against liquefaction triggering has been calculated from SPT, CPT and Shear Wave velocity (VST) results, following the recommendations of Idriss and Boulanger (2008); in particular, the calculation of the stress reduction factor r_d has been performed using the NCEER method. The obtained results demonstrate that potentially liquefiable soils are present mainly in the upper sand layers. The maximum depth of liquefaction obtained from the in-situ tests examined is variable over the whole area; in the design analyses all the cohesionless strata above the HWF and above cohesive units were assumed liquefiable.

4 FOUNDATION PILES

4.1 DISEGN CRITERIA

Structural numerical models were developed in order to design all the structures and the piles; the models analyze the behavior of Jetties and Quays under design loads by means of linear elastic analysis. Refer to § 6 for structural analysis details.

The geotechnical verifications included:

- evaluation of horizontal pressures on piles due to liquefaction occurrence;
- evaluation of negative skin friction on piles due to settlements after liquefaction event;
- calibration of the parameters for the linear elastic constrains to be included in the numerical models in order to consider the interaction between piles under horizontal loading and the surrounding soil;

- development of the axial bearing capacity curves (compression and tension) for piles with diameter $\text{Ø}600$ mm and $\text{Ø}800$ mm;

The depth of potential liquefiable soils is estimated for each section of Jetties and Quays; each pile in the structural model is loaded with an equivalent force equal to 30% of lithostatic pressure along the pile (as suggested in JRA, 1996-2002). The liquefaction phenomena is not combined with maximum seismic peak acceleration according to main literature references – see figure below (cfr. Subhamoy Bhattachary, Domenico Lombardi – 2012). All verifications have been carried out according to Eurocodes and to API RP WSD 2007 recommendations.

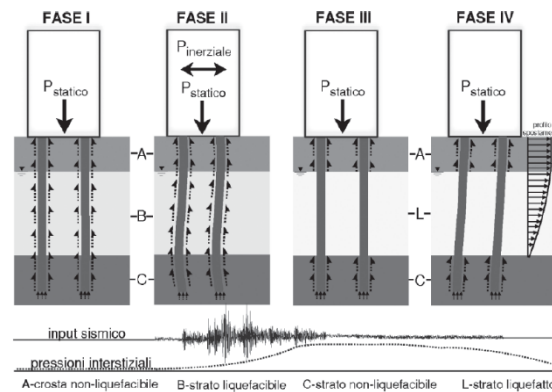


Figure 4. Different stage of loading on pile foundation during an earthquake- qualitative trend of the excess pore pressure due to liquefaction

Deep foundations of decks extend through potentially liquefiable sand layers near the seabed surface and are socketed in more competent layers (Flysch bedrock). When liquefaction occurs, the skin friction in the liquefied layer would be expected to decrease to zero and, as the liquefiable layer settles, negative skin friction could develop around the pile in this layer. Vertical ground settlement will occur as excess pore-water pressures induced by liquefaction dissipate, resulting in down-drag loads on and loss of vertical support for deep foundations.

The down-drag load (or skin friction) within a liquefied soil increases over time as excess pore water pressures dissipate (effective stresses increase) during the sand reconsolidation process after liquefaction. The down-drag load has been calculated assuming the conservative hypothesis that $ru=0$, corresponding to the end of the reconsolidation process. Down-drag loads are a consequence of seismic shaking, then they should not be applied in conjunction with the design seismic loads, because they will not occur at the same time. The sum of the expected axial pile load and down-drag load from the liquefied layer has to be less than the pile bearing capacity.

4.2 CONSTRUCTION APPROACH AND EXPERIENCES

Concrete bored piles were preferred to driven piles, due to the presence of the aforesaid flysch bedrock under sea level, which comprises inter-bedded layers of mudstones and sandstones. As highlighted in § 3.1, the uniaxial compressive strength varies within the range 0.6 to 12.5 MPa and piles are socketed into the Flysch at least 5-6 m, as defined by calculations. It was not possible to further limit the depth of embedding of the piles tip, because the alteration of superficial Flysch strata cannot be predicted and piles work under relevant

compressive forces. Bored piles were executed by sea using pontoons equipped with crawl, hammer, drilling tools or by earth, where possible along West and North Quay. The figures below shows the piling works by sea and by earth.

The use of vertical piles in lieu of battered piles was considered the optimal solution, both for design and construction purposes, taking into account that:

1. all standards and codes (i.e. Eurocodes, Italian NTC 08. § 7.2.5), and specific recommendations for piers and wharves such as Polb 2009 (§§ 4.2 e 5.5.4) and Pola Seismic code 2010 (§ 1.4.1 pt. C) state that battered piles should avoided - if possible - in order to increase safety of constructions in seismic zone;
2. vertical piles offer also the possibility to simplify construction sequences with not negligible time and cost savings.

There were no relevant problems during execution with a daily production up to 3 piles within a day by rig installed on site.

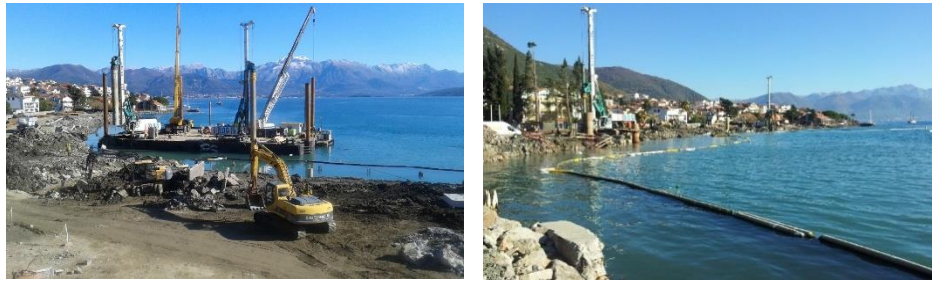


Figure 5. Piling works from pontoons and along the future quays

Structural numerical models were developed in order to design all the structures and the piles; the models analyze the behavior of Jetties and Quays under design loads by means of linear elastic analysis. Refer to § 6 for structural analysis details.

5 QUAYS DESIGN

5.1 DISEGN CRITERIA

The Quays were designed as opened quays and not linked to the ground behind them; a longitudinal joint separates quays from ground retaining structures. For this reason, piles were designed in order to withstand forces caused by self – weight and live loads applied on the deck and not for static and seismic ground pressures.

Ground behind quays was preliminary improved with specific deep soil mixing techniques (cfr. § 5.2) in order to create a continuous diaphragm wall.

The MIP wall works as a retaining structure to allow dredging necessary for the port, but at the same time it is conceived also as a liquefaction countermeasure against lateral spreading.

In fact, the cells of the containment wall isolate and enclose the potential liquefiable soil during an earthquake event in such a way that the soil contained within the grid cells will not liquefy. As a result, the wall constitutes a stable block capable to withstand the lateral forces due to the spreading of liquefied soil outside the structure itself.

The open quays solution was proposed, instead of gravity block walls initially designed, for several reasons:

1. the solution on piles is suitable in a port for nautical tourism in order to mitigate the effects of wave reflection inside the basin and to guarantee the optimum conditions for vessel berthing;
2. along the north quay the bedrock depth is continuously increasing in the east direction up to 15-16 m. The marine deposits above this bedrock are subjected to liquefaction. A design solution with a block wall founded on rocky strata should be not feasible or in any case of high impact on construction time and costs. The following figure shows that for values of a_g/g higher than 0.25 to 0.35 the minimum width B of the block wall is greater than the height and the work becomes inconvenient from the economical point of view;
3. if the concrete wall was founded on the marine sediments and not on bedrock, it would require a ground improvement under foundation extended also towards sea in order to avoid liquefaction and slope instability with related significant costs and time for execution;
4. due to the presence of a longitudinal joint along the quays, movements or liquefaction phenomena will not affect quays behavior during an earthquake;
5. it is possible to use a single structural solution for both jetties and for quays with optimization of times and execution costs;
6. the proposed solution allows to separate marine works and ground improvement works behind quays.

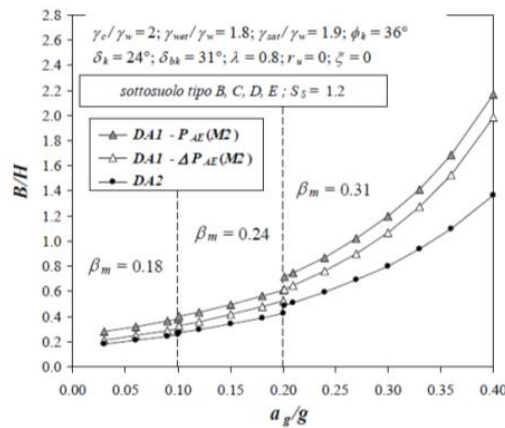


Figure 6. B/H ratio for walls vs a_g/g values

5.2 CONSTRUCTION APPROACH AND EXPERIENCES

The ground improvement along the quays was executed by means of deep mixing. The heavy-duty drilling rigs used for the site were equipped with three parallel continuous flight augers each with a diameter of 0,55 m. To execute the panels, the triple counter-rotating auger unit drills the ground whilst binder slurry is simultaneously injected. Once final depth is reached, the soil-cement mixture is homogenized by alternating rotation of the individual

augers and concurrent upward and downward movement of the entire auger assembly, depending on soil-cement properties (Figure 7).

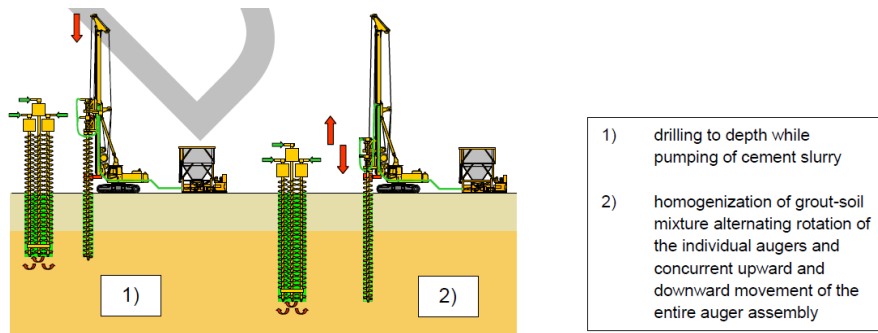


Figure 7. Production sequence for MIP

The result will be a continuous body of cemented soil, with dimensions defined by the geometry of the auger unit: a length of 1,70m (as measured from the outside of the outer augers) and a width of 0,55 m corresponding to the auger's diameter. When the panels form the continuous wall along the Quays the elements are performed fresh to fresh. Before hardening of the panels, steel beams were installed in order to guarantee the necessary bending stiffness to the ground improved panels (Figure 8). All these steel beams were finished with a continuous reinforced concrete capping beam (Figure 9). The ground improvement works were completed on site with great efficiency, proving the advantages of the design choice compared to other conventional techniques (gravity wall, concrete diaphragm wall and so on).

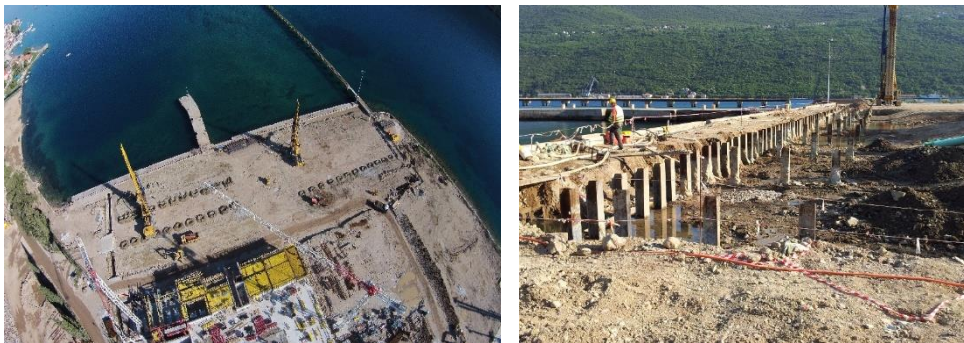


Figure 8. Ground improvement works along the shore line with deep soil mixing.



Figure 9. Capping beam of the future West Quay Wall

6 DESIGN OF DECK STRUCTURES

6.1 DESIGN CRITERIA

6.1.1 Structural analysis

In order to verify deck structures and piles, 3D finite elements models were developed for each part between two structural joints (structural analysis were performed with the aid of the software Sap 2000): piles and deck beams are modelled by means of beam elements while deck slab is modelled using plate orthotropic elements to take into account the mono-directional behavior of concrete slab on planks. Spring elements, which stiffness was evaluated using geotechnical data, are assigned at pile tips socketed into the flysch (F) strata.

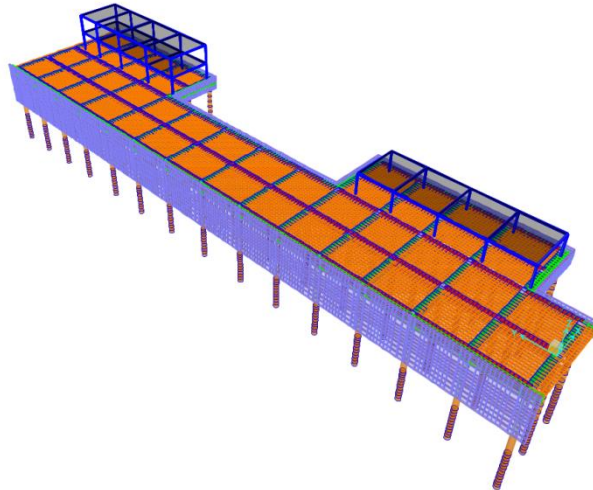


Figure 10. Example of Sap Model for structural analysis

6.1.2 Corrosion protection strategy

Corrosion of structures in the marine atmosphere and in the splash zone was achieved using the following strategy:

1. **Bored concrete piles protection:** re-bars in the concrete piles are protected by permanent casing and by means of a proper cover; the head of the casing in the splash zone (the most critical one) will be coated with an epoxy layer;
2. **Concrete elements protection:** the corrosion of reinforcements is prevented by:
 - ✓ using a concrete C40/50 with an exposure class XS3, providing an adequate concrete non less than 6 cm.
 - ✓ requiring a strict crack width limitation
 - ✓ using galvanized reinforcements for structural elements in contact with the wave splash (splash zone) and up to a thickness of 100mm from the face exposed to seawater.

7 CONCLUSIONS AND ACNOWLEDGES

Technical solutions designed for the new port of nautical tourism in Portonovi were described: solutions were specifically developed in order to face the challenging conditions related to geotechnical aspects, liquefaction, seismicity, durability, project requirements. Experiences on site demonstrated their validity and feasibility in terms of construction site organization time and cost savings.

The authors are grateful to Azmont Investments and to all Rare team for fruitful cooperation for the project development; we express our appreciation to Bauer and Aquamont teams for providing the necessary data and information concerning piling and grouting technologies and for their kind collaboration during the works.

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SANACIЈA NOSEĆE DRVENE KONSTRUKCIЈE KUĆE STEVANA MOKRANJCA U NEGOTINU

Rezime:

S-80

U radu je prikazan postupak sanacije noseće drvene konstrukcije kuće Stevana Mokranjca u Negotinu. Specifičnosti gradnje i oblikovanja konstrukcije karakteristične za tradicionalne stambene kuće građene sredinom XIX veka i kasnije, morale su se sačuvati s obzirom da objekat ima status spomenika kulture od izuzetnog značaja. Posebna pažnja je posvećena potpornoj konstrukciji (skeli) koja mora da obezbedi potpunu stabilnost dela konstrukcije objekta na kojoj nema intervencije. Ugradnjom posebno oblikovanih greda i ojačanja u pojedinim zonama međuspratne konstrukcije sačuvana je autentičnost enterijera kuće.

Ključne reči: drvena konstrukcija, skela, autentičnost

RENOVATION OF TIMBER STRUCTURES OF STEVAN MOKRANJAC HOUSE IN NEGOTIN

Summary:

The paper describes the process of renovation of timber structure of the house of Stevan Mokranjac in Negotin. Specifics of construction and design characteristic for the traditional residential houses built in the mid of nineteenth century and later, had to be preserved because the building has the status of cultural monuments of great importance. Special attention was paid to the supporting structure (scaffold), which must ensure complete stability of the part of house structure where there is no work. Incorporating a specially formed beams and reinforcements in certain areas of the floor structure, the authenticity of the interior of the house is preserved.

Key words: timber structures, scaffold authenticity

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1 UVOD

Kuća Stevana Stojanovića Mokranjca u Negotinu je rodna kuća istaknutog kompozitora i dirigenta i ima status spomenika kulture od velikog značaja. Nastala je sredinom 19. veka, a pouzdano se zna da je 1867. bila u vlasništvu Marije Mokranjac, što se vidi po tapiji iz 1889. godine. Kuća se odlikuje karakteristikama tradicionalne balkanske varoške stambene kuće, gde je u suterenu smešten podrum ozidan kamenom, dok je prizemni deo, namenjen stanovanju, sagrađen u bondručnom konstruktivnom sistemu (drvena skeletna konstrukcija sa zidnom ispunom u pečenoj ili nepečenoj opeci), a sačinjavale su ga, u to vreme, dve sobe i kuhinja. Kuća je 1964. godine adaptirana za potrebe memorijalnog muzeja posvećenog Mokranjcu, tako da su podrumске prostorije uređene za manje muzičke koncerte, dok su prostorije u prizemlju muzeološki opremljene i sadrže veći deo zaostavštine u muzičkim instrumentima i drugim predmetima koji su pripadali istaknutom kompozitoru i dirigentu. Sve prostorije opremljene su eksponatima iz tog perioda vezanim za život i delo Stevana Mokranjca i njegove porodice. Objekat je pravougaonog oblika u osnovi dimenzija 16,75x8,55-9,70m. Na severnoj strani objekta nalazi se drvene konstrukcije trema (doksat) površine 25,3m², koji vodi ka ulazu u prostor na spratu kuće. Veći radovi na uređenju kompleksa oko Mokranjčeve kuće u Negotinu izvedeni su 1980. godine, poslednji put. U ovom trenutku kuća ne radi kao muzej, ne prima turističke posete, zbog ozbiljnih oštećenja drvene međuspratne konstrukcije koja odvaja podrumsku od spratne etaže.



Slika 1 – Memorijalni muzej – kuća Stevana Mokranjca

2 OPIS POSTOJEĆEG STANJA

Postojeće stanje konstrukcije i određenih konstrukcijskih celina ogleda se u sledećim činjenicama:

- Podrum i prizemlje objekta podeljeni su klasičnom drvenom međuspratnom konstrukcijom (MSK) od hrastovih tesanih greda dimenzija 14/14-15cm koja je u veoma lošem stanju, Slika 2. Međuspratne grede (MSG) postavljene su na rasteru od 30-40cm i raspona 3,50m, statičkog sistema proste grede. Prilikom poslednje rekonstrukcije ovi konstruktivni elementi nisu adekvatno ni obrađeni ni zaštićeni, čak ni kora, ponegde, nije oljuštena u potpunosti, tako da su bile pogodno tlo za razvoj mikroorganizama, insekata, sipaca koji su u velikoj meri uticali na smanjenje nosivosti poprečnog preseka i pojavu

prekomernih ugiba i vibracija. Pored jasnih vizuelno uočljivih oštećenja, ove MSG se karakterišu i tupim zvukom pri udaru, što je još jedan pokazatelj dotrajalosti ovih elemenata, Slika 3.

- MSG se oslanjaju na ležišne podužne grede na kamenim podrumskim zidovima i na centralnu drvenu gredu postavljenu po sredini kuće oslonjenu na drvene stubove koji su raspoređeni na nejednakim međusobnim rastojanjima.



Slika 2 –Deterioracija neadekvatno zaštićene drvene međuspratne konstrukcije usled bioloških uticaja

- Preko MSG, napred opisanih, izveden je daščani sloj, preko koga je nanešena ispuna od pepela i šljake u vidu podne podloge preko koje je ugrađena završna podna obloga od dasaka tipa “brodski pod”. Daske u konstrukciji podne podloge pričvršćene za MSG su u stanju raspadanja, zahvaljujući aktivnosti crvotočine i sipaca tako da se pojedini elementi mogu rukom uklanjati.



Slika 3 –Međuspratna greda sa slikom tipičnih oštećenja izazvanih dejstvom insekata, lošom preventivnom zaštitom i selekcijom materijala

- U delu hodnika i prostora ostave, podna obloga se sastoji od pečenih opeka, koje su postavljene u sloju maltera direktno preko daščane podloge bez ikakvih izolacionih slojeva.

- U centralnom delu dužine hodnika smeštena je zidana, veoma teška i prostrana konstrukcija ložišta-odžaklije, Slika 4. Sve to je oslonjeno na MSG čiji se nivo oštećenosti i dotrajalosti jasno vidi sa pozicije poda podruma, kao i prekomerna deformacija kojom se one karakterišu. Kota poda ispred ložišta je na nižem nivou u odnosu na kotu istog poda u drugim prostorijama za više od 10 cm, kao direktna posledica te deformacije.
- Prostorije koje se nalaze u neposrednoj blizini ognjišta-odžaklije imaju nagnute podove prema ognjištu kao posledicu prekomernih deformacija čitave MSK usled velikog koncentrisanog opterećenja.
- Duž hodničkih zidova uočene su pukotine u podnoj oblozi od opeke iz navedenih razloga. Na osnovu detaljnih merenja i statičke analize došlo se do zaključka da je ova zona najugroženija i najnestabilnija, a jedan od uzroka se može pronaći u podrumu kuće.



Slika 4 –Ognjište – odžaklija u centralnom delu kuće

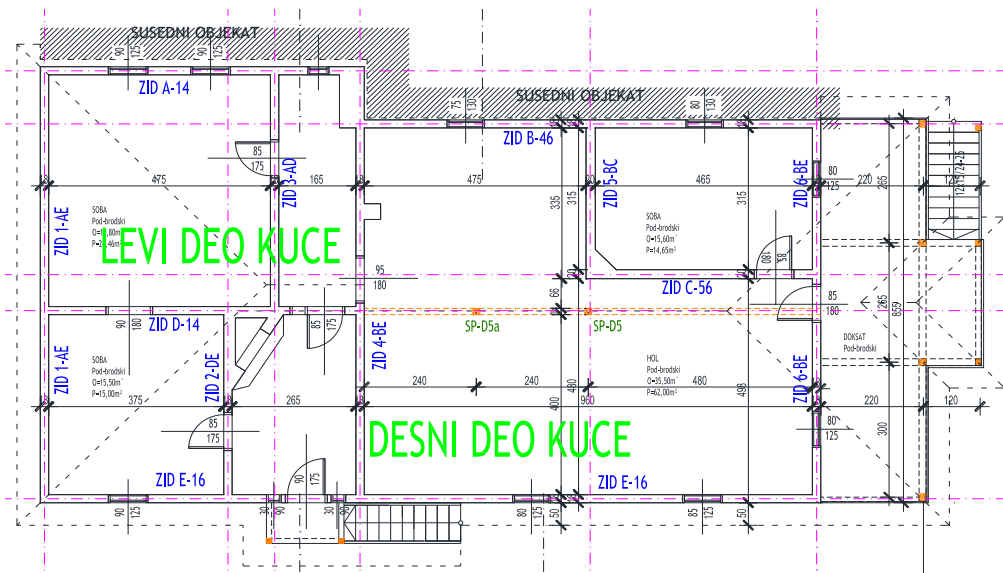
- U podrumu, u polovini rastojanja između nosećih podužnih zidova, preko niza stubova pruža se centralna greda od hrastovine dimenzija poprečnog preseka $b/h=25/25\text{cm}$ koja prihvata međuspratne grede, odnosno polovinu ukupnog tereta koji dolazi od prizemlja kuće, od tavanice i od krovne konstrukcije. U zoni oslanjanja centralne grede evidentne su strukturne nepravilnosti drveta koje se pre svega ogledaju u prisustvu čvorova dužine 30-40cm i izrazito naglašene anizotropije po dužini elementa. (Slika 5).
- Centralna greda, usled značajnih fizičkih i strukturnih oštećenja, ima veliki ugib, izazvan teretom prizemlja, odnosno opterećenjem od zidane konstrukcije ložišta koja se nalazi neposredno iznad, upravo u toj zoni. Drveni stubovi dimenzija $b/h=14/16$ kao i naglavni drveni jastuci istih dimenzija i materijala u sličnom su stanju kao i ostali oštećeni elementi konstrukcije, napadnuti crvotočinom, sipcima i truleži.
- Centralna greda se nastavlja i u drugom delu objekta, po celoj dužini i ima istu dispoziciju, ali je u mnogo boljem stanju, s obzirom na kvalitetniju hrastovu građu od koje je izvedena ali i zbog manje opterećenosti.
- Pregledom objekta ustanovljeno je da ne postoji mogućnost adekvatnog prirodnog provetravanja podrumskog prostora što treba na odgovarajući način rešiti, kako bi novi rekonstruisani elementi imali povoljnije eksploatacione uslove.



Slika 5 – Oštećenja centralne, noseće grede MSK

3 POSTUPAK SANACIJE I REKONSTRUKCIJE

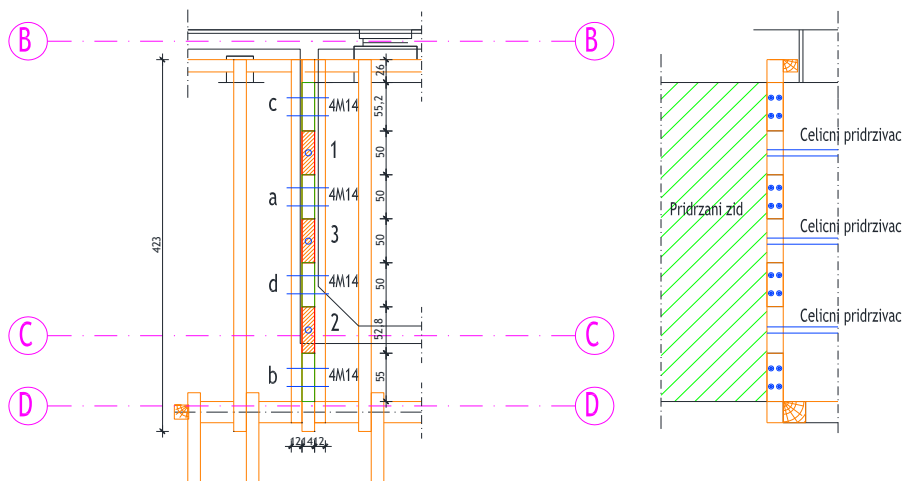
Sanaciona procedura u projektu sanacije i rekonstrukcije je prikazana kroz tekstualnu i grafičku dokumentaciju. Svaka pozicija je detaljno opisana i praćena odgovarajućim skicama i detaljima. Objekat je podeljen na dva dela (Slika 6) koji su posmatrani ponaosob i na osnovu kojih su date dve procedure sanacije. U daljem, one će biti date po redosledu koji se mora poštovati prilikom izvođenja radova.



Slika 6 – Osnova sprata

Postupak sanacije MSK salona i dnevnog boravka („desni deo kuće“) sastoji se u sledećim koracima i otpočeće nakon preseljenja svih eksponata na adekvatnu lokaciju:

- uklanjanje daščane podne obloge u potpunosti zajedno sa ispunom od šljake i pepela;
- uklanjanje daščane podne podloge pričvršćene za međuspratne grede;
- izrada potporne konstrukcije (skele) od čeličnih cevastih elemenata ispod svih pregradnih zidova;
- uklanjanje jednog dela starih MSG u zoni oko poduprtih zidova;
- izrada sloja za naleganje preko dela podrumskih kamenih zidova o koji će se osloniti MSG od tesane hrastovine, dimenzija poprečnog preseka $b/h=12/14\text{cm}$, u cementnom malteru i oblaganje istog bitumeniziranom Al-folijom kako bi se sprečilo higroskopsno upijanje vlage drveta iz cementnog maltera;
- ugradnja specijalno projektovanih i oblikovanih greda složenog poprečnog preseka ispod poduprtih zidova (Slika 7). Nove grede su od tesane hrastovine vlažnosti $w<18\%$, zaštićene odgovarajućim sredstvima od gljivica i insekata. Osnovna zaštita svih elementa je premazivanje u dva sloja zaštitnim i impregnacionim sredstvom;



Slika 7 –Greda složenog poprečnog preseka ispod poduprtog zida

- uklanjanje podupirača;
- uklanjanje preostalog dela starih MSG u ovom delu kuće;
- izrada sloja za naleganje preko ostalog dela podrumskih kamenih zidova o koji će se osloniti nove MSG od tesane hrastovine, u cementnom malteru i oblaganje istog bitumeniziranom Al-folijom;
- ugradnja novih MSG od tesane hrastovine, na osovinskom rasteru od $e=40\text{cm}$, dimenzija poprečnog preseka $b/h=12/14\text{cm}$;
- ugradnja podne konstrukcije. Podna konstrukcija se sastoji od specijalno profilisanih drvenih talpi četinara I klase debljine 45mm i širine 12-14cm, koje se međusobno uklapaju vezom na pravi sučeljak ili pero-žljeb. Ove daske se direktno pričvršćuju za nove MSG postavljene na rasteru $e=40\text{cm}$. Podna konstrukcija takođe treba biti zaštićena odgovarajućim premazima.

- detaljima koji se odnose na oslanjanje grede njenim krajevima o podrumski zid, konstrukciju u zoni granice između „levog“ i „desnog“ dela kuće i o stub;
- ugradnja specijalno projektovanih i oblikovanih greda složenog poprečnog preseka ispod pozicija pregradnih zidova, u svemu prema datim grafičkim priložima i opisanoj proceduri. Nove grede su od tesane hrastovine, osušene, zaštićene odgovarajućim sredstvima od žižka, insekata i mogućnosti razvoja gljivičnih kolonija i premazanih u dva sloja zaštitnim i impregnacionim sredstvom;
 - ugradnja novih MSG od tesane hrastovine, na osovinskom rasteru od $e=40\text{cm}$, dimenzija poprečnog preseka $b/h=12/14\text{cm}$, slika 8;
 - ugradnja novih posebnih MSG od tesane hrastovine, dimenzija poprečnog preseka $b/h=12/14\text{cm}$, u zoni ložišta;
 - zidanje novih pregradnih zidova od iste opeke u debljini od 12cm;
 - uklanjanje potporne konstrukcije (skele) i prenošenje opterećenja od krovne konstrukcije na novoizvedene konstrukcijske elemente;
 - ugradnja podne konstrukcije tipa „brodski pod“ u sobnom prostoru. Podna konstrukcija se sastoji od specijalno profilisanih drvenih talpi četinara I klase debljine 45mm i širine 12-14cm, koje se međusobno uklapaju vezom na pravi sučeljak ili pero-žljeb. Ove daske se direktno pričvršćuju o nove MSG postavljene na rasteru $e=40\text{cm}$. Podna konstrukcija se takođe štiti odgovarajućim premazima;
 - ugradnja daščane podloge u zoni hodnika i kuhinje, hidroizolacione obloge preko nje i izrada podne obloge od pune opeke u cementnom malteru;

4 ZAKLJUČAK

Izložena procedura sanacije i rekonstrukcije noseće drvene konstrukcije kuće Stevana Mokranjca u Negotinu predstavlja optimalan i realno izvodljiv građevinski postupak. Prilikom osmišljavanja rešenja kojima se omogućavaju određeni postupci zamene konstrukcijskih elemenata vodilo se računa o preporukama Zavoda za zaštitu spomenika kulture iz Niša o očuvanju autentičnosti konstrukcije objekta, oblikovanju konstrukcijskih elemenata i samog procesa gradnje karakterističnog za sredinu XIX i početak XX veka na prostorima Srbije. Postoji strepnja da će veliki problem biti izbor kompetentnog, referentnog i stručnog izvođača predloženih postupaka sanacije noseće drvene konstrukcije kuće imajući u vidu delikatnost pojedinih izvođačkih pozicija, kao i kvalitet drvene građe koja mora biti pažljivo odabrana i obrađena.

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Duška Nedeljković¹, Jelena Bojić²

IZRADA PRIBOJA U ZONI STUBA S3, ŽEŽELJEV MOST, NOVI SAD

Rezime:

S-81

Glavnim projektom mosta predviđeno je da se za fundiranje rečnog stuba (stub S3) iskoristi postojeći temelj srušenog "Žeželjevog" mosta. U radu je prikazan postupak istražnih i pripremnih radova koji su prethodili radovima na izradi priboja u zoni stuba S3 kao i radovi na samoj izradi priboja. Sve radove je izvodila GP "Mostogradnja".

Ključne reči: Žeželjev most, Istražni radovi, Pripremnih radovi, Priboj, Mostogradnja

CONSTRUCTION OF A COFFERDAM IN ZONE OF PIER P3, ŽEŽELJ BRIDGE, NOVI SAD

Summary:

Final Design of the bridge anticipates the use of existing foundation of the demolished "Zezelj" bridge for founding of river pier (pier P3). This work presents procedure of investigations and preparatory works that preceded to works on cofferdam construction in zone of pier P3, and works on cofferdam construction. All works has been done by GP "Mostogradnja".

Key words: Žeželj bridge, Investigations, Preparatory works, Cofferdam, Mostogradnja

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1 TEHNIČKI OPIS I KARAKTERISTIKE MOSTA

Glavnim projektom mosta predviđeno je da se za fundiranje rečnog stuba (stub S3) iskoristi postojeći temelj srušenog "Žeželjevog" mosta. Postojeći temelj je masivni armirano-betonski element gabaritnih dimenzija u osnovi 24,50 x 39,50m, sa zaobljenjima $R=12,25m$ na uzvodnoj i nizvodnoj strani. Donja ivica temelja je na koti 55,00m.n.m. a gornja ivica po obimu temelja na koti 61,50m.n.m. Najviši deo gornje površine temelja, koja je formirana prema statičkim zahtevima i tehnologiji izgradnje srušenog mosta, nalazi se na koti 64,00m.n.m. Dno korita reke na mestu postojećeg temelja stuba S3 je, zbog ostataka srušene konstrukcije u toj zoni, veoma neravno - uglavnom je bilo između kota 66,00 - 68,00m.n.m.

2 ISTRAŽNI RADOVI

Radi izvođenja radova na priboju oko postojećeg temelja stuba S3, pre svega je urađeno pozicioniranje postojećeg temelja, pobijanje pomoćnih čeličnih šipova, pobijanje čeličnih talpi i izrada šipova postupkom mlaznog injektiranja. Bilo je neophodno da se u zoni ovih radova prethodno očisti dno, odnosno da se uklone delovi srušene konstrukcije mosta kao i druge veštačke ili prirodne prepreke.

Pre početka radova na čišćenju dna izvršeni su istražni radovi. Ti radovi su obuhvatili:

- Batimetrijsko snimanje dna: Batimetrijski snimak tla obuhvatio je korito reke u široj zoni mosta. Na osnovu ovog snimka izvučeni su profili korita reke Dunav (sa upisanim visinskim kotama) u zoni stuba S3.
- Geoistražni radovi sondažnim bušenjem: izvršeno je na lokacijama koje su određene na osnovu podataka dobijenih batimetrijskim snimanjem dna. Posebna pažnja je posvećena istražnim bušotinama na mestima karakterističnih uzvišenja koja su se nalazila na više mesta u zoni postojećeg temelja.
- Snimanje dna pomoću ronioca.

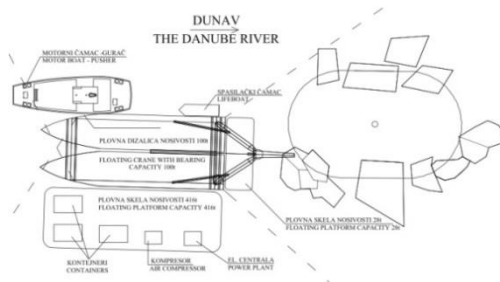
Po završetku istražnih radova, konstatovano je prisustvo većih i manjih betonskih blokova (ostataka srušene konstrukcije) u zoni oko postojećeg kesona stuba S3. Da bi se pristupilo izradi priboja oko kesona, bilo je potrebno je izvršiti uklanjanje betonskih elemenata.

3 IZRADA PRIBOJA U ZONI STUBA S3

3.1 PRIPREMNI RADOVI

Zona uklanjanja ostataka starog mosta sa dna korita reke Dunav je određena na osnovu položaja priboja stuba S3. Čišćenje je izvršeno u širini od tri metra po obimu kesona odnosno po 1,5m unutar i izvan priboja.

Uklanjanje betonskih elemenata je vršeno pomoću plovne naprave, koja je u vlasništvu GP "Mostogradnja" AD, "Miljacka" nosivosti 135t. Sa nizvodne strane na plovnu napravu bila je pričvršćena plovna skela, na koju je namontirana konstrukcija za spuštanje i podizanje čelične cevi $\varnothing 1600mm$, koja je služila za zaštitu ronioca.



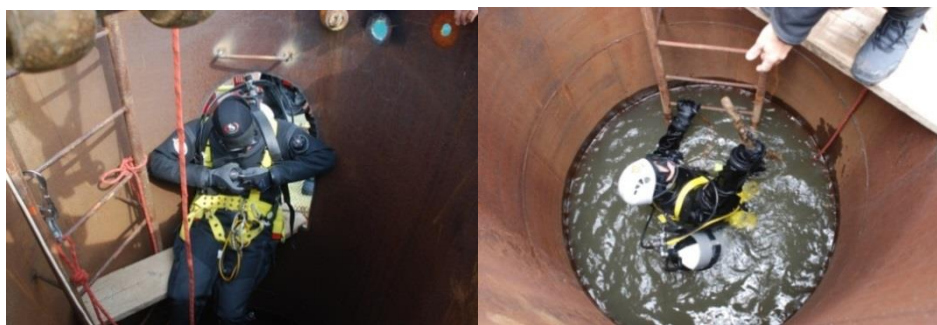
Položaj i raspored plovila

3.2 UKLANJANJE BETONSKIH ELEMENATA

Usled visokog vodostaja reke Dunav, podvodni radovi izvođeni su u veoma opasnom okruženju po ronioce:

- Veoma mala vidljivost pod vodom (5-10 cm);
- Reka je nosila granje, debla i ostale materijale opasne po ronioce i opremu kako po površini reke tako i ispod nje;
- Velika brzina toka reke.

Zbog svega gore navedenog ronioци su se spuštali na poziciju rada tj. na dno korita reke kroz radnu čeličnu cev Ø1600mm. Na dnu cevi je bio otvor kroz koji su ronioци izlazili i vršili inspeksijske radove. U toku izvođenja radova sve vreme su bili vezani za radnu cev i izvodili radove do 3m nizvodno od radne cevi. S obzirom na takvu tehnologiju, definisani su tačni položaji radne cevi u 48 tačaka, tako da se pokrije cela zona radova.



Ulazak ronioца u radnu cev

Kompletna trajektorija pretražene trase oko kesona u širini od 3m bila je prekrivena armirano betonskim elementima, manjih i većih dimenzija, povezanih međusobno armaturom i ručno nepomerljivih. Takođe, trasa je bila prekrivena drobljenim betonom i kamenim nabačajem manjih i većih dimenzija.

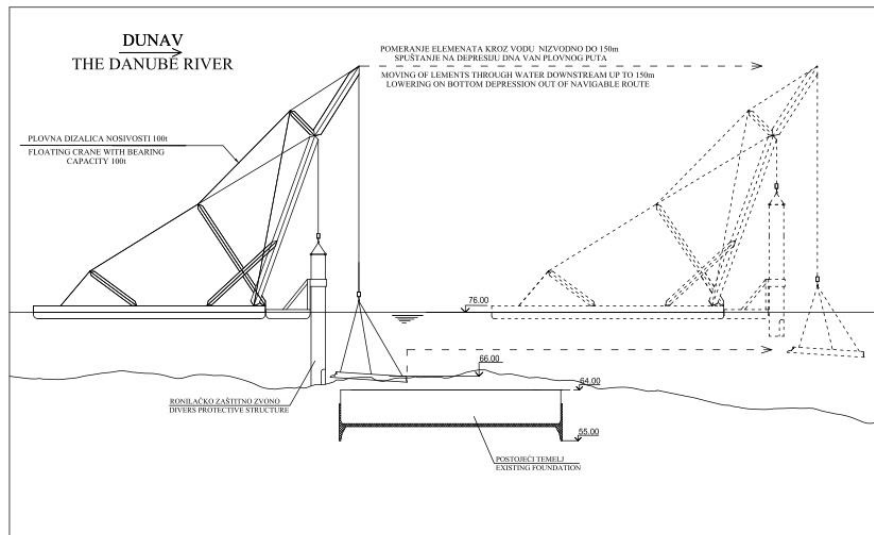
Svi elementi su bili prekriveni nanosima peska i nalazili su se zatrpani u materijalu na dnu reke.

Armatura na koju je ronilačka ekipa nailazila je bila iskidana i zgužvana i dosta otežavala rad ronionicima, jer se morala obratiti dodatna opreznost, kako ne bi došlo do povređivanja. Velika količina armature je morala biti iščupana i isečena da bi se betonski elementi mogli osloboditi i ukloniti.

Postupak vađenja betonskog bloka:

Pošto su plovila postavljena u određeni položaj uz geodetsko praćenje, plovna dizalica je spuštala radnu čeličnu cev prečnika Ø1600mm na dno, nakon čega su ronionci silazili i vršili inspekciju. Posle ronilačkog pregleda, pristupalo se podvodnom bušenju rupa i ugradnji ankera u betonske blokove. Ronionci su na ankere montirali ankerne ploče koje su šeklama vezivali za noseću užad. Broj i raspored ankera je zavisio od veličine i položaja svakog elementa.

Nakon kačenja i podizanja betonskog elementa dizalica ga je transportovala do depresije koja se nalazi 150m nizvodno od kesona stuba S3, a koja je otkrivena prilikom batimetrijskog snimanja dna.



Transport AB elementa

Više puta je pokušavano da se izvuku kompletni betonski blokovi čija nam težina nije bila poznata, s obzirom da su bili zatrpani i zasuti materijalom. Zbog nepoznate težine elemenata često su se kidale sajle i izvlačili ankeri iz betona. Da bi se takvi elementi izvukli, vršeno je „usisavanje“ materijala iznad tih elemenata, štemovanje, sečenje armature i usitnjavanje krupnih betonskih blokova.



Transport betonskog bloka

Manji betonski blokovi su izvlačeni i odlagani na plovne skele a kasnije transportovani i odlagani u depresiji koja se nalazi 150 m nizvodno od zone stuba S3.

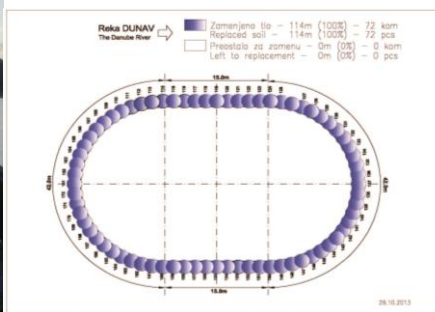


Vađenje manjih betonskih blokova

3.3 DUBINSKO UKLANJANJE BETONSKIH ELEMENATA-ZAMENA TLA U ZONI PRIBOJA

Posle uklanjanja velikih betonskih blokova u zoni stuba S3 neophodno je bilo ukloniti i sitnije ostatke starog mosta – armaturu, manje AB elemente, drobljeni kamen i sl. i izvršiti zamenu tog materijala šljunčanim materijalom kako bi se stvorili uslovi za pobijanje čeličnih šipova i talpi.

Zona dubinskog čišćenja dna korita reke Dunav je određena na osnovu položaja budućeg priboja oko postojećeg temelja stuba S3. Očišćena je zona u širini od 2,2 metra pored temelja starog mosta. S obzirom na takvu tehnologiju, definisani su tačni položaji radne cevi u 72 tačke, tako da se uklanjanjem „pokrije“ cela zona radova. Radovi su vršeni HW opremom teškom 30t. Navedena oprema je bila vođena posebnom konstrukcijom postavljenom na plovni objekat.

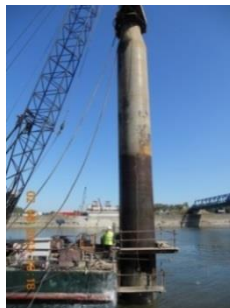


Posebna konstrukcija za vođenje radnecevi – Šema položaja radne cevi

U okviru radne cevi Ø2200mm vršio se iskop grajferom teškim 7t do dubine prirodnog sloja do maksimalno 5m dubine ispod rečnog dna. Prilikom spuštanja grajfera slobodnim padom, materijali u zoni iskopa (ostaci srušene konstrukcije mosta, kameni nabačaj, drveni šipovi) bivaju „smrvljeni“. Nakon iskopa u cev je sipan šljunčani materijal.



Grajfer



Radna cev

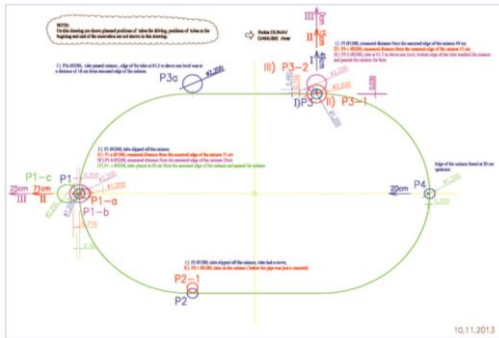


Izvađena armatura

3.4 UTVRĐIVANJE POLOŽAJA TEMELJA STAROG MOSTA

Utvrdjivanje položaja postojećeg temelja stuba se vršilo po HW tehnologiji pomoću radne cevi prečnika Ø1200mm.

Praćenje pozicioniranja cevi u projektovanu tačku vršili su geometri. Cev je pozicionirana tako da se položaj centra cevi poklapa sa položajem merne tačke. S obzirom da je vršeno čišćenje dna Dunava samo u zoni pobijanja talpi i položaj kesona je određen pobijanjem cevi Ø1200 pored temelja kesona.



Položaj izvršenih bušotina



Utvrdjivanje položaja dve radne cevi

Nakon pozicioniranja cevi iznad kesona vršen je iskop grajferom do kote postojećeg temelja. Posle iskopa grajferom ronionci su silazili do kote iskopa i čistili materijal do površine postojećeg temelja (kesona). Za čišćenje su korišćene pumpe koje usisavaju materijal i izbacuju ga van radne cevi. Nakon čišćenja stvarali su se uslovi za podvodno utvrđivanje položaja kesona u odnosu na pobijene cevi. Pošto nam je položaj cevi ranije definisan (geodetskim navođenjem) definisali smo i položaj ivice postojećeg temelja stuba za tu mernu tačku.



Priručni alat za geodetsko određivanje položaja šipa



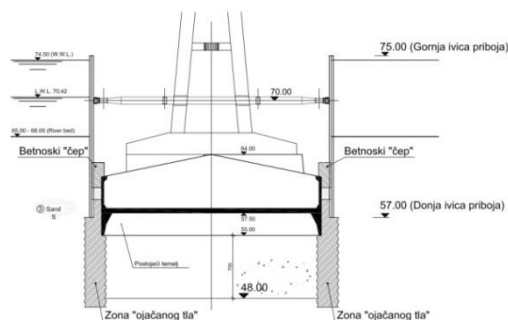
Ronilac unutar radne cevi

Oprema i mehanizacija za bušenje, tehnološki postupak i organizacija radova je ista kao za fazu zamene tla tj. u tački 3.3

Na osnovu ukupno 11 mernih tačaka definisan je stvarni položaj kesona. Utvrđeno je da je stvarni položaj kesona za **20 cm** translatorno pomeren **uzvodno** od projektovanog položaja.

3.5 IZRADA PRIBOJA U ZONI STUBA S3

Priboj se, kao celina, sastoji od čeličnih delova (zida priboja od talpi i ramovskog razupirača) i betonskih delova, "betonske zavese" izvedene postupkom mlaznog injektiranja i "čepa" od podvodnog betona koji služi kao zaptivka i preko koga se talpe oslanjaju na postojeći temelj.



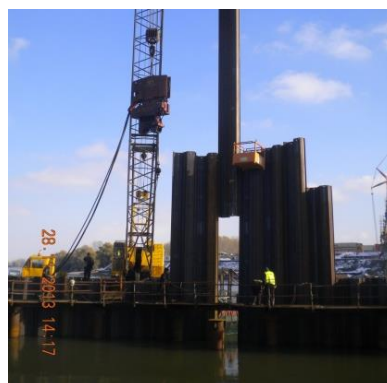
Dispozicija priboja

Posle završetka svih pripremnih radova, (uklanjanja velikih betonskih blokova, zamene materijala, pozicioniranja kesona) započeta je izrada priboja.

Radovi na pobijanju šipova po obimu kesona i za oslanjanje pomoćnog rama trajali su 8 dana, a iskorišćeni su i za kontrolu položaja kesona. Pobijeno je ukupno 32 šipa i postavljen je pomoćni ram za pobijanje talpi-zida priboja (obim priboja je 114m). U periodu od 10 dana pobijeno je 190 čeličnih talpi dužine 18 m. Pobijanje talpi na projektovanu kotu trajalo je još 18 dana.



Šipovi po obimu kesona i pomoćni ram



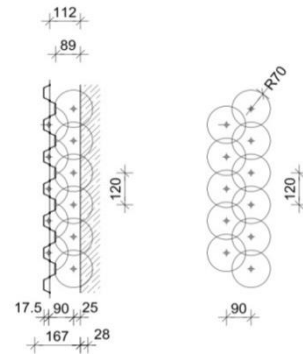
Zatvaranje zida priboja

Nakon završetka izrade zida od čeličnih talpi počeli su pripremi radovi za izradu šipova mlaznim injektiranjem (Priprema platforme za mašinu za mlazno injektiranje itd.).

Izrada šipova mlaznim injektiranjem vršila se sa plovne skele. Ukupno je urađeno 190 šipova. Veliki problem predstavljali su armatura i užad za prednaprezanje koji su virili iz temelja starog stuba. Često je dolazilo do oštećenja cevi kojom su injektirani šipovi.



Izrada šipova mlaznim injektiranjem



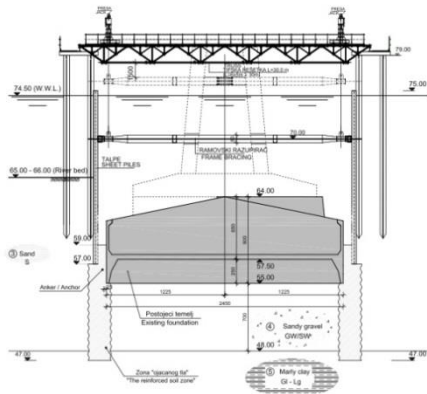
Dispozicija šipova

Ramovski razupirač je zbog svoje velike težine transportovan do priboja iz tri dela pomoću plovne skele „Miljacka“. Spajanje je izvršeno na licu mesta u zoni priboja stuba S3.



Montaža ramovskog razupirača

Za spuštanje ramovskog razupirača na projektovanu kotu izrađena je posebna pomoćna konstrukcija koja se sastojala iz četiri baterije šipova spojenih tipskim rešetkama. Spuštanje ramovskog razupirača je obavljeno opremom za „heavy lifting“ I to pumpa + presa nosivosti 2000 kN – 4 komada. Ukupna težina ramovskog razupirača je 1100kN + 300kN , tako da opterećenje po jednom kompletu iznosi 350kN. Prilikom spuštanja ram je bio okačen u četiri tačke iznad kojih se nalazio po jedan komplet „heavy lifting“ opreme. Pre spuštanja razupirača namontirani su teleskopski (podesivi) oslonci koji prenose opterećenje sa talpi na ram. Radove na podešavanju oslonaca izvodili su ronionci uz nadzor tehničkog osoblja Mostogradnje.



Skica konstrukcije za spužtanje rama



Montaža tipskih rešetki



Montaža tipskih rešetki



Kačenje rama

Nakon spužtanja ramovskog razupirača a pre crpljenja vode iz priboja Mostogradnja je izvršila betoniranje „čepa“. Za betonažu „čepa“ angažovana je stabilna pumpa koja je pumpala beton kroz ~200 m dugačak cevovod koji je išao preko pristupnog mosta do plovne skele na kojoj se nalazila autopumpa.



Podvodno betoniranje „čepa“



Priboj nakon ispužavanja vode

Vicko Letica¹

PRIKAZ GLAVNOG PROJEKTA BRANE I MALE HE ČIFLIK NA RECI NIŠAVI

Rezime:

S-82

Brana sa pripadajućim objektima, HE Čiflik, locirana je u blizini sela Čiflik na reci Nišavi, uzvodno od Bele Palanke. MHE Čiflik je protočna elektrana, pribranskog tipa. Brana je konstruktivno podeljena dilatacionim spojnicama na pet odvojenih blokova. Ukupna dužina betonskih struktura, u koje ulaze i delovi ukorporirani u nasutu branu iznosi 90,00 m. Maksimalna visina brane iznosi 14,50 m. Maksimalna denivelacija uzvodnog i nizvodnog nivoa vode pri zatvorenim ustavama iznosi 5,05 m, za radni nivo u akumulaciji.

Ključne reči: brana, hidroelektrana, beton, nasip

REVIEW OF THE FINAL DESIGN OF THE SMALL HPP CIFLIK ON NISAVA RIVER

Summary:

Small storage hydropower plant Ciflik, dam and appurtenant structures are located near the village Ciflik, on Nisava River, upstream of Bela Palanka. The dam is divided into five blocks by expansion joints. The total length of the concrete structure, with components which are part of the earthfill dam, is 90.00 m. The maximum height of the dam is 14.50 m. Maximum difference between headwater and tailwater for reservoir operating conditions (with lowered gates) is 5.05 m.

Key words: dam, hydropower plant, concrete, embankment

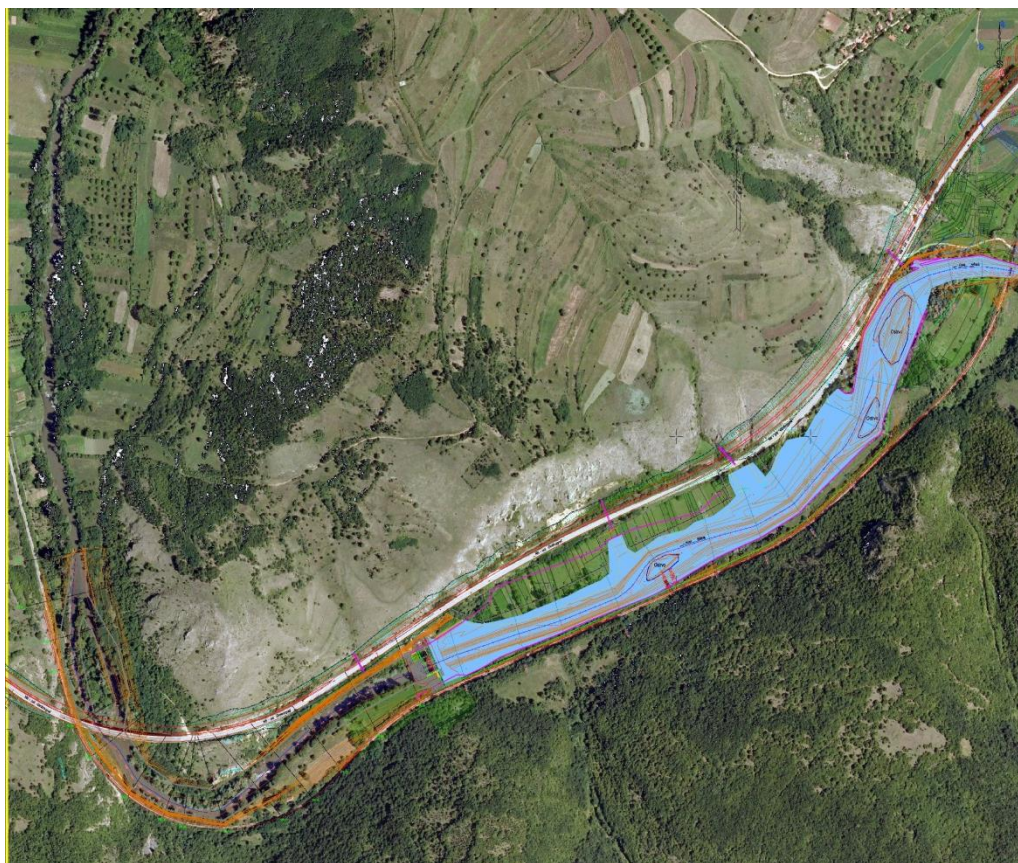
¹ Mr.dipl.grad.inž., Beograd BRANE-HPP-WTP. d.o.o office@brane-hpp-wtp.com

1. OPIS

Brana sa pripadajućim objektima, HE Čiflik, locirana je u blizini sela Čiflik na reci Nišavi, a uzvodno od manastira Sveti Otac. Lokacija se nalazi uzvodno od Sićevačke klisure.

Do lokaliteta, praktično, postoji međunarodna saobraćajnica Niš-Bela Palanka-Pirot-Sofija koja je udaljena oko 100 m od desne obale. Postoji takođe put levom obalom, od mesta Čiflik do lokacije elektrane u dužini od 800 m sa makadamskom podlogom. Lokacija je između saobraćajnice i pruge i van koridora postojećih i planiranih infrastrukturnih objekata, kao i izvan koridora 10.

MHE Čiflik je protočna hidroelektrana pribranskog tipa, projektovana tako da objekti za usporavanje rečnog toka i hidroelektrana čine jednu funkcionalnu celinu. Izgradnjom ovih objekata usporiće se vodni tok reke i u pregradnom profilu izdići će se na kotu 313,00 m.n.m. Sa ciljem da se poveća pad hidroelektrane, rečno korito nizvodno od pregradnog profila je prokopano.



Brana je konstruktivno podeljena dilatacionim spojnica na pet odvojenih blokova. Veza levog boka kanjona ostvaruje se sa betonskim gravitacionim blokom. Između ovog bloka i

prelivenog bloka sa radijalnim ustavama, nalazi se gravitacioni betonski blok u koji su smešteni dopunski slobodan preliv i riblja staza. Uz glavni prelivni blok sa kontrolisanim propuštanjem vode, koji je smešten u centralnom delu, locirana je mašinska zgrada. Montažni blok se naslanja na mašinsku zgradu, a nalazi se u telu nasute brane, koja se proteže do desnog boka kanjona. Veza nasutog dela i betonskog gravitacionog, ostvaruje se uz pomoć potpornih zidova. Uz montažni blok locirana je i administrativna zgrada.

Betonska brana je fundirana na koti 305,00–302,50 m.n.v. i ima ukupnu dužinu betonskog dela u reci bez delova za vezu sa nasutom branom 48,10 m. Ukupna dužina betonskih struktura, u koje ulaze i delovi ukorporirani u nasutu branu iznosi oko 90,00 m. Maksimalna visina brane iznosi 14,50 m. Maksimalna denivelacija uzvodnog i nizvodnog nivoa vode pri zatvorenim ustavama iznosi 5,05 m, za radni nivo u akumulaciji. Prilaz brani i svim njenim objektima predviđen je preko pristupnog puta u nizvodnom delu na levoj obali. Vododrživost akumulacije se ostvaruje preko injekcione zavese dubine 7,00 m.

Prema geološko-geomehaničkom elaboratu, konstatovano je da podlogu mesta brane MHE Čiflik čine karbonatne stenske mase, strukturno i teksturno. Ove tvorevine čine podlogu levoj i desnoj dolinskoj strani kao i podlogu u samom koritu reke. U prostoru obe dolinske strane i dna doline, gde su u podlozi karbonatne stene, geofizičkim terenskim istraživanjima i ispitivanjima, konstatovane su dve zone sa neznatno različitim geotehničkim svojstvima.

Prva zona nešto ispucanije karbonatne serije, neposredno ispod aluvijalnog, aluvijalno-terasnog, deluvijalnog glinovitog i antropogenog materijala, je promenljive debljine oko 0,5-0,7 m, čija prirodno vlažna zapreminska težina iznosi 25 kN/m^3 . Vrednosti ugla unutrašnjeg trenja iznose od $\varphi=35^\circ$ za alterisano stanje do maksimalno $\varphi=53^\circ$, uz koheziju sa vrednosti $c=134 \text{ kN/m}^2$. Vrednost modula elastičnosti iznosi $E_s=1,7 \times 10^6 \text{ kN/m}^2$. Dozvoljeno opterećenje u ovoj zoni iznosi 5,50 Mpa.

Druga zona karbonatne serije, neposredno ispod prve, je kontinualna do dubine od 30 m. Ona je značajnije kompaktnija od prethodne. Prirodno vlažna zapreminska težina iznosi 26 kN/m^3 . Vrednosti parametra smicanja su relativno visoke i iznose $\varphi=59^\circ$, uz koheziju sa vrednosti $c=755 \text{ kN/m}^2$. Vrednosti modula elastičnosti za mesto brane iznosi $E_s=9,1 \times 10^6 \text{ kN/m}^2$. Dozvoljeno opterećenje u ovoj zoni iznosi 18,60 Mpa.

Fundiranje betonskih objekata će se vršiti u krečnjačkom materijalu. Tako, da su za proračun gravitacionih struktura na pregradnom mestu usvojeni sledeći parametri fundamenta: modul elastičnosti $E_c=2.000.000 \text{ kN/m}^2$, osnovni ugao unutrašnjeg trenja $\varphi=35^\circ$ i rezidualni ugao unutrašnjeg trenja $\varphi_R=35^\circ$.

Prelivni blok se generalno sastoji od ploče i podužnih vertikalnih zidova između kojih je prostor premošćen sa uzvodnim i nizvodnim mostom. Donja ploča i zidovi su monolitna konstrukcija, dok su mostovi nezavisni. Usvojeni su armirano-betonski mostovi sistema proste grede sa četiri glavna nosača, kolovoznom pločom i pešačkom stazom na uzvodnom mostu.

Mašinska sala je ukorporirana u branu i sa ostalim blokovima formira pregradno mesto. Blok mašinske zgrade se generalno sastoji od masivnog dela, koji je sastavljen od monolitne donje ploče i podužnih i poprečnih vertikalnih zidova. Na najvišoj koti brane, locirani su uzvodni i nizvodni monolitni mostovi. Iznad kote krune brane, a na mestu mašinske sale predviđena je betonska superstruktura za pokrivanje mašinskog prostora.

Opterećenja za proračun, su uzeta prema važećim propisima i prema uobičajenim pravilima za proračun ovakvog tipa konstrukcija, pri čemu su za seizmički proračun kvazistatičkom metodom usvojeni sledeći koeficijenti: $K= 0,0430$ za zemljotres Z1-OBE i $K= 0,0715$ za zemljotres Z2-MDE.

Opšta stabilnost konstrukcije je dokazana, tako što je proverena njena stabilnost na isplivavanje, klizanje i prevrtanje. Istovremeno je proveren i maksimalni pritisak na fundament od uticaja konstrukcija. Proračunom su dobijene sledeće vrednosti koeficijenata stabilnosti za masinsku zgradu, kao merodavnu za stabilnost na klizanje:

KOEFIČIJENTI STABILNOSTI <u>MAŠINSKA</u> <u>ZGRADA</u>	KOMBINACIJE OPTEREĆENJA			
	OSNOVNO	DOPUNSK O	IZUZETNO (Z1 - OBE)	IZUZETNO (Z2- MDE)
Protiv klizanja	3,73	2,77	2,68	2,02
Protiv isplivavanja	2,86	2,15	2,86	2,38/1,60
Protiv preturanja	2,62	1,88	2,45	1,95/1,50

Može se videti da kriterijumi stabilnosti nisu bili ključni pri izboru dimenzija glavnih objekata. U ovom slučaju hidrološko-hidraulički uslovi su diktirali poprečne i podužne dimenzije objekata, a lokalna nosivost dimenzije poprečnih preseka pojedinih elemenata.

Izvršen je proračun za karakteristične delove konstrukcije, gde se očekuju uticaji koji prevazilaze konstruktivnu armaturu. Dimenzionisanje betonskih elemenata je izvršeno prema teoriji graničnih stanja, a u skladu sa pravilnikom BAB 87. Dimenzionisanje čeličnih elemenata je izvršeno u svemu prema Pravilnicima i Standardima za čelične konstrukcije. Kontrola širine otvora prslina je proračunata prema Ruskim normama SNIP 2.03.01-84, pri čemu je za maksimalno dozvoljenu širinu otvora prslina usvojeno $a=0,20$ mm.

Za sve nosive strukture predviđen je hidrotehnički beton marke MB 30, klase vodonepropusnosti V-8 i otpornost na mraz M-100. Za delove izložene struji vode predviđena je i povećana otpornost na abraziju. Za armiranje su predviđeni visoko vredni prirodno tvrdi rebrasti čelik RA 400/500-2 (B500A) i meki betonski glatki čelik GA 240/360. Zaptivanje spojnica će se izvršiti PVC trakama.





Vicko Letica¹

SANACIJE NA ČELIČNIM VRATIMA NA SREDNJOJ GLAVI PREVODNICE ĐERDAP I

Rezime:

S-83

Brodaska prevodnica na čijoj su srednjoj glavi locirana vrata, sastavni je deo Hidroenergetskog sistema "Đerdap I" na Dunavu. Na čeličnim vratima srednje glave, u proteklom periodu, dolazilo je do pojave pukotina iz različitih razloga. U ovom radu su prikazane neke od njih, sa izvršenim analizama uzroka nastanka pukotina. Prikazane su sanacije na mestima pojave pukotina sa kontinualnim digitalnim praćenjem napona za vreme eksploatacije vrata.

Ključne reči: brodska prevodnica, vrata, sanacija

MIDDLE HEAD STEEL GATES OF THE DJERDAP I NAVIGATION LOCK REPARATIONS

Summary:

Navigation lock on whose middle head are located gate is a component of the Hydro-energetic system Djerdap I on the Danube River. On the steel gate of the middle head, in the recent period, cracks have been showing up out of different reasons. In this paper are shown some of them, with analysis on what causes cracks. Improvement and reparations on the places where cracks appear are shown, as well as continual digital monitoring during the exploitation of the gate.

Key words: navigation lock, operating gate, reparation

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1 OPŠTE

U okviru hidroenergetskog i plovidbenog sistema „Đerdap I“ na reci Dunavu predviđene su dve brodske prevodnice, jedna duž srpske, a druga duž rumunske obale kojima je omogućeno nesmetano odvijanje plovidbe u Đerdapskom sektoru. Čelična vrata na srednjoj glavi srpske prevodnice, su jedan od vitalnih objekata koji deli prevodnicu na uzvodnu i nizvodnu komoru. Vrata su ravna, klizna, spuštajuća. Statički sistem vrata je prosta greda kutijastog preseka. Sve sekcije imaju po dva glavna horizontalna nosača i po dva vertikalna lima (uzvodni, koji je ujedno i zaustavni lim, i nizvodni) ukrućenih i povezanih sa poprečnim rebrima. Od oko četvrtine raspona, poprečni presek se linearno smanjuje, tako što uzvodni vertikalni lim ostaje u pravcu, a nizvodni vertikalni lim dobija prelom.

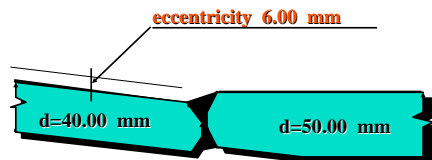


Slika 1 - Izgled vrata

Svetli otvor vrata iznosi po horizontali 34,00 m i po visini 23,10 m. Maksimalni pritisak na vrata iznosi 22,50 mVs. Jasno je da se radi o čeličnoj konstrukciji vrlo velikog raspona koja prima značajno stalno opterećenje od vodenog stuba. Pored toga konstrukcija trpi dinamičko opterećenje u vidu malocikličnog ponavljanja, pri stalnom punjenju i pražnjenju komora, pri prevođenju brodova. Stara vrata su ugrađena i puštena u pogon 1967. godine, a 2003. godine je spuštajući deo vrata zamenjen sa potpuno novom čeličnom konstrukcijom.

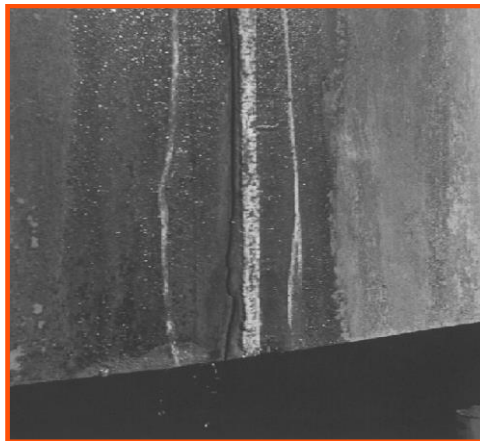
2 PROBLEMI NA STARIM VRATIMA

Nizvodni vertikalni lim je projektovan i izveden sa vertikalnim geometrijskim prelomima i to baš na mestu diskontinuiteta u vertikalnim limovima, tj. prelasku sa lima debljine 50,00 mm na lim debljine 40,00 mm. Na tom mestu je ujedno izveden i montažni nastavak konstrukcije. Međutim, pri izvođenju ove veze, došlo je do pojave znatnijeg ekscentriciteta.



Slika 2 – Ekscentricitet u montažnom nastavku

Prva i najznačajnija pukotina pojavila se 1980. godine. Njena lokacija je bila na donjoj sekciji u zoni pomenutog geometrijskog preloma na nizvodnoj strani prema obali. Sanacija je izvršena premoščavanjem degradiranog mesta sa sistemom horizontalnih i vertikalnih podvezica. Ugrađeno je pet vertikalnih sa spoljne strane i pet horizontalnih podvezica sa unutrašnje strane lima i jedna horizontalna podvezica na dnu nosača. Međutim, nije se dovoljno vodilo računa o lokalitetima u kojima se uvode sile zatezanja iz vertikalnih nosača u ojačane preseke. Tako je 1993. godine došlo do pojave nove vertikalne pukotine u zoni unošenja sile ali sa unutrašnje strane vertikalnog lima. Sanacijom iz 1993. i 1995. godine izvršeno je preoblikovanje horizontalnih podvezica radi izbegavanja dopunskih koncentracija naprezanja. U pojasu geometrijskog preloma, dodat je kompletan set istih podvezica ali u višim zonama radi smanjenja globalne koncentracije krutosti. Istovremeno je izvršeno pokrivanje sa podvezicama rečnog preloma, na isti način kao i za obalni prelom, kod koga je takođe konstatovan ekscentricitet u vezi.



Slika 3 - Glavna pukotina

Numeričkom analizom, utvrđeno je, da je po projektnom rešenju na lokaciji preloma skoro potpuno iskorišćen dozvoljeni napon za osnovno opterećenje. Uzimajući u obzir ekscentričnost u vezi, ukupni naponi dostižu zonu granice tečenja za projektovani materijal. Pošto je konstrukcija ušla u opseg malocikličnog opterećenja, u kombinaciji sa nekim od dopunskih uticaja kao što su: temperaturni, seizmički, manipulativni i sl., potvrđeno je da su naponi prešli granicu nosivosti materijala u zoni geometrijskog preloma. Takođe je potvrđeno da su

nepravilnosti unošenja sile u zonama sanacije, tojest koncentracije napona, izazvale dopunske prskotine.



Slika 4 – Rešenje sanacije

Na osnovu svih tadašnjih analiza izveden je zaključak, imajući u vidu stanje celokupne donje sekcije, da je i pored dopunskih sanacija, eksploatacija vrata pod rizikom. Predložena je izrada nove konstrukcije donje sekcije vrata.

3 PROBLEMI SA NOVIM VRATIMA

Odmah nakon izvođenja i montiranja predmetnih vrata na srednjoj glavi, 2003. godine, izvršeno je prvo kontrolno merenje napona na nizvodnom licu na sredini i u zonama geometrijskog loma. Izmereni naponi su se kretali od 180 Mpa pritiska, na uzvodnom limu do 205 Mpa zatezanja na nizvodnom limu.

Iz analize rezultata merenja konstatovano je, da se na mestu montažnog nastavka na nizvodnom limu, u samoj sredini raspona, javlja napon, koji je za više od 20% veći od najvećeg računskog, ali u granicama dozvoljenog. Interesantno je, da je na mestu neposredno uz ovu lokaciju izmereni napon bio u računskim granicama. Iste vrednosti napona, dobijene su i u sledećim merenjima, a i u zapisima iz kontinualnog merenja napona. Ovakvi rezultati merenja su upućivali na to da postoji neki vid greške koji neočekivano utiče na povećavanje napona.

Tako je u vreme redovnog remonta Brodske prevodnice 2010. godine, primećena pukotina u sredini vrata, na mestima ranije konstatovanih povećanih napona.



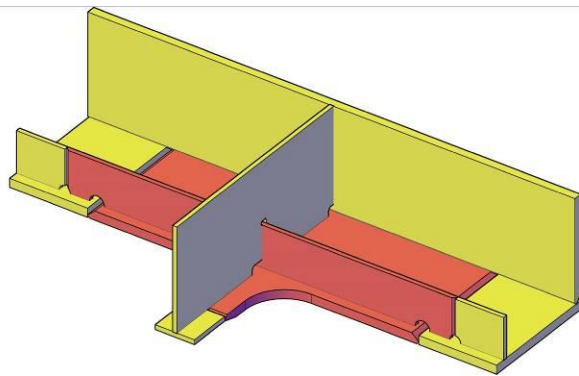
Slika 5 - Mesto pojave pukotine u sredini vrata

Konstatovano je, da je u fazi montaže došlo do značajnih zazora između dva glavna elementa pred spajanje. Tada je odlučeno da se izvrši dosecanje na jednaku meru, kako bi se spoj doveo u relativno jednake distance. Izvršeno je odsecanje oko 5,00 mm, a zatim navlačenje sekcija. Ekscentricitet je bio veći u gornjim zonama donje sekcije. Pored ovih zazora postojao je i ekscentricitet u vezi gornjih i donjih delova pre ukрупnjavanja za konačno spajanje. Kontrolama pre izvođenja sanacije ovog dela konstrukcije donje sekcije, konstatovano je, da je izvršeno naknadno punjenje montažnog otvora sa materijalom za varenje.



Slika 6 - Montažni otvor sa punjenjem

Tada je bilo jasno, da je došlo do pojave pukotine na najlošijem mestu na varu, odakle je krenuo razvoj pukotine. Zbog toga je odlučeno da se oštećeni deo vertikalnog lima zameni sa polukružnim umetkom lima iste debljine i istih karakteristika. Pored sanacije prsline u vertikalnom limu, predviđena je i zamena donje flanše u celosti i to na dužini od 190,00 cm. Kako su se radovi izvodili na najopterećenijem delu vrata, posvećena je specijalna pažnja i sečenju, a naročito varenju novih delova konstrukcije.



Slika 7 - Geometrija dela predviđenog za zamenu u centralnoj zoni



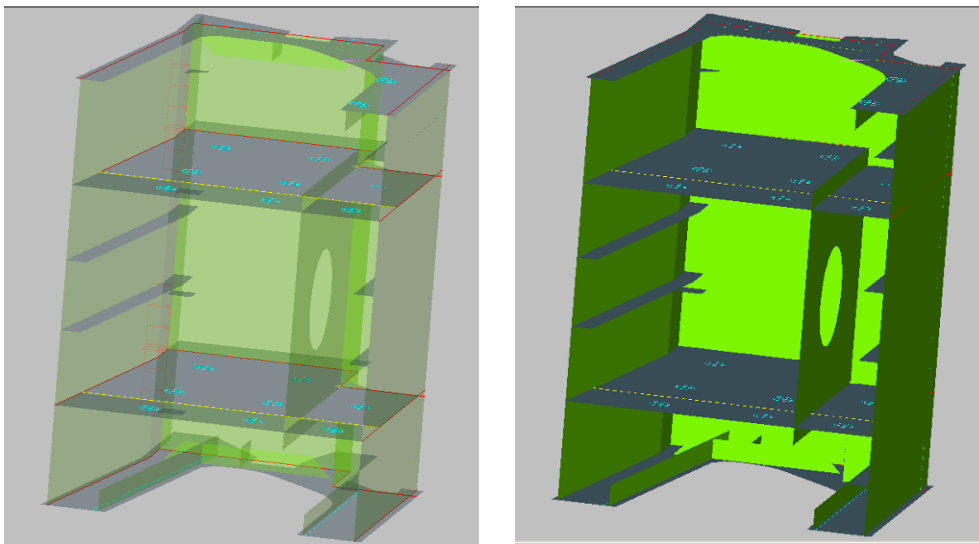
Slika 8 - Radovi na sečenju i uklanjanju dela konstrukcije

U toku redovnog remonta 2007. godine, konstatovane su prsline u zoni oslonaca na uzvodnom geometrijskom prelomu, na mestu gde se sučeljavaju tri vertikalna lima i jedan horizontalni.



Slika 9 - Prsline na geometrijskom prelomu

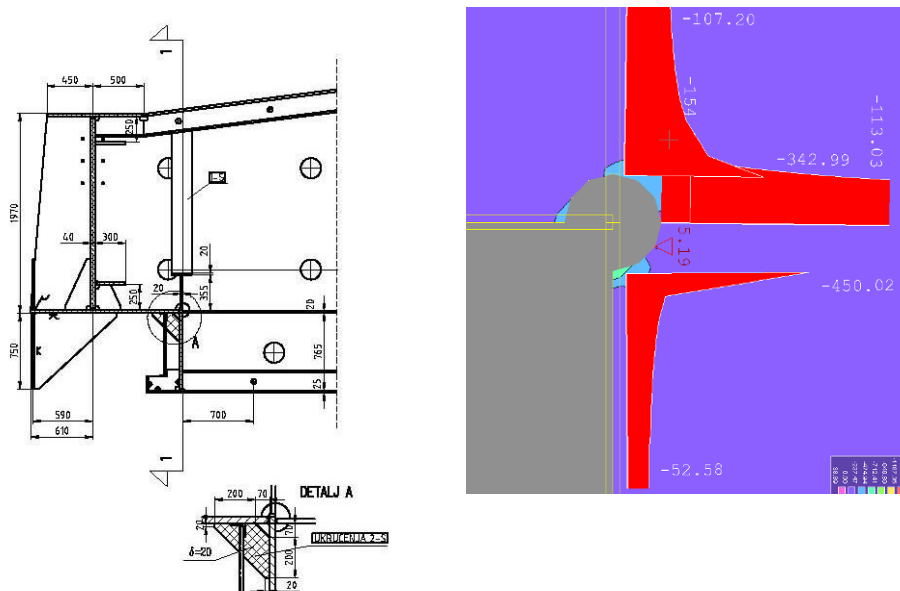
Izvršen je proračun srednje sekcije. Za proračun je izabrana zona vrata od oslonca do prvog glavnog poprečnog ukrućenja u ukupnoj dužini od 393 cm. Konstrukcija je čelična, sandučasta, a debljine vertikalnih i horizontalnih limova su 16 do 40 mm. U statičkom smislu ovaj segment je konzolni element.



Slika 10 – Model za proračun

Numerički proračun napona i deformacija je sproveden po metodi konačnih elemenata (FEM). Geometrija vrata je simulirana pločastim elementima sa preko 31000 čvorova. Stabilnost modela je obezbeđena sa sprečenim pomeranjima u čvorovima na mestima poprečnog ukrućenja. Proračun je izvršen po teoriji elastičnosti. Uticaj težine konstrukcije je sračunat preko sopstvene težine konstrukcije. Hidrostatički uticaji su računati sa podeljenim opterećenjem na uzvodnom limu i sa koncentrisanim horizontalnim silama na mestima glavnih oslonaca.

Proračunom je pokazano da u zoni pojave prslina u glavnim horizontalnim limovima i na spoljašnjim licima flanši dolazi do značajne koncentracije napona, što sa uticajem od zaostalih napona nastalih pri izvođenju konstrukcije, jeste najverovatniji uzrok pojave prslina na ovim mestima.



Slika 11 – Rezultati analize sa sanacijom

Sanacija je izvršena tako što je sa spoljašnje strane u nivou glavnih horizontalnih nosača dodat trougaoni lim. Na ovaj način je u najopterećenijem preseku smanjen generalni napon i izvršena preraspodela koncentracije napona.

Istovremeno sa prethodnim, izvršena je sanacija na sekundarnim nosačima u zoni geometrijskog preloma na nizvodnom licu. Predviđeno je, da se doda dopunsko rebro na sekundarne nosače kako bi se premostio montažni nastavak na flanši, gde su konstatovane prsline.



Slika 12 - Radovi na postavljanju i varenju novih limova na mestu sekundarnih nosača

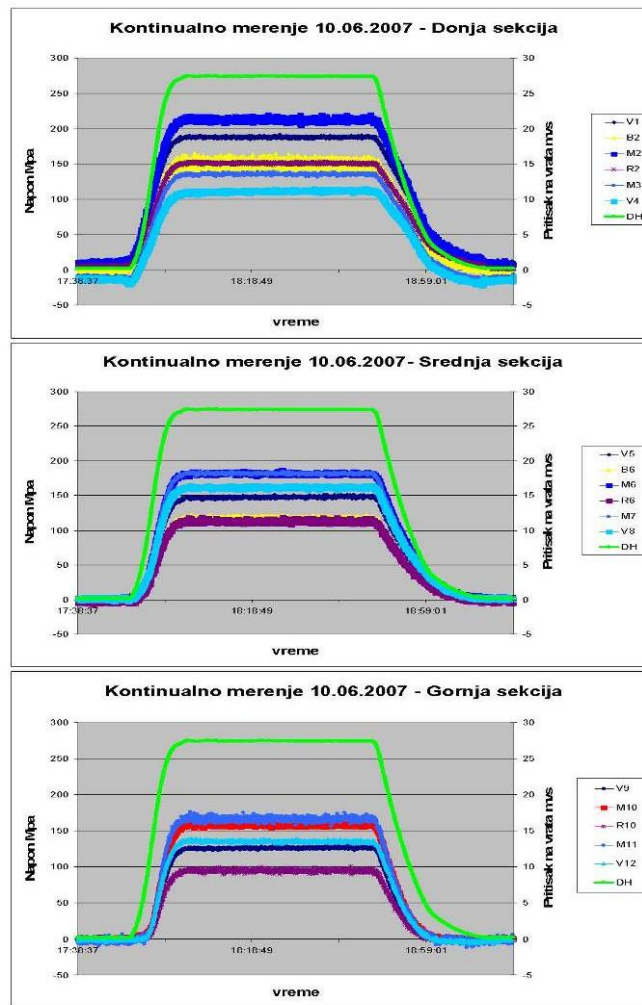
4 KONTINUALNO MERENJE NAPONA

Još na starim vratima, zbog postojanja sanacije na duži period, uvedeno je kontinualno osmatranje naponske slike na konstrukciji vrata. Projektom je bilo predviđeno postavljanje stalnih mernih traka digitalno povezanih u centralnu jedinicu gde se prati ponašanje konstrukcije pri svakoj manipulaciji. Predviđeno je upoređenje merenih vrednosti sa preciziranim apsolutnim maksimalnim vrednostima, sa relativnim maksimumima za svaki period i sa maksimalnim priraštajima za svaku fazu prevođenja. U slučaju da bilo koja od tih vrednosti pređe dozvoljene granice, signalizirale bi se odgovarajuće mere u zavisnosti od inteziteta uticaja. Predviđene su dve granične vrednosti: upozoravajuća i kritična.



Slika 13 - Izgled jedne od mernih traka

Prema istom principu i na sličnim mestima i na novim vratima je obezbeđeno kontinualno merenje napona.



Slika 14 - Mereni naponi juna 2007. godine

Koliko je poznato, ovakav sistem za kontinualno praćenje naponske slike za ovakav objekat u eksploataciji, a da nije ekperimentalna ili naučna svrha, prvi put se primenio na objektu „Djerdap I“.

Konačno, zadnjih šest godina na vratima srednje glave nema značajnih problema.

Slobodan Radovanović¹, Dejan Divac², Dragan Danilović³, Marko Radnić⁴

NOSIVOST PRSKANOG BETONA SA ČELIČNIM FIBER VALKNIMA U PODGRADI TUNELA

Rezime:

S-84

U okviru ovog rada prikazan je primer određivanja momenta nosivosti preseka od prskanog betona sa čeličnim fiber vlaknima. Pored toga prikazana je metodologija naponsko-deformacione analize iskopa tunela i određivanja uticaja u primarnoj podgradi tunela. Analizirana je primarna podgrada koja se sastoji od prskanog betona i čeličnih fiber vlakana. Ovakav beton pri dostizanju čvrstoće na zatezanje prelazi u duktilan lom sa odgovarajućom rezidualnom čvrstoćom na zatezanje. U okviru rada prikazan je postupak za određivanje momenta nosivosti na proračunskom profilu na tunelu Preseka u Makedoniji na osnovu rezultata testa savijanja prskanog betona sa čeličnim fiber vlaknima.

Ključne reči: prskani beton, fiber vlakna, rezidualna čvrstoća, moment nosivosti

LOAD-BEARING CAPACITY OF SPRAYED CONCRETE REINFORCED WITH STEEL FIBRES IN TUNNEL SUPPORT

Summary:

This paper contains an example of how to determine load-bearing capacity of a cross-section made of sprayed concrete reinforced with steel fibres. Apart from this, it shows the methodology of stress-strain analysis of the tunnel excavation and determining the influence in the primary tunnel support. The primary support containing sprayed concrete reinforced with steel fibres was analyzed. When such a concrete reaches the tensile strength, there appears ductile rupture with relevant residual tensile strength. The paper contains a diagram for determining load-bearing moment of the calculated section at the Preseka Tunnel, FYR Macedonia, on the basis of the results of bending test of sprayed concrete reinforced with steel fibres.

Key words: sprayed concrete, fibres, residual strength, load-bearing moment

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1 UVOD

U okviru ovog rada prikazana je metodologija sa primerom sprovođenja naponsko-deformacione analize iskopa tunela i proračuna momenta nosivosti primarne podgrade. Primarna podgrada se sastoji od prskanog betona i čeličnih fiber vlakana.

U okviru poglavlja 2 prikazana je metodologija sprovođenja naponsko-deformacione analize pri analizi iskopa tunela. Zatim prikazana je i analiza ugradnje primarne podgrade kao određivanje momenta nosivosti primarne podgrade od prskanog betona i fiber vlakana. Fiber vlakna se sve više koriste u izgradnji tunela kao sastavni deo primarne podgrade. Fiber vlakna mogu da budu plastična i čelična. Osnovna prednost betona sa fiber vlaknima u odnosu na nearmiran beton jeste pojava duktilnog loma, koji omogućava da se i nakon dostizanja zatezne čvrstoće u betonu javljaju rezidualni naponi, te beton ima odgovarajući stepen nosivosti i nakon pojave prslina (1,2).

Razni strani propisi poznaju problematiku korišćenja fiber vlakana u betonu opšte, a u evrokodovima (3,4) dat je poseban osvrt na prskani beton sa fiber vlaknima koji se između ostalog najčešće upotrebljava u tunelogradnji pri izradi podgradne konstrukcije. U (3,4) dat je osvrt na metode ispitivanja prskanog betona sa fiber vlaknima. U radu (5) dat je osvrt na proračun graničnog stanja upotrebljivosti za prefabrikovanu segmentnu oblogu u hidrotehničkom tunelu.

Cilj ovog rada je da se prikaže kako se može odrediti moment nosivosti preseka prskanog betona sa fiber vlaknima koji je opterećen na složeno savijanje. Ovo je osobitovažno jer za projektantske aktivnosti u okviru dimenzionisanja betona sa fiber vlaknima u okviru primarne konstrukcije u tunelima je važno odrediti nosivost takvog betona. Proračun je prikazan za jedan proračunski profil na tunelu Preseka u Republici Makedoniji. Ovaj tunel se nalazi na auto-putu Kičevo-Ohrid.

U okviru ovog rada prikazani su i rezultati ispitivanja testom savijanja uzoraka betona sa čeličnim fiber vlaknima.

2 METODOLOGIJA PRORAČUNA

2.1 OPŠTE POSTAVKE O STANJU NAPONA U TUNELSKIM ISKOPIMA

Stenske mase su prirodno napregnute i ta primarna naponska stanja su jedna od bitnih karakteristika posmatranog terena. Primarno naponsko stanje je dominantno prouzrokovano gravitacijom odnosno sopstvenom težinom i dejstvom tektonike. Kada se u stenskim masama grade podzemni objekti, stvara se novo naponsko stanje koje nazivamo sekundarnim naponskim stanjem. Preraspodela primarnog naponskog stanja u stenskim masama se ispoljava pri iskopu podzemne prostorije. Posle izvršenog iskopa, u zoni oko podzemne konstrukcije uvek dolazi do novog stanja napona. Naponsko-deformaciona analiza tunela (kao linijskog objekta čija je jedna dimenzija izražena u odnosu na ostale dve) je trodimenzionalni problem samo u jednom kratkom periodu građenja (uticaj čela iskopa), jer se tunel u periodu eksploatacije nalazi u uslovima ravnog stanja deformacije.

Posmatrajmo reprezentativni poprečni presek tunela. Definišemo pojam "stanje podzemne prostorije" kao stanje napona, deformacija i pomeranja u stenskoj masi i konstrukciji, u razmatranom poprečnom preseku. Stanje podzemne prostorije je funkcija vremena i udaljenosti

čela iskopa od posmatranog poprečnog preseka. Kako se čelo iskopa približava posmatranom poprečnom preseku deformacije se u istom preseku povećavaju. Postoje dve situacije: prva je da se pomeranja stabilizuju i da polako počnu da teže nekoj konačnoj vrednosti. U tom slučaju postoji mogućnost da se ne vrši podgrađivanje ako su pomeranja mala, a ako imaju veće vrednosti onda se vrši podgrađivanje i pomeranja onda dostignu neku konačnu vrednost. Druga situacija je, kada su stenske mase slabijih mehaničkih karakteristika, onda pomeranje poprečnog preseka počinje naglo da se povećava kada čelo iskopa prođe posmatrani profil i u tom slučaju je neophodno podgrađivanje kako bi se sprečilo rušenje tunelskog otvora (6).

2.2 PRIMENA MKE ZA MODELIRANJE ISKOPA TUNELA U 2D NUMERIČKIM ANALIZAMA

Najčešći pristup numeričkom rešavanju problema naponsko-deformacione analize nelinearnog kontinuuma je metod konačnih elemenata (MKE). Za potrebe numeričke analize korišćen je programski paket *Phase 2*.

Za definisanje mehaničkog ponašanja stenskih masa u okviru ovog rada korišćen je *Hoek-Brown*-ov materijalni model (7).

Prethodno opisan problem uvođenja trodimenzionalnih efekata pri dvodimenzionalnoj numeričkoj analizi vršen je metodom redukcije modula deformacije unutar tunelskog iskopa. Proračunska analiza se zasniva na sledećem: tunelski iskop se nalazi u primarnom naponskom stanju, zatim se u inkrementima vrši redukcija modula deformacije unutar tunelskog iskopa. Modulo deformacije se redukuje u nekoliko koraka od početne vrednosti pa do vrednosti 0 što predstavlja situaciju kada je čelo iskopa daleko od posmatranog profila. Za svaku vrednost modula deformacije unutar tunelskog iskopa vrši se rešavanje naponsko-deformacionog problema, na osnovu koga se dobija karakteristična linija stene koja reprezentuje promenu pomeranja na tunelskom otvoru usled udaljenja čela iskopa u posmatranom profilu.

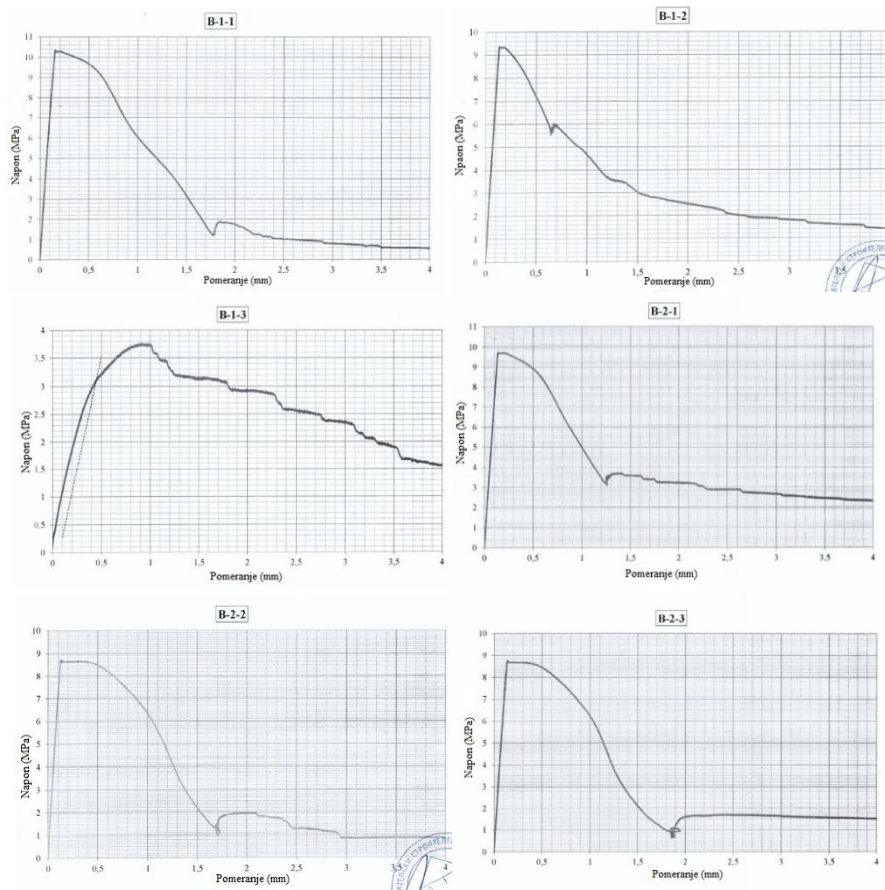
Na osnovu karakteristične linije stene, treba odrediti odgovarajući tip primarne podgrade kao i vreme kada se podgrada postavlja. Primarna podgrada treba da se postavi kada su se već desila odgovarajuća pomeranja, odnosno kada je došlo do odgovarajuće relaksacije napona oko tunelskog otvora. Određivanje trenutka postavljanja podgrade vrši se koristeći empirijske veze date od strane Vlachopoulos i Diederchs (8). Podgrada može da se sastoji od prskanog betona, prskanog betona sa ankerima i prskanog betona sa ankerima i/ili remenatama. U zavisnosti od veličine pomeranja, karakteristične linije, uticaja podzemnih voda pretpostavlja se odgovarajući podgradni sistem za koji se vrši proračun i dimenzionisanje (9).

Kada se postavi podgrada onda se vrši proračun u istoj mreži konačnih elemenata. Na taj način se dobijaju uticaji u podgradnoj konstrukciji. Nakon toga, na osnovu dobijenih uticaja u primarnoj podgradi vrši se odgovarajuća kontrola napona i dimenzionisanje podgrade.

2.3 PRORAČUN PRIMARNE PODGRADE SA FIBER VLAKNIMA

Za ispitivanje mehaničkih karakteristika prskanog betona sa fiber vlaknima koristi se test savijanja i test određivanja energije loma (3). Za analizu u okviru ovog rada korišćeni su rezultati ispitivanja testom savijanja betona sa čeličnim fiber vlaknima koji su ispitani za potrebe građenja tunela Preseka u Makedoniji.

Ispitivane su prizme 125x75x500 sa dve koncentrisane sile testom savijanja. Na sledećim slikama prikazani su dijagrami napon-deformacija dobijeni testom savijanja.

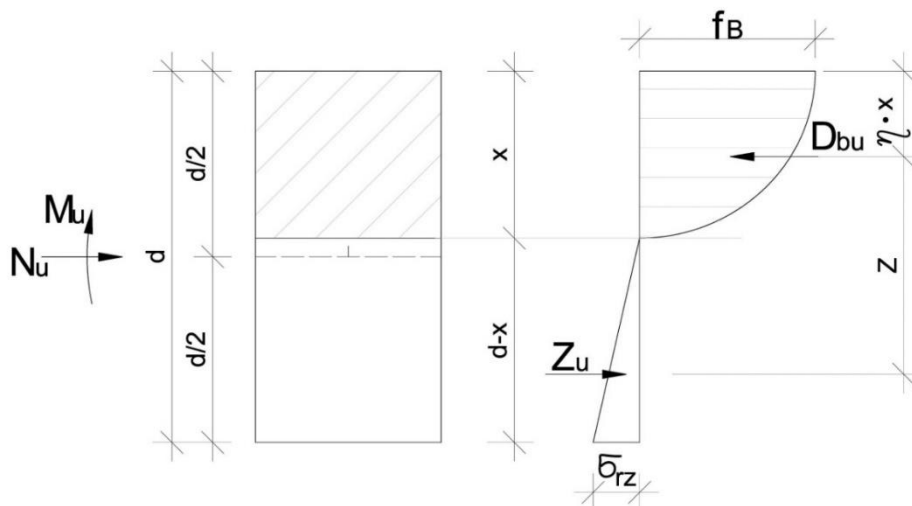


Slika 1 – Dijagrami napon-pomeranje dobijeni testom savijanja na gredicama od prskanog betona sa čeličnim fiber vlaknima

Na osnovu testa savijanja posmatraju se 3 karakteristične tačke, i to: tačka sa maksimalnom vrednošću napona zatezanja, zatim tačke sa vrednostima rezidualnih napona zatezanja za pomeranje $L/600$ i $L/150$, gde je L razmak oslonaca prizme. Za određivanje momenta nosivosti preseka od prskanog betona i fiber vlakana koristiće se vrednost rezidualnog napona zatezanja pri deformaciji $L/150$. Na osnovu rezultata ispitivanja na 6 ispitnih prizmi, srednja vrednost rezidualnog napona zatezanja pri deformaciji $L/150$ iznosi 1,87 MPa.

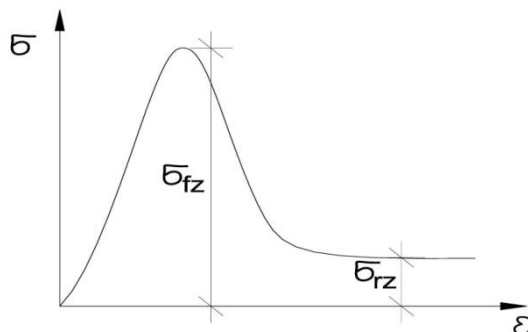
Nosivost betona na pritisak nije analizirana jer se dobija da su naponi pritiska ispod dozvoljenih napona na pritisak u betonu. U slučaju pojave zatezanja, određuje se nosivost preseka na savijanje u slučaju pojave prslina. U skladu sa izvršenim ispitivanjima i mehaničkim ponašanjem betona sa fiber vlaknima, prikazan je algoritam za utvrđivanje nosivosti na savijanje preseka od fiber vlakana:

Na slici 2 je prikazan presek betona u kome je prekoračena čvrstoća na zatezanje u betonu i došlo do otvaranja prsline.



Slika 2 – Raspodela napona u preseku pri pojavi prsline

Kada dođe do pojave prsline u betonu, preostalu silu zatezanja prihvataju fiber vlakna. Naponi koji se tada ostvaruju u zategnutom delu preseka odgovaraju rezidualnoj čvrstoći na zatezanje koja se propagira po celom zategnutom delu preseka. Karakterističan dijagram napon-dilatacija betona sa fiber vlaknima šematski je prikazan na sledećoj slici.



Slika 3 – Karakterističan dijagram napon deformacija betona sa fiber vlaknima

Sa slike 3 se vidi da prilikom dostizanja vršne čvrstoće, napon u betonu pada na rezidualnu vrednost i javlja se duktilno ponašanje betona. Da bi se odredilo koliki moment nosivosti može da prihvati presek sa prsline ojačan fiber vlaknima, potrebno je odrediti položaj neutralne linije u trenutku formiranja prsline. Položaj neutralne linije se određuje iz uslova ravnoteže unutrašnjih sila koje se javljaju u preseku u trenutku pojave prsline

$$\sum N = 0 \Rightarrow D_{bu} - Z_u = N_u$$

$$\sum N = 0 \Rightarrow D_{bu} - Z_u = N_u$$

$$D_{bu} = \alpha_b x b f_B$$

$$\alpha_b = \frac{3\varepsilon_b - 2}{3\varepsilon_b}$$

gde je:

α_b koeficijent punoće napona pritiska u betonu

ε_b dilatacija u betonu

x visina pritisnute zone betona

ε_b dilatacija u betonu

f_b računski čvrstoća betona pri pritisku

b širina preseka

D_{bu} sila pritiska u betonu

N_u granična normalna sila

$$Z_u = \sigma_{rz} b(d - x) / 2$$

gde je:

σ_{rz} rezidualna čvrstoća betona na zatezanje

d visina preseka

Z_u sila zatezanja u fiber vlaknima nakon pojave prsline

Kada se iz uslova ravnoteže normalnih sila odredi visina pritisnutog betona u preseku, onda se može odrediti i vrednost momenta nosivosti datog preseka. Moment nosivosti se određuje iz sume momenata oko težišta zategnutog dela preseka:

$$\sum M = 0 \Rightarrow M_u = D_{bu} z - N_u \left(\frac{d}{2} - \frac{d - x}{3} \right)$$

M_u granični moment nosivosti

Za analize u okviru ovog rada korišćena je prosečna vrednost rezidualnog napona zatezanja od svih 6 prizmi koja iznosi 1,87 MPa.

S obzirom da se radi o malom broju uzoraka, teško je reći da su ove vrednosti čvrstoća u potpunosti reprezentativne, tako da će se radne verzije ovih napona podeliti sa faktorom sigurnosti 1,5 što je i u skladu sa evropskim propisima, kako bi se dobile računski vrednosti ovih rezidualnih napona. Računski vrednost rezidualnog napona bi iznosila 1,25 MPa.

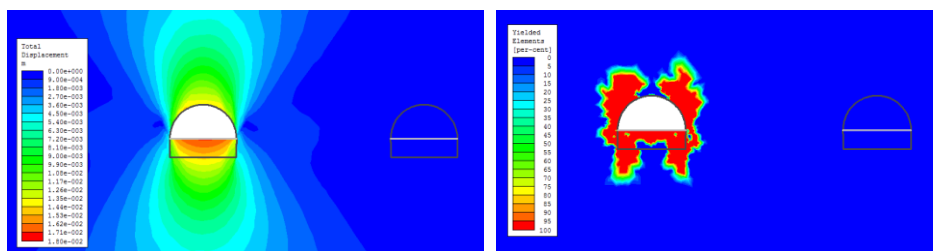
3 REZULTATI PRORAČUNA

U okviru ovog poglavlja dat je jedan primer sprovođenja naponsko-deformacione analize iskopa tunela i proračuna primarne obloge koja se sastoji od prskanog betona i čeličnih fiber vlakana. Proračunski profil se nalazi na tunelu Preseka u Makedoniji u kvarc-sericitskim škriljcima sa visinom nadsloja od 120 m. U tabeli 1 dati su parametri stenske mase koji su neophodni pri naponsko-deformacionoj analizi.

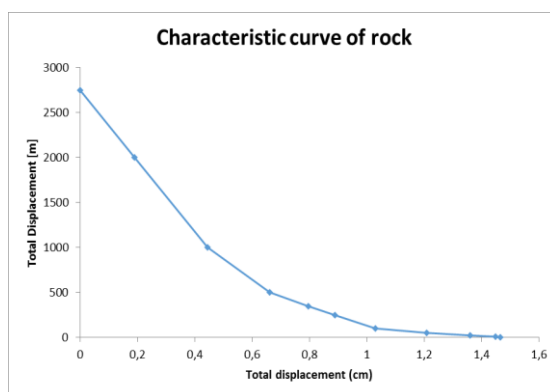
Tabela 1 – Fizičko-mehaničke karakteristike stenske mase

Parametar stenske mase	Vrednost
Zapreminska težina (kN/m ³)	26,8
Jednoaksijalna čvrstoća monolita (MPa)	33
Modul deformacije (MPa)	2750
Vrednost RMR poena	42
Hoek-Brown-ova konstanta mi	10
Parametar mb	1,054
Parametar s	0,0009119
Parametar a	0,5139

Iskop tunela se vrši u dve faze: prva faza podrazumeva iskop kalotnog dela visine 5,7 m, druga faza predstavlja iskop donjeg dela tunelskog profila. Polje ukupnih pomeranja kada je čelo iskopa daleko od posmatranog profila, prikazano je na sledećoj slici:



Slika 4 – Polje ukupnih pomeranja i polje plastičnih deformacija za nepodgrađen iskop



Slika 5 – Karakteristična linija stene

Na slici 5 je prikazana karakteristična linija stene za posmatrani profil: Sa slike 5 se vidi da pomeranje konture iskopa ima progresivan rast, numerička analiza ima konvergenciju na 1,47 cm, međutim ta vrednost može da bude veća u slučaju da se nastavi trend rasta pomeranja, te se stoga predviđa ugradnja podgrade. Pretpostavlja se da se podgrada ugradi 2 m nakon što čelo iskopa prođe posmatrani profil. U skladu sa (5,4) određuje se vreme ugradnje primarne podgrade.

$$u_{\max} = 1,47 \text{ cm}$$

$$Rt = 5,5 \text{ m}$$

$$Rpl = 11 \text{ m}$$

$$Rpl / Rt = 1,93$$

$$X = 4,0 \text{ m}$$

$$X / Rt = 2 / 5,5 = 0,73$$

Sa dijagrama iz (5) dobija se da je vrednost pomeranja konture iskopa u trenutku ugradnje podgrade jednaka:

$$u = 0,57 * u_{\max} = 0,83 \text{ cm}$$

Sa karakteristične linije stene se vidi da se u koraku 5 dogodilo pomeranje 0,83 cm. Vrednost modula deformacije u fazi relaksacije u ovom koraku iznosi 350 MPa. Proračun je sproveden na prethodno prikazanom modelu konačnih elemenata u sledećim fazama:

Faza 1 - primarno naponsko stanje

Faza 2 - relaksacija materijala u zoni iskopa sa modulom deformacije do vrednosti 350 MPa.

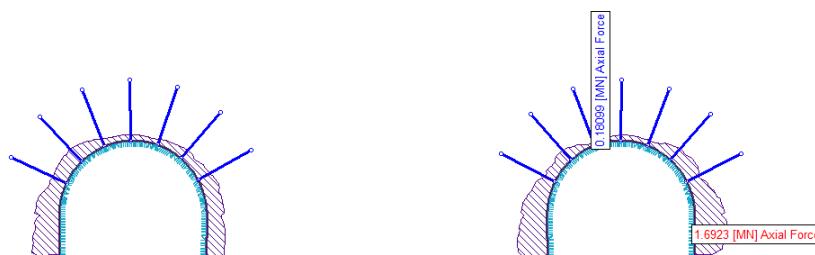
Faza 3 - u koraku pri relaksaciji modula deformacije 250 MPa kalotnog dela ugrađuje se primarna podgrada. U istom koraku vrši se iskop materijala. To znači da je čelo iskopa daleko od posmatranog profila.

Faza 4 - relaksacija materijala u zoni donjeg dela tunelskog iskopa do vrednosti modula deformacije od 350 MPa.

Faza 5 - U koraku pri relaksaciji stenske mase od 250 MPa u donjem delu ugrađuje se primarna podgrada. U istom koraku vrši se iskop materijala iz donjeg dela tunelskog otvora. To znači da je čelo iskopa daleko od posmatranog profila.

Faza 6 - U ovom koraku je ugrađena obloga i u drugoj tunelskoj cevi po istom principu kao i za prvu tunelsku cev.

Na slici 6 i slici 7 prikazani su dijagrami normalnih sila i momenata savijanja za obe tunelske cevi kada je potpuno ugrađena obloga i u jednoj i u drugoj cevi (faza 6).



Slika 6 – Dijagram normalnih sila



Slika 7 – Dijagram momenata savijanja

Za modeliranje je korišćen beton koji ima visinu preseka 15 cm, MB 30. Prilikom analize nosivosti posmatra se presek širine 1 metar.

Za određivanje momenta nosivosti korišćen je presek na primarnoj podgradi u kome se javlja napon zatezanja, a koji ima sledeće uticaje:

Vrednost normalne sile je: $N = 926kN$

Vrednost momenta savijanja je: $M = 27,94kNm$

U skladu sa metodologijom opisanom u poglavlju 2.3 dobija se vrednost momenta nosivosti datog preseka:

$M_u = 60,79kNm$, vrednost momenta savijanja koji se javlja u preseku pomnožen koeficijentom sigurnosti iznosi $M = 1,6 * 27,94 = 44,70kNm$.

Faktor sigurnosti ovakvog preseka za slučaj pojave prsline bi iznosio:

$F_s = 60,79 / 44,70 = 1,36$

4 ZAKLJUČAK

U okviru ovog rada prikazana je metodologija proračuna momenta nosivosti prskanog betona sa fiber vlaknima pri analizi primarne podgrade u tunelu. Pored toga prikazana je i metodologija koja se generalno koristi za proračun iskopa i primarne konstrukcije tunela primenom metode konačnih elemenata. S obzirom da u srpskim propisima nema postupak za proračun preseka sa fiber vlaknima, autori su hteli da kroz ovaj rad pokažu kako se to i na koji način može izvršiti. Treba naglasiti da se navedeni primer proračuna odnosi na proračunski profil na tunelu Preseka, ali se prikazani postupak može primeniti pri bilo kojoj analizi prskanog betona sa fiber vlaknima. Drugi način za proračun prskanog betona ojačanog fiber vlaknima jeste korišćenje metode konačnih elemenata. Na taj način se beton sa fiber vlaknima može modelirati preko modela sa plastičnim oštećenjem ili drugih materijalnih modela koji imaju plastično oštećenje. Ovaj tip proračuna nije analiziran u okviru ovog rada.

ZAHVALNICA

Autori se zahvaljuju projektu tehnološkog razvoja TR37013.

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Dunja Vla¹, Predrag Popovic,² Tracy R. Naso³

SANACIJA DVANAEST TUNELA NA NORTHWESTERN UNIVERZITETU

Rezime:

S-85

Mreža tunela na univerzitetskom kompleksu je dugačka oko dva kilometara. Tuneli su građeni u 1914-oj i u ranim 1970-im godinama. U tunelima su smeštene cevi za toplu vodu i parno grejanje, i elektro i komunikacione kablove. Konstrukcija tunela se sastoji od betonskih zidova, krovne i podne ploče i temelja. Čelični ramovi u tunelima služe kao noséca konstrukcija za infrastrukturu mrežu. Uzroci oštećenja u tunelima su bili stalna izloženost podzemnim vodama, i visoka temperatura u tunelima. Opisane su metode korišćene za sanacije tunela, pri punoj funkciji tunela tokom popravki.

Ključne reči: tunel, podzemne vode, oštećenja, sanacija

RESTORATION OF TWELVE UTILITY TUNNELS AT NORTHWESTERN UNIVERSITY

Summary:

The utility tunnel system at the university campus is about two kilometers long and was built in 1914 and in the early 1970s. The tunnels house pipes for hot water, steam and fire protection systems, and electric and communication cables. The structure of the tunnels consists of concrete walls, roof and floor slabs and footings. Steel racks in the tunnel are used to support pipe and cable infrastructure. Concrete distress in the tunnels was caused by constant exposure to underground water and high temperatures inside the tunnels. Repair methods used to restore the tunnels while the tunnels remain fully in service are presented.

Key words: tunnel, underground water, distress, repair, rehabilitation

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1 INTRODUCTION

Northwestern University in Evanston, Illinois, USA, is a private research university established in 1851. The university has approximately 21,000 students and is composed of twelve schools and colleges. The main campus has an area of about 100 hectares in Evanston, along the shores of Lake Michigan. There are over one hundred buildings, and twelve tunnels are used to house the steam, hot water, fire safety, and communication lines for a large number of the buildings. The tunnels pass beneath several different surface conditions. Sections of the tunnels were located in the basements of the buildings; beneath the front stairs at the entrance to buildings; beneath asphalt-paved parking lots and roadways, concrete sidewalks, and lawns; and beneath landscaped planting beds. The total length of the tunnels is about 1.9 kilometers, and the length of each individual tunnel varies from approximately 60 meters to 200 meters.

A condition evaluation determined that the tunnels are serviceable, but require a fair amount of repair to address deterioration of the concrete structure and pipe supports. Water intrusion was the most pressing problem in all of the tunnels, and was the cause of deterioration of both the concrete structure and the metal pipe supports. Many locations were observed where previous concrete repairs had failed due to continued water ingress, particularly in the newer tunnels.

The evaluation concluded that conditions of the utility tunnels ranged from very poor to fair. Extensive corrosion of the embedded reinforcing steel was present at walls and the underside of the roof slab, resulting in delaminations and spalling. The observed depths of the spalls had compromised the load-carrying capacity of the structures at some locations, including some under operating roadways. Extensive repairs of these structures were required to restore their structural integrity.

The challenge of the restoration project was that utility tunnels had to remain operable with all piping systems in place. Steam and fire protection pipes in one of the tunnels were scheduled to be replaced with new in conjunction with implementing the needed structural repairs, and the restoration work had been coordinated with the pipe replacement project.

2 DESCRIPTION OF THE TUNNELS

2.1 TUNNELS BUILT IN 1914

The tunnel structures built in 1914 consist of reinforced concrete walls and roof slab and an unreinforced concrete floor slab, as shown in the cross-sections in Figure 1. The interior height of the tunnels is 6 ft to 6 ft-6 in. and the interior width is 4 ft-6 in. to 5 ft. The walls are 7 in. thick. The roof of the tunnel is either arched or consists of a crowned slab with an average thickness of approximately 7 1/2 in. Steam pipes are supported on steel pipe racks along one wall of the tunnel, and conduit and communications lines run along the opposite wall.

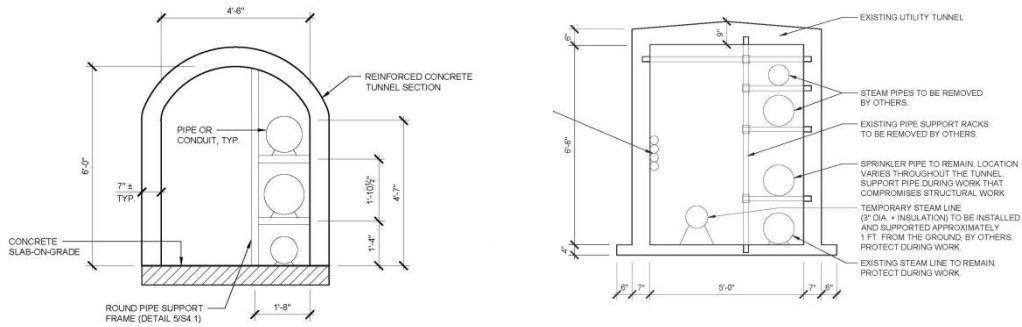


Figure 1 - Cross section of 1914 tunnels

2.2 TUNNELS BUILT IN 1970S

The tunnel structures built in 1970s consist of reinforced concrete walls, roof and floor slabs, as shown in the cross-sections in Figure 2. The interior height of the tunnels varies from 7 ft-2 in. to 8 ft-4 in. and the interior width varies from 6 ft. to 12 ft. Walls are 7 in. to 9 in. thick and the roof slab of the tunnel is flat, with a thickness of 7 in. Steam pipes were supported on unistrut steel racks along the one wall of the tunnels.

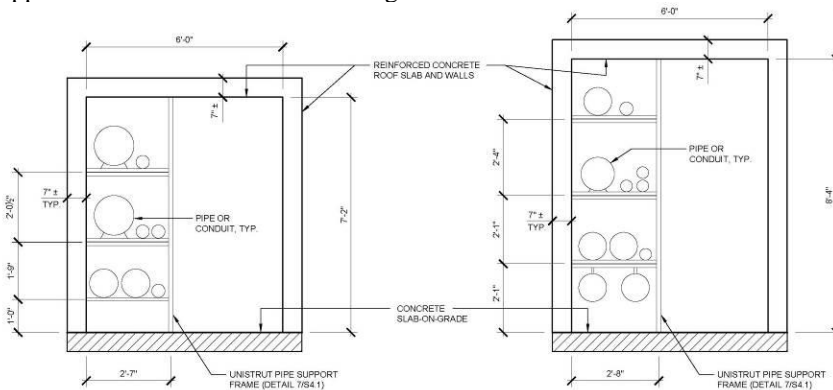


Figure 2 - Cross section of 1970 tunnels

3 TYPE OF DISTRESS

Distress observed in the utility tunnels included the following:

- Deterioration of reinforced concrete walls due to corrosion of embedded reinforcing bars. Significant section loss of reinforcing bars was observed at some locations. Both partial and full depth deterioration of walls was present. Figures 3 and 4 show these conditions.



Figure 3 - Deterioration observed on concrete walls



Figure 4 - Deterioration observed on the underside of the roof slab

- Deterioration of pipe steel racks, most severely at the bases, as shown in Figure 5.



Figure 5 - Deterioration observed at steel pipe rack bases

- Water infiltration into the tunnel through cracks, construction joints, and through pipe penetrations, as shown in Figure 6.



Figure 6 - Actively leaking crack in the roof slab and water ponding on the floor

- Missing or insufficient thermal protection over hot water and steam pipes causing constant exposure of concrete to temperatures over 110°F.

4 REPAIR OPTIONS

Depending on the extent of the concrete deterioration and extent of compromised load-carrying capacity, the following repair options were implemented on these structures to restore their structural integrity:

- Repair Option 1: Perform localized concrete repairs on the inside surfaces of the tunnel at observed areas of deterioration and inject leaking cracks and joints. This option consisted of removing unsound and sound concrete from the interior of the tunnel, replacing reinforcing steel that exhibited significant cross-sectional loss or damage, installing repair concrete, and injecting leaking cracks and joints. An illustration of this repair option and a photograph of the repair work in progress are shown in Figure 7. The longevity of these repairs is expected to be limited, as this option does not address moisture ingress into the tunnel, which is promoting corrosion of the embedded reinforcing steel and deterioration of the concrete. The localized repairs also do not address the general poor condition of the concrete.

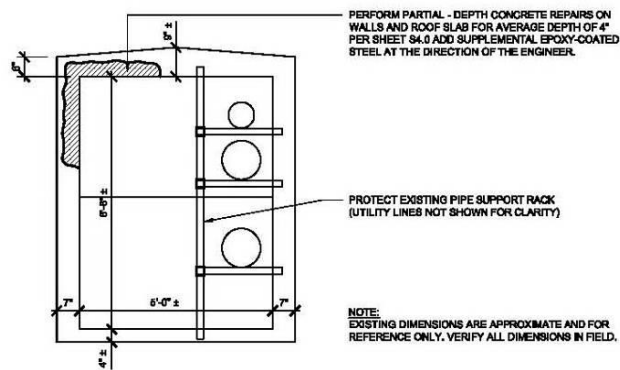


Figure 7 - Repair Option 1, partial depth interior repairs on walls and ceilings

- Repair Option 2: Excavate tunnel, replace roof slab, perform localized concrete repairs, install exterior waterproofing, and inject leaking cracks and joints. This second option for repair requires excavation of the soil and other overburden material to expose the exterior surface of the tunnel. This approach allows for localized full-depth concrete repairs, including replacement of significantly deteriorated reinforcing steel, replacement of the roof slab and full replacement of the exterior waterproofing protection with a new hot-applied fiber-reinforced rubberized asphalt waterproofing membrane system. This method is illustrated and photographs of the work in progress are shown in Figures 8 and 9. This method allows for more thorough replacement of deteriorated and poor-quality concrete and protection of the tunnel from water ingress, therefore greatly increasing the expected service life of the repairs and the tunnel. This option will not, however, increase the load capacity of the existing tunnel, which is not sufficient to carry modern truck loading under roadways.

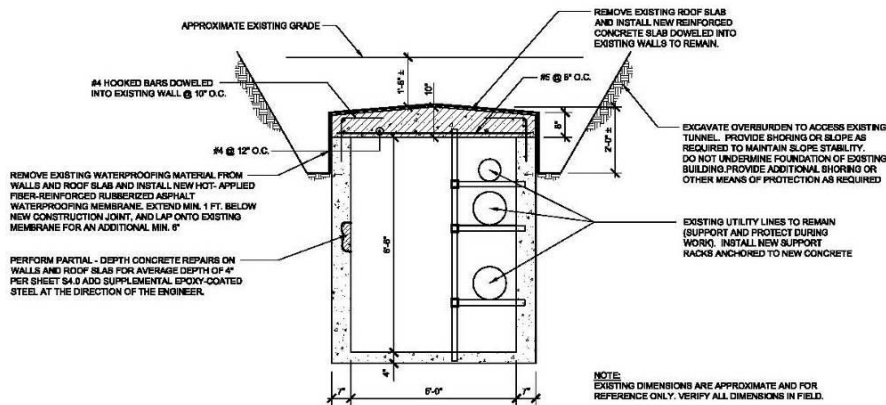


Figure 8 - Repair Option 2, partial replacement and waterproofing of tunnel structure. Replacement of roof slab with full and partial depth concrete repairs on walls.



Figure 9 - Repair Option 2, partial replacement of tunnel structure. Installation of new roof slab

- Repair Option 3: Tunnel replacement, with encasement of existing walls. A third option is full replacement of the tunnel cross-section, as illustrated in Figure 10. Photographs of the repair work in progress are shown in Figure 11. This option involves excavation of the overburden, including soil and pavements, demolition and removal of the existing roof slab, construction of new cast-in-place concrete walls and roof slab spanning over the original tunnel structure, and installation of a new hot-applied fiber-reinforced rubberized asphalt waterproofing membrane system and toe drains at the bases of the new walls. The existing walls are left in place, and any unsound concrete is repaired on accessible portions of the interior surfaces of these walls to remain. This repair option allows the new tunnel structure to be designed to withstand a modern truck loading beneath roadways while keeping the existing pipes in place without any disruption to service. The new waterproofing membrane will cut off future moisture ingress, protecting the concrete structure and the steel pipe racks. In addition to these benefits, the service life of the new tunnel would be expected to be another 50 years with periodic maintenance.

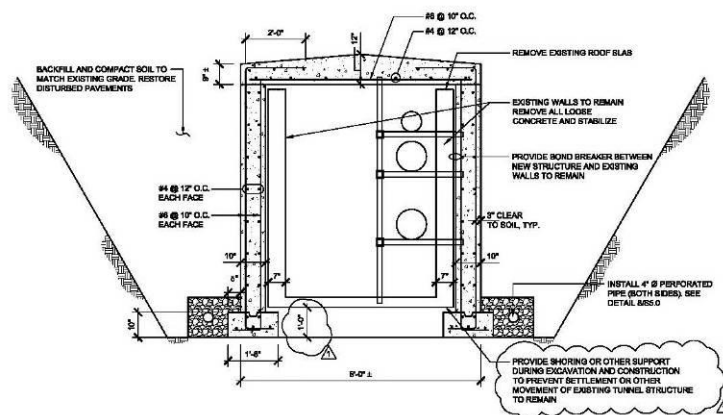


Figure 10 - Repair Option 3, structural replacement with original walls to remain.



Figure 11 - Repair Option 3, structural replacement with original walls to remain. Encasing existing walls with new and pouring the new roof slab.

As part of each of the three repair options, deteriorated pipe support racks were replaced or repaired.

Each section of tunnel was repaired using one of the three options. The appropriate repair options were selected based on the condition of the tunnel and feedback from the owner regarding accessibility for excavation, current and planned loading requirements for the tunnel, and other work restrictions that affected specific sections of the tunnel.

5 SUMMARY

Sections of the utility tunnel system exhibited significant levels of deterioration. In addition, sections of the structures had limited capacity to support the required truck loading for roadways that pass over portions of the tunnels. The tunnels had to be repaired to address these conditions, and certain portions were replaced in order to allow for continued use and serviceability of the tunnels. All repair and replacement work was completed while the infrastructure system inside them remained fully functioning.

Violeta Mircevska¹

UTICAJ OBLIKA REZERVOARA I PRAVCA SEIZMIČKOG DEJSTVA NA FSI EFEKTE

Rezime:

Pravac seizmičkog dejstva i nepravilni oblici terena su važni faktori za dinamičku analizu interakcije brana - fluid. U ovom radu, tretiran je kompleksni model konačnih i granicnih elemenata. Hidrodinamički efekat izračunat je pomoću hidrodinamičke matrice uticaja. Sistem brana-fluid je podvrgnut triju tipičnih slučajeva horizontalne seizmičke akcije : uzvodno, poprecno na tok, i pod uglom od 45 stepeni u odnosu na osu brane , dok je vertikalna komponenta umanjena na 75% od horizontalnog dejstva zemjotresa. Rezultati pokazuju najviši hidrodinamički pritisci za seizmičko dejstvo u pravcu recnog toka, koji se mogu dodatno pojačati od strane nepravilnosti terena.

Ključne reci: lučna brana, brana-fluid interakcija, nepravilnost terena

INFLUENCE OF RESERVOIR SHAPE AND DIRECTION OF EXCITATION OVER FSI EFFECTS

Summary:

During an earthquake, the vibration energy is transferred by dam and the reservoir to the fluid. The direction of the seismic excitation and the terrain irregularities are important factors for dynamic dam-fluid-reservoir interaction analysis. In this paper, a complex finite element – boundary element model of an arch dam-reservoir system is computed using the hydrodynamic matrix of influence. The dam was subjected to three typical cases of the horizontal seismic action: stream-wise, cross-stream and at an angle of 45 degrees with respect to the dam axis, accompanied with a vertical component scaled at 75% of the horizontal ground motion. The results show that highest hydrodynamic pressures occur for seismic excitation in the stream-wise direction, which can be additionally amplified by the terrain irregularities.

Key words: arch dam, fluid-structure interaction, reservoir topology

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1 INTRODUCTION

Various time domain direct-coupling algorithms (Yu et al. [2001], Czygan and von Estroff [2002]) and iterative-coupling algorithms (Mansur [1983], Lin et al. [1996], Kamiya et al. [1996], von Estorff and Hagen [2006], Soares et al. [2005]) integrate the advantages of both BEM and FEM techniques to simulate fluid-structure interaction effects. Both types of algorithms, however, show certain difficulties especially when solving more complex coupled BEM-FEM models. The direct-coupling techniques are affected by instability of the solution, whereas the iterative-coupling methods are based on time consuming iterations and often exhibit instability caused by equilibrium failure at the interface. To rapidly attain a stable solution, especially considering the complex influence of the combined vibration of the dam and the irregular terrain, an original method is developed applying a FEM-BEM fluid-structure interaction solution based on independent analysis of the two physically different domains. It is based on the determination of the matrix of hydrodynamic influence (HDI) that enables definition of the vector of hydrodynamic forces (HDF). (ADAD-IZIIS [2008]). The interaction effect at the dam-fluid interface is enforced by adding the vector of hydrodynamic forces to the vector of inertial forces within the standard equation of dynamic motion of the FEM solid sub-domain.

2 NUMERICAL MODEL FOR ANALYSES

The considered double-curved arch dam is 405 m long and 130 m high, and is composed of 27 monolith blocks. The thickness of the crown cantilever is 26 m at the base and decreases gradually to 7.6 m at the crest. The material properties considered in the analyses were: Young's modulus $E=31.5\text{GPa}$, mass density of concrete $\rho=2450\text{ kg/m}^3$, Poisson's ratio $\nu=0.2$, mass density of water $\rho=1000\text{ kg/m}^3$, and acoustic wave velocity in water $c=1440\text{ m/s}$. The water depth vs. dam height ratio is 0.77. In the simulations, the material behaviour is in the linear domain and the rock foundation was assumed as perfectly rigid. The dam-fluid-reservoir system was subjected to the well-known El Centro record of the 1940 Imperial Valley earthquake, measured at rock outcrop. Observational nodes at the centroids of the boundary elements (C1, D1, E, F and G) provide insight into the hydrodynamic effects exerted over the interface, whereas the dynamic response of the dam is displayed at the finite element nodes (A, B, C, and D), Fig. 1.

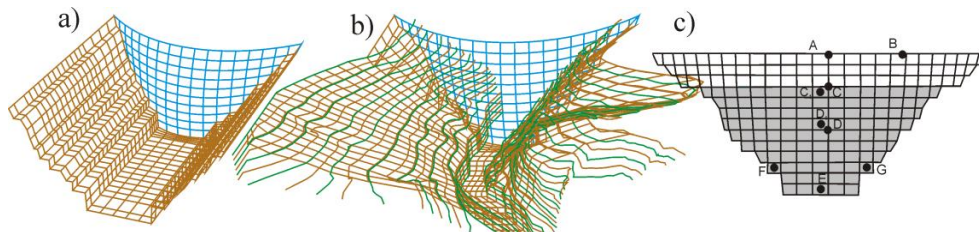


Fig.1 BEM discretization of the fluid-dam-foundation rock system for a) regular terrain, and b) complex terrain c) Location of the observation nodes

3 SEISMIC EXCITATION IN STREAM-WISE DIRECTION

Time histories of relative displacement, velocity and acceleration for node B (dam crest), where the highest dynamic response occurs, are given in Fig. 2. The response acceleration is undoubtedly the most relevant dam response parameter for quantifying the interaction effects. Namely, the dam and the fluid are dynamically interacting during the seismic action in a way that the dam acceleration exerted along the normal at the any point generates the energy of compressive or dilatational waves, which in return modify the dam acceleration response.

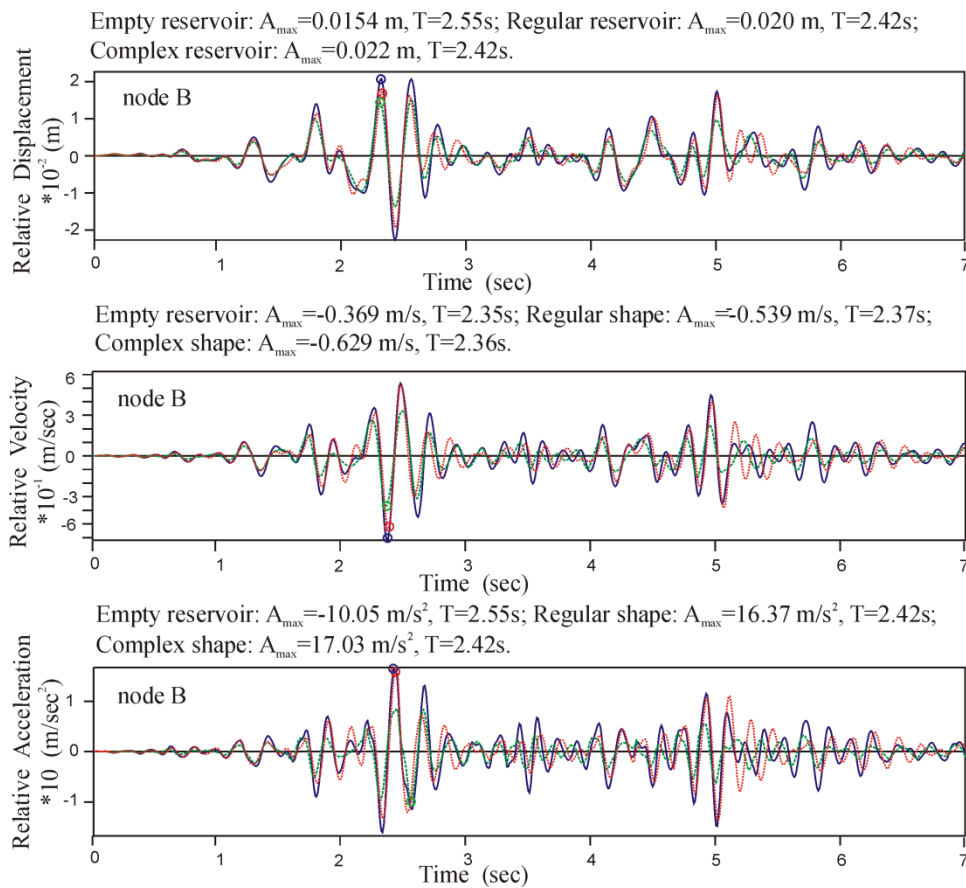


Fig.2 Dynamic response for seismic excitation in the stream-wise direction. Dashed line indicates response of empty reservoir, dotted line indicates response of reservoir with regular geometry, and solid line shows response of reservoir with complex geometry

In order to depict the influence of terrain irregularities on evaluation of the dynamic response along the crown cantilever, time histories of relative accelerations are presented for nodes A, C and D , Fig. 3. As expected, the dam response is strongly affected by the dam-fluid interaction. The ratio between the water depth and dam height of 0.77 is sufficiently high to

contribute to the development of considerable hydrodynamic forces. What is more important, these figures emphasize the influence of the considered direction of excitation and the terrain irregularities.

A simple quantitative comparison made by means of the ratio between the observed maximal responses reveals that due to the effect of reservoir irregularities, the maximal relative displacement, velocity and acceleration responses at the dam crest (node B) increase for 10%, 17% and 4%, respectively. Fig. 2. Likewise, the maximal relative acceleration responses increase for 4%, 4%, 14% and 49% with the increase of the depth of the water for nodes A, B, C and D, respectively. Fig. 3. In Fig. 4 are shown the developed hydrodynamic pressures at the centroids of the selected boundary elements (observational points C1, D1, E, F and G). The corresponding hydrostatic pressure HSP is also given. The influence of the reservoir irregularities is again quantified with the ratio between the maximal responses generated by reservoirs with complex and regular geometry respectively.

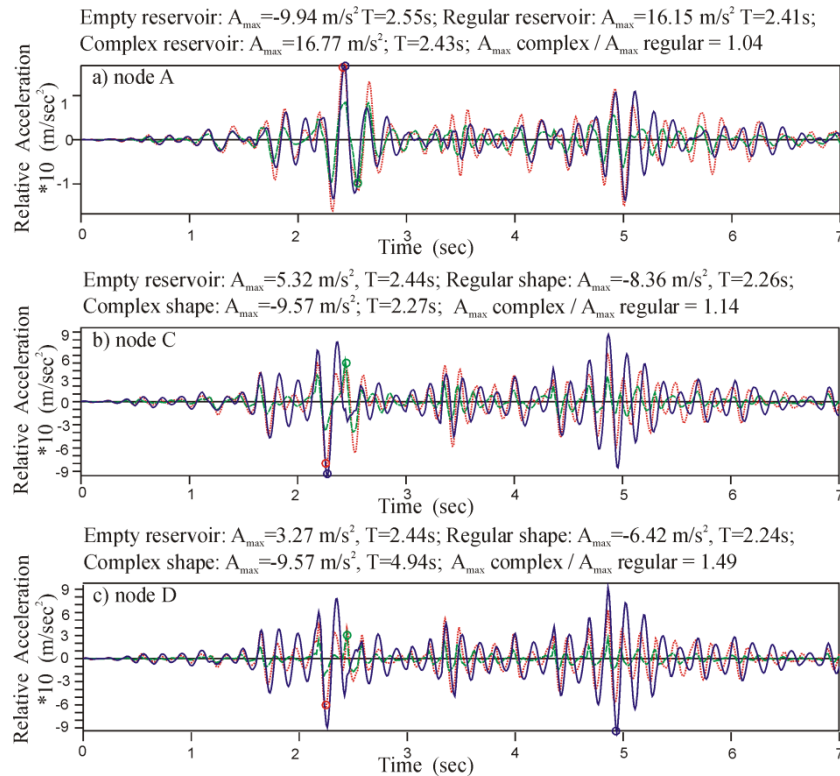


Fig.3 Relative acceleration response for seismic excitation in the stream-wise direction. Dashed line indicates response of empty reservoir, dotted line indicates response of full reservoir with regular geometry, and solid line shows response of full reservoir with complex geometry.

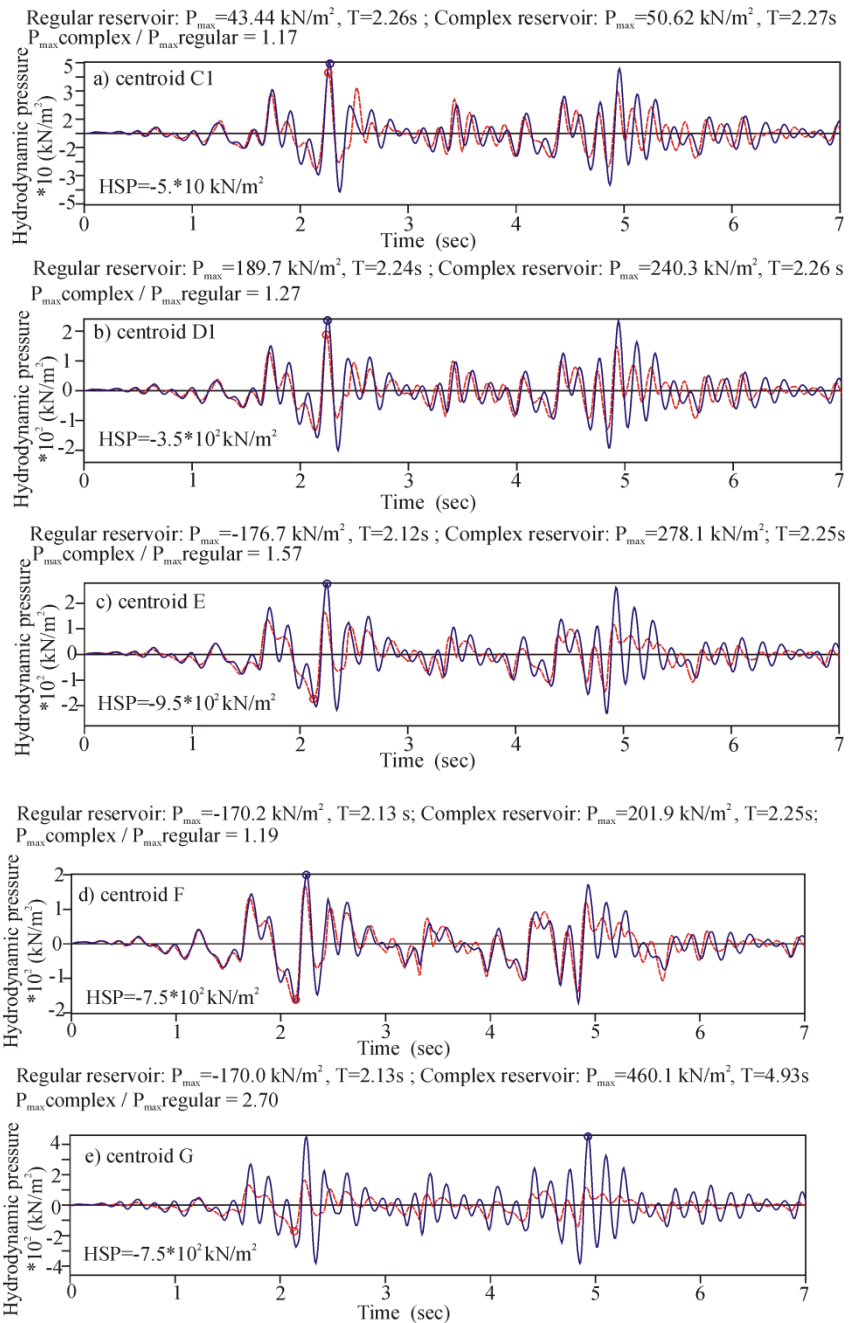


Fig.4 Hydrodynamic pressures under seismic excitation in the cross-stream direction. Dashed line indicates response of reservoir with regular geometry, and solid line indicates response of reservoir with complex geometry.

Thus the response is increased from 17% at the water mirror (centroid C1) to 170% close to the right canyon wall (centroid G). The relatively narrow canyon and twisted terrain topology particularly in the lower right corner part of the reservoir contribute to correspondingly stronger hydrodynamic shaking effects at the bottom of the dam. At dam height of 25 m close to the right canyon wall (centroid G), the angle between the dam and the terrain is acute and contributes to more than a two-fold increase of the magnitude of the maximal hydrodynamic pressure. At the same height on the opposite side of the dam, close to the left canyon wall (centroid F) the terrain is considerably smoother, which is translated in moderate amplification of the maximal hydrodynamic pressure of 19% in case of complex reservoir

4 SEISMIC EXCITATION IN CROSS-STREAM DIRECTION

Due to the cross-stream excitation, the dam exerts a buckling type of deformation with a deflection point somewhere along the central vertical axis of the dam. Therefore, in this zone are observed relatively low absolute accelerations, in particular, in the lower part of the dam. The geometric symmetry of the regular reservoir combined with the symmetric shape of the dam do not allow any significant hydrodynamic action to be exerted in the lower part of the dam, centroids D1 and E (Figs. 5b and 5c). In contrast, in the case of irregular reservoir, the analysis shows many-fold amplification of the hydrodynamic pressures in the lower part of the dam.

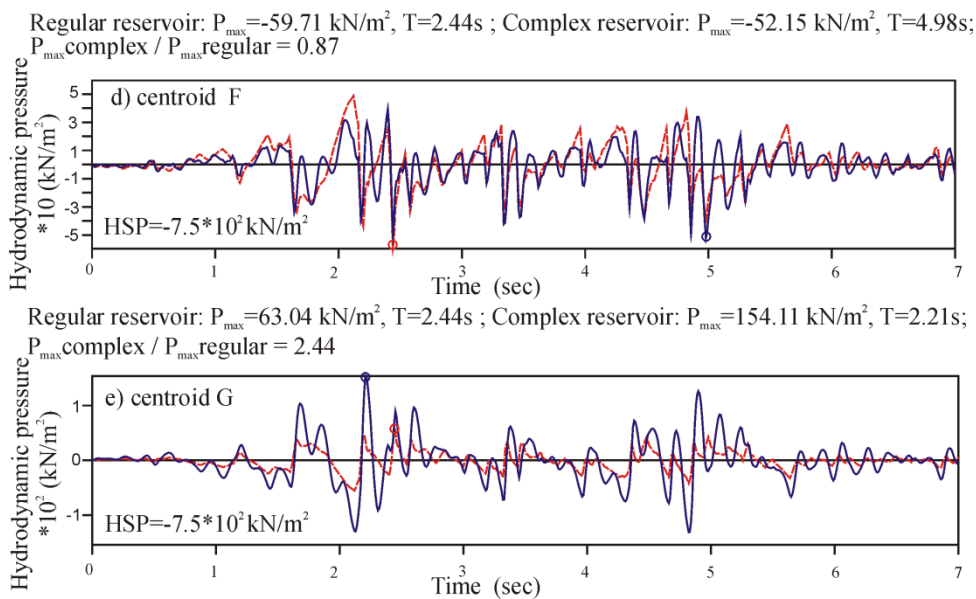


Figure 5. Hydrodynamic pressures under seismic excitation in the cross-stream direction. Dashed line indicates response of reservoir with regular geometry, and solid line indicates response of reservoir with complex geometry.

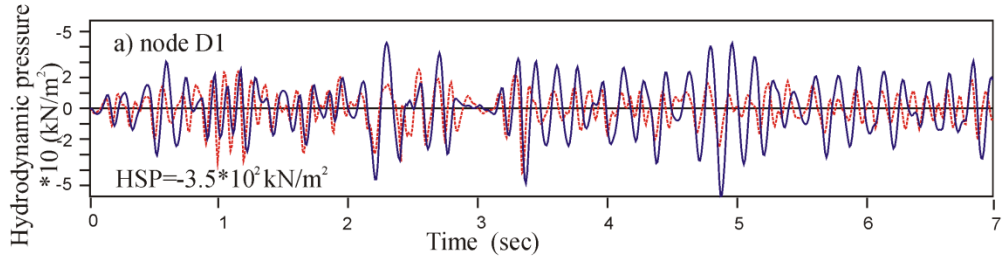
Furthermore, the maximal hydrodynamic pressures at centroids F and G for regular terrain are of the same order of magnitude but in the opposite phase (Figs. 5d and 5e). At the right abutment, centroid G, the maximal hydrodynamic pressure of 63 kN/m² is generated at 2.44 sec, whereas at the left abutment, centroid F, the maximal hydrodynamic pressure is -59 kN/m² also at 2.44 sec. These slightly asymmetrical hydrodynamic pressures are consequence of the horizontal rotation of the dam central axis for 3 degrees with respect to the stream-wise direction decided during the dam design phase. In the case of complex terrain, however, the hydrodynamic pressure is significantly amplified by the vibration of a considerably irregular right wall close to the centroid G with a maximal value of 154 kN/m² observed at 2.21 sec, (Fig. 5e), but for the centroid F, the maximal hydrodynamic pressure is -52 kN/m² and computed at 4.98 sec. (Fig. 5d). The common characteristic of both scenarios is that the pressure waves generated by the left canyon wall are followed by dilatation waves generated by the right wall and vice versa.

5 THREE – COMPONENT SEISMIC EXCITATION

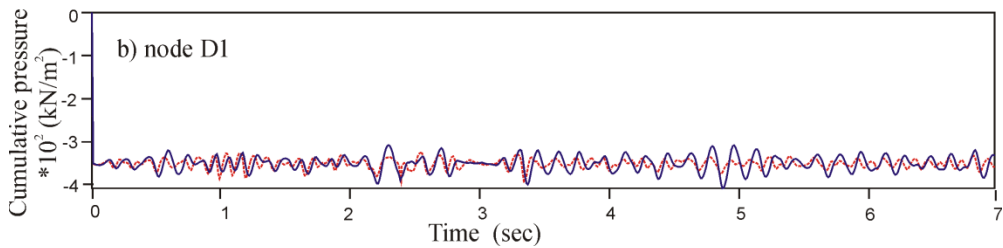
In the following example, the effects of the complex reservoir are considered under the same seismic action as in the previous cases, but set at 45 degrees with respect to the dam axis. In the simulations, the horizontal component was decomposed into one stream-wise and another cross-stream component. The obtained dynamic responses of the dam (relative displacement, velocity and acceleration; not presented graphically) are considerably lower from that obtained for stream-wise excitation. For example, the maximal relative acceleration measured at node B in the stream-wise direction is 8.3 and 9.8 m/s² for the regular and complex reservoirs, respectively. At the same time, the seismic action in the stream-wise direction yields respective maximal relative accelerations of 16.4 and 17.0 m/s², Fig. 2. This was expected, as in this case the horizontal components of the seismic excitation are about 30% lower (PGA=0.21g).

In Figs. 6 are shown the time histories of the cumulative (hydrodynamic + hydrostatic) pressures for node D1 and G. At these nodes, the hydrostatic pressure largely dominates the hydrodynamic pressure. Thus, in the case of regular reservoir, the ratio between the maximal hydrodynamic pressure and the hydrostatic pressure is 12% and 16.8% for nodes D1 and G respectively. In the case of complex reservoir, these ratios increase to 16.3% and 20.4%. On the other side, whereas the ratios between the maximal hydrodynamic pressures developed in the complex and regular reservoirs are 36% and 21% for nodes D1 and G respectively, these ratios for the cumulative pressures decrease to only 4% and 3%. In the simulations, the hydrostatic pressure remains constant and contributes to the initial stress conditions in the dam before the seismic excitation actually takes place. During the seismic excitation, however, the hydrodynamic pressure varies considerably in intensity and distribution for both complex and regular reservoirs. It modifies the dynamic response of the dam respectively to the differences in the hydrodynamic loading. Note also that the above ratios depict the differences between the maxima only.

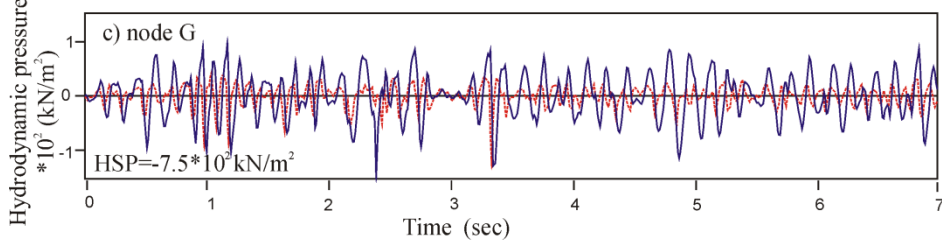
Regular reservoir: $P_{\max} = -41.9 \text{ kN/m}^2$, $T = 3.35 \text{ s}$; Complex reservoir: $P_{\max} = -57.0 \text{ kN/m}^2$, $T = 4.89 \text{ s}$;
 $P_{\max \text{ complex}} / P_{\max \text{ regular}} = 1.361$



Regular reservoir: $P_{\max} = -391.9 \text{ kN/m}^2$, $T = 3.35 \text{ s}$; Complex reservoir: $P_{\max} = -407. \text{ kN/m}^2$, $T = 4.89 \text{ s}$;
 $P_{\max \text{ complex}} / P_{\max \text{ regular}} = 1.04$



Regular reservoir: $P_{\max} = -126.3 \text{ kN/m}^2$, $T = 2.31 \text{ s}$; Complex reservoir: $P_{\max} = -152.9 \text{ kN/m}^2$, $T = 2.31 \text{ s}$;
 $P_{\max \text{ complex}} / P_{\max \text{ regular}} = 1.21$



Regular reservoir: $P_{\max} = -876.3 \text{ kN/m}^2$, $T = 2.31 \text{ s}$; Complex reservoir: $P_{\max} = -902.9 \text{ kN/m}^2$, $T = 2.31 \text{ s}$;
 $P_{\max \text{ complex}} / P_{\max \text{ regular}} = 1.03$

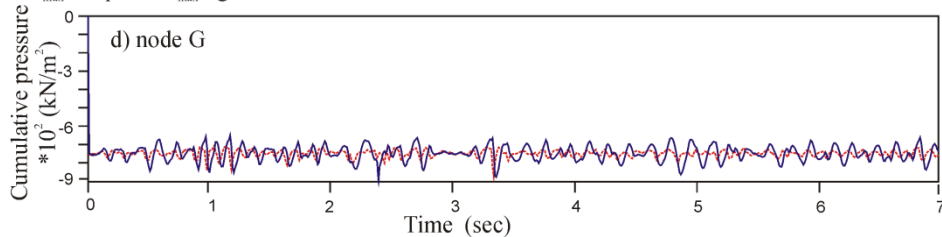


Figure 6. Hydrodynamic and cumulative (hydrodynamic plus hydrostatic) pressures at dam-fluid interface for three-component seismic excitation for centroids D1 and G. Dotted line indicates pressures for reservoir with regular geometry, and solid line shows pressures for reservoir with complex geometry.

6 MOMENTUM INDEXES

In order to quantitatively measure the hydrodynamic energy exerted at the entire dam-fluid interface for both considered reservoir configurations (regular and complex terrain), time dependent indicators referred to as ‘momentum indexes’ were developed. First, the time histories of the hydrodynamic forces, were calculated along the normal of each boundary element node, i . The momentum indexes were then determined as a sum of the integrals of the resulting time histories of the hydrodynamic forces over the interface nodes as follows,

$$I(t) = \sum_{i=1}^{NP} \int_0^t |f_i(t)| dt \quad (1)$$

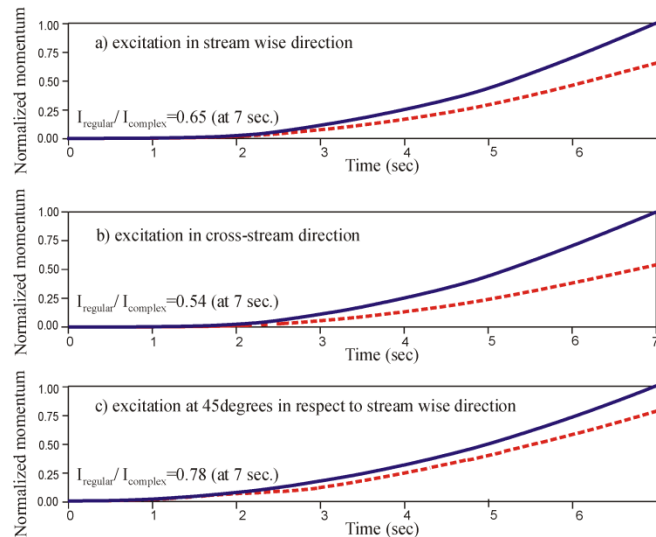


Figure 7. Normalized momentum indexes. Solid blue line indicates the momentum indexes in case of complex reservoir, and dashed red line indicates momentum indexes for regular reservoir.

For easier comparison, the momentum indexes were normalized to the maximal value obtained at 7 sec. for complex reservoir geometry. It can be observed in Figs. 7 that the normalized momentum indexes increase with time because the area bounded by the time history of the hydrodynamic force is increasing time function. What is more important, these figures show that the complex reservoir topology generates considerably higher hydrodynamic forces and respective momentum at the dam-fluid interface than the regular terrain.

7 CONCLUSIONS

The presented analyses contribute to the improved understanding of the influence of the direction of the seismic excitation on dynamic interaction with the water in the reservoir with regular and complex configuration. The analysis of the interaction was conducted applying the

BEM-FEM technique and the matrix of hydrodynamic interference. The considered direction of the incoming seismic waves was stream-wise, cross-stream and at 45 degrees with respect to the dam axis along with the concomitant vertical component scaled at 75% of the horizontal ground motion. In case of a three-component seismic excitation with direction other than the stream-wise direction i.e, parallel to the central cross section, the obtained hydrodynamic responses of the dam are considerably lower. Compared to a reservoir with regular geometry, the vibration of the arch dam and the canyon walls with complex geometry generates different wave fields of compressive and dilatational waves. This results in modification of the distribution pattern of the hydrodynamic pressures and to considerably higher magnitudes. For the stream-wise seismic excitation, the maxima of the response time histories of the hydrodynamic pressures calculated at selected observational points at the dam-fluid interface can be between 20% and 170% higher in the case of irregular reservoir. In the case of seismic force acting in the cross-stream direction, the hydrodynamic effects seem subdued for both reservoir configurations. However, the impact of the terrain irregularities is relatively higher and can be amplified for more than 10 times.

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Ivančo Simonovski¹

SANACIJA PRODORA VODE U OBJEKAT „BUDOŠ“ – NIKŠIĆM CRNA GORA

Rezime:

S-87

Kao posledica prodora vode u tunelu Budoš u Nikšiću, Crna Gora u toku eksploatacije tunel je pretrpeo vidljiva oštećenja betonske obloge koja se manifestuju u obliku prslina i proticanja vode na kolovoznu konstrukciju. U radu su prikazane faze sanacije betona prema Evropskim sandardima za zaštitu i sanaciju betonskih i AB konstrukcija EN 1504. Pri sanaciji ovog tunela primenjen je metod za sanaciju i zaustavljanje aktivnih prodora vode injektiranjem poliuretanskih smola i reparaturnih maltera.

Ključne reči: sanacija, tunel, injektiranje

REPAIR OF WATER INGRESS IN “BUDOSH” TUNNEL – NIKŠIĆ, MONTENEGRO

Summary:

As a result of water ingress in the Budosh tunnel in Nikšić, Montenegro, during its service life the structure has suffered significant damages to the concrete tunnel lining, which are visible in the form of cracks and water leakages into the road surface. This paper examines the phases of repair according to European Standard for Protection and Repair of Concrete and Reinforced Concrete Structures – EN 1504. The repair works on this tunnel were carried out by applying the method of repair and stopping of water ingress by injecting polyurethane resins and repair mortars.

Key words: repair, injecting, tunnel

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1 INTRODUCTION

Water is the biggest enemy of tunnel structures since it causes problems during the excavation and leads to additional costs for rock/soil stabilisation. It usually causes difficulties during the working life of the tunnel, which sometimes have an impact not only on the concrete tunnel lining, but also on the tunnel fixtures (lighting, traffic signalisation).

Damaging effects of water on tunnels may be classified by:

External effects (on the surroundings of the tunnel, but without any effect on the structure)

Structural effects (on the structure of the tunnel)

Functional effects (on the functionality of the tunnel)

Not all the defects that can arise as a result of damaging effects of water, can be predicted at the time of tunnel design. Some of them can be noticed only after longer period of use of the tunnel.

Damages can be observed in the concrete lining of many tunnels built in the 1970s. Usually, they appear as a result of water infiltration as the leading factor that poses a threat to the service life of the structure.

One of the tunnels in which such a problem was identified is the “Budosh” tunnel built in 1977. It is located near the town of Nikšić in Montenegro. The tunnel is 1,114.0 m long, 8.0 m wide and 6.0 m high.



Figure 1 - The “Budosh” tunnel before repair.

Establishing the causes of the problems that have occurred during the working life of the structure is quite a complex issue and most often it is a result of several factors.

Main problem is the occurrence of water ingress through cracks in the concrete, at the joints in concrete building and visible damages to the concrete lining caused by aggressive forces, such as those caused by ice, salt and carbonation.

One of the reasons for the appearance of water inflow occurred at the phase of blasting of the tunnel. The loosening of the rock above the tunnel is more expressed at the entrance end exit of the tunnel which probably comes from the change in geological composition of the rock.

The reason for decreased functionality of the drainage channel is believed to be the irregular check and cleaning of the channel, so it cannot accept all underground waters.

This enabled the water to cause partial washing out of the cement mortar from the concrete, which led to deterioration of the concrete. Consequently, cracks have appeared in the concrete structure over time and water ingresses through it. Water ingress is also observed in the joints in concrete building, which threatens traffic safety, especially in times of heavy rain.

2 STATE OF THE TUNNEL BEFORE REPAIRATION

Across its entire length, the tunnel is exposed to constant hydrostatic pressure from the water penetrating through the rock above the tunnel. The problems with water ingress have been observed at the entrance and exit of the tunnel covering a total length of 600 m, while the damages to the protective coating on the concrete surface is registered along the tunnel at a height of 2.5 m. Water ingress is noticeable across the entire section of the tunnel, as well as in the joint between the reinforced concrete wall and the sidewalk. It makes the road surface wet and damages the protective coating of the concrete surface up to 2.5 m in height. The width of the cracks varies, ranging from 1.0 mm up to several centimetres.

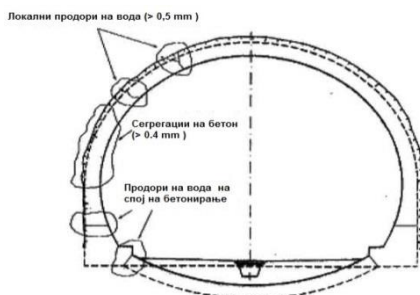


Figure 2 - Characteristic positions of water ingress



Figure 3 - Damages to the protective coating of the concrete surface



Figure 4 - Water ingress at the crown



Figure 5 - Water ingress at a joint of concrete building.

3 THE PROJECT PHASES OF THE CONCRETE REPAIR AND PROTECTION PROCESS

The European Standard for Protection and Repair of Concrete and Reinforced Concrete Structures - EN 1504 provides for the following phases:

1. **Information about the Structure** (history of the structure - documentation, maintenance)
2. **Process of Assessment** (Identifying the defects, Identifying the causes of the potential defects, Assessment of the state of the structure)
3. **Management Strategy/Plan** (Considering the options, Establishing the principles of repair/protection, Establishing the necessary safety measures)
4. **Design of Repair Work** (Intended use of materials and technologies, Defining the conditions of – the substrate, products, application, specification, technical documentation)
5. **Repair Work** (Application of the selected work materials and technologies, Quality control, Application of the selected safety measures)
6. **Acceptance of Repair Work** (Testing the repair work, Acceptance of the documentation, Establishing maintenance measures)

The project of the concrete repair works at the Budosh tunnel includes stopping any active water ingress through the cracks that would enable continuous flow of traffic. It is also necessary to apply anti-corrosion coatings on the concrete lining of the tunnel for protection against aggressive attacks of ice, salt and carbonation. Primary factors that are taken into consideration for selecting the materials for repairing the tunnel are the size and number of cracks, their position and the reason they occurred.

Considering to the state of the concrete lining and the amount of water that exerts pressure on the tunnel, it was decided that the cracks measuring 1.0-5.0 mm in width should be treated by injecting polyurethane resins for quick stopping of the water and filling the cracks with highly elastic material.

In the case of cracks measuring more than 5.0 mm in width, water ingress should be stopped by applying the quick-setting agent **Ubrzuvac C** (Accelerator C) and then sealing the cracks by applying **Reparatur Malter – F** (repair mortar).

The injection materials differ in their characteristics (setting time, viscosity) depending whether they are used for stopping active water ingress, injection in media (porous, closed structure), rock and soil stabilisation, etc.

Depending on the intended purpose of use, there are three categories of injection materials:

- structural fulfilling cracks with a material that transmits strain.
- ductile filling of cracks.
- filling cracks with expanding material.

3.1 REPAIR PHASES

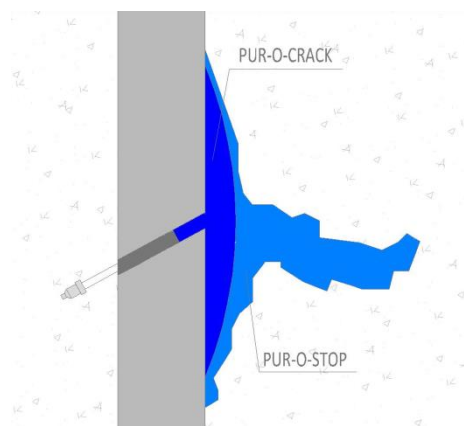
In order to repair the active water ingress through the cracks of the Budosh tunnel, a system of polyurethane expanding foam and polyurethane resin was selected. One of the preconditions for selection of these materials is that they can be injected by using a manual

one-component injector, which renders the repair works more easy and economical. The repair process developed in 3 phases:

1st PHASE - elimination of all unstable parts from the substrate by sandblasting.

2nd PHASE - sealing and injecting any active water ingress.

The cracks were treated by injecting **PUR-O-STOP** (TPH - Hamburg) – one-component polyurethane resin – which blocks the active water ingress by quick expansion and fills the caverns and cracks in the concrete structure. The material has a relatively low viscosity of 230 (mPaS) which makes it suitable for injection in media with cracks measuring 1.0 to 5.0 mm in width.



As soon as the water ingress has been stopped, **PUR-O-CRACK** (TPH - Hamburg) – elastic two-component polyurethane resin – should be injected in the cracks. It has high ductility and can match the expansions of the structure (compliant to EN 1504-5). The viscosity of the material (160 mPas) makes it possible for all micro-cracks in the concrete to be easily filled. By injecting the material, the resistance of the polyurethane foam (**PUR-O-STOP**) to hydrostatic pressure is increased.

3rd PHASE - restoration of the damaged concrete surface by applying **REPARATUR MALTER F** (ADING AD) – polymer-modified cement-based repair mortar, as well as by applying **ANTI-KOROZIN BB** (ADING AD) – anti-corrosion protective coating on the concrete surface.

I. Preparing of the substrate

First of all, sandblasting was carried out across the entire length of the tunnel up to 2.5 m in height and all unstable positions of the substrate were removed.

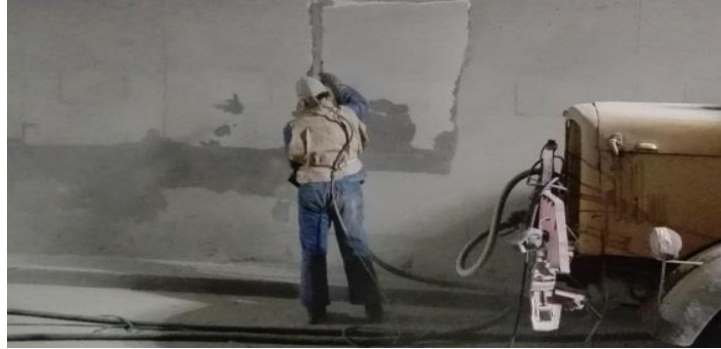


Figure 6 - Dry sandblasting

II. Sealing and injecting of the active water ingress

The cracks in which water ingress has been observed were sealed by injecting and filling them with quick-setting cement.

On both sides of the crack, bores were drilled at an angle of 45° from the substrate and at a distance of 30 cm from each other, measuring around 15 cm in depth by using a drill. The position and the distance between the bores are defined depending on the crack and the amount of water that penetrates through.

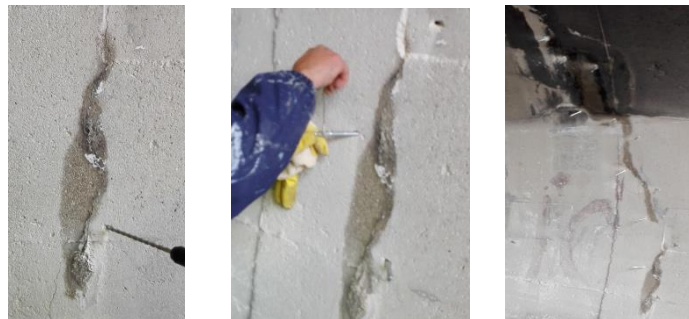


Figure 7 - Drilling and fixing metal packers

Metal packers were fixed in the bores. The polyurethane resins (Pur-O-Stop / Pur-O-Crack) were later injected through them by using a manual injector (AKKUIINJEKTOR). Injecting has been carried out under a pressure of 150 -200 bar, bottom-up, in order to avoid any air entrapment.



Figure 8 - Injecting a crack

In the cracks that appear along the joint between the crown and the invert of the tunnel, which have active water ingress, a slit (triangle) was cut mechanically along the length of the crack.

The active water ingress was closed by applying the quick-setting agent Ubrzuvac C. After having sealed the crack, the entire slit is filled with Reparatur Malter-F – polymer-modified cement-based repair mortar.



Figure 9 - Sealing with Ubrzuvac C and Reparatur Malter-F

III. Anti-corrosive protection

After closing of all active water ingress on the surface, on the already prepared substrate should be applied anti-corrosive acrylate-based coating Antikoroziin BB , by using an airless pump.



Figure 10 - Protective coating - Antikoroziin BB

4 CONCLUSION

This type of repair works are relatively complex because the reparation is made during exploitations of the tunnel and most often there are not enough accurate information about the area surrounding the structure. Different types of injection products have been developed, which require sophisticated equipment, high precision and extensive experience of the equipment operator.

The entire process of injecting can be carried out in several different ways, depending on the conditions and requirements of the structure. The Budosh tunnel required the injection of cracks with relatively small length, without the installation of a waterproofing system and full shotcrete application on the secondary layer, which would largely increase the price of the overall repair works on water ingress in the structure. By injecting polyurethane resins in the cracks, 90% of the active water ingress was sealed in a relatively short period of time. This prevented further deterioration of the concrete and enabled continuous flow of traffic in the tunnel.

This type of repair works involving the injection of polyurethane resins is applied in many residential buildings, hydro-technical facilities, biogas tanks, etc. It is necessary to increase the caution when designing the waterproofing of the structure, in order to avoid multiple repair works that would increase the cost of the tunnel.



Vladan Kuzmanović¹, Veljko Koković², Milan Spremić³

REKONSTRUKCIJA SIFONA HIDROELEKTRANI „ZVORNIK“

Rezime:

88
S-S

Šezdeset godina posle završetka izgradnje Hidroelektrane Zvornik, započela je revitalizacija agregata A1. Povećanje instalisane snage uslovalo je zamenu agregata i povećanje protoka. Da bi se ovo omogućilo, neophodno je izvesti rekonstrukciju sifona, koja podrazumeva promenu oblika i površine svetlog otvora sifona neposredno ispod agregata. Promena geometrije sifona obuhvata demontažu postojeće čelične obloge sifona, uklanjanje okolnog dela sekundarnog betona, montažu nove čelične obloge i njenu monolitizaciju sa postojećom armirano betonskom konstrukcijom brane.

Ključne reči: Hidroelektrana, rekonstrukcija, sifon

REVITALIZATION OF „ZVORNIK“ HYDROPOWER PLANT

Summary

Sixty years after the construction of HPP Zvornik, revitalization of the aggregate A1 began. An increase in the installed capacity is provided by the water flow rate rise and the replacement of turbine generators. To make this possible, it is necessary to modify the geometry of the tube cone below the unit. Civil engineering part of the reconstruction works includes: dismantling of the existing steel tube cone lining, removal of the surrounding secondary concrete, assembling the new steel draft tube cone and connecting it with the existing dam concrete structure.

Key words: Hydropower plant, reconstruction, draft tube cone

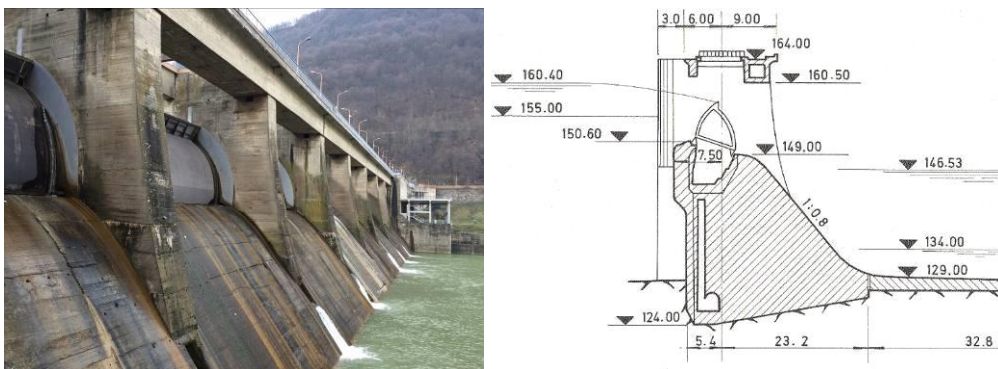
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1 UVOD

HE „Zvornik“ je pribransko postrojenje koje se sastoji od dve priobalne mašinske zgrade i prelivnog dela brane između njih. U mašinskim zgradama su smeštena po dva turbinska agregata tipa „Kaplan“, svaki instalisane snage od 24 MW, sa protokom od $155,00 \text{ m}^3/\text{s}$. Gravitaciona betonska brana „Zvornik“ izgrađena je 1955. godine u donjem toku reke Drine, kod mesta Mali Zvornik. Prelivni deo brane ima 8 prelivnih polja dužine po 18,0 m, a prelivi su kontrolisani radijalnim ustavama visine 8,0 m. Na brani postoje 4 temeljna ispusta za pražnjenje akumulacije i riblja staza uz levu obalu za migraciju riba. U branu je ugrađeno 316.000 m^3 betona i formirana je akumulacija zapremine od $89,0 \times 10^6 \text{ m}^3$.



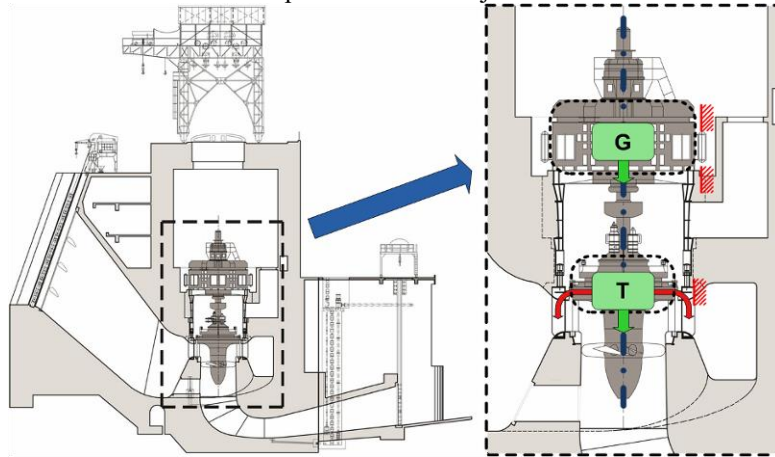
Slika 1 – Izgled sa nizvodne strane, poprečni presek brane

Građevinska visina brane je 41,00 m, a dužina u kruni 269,40 m. Uzvodno lice je vertikalno, sa previsom, dok je nizvodna kosina u nagibu 1:0,8. Nizvodno od preliva je slapište tipa „bučnice“, dužine 32,80 m. Kota normalnog uspora je 155,00 m, kota maksimalnog uspora hiljadugodišnje velike vode je 160,40 m, a maksimalna visina prelivnog mlaza 9,80 m. Kota krune brane je na 164,00 m (slika 1). Preko preliva je izgrađen most širine 15,0 m, na kome je kran koji opslužuje hidromehaničku opremu. Nakon šezdeset godina upotrebe, započeta je revitalizacija prvog agregata.

Revitalizacija je podrazumevala povećanje instalisane snage sa 24 na 37,5 MW po agregatu, što je obuhvatalo zamenu celokupnog agregata (generatora i turbine) i rekonstrukciju sifona kako bi se omogućilo uvećanje protoka sa 155 na $165 \text{ m}^3/\text{s}$. Novi agregat je konstruisan da gabaritno odgovara postojećoj konturi plašta turbinske jame, dok je u cilju povećanja protoka bilo neophodno izmeniti geometriju izlaznog sifona. Na delu između kućišta radnog kola turbine i kolena sifona, rekonstrukcija je podrazumevala povećanje prečnika svetlog otvora sifona i izmenu geometrije na dužini od 4,8 m, do postojećeg kolena sifona.

Na slici 2 su prikazani osnovni delovi agregata, koji se sastoji od generatora i turbine povezanih vertikalnim vratilom. Konstrukcijski, rotor generatora, vertikalno vratilo i turbina čine jednu, pokretnu celinu, a stator generatora drugu, nepokretnu. U gravitacionom pravcu, agregat se na konstrukciju brane oslanja posredno, preko statorskih lopatica, i to: pokretna celina preko poklopca turbine, a nepokretna preko plašta turbinske jame. Bočno pomeranje

vratila je sprečeno vodećim ležajevima u tri tačke po visini, dok je horizontalna nepomerljivost statora obezbeđena krutom vezom sa plaštom turbinske jame.



Slika 2 - Poprečni presek kroz mašinsku zgradu

Imajući u vidu različite karakteristike postojećeg i novog agregata, bilo je neophodno izvršiti kontrolu naprežanja u elementima postojeće konstrukcije. Proračun statičkih i dinamičkih uticaja i dokaz nosivosti postojećeg plašta turbinske jame i statorskih lopatica su bili deo mašinskog projekta, dok je kontrola veza sa okolnom masivnom betonskom konstrukcijom brane činila deo građevinskog projekta. Prema rezultatima proračuna, zamena ili ojačanje ovih konstruktivnih elemenata nije bila potrebna, tako da se najveći i najznačajniji deo građevinskih radova odnosi na radove u postojećem sifonu, koji su opisani u nastavku rada.

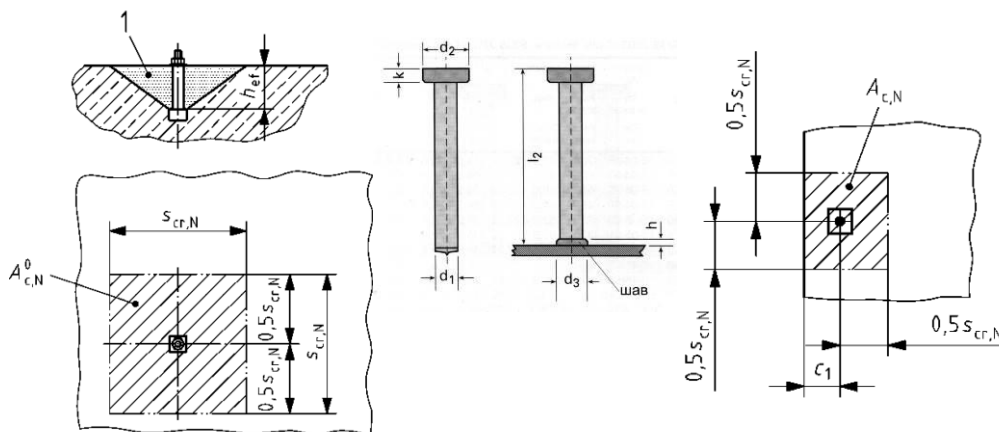
2 REKONSTRUKCIJA SIFONA

Ugradnja agregata sa većom instalisanom snagom je podrazumeva i povećanje protoka, što je postignuto izmenom geometrije sifona. Voda se u postrojenje uvodi preko dva turbinska zahvata, nakon čega se sprovodnom spiralom usmerava na stacionarne vertikalne lopatice turbine. Prolaskom preko stacionarnih lopatica nastaje vrtloženje vode koje pokreće lopatice turbine, a voda se dalje preko turbinskih lopatica usmerava kroz sifon na izlazne vade. Geometrija sifona je složenog geometrijskog oblika koji se sastoji od cilindra prečnika 465 cm i dužine cca 175 cm na vrhu, nakon čega se poprečni presek postepeno menja sa kružnog na pravougaoni, do kolena sifona. Pri tome, površina poprečnog preseka je promenljiva, odnosno, povećava se sa udaljavanjem od turbine. Usled vrtložnog kretanja vode velikom brzinom, površina sifona je izložena sišućim naponima, koji prema modelskim ispitivanjima proizvođača opreme dostižu vrednost i do 0,1 MPa. Sa povećanjem poprečnog preseka sifona naponi zatezanja opadaju, tako da na delu kolena prelaze u napone pritiska. Zbog ovakvog naponskog stanja i velike brzine vode, celokupna površina sifona je obložena čeličnim limom debljine 30 mm, koji je u okolnu masivnu betonsku konstrukciju brane ankerovan horizontalnim ukrućenjima dužine 15-20 cm. Rekonstrukcija sifona je u cilju uvećanja protoka obuhvatala povećanje najmanjeg poprečnog preseka sifona. Prečnik cilindričnog dela trebalo je povećati sa 465 na 485 cm, a na ostatku dužine sifona obezbediti postepeni prelaz na poprečni

2.2 VEZA OBLOGE SA KONSTRUKCIJOM BRANE

Projektjni kriterijum za vezu čelične obloge konusa sifona sa postojećom armirano betonskom konstrukcijom je minimalni pritisak (sisanje) na površini konusa koji nastaje pri protoku vode. Računsko površinsko opterećenje od $-0,1$ MPa je korišćeno i za proračun moždanika sa glavom (tj. za vezu obloge sifona sa sekundarnim betonom) iz za vezu sekundarnog betona sa postojećim na konturi sečenja.

Proračun ankera i veza je sproveden prema odredbama grupe standarda SRPS CEN/TS 1992-4 [2]. Pri proračunu su usvojene pretpostavke da je beton nearmiran i da se ankeri ugrađuju u neispucali beton. Proračun nosivosti na zatezanje se zasniva na konceptu nosivosti na zatezanje efektivnog konusa betona u koji je moždanik ankerovan (slika 4).



Slika 4 – Efektivne površine za proračun nosivosti na zatezanje ankera

Karakteristična nosivost ankera na čupanje za neispucali beton prema [3] data je izrazom:

$$N_{Rk,c}^0 = 11,9 \cdot \sqrt{f_{ck,cube}} \cdot h^{1,5}_{ef}$$

Karakteristična nosivost za grupu ankera prema pomenutom standardu računa se na sledeći način:

$$N_{Rk,c} = N_{Rk,c}^0 \cdot \frac{A_{c,N}}{A_{c,N}^0} \cdot \Psi_{s,N} \cdot \Psi_{re,N} \cdot \Psi_{ec,N}$$

gde su:

$\Psi_{s,N}$ – koeficijent kojim se uzima u obzir rastojanje ankera od ivice betona,

$\Psi_{re,N}$ – koeficijent kojim se uzima u obzir uticaj armature,

$\Psi_{ec,N}$ – koeficijent efekta ekscentriciteta opterećenja,

Odnos površina $A_{c,N}/A_{c,N}^0$ predstavlja odnos efektivne površine za grupu ankera i efektivne površine jednog ankera. Površina $A_{0c,N}$ je funkcija dubine ankerovanja i jednaka je:

$$A_{c,N}^0 = s_{cr,N}^2 = (3 h_{eff})^2 = 9 h_{eff}^2$$

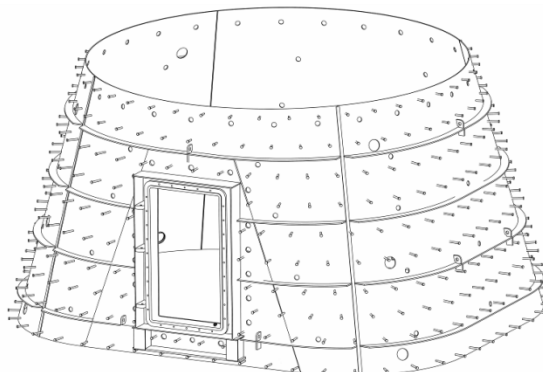
Za vezu nove obloge konusa sa betonom projektovani su moždanici sa glavom prečnika 19 mm, dužine 100 i 150 mm. Moždanici su izvedeni u pogonu. Moždanici su postavljeni u

rasteru od 365 mm u dva pravca po celoj površini obloge sifona, videti sliku 5 i 6. Proračunom je dokazana nosivost moždanika na zatezanje u sekundarnom betonu. Sekundarni beton je armiran armaturom u dva pravca. Deo armature je izveden na licu mesta u sifonu, a deo armature je montiran zajedno sa oblogom sifona.

Takođe, imajući u vidu nemogućnost kontrole veze postojećeg primarnog i naknadno ugrađenog sekundarnog betona, spoj je dodatno ojačan armaturnim ankerima, prečnika 22 mm, videti sliku 6. Raster ankera za vezu sekundarnog i primarnog betona je 500 mm. Dubina bušenja ankera je 250 mm.



Slika 5 – Ankeri za vezu obloge sifona sa primarnom AB konstrukcijom



Slika 6 – Model nove obloge konusa sifona [1, 2]

Na delu konusa na kome je obloga uklonjena sečenjem, projektom je predviđena zamena isečene armature odgovarajućom armaturom sa jednakim prečnikom i rastojanjem. Nova armatura se montira na prethodno pomenute armaturne ankere, za koje je predviđeno hemijsko ankerovanje dvokomponentnom epoksidnom anker smesom. Izabrana je epoksidna smesa za ankerovanje, koja sme da se koristi u vodi, zbog mogućnosti da se u zonama betona u kojima je predviđeno ankerovanje pojavi voda. Veza starog i novog betona projektovana je takođe na maksimalno sišuće opterećenje koje se može javiti unutar sifona.

Po završenoj montaži armature, pristupa se montaži nove obloge konusa sifona (slika 6), koja se u fazi montaže dovodi u projektovani položaj pomoću montažnih ankera. Projektovani

3 ZAKLJUČAK

Revitalizacijom Hidroelektrane Zvornik predviđena je sukcesivna zamena sva 4 agregata. Pošto se karakteristike novih agregata razlikuju od postojećih, izvršena je kontrola nosivosti veza agregata sa betonskom konstrukcijom brane. Za vezu obloge sifona sa sekundarnim betonom projektovani su moždanici sa glavom, a za vezu primarnog i sekundarnog betona armaturni ankeri. Rešen je prolazak sabirnica kroz masivni nizvodni zid brane i izvršena rekonstrukcija montažne platforme novog rotora. Projektovanim građevinskim radovima omogućena je uspešna revitalizacija Hidroelektrane Zvornik, kojom se ukupna instalisana snaga povećava sa 98 na 150 MW.

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PRORAČUN ZAŠTITNE KOŠINE NA TRASI AUTOPUTA E75 TUNEL PREDEJANE-CARIČINA DOLINA

Rezime:

S-89

U radu je prikazan statički proračun zaštitne konstrukcije padine na trasi autoputa E75 tunel Predejan- Caričina dolina. Zaštitna konstrukcija se sastoji od zida u vidu armiranobetonskih sanduka koji su sa donje strane uklješteni u stenski masiv, a sa gornje strane pridržani geotehničkim ankerima. Primenom MKE i računskog programa PLAXIS 2D dobijeni su rezultati na osnovu kojih je izvršeno dimenzionisanje konstruktivnih elemenata. Takodje, u radu je dat kratak opis faza izvođenja ovih radova.

Ključne reči: zaštita kosine, armiranobetonski sanduci, geotehnički ankeri

STATIC CALCULATION OF SLOPE PROTECTION ON THE ROUTE OF THE HIGHWAY E75 TUNEL PREDEJANE- CARIČINA DOLINA

Summary:

The paper presents a static calculation of slope protective structure on the route of highway E75, tunnel Predejane-Caričina dolina. Slope protective structure consists of a wall from reinforced concrete crates which is at the bottom side restrained in the rock, and supported on the top by geotechnical anchors. Using FEM and program PLAXIS 2D results used for the dimensioning of structural elements are obtained. Also, the paper presents a brief description of the execution of all phases of work.

Key words: slope protection, reinforced concrete crate, geotechnical anchors

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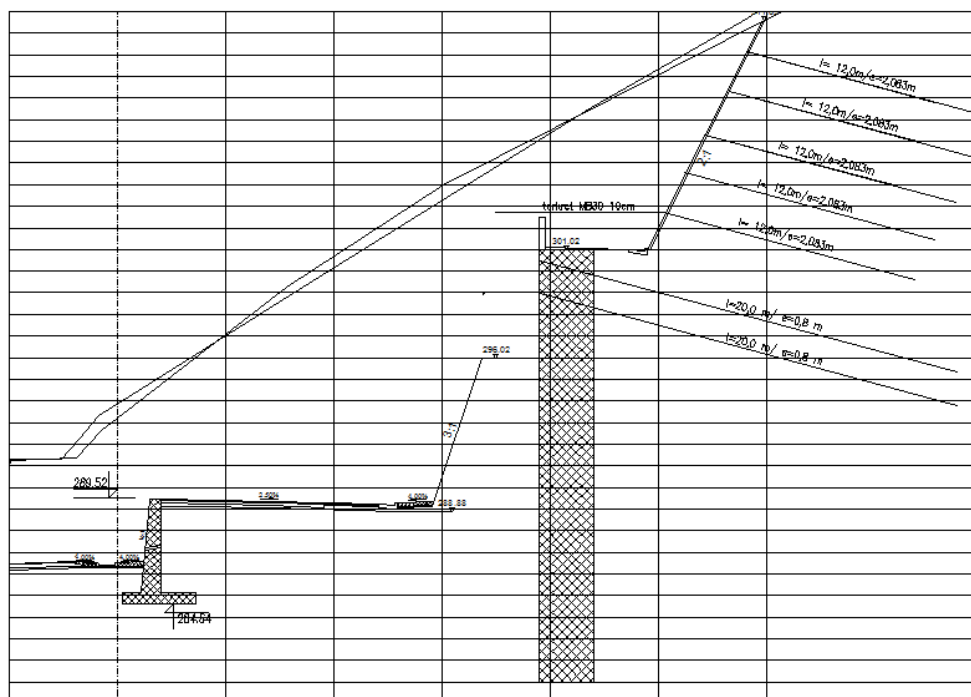
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1 UVOD

Za potrebe izgradnje autoputa E75, urađen je projekat zaštite kosine 6 od km 882+200 do km 882+720. Na ovome delu autoput prolazi po ivici padine i za njegovu izgradnju potrebno je značajno zasecanje padine. Visina ovog zasecanja je od 5.00m do preko 40.00m. Usled ovog zasecanja padina, koja je predhodno bila stabilna, postaje nestabilna, pa je potrebno projektovati neku zaštitnu konstrukciju kako bi se obezbedila potrebna stabilnost. U radu je prikazano projektovano rešenje u kome se zaštitna konstrukcija izvodi u vidu zida od armiranobetonskih sanduka koji su sa donje strane uklješteni u stenu, a sa gornje strane pridržani geotehničkim ankerima, koji se takođe sidre u stenski masiv iza potencijalnih kliznih ravni. Sanduci su u osnovi dimenzija 2.50x1.20m, a njihove prosečne visine su oko 20.00m. Iznad sanduka izvodi se berma širine oko 4.00m, a iznad nje kosina u nagibu 2:1 sve do površine postojećeg terena. Da bi se obezbedila stabilnost ovakve kosine izvode se geotehnički ankeri, a površina terena oblaže se torkret betonom koji je armiran mrežama. Na donjem delu između sanduka i autoputa predviđeno je izvođenje berme koja se oblaže geotehničkim mrežama.

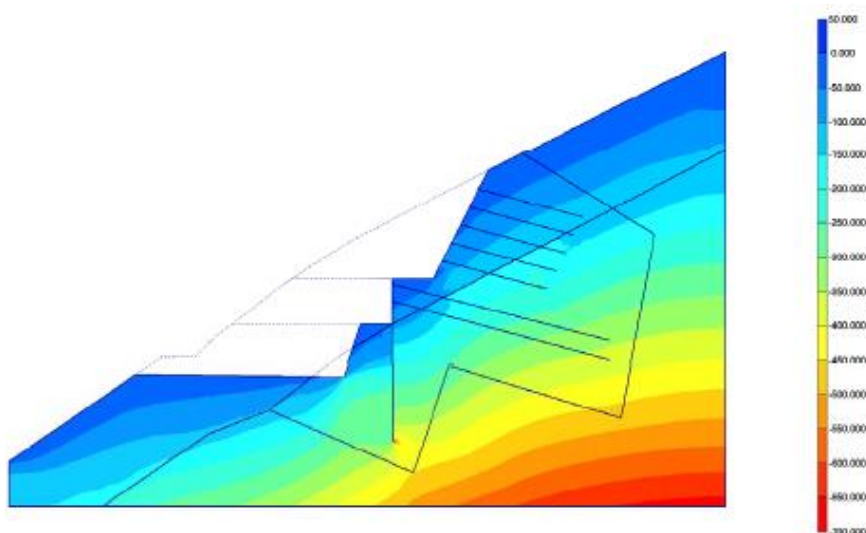
Na delu gde su visine zasecanja padine veće projektovano je izvođenje sanduka i ankeri u dva reda po visini. Na ovome delu, prvo se izvode sanduci i ankeri gornjeg reda, a zatim sanduci i ankeri donjeg reda. Ovi redovi su međusobno smaknuti, tako da se ankeri donjeg reda mimoilaze sa izvedenim sanducima gornjeg reda.



Slika 1 – Poprečni presek km 882+275

2 PRORAČUN ZAŠTITNE KOSINE

Proračun opšte i lokalne stabilnosti padine urađen je u programu PLAXIS 2D u dva karakteristična poprečna preseka. Korišćenjem podataka iz geomehaničkog elaborata usvojeni su merodavni parametri otpornosti tla za proračun globalne i lokalne stabilnosti padine u toku izvođenja radova na zasecanju padine, kao i kasnije, u fazi eksploatacije. U programu PLAXIS 2D korišćenjem MKE stačunata su naponsko deformacijska stanja u tlu usled sopstvene težine tla, kao i usled seizmičkih uticaja. Ovi proračuni urađeni su za postojeće stanje padine, kao i za stanja u kojima se padina nalazi u toku njenog zasecanja i izvođenja zaštitne konstrukcije. U programu postoje mogućnosti za korišćenje različitih konstitutivnih modela tla kako bi se proračuni što više približili realnom stanju na terenu. Pored proračuna naponsko deformacijskih stanja u svim elementima, proračunom su obuhvaćene i analize stabilnosti padine. Korišćenjem postupka redukcije parametara otpornosti tla, dolazi se do stanja kada padina postaje nestabilna. Na osnovu odnosa čvrstoće tla pre redukcije i posle redukcije parametara određuju se globalni i lokalni faktori sigurnosti padine na klizanje.



Slika 2 – Prikaz totalnih napona - presek km 882+275

U sanducima su za potrebe dimenzionisanja armature sračunati statički uticaji za tri karakteristične faze:

U ovoj fazi sanduci se ponašaju kao konzolni elementi koji su elastilno uklješteni u stenski masiv. Ova faza odgovara fazi izvođenja radova kada su izvedeni sanduci, a nisu izvedeni geotehnički ankeri. Za ovu fazu pri dimenzionisanju preseka računato je sa faktorima sigurnosti 1.40, odnosno 1.30 sa seizmikom.

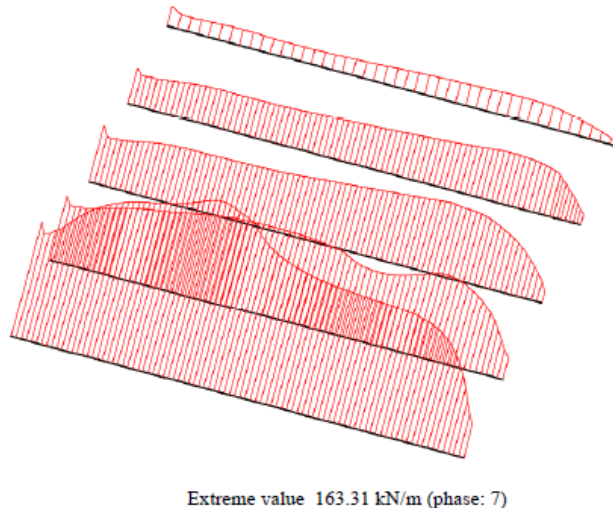
Ova faza je faza eksploatacije, iskop za sanduke je izveden do konačne dubine i izvedeni su geotehnički ankeri koji su predhodno napregnuti sa 1/3 od njihove nosivosti. Pri dimenzionisanju sanduka za ovu fazu korišćeni su faktori sigurnosti od 1.80. Ovi faktori su usvojeni, jer se opterećenje koje deluje na sanduke može tretirati kao stalno, ali i kao i

pokretno. Usled lokalnih pomeranja u tlu na pojedinim sanducima može doći do povećanja optrećenja, pa su zbog toga usvojeni veći faktori sigurnosti.

U ovoj fazi analizirano je stanje kada je usled povećanih bočni pritisaka tla došlo do iscrpljenja nosivosti izvedenih ankeri. Pri dimenzionisanju sanduka u ovoj fazi korišćeni su faktori sigurnosti 1.60.

Pored dimenzionisanja sanduka izvršeno je i dimenzionisanje geotehničkih ankeri. Na osnovu sračunatih sila određen je potreban broj ankeri i usvojen tip ankeri. Usvojeni su samobušajući ankeri IBO sistema DYWIDAG. Ankeri su dimenzionisani za nosivost materijala od koga su izvedeni, kao i za nosivost sidrišnog dela ankeri, odnosno nosivosti tla na trenje po omotaču bušotine.

Na delu iznad sanduka predviđeno je izvođenje zasecanja i formiranja kosine u nagibu 2:1. Visine ovih zasecanja terena su različite i u zavisnosti od toga usvojeni su geotehnički ankeri u više redova. Na ovome delu kosina se pokriva prskanim betonom koji se armira armaturnim mrežama. Na osnovu sračunatih sila određen je potreban broj i raspored geotehničkih ankeri.

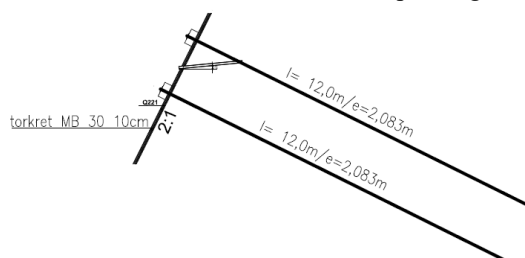


Slika 3 – Aksijalne sile u geotehničkim ankerima - presek km 882+275

Na delu ispod sanduka projektovano je izvođenje bermi čije su visine od 6.00 do 8.00m i koje se izvode u nagibu 3:1. Na gornjem delu berme izvodi se ravan deo čija je širina oko 2.50m. Ovim bermama prihvata se deo bočnih pritisaka tla, tako da se smanjuju statički uticaji u sanducima i njihova pomeranja. Da bi se obezbedila trajna stabilnost ovih bermi, projektom je predviđeno da se izvrši njihovo oblaganje mrežom koja se kratkim ankerima pričvršćuje za stabilne slojeve tla. Posle određenog vremena na mestima ovih mreža formira se rastinje koje potpuno pokriva bermu.

3 IZVOĐENJE ZAŠTITNE KONSTRUKCIJE PADINE

Zaštita padine generalno se izvodi odozgo na dole. Prvo se izvodi pristupni put sa koga se manjim bagerima vrši iskop kosine iznad zaštitne konstrukcije od sanduka. Ovaj iskop izvodi se kampadno (dužine kampada ne veće od 15.00m) sa visinama zasecanja oko 2.00m. Posle završenog iskopa kosine izvodi se sloj torkret betona debljine oko 10cm. Ovaj sloj se izvodi u dve faze i armira se armaturnom mrežom. Posle završenog torkreta, izvode se geotehnički ankeri. Ankeri su samobušajući IBO (R32N, R38N, R51L i R51N), prema standardu EN 10083. Dužine ovih ankera su od 6.00m do 15.00m i oni se izvode pod nagibom od 15°.



Slika 4 – Detalj izvođenja torkret betona

Bušenje za ankere izvodi se krunicom prečnika $\varnothing 76\text{mm}$. Na mestima čvršće stene, za bušenje može se upotrebiti udarni čekić. Popunjavanje bušotine oko ankera vrši se injektiranjem cementnom emulzijom. Da bi se obezbedila dovoljna nosivost ankera, predviđeno je da se iznad torkreta izvode ankerni armiranobetonski blokovi dimenzija $50 \times 50 \times 25\text{cm}$. Iznad blokova montiraju se podložne čelične ploče, a utezanje ankera ostvaruje se maticama.



Slika 5 – Detalj izvođenja armiranobetonskih sanduka

Pošto se završi izvođenje kosina i njihova zaštita torkret betonom i geotehničkim ankerima, sa formiranog radnog platoa može se pristupiti izvođenju projektovanih armiranobetonskih sanduka. Sanduci se izvode tako što se sukcesivno vrši iskop dubine od 1.00 do 2.00m i betoniranje zaštitnog plašta. Iskop se izvodi delimično mašinski, a delimično ručno uz

korišćenje opreme za štemovanje i iskop stenske mase. Na nekim delovima, gde je čvršća stena potrebno je da se izvede mikrominiranje, kako bi se pre iskopa stena usitnila. Kada se završi iskop za jednu lamelu, vrši se montaža armature plašta, montaža oplata i betoniranje plašta. Kada se završi izrada plašta u iskopani sanduk montira se glavna armatura. Betoniranje sanduka se izvodi kontraktorski odozdo na gore, s tim što se na svakih nekoliko metara novoizvedenog betona, vrši nabijanje betona pervibratorima.

Na gornjem delu sanduka ugrađuju se čelične cevi Ø100mm kroz koje se vrši bušenje ankeri koji pridržavaju sanduke. Po završetku bušenja i ugradnje ankeri vrši se injektiranje u tlu oko ankeri i unutar izvedenih cevi. Na vrhu sanduka ugrađuje se armatura koja služi za povezivanje izvedenih sanduka i armiranobetonske ploče iznad njih. Sa prednje strane iznad ploče izvodi se zaštitni armiranobetonski zid. Na ovaj način završeno je izvođenje sanduka. Pošto se pri iskopu za sanduk i betoniranju plašta delimično remeti naponsko stanje u tlu, iskop za sanduke mora se izvoditi uz permanentno praćenje ponašanja padine.

4 ZAKLJUČAK

U radu je prikazan proračun zaštitne konstrukcije padine na deonici osnovne trase autoputa E75 tunel Predejane-Caričina dolina. Zaštitnu konstrukciju čine armiranobetonski sanduci koji su sa donje strane uklješteni u stenu, a sa gornje strane pridržani geotehničkim ankerima koji se takođe sidre u stenski masiv iza potencijalnih kliznih ravni. Primenom računskog programa PLAXIS 2D i odgovarajućih konstitutivnih modela dobijeni su statički uticaji na osnovu kojih je izvršeno dimenzionisanje konstruktivnih elemenata zaštitne konstrukcije. Predloženo rešenje zaštitne konstrukcije padine ima niz prednosti u odnosu na druga rešenja: nije potrebna teška mehanizacija za izvođene radova, ostvaruje se znatna ušteta u vremenu građenja, značajna ekonomska opravdanost. Kako je reč o složenim izvođačkim radovima, u radu je prikazan kratak opis faza izvođenja ovih radova.

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Dragan Lukić¹, Stanko Brčić², Slobodan Maksimović³

RACIONALIZACIJA FUNDIRANJA OBJEKTA CVIJIĆEVA BR. 42 U BEOGRADU

Rezime:

Zgrada u ulici Cvijićeve 42 u Beogradu po prvobitnom tehničkom rešenju predviđena je da se fundira na šipovima. Racionalizacijom tehničkog rešenja objekat je fundiran na armirano betonskoj ploči uz neophodnu zamenu materijala ispod ploče. U radu se prikazuju prvobitno tehničko rešenje fundiranja na osnovu koga je dobijena građevinska dozvola i izmenjeno tehničko rešenje na osnovu koga je izgrađen objekat. Takođe, u radu se prikazuju i rezultati sleganja objekta.

Ključne reči: šipovi, ploča, sleganje, zamena materijala, racionalizacija

RATIONALIZATION OF FOUNDATION OF THE OBJECT CVIJIĆEVA No. 42 IN BELGRADE

Summary:

Due to the initial technical solution, the building in Civjiceva 42 in Belgrade was founded on piles. By the rationalization of that solution, the building is founded on the RC slab, with necessary replacement of the soil below the slab. The paper briefly presents the initial technical solution, upon which the building permit is obtained, and the corrected solution, upon which the building is built. Also, the paper presents the results of the settlement of the object.

Key words: piles, plate, settlement, substitute soil, rationalization

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1 UVOD

Investitor objekta u ulici Cvijićeva br. 42 pribavio je građevinsku dozvolu na osnovu urađene kompletne tehničke dokumentacije. Sastavni deo navedene dokumentacije je Projekat obezbeđenja temeljne jame koji pored osiguranja susednih objekata sadrži i način fundiranja objekta. Investitor je lokaciju kupio od prethodnog investitora koji je za ovu lokaciju predvideo izgradnju stambeno-poslovnog objekta sa podzemnom etažom. U okviru pripremnih radova prethodni investitor je uradio obodne šipove $\phi 400$ mm kao zaštitu postojećih susednih objekata. Novi investitor je predvideo izgradnju poslovnog objekta P+4 bez podzemne etaže, a uzimajući u obzir izgrađene obodne šipove. Temeljenje novog objekta je predviđeno na šipovima $\phi 600$ mm (27 šipova dužine 8 m i 20 šipova dužine 9 m). Za ovakvo rešenje temeljenja objekta projektant se opredelio na osnovu urađenog Elaborata o geotehničkim uslovima fundiranja poslovnog objekta – Doma zdravlja u ulici Cvijićeva br. 42. Investitor je zatražio mišljenje i ocenu ovakvog rešenja. Analizirajući podatke date u Elaboratu o geotehničkim uslovima i karakteristike terena utvrđene na osnovu izgradnje obodnih šipova, zaključeno je da je rešenje fundiranja objekta predimenzionisano i neracionalno. Naime, moglo se zaključiti da navedeni parametri u Elaboratu nisu u skladu sa konstatovanim stanjem pri izradi obodnih šipova pa je predložena racionalizacija i izrada novog rešenja fundiranja objekta na armiranobetonskoj ploči. Investitor je prihvatio izradu novog tehničkog rešenja temeljenja uz neophodnu izmenu konstrukcije ne menjajući arhitektonski deo iz odobrenog projekta od strane nadležnog organa.

2 TEHNIČKO REŠENJE PO ODOBRENOM PROJEKTU

U cilju izrade projekta i pribavljanja građevinske dozvole i zakonske obaveze urađen je Elaborat o geotehničkim uslovima fundiranja poslovnog objekta – Doma zdravlja u ulici Cvijićeva br.42 u Beogradu. U elaboratu su dati sledeći geotehnički parametri:

1. NASUTO TLO

$$\varphi=23^{\circ} \quad c=0 \text{ kN/m}^2 \quad \gamma=19,00 \text{ kN/m}^3$$

2. HUMUS

$$\varphi=23^{\circ} \quad c=0 \text{ kN/m}^2 \quad \gamma=19,00 \text{ kN/m}^3$$

3. GLINOVITA PRAŠINA

$$\varphi=19^{\circ} \quad c=19 \text{ kN/m}^2 \quad \gamma=20,00 \text{ kN/m}^3 \quad M_s=5400 \text{ kN/m}^2$$

4. PRAŠINASTA GLINA

$$\varphi=22^{\circ} \quad c=20 \text{ kN/m}^2 \quad \gamma=20,00 \text{ kN/m}^3 \quad M_s=7400 \text{ kN/m}^2$$

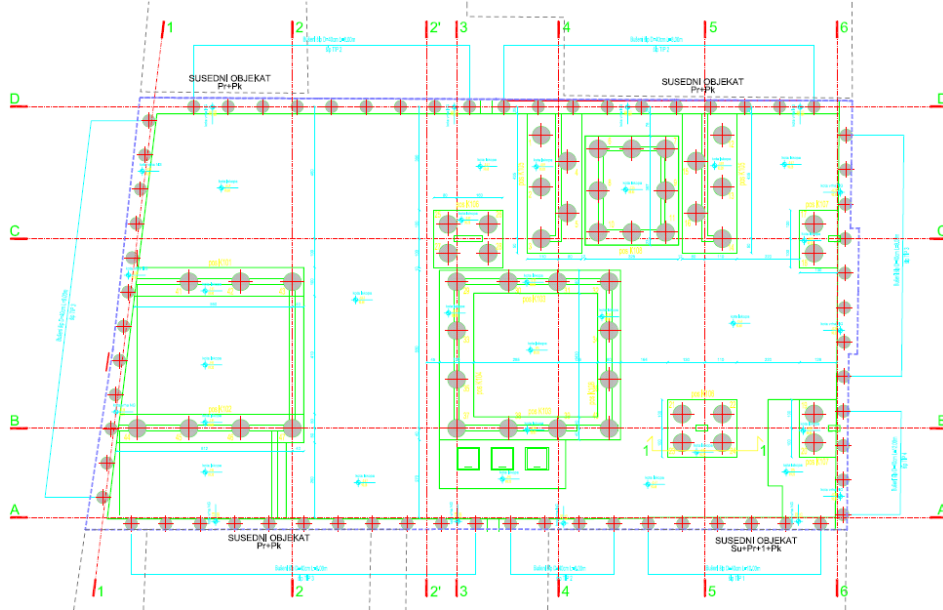
5. LAPOROVITA GLINA

$$\varphi=20^{\circ} \quad c=30 \text{ kN/m}^2 \quad \gamma=20,00 \text{ kN/m}^3 \quad M_s=12000 \text{ kN/m}^2$$

Na osnovu ovako dobijenih parametara projektant konstrukcije je dao tehničko rešenje fundiranja objekta u tehničkom izveštaju konstatovao sledeće: "Prema projektu pripremnih radova izvedeni su šipovi $\phi 400$ mm kao deo zaštitne konstrukcije. Pošto je promenjena namena objekta i nisu potrebne podzemne etaže, prilikom novog statičkog proračuna konstrukcije objekta utvrđeno je da će se javiti diferencijalno sleganje objekta jer se kalkanski i prednje

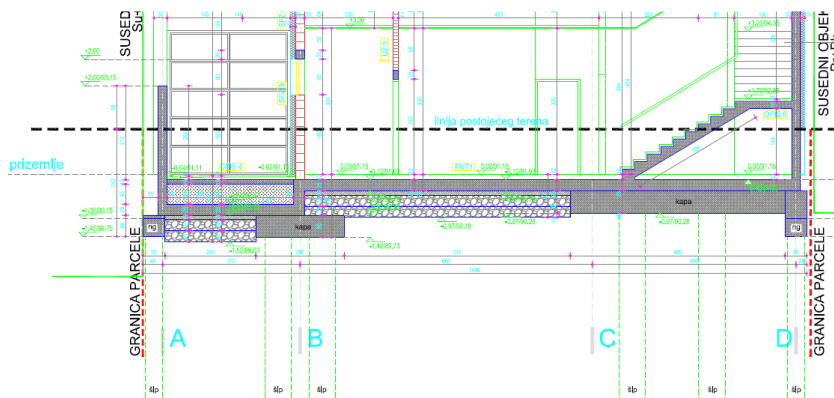
fasadni zidovi naslanjaju na naglavne grede tj. šipove. Na tim mestima je sleganje objekta zanemarljivo. Kako se objekat fundira u slojevima tla koji nemaju veliki modul elastičnosti a samim i modul stišljivosti na elementima u sredini objekta će se javiti velika sleganja. Zbog svega ovoga je zadat projektni zadatak da se ceo objekat fundira na bušenim šipovima.

Za bušene šipove Ø600mm se odlučilo zbog male nosivosti šipova Ø400mm, a na osnovu dobijenih podataka iz statičkog proračuna konstrukcije. Bušeni šipovi su dužine od 8,00m i 9,00m i baza im je u sloju laporovite gline." Na slici broj 1 prikazana je dispozicija projektovanih šipova /prema odobrenom projektu/.



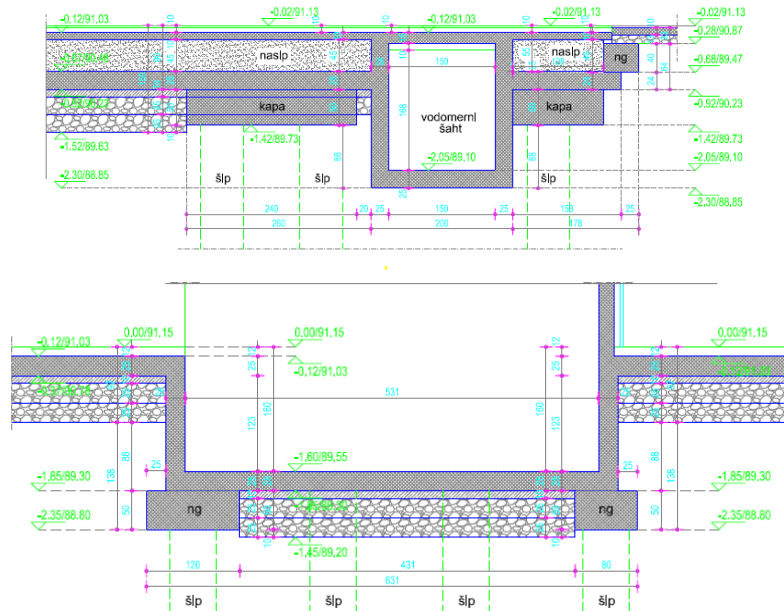
Slika 1 – Dispozicija šipova [1]

Na slici broj 2 prikazan je presek kroz objekat prema odobrenom projektu. Prikazan je deo koji se odnosi na temeljenje.



Slika 2 – Presek kroz objekat u zoni temelja [1]

Na slici broj 3 prikazani su detalji temeljne konstrukcije.

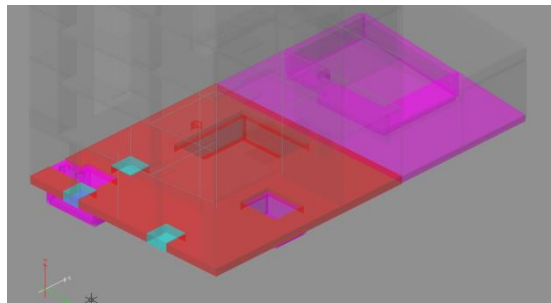


Slika 3 – Detalji temeljne konstrukcije sa šipovima i naglavnim gredama [1]

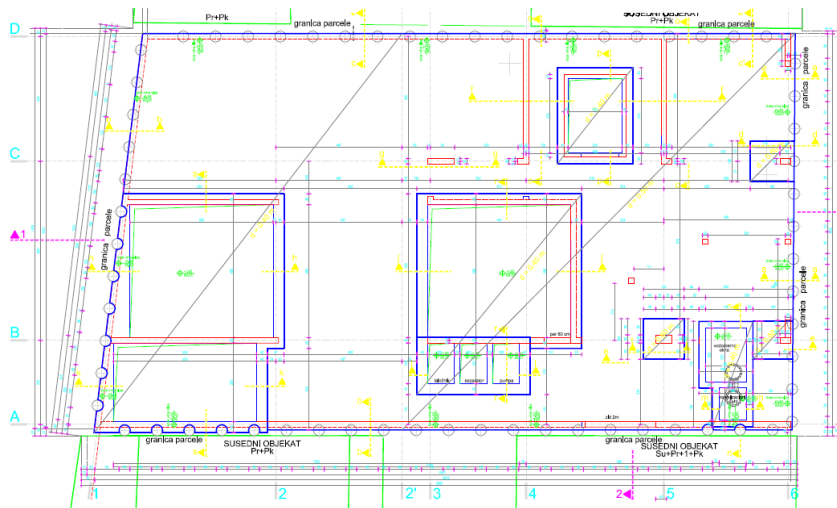
U projektovanom rešenju temeljenja nije data tehnologija izvođenja radova što predstavlja nedostatak imajući u vidu potrebe osiguranja dubljih delova iskopa. Posebno je bilo neophodno dati redosled izvođenja radova.

3 TEHNIČKO REŠENJE PO IZMENJENOM PROJEKTU

U projektu temeljenja predviđena je zamena materijala i ugradnja tampon sloja debljine 2 x 25 cm sa valjanjem. Objekat se temelji na armiranobetonskoj ploči debljine 50 cm ispod višeg dela objekta P+4 i 40 cm ispod dvorišnog dela objekta. Osnova temeljne ploče i model u računarskom programu prikazani su na grafičkim detaljima (slike 4 i 5).

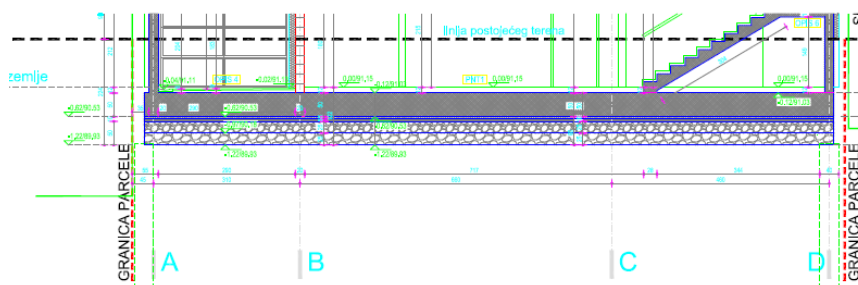


Slika 4 – Model temeljne konstrukcije [2]

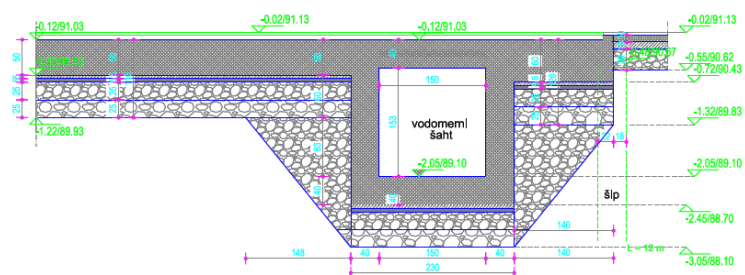


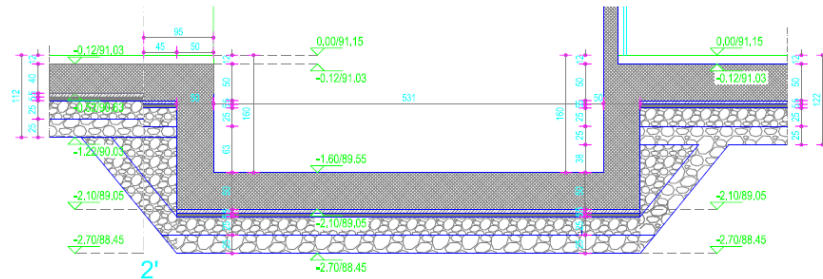
Slika 5 – Osnova temeljne konstrukcije [2]

Novo rešenje temeljenja uslovalo je određene izmene na konstrukciji objekta što je prikazano na grafičkim priložima (slike 6 i 7).



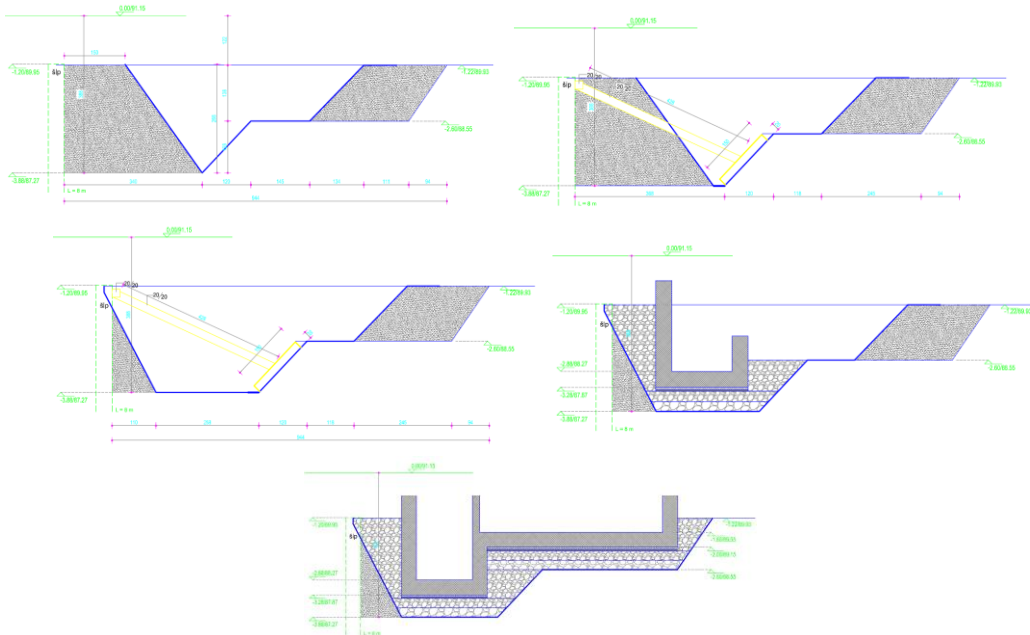
Slika 6 – Presek kroz objekat u zoni temelja [2]





Slika 7 – Detalji temeljne konstrukcije [2]

Pored tehničkog rešenja i statičkog proračuna data je detaljna tehnologija izvođenja radova što je delimično prikazano na grafičkim priložima (slika 8). Pored toga u tehničkom izveštaju dat je redosled izvođenja radova:



Slika 8 – Tehnologija izvođenja radova [2]

"Prva faza iskopa je široki iskop do kote 89.93 na delu objekta (od ose 6 do ose 2') i do kote 90.03 na delu aneksa (od ose 2' do ose 1). Pri iskopu do objekata ostaviti 0.5 m osim na delu rampe gde do objekta ostaviti 1.0 m. Ova mera obezbeđenja je u cilju sprečavanja eventualnog raskvašavanja tla ispod temelja objekata prema iskopu. Ovaj deo iskopa uradiće se sukcesivno sa nasipanjem tampon sloja.

Druga faza iskopa je iskop dubljeg dela za automobile u aneksu do kote prema detalju, sa razupiranjem drvenim gredama 20/20 cm (ili čeličnim nosačima). Razupirače razupreti u betonsku ploču (donja ploča).

Paralelno na delu aneksa uz osu D može se uraditi iskop do objekata i nasipanje sa nabijanjem (bez vibracija). Iskop vršiti pažljivo i uz kontrolu.

Treća faza iskopa je iskop dubljeg dela za automobile u samom objektu. Ovde se ne očekuje razupiranje. Sve radove raditi prema detaljima iz projekta.

Paralelno sa navedenim radovima može se završiti iskop do objekata aneksnog dela, uraditi tampon sloj sa nabijanjem (bez vibracija), uraditi nabijeni beton i armirati donja ploča . Posebno se naglašava da treba sačekati da zidovi dubljih delova postignu određenu čvrstoću i onda vršiti sabijanje pored njih.

Četvrta faza je iskop za vodovodni i kanalizacioni šaht. Ovde su dva šipa ugrožena i njihovo osiguranje izvršiće se jednim nosačem koji je vezan za po dva šipa levo i desno, a po potrebi i razupreti. Ovo osiguranje će se rešiti na licu mesta kada se izvrši iskop do šipova i u zavisnosti od nađenog stanja.

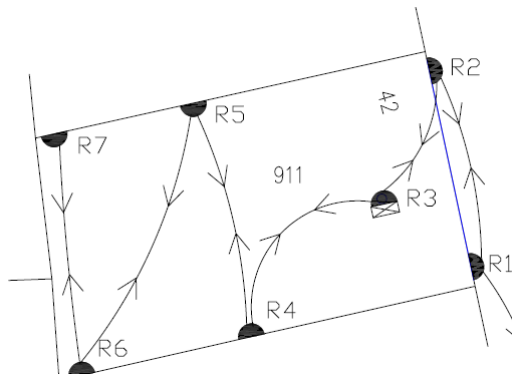
Peta faza je iskop šahta za lift. Ovde se ne očekuje razupiranje. Iskop uraditi prema detalju.

Šesta faza iskopa je iskop za kapitele donje ploče dubine 10 cm.

Posle iskopa u urađenih dubljih delova armirati i betonirati donju ploču. Deo donje ploče u aneksu može se uraditi i paralelno sa radovima na delu objekta."

4 PRAĆENJE SLEGANJA OBJEKTA

U cilju verifikacije predloženog rešenja u toku izgradnje praćena su sleganja objekta. Pre početka izgradnje objekta urađeno je nulto snimanje. U toku izgradnje konstrukcije i po završetku izvršena su kontrolna snimanja. Na slici 9 prikazan je položaj repera za praćenje sleganja objekta.



Slika 9 – Položaj repera za praćenje sleganja

U tabeli 1 prikazani su rezultati osmatranja sleganja objekta. Na osnovu dobijenih merenja sleganja objekta može se zaključiti da je racionalisano novo rešenje fundiranja objekta u potpunosti zadovoljavajuće ne samo sa aspekta ekonomičnosti, jednostavnijeg izvođenja

radova već i sa aspekta sleganja. Rezultati su u potpunosti demantovali zaključke iz Elaborata o geotehničkim uslovima fundiranja objekta u pogledu sleganja.

Tabela 1 – Rezultati praćenja sleganja objekta

Datum: 14.09.2015.		Datum: 25.09.2015.		Datum: 21.10.2015.		
Broj repera	Nulto merenje	1.Merenje	H1-Ho Δh [mm]	2.Merenje	H2-H1 $\Delta h'$ [mm]	H2-Ho Δh [mm]
	Ho	H1		H2		
R1	91.6468	91.6468	0.0	91.6454	-1.4	-1.4
R2	91.3989	91.3987	-0.2	91.3976	-1.1	-1.3
R3	91.4457	91.4453	-0.4	91.4436	-1.7	-2.1
R4	91.3904	91.3902	-0.2	91.3899	-0.3	-0.5
R5	91.3339	91.3337	-0.2	91.3326	-1.1	-1.3
R6	91.4178	91.4177	-0.1	91.4176	-0.1	-0.2
R7	91.3947	91.3945	-0.2	91.3944	-0.1	-0.3
Opažao:S. Milošević		Opažao:S. Milošević		Opažao:S. Milošević		

Kota stalnog repera:		
RP1	91.4790	Kontrolni reper
RP2	91.3248	Kontrolni reper
RP2	91.6164	Kontrolni reper

5 ZAKLJUČAK

Izmenom tehničkog rešenja temeljenja objekta izvršena je značajna ušteda i rešenje je jednostavnije za izvođenje. Data je kompletna tehnologija izvođenja radova i njihov redosled. Posebno je potrebno naglasiti da su sleganja objekta zanemarljiva.

Na osnovu ovog rešenja može se izvesti zaključak da pri rešavanju temeljenja posebnu pažnju treba posvetiti na realnost dobijenih parametara dobijenih istraživanjem, a posebno na njihovu interpretaciju i na osnovu njih preporuke.

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Lidja Kurešević¹, Ivana Delić-Nikolić², Olivera Vušović³

LINEARNO TERMIČKO ŠIRENJE ARHITEKTONSKOG KAMENA VULKANSKOG POREKLA IZ VARDARSKE ZONE

Rezime:

S-91

Linearno termičko širenje prirodnog kamena ima veliki značaj pri izvođenju građevinskih i arhitektonskih radova, jer vertikalne i horizontalne površine na kojima nije predviđen prostor za termičko širenje kamena mogu biti narušene do potpuno uništene usled mehaničkih deformacija do kojih dolazi zbog nastajanja napona pri promeni ambijentalne temperature, na koju prirodni kamen reaguje dilatacijom i kontrakcijom. Ispitivanje linearnog termičkog širenja kamena poreklom iz pet vulkanitskih kompleksa Vardarske zone Srbije pokazalo je da su njihove vrednosti niže u poređenju sa vrednostima linearnog termičkog širenja poznatih varijeteta granita iz sveta, ali u nivou vrednosti linearnog termičkog širenja granitoidnih stena poreklom iz iste geotektonske jedinice.

Кljučне речи: linearno termičko širenje, dilatacija, vulkaniti

LINEAR THERMAL EXPANSION OF DIMENSION STONE OF VOLCANIC ORIGIN FROM THE VARDAR ZONE OF SERBIA

Summary:

Linear thermal expansion is of great importance for architectural works which include natural stone pieces because the surfaces covered with them can be disturbed or completely ruined due to mechanical deformation caused by strain produced during the change of the environmental temperature. Testing of the linear thermal expansion of the stone originating from the five volcanic massifs of the Vardar zone of Serbia has shown that their values are lower than the same values of the well-known granites from other countries, but are quite similar to the values of the linear thermal expansion of the stone originating from plutonic masses of the same geotectonic unit.

Key words: linear thermal expansion, dilatation, volcanic rocks

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1 UVOD

Iznos linearnog širenja arhitektonskog građevinskog kamena pri porastu ambijentalne temperature ima veliki značaj pri oblaganju horizontalnih, a naročito vertikalnih površina građevinskih objekata. Ako temperaturne promene dovode do dilatacije koja premašuje prostor koji je pri ugradnji kamena za ovu namenu ostavljen, dolazi do porasta napona i deformacije kamenih ploča, i, u krajnjem slučaju – pucanja ploča, što naružuje estetski izgled površine objekta na kojem su ugrađene. Takođe, vrednosti linearnog termičkog širenja za prirodni kamen, razlikuju se od vrednosti za veziva koja se koriste za postavljanje i fiksiranje kamenih elemenata pri ugradnji. Koeficijent linearnog toplotnog širenja cementnih veziva je veći od koeficijenta prirodnog kamena (Tabela 1), što dovodi do stvaranja napona na graničnim površinama između kamenih elemenata i veziva, do njihovog odvajanja, i ispadanja kamenih ploča.

Tabela 1. Vrednosti koeficijenta linearnog termičkog širenja različitih vrsta stena i maltera (iz: [1], osim*).

R. br.	Vrsta stene	Koeficijent linearnog termičkog širenja [$\cdot 10^{-6} 1/^{\circ}\text{C}$]
1.	Granit	3.7-6.0
2.	Gabro, diorit, dijabaz	2.0-3.0
3.	Bazalt	2.2-3.5
4.	Peridotit	4.5
5.	Krečnjak	1.7-6.8
6.	Kvarcit	6.0
7.	Mermer	2.7-5.1
8.	Malter	7.3-13.5*[2]

Ovakva situacija se može videti na više mesta u Beogradu (palata Albanija – pre restauracije, ali i danas; plato oko Vukovog spomenika; plato oko spomenika borcima palim u NOR na Avijatičarskom trgu u Zemunu).

Linearnog termičko širenje složenih, nehomogenih materijala, kao što je stena (kamen), ne zavisi samo od koeficijenta dilatacije minerala od kojih je izgrađena i promene ambijentalne temperature, već i od sklopa stene, orijentacije mineralnih zrna, njihovih dimenzija, prisustva pora, šupljina i drugih diskontinuiteta (pukotina, prslina, stilolita, žica), njihove ispunjenosti i tipa ispune. Usled prirodne anizotropnosti, mineralna zrna se šire u različitim iznosima duž različitih kristalografskih pravaca. S obzirom da kvarc ima najveću toplotnu provodljivost od svih petrogenih minerala, njegov udeo u građi stene značajno utiče na toplotnu dilataciju. Što je veći udeo kvarca u steni, veći je i koeficijent njenog linearnog termičkog širenja [1,3].

U sklopu ispitivanja upotrebljivosti kamena poreklom iz magmatskih kompleksa Vardarske zone Srbije [4], pored kamena poreklom iz plutonskih kompleksa, koji je najčešće korišćen kao arhitektonski građevinski kamen, ispitivanjem su obuhvaćeni i vulkanitski kompleksi.

Kamen poreklom iz vulkanitskih masiva, češće nego kamen poreklom iz plutonskih, može imati svojstva koja ograničavaju njegovu primenu za svrhu oblaganja građevinskih objekata: značajnije prisustvo pora i šupljina, zone oksidacije i hidrotermalnih alteracija, intenzivnija ispucalost, nepovoljno lučenje, male dimenzije prirodno lučenih blokova, promenljivost sklopa

i boje (izgleda) stene čak i na malim rastojanjima itd. Sve ove karakteristike, negativne sa aspekta primene kamena u arhitektonsko-građevinske svrhe, uzrokovane su genetskim procesom (generalno brzim hlađenjem lave na površini Zemlje), ali i toplotnim uticajem magme u nižim delovima terena, koja, hladeći se, otpušta toplotu u okolne stene i zagreva podzemne vode, koje zatim vrše alteraciju očvrslilih stena na površini terena.

Iz ovih razloga, vulkanske stene se ređe koriste kao arhitektonski građevinski kamen, pa se retko za ove namene i ispituju njihova tehnička svojstva. Tokom prethodnih ispitivanja [4], utvrđeno je da postoje značajne razlike npr. u vrednostima pritiska i savojne čvrstoće između ispitanih plutonskih i vulkanskih stena, koje su daleko povoljnije kod plutonskih, osim u slučajevima kada su vulkaniti silifikovani. Međutim, kada su u pitanju vrednosti linearnog termičkog širenja kamena, ovo nije pravilo. Zbog retkosti ove vrste podataka, uzorci odabranih vrsta kamena su uzorkovani i ispitani sa svrhom utvrđivanja vrednosti linearnog termičkog širenja i upoređivanja sa referentnim vrednostima za druge tipove kamena, koji potiču od genetski različitih tipova stena.

2 METODA ISPITIVANJA

Od svakog uzorka odabranog kamena isecaju se opitna tela (u našem slučaju tri) pravilnog oblika, čije su dimenzije u skladu sa uređajem za merenje dilatacije, ali dužine ne manje od 50 mm. Poprečni presek opitnih tela je kvadratnog oblika, ivice dužine ne veće od 6 mm. U konkretnom slučaju sprovedenih ispitivanja, dimenzije prizmatičnih opitnih tela su (što bliže): 50mm x 5mm x 5mm. Ispitivanja se vrše na temperaturi do 100°C, jer pod normalnim uslovima ambijentalnih temperatura ugrađeni kamen neće biti izložen višim temperaturama. Osim toga, iznad ove temperature, u zagrevanom uzorku usled ekspanzije može doći do nastanka trajnih promena – nastanka mikroprslina, faznih transformacija, hemijskih reakcija (npr. oksidacije), ili narušavanja kristalne rešetke [5]. Dužina potpuno osušenih uzoraka se meri pomičnim merilom ili dilatometrom na sobnoj temperaturi, i obe vrednosti (dužina opitnog tela i sobna temperatura) se beleže. Dužina opitnog tela na sobnoj temperaturi se beleži kao vrednost L_0 , a sobna temperatura na kojoj je dužina merena – kao T_0 . Nakon zagrevanja do temperature od 100°C (postupak u svemu usklađen sa međunarodnim standardom ISO 10545-8), meri se dužina dilatiranih opitnih tela, i beleži kao L_{100} . Iznos linearnog širenja (ΔL) usled zagrevanja iznosi:

$$\Delta L = L_{100} - L_0 \quad (1)$$

Radi praktičnosti primene ovog rezultata, vrednost ΔL izražena u milimetrima, deli se sa prvobitnom dužinom opitnog tela L_0 izraženom u metrima. Na ovaj način, dobija se jedinična vrednost linearnog termičkog širenja po dužnom metru ispitivanog kamena, izražena u milimetrima ($\Delta L'$).

Izduženje tela jedinične dužine pri povišenju temperature za jedan stepen, ili koeficijent linearnog termičkog širenja (α) izračunava se prema formuli:

$$\alpha = \Delta L/L_0 \cdot (100^\circ\text{C} - T_0) = \Delta L/L_0 \cdot \Delta T \quad (2)$$

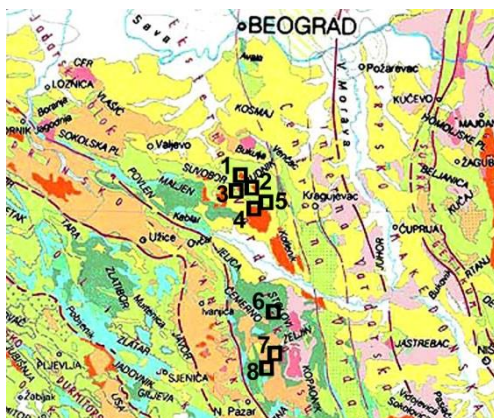
3 REZULTATI ISPITIVANJA

Ispitani su uzorci kamena uzeti iz pet vulkanitskih masiva, sa osam lokaliteta (Slika 1).

Tabela 2. Vrednosti linearnog termičkog širenja i koeficijenta linearnog termičkog širenja uzoraka iz vulkanitskih kompleksa Eksterne podzone Vardarske zone Srbije.

Redni broj	Vulkanitski masiv Lokalitet	Petrološki tip stene	Termička dilatacija $\Delta L'$ [mm/m]	Koef. termičke dilatacije α [$\cdot 10^{-6}$ 1/°C]
1.	Rudnik Gradska Gora	andezit sa prelazima u dacidit	0.439 0.537 0.600	5.63 6.97 7.59
2.	Rudnik Ćeramide	dacidit	0.630 0.633 0.647	7.87 8.19 8.23
3.	Slavkovic Kadina Luka	biotitski dacidit	0.815 0.818 0.829	10.4 10.7 10.9
4.	Borač-Ješevac Ravno Brdo	dacidit	0.419 0.421 0.472	5.18 5.32 5.98
5.	Borač-Ješevac Talambas	trahandezit do trahibazalt	0.444 0.512 0.608	5.70 6.56 7.89
6.	Stolovi Kamenica	hornblenda andezit	0.536 0.583 0.602	7.15 7.38 7.52
7.	Ibarski vulkaniti Brvenik-Šumnik	propilitisani pirokzensko-amfibolski andezit	0.391 0.492 0.530	4.89 6.15 7.09
8.	Ibarski vulkaniti Bisina	augit-biotit andezit	0.496 0.511 0.526	6.12 6.56 6.58

U Tabeli 2, prikazani su rezultati ispitivanja linearnog termičkog širenja i koeficijenta linearnog termičkog širenja vulkanitskih masiva lociranih u Eksternoj podzoni Vardarske zone Srbije, zajedno sa lokalitetima sa kojih potiču uzorci, kao i petrološki tipovi stena iz kojih su uzorci kamena uzeti. Radi poređenja, u Tabeli 3, date su i neke referentne vrednosti – za granitoide, krečnjake i mermere.



Slika 1 – Lokalizacije na kojima su uzeti uzorci za ispitivanje (1. Gradska Gora, 2. Čeramide, 3. Slavkovića, 4. Ravno brdo, 5. Talambas, 6. Kamenica, 7. Brvenik-Šumnik, 8. Bisina)

Tabela 3. Vrednosti linearnog termičkog širenja i koeficijenta linearnog termičkog širenja odabranih vrsta stena iz sveta i Srbije.

Redni broj	Zemlja, lokalitet Komerrijalni naziv	Petrološki tip stene	Termička dilatacija $\Delta L'$ [mm/m]	Koef. termičke dilatacije α [$\cdot 10^{-6} 1/^{\circ}C$]
1.	Brazil "Royal red"	granit	1. 0.575 2. 0.682 3. 0.708	1. 7.77 2. 8.97 3. 9.45
2.	Italija "Bianco Sardo"	granit	1. 0.637 2. 0.788 3. 0.915	1. 8.17 2. 9.85 3. 11.6
3.	Srbija, Bukulja "Bukovik"	granit	1. 0.588 2. 0.596 3. 0.604	1. 7.55 2. 7.73 3. 7.74
4.	Srbija, Kremiči	granodiorit	1. 0.466 2. 0.486 3. 0.544	1. 5.76 2. 6.00 3. 6.88
5.	Crna Gora, Čanj "Bokit"	krečnjak	1. 0.280 2. 0.332 3. 0.335	1. 3.73 2. 4.47 3. 4.55
6.	Srbija, Sirogojno "Zidine"	krečnjak	1. 0.367 2. 0.368 3. 0.464	1. 4.96 2. 4.97 3. 6.27
7.	Srbija, Požega "Plavi tok"	mermer	1. 0.790 2. 0.850 3. 0.910	1. 10.90 2. 11.60 3. 11.80

4 DISKUSIJA

Vrednosti linearnog termičkog širenja kamena poreklom iz vulkanitskih masiva Vardarske zone Srbije pokazuju variranje u opsegu od 0.391 mm/m (Ibarski vulkaniti, lokalitet Brvenik-Šumnik) do 0.829 mm/m (slavkovički vulkaniti, lokalitet Kadina Luka).

Što je vrednost linearnog termičkog širenja kamena veća, veći je i njegov koeficijent linearnog termičkog širenja. Ove vrednosti variraju od $4.89 \cdot 10^{-6}$ 1/°C (Ibarski vulkaniti, lokalitet Brvenik-Šumnik) do $10.9 \cdot 10^{-6}$ 1/°C (slavkovički vulkaniti, lokalitet Kadina Luka).

Najveće rasipanje vrednosti pokazuju uzorci sa lokaliteta Brvenik-Šumnik, što je prirodno s obzirom na izrazitu heterogenost primarnog stenskog materijala uzorkovanog u kamenolomu. Stene na lokalitetu Brvenik-Šumnik su intenzivno silifikovane, usled čega imaju i veoma visoke vrednosti drugih tehničkih svojstava – pritisne i savojne čvrstoće – vrednosti koje premašuju iznose karakteristične za plutonske stene. Za razliku od ovog lokaliteta, stene čitavog slavkovičkog masiva, pretrpele su izvestan stepen hidrotermalne alteracije, usled koje se u steni mogu naći i sekundarni sulfidni minerali (kao pirit), hlorit, kalcit i minerali glina (svi imaju veći koeficijent linearnog termičkog širenja nego petrogeni minerali, uključujući i kvarc). Drugo svojstvo karakteristično za slavkovički masiv je da stene nisu tipični vulkaniti, već subvulkanske stene. Sadrže agregacije zrna kvarca, ponekad dužine i po nekoliko centimetara. Uticaj krupnoće zrna [6] i značajno većeg iznosa koeficijenta linearnog termičkog širenja kvarca, doprinose većem iznosu linearnog termičkog širenja slavkovičkog dacita, kao i prisustvo mikropukotina, uzrokovanih načinom eksploatacije (koja se jedno vreme vršila masovnim miniranjem).

Zatim, upadljivo je da, iako su svi ispitani vulkanitski masivi bili izloženi promenljivoj stepenu i tipu hidrotermalnih alteracija, ova pojava se reflektuje na rasipanje vrednosti rezultata kod trahandezita-trahibazalta Talambasa, andezita lokaliteta Brvenik-Šumnik, andezita Gradske Gore, dok kod andezita lokaliteta Kamenica i Bisina i dacita Čeramida, Ravnog Brda i lokaliteta Kadina Luka – rasipanje rezultata nije toliko izraženo (Slika 2).

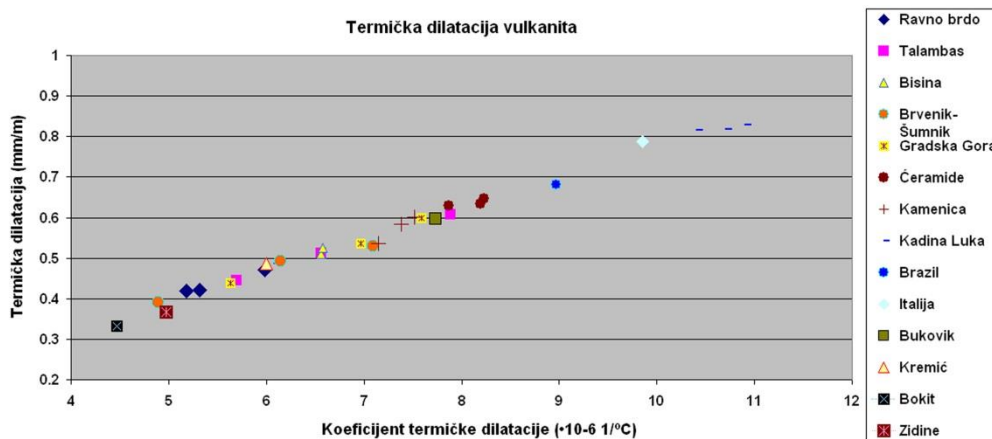
Kada se uporede dobijene vrednosti ispitanih uzoraka kamena iz vulkanitskih masiva sa referentnim vrednostima iz Tabele 3, može se konstatovati da su vrednosti linearnog termičkog širenja vulkanita niže nego iste vrednosti za plutonske magmatske stene iz Brazila i Italije i mermer, a više nego iste vrednosti za krečnjake. Međutim, uočljiv je sličan opseg vrednosti za linearno termičko širenje kamena poreklom iz vulkanitskih i plutonskih kompleksa Srbije (Tabela 3, redovi 3 i 4). Ovi uzorci potiču iz iste geotektonske jedinice – Eksterne podzone Vardarske zone Srbije, i pripadaju istoj fazi magmatske aktivnosti – predstavljaju dve facije formacije kasnopaleogeno-neogene starosti.

Budući da među petrogenim mineralima, kvarc ima najveći koeficijent linearnog termičkog širenja, logično je očekivati da stene sa najvećim udelom kvarca u svom sastavu, takođe imaju viši koeficijent linearnog termičkog širenja u odnosu na stene sa nižim udelom kvarca. Ovo podrazumeva da bi, generalno, daciti trebalo da imaju veći koeficijent linearnog termičkog širenja nego andeziti. U našim ispitivanjima, dobijeni podaci delimično potvrđuju ovu postavku. Linearno termičko širenje ispitanih dacita varira od 0.419 do 0.829 mm/m, odnosno do 0.647 mm/m kada zanemarimo lokalitet Kadina Luka. Linearno termičko širenje ispitanih andezita (koji sadrže manji udeo kvarca) varira od 0.391 (odnosno, od 0.439 mm/m, kada izuzmemo propilitisane andezite lokaliteta Brvenik-Šumnik) do 0.602 mm/m. Vrednosti koeficijenta linearnog termičkog širenja za trahandezit do trahibazalt Talambasa, koji

generalno sadrži još manje kvarca nego andezit, variraju od 0.444 do 0.608 mm/m, što ne potvrđuje gore navedenu pretpostavku. Razlog za odstupanje je intenzivna silifikacija stene na ovom lokalitetu.

5 ZAKLJUČAK

Promenljiv stepen variranja vrednosti linearnog termičkog širenja različitih tipova stena sa ispitanih lokaliteta u vulkanitskim masivima Eksterne Vardarske zone Srbije, pokazao je da ove vrednosti nisu vezane isključivo za genetske faktore, i da su vrednosti linearnog termičkog širenja za vulkanite i plutonite iz iste geotektonske jedinice i iste facije magmatizma vrlo bliske. Vulkanske stene se karakterišu izraženom nehomogenošću i promenljivošću sklopa, stepena i tipa hidrotermalnih alteracija, čak i na relativno malim rastojanjima na površini terena, i teško je odabrati homogene uzorke ujednačenih svojstava na terenu; zatim, u laboratorijskim uslovima, nehomogenost uzoraka još više dolazi do izražaja, jer se od količine uzorka koja je na raspolaganju, odabiraju najkvalitetniji komadi za izradu opitnih tela – što se u slučaju različitih tipova stena sa različitih lokaliteta veoma razlikuje.



Slika 2 – Dijagram rasipanja vrednosti linearnog termičkog širenja $\Delta L'$ i koeficijenta linearnog termičkog širenja α za vulkanite sa osam uzorkovanih lokaliteta i šest referentnih vrednosti za granitoide i krečnjake

Petrološki tip stene utiče na pojavu razlika u vrednostima linearnog termičkog širenja samo u pojedinim slučajevima. Vrednosti dobijene za dacite i andezite veoma su bliske, bez obzira na razliku u sadržaju kvarca. Pojava sekundarne hidrotermalne alteracije, naročito silifikacije, koja je daleko ređa kod plutonskih stena, menja primarni sadržaj kvarca u vulkanskoj steni, i time remeti očekivane šeme u promeni vrednosti linearnog termičkog širenja koje važe za nealterisane stene.

Značaj ovih ispitivanja je daleko veći za praktične namene, jer pruža uvid u veličinu razmaka između dekorativnih elemenata od prirodnog kamena, koji se mora ostaviti kako bi se ugrađenom kamenu omogućilo neometano širenje i skupljanje pod uticajem dnevnih i godišnjih oscilacija temperature, i sprečilo narušavanje celovitosti i estetskog izgleda kamenih

površina i povređivanje prolaznika usled obrušavanja kamenih fasada. Najviše vrednosti linearnog termičkog širenja utvrđene su kod uzoraka dacita iz slavkovičkog masiva, i upravo je kod površina obloženih ovim kamenom primećeno neočekivano intenzivno propadanje i osipanje kamenih elemenata pod uticajem atmosferilija na jednom lokalitetu u Beogradu [7].

S obzirom na razlike u vrednostima linearnog termičkog širenja kamenih materijala i cementnih veziva, neophodno je ostavljati odgovarajuće razmake kako bi se omogućilo slobodno širenje i skupljanje kamenih elemenata. Budući da će se cementno vezivo i širiti i skupljati više nego kameni elementi, treba uzeti u obzir i vremenski period tokom godine kada se vrši ugradnja.

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TRAJNOST BETONA IZLOŽENOG NA DEJSTVO VODE, ATMOSFERSKIH UTICAJA I HEMISKU KOROZIЈU

Rezime:

Trajnost betonskih i AB konstrukcija u uslovima izloženosti na atmosferfke uticaje, ciklusa smrzavanja i odmrzavanja, hlorida, karbonizacije ili druge hemiske agresije, direkto zavisi od nepropusnosti betona na prodor vode i hemiskih kontaminenata. Cilj ovog rada je da prezentira rezultate istraživanja otpornosti na prodor vode kod različitih tipova betona, sa različitom črstoćom i sa primenom različituh tipova aditiva za beton. Testiranje je izvršeno prema evropskim standardima, a testirana je propustljivost betona na prodor vode pod pritiskom, kako i prodor kailarne vlage.

Ključne reči: Beton, vodonepropusni beton, kapilarna apsorpcija, EN 206

DURABILITY OF CONCRETE EXPOSED TO WATER, ATMOSPHERIC INFLUENCES AND CHEMICALS

Summary:

Durability of concrete and steel reinforced concrete structures subjected to atmospheric influences, freeze-thaw cycles, chlorides, carbonation and other chemical aggression, largely depends on the concrete impermeability to water and other chemical contaminations. This text includes presentation of results from testing of resistance of water ingress for different types of concrete, with different strength characteristics, prepared with different types of admixtures. Testing is conducted according to European standards, and it includes testing of penetration of water under pressure and capillary absorption.

Key words: Concrete, waterproof concrete, capillary absorption, EN 206

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1 UVOD - KLASSE IZLOŽENOSTI BETONSKIH KONSTRUKCIJA U SAGLASNOSTI SA EVROPSKIM STANDARDOM EN 206

Različite betonske i AB konstrukcije tokom svoje eksploatacije mogu da budu izložene fizičko-mehaničkim uticajima, atmosferskim uticajima, hemiskoj agresiji itn. Evropski standard EN 206 Concrete — Part 1: Specification, performance, production and Conformity definiše Klase izloženosti betona na uticaje okoline.

Tabela 1 - Klase izloženosti betona

Obeležavanje klase	Opis okoline	Informativni primeri
1. Nema rizika od korozije i hemiske agresije		
X0	Nearmirani beton koji nije izložen smrzavanju, abraziji ili hemiskoj agresiji Armirani beton u veoma suvoj sredini	Beton unutar zgrade gde je vazduh veoma suv
2. Korozija izazvana karbonizacijom		
Armirani beton ili beton u kome je ugrađen metal, koji je izložen vazduhu i vlazi, izloženost može da se klasifikira kao što sledi: * Sadržaj vlage se odnosi na zaštitni sloj preko armature, koji uobičajeno odražava uslove okoline. Ovo je slučaj ukoliko ne postoji barijera koja odvaja beton od okoline		
XC1	Suva ili trajno vlažna	Beton unutar zgrade gde je vlažnost vazduha niska Beton stalno potopljen pod vodom
XC2	Vlažna, retko suva	Betonske površine izložene dugim kontaktom sa vodom Mnogo temeljnih konstrukcija
XC3	Umerena vlažnost	Vazduh u zgradama gde je vlažnost visoka Spoljašne betonske površine zaštićene od kiše
XC4	Ciklično vlažno i suvo	Konstrukcije izložene na kontakt sa vodom, koje nisu klasifikovane kao XC2
3. Korozija izazvana hloridima koji ne potiču od morske vode		
Armirani beton ili beton u kome je ugrađen metal je izložen solima za razmrzavanje ili drugim hloridima koji ne potiču od morske vode		
XD1	Umerena vlažnost	Betonske površine izložene hloridima u atmosferi
XD2	Vlažna, retko suva	Bazeni za plivanje Industrijske vode gde su sadržani hloridi
XD3	Ciklično vlažno i suvo	Mostovi, pločnici, parkirališta
Obeležavanje klase	Opis okoline	Informativni primeri
4. Korozija izazvana hloridima u morskoj vodi		
Armirani beton ili beton u kome je ugrađen metal, koji je izložen morskoj vodi ili hloridima u vazduhu koji potiču od morske vode, klasifikira se kao što sledi:		
XS1	Izložena soli iz vazduha ali ne i direktno u kontaktu sa morskom vodom	Objekti u blizini morske obale
XS2	Trajno potopljeni	Delovi pristanišnih konstrukcija
XS3	Konstrukcije u zoni plime, prskanja i talasa	Delovi pristanišnih konstrukcija

Obeležavanje klase	Opis okoline	Informativni primeri
5. Ciklusi smrzavanja sa i bez soli za razmrzavanje		
Beton izložen ciklusima smrzavanja i razmrzavanja dok je u vlažnom stanju, klasificira se kao što sledi:		
XF1	Umerena zasićenost vodom bez soli za razmrzavanje	Vertikalne betonske površine izložene kiši I smrzavanju
XF2	Umerena zasićenost vodom sa sredstvom za razmrzavanje	Vertikalne betonske površine na autoputnoj infrastrukturi izložene smrzavanju i prskanju sredstvom za odmrzavanje
XF3	Visoka zasićenost vodom bez soli za razmrzavanje	Horizontalne betonske površine izložene kiši I smrzavanju
XF4	Visoka zasićenost vodom sa sredstvom za razmrzavanje	Mostovske ploče I betonski kolovozi i druge konstrukcije izložene direktnom nanošenju sredstva za razmrzavanje I smrzavanju. Pristanišne konstrukcije izložene smrzavanju
6. Hemiska agresija		
Beton izložen hemijskoj agresiji od podzemne vode kako je navedeno u Tabeli 2 klasificira se kao što sledi: Klasifikacija morske vode zavisi od geografske lokacije. * Specijalna istraživanja su potrebna da se utvrdi relevantna klasa izloženosti u uslovima gde		
<ul style="list-style-type: none"> - Prisutne su druge agresivne hemikalije - Hemiski su kontaminirani tlo ili voda - granice prevazilaze vrednosti date u Tabeli 2 - Prisutno je više kombinacija hemikalija iz Tabele 2 		
XA1	Slabo hemiski agresivna sredina prema Tabeli 2	
XA2	Umereno hemiski agresivna sredina prema Tabeli 2	
XA3	Visoko hemiski agresivna sredina prema Tabeli 2	

Tabela 2 - Limitirajuće vrednosti klase izloženosti betona na hemisku agresiju od prirodng tla ili podzemne vode

Klasifikacija hemiski agresivnih sredina bazira se na agresivnost tla i vode na temperaturi od 5 do 25°C, gde voda teče dovoljno sporo da uslovi budu adekvatni statičnim uslovima. Najnepovoljnija hemiska karakteristika određuje klasu izloženosti. Kada dve različite agresije dovode do jednu istu klasu izloženosti, usvaja se sledeća klasa, osim ako se specijalno ne dokaze da to nije neophodno.				
Hemiska karakteristika	Referentna test metoda	XA1	XA2	XA3
Podzemna voda				
SO ₄ ²⁻ mg/l	EN 196-2	≤ 200 i ≤ 600	≤ 600 i ≤ 3000	≤ 3000 i ≤ 6000
pH	ISO 4316	≤ 6.5 i ≤ 5.5	≤ 5.5 i ≤ 4.5	≤ 4.5 i ≤ 4.0
CO ₂ mg/l aggressive	EN 13577	≤ 15 i ≤ 40	≤ 40 i ≤ 100	≤ 100 do zasićenja
NH ₄ ⁺ mg/l	ISO 7150-1 or ISO 7150-2	≤ 15 i ≤ 30	≤ 30 i ≤ 60	≤ 60 i ≤ 100
Mg ²⁺ mg/l	ISO 7980	≤ 300 i ≤ 1000	≤ 1000 i ≤ 3000	≤ 3000 do zasićenja
Tlo				
SO ₄ ²⁻ mg/kg ^a total	EN 196-2 ^b	≤ 1000 i ≤ 3000 ^b	≤ 3000 ^b i ≤ 12000	≤ 12000 i ≤ 24000
Acidity/ml/kg	DIN 4030-2	200 Baumann Gully	Nije poznat slučaj u praksi	
<ul style="list-style-type: none"> a) Glinasto tlo sa propustljivošću nižom od 10⁻⁶ m/s može da se premesti u nižu klasu b) Metod prevída ekstrakciju SO₄²⁻ hloro vodonodnom kiselinom, alternativno može da se koristi vodena ekstrakcija, ako postoji iskustvo na mesto upotrebe betona c) Limit od 3000 mg/kg bice smanjen na 2000 ako postoji rizik od akumulacije jona sulfatnih jona u betonu kao posledica ciklusa susenja I kvasenja ili kapilarne apsorpcije 				
U zavisnosti od tipa konstrukcije i njenog okruženja jedna konstrukcija može istovremeno da bude izložena na više različitih uticaja. Dati primeri služe za ilustraciju.				

1.1 PREPORUKE ZA SASTAV I SVOJSTVA BETONA

Evropski standard EN 206 daje i preporuke za minimalne uslove koje beton mora da zadovolji u zavisnosti od toga u kojoj je klasi izloženosti klasificiran. Ove se preporuke odnose na minimalnu čvrstoću betona, minimalnu količinu cementa, tipa cementa i sadržaja uvučenog vazduha. Dati su u Tabeli 3.

Tabela 3 - Preporuke za ograničujuće vrednosti sastava i svojstva betona

	Nema opasnosti od korozije	KLASE IZLOŽENOSTI																	
		Krozija izazvana hloridima															Agresivna hemiska sredina		
		Krozija izazvana karbonizacijom					Morska voda			Hloridi koji ne potiču od morke vode			Ciklusi smrzavanje -razmrzavanje						
		X0	XC1	XC2	XC3	XC4	XS1	XS2	XS3	XD1	XD2	XD3	XF1	XF2	XF3	XF4	XA1	XA2	XA3
Max W/C	-	0,65	0,6	0,55	0,5	0,5	0,45	0,45	0,55	0,55	0,45	0,55	0,55	0,50	0,45	0,55	0,50	0,45	
Minimum strength class	C12/15	C20/25	C25/30	C30/37	C30/37	C30/37	C35/45	C35/45	C30/37	C30/37	C35/45	C30/37	C30/37	C30/37	C30/37	C30/37	C30/37	C35/45	
Minimum količina cementa	-	260	280	280	300	300	320	340	300	300	320	300	300	320	340	300	320	360	
Minimum količina vazduha	-	-	-	-	-	-	-	-	-	-	-	-	-	4,0 ^a	4,0 ^a	4,0 ^a	-	-	
Drugi zahtevi	-	Agregat u saglasnosti sa EN12620 sa zadovoljavajućom otpornošću na smrzavanje-razmrzavanje															Sulfato-otporni cement ^b		
a)	Kada beton nije aeriran, otpornost betona treba da se ispita sa odgovarajućim metodama u poređenju sa betonima za koje je otpornost na smrzavanje-razmrzavanje dokazana																		
b)	Kad SO ₄ ²⁻ uzrokuje klase izloženosti XA2 i XA3 ključno je da se upotrebi sulfato-otporni cement. Kada je cement klasificiran prema otpornosti na sulfatesrednje ili visoko S-otporni cement treba da se koristi kod XA2 (i XA1 kad postoji mogućnost), i visoko sulfato-otporni cement treba da se koristi kod klase izloženosti XA3.																		
Preporuke se odnose na prepostavljeni eksploatacioni period konstrukcije od 50 godina. Odnosi se na cement CEM I i maksimalnu granulaciju od 20-32mm. Limitirajuće vrednosti W-C faktora i minimalne količine cementa odnose se na sve slučajeve, dok se jakost betona može dodatno specificirati																			

Naravno, pravilni izbor komponenta i performansi betona su samo prvi uslov da se obezbedi trajnost izložene konstrukcije. Način i kvalitet izvođenja, kao i primenjeni sistemi za zaštitu konstrukcija su ključni za produženje trajnosti i eksploatacionog perioda određene konstrukcije.

2 VODONEPROPUSNOST BETONA – FAKTOR ZA POVEĆANJE TRAJNOSTI IZLOŽENOG BETONA

Osnovni medijum kroz koji potencijalno štetni hemijski agensi mogu da uđu u beton je voda. U vodi su često rastvoreni hloridi (pr. morska voda, podzemne vode ili voda sa kolovoza koji su tretirani solju za odmrzavanje). Hloridi izuzetno nepovoljno utiču kako na beton, tako i na armaturu koju korodiraju. Pored hlorida, kroz vodu u beton mogu da uđu i sulfati, organske materije, karbonati, itd. Dodatno, sama voda koja uđe u beton, ukoliko smrzne može fizički da naruši njegovu strukturu. Čista izvorska voda (takozvana “mekka” voda) može hemijski da rastvori kalcium iz krečnjaka (agregata) i hemijski da degradira beton.



Posledice smrzavanja



Rastvoreni krečnjak - agregat “mekom” izvorskom vodom



Degradacija betona izloženog na karbonizaciju

Sam po sebi, beton je hidrofilni material, odnosno nikako ne može da bude 100% otporan na prodor vode - struktura cementnog maltera (a time i samog betona) je takva da je porozan – u njemu neminovno postoje mikro prsline i pore kroz koje voda može da proдре. Prodiranje može da bude posledica hidrostatskog pritiska, ili kapilarnih pojava.

2.1 OTPORNOST BETONA NA PRODOR VODE POD PRITISKOM

Otpornost betona na prodor vode pod pritiskom (VDP betona), zavisi od čvrstoće betona i njegovih performansi. Beton visoke čvrstoće, koji je homogen, dobro ugrađen, vibriran i negovan, ima zatvoreniju strukturu i monogo manju mogućnost prodiranja vode. Evropski standard EN 12390-8 Depth of penetration of water under pressure definiše način ispitivanja nepropusnosti betona na prodor vode pod priskom. Standard predviđa izlaganje betona na konstantni vodeni pritisak od 5,0 Bar, u trajanju od 72 sata. Projektantima ostaje da sami odrede koja otpornost na prodiranje vode zadovoljava uslove na određenom projektu – naravno, uzimajući u obzir stepen izloženosti betona u datoj konstrukciji.



Ispitivanje otpornosti betona na prodor vode pod pritiskom

Sa ciljem da se utvrdi kako različitih performansi betona i aditivi utiču na otpornost betona na prodiranje vode, sproveli smo uporedno ispitivanje vodo-nepropusnosti na 3 klase betona. Beton je proizveden na fabrici betona koristeći standardne recepture za proizvodnju – jedino je dodatak za vodonepropusnost bio dodat naknadno. Korišćen je cement TITAN “Usje” – Skopje, CEM II/A-V 42.5 R. Agregat je krečnjačkog porekla, drobljeni, osim frakcije 0-4mm koja je mešana sa 20% peska rečnog porekla. Betoni su 4-frakciski, sa maksimalnom frakcijom do 32mm. Konzistencija betona je visoke klase S3 i S4 (pumpabilni beton). Ispitivanja su izvršena u nezavisnoj akreditovanoj laboratoriji GIM – Skopje.

Tabela 3

Klasa betona	Količina cementa [kg/m ³]	Plastifikaor / Superplastifikator	Aditiv za vodonep.	Konz. Slump	Težina betona [kg/m ³]	Čvrstoća betona [MPa]	Max prodor vode [mm]
C25/30	360	Fluiding-M	/	S3	2364	43,6	35
C25/30	360	Fluiding-M	Hidrofob-T (1%)	S3	2353	46,8	21
C30/37	390	Superfluid-21M EKO	/	S4	2413	58,2	19

Zaključak: Rezultati pokazuju da na otpornost betona na prodiranje vode pod pritiskom utiču čvrstoća betona, kvalitet ugradnje, zbijenost i homogenost betona. Svi ovi činioci doprinose da struktura betona bude zatvorenija i da smanji procenat mikroprrsline kroz koje voda može da prode u beton. Postižu se upotrebom aditiva za beton od grupe Superplastifikatora, koji omogućuju redukciju vode u betonu, veće čvrstoće i kvalitetniju ugradnju betona. Dodatno, upotrebom specijalizovanih aditiva za VDP beton, postiže se zatvaranje pora u betonu i smanjuje se mogućnost za prodiranje vode.

2.2 KAPILARNA APSORPCIJA VODE U BETONU

Drugi način na koji voda može da uđe u beton je kapilarno upijanje – prenos vode kroz sistem kapilarnih prsline prisutnih u betonu. Da bi došlo do pojave kapilarnog upijanja nije potrebno da postoji značajan hidrostatski pritisak - dovoljno je da beton bude u kontaktu sa vodom ili vlagom. Kapilarna vlaga u betonu često se “penje” i prodire u delove konstrukcije koji nisu direktno u kontakt sa vodom.



Efekt kapilane apsorpcije vode u betonu



Vodo-repелentna betonska površina

Smanjenje mogućnosti za pojavu kapilarne apsorpcije vode u betonu postiže se na način što se proizvede beton visokih performansi, koji je pravilno ugrađen i ima zatvoreniju strukturu (za

to se koriste aditivi – superplastifikatori). Dodatno, primenom specijalizovanih aditiva za beton (Hidrofob-T) postiže se formiranje vodo-nerastvorljivih kristala koji (delimično) zatvaraju strukturu betona.

Najefikasniji način za sprečavanje kapilarnog upijanja vode u betonu je upotreba aditiva na bazi silana-siloksana (Hidrofob-21), koji utiču na površinski napon betona, praveći da betonska površina odbija molekule vode koji ne mogu da prodru u kapilare u betonu. Na taj način postiže se da površina betona postane visoko hidrofobna, odnosno Vodorepelentna. Isti materijali (najčešće prilikom sanacija) mogu da se koriste i kao premazi za površinsku impregnaciju betona (EN1504-2, metoda 1- hidrofobna impregnacija). Evropski standardi predviđaju više metoda kako da se utvrdi stepen otpornosti na kapilarnu apsorpciju vode kod betona i drugih građevinskih materijala koji imaju otvorenu strukturu.

Prema standardu EN480-5 Determination of capillary absorption, ispituje se količina apsorbarane vode nakon 7 dana (ispitivanje se vrši na standardni malter, i ispituje se u odnosu na etalon). Uzorci se čuvaju u zatvorenoj komori, postavljeni na rešetku, konstantno potopljeni u vodi visine 2-4mm. Nakon 7 dana meri se masa uzoraka i određuje količina apsorbarane vode. Naša su laboratoriska ispitivanja pokazala smanjenje kapilarne apsorpcije vode kod uzorka maltera sa dodatkom Hidrofob-21 (dozaža 0,7%) za oko 75% u odnosu na etalon (po standardu mora da bude više od 50%). Drugi deo ispitivanja vrši se testiranjem kapilarne apsorpcije na 90 dana star uzorak, tretiran u vodi 28 dana. Naša su laboratoriska ispitivanja pokazala smanjenje kapilarne apsorpcije vode kod uzorka maltera sa dodatkom Hidrofob-21 (dozaža 0,7%) za oko 70% u odnosu na etalon (po standardu mora da bude više od 40%).



Ispitivanje otpornosti betona na kapilarnu apsorpciju



Karsten tube metoda

Druga metoda za određivanje kapilarne apsorpcije koju smo primenili je Karsten tubes metoda. Prema ovoj metodi ispituje se volumen vode koji se apsorbovao - “upio” u neki medium za određeno vreme. Aplikacija vode vrši se preko cevi (Karsten tube), koja se fiksira na površinu poroznog materijala. Ovim metodom se ispituju i različiti materijali koji su površinski impregnirani (beton, malter, prirodni kamen, opeka).

Naša su ispitivanja obuhvatila dva tipa betona sa različitom čvrstoćom:

Prvo je testiranje izvršeno na beton C30/37 ,4 fr. $d_{max}=32mm$, sa $350kg/m^3$ CEM I

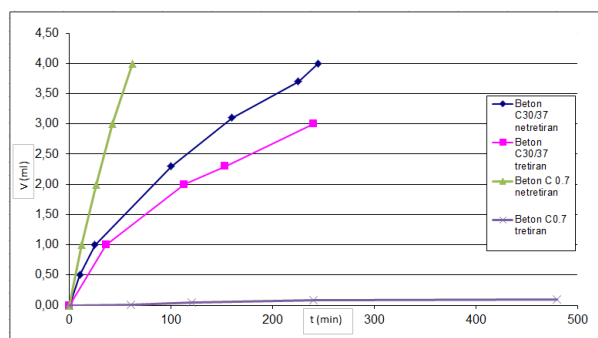
Tabela 4 - Zavisnost vreme – upijena voda kod betona C30/37

Etalon	t (min)	0	10	25	100	160	225	245
	V(ml)	0	0,5	1	2,3	3,1	3,7	4
Beton tertian Hidrofob-21	t (min)	0	36	113	153	240	/	/
	V(ml)	0	1	2	2,3	3	/	/

Drugo je testiranje izvršeno na beton C 0.70 (prema EN 1766) sa 275kg/m³ CEM I i čvrstoćom na pritisak od 25-35MPa.

Tabela 5 - Zavisnost vreme – upijena voda kod betona C 0.7

Etalon	t (min)	0	12	26	42	62
	V(ml)	0	1	2	3	4
Beton tertian Hidrofob-21	t (min)	0	60	120	240	480
	V(ml)	0	0	0	0	0



Objašnjenje: Velika razlika u efektu površinske impregnacije betona je posledica različitog kvaliteta, odnosno različite otvorenosti strukture i poroznosti materijala za dve klase betona. Slabiji beton klase C0.7 ima otvoreniju strukturu i u njemu lakše prodire aditiv Hidrofob-21. Zbog toga je i efekat površinske hidrofobne impregnaciji bolji – dobijena je kompletno vodo-repelentna površina betona.

3 MERE ZA POVEĆANJE TRAJNOSTI I ZAŠTITU IZLOŽENOG BETONA I KONSTRUKCIJA

Da bi se produžio vek trajanja betona koji je pored prodora vode izložen i na jaku hemijsku agresiju, neophodno je da se primene dodatne mere zaštite. Kod već postojećih konstrukcija nije moguće da se interveniše u samu strukturu betona nego potrebno je da se obezbedi dodatna

površinska zaštita izloženog betona, i eventualno sanacija nekih kontaminiranih ili degradiranih delova konstrukcije.

Za tu svrhu, Evropski standard EN1504-2 predviđa tri različite metode za površinsku zaštitu izloženog betona od prodiranja vode i hemiski agresivnih materijala – Hidrofobna impregnacija, Impregnacija i Oblaganje (coating).

3.1 HIDROFOBNA IMPREGNACIJA

Hidrofobna impregnacija se najčešće primenjuje u slučajevima gde postoji opasnost od prodora kapilarne vlage u betonu. Za tu namenu najčešće se primenjuju materijali na bazi silana i siloksana (Hidrofob-21), ili silikonata (Fasil-V). Prednost primene ovakvih materijala je taj što oni su najčešće bezbojni i ne menjaju izgled površine koja se tretira – što omogućuje njihovu primenu kod zaštite kuturno istorijskih spomenika, religijskih objekata, i dr. Druga osobina ovog tipa zaštite je ta što materijali za impregnaciju dozvoljavaju da konstrukcija “diše” – odnosno da ostane paropropusna, što sprečava do pojavu kondenzacije u samom objektu. Hidrofobna impregnacija primenjuje se za zaštitu konstrukcija prema sledećim principima (EN1504-9): P1 Zaštita od prodora, P2 Kontrola Vlage, P8 Povećanje otpora preko limitiranja sadržaja vlage.

3.2 IMPREGNACIJA

Impregnacija je metod za zaštitu konstrukcija koja predviđa delimično zatvaranje pora u betonu i formiranje isprekidanog zaštitnog filma na površini konstrukcije. Za tu namenu najčešće se primenjuju materijali na bazi organskih polimera. Primeri materijala za impregnaciju su Veza-U i Dekosil. Pored zaštite konstrukcije od prodiranja vodi kotaminenata, primenom materijala za impregnaciju postiže se i površinsko pojačavanje i stabilizacija betona. Impregnacija primenjuje se za zaštitu konstrukcija prema sledećim principima (EN1504-9): P1 Zaštita od prodora, P5 Fizička otpornost (površinsko pojačanje).

3.3 OBLAGANJE (COATING)

Oblaganje konstrukcije podrazumeva formiranje kontinuiranog neprekinutog zaštitnog sloja na površini betona, debljine do 5mm. Vakav postupak zaštite preporučuje se u svim slučajevima veće izloženosti betona, kada postoji značajna izloženost konstrukcije na dejstvo karbonizacije, ciklusa mržneja i hlorida (pr. kod putne infrastrukture, termo-elektro industrije i u urbanim područjima). Antikorozivno oblaganje je neophodno kad je stepen izloženosti konstrukcije XC3, XC4; XD2, XD3; XS3; XF3÷4; XA1÷3. U zavisnosti od tipa hemiske agresije, postoji više tipova materijala koji se koriste za oblaganje-zaštitu konstrukcija (na bazi akrilatnih i metakrilatnih smola, poluretana, epoksidnih smola, itd). Oblaganje se primenjuje za zaštitu konstrukcija prema sledećim principima (EN1504-9): P1 Zaštita od prodora, P2 Kontrola Vlage, P5 Fizička otpornost (površinsko pojačanje), P6 Otpornost na hemikalije, P8 Povećanje otpora preko limitiranja sadržaja vlage.

Iskustvo pokazuje da se infrastrukturni objekti (mostovke konstrukcije, tueli, potporni zidovi i sl), najčešće izloženi na dejstvo karbonizacije i hlorida (klasa izloženosti XC3, XC4; XD1, XS1; XF3÷2; XA1). Za njihovu zaštitu trend je da se primenjuju premazi na bazi akrilnih smola. Prednost ovakvih materijalje što su ekološki (ne sadrže rastvarače), lako se nanose, podnose nanošenje na vlažnom betonu, ekonomični su, dok istovremeno efikasno štite beton od

prodora i korozije. Jedan ovakav materijal koji smo ispitivali i za koga pratimo referente objekte stare više od 10 godina je Antikorozin-BB.



*Zaštita mostovskog nosača oblaganjem -
coating (Antikorozin-BB)*



*Obrada pešačke staze na mostu – sistem na
bazi epoksida i metakrilata*

Druga kritična pozicija na infrastrukturnim objektima su betonski elementi i horizontalni delovi konstrukcija koji su pored hemiske izloženosti, dodatno izloženi na mehanička opterećenja – saobraćaj, habanje, udarce i zadržavanje vode. U ovakvim slučajevima potrebno je da se primene materijali i sistemi za zaštitu koji su mehanički otporni, otporni na habanje, protivklizni, UV-stabilni i otporni na atmosferske uticaje. Primer ovakvog sistema je protivklizni sistem koji se koristi za obradu pešačkih staza na mostovima. Sistem sačinjavaju epoksidni prajmer i izravnavajući sloj, kvarcni pesak koji daje protivkliznost i završni premaz na bazi metakrilata (Adingparker-P).

Nikola Tošić¹, Aleksandar Savić², Vedran Carević³

METODA ZA KONTINUALNO ODREĐIVANJE UPIJANJA VODE KRUPNOG AGREGATA OD RECIKLIRANOG BETONA

Rezime:

Agregati od recikliranog betona poseduju značajno veće upijanje vode u odnosu na prirodne agregate zbog prisustva cementne paste zaostale nakon drobljenja betonskog otpada. Visoka vrednost upijanja vode ovih agregata utiče na procedure spravljanja receptura betona sa agregatom od recikliranog betona. U ovom radu je predstavljena metoda za kontinualno određivanje upijanja vode agregata od recikliranog betona koja je bazirana na kontinualnom merenju mase uzorka. U istraživanju je ispitan i uticaj temperature sušenja na upijanje vode i analizirani su različiti efekti na agregat od recikliranog betona i prirodni agregat.

Ključne reči: agregat od recikliranog betona, prirodni agregat, upijanje vode

A METHOD FOR THE CONTINUOUS MEASUREMENT OF WATER ABSORPTION OF COARSE RECYCLED AGGREGATES

Summary:

Recycled concrete aggregates have a significantly higher water absorption compared to natural aggregates due to the presence of the residual cement paste after concrete waste crushing. This high water absorption determines the mixture proportioning for recycled aggregate concrete. In this study a method for the continuous measurement of the kinetics of water absorption is proposed based on the continuous measurement of the sample's mass. Also investigated, is the influence of the drying temperature on water absorption and different effects on recycled and natural aggregates are analysed and discussed.

Key words: recycled concrete aggregate, natural aggregate, water absorption

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1 INTRODUCTION

1.1 RECYCLED CONCRETE AGGREGATE

Concrete is the most widely used construction material with global production reaching 21 billion tons annually [1]. This poses an enormous strain on natural resources used in its production. Global annual consumption of natural aggregates is around 15 billion tons [2].

At the same time, aging structures are being demolished and large amounts of construction and demolition (C&D) waste are being generated. In the EU alone, almost 850 million tons of C&D waste are produced annually, and waste concrete is a large portion of it [3].

These problems have prompted researchers to study alternative materials for use as aggregates. One of the most promising alternatives is recycling of waste concrete which aims at solving both problems at once—reducing the consumption of natural resources and the amount of waste being landfilled. This approach has been studied for several decades [4].

Usually, waste concrete is crushed into aggregates called recycled concrete aggregates (RCA) which can be used in new applications. This crushing is performed in mobile or stationary recycling facilities using single or multiple-stage crushing in different crusher types, together with the removal of any impurities such as steel, wood, gypsum, masonry and glass. When this new aggregate, RCA, is used for the production of concrete, that new concrete is called recycled aggregate concrete (RAC).

The main difference between RCA and natural aggregate (NA), river gravel or crushed stone, is the so-called ‘residual cement paste’ that is left attached to RCA particles after crushing. This is actually a part of the hardened cement paste around the NA aggregate particles in the original concrete being recycled. Because of this, RCA can be viewed as a two-phase material containing natural aggregates and cement paste.

The main implication of the residual cement paste is higher water absorption of RCA compared with NA. Water absorption of RCA can range from 2% to 10% in the case of coarse aggregate (particle size >4 mm) and up to 15% in the case of fine RCA (particle size <4 mm) [5]. The extremely high absorption of fine RCA has led many standards to impose restrictions on its use in concrete [6]; hence, it was not considered in this study.

The large water absorption of RCA poses problems when designing RAC mixtures. During mixing RCA can absorb a certain amount of water, thus reducing the w/c ratio, causing workability problems. This has led some researchers to adopt an effective water-to-cement ratio (w/c_{eff}) which is maintained constant in the mix by adding additional water to saturate oven-dry (OD) aggregates or by using saturated surface dry (SSD) aggregates. Researchers have studied the effect of the moisture state of RCA on RAC properties, investigating the differences between using dry, semi-saturated and fully saturated RCA, and found a non-negligible influence of the moisture state [7,8]. Also investigated were different mixing procedures that would more adequately take into account the large water absorption of RCA [9].

Another important aspect of the residual cement paste attached to RCA particles is its potential for carbonation and consequently, CO₂ sequestration. Previous research has studied the possibilities of using RCA as a low-cost carbon sink, through open-air exposure during stockpiling. Such carbonation modification of RCA was shown to potentially decrease water

absorption and reduce porosity, thereby improving RCA properties making it better-suited for use in RAC [10,11].

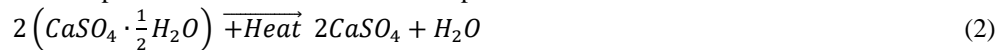
1.2 EXISTING PROCEDURES FOR MEASURING WATER ABSORPTION

All of the aforementioned only highlights the importance of adequately determining water absorption of RCA. So far, the most common method, at least in Europe, has been the EN 1097-6 standard's procedure used for NA [12]. In this method an aggregate sample is submerged in water for 24 h after which its SSD mass (M_a) is recorded. The SSD state is subjectively assessed by drying the wet aggregates with a cloth. The aggregates are then placed in a dryer and left at a temperature of $110 \pm 5^\circ\text{C}$ until mass stabilization after which the OD mass (M_s) is measured. Water absorption (WA) is then calculated from Equation 1:

$$WA = 100 \cdot \frac{M_a - M_s}{M_s} \quad (1)$$

However, several problems exist with this procedure, especially when it is used on RCA and this has already been highlighted in previous studies [13]:

When RCA is dried at 110°C , not only free water, but also a part of the chemically bound water is evaporated from the residual cement paste:



There is no measurement of the kinetics of water absorption i.e. the development of absorption through time, which can be considered different in RCA compared with NA and significant for mixture proportioning [14].

There is no perturbation of the aggregates during soaking which means that air bubbles can remain trapped between aggregate particles and prevent complete absorption.

The SSD condition is subjectively assessed by wiping the aggregates with a cloth, although it was shown that this subjectiveness can introduce differences usually smaller than 5% [14].

This situation has led several researchers to search for alternative methods of measuring water absorption of RCA [13–18]. All of the proposed methods concentrate on using hydrostatic weighing as a way of measuring RCA absorption kinetics, sometimes in combination with pycnometer measurements.

2 EXPERIMENTAL PROGRAMME

2.1 SCOPE AND OBJECTIVE

In this study, water absorption of natural and recycled non-carbonated and fully carbonated aggregates was measured. For this purpose, a new test procedure was developed which relies on previously proposed methods by other authors [13–18].

The aim was, first, to determine whether the standard drying temperature proposed in EN 1097-6 [12] is adequate for RCA, and second, to determine whether carbonation modification of RCA has an effect on water absorption.

2.2 MATERIALS

Two particle sizes of both NA and RCA were used in this study – fraction II (4-8 mm) and fraction III (8-16 mm).

Natural aggregate was standard, commercially available river gravel from the Danube river, purchased from Gradient d.o.o., Belgrade.

Recycled concrete aggregate was obtained from a demolished 40 year old highway bridge in the vicinity of Belgrade. The aggregate was obtained by crushing columns and the deck of the bridge in a construction site mobile crusher. The demolished structure was relatively clean from impurities as the asphalt had been scraped of the deck prior to demolition. After crushing, RCA was sieved into fractions II and III. Cores taken from the existing structure prior to crushing, showed a compressive strength of 23 MPa in the column and 35 MPa in the deck of the bridge. Carbonation depth of the parent concrete was tested and shown to be 25 mm in the column and 0 mm in the deck.

Immediately after crushing and sieving, a part of both RCA fractions were sealed in plastic bags to prevent further carbonation while another part of both fractions was put in a Memmert ICH260C carbonation chamber and exposed to $5 \pm 1\%$ of CO_2 concentration, $55 \pm 5\%$ relative humidity and a temperature of $20 \pm 2^\circ\text{C}$ until full carbonation i.e., mass stabilization.

2.3 METHODS

For the purpose of this study, a new method was used, based on hydrostatic weighing and on methods proposed in previous studies [13–18]. Since hydrostatic weighing was used, the sequence of the measurement procedure had to be reversed, compared with EN 1097-6 [12].

First, the aggregates were washed to clean them from fine particles. Afterwards, they were placed in an oven and dried at $110 \pm 5^\circ\text{C}$ or $45 \pm 5^\circ\text{C}$ for 24 h or until mass stabilization.

After this, the OD mass of the sample (m_0) was measured. Then, the sample was placed in a perforated steel basket. The sample size was chosen in such a way that no more than two or three layers of aggregate were in the basket. This amounted to ca. 400 g for fraction II and ca. 800 g for fraction III. For fraction II a basket with a finer perforation was used, compared with the basket for fraction III, as is shown on Figure 1.



Figure 1 – Apparatus for absorption measurement of aggregate fraction II (left) and III (right)

The basket with the sample inside was hung from the scale and the water container was raised so that the basket was completely immersed in water. The measurement procedure was

as follows—the mass of the immersed sample with the basket ($M(t)$) was recorded after 1, 3, 5, 10, 15, 30 minutes, every hour until 6 hours and the final reading (M_{24}) was taken after 24 hours. Before each reading the basket was unhung from the scale, shaken to remove entrapped air bubbles and hung again on the scale.

After 24 h, the water container was lowered and the aggregates were spread on a dry cloth and left to dry in open-air until the SSD condition was achieved, judged by naked eye observation. Then, the SSD mass of the sample (m_{24}) was measured.

The increase in sample mass at time t , equal to the mass of the absorbed water ($\Delta m_w(t)$) was calculated according to Equation 3:

$$\Delta m_w(t) = m_{24} - m_0 - (M_{24} - M(t)) \quad (3)$$

Finally, water absorption at time t , was calculated according to Equation 4:

$$WA(t) = \Delta m_w(t)/m_0 \quad (4)$$

3 RESULTS AND DISCUSSION

The sample designation was as follows: for NA “NA-II(III) 45(110)”, with II and III signifying the fraction and 45 and 110 the drying temperature; for RCA “RCA-II(III)-NC(FC) 45(110)”, the same as for NA, with and addition of NC signifying non-carbonated and FC fully carbonated RCA.

The results for NA, fractions II and III, are shown in Figure 2. Vertical lines were drawn at 1 h and these absorption values highlighted because this time was deemed typical for ready-mixed concrete transport in Serbia and hence, important for mix design.

Several things can be seen from the figures. First, the absolute values of absorption are low, just above 1% for fraction II, although this is relatively high for NA. Second, the absorption kinetics are such that it can be said that most of the absorption occurs within the first hour. Finally, and most importantly, there is, for practical purposes, no significant difference between drying the samples at 45 or 110°C. This justifies the use of EN 1097-6 in the case of NA.

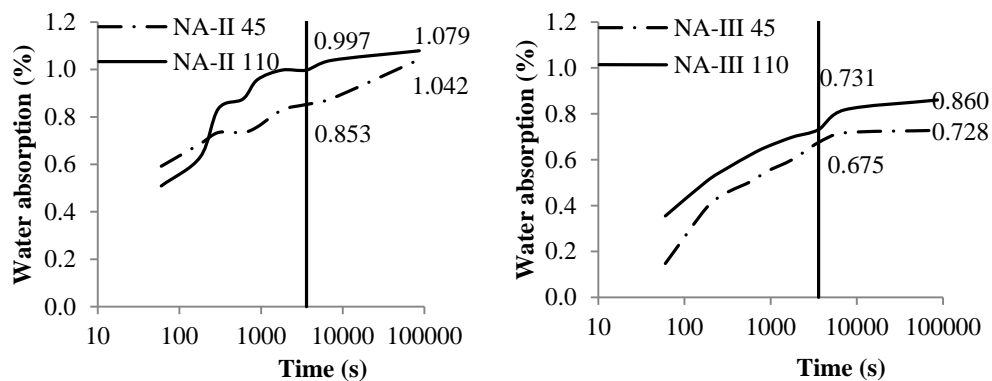


Figure 2 – Water absorption kinetics of NA fraction II (left) and fraction III (right)

The results for RCA, fractions II and III, non-carbonated and fully carbonated, are shown in Figures 3 and 4. From these figures, several important conclusions can be drawn. First, there is

a significant difference between drying the samples at 45 and 110°C, for both non-carbonated and fully carbonated RCA, although the difference is smaller in the case of fully carbonated aggregates.

For non-carbonated fraction II, the decrease in absorption is 29.5% after 1 h and 27.1% after 24 h. For fully carbonated fraction II, the decrease is 11.6% after 1 h and 10.4% after 24 h. For non-carbonated fraction III, this decrease is 47.1% after 1 h and 46.5% after 24h. For fully carbonated fraction III, the decrease is 26.4% after 1 h and 24.6% after 24 h.

Compared to NA, the kinetics of RCA absorption seem to be much faster in the first hour and they don't change with carbonation modification. Finally, there is a substantial decrease in water absorption after carbonation modification. For fraction II dried at 110°C the decrease in absorption after 24 h is 17.6% and an increase of 1.2% for drying at 45°C. For fraction III dried at 110°C the decrease in absorption after 24 h is 33.9% and 6.7% for drying at 45°C.

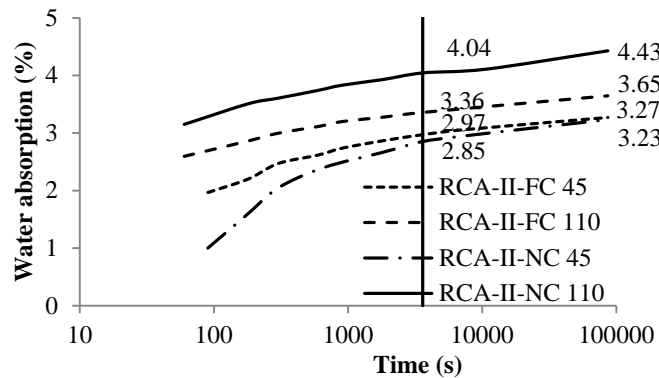


Figure 3 – Water absorption kinetics of RCA fraction II

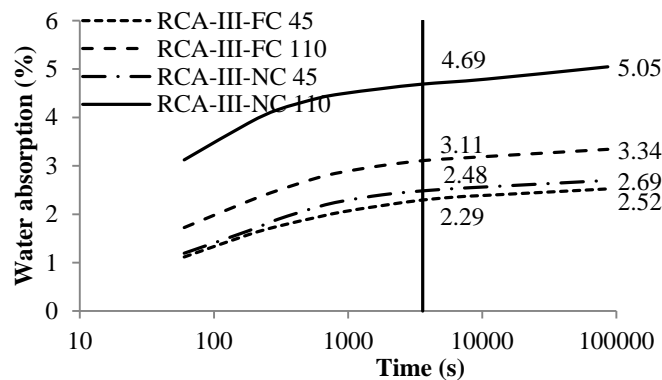


Figure 4 – Water absorption kinetics of RCA fraction III

These results have important implications for mixture proportioning of RAC. Since in practice RCA will never be exposed to temperatures above 100°C, measuring absorption by drying RCA at 110°C will lead to overestimated water absorption values. The overestimation

decreases with increasing carbonation of RCA, but it does not fall below 10% even in the extreme case of fully carbonated RCA.

Using this overestimated absorption in mixture proportioning can lead to the usage of unnecessarily high amounts of additional water for absorption compensation. This can then lead to higher w/c ratios and weaker bonds between the cement paste and aggregates. It must be concluded then, that the observed problems of workability of RAC arise not from high water absorption, but from the irregular and jagged shape of the particles. This problem should be solved by using plasticizers and not by increasing the water content.

It is necessary therefore, to adopt new standards for RCA water absorption measurement which would take into account its specificities in an appropriate manner.

4 CONCLUSIONS

In this study, the water absorption of NA and RCA was measured using a hydrostatic weighing technique enabling continuous measurement and different drying temperatures were analysed. Carbonation modification of RCA and possible improvements in water absorption were also analysed. Based on the results presented in the previous section, the following conclusions can be drawn:

- It is possible to measure water absorption continuously using the hydrostatic weighing technique and the procedure described in this paper.
- For the measurement of water absorption of NA, the standard technique described in EN 1097-6 can be used.
- For RCA, significant differences in absorption are obtained when drying the aggregates at 45°C compared with 110°C. These differences range from 47.1% in the case of non-carbonated fraction III after 1 h, to 10.4% for fully carbonated fraction II after 24h.
- Carbonation modification of RCA can significantly change water absorption, ranging from 33.9% decrease in the case of fraction III dried at 110°C to an increase of 1.2% in the case of fraction II dried at 45°C.
- It's necessary to develop new standards for RCA water absorption that will take into account its specificities, thus eliminating potentially erroneous mixture proportioning of RAC.

ACKNOWLEDGMENTS

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ISPITIVANJE KONTINUALNIH NOSAČA OD SCC BETONA SA RECIKLIRANIM AGREGATOM

Rezime:

S-94

U okviru ovog rada prikazani su i diskutovani rezultati eksperimentalnih ispitivanja kontinualnih grednih nosača od samougrađujućeg betona sa rečnim i recikliranim agregatom izloženih savijanju do loma. Izvršena je sistematizacija u literaturi postojećih znanja vezanih za mehaničke karakteristike samougrađujućeg betona sa recikliranim agregatom. Na osnovu eksperimentalnog ispitivanja na kontinualnim grednim nosačima, uporedne analize rezultata ispitivanja nosača sa recikliranim agregatom u odnosu na nosač sa prirodnim agregatom ispitana je mogućnost primene recikliranog agregata za izradu konstrukcijskih elemenata.

Ključne reči: Samougrađujući beton, Reciklirani agregat, Kontinualni gredni nosač

TESTING OF CONTINUOUS BEAMS OF SCC CONCRETE WITH RECYCLED AGGREGATES

Summary:

The paper presents and discusses the results of experimental tests of continuous beams of self-compacting concrete with recycled aggregate and river exposed to bending fracture. Systematization of existing literature knowledge regarding the mechanical properties of self-compacting concrete with recycled aggregate was done. Based on the experimental research on continuous beam carriers, comparative analysis of the results of beams with recycled aggregates in relation to the carrier with natural aggregates examined the possibility of using recycled aggregates to produce structural elements.

Key words: Self-compacting concrete, Recycled aggregates, Continuous beams

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1 UVOD

Samougrađujući beton (SCC) kao relativno nov materijal predstavlja još uvek jednu nedovoljno istraženu oblast, kako u pogledu karakteristika u svežem, tako i u očvrsлом stanju. Prve preporuke za pravilno projektovanje samougrađujućeg betona objavljene su od strane EFNARC 2002. godine [1]. Evropska projektna komisija, koju sačinjava pet evropskih asocijacija je skicirala evropske preporuke za samougrađujući beton ("The European Guidelines for Self Compacting Concrete") 2005. godine [2]. Od strane Tehničkog komiteta CEN/TC 104 "Concrete and related products" pripremljen je standard EN 206-9:2010 [3] koji je u februaru 2010. godine objavljen od strane Evropskog komiteta za Standardizaciju. Ovaj standard, koji definiše pravila za projektovanje samougrađujućeg betona, je dodatak standardu EN 206-1:2000 i postaće njegov sastavni deo.

Sve veći nedostatak prirodnog agregata, kao i sve veće količine građevinskog otpada krajem prošlog veka otvorili su pitanje njegove prerade i ponovne upotrebe u građevinarstvu [4]. Primena recikliranog agregata dobijenog od građevinskog otpada nastalog rušenjem dotrajalih objekata do sada se mahom koristila za izradu nekonstrukcijskih betonskih elemenata (ivičnjaka, ograda) kao i tampon slojeva i podloga [5]. Poslednjih godina sve je veći broj istraživanja u kojima se ispituje mogućnost spravljanja samougrađujućeg betona sa recikliranim agregatom, kao i njegova primena za spravljanje konstrukcijskih elemenata.

U okviru rada prikazani su u literaturi dostupni podaci do sada sprovedenih istraživanja vezanih za mehaničke karakteristike samougrađujućeg betona u svežem i očvrsлом stanju i u njima date preporuke upoređene su sa eksperimentalno dobijenim rezultatima ispitivanja sprovedenog u okviru doktorske disertacije. Prikazano je i ispitivanje sprovedeno na kontinualnim grednim nosačima opterećenim kratkotrajnim statičkim opterećenjem sve do loma. Za isti procenat armiranja nosača sprovedena je analiza rezultata ispitivanja nosača izrađenih od samougrađujućeg betona sa kvalitetnim recikliranim agregatom u odnosu na nosač spravljen od samougrađujućeg betona sa prirodnim agregatom. Na osnovu analize i diskusije dobijenih rezultata potvrđena je mogućnost primene recikliranog agregata za izradu konstrukcijskih elemenata od samougrađujućeg betona.

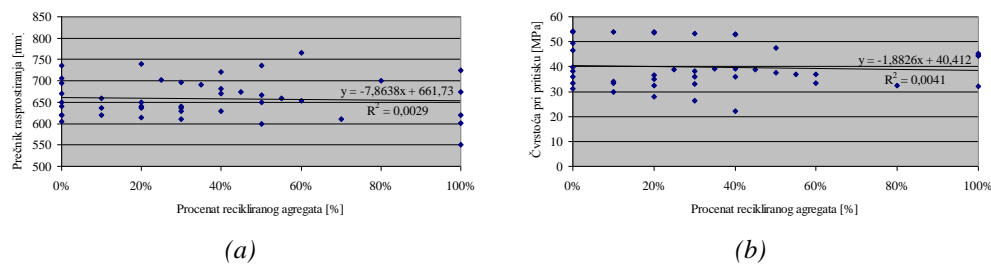
2 KRATAK PREGLED MEHANIČKIH KARAKTERISTIKA SAMOUGRAĐUJUĆEG BETONA

Brojna istraživanja vezana za samougrađujući beton odnosila su se na njegove karakteristike u svežem stanju, dok se verovalo da su njegove karakteristike u očvrsлом stanju slične vibriranom betonu. Poznavanje mehaničkih karakteristika samougrađujućeg betona u očvrsлом stanju, od primarnog je značaja za projektovanje konstrukcija, pa je razumljivo pojačano interesovanje građevinskih konstruktera za tu oblast.

U analiziranim radovima [6-9] prikazani su podaci u vezi sa sastavom betonske mešavine, kao i karakteristike samougrađujućeg betona u svežem i očvrsлом stanju. Sva sprovedena eksperimentalna istraživanja bazirana su na određivanju čvrstoće pri pritisku samougrađujućeg betona, kao merodavnog parametra za definisanje njegove klase. Druge mehaničke karakteristike nisu značajnije istraživane, pri čemu veličina uzorka, uslovi negovanja i metode ispitivanja variraju kod različitih istraživača.

Za kvalitetno projektovanje konstrukcijskih elemenata u praksi potrebno je i poznavanje analitičkih veza kojima je definisan odnos između čvrstoće pri pritisku i čvrstoće pri zatezanju, kao i odnos između čvrstoće pri pritisku i modula elastičnosti. Na osnovu do sada sprovedenih istraživanja u nekim od analiziranih radova prezentovane su i analitičke veze za samougrađujući beton sa prirodnim agregatom [9].

Poslednjih godina sproveden je veliki broj istraživanja vezanih za mogućnost primene recikliranog građevinskog otpada kao agregata za spravljanje samougrađujućeg betona. U radu [10] izvršena je sistematizacija i analiza dosadašnjih istraživanja vezanih za mogućnost primene recikliranog agregata za spravljanje samougrađujućeg betona. Na osnovu analize rezultata Slump-flow testova sprovedenih od strane mnogih autora, na Slici 1 (a) prikazana je zavisnost između količine recikliranog agregata izraženog u procentima i prečnika rasprostiranja sveže betonske mase. Uočeno je da promena prečnika rasprostiranja sveže betonske mase samougrađujućeg betona sa 100% krupnozrnog recikliranog agregata u odnosu na mešavinu sa prirodnim agregatom iznosi 7,8 cm.



Slika 1 - (a) Odnos između procenta recikliranog agregata i prečnika rasprostiranja, (b) Odnos između procenta recikliranog agregata i čvrstoće pri pritisku [10]

Do sada sprovedena eksperimentalna istraživanja mehaničkih karakteristika samougrađujućeg betona sa recikliranim agregatom u očvrsлом stanju, koja su analizirana u radu [10], bazirana su na određivanju čvrstoće pri pritisku. Određivanje čvrstoće pri zatezanju vršeno je kako cepanjem tako i savijanjem, dok određivanje modula elastičnosti gotovo da nije ni prikazano. Na osnovu analize rezultata čvrstoće pri pritisku, sprovedenih od strane mnogih autora, na Slici 1 (b) prikazana je zavisnost između količine recikliranog agregata izraženog u procentima i čvrstoće pri pritisku. Uočen je pad čvrstoće pri pritisku samougrađujućeg betona sa 100% recikliranog agregata u odnosu na mešavinu sa prirodnim agregatom od 1,88 MPa.

3 EKSPERIMENTALNO ISPITIVANJE

Eksperimentalna laboratorijska ispitivanja sprovedena su u dve faze. Prva faza je obuhvatila ispitivanje fizičko-mehaničkih svojstava materijala. Ispitane su mehaničke karakteristike samougrađujućeg betona spravljenog sa rečnim agregatom (etalon uzorak), kao i betona spravljenog sa kvalitetnim recikliranim agregatom. Druga faza obuhvatila je ispitivanja kontinualnih grednih nosača izrađenih od samougrađujućeg betona sa recikliranim agregatom i nosača izrađenih od samougrađujućeg betona sa prirodnim agregatom (etalon uzorci) pod dejstvom kratkotrajnog probnog opterećenja sve do loma [11].

Za spravljanje betonskih mešavina korišćene su frakcije 0/4, 4/8 i 8/16 mm rečnog agregata i frakcije 4/8 i 8/16 mm recikliranog agregata. Spravljena je mešavina SCC sa rečnim agregatom, kao i mešavine sa recikliranim agregatom i to: SCC R 50 kod koje je frakcija rečnog agregata 4/8 mm zamenjena recikliranim agregatom i SCC R 100 kod koje su frakcije rečnog agregata 4/8 mm i 8/16 mm zamenjene recikliranim agregatom. Korišćeni reciklirani agregat dobijen je drobljenjem mehanički oštećenih prefabrikovanih betonskih ploča.

Svi uzorci mešavina koje su ispitivane u okviru eksperimentalnog ispitivanja izrađeni su od cementa CEM I 42.5 R proizvođača HOLCIM. Korišćeno je kameno brašno dobijeno mlevenjem krečnjaka, a za spravljanje mešavina korišćen je aditiv MC PowerFlow 1102, tipa hiperplastifikator.

Programom eksperimentalnog ispitivanja definisane su geometrijske karakteristike uzoraka za ispitivanje, način njihove izrade, raspored i vrsta mernih instrumenata, kao i procedura ispitivanja. Osim vrste agregata, variran je i procenat armiranja glavne armature (0,65%, 0,94%, 0,86%), dok je podeona armatura za sve nosače bila ista. U ovom radu prikazani su rezultati ispitivanja kontinualnih grednih nosača sa procentom armiranja od 0,65%.

3.1 REZULTATI ISPITIVANJA SVEŽE BETONSKE MASE

Projektovanje betonskih mešavina (SCC, SCC R 50, SCC R 100) za potrebe eksperimenta obavljeno je tako da se postigne jednaka konzistencija kako etalona (SCC) tako i mešavina sa recikliranim agregatom (SCC R 50 i SCC R 100). Tokom *Slump-flow* testa mereno je vreme (t_{500}) za koje beton dostigne prečnik od 500 mm. Za sve mešavine određena je i zapreminska masa u svežem stanju, a rezultati su prikazani u Tabeli 1.

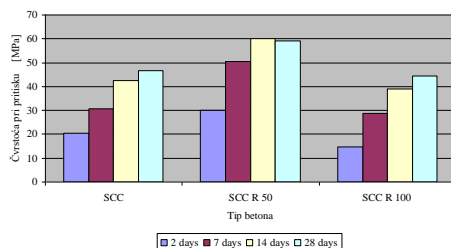
Tabela 1. Rezultati ispitivanja sveže betonske mešavine

Vrsta betona	Zapreminska masa, [kg/m ³]	Slump-flow test, D[cm]	t_{500} , [s]	ω_c (vodocem. faktor)
SCC	2305	60,5	4,8	0,49
SCC R 50	2293	60,0	5,0	0,50
SCC R 100	2268	60,2	5,0	0,51

Poređenjem dobijenih rezultata Slump-flow testova sa preporukama definisanim u EFNARC i EN 206-9:2010 projektovane mešavine svrstavaju se u klasu SF1, dok se na osnovu izmerenog vremena (t_{500}) zaključuje da mešavine pripadaju klasi VS2. Tokom eksperimenta nije uočeno izdvajanje vode.

3.2 REZULTATI ISPITIVANJA OČVRSLOG BETONA

Ispitivanje čvrstoće pri pritisku za sve mešavine sprovedeno je na kockama ivice 150 mm posle 2, 7, 14 i 28 dana, a dobijeni rezultati su prikazani na Slici 2.

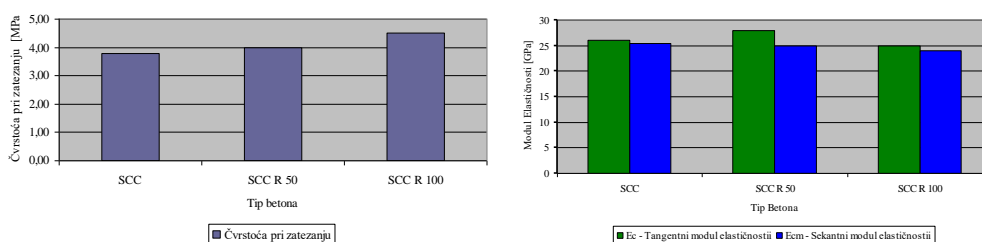


Slika 2 - Grafički prikaz čvrstoće pri pritisku

Čvrstoća betona pri pritisku pri istoj starosti betona (28 dana) je različita u zavisnosti od tipa mešavine. Kod sve tri mešavine izražen je priraštaj ranih čvrstoća, koje u prvih 7 dana dostižu više od 60 % čvrstoće betona pri pritisku za starost od 28 dana. Mešavina SCC R 50 ima veću čvrstoću za 26,85 %, u odnosu na čvrstoću etalon uzorka dok mešavina SCC R 100 ima za 4,40 % manju čvrstoću u odnosu na čvrstoću etalon uzorka.

Promene u čvrstoći pri pritisku kod mešavina sa recikliranim agregatom treba tražiti u mikrostrukturi betona. Povećanje čvrstoće pri pritisku mešavine SCC R 50 treba tražiti u mogućoj dopunskoj hidrataciji stare cementne paste koju sadrži reciklirani agregat, većoj količini drobljenog kamenog agregata u zamenjenoj frakciji (8/16) recikliranog agregata ili efikasnijem međusobnom pakovanju uglastih zrna recikliranog i zaobljenih zrna prirodnog agregata. Kod mešavine SCC R 100 došlo je do neznatnog pada čvrstoće pri pritisku, jer je druga frakcija, koja je u ovom slučaju reciklirani agregat, sadržala veći procenat sitnih čestica (prašine), čime je smanjena količina krupnog agregata koji ima uticaj na čvrstoću betona.

Čvrstoća pri zatezanju cepanjem određena je posle 28 dana na cilindrima Ø150/300 mm u skladu sa EN 12390-6. Rezultati ispitivanja čvrstoće pri zatezanju cepanjem prikazani su na Slici 3 (a). Povećanje čvrstoće pri zatezanju cepanjem kod mešavina sa recikliranim agregatom je neznatna u odnosu na etalon uzorak.



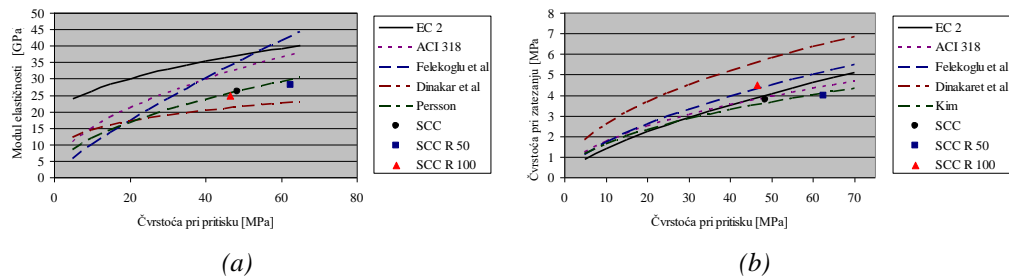
(a)

(b)

Slika 3 - (a) Grafički prikaz rezultata ispitivanja čvrstoće pri zatezanju cepanjem; (b) Grafički prikaz rezultata tangentnog i sekantnog modula elastičnosti

Određivanje tangentnog i sekantnog modula elastičnosti obavljeno je posle 28 dana na cilindrima Ø150/300 mm u skladu sa standardom EN 1992-1-1:2004. Dobijene su relativno

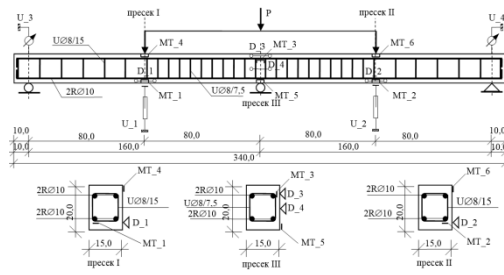
niske vrednosti modula elastičnosti kako za etalon tako i za mešavine sa recikliranim agregatom, Slika 3 (b).



Slika 4 - (a) Grafički prikaz zavisnosti modula elastičnosti od čvrstoće betona pri pritisku; (b) Grafički prikaz zavisnosti čvrstoće pri zatezanju od čvrstoće pri pritisku

Od svih razmatranih analitičkih formulacija kojima je definisana zavisnost između modula elastičnosti i čvrstoće pri pritisku, eksperimentalno dobijeni rezultati su najbliži analitičkoj formulaciji predloženoj od strane Persona, Slika 4 (a). Eksperimentalno dobijene vrednosti čvrstoće pri zatezanju nalaze se između krivih koje su definisane analitičkim izrazima predloženim od strane Felekoglua i ostalih sa jedne i Kima sa druge strane, Slika 4 (b).

3.3 REZULTATI I ANALIZA REZULTATA ISPITIVANJA KONTINUALNIH NOSAČA



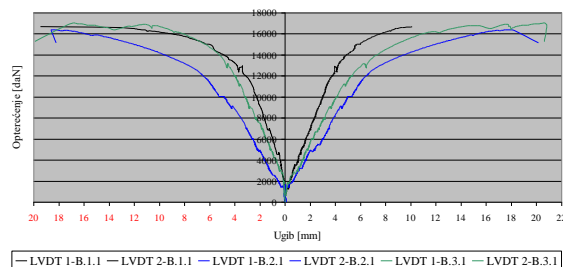
(a) (b)

Slika 5 - (a) Dispozicija kontinualnog grednog nosača u fazi ispitivanja (b) Raspored mernih instrumenata na kontinualnom grednom nosaču armiranom sa $\pm 2R\text{Ø}10$ mm

Ispitivani nosači su pravougaonog poprečnog preseka dimenzija $b/d=15/20$ cm, ukupne dužine 340 cm, raspona polja nosača 160 cm, armirani rebrastom armaturom RA 400/500–2, projektovane marke samougrađujućeg betona MB 40. U ovom radu su prikazani rezultati ispitivanja kontinualnih grednih nosača armiranih sa $\pm 2R\text{Ø}10$ mm, procenta armiranja 0,65%. Kontinualni gredni nosači izrađeni od samougrađujućeg betona na bazi prirodnog agregata

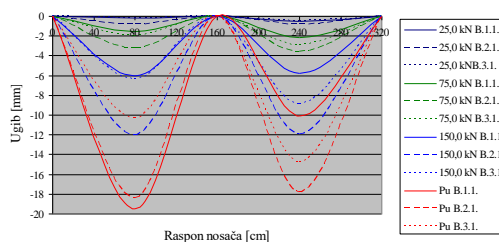
(SCC) kao i samougrađujućeg betona spavljenog sa agregatom od recikliranog agregata (SCC R 50, SCC R 100). Praćene su globalne deformacije nosača (ugibi), kao i pojava i razvoj prslina u funkciji priraštaja opterećenja.

Analizi ugiba, kao najočiglednijeg pokazatelja ponašanja konstrukcije pod uticajem opterećenja posvećena je naročita pažnja. Sprovedena je uporedna analiza eksperimentalno utvrđenih vrednosti ugiba u preseccima I i II, za nosače od samougrađujućeg betona sa recikliranim agregatom (SCC R 50, SCC R 100) u odnosu na nosač izrađen od prirodnog agregata (SCC).



Slika 6 - Veza opterećenje-ugib, preseccima I i II kontinualnih nosača armiranih sa $\pm 2R\text{Ø}10$ mm

Na Slici 6 sa leve strane u odnosu na ordinatnu osu prikazane su dobijene vrednosti ugiba preseka I, dok su sa desne strane prikazane dobijene vrednosti ugiba preseka II, u funkciji opterećenja. Uočavaju se karakteristične faze kroz koje nosač prolazi tokom nanošenja probnog opterećenja: 1) do pojave prvih prslina, 2) od pojave prvih prslina do pojave tečenja u zategnutoj čeličnoj armaturi i 3) od pojave tečenja u čeličnoj armaturi do loma nosača. Za sve nosače se u elastičnoj fazi uočava gotovo identično ponašanje, pre formiranja prvih prslina.



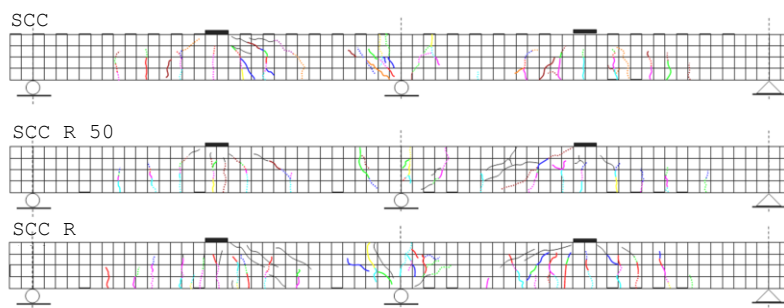
Slika 7 - Grafički prikaz elastičnih linija kontinualnih grednih nosača različitih vrsta betona armiranih sa $\pm 2R\text{Ø}10$ mm

Na osnovu sprovedene analize zaključuje se da su kontinualni gredni nosači (SCC R 50, SCC R 100) imali veće ugibe u odnosu na uporednu (etalon - SCC) gredu, što je prikazano na Slici 7. Ovo se, između ostalog, može tumačiti kao posledica različitog ponašanja u zoni zatezanja u zavisnosti od sastava betona, odnosno vrste agregata. Imajući u vidu da su moduli elastičnosti i slika prslina za sve tri vrste betona vrlo slični, razlog za uočene razlike u ugibima treba tražiti u sadejstvu zategnutog betona između prslina.

Tabela 2. Rezultati ispitivanja kontinualnih grednih nosača

Tip betona	Opterećenje koje dovodi do pojave prve prsline		Opterećenje koje dovodi do pojave tečenja u zategnutoj armaturi		Granično opterećenje $F_{L,NL}$	Indeks duktilnosti
	nad srednjim osloncem [kN]	u polju [kN]	nad srednjim osloncem [kN]	u polju [kN]		
SCC	/	46,26	76	70	166,5	8,9
SCC R 50	18,75	17,2	18	56	163,8	5,4
SCC R 100	28,26	32,8	50	58	167,8	7,3

U Tabeli 2. prikazani su rezultati opterećenja koje dovodi do pojave prve prsline i tečenja u zategnutoj armaturi kako u polju, tako i nad srednjim osloncem. Opterećenje koje dovodi do loma nosača sa prirodnim agregatom, kao i nosača sa recikliranim agregatom gotovo da je identično. Duktilnost nosača prikazana u Tabeli 2. izražena je u funkciji indeksa duktilnosti i ukazuje i da kod nosača sa prirodnim i sa recikliranim agregatom dolazi do duktilnog loma i formiranja mehanizma loma.



Slika 8 - Mapa prsline kontinualnih grednih nosača od različitih vrsta betona armiranih sa $\pm 2R\text{Ø}10\text{ mm}$

Praćenje formiranja, razvoja i širine prsline u betonu vršeno je vizuelno, iscrtavanjem na prethodno izšrafiranoj bočnoj površini ispitivanih grednih nosača, Slika 8. Kod grednih nosača izrađenih od samougrađujućeg betona sa recikliranim agregatom prve prsline javile su se pri niskom intenzitetu opterećenja, što nije bio slučaj kod nosača izrađenih od samougrađujućeg betona sa prirodnim agregatom. Veće otvaranje prsline uočeno je kod nosača izrađenih od samougrađujućeg betona sa recikliranim agregatom.

4 ZAKLJUČAK

Na osnovu analize eksperimentalno dobijenih rezultata ispitivanja mogu da se izvedu dve grupe zaključaka. Prva grupa zaključaka odnosi se na fizičko–mehanička svojstva samougrađujućeg betona spravljenog sa prirodnim i recikliranim agregatom, kako u svežem tako i u očvrslom stanju. Drugu grupu čine zaključci koji se odnose na graničnu nosivost kontinualnih nosača spravljenih od samougrađujućeg betona sa prirodnim i recikliranim agregatom, izloženih dejstvu kratkotrajnog opterećenja.

Ispitivanjem sveže betonske mešavine uočava se da karakteristike samougrađujućeg betona sa recikliranim agregatom zadovoljavaju uslove koji su definisani u EFNARC kao i EN 206-9:2010, odnosno primena agregata od recikliranog betona nije ograničavajući faktor za proizvodnju samougrađujućih betona. Mehaničke karakteristike betona sa recikliranim agregatom u očvrslom stanju ukazuju na pad čvrstoće pri pritisku i minimalna odstupanja čvrstoće betona pri zatezanju i modula elastičnosti kod betona sa recikliranim agregatom u odnosu na beton sa prirodnim agregatom.

Duktilnost kontinualnih nosača izrađenih od samougrađujućeg betona sa prirodnim agregatom, kao i onih sa recikliranim agregatom, iskazana je preko indeksa duktilnosti (ID) i ima zadovoljavajuće vrednosti. Kod svih nosača došlo je do vidljive iscrpljenosti nosača i duktilnog loma. Uočeno je češće lokalno mrvljenje betona ispod pločica za unos opterećenja, kao i u regionu srednjeg oslonca, kod nosača koji su izrađeni od samougrađujućeg betona sa recikliranim agregatom nego kod nosaca sa prirodnim agregatom. Po pitanju granične nosivosti na savijanje, kod nosača sa procentom armiranja od 0,65 %, svi podaci i analize ukazuju da nema prevelike razlike između nosača sa prirodnim i recikliranim agregatom. Razlike u ugibima kontinualnih nosača sa prirodnim i recikliranim agregatom pri eksploatacionom opterećenju su neznatne, dok pri graničnom opterećenju dolazi do značajnije razlike u vrednostima ugiba. Mapa prslina menja se u skladu sa količinom recikliranog agregata u nosaču, tako da sa povećanjem količine recikliranog agregata broj prslina raste, a njihova širina na nosaču se povećava. Intenzitet opterećenja koji dovodi do pojave prve prslina na nosaču, za isti procenat armiranja nosača, u funkciji je količine recikliranog agregata, pa se prve prsline javljaju pri veoma niskom opterećenju kod nosača sa recikliranim agregatom u odnosu na nosače sa prirodnim agregatom.

Prezentovane analize i zaključci pokazuju da se industrijski nusproizvodi, kao i agregat dobijen od otpadnog betona, mogu primeniti za spravljanje konstrukcijskih elemenata od samougrađujućeg betona. Izrada konstrukcijskih armiranobetonskih nosača sa delimičnom zamenom krupnog prirodnog agregata, recikliranim agregatom je sa stanovišta zaštite životne sredine opravdana, a u konstrukcijskom smislu izvodljiva.

Svi izvedeni zaključci zasnovani su na analizi rezultata sprovedenog sopstvenog eksperimentalnog istraživanja. Za izvođenje opštih zaključaka neophodno je sprovesti opsežna eksperimentalna ispitivanja i formirati bazu eksperimentalnih podataka sa ispitivanjima u laboratorijskim i terenskim uslovima linijskih nosača izloženih kako čistom savijanju tako i složenom naprezanju.

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PRIMENA METODE ZRELOSTI ZA ODREĐIVANJE ČVRSTOĆE BETONA U KONSTRUKCIJI

Rezime:

S-95

Određivanje trenutka dostizanja željene čvrstoće pri pritisku betona u konstrukciji predstavlja složen zadatak, jer ovo svojstvo zavisi od brojnih faktora (kao što su: sastav betona, način nege, uslovi okoline, itd.). U radu je dat kratak istorijat razvoja koncepta zrelosti betona, sa osvrtom na mogućnost njegove primene u određivanju priraštaja čvrstoće betona tokom vremena u realnim uslovima. Prikazani su i analizirani rezultati merenja koja su obavljena pomoću savremenog "Doka Concremote" sistema, a koji govore u korist primene ovakvog koncepta u praksi.

Ključne reči: zrelost betona, čvrstoća pri pritisku, "Doka Concremote" sistem

APPLICATION OF MATURITY METHOD IN DEFINING THE CONCRETE STRENGTH IN SITU

Summary:

Defining the moment of achieving the desired concrete compressive strength represents a very complex task, because it depends on many factors (such as: concrete composition, curing method, environmental conditions, etc.). Short historical overview of maturity method development has been presented in the paper, with its possible application in determining the compressive strength increase of concrete when subjected to real in situ conditions. The results of conducted measurements using the contemporary "Doka Concremote" system, which showed positive effects, were also presented and analyzed.

Key words: maturity of concrete, compressive strength, "Doka Concremote" system

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1 UVOD

U cilju povećanja produktivnosti i brzine napredovanja radova u oblasti projektovanja, a posebno izgradnje objekata od betona, teži se ka novim i ubrzanim tehnikama i načinima gradnje. Prema Pravilniku za beton i armirani beton PBAB'87[1], oplata se može ukloniti sa vertikalnih elemenata konstrukcije nakon dostizanja 30% od projektovane čvrstoće, dok se kod horizontalnih elemenata oplata može ukloniti nakon dostizanja 70% od projektovane čvrstoće pri pritisku (marke betona). Za određivanje stvarne vrednosti dostignute čvrstoće pri pritisku u konstrukciji preporučuje se uzimanje kontrolnih uzoraka koji će biti negovani na isti način kao i posmatrani elementi. Na osnovu ispitane čvrstoće pri pritisku uzetih uzoraka određuje se da li su posmatrani elementi dostigli potrebnu vrednost čvrstoće. Ipak, način procene trenutka kada je potrebno izvršiti ispitivanje kontrolnih uzoraka nije precizno definisan, pa se u najvećem broju slučajeva inženjeri oslanjaju na prethodna iskustva. Svakodnevno ispitivanje od trenutka ugradnje zahtevalo bi preveliki broj uzoraka po elementu konstrukcije, posebno imajući u vidu da je uzorke potrebno uzimati i u cilju redovne kontrole kvaliteta betona i dokazivanja postignute marke betona. Pogrešna procena, u smislu ispitivanja uzoraka kada je zahtevana čvrstoća pri pritisku prekoračena, dovodi do kašnjenja u izvođenju radova i smanjivanja efikasnosti procesa izgradnje.

Pored toga, poznavanje čvrstoće betona in situ može biti važno i u slučajevima određivanja: trenutka nanošenja opterećenja, vremena prednaprezanja elementa, potrebnog perioda negovanja betona, dostizanja manipulativne čvrstoće kod prefabrikovanih elemenata, itd.

1.1 FUNKCIJE ZRELOSTI

Jedan od rasprostranjenih načina prevazilaženja pomenutih nedostataka je primena koncepta zrelosti u određivanju ranih čvrstoća pri pritisku betona u konstrukciji. Zrelost betona je veličina (faktor) koji uzima u obzir starost i istoriju temperature betonskog elementa. Promena temperature betona je posledica procesa hidratacije cementa i uticaja temperature okoline. Razvoj koncepta zrelosti započet je nakon što je primećeno da betonski elementi negovani na različitim temperaturama dostižu različite vrednosti čvrstoće pri pritisku, posebno pri malim starostima.

Jedna od prvih i najpoznatijih funkcija zrelosti je Nurse-Saul-ova funkcija[2]:

$$M(T) = \sum (T_a - T_0) \times \Delta t \quad (1)$$

gde su:

Δt - vremenski interval (sati ili dani),

T_a - prosečna temperatura betona u toku vremenskog intervala ΔT ($^{\circ}\text{C}$),

T_0 - kritična temperatura ($^{\circ}\text{C}$).

Pod kritičnom temperaturom podrazumeva se temperatura ispod koje se proces hidratacije cementa u potpunosti obustavlja. Saulov princip može se izraziti na sledeći način: "Beton istog sastava i iste zrelosti (sračunate kao proizvod vremena i temperature), ima približno istu čvrstoću bez obzira na kombinaciju temperature i vremena za koju je dostignuta pomenuta zrelost". Ovaj princip se zbog svoje jednostavnosti i danas primenjuje, iako njegova osnovna pretpostavka - da je početna brzina priraštaja čvrstoće linearna funkcija temperature u mnogim

slučajevima nije ispunjena. Još jedan nedostatak Nurse-Saul-ove funkcije, kao i kasnije razvijene Arrhenius-ove funkcije, je taj što nisu uzimale u obzir sastav betonske mešavine, a pre svega vrstu korišćenog cementa.

Bresson i Papadakis (1973) predstavili su metod zrelosti koji uključuje i faktor koji uzima u obzir razlike u odvijanju hidratacije cementa na različitim temperaturama. Prema ovom metodu, na višim temperaturama zabeležena je veća osetljivost (reaktivnost) u odnosu na niže temperature. Ovakav pristup nazvan je "Weighed maturity" ("težinska zrelost"). De Vree je unapredio ovaj pristup uzevši u obzir i različito ponašanje cementa i na temperaturama nižim od 20°C [2].

Vrednost zrelosti prema De Vree-u računa se kao:

$$M(T) = \sum t \times T \times C^n \quad (2)$$

gde su:

M(T) - zrelost (°C h),

T - širina temperaturnog segmenta (u većini slučajeva iznosi 5°C),

t - vreme u satima za koje je temperatura betona prevazilazila donju granicu segmenta temperature širine 5°C, počevši od -10 °C,

C - parametar koji zavisi od vrste cementa,

n - eksponent pridružen odgovarajućem temperaturnom segmentu.

Kao što je ranije navedeno, C predstavlja parametar koji zavisi od vrste cementa i eksperimentalno se određuje. Vrednosti koeficijenta n za različite temperaturne segmente prikazane su u tabeli 1.

Tabela 1 – Vrednosti koeficijenta n za temperaturne segmente prema metodi De Vree-a

T. s. (°C)	-10/-5	-5/0	0/5	5/10	10/15	15/20	20/25	25/30	30/35	35/40
n	-2,0	-1,5	-1,0	-0,5	0	0,5	1,0	1,5	2,0	2,5
T. s. (°C)	40/45	45/50	50/55	55/60	60/65	65/70	70/75	75/80	80/85	85/90
n	3,0	3,5	4,0	4,5	5,0	5,5	6,0	6,5	7,0	7,5

Radi povećanja efikasnosti proračuna, umesto prikaznih vrednosti predložena je kontinualna zavisnost:

$$n = 0,1 \times T - 1,245 \quad (3)$$

Nakon zamene diskretnih vrednosti n izrazom (3) i integracije izraza (2), dobija se sledeća funkcija za priraštaj zrelosti tokom 1h:

$$M(T) = \frac{10(C^{(0,1T-1,245)} - C^{(-2,245)})}{\ln C} \quad (4)$$

gde je: T - osrednjena vrednost temperature u °C [3].

1.2 "DOKA CONCREMOTE" SISTEM

"Doka Concremote" sistem omogućava približno određivanje čvrstoće pri pritisku betona primenom funkcije zrelosti prema De Vree-u. Na ovaj način, moguće je odrediti dijagram priraštaja čvrstoće betona ugrađenog u različite konstruktivne elemente, kao što su:

horizontalni i vertikalni AB elementi u visokogradnji ali i mostovi, tuneli ili brane u niskogradnji [4]. Sistem obuhvata tri faze [5]:

1. kalibraciju merenja za određenu vrstu betona,
2. merenje vremena i temperature na mestu ugradnje betona uz pomoć senzora,
3. obradu dobijenih podataka.

Prva faza se sastoji iz uzimanja uzoraka ispitivanog betona na fabrici betona (ili na gradilištu) i njihovog smeštanja u posebne kalibracione kutije (slika 1). Na svakom od uzoraka ispituje se čvrstoća pri pritisku pri određenoj starosti, dok se uz pomoć senzora ugrađenih u kutijama kontinuirano prati razvoj temperature u uzorku i u njegovoj neposrednoj okolini. Na osnovu dobijenih podataka, formira se funkcija zrelosti (formula 2), a zatim i zavisnost čvrstoće pri pritisku od zrelosti za ispitivani beton [6]. Dobijena zavisnost se zatim pomera za određeni faktor sigurnosti, koji se određuje prema izrazu [7]:

$$\delta = s \times \sigma_{ij} \quad (5)$$

gde su:

- s - parametar koji zavisi od svrhe ispitivanja (s=0 za određivanje trajanja nege betona, s=1 za određivanje trenutka uklanjanja oplata i s=1,5 za određivanje trenutka dostizanja dovoljne čvrstoće za prednaprezanje ispitivanog elementa),
- σ_{ij} - standardna devijacija (vrednost koja se može dobiti od proizvođača ispitivanog betona).



Slika 1. Kalibraciona kutija "Doka Concremate" sistema

Druga faza obuhvata postavljanje senzora na elemente konstrukcije prilikom ugradnje betona. Ovi senzori, takođe, kontinuirano mere temperaturu u betonu i okolini. U zavisnosti od elementa koji se ispituje bira se tip senzora koji se ugrađuje: Concremate senzor za ploče ili Concremate senzor u vidu kabla. Senzori u vidu kabla se mogu koristiti kao senzori za zidove koji se mogu višekратно upotrebljavati, ili kao senzori u vidu kabla sa 3 merna mesta i senzori u vidu kabla sa 1 mernim mestom, koji ostaju trajno zarobljeni u betonu nakon ugrađivanja.

Treća faza obuhvata obradu merenih podataka i njihovo poređenje sa rezultatima kalibracionog merenja. Proračunavanjem postignute zrelosti betona, na osnovu izmerenih vrednosti temperature u realnom vremenu, a zatim i određivanjem čvrstoće pri pritisku, preko funkcije zavisnosti čvrstoće pri pritisku od zrelosti betona dobijene u procesu kalibracije, prati se razvoj čvrstoće pri pritisku u ugrađenom betonskom elementu.

2 REZULTATI I ANALIZA SPROVEDENIH ISPITIVANJA

Ispitivanje mogućnosti primene koncepta zrelosti u određivanju razvoja čvrstoće pri pritisku betona ugrađenog u elemente konstrukcije pomoću "Doka Concremote" sistema izvršeno je u saradnji sa fabrikom betona "CRH" u Krnjači i firmom "M Enterijer Gradnja". Ispitivan je trofrakcijski pumpani beton, projektovane marke MB 30, ugrađen u ploče na stambenom objektu "Sunnyville" u Višnjici (izvođač radova: "M Enterijer Gradnja").

U sastav ovog betona ulazili su: prirodni agregat separisan na tri standardne frakcije, cement oznake CEM II/A-L 42,5R proizvođača "CRH" Novi Popovac, voda iz vodovoda i superplastifikator pod nazivom BEVETOL-SPL, proizvođača "Isomat".

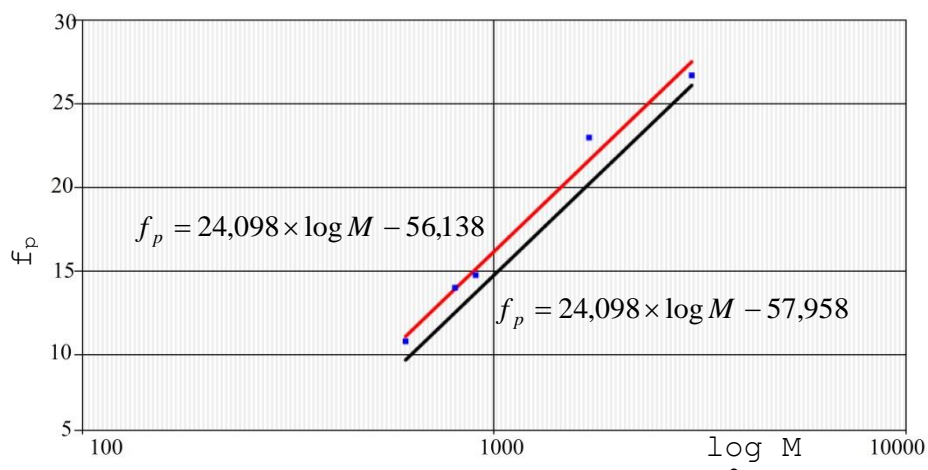
2.1 PROCES KALIBRACIJE SISTEMA

U saradnji sa pomenutom fabrikom betona i centralnom laboratorijom u Holandiji, ovlašćenom da vrši obradu podataka za sistem "Doka Concremote", urađen je proces kalibracije za ispitivani beton projektovane marke MB 30 i napred navedenog sastava. Prema preporuci ovlašćene laboratorije, za upotrebljeni cement usvojena je C vrednost od 1,25.

Šest uzoraka oblika kocke ivice 15 cm uzeto je na fabrici betona (21.04.2016. u 14:00 h) i smešteno u dve kalibracione kutije, u kojima su negovani u kontrolisanim ambijentalnim uslovima do trenutka ispitivanja. Vreme ispitivanja svakog od uzoraka određeno je u dogovoru sa centralnom laboratorijom, a na osnovu unapred postavljenog cilja. Kao konačni cilj ispitivanja, definisano je određivanje trenutka kada će beton u konstrukciji dostići vrednost od 70% u odnosu na projektovanu marku betona, tj. u konkretnom slučaju: $0,7 \times 30 = 21$ MPa (vrednost pri kojoj se prema Pravilniku BAB'87 može ukloniti oplata sa horizontalnih delova elemenata konstrukcije [1]). Na osnovu kontinualnog merenja temperature betona, izvršen je proračun veličina zrelosti tokom vremena za svaki od uzoraka. Pri određenim starostima betona ispitivana je njegova čvrstoća pri pritisku i, konačno, određena je kalibraciona kriva zavisnosti čvrstoće pri pritisku od zrelosti betona, a zatim i koeficijent sigurnosti za koji se kriva pomera ($\delta = 1,82$). Rezultati ovog ispitivanja prikazani su u tabeli 2 i na slici 2. Zbog izrazito niske vrednosti čvrstoće pri pritisku (0,5 MPa), odgovarajuća zrelost dobijena na uzorku broj 041-1 nije uzeta u obzir prilikom određivanja kalibracione krive.

Tabela 2 – Rezultati ispitivanja kontrolnih uzoraka

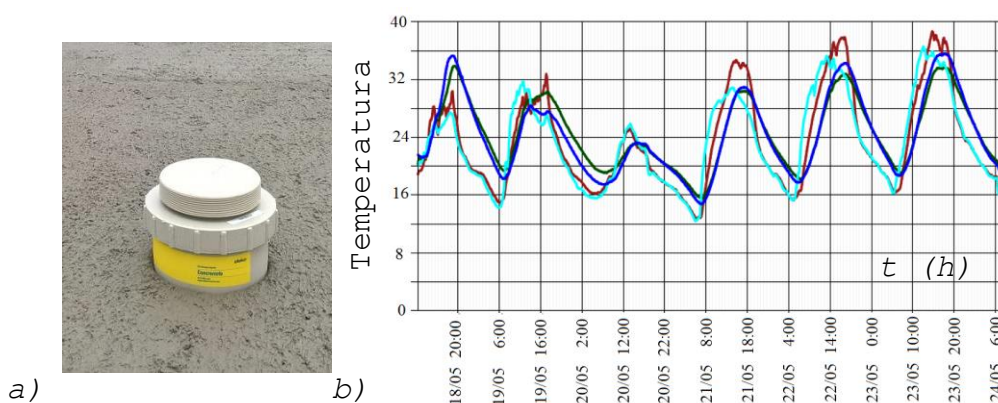
Uzorak broj	Datum ispitiv.	Vreme ispitiv.	Starost uzorka (h)	Masa (kg)	Zaprem.masa γ (kg/m ³)	Sila P (kN)	Čvrstoća pri pritisku f_p (MPa)	Zrelost M (°Ch)
041-1	21.04.16.	22:20	8,3	7,900	2341	10,7	0,5	223,2
041-2	22.04.16.	8:30	18,5	7,905	2342	243,8	10,8	610,7
041-3	22.04.16.	13:00	23,0	7,960	2359	316,1	14,0	805,0
246-1	22.04.16.	16:20	26,3	7,855	2327	332,8	14,8	903,0
246-2	23.04.16.	18:20	52,3	7,940	2353	517,6	23,0	1782,6
246-3	25.04.16.	16:20	74,3	7,965	2360	601,7	26,7	2885,1



Slika 2. Korelaciona zavisnost između čvrstoće pri pritisku (f_p) i logaritma zrelosti ispitivanog betona ($\log M$) dobijena proračunom, a zatim pomerenjena za koeficijent sigurnosti

2.2 POSTAVLJANJE SENZORA I OBRADA REZULTATA MERENJA

Prilikom betoniranja armiranobetonske ploče međuspratne konstrukcije na objektu u naselju "Sunnyville", u svež beton su postavljena dva senzora (slika 3a). Osim temperature betona, senzori su kontinuirano merili i temperaturu okoline. Od trenutka postavljanja, ovi senzori su vršili slanje podataka o temperaturi betona, kao i temperaturi okoline, u centralnu laboratoriju - preko mrežnog signala [9]. Dijagram izmerenih vrednosti temperatura betona i okoline (sa po dva senzora), u funkciji vremena prikazan je na slici 3b.



Slika 3. Postavljanje senzora a) i zapis izmerenih temperatura betona i okoline (b)

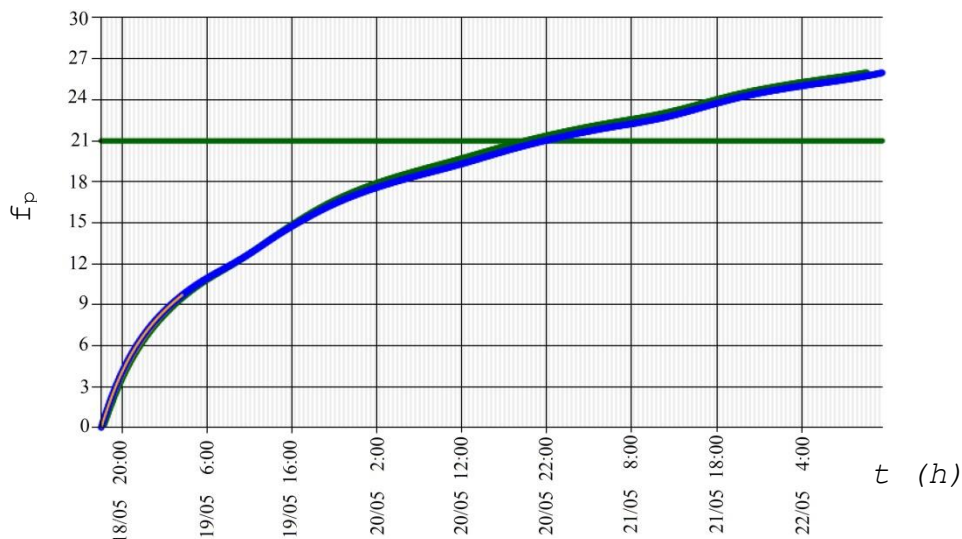
U principu, senzori se postavljaju tako da pokrivaju najkritičnije tačke (u pogledu naprežanja) i najnepovoljnije tačke u pogledu priraštaja čvrstoće [4]. Za potrebe ovog ispitivanja, istovremeno sa postavljanjem dva senzora u ploči, uzeti su i uzorci betona oblika

kočke ivice 15 cm, koji su negovani na isti način kao beton u konstrukciji do trenutka ispitivanja. Rezultati dobijeni analizom podataka dobijenih od senzora i prethodno formirane korelacije, upoređeni su sa vrednostima čvrstoće pri pritisku izmerene na uzetim uzorcima (tabela 3). U koloni 4 dati su prosečni rezultati čvrstoće pri pritisku - f_p (dobijeni na dva ispitana uzorka), dok su u koloni 5 date vrednosti dostignute zrelosti betona u ploči u vreme ispitivanja čvrstoće. U koloni 6 date su računске vrednosti čvrstoće pri pritisku - $f_{p,rač}$, određene na osnovu prethodno formirane korelacije (slika 2).

Tabela 3. Uporedni rezultati ispitivanja čvrstoće pri pritisku (f_p) i čvrstoće dobijene proračunom ($f_{p,rač}$)

Datum ispitivanja	Vreme ispitivanja	Datum spravljanja uzorka	f_p (MPa)	M (°Cdani)	$f_{p,rač}$ (MPa)
(1)	(2)	(3)	(4)	(5)	(6)
20.05.2016.	14:20	18.05.2016.	21,5	1680,8	19,8
24.05.2016.	8:20	18.05.2016.	29,2	4532,5	30,2

Uz pomoć prethodno opisanih merenja i analiza, određena je odgovarajuća veličina čvrstoće pri pritisku betona za svaku vrednost zrelosti. Zatim je generisan dijagram razvoja čvrstoće pri pritisku betona u toku vremena i određen trenutak kada je beton dostigao ciljane vrednosti čvrstoće od 21 MPa, kao što je prikazano na slici 4.



Slika 4. Razvoj čvrstoće pri pritisku betona (f_p) u ispitivanom elementu konstrukcije tokom vremena (t)

3 ZAKLJUČAK

Razvojem koncepta zrelosti i povezivanjem ovog parametra sa vrednošću čvrstoće pri pritisku, dobijen je sigurniji način za praćenje razvoja čvrstoće betona ugrađenog u konstrukciju. Funkcija De Vree-a u najvećoj meri prevazilazi nedostatke ranijih metoda, uzimajući u obzir sastav betona i različito ponašanje cementa pri različitim temperaturama (koncept "težinske zrelosti"). Prednosti ovakvog koncepta su prepoznate u svetskim okvirima, što je dovelo do razvika brojnih standarda koji propisuju upotrebu metoda za određivanje zrelosti (na primer: NEN 5970:2001 nl, ASTM C1074–11, DIN 1045-3, BS 1881-201, Probeton RN010, TS 13508, NF-P 18-406).

U poređnim ispitivanjem mogućnosti primene koncepta zrelosti betona prema De Vree-u (korišćenjem savremenog "Doka Concremote" sistema) i kontrolnih betonskih uzorka, došlo se do zaključka da je moguće koristiti navedeni sistem za određivanje ranih čvrstoća betona u konstrukciji, sa zadovoljavajućim stepenom pouzdanosti. Međutim, ovde treba naglasiti da se prilikom primene ovog koncepta podrazumeva da ugrađivani beton treba da bude ujednačenog sastava i da se nega svih betonskih elemenata vrši na isti način. Naime, ukoliko dođe do bilo kakve promene u sastavu betona (u smislu vrste i/ili količine upotrebljenih komponentnih materijala), to obavezno zahteva ponavljanje procesa kalibracije.

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Jelena Dragaš¹, Snežana Marinković²

NEGA BETONA SA VELIKIM SADRŽAJEM LETEĆEG PEPELA

Rezime:

S-96

Poslednjih decenija došlo je do razvoja betona sa velikim sadržajem letećeg pepela, koji imaju sporiji prirast čvrstoće pri pritisku i niže rane čvrstoće betona. Kako bi se ovaj problem prevazišao potrebno je odabrati odgovarajuću vrstu nege betona. Voda je ključan faktor za efikasno odvijanje hidratacije i pucolanske reakcije ove vrste betona pa je standardna nega pri relativnoj vlažnosti od 95% najčešće najbolji izbor. Više temperature nege pospešuju odvijanje hemijskih reakcija u betonu pa se zaparivanje često koristi kao nega ovih betona. Kako bi se postigle dovoljne rane i kasne čvrstoće potrebno je pronaći balans između dužine nege uzoraka u standardnim uslovima pre početka zaparivanja i dužine trajanja ciklusa zaparivanja.

Ključne reči: leteći pepeo, k-vrednost, nega betona, zaparivanje

CURING OF HIGH-VOLUME FLY ASH CONCRETE

Summary:

High-volume fly ash concrete has been developed for the last few decades. One of the main disadvantages of this type of concrete is slow compressive strength development and low early-age strengths. In order to overcome this adequate curing regime should be chosen. Water is the main factor influencing the hydration and pozzolanic reaction in concrete so water curing is usually the best choice. Higher curing temperatures have a positive effect on the chemical reactions taking place in concrete so steam curing is usually used for high-volume fly ash concrete. In order to obtain good early and later compressive strengths, a balance between standard water curing and steam curing duration is needed.

Key words: fly ash, k-value, concrete curing, steam curing

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1 UVODNI DEO

Leteći pepeo je nusprodukt sagorevanja uglja koji se poslednjih decenija sve više koristi u betonima kao delimična zamena cementa ili sitnog agregata. Prvobitna motivacija za upotrebu letećeg pepela (tridesetih godina prošlog veka) je bila smanjenje toplote hidratacije betona prilikom izgradnje brana. Posmatrajući ekološki i ekonomski aspekt proizvodnje betona, smanjenje količine cementa zamenom jednog njegovog dela letećim pepelom dovodi do manjeg štetnog uticaja proizvodnje betona na životnu sredinu i njegove manje cene.

Upotreba letećeg pepela kao dodatka betonima definisana je u mnogim standardima. Prema standardu EN 206-1:2000 [1] leteći pepeo treba da zadovolji fizičke i hemijske karakteristike definisane u EN 450-1:2012 [2] kako bi mogao da se primenjuje u betonima. Nije eksplicitno definisana maksimalna količina letećeg pepela koja se može koristiti u betonima ali standard EN 206-1:2000 [1] definiše maksimalnu količinu letećeg pepela u odnosu na masu cementa koji može da se uzme u obzir u okviru koncepta k-vrednosti i ograničava je na vrednost $\text{leteći pepeo/cement} = 0.33$.

Koncept k-vrednosti definiše potencijal letećeg pepela kao vezivnog materijala u odnosu na količinu i vrstu cementa i produkata hidratacije koji su neophodni za odvijanje pucolanske reakcije kroz zamenu vodocementnog faktora (v/c) sa faktorom koji uzima u obzir i leteći pepeo: $v/(\text{cement} + k \times \text{leteći pepeo})$. K-vrednost uzima u obzir različitu vrstu cementa a količina letećeg pepela je bez obzira na ukupnu količinu upotrebljenog pepela ograničena na maksimalnih 33% u odnosu na masu cementa.

Koncept k-vrednosti koriguje i minimalnu količinu cementa koja može biti upotrebljena u betonima sa letećim pepelom kako bi se postigla odgovarajuća trajnost ovih betona: nova minimalna masa cementa za određenu klasu izloženosti se računa kao $k \times (\text{minimalna masa cementa} - 200) \text{ kg/m}^3$. Kao rezultat ovoga, procenat zamene cementa letećim pepelima je ograničen na 25–30% za betone kod kojih se neće eksperimentalno dokazivati trajnost betona. Korišćenje većih količina letećeg pepela u betonima je moguće uz dokazivanje da su karakteristike tih betona iste kao referentni cementni beton tj. da se primenjuje koncept ekvivalentnih performansi betona (*equivalent concrete performance concept*) za potrebnu klasu izloženosti betona.

Poslednjih decenija došlo je do razvoja betona sa velikim sadržajem letećeg pepela (*high-volume fly ash concrete*) koji su prvi put ispitivani u CANMET-u (Canada Centre for Mineral and Energy Technology) osamdesetih godina prošlog veka [3]. Betoni sa velikim sadržajem letećeg pepela imaju više od 50% letećeg pepela klase F ili C [4] u ukupnoj količini vezivnog materijala (cement i leteći pepeo). I pored toga što upotreba ovih betona nije u skladu sa konceptom k-vrednosti koje propisuje EN 206-1:2000 [1] pravilnim projektovanjem mešavine betona mogu se dobiti betoni dobrih mehaničkih karakteristika [5, 6, 7, 8].

1.1 UPOTREBA BETONA SA VELIKIM SADRŽAJEM LETEĆEG PEPELA U SRBIJI

Srbija, kao i većina zemalja koje električnu energiju dobijaju prvenstveno iz termoelektrana, ima problem sa velikim količinama deponovanog pepela. Trenutno se u svim termoelektranama u Srbiji proizvodi oko šest miliona tona letećeg pepela godišnje, a količina deponovanog pepela iznosi preko 200 miliona tona. Imajući ovo u vidu ne čudi činjenica da se

sve više istraživanja moguće primene letećeg pepela sprovodi i u Srbiji. Istraživanje primene letećeg pepela iz termoelektrane Nikola Tesla B u betonima sa velikim sadržajem letećeg pepela sprovodi se na Građevinskom fakultetu Univerziteta u Beogradu. Rezultati dosadašnjih ispitivanja [8] pokazuju da je moguće napraviti betone sa 200 kg/m³ cementa i 200–400 kg/m³ letećeg pepela koji imaju zadovoljavajuću ugradljivost, rane i kasne čvrstoće pri pritisku, čvrstoću pri zatezanju cepanjem i modul elastičnosti. Čvrstoće pri pritisku ovih betona na 7 dana su 22.2–27.1 MPa, na 28 dana 34.2–42.0 MPa a na 90 dana 42.3–54.2 MPa. Čvrstoća pri zatezanju na 28 dana je u opsegu od 2.0 MPa do 3.7 MPa, a modul elastičnosti na 28 dana 31.3–33.2 GPa. Rezultati ovog ispitivanja pokazuju da se betoni sa velikim sadržajem letećeg pepela iz termoelektrane Nikola Tesla B mogu koristiti kao konstruktivni betoni.

Jedan od ograničavajućih faktora moguće primene betona sa letećim pepelom je i pitanje njihove radioaktivnosti koja potiče iz prirodne radioaktivnosti uglja čijim se sagorevanjem dobija leteći pepeo. Radioaktivnost građevinskih materijala se ispituje kroz I-indeks koji uzima u obzir aktivnost prirodnih radionuklida ²²⁶Ra, ²³²Th i ⁴⁰K i ograničen je na maksimalnu vrednost od 1.0. Rezultati ispitivanja radioloških karakteristika betona sa velikim sadržajem letećeg pepela iz termoelektrane Nikola Tesla B [9] pokazuju da je I-indeks svih ispitanih betona (masa cementa 200 kg/m³, masa letećeg pepela 200–400 kg/m³) značajno manji od maksimalne propisane vrednosti i da se oni mogu bezbedno koristiti u građevinarstvu.

2 NEGA BETONA

Kako bi dobili beton dobrog kvaliteta neophodno je, nakon spravljanja po određenoj recepturi i ugrađivanja, negovati taj beton na odgovarajući način tokom prvog perioda njegovog očvršćavanja. Nega betona podrazumeva sve postupke usmerene na što uspešniju hidrataciju cementa a koji se sastoje u zaštiti od spoljašnjih nepovoljnih uticaja, održavanju povoljne temperature i vlažnosti betona. Beton mora biti održavan u stanju vlažnosti, zasićen ili skoro zasićen vodom zato što se hidratacija cementa može odvijati samo u zasićenim kapilarnim porama. Od suštinskog je značaja sprečiti isparavanje vode iz kapilarnih pora a voda koja se gubi samoisušivanjem mora se nadokanditi vodom dodatom spolja. Betoni sa običnim portland cementnom se neguju bar sedam dana.

Vrsta nege koja se primenjuje kod betona sa velikim sadržajem letećeg pepela ima mnogo veći uticaj na mehaničke karakteristike i trajnost betona nego što je to slučaj kod običnih cementnih betona. Tokom pucolanske reakcije leteći pepeo (amorfni silicijum-dioksid iz pepela) hemijski reaguje sa kalcijum-hidroksidom na običnim temperaturama i stvara jedinjenja koja imaju vezivna svojstva. Ova reakcija je znatno sporija od hidratacije cementa, pa je kod betona sa većom količinom letećeg pepela poželjno produžiti trajanje nege betona kako bi pucolanska reakcija bila što uspešnija. Pucolanska reakcija je sekundarna reakcija koja se dešava nakon početka hidratacije cementa i stvaranja dovoljno kalcijum-hidroksida pa se voda koja se nalazi u mešavini betona troši prvo tokom reakcije rane hidratacije cementa.

Ako se uzorak betona ne nalazi u sredini koja ima dovoljnu vlažnost ili uzorak nije dobro zaštićen, može doći do isparavanja vode iz betona usled suve sredine u kojoj se nalazi. Kako bi se to sprečilo i omogućilo da se reakcija cementa i letećeg pepela sa vodom nesmetano odvija, potrebno je produžiti vreme nege betona sa velikim sadržajem letećeg pepela. Ovi betoni su najčešće napravljeni sa niskim vodovezivnim faktorom kako bi se dostigle željene čvrstoće betona. Proces samoisušivanja se odvija dosta brže u betonima sa niskim vodovezivnim faktorom [10] pa zbog ovoga treba specijalno voditi računa o negovanju ovih betona.

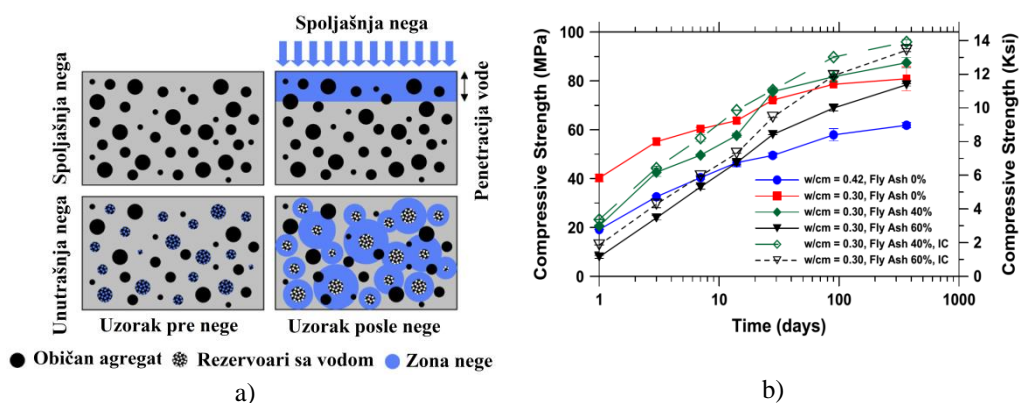
Postupci nege betona u praksi su raznovrsni i zavise od uslova na gradilištu, vrste betona i konstrukcije, tehnologije građenja i dostupne tehnologije za negu betona. Nega obično podrazumeva pokrivanje betonskih elemenata raznim materijalima ili zaptivajućim membranama kako bi se sprečilo isparavanje vode. Kako bi se nadokadila izgubljena voda betonski elementi se često polivaju vodom kako bi se održali u vlažnom stanju. Nega betona nakon ugrađivanja je bitna i sa aspekta smanjenja prslina usled skupljanja betona i bržeg dostizanja potrebnih ranih čvrstoća.

Povećanje temeprature negovanja ubrzava hemijsku reakciju hidratacije i povoljno utiče na rane čvrstoće betona. Ovo je posebno bitno kod betona sa velikim sadržajem letećeg pepela koji imaju sporiji prirast čvrstoće u poređenju sa običnim cementnim betonima. Međutim, ako je temperatura iznad određene granice za vreme vezivanja, uticaj na čvrstoću nakon sedam dana može biti negativan. To se objašnjava brzom ranom hidratacijom tokom koje se formiraju proizvodi slabije fizičke strukture i veće poroznosti. Na ovaj način može doći do neujednačenog rasporeda proizvoda hidratacije u cementnoj pasti [10].

Sličan efekat nege betona se može postići zaparivanjem betona. Ako se zaparivanje vrši pri normalnom atmosferskom pritisku i temperaturi do 100°C ovaj postupak se može smatrati specijalnim postupkom nege u vlažnom stanju. Ovaj postupak se obično primenjuje kod prefabrikovanih betona kako bi se što pre dostigle dovoljno velike početne čvrstoće i oslobodila oplata. Bitni parametri o kojima treba voditi računa su temepratura zaparivanja i ciklus tokom kog se dostižu potrebne temperature zaparivanja. Tempo kojim se temepratura podiže do željene vrednosti je jako bitan s obzirom da brzo dostizanje temeprature zaparivanja može negativno uticati na kasne čvrstoće betona. Odlaganje početka ciklusa zaparivanja je često poželjno kako bi se napravio balans između početnih i kasnih čvrstoća betona koje se postižu procesom zaparivanja betona.

Tokom poslednje decenije razvijeni su i novi načini nege betona. Jedan od njih je metoda unutrašnje nege—*internal curing (IC)* [11, 12]. Unutrašnja nega podrazumeva da se voda za hidrataciju cementa nalazi unutar cementne paste u vidu različitih rezervoara. Najčešće se koriste laki agregat koji je prethodno zasićen vodom, superupijajući polimeri ili zasićena drvena vlakna. Ovaj princip funkcioniše tako što se ovi rezervoari sa vodom ravnomerno rasporede unutar matrice betona i neguju beton iznutra. Na ovaj način se smanjuju prsline tokom razvoja ranih čvrstoća i poboljšavaju mehaničke karakteristike betona tako što se efikasno pospešuje hidratacija cementa i pucolanska reakcija letećeg pepela.

Efikasnost unutrašnje nege ispitana je u istraživanju koje su sproveli D.P. Bentz i W.J. Weiss [12]. Ispitivanje je vršeno na malterima u kojima je zapreminski zamenjeno 40% i 60% cementa letećim pepelom a kao rezervaor za vodu korišćen je laki sitni agregat (upijanje nakon 24 h 17.5%) kao zamena dela normalnog agregata. Vodocementni faktor kontrolnog cementnog betona je 0.42 a betona sa letećim pepelom nešto niži - 0.3, kako bi se postigle potrebne čvrstoće. Na *Slici 1a* prikazan je princip funkcionisanja unutrašnje nege u poređenju sa standardnom spoljašnjom negom betona. Jasno je da se na ovaj način zona koja se neguje znatno proširuje. Na *Slici 1b* prikazan je prirast čvrstoće pri pritisku kontrolnih cementnih mešavina i mešavina sa letećim pepelom koje su negovane standardnom i unutrašnjom negom (IC). Sa dijagrama se može videti pozitivan efekat unutrašnje nege na prirast čvrstoće betona, posebno pri starosti od 28 dana i većoj. Ovo istraživanje je takođe pokazalo smanjenje autogenog skupljanja ali i pad modula elastičnosti (do 10%) usled primene unutrašnje nege.



Slika 1 - a) Prikaz razlike između spoljašnje i unutrašnje neze, b) Razvoj čvrstoće pri pritisku betona sa različitom količinom letećeg pepela i različitom vrstom neze (IC-unutrašnja neza) [12]

2.1 NEGA BETONA SA VELIKIM SADRŽAJEM LETEĆEG PEPELA

Imajući u vidu da je specifičnost betona sa velikim sadržajem letećeg pepela sporiji prirast čvrstoće pri pritisku mnoga istraživanja se bave tim pitanjem i pokušavaju da odgovore na pitanje koja je vrsta neze najpogodnija za ovu vrstu betona. Među prvim istraživanjima ove vrste, 1995. godine A. A. Ramezianpour i M. Malhotra [13] su ispitali uticaj različitih vrsta neze betona sa velikim sadržajem letećeg pepela. Na tri različite mešavine (referentni cementni beton, beton sa 25% i 58% zemene cementa letećim pepelom) ispitali su uticaj četiri različita režima neze: (1) standardna neza u bazenu nakon raskalupljanja uzorka, (2) neza na sobnoj temperaturi nakon raskalupljanja, (3) neza na sobnoj temperaturi nakon dva dana neze u bazenu i (4) neza na 38°C i 65% relativne vlažnosti. Beton sa velikim sadržajem letećeg pepela (58% zamene cementa) imao je najveću čvrstoću pri pritisku na uzorcima negovanim režimom (4) i imao veću čvrstoću i od referentnog cementnog betona pri starosti od 28 dana ali sporiji prirast pri većim starostima. Usled povećane temperature neze (38°C) dolazi do bržeg odvijanja hidratacije i pucolanske reakcije u poređenju sa standardnom negom betona (28 dana u bazenu) i do većih čvrstoća betona.

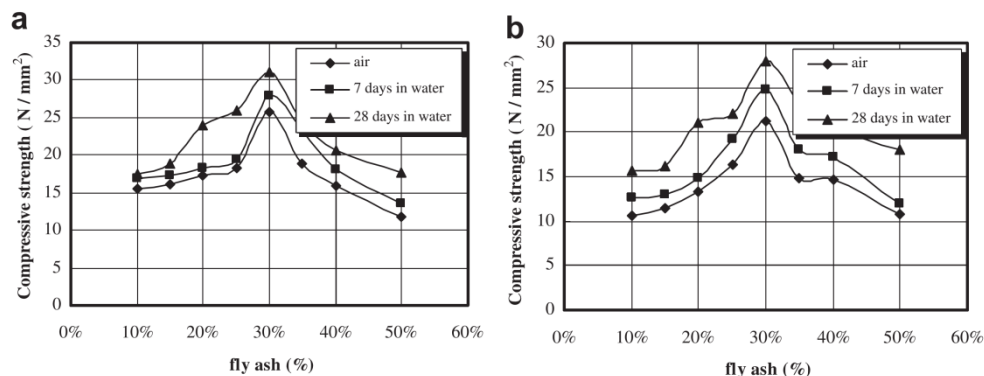
Pojava znatno sporijeg prirasta nakon 28 dana može se objasniti nedostatkom vode za dalji razvoj hemijskih reakcija u betonu [13]. Neza režimom (2) je dala najmanje čvrstoće betona zbog nedostatka vode za odvijanje reakcija u betonu, dok su čvrstoće nešto veće pri režimu (3) gde su uzorci negovani bar dva dana u bazenu pre neze na sobnoj temperaturi. Standardna neza u bazenu u trajanju od 28 dana daje dobre čvrstoće betona, očekivano veće nego tokom režima (2) i (3). Razlika u odnosu na običan cementni beton je u tome što su najveće čvrstoće kod tih betona ostvarene pri standardnoj nezi betona a pri režimu (4) dobijene su najveće rane čvrstoće ovih betona i dobar prirast čvrstoće ali nešto manja čvrstoća nakon 28 dana.

Različita vrsta neze betona sa letećim pepelom ispitana je i u istraživanju H. A. Mohamed [14] na betonima sa 10–50% zamene cementa letećim pepelom negovanim kroz tri režima: (1) neza na sobnoj temperaturi, (2) neza u bazenu u trajanju od 7 dana i (3) neza u bazenu u trajanju od 28 dana. Sa Slike 2 se vidi da je čvrstoća pri pritisku najveća na uzorcima negovanim u bazenu 28 dana a da je čvrstoća u proseku manja za 18% i 10% pri režimima

nege (1) i (2) respektivno. Na osnovu ovih istraživanja može se zaključiti da je voda ključan faktor za efikasno odvijanje hidratacije i pucolanske reakcije u betonima sa velikom količinom letećeg pepela ali da povišene temperature nege mogu ubrzati rani prirast čvrstoće [14].

Leteći pepeo se često koristi u proizvodnji valjanih betona. Uticaj različite vrste nege na valjane betone sa 0%, 50% i 70% zamene cementa sa dve vrste letećeg pepela ispitano je u istraživanju koje je sproveo C. D. Atis [15]. Korišćene su dve vrste letećeg pepela klase F [4]: iz termoelektane *Drax* (SiO_2 : 50.2% i Al_2O_3 : 28.6% a gubitak pri žarenju LOI: 2.8) i pepeo iz termoelektane *Aberthaw* (SiO_2 : 44.9% i Al_2O_3 : 25.2% a gubitak pri žarenju LOI: 15.6). Pepeo *Aberthaw* je lošijeg kvaliteta i ne zadovoljava uslove potrebne za primenu letećeg pepela u betonima [4]. Uzorci su negovani na sobnoj temperaturi od 20°C pri relativnoj vlažnosti od 65% i 100%. Čvrstoće pri pritisku, zatezanju cepanjem i savijanjem betona napravljenih sa kvalitetnijim pepelom (*Drax*) su veće na uzorcima negovanim na relativnoj vlažnosti od 100%. Kod betona napravljenih sa pepelom lošijeg kvaliteta (*Aberthaw*) ova zavisnost nije jasno izražena. Čvrstoće pri pritisku, zatezanju cepanjem i savijanjem betona pri različitim starostima je generalno veća pri nezi na relativnoj vlažnosti od 65% uz neke izuzetke.

Kako bi se povećala reaktivnost letećeg pepela često se koristi mehanička aktivacija kao princip za poboljšanje. Mehanička aktivacija podrazumeva mlevenje pepela kao bi se dobile sitnije čestice koje su reaktivnije. Rezultati jednog takvog istraživanja [16] tokom kog je vršeno ispitivanje maltera napravljenih sa različitom količinom letećeg pepela (10–60% zamene cementa letećim pepelom) koji je usitnjen na četiri različita načina dati su u nastavku.



Slika 2 - Čvrstoća pri pritisku betona u funkciji količine letećeg pepela i vrste nege za betone sa a) 550 kg/m³ cementa, b) 450 kg/m³ cementa [14]

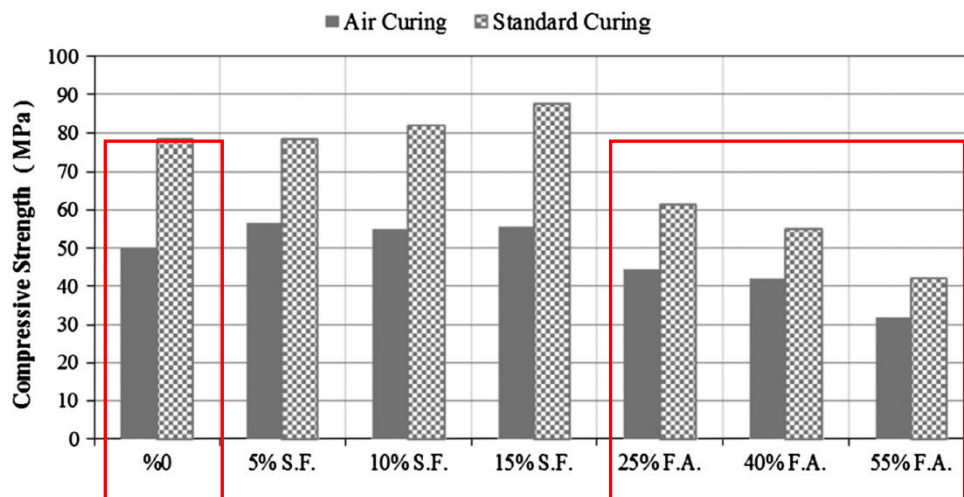
Leteći pepeo koji je korišćen tokom ovog istraživanja mehanički je aktiviran (u mlinu za mlevenje sa kuglama) u trajanju od 0, 10, 40 i 60 minuta - uzorci T0, T10, T40 i T60) a uzorci maltera su negovani u bazenu sa vodom na četiri različite temperature: 20, 40, 60 i 80°C. Istraživanje je pokazalo da se reaktivnost letećeg pepela povećava usitnjavanjem i da se najveće čvrstoće betona dobijaju sa pepelom koji je najviše usitnjen (uzorak T60). Najveće rane čvrstoće postignute su na uzorcima koji su negovani na višim temperaturama (80°C) ali je dalji prirast čvrstoće u ovom slučaju neznatan. Povećanjem temperature se povećava aktivnost pepela koji u većoj meri reaguje tokom perioda do tri dana pa je dalji prirast zanemarljiv. Najveći prirast čvrstoće i najveće kasne čvrstoće ostvarene su pri temperaturama od 20°C i

40°C, dok je prirast manji za temperaturu od 60°C a praktično zamenaerljiv pri temperaturi od 80°C.

Mehanička aktivacija pepela je dovela do sve veće upotrebe jako sitnog letećeg pepela karakterisanog kao ultrasitni pepelo (*ultrafine fly ash*). Veličina srednjeg zrna letećeg pepela je 20–30 mikrona dok su pepeli koji imaju srednje zrno 1–5 mikrona karakterisani kao ultrasitni pepeo [17]. Istraživanje koje su sproveli B. Liu i dr. [18, 19] imalo je za cilj utvrđivanje uticaja zaparivanja betona napravljenog sa 25–50% [2001] i 30–40% [2005] zamene cementa sa ultrasitnim letećim pepelom. Uzorci su negovani u dva režima: (1) standardna nega u bazenu i (2) zaparivanje na 60°C tokom ciklusa od dva časa postepenog podizanja temperature, 8 časova pri temperaturi od 60°C i jednog časa postepenog hlađenja. Proces zaparivanja je počeo dva časa nakon ugrađivanja betona u kalupe. Ispitivanje je pokazalo da betoni koji su negovani zaparivanjem imaju manje čvrstoće pri pritisku na 28 dana i manji prirast čvrstoće u odnosu na betone negovane u bazenu. Ovo je posledica toga što se prilikom zaparivanja betona na većim temperaturama hidratacija cementa odvija brzo što dovodi do bržeg stvaranja gela. Gel obavlja čestice cementa i letećeg pepela stvarajući postepeno oko njih čvrstu i gustu membranu. Prodiranje vode kroz gel i razlaganje čestica cementa i letećeg pepela je otežano pa se usporava dalja hidratacija i pucolanska reakcija u betonu. Što je veća temperatura i duže vreme zaparivanja, manji je prirast čvrstoće pri pritisku [19].

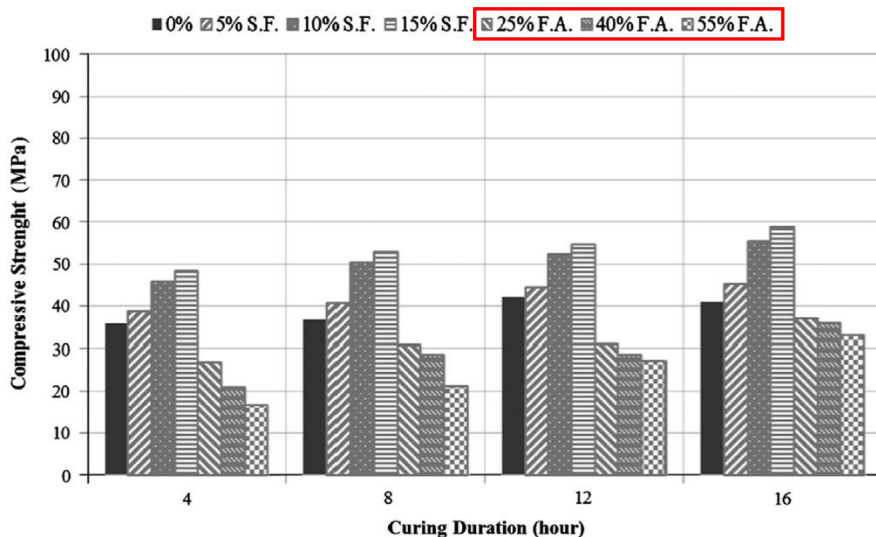
Leteći pepeo ima pozitivan uticaj na ugradljivost betona pa se često koristi u proizvodnji samougrađujućih betona (*self-compacting concrete*). Istraživanje koje su sproveli A. F. Bingöl i I. Tohumcu [20] analizira uticaj različite vrste nege na samougrađujuće betone koji imaju 0% (referentni beton), 25%, 40% i 55% zamene cementa letećim pepelom i vodovezivni faktor 0.35. Betoni su negovani na tri načina: (1) nega na sobnoj temperaturi od $20 \pm 2^\circ\text{C}$ i relativnoj vlažnosti od 55%, (2) nega na sobnoj temperaturi u bazenu u trajanju od 3, 7 i 28 dana i (3) nega zaparivanjem na temperaturi od 60, 70 i 80°C u trajanju od 4, 8, 12 i 16 časova. Zaparivanje se vrši u ciklusima koji podrazumevaju negu na sobnoj temperaturi u trajanju od 4 časa, postepeno povećanje temperature do maksimalne (60, 70 i 80°C) u trajanju od 2 časa, zagrevanje na maksimalnoj temperaturi u trajanju od 4, 8, 12 i 16 časova, postepeno spuštanje na sobnu temperaturu u trajanju od 2 časa i negu na sobnoj temperaturi u trajanju od 2 časa pre raskalupljanja.

Na *Slici 3* prikazani su rezultati ispitivanja čvrstoće pri pritisku betona starosti 28 dana sa 0%, 25%, 40% i 55% zemene cementa letećim pepelom (0%, 25% F.A, 40% F.A. i 55% F.A. respektivno) negovani na sobnoj temperaturi i relativnoj vlažnosti od 55% (*Air curing*) u poređenju sa rezultatima na uzorcima negovanim u bazenu 28 dana (*Standard curing*). Usled nege na sobnoj temperaturi dolazi do pada čvrstoće u poređenju sa negom u bazenu i za referentni beton i za betone sa pepelom. Čvrstoća pri pritisku se smanjila za 37%, 28%, 24% i 25% za referentni beton, betona sa 25%, 40% i 55% zamene cementa letećim pepelom respektivno. Usled nedostatka vode proces hidratacije ne može u potpunosti da se obavi pa očekivano dolazi do smanjenja čvrstoće [21]. Uzorci koji su negovani u bazenu 3, 7 i 28 dana imaju veće čvrstoće u poređenju sa uzorcima negovanim na sobnoj temperaturi. Za uzorke negovane 28 dana u bazenu povećanje čvrstoće je u proseku za 35%, 30%, 25%, 24% za referentni beton i betone sa 25%, 40% i 55% zamene cementa respektivno. Sa povećanjem trajanje nege u bazenu povećava se i čvrstoća betona za sve uzorke - najveće čvrstoće su ostvarene pri nezi od 28 dana a najmanje pri trajanju nege od 3 dana. Prirast čvrstoće sa povećanjem trajanja nege je najizraženiji kod referentnog betona.



Slika 3 - Efekat nege na sobnoj temperaturi i relativnoj vlažnosti od 55% na čvrstoću pri pritisku samougrađujućih betona [20]

Na slikama 4, 5 i 6 prikazane su čvrstoće pri pritisku betona negovanih zaparivanjem sa ciklusima u trajanju od 4, 8, 12 i 16 časova na temperaturama od 60, 70 i 80°C. Sa slika 4, 5 i 6 se vidi da se čvrstoća povećava sa povećanjem trajanja procesa zaparivanja za sve vrste betona pri čemu je razlika najveća između ciklusa od 8 i 12 časova. Čvrstoća se neznatno smanjuje sa povećanjem temperature za negu u trajanju od 4 i 8 časova, a ostaje ista ili se povećava za negu u trajanju od 12 i 16 časova.



Slika 4 - Čvrstoća pri pritisku uzoraka negovanih zaparivanjem na temperaturi od 60°C u trajanju od 4, 8, 12 i 16 časova [20]

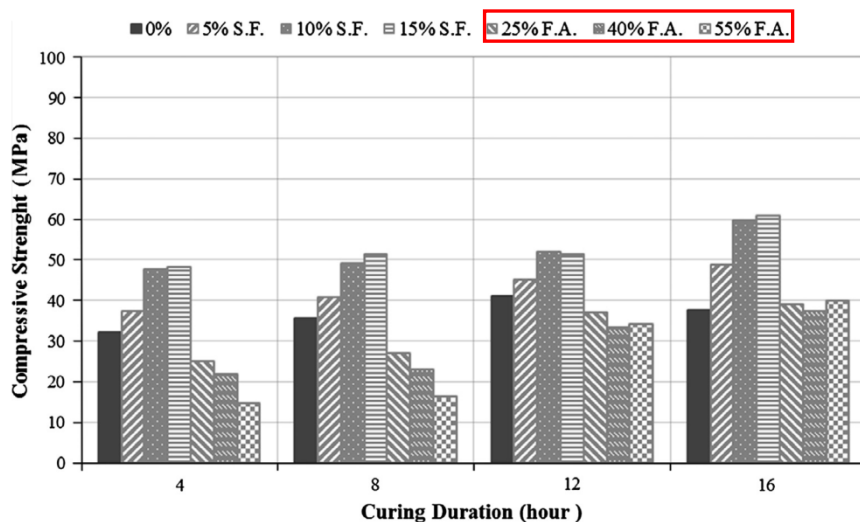
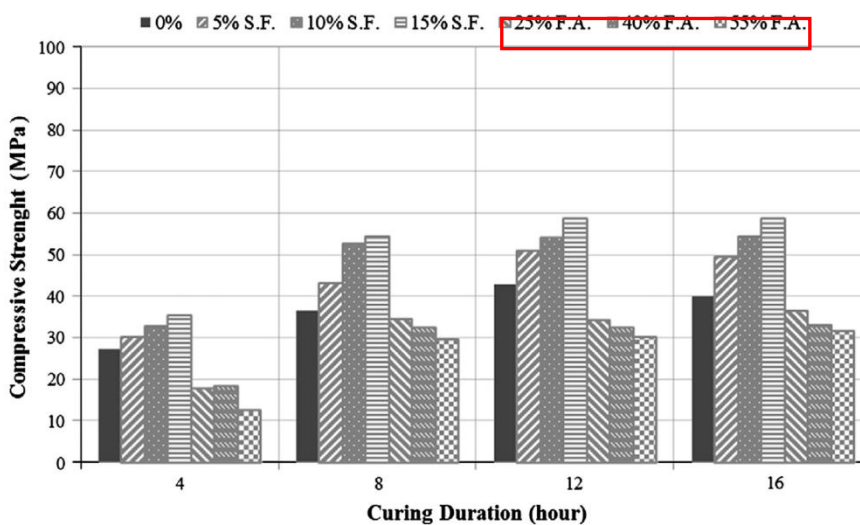


Fig. 5. Effect of steam curing durations on the compressive strength for 70 °C.

Slika 5 - Čvrstoća pri pritisku uzoraka negovanih zaparivanjem na temperaturi od 70°C u trajanju od 4, 8, 12 i 16 časova [20]



Slika 6 - Čvrstoća pri pritisku uzoraka negovanih zaparivanjem na temperaturi od 80°C u trajanju od 4, 8, 12 i 16 časova [20]

Ovo ukazuje na to da postoji optimalna temperatura i trajanje ciklusa zaparivanje koje zavisi i od mešavine betona i kvaliteta letećeg pepela. Kod betona sa letećim pepelom najveću čvrstoću imaju betoni sa 55% zamene cementa pepelom negovani na 70°C u trajanju od 16 časova (40 MPa). Referentni beton ima najveću čvrstoću na uzorcima negovanim na 80°C u trajanju od 12 časova (43 MPa). Svi betoni negovani zaparivanjem imaju manje čvrstoće u

poređenju sa standardnom negom u bazenu zbog neravnomerne raspodele produkata hidratacije pri nezi na višim temperaturama. Uticaj nege zaparivanjem na strukturu pora, čvrstoću pri pritisku, zatezanju cepanjem i modul elastičnosti tema je istraživanja koje su sprovedli A. Gonzalez-Corominas i dr. [22]. Tokom ovog istraživanja ispitani su betoni sa prirodnim i recikliranim agregatom sa 30% zamene cementa letećim pepelom i vodovezivnim faktorom 0.285.

Uzorci su negovani na dva načina: (1) na sobnoj temperaturi (23°C) pokriveni mokrom tkaninom i plastičnom folijom u trajanju od 24 časa i (2) zaparivanjem u ciklusu koji podrazmeva negu na temperaturi od 30°C u trajanju od 4 časa, postepeno povećanje do maksimalne temperature od 65°C u trajanju od četiri časa, negu na maksimalnoj temperaturi u trajanju od osam časova i postepeno spuštanje na temperaturu od 30°C u trajanju od osam časova. Nakon završetka nege uzorci su do ispitivanja čuvani na sobnoj temperaturi u prostoriji sa relativnom vlažnošću od 95%. Prosečna veličina pora je manja nakon nege zaparivanjem u poređenju sa uzorcima negovanim na sobnoj temperaturi. Za sve ispitane betone čvrstoća pri pritisku za starosti od 28 i 90 dana je veća kod uzoraka negovanih na sobnoj temperaturi u odnosu na uzorke negovane zaparivanjem dok su rane čvrstoće (starost od jednog dana) veće kod uzoraka negovanih zaparivanjem. Čvrstoća pri zatezanju cepanjem je manja prilikom nege zaparivanjem za beton sa prirodnim agregatom ali veća za betone sa recikliranim agregatom. Modul elastičnosti je veći za sve betone koji su negovani zaparivanjem u odnosu na negu na sobnoj temperaturi.

3 ZAKLJUČAK

Kako bi se dostigle potrebne rane čvrstoće betona sa velikim sadržajem letećeg pepela potrebno je koristiti leteći pepeo koji odgovara standardu za primenu pepela u betonima, koji ima što manju veličinu srednjeg zrna i nizak vodovezivni faktor. Kao moguće rešenje često se vrši usitnjavanje letećeg pepela ili se koristi ultrasitni pepeo. Na osnovu rezultata istraživanja koja se bave uticajem različitih vrsta nege na mehaničke karakteristike betona sa velikim sadržajem letećeg pepela može se zaključiti da je voda ključan faktor za efikasno odvijanje hidratacije i pucolanske reakcije ali i da povišene temperature nege mogu ubrzati rani prirast čvrstoće.

Nega zaparivanjem betona sa velikim sadržajem letećeg pepela dovodi do nižih čvrstoća pri pritisku betona pri starosti od 28 dana ali pozitivno utiče na rane čvrstoće koje su generalno problem kod ove vrste betona. Kako bi se postigle dovoljne rane i kasne čvrstoće potrebno je pronaći balans između dužine nege uzoraka u standardnim uslovima pre početka zaparivanja i dužine trajanja ciklusa zaparivanja. Temperatura, dinamika i trajanje ciklusa zaparivanja bitno utiču na mehaničke karakteristike betona sa velikim sadržajem letećeg pepela. Neadekvatan režim zaparivanja može dovesti do promena u poroznosti betona i sprečavanja adekvatnog razvoja hidratacije i pucolanske reakcije betona, što može bitno smanjiti njegove mehaničke karakteristike. Stoga je bitno odabrati adekvatnu starost u kojoj se počinje sa režimom zaparivanja na povišenim temperaturama, trajanje ciklusa zaparivanja i maksimalnu temperaturu. Ove parametre treba odabrati u zavisnosti od kvaliteta letećeg pepela i mešavine betona.

ZAHVALNOST

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UTICAJ ZAMRZAVANJA I ODMRZAVANJA NA STRUKTURU BETONA

Rezime:

S-97

U toku eksploatacije, betoni izloženi dejstvu mraza i/ili dejstvu mraza i soli za odmrzavanje moraju biti otporni. Spravljeni su i ispitani uzorci, pri čemu su zadovoljeni svi zahtevi i preporuke da bi beton bio otporan. Stavljen je poseban akcenat na trajnost betona pa su uzorci ispitani na M-250 i na 1200 ciklusa zamrzavanja i odmrzavanja, odnosno htelo se videti da li je u dužem vremenskom periodu došlo do promena u strukturi betona. Ispitivanja su obavljena destruktivnom, i nedestruktivnom metodom. Nakon više od godinu dana nije došlo do promene u strukturi betona.

Ključne reči: zamrzavanje i odmrzavanje, pore, RapidAir 457

INFLUENCE OF FREEZING AND THAWING ON THE STRUCTURE OF CONCRETE

Summary:

During the exploitation, concretes exposed to frost and/or frost and de-icing salts must be resistant. Samples were made and tested and they met all the requirements and recommendations for concrete resistance. Special emphasis was put on the durability of concrete, therefore the samples were tested on M-250 and at 1200 cycles of freezing and thawing, that is, to be found out if during a longer time period there were changes in the concrete structure. Tests were carried out using the standard destructive, as well as non-destructive method. After more than a year there was no change in the structure of concrete.

Key words: freezing and thawing, pores, RapidAir 457

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1 INTRODUCTION

Freezing and thawing of hardened concrete is one of the most important problems of the durability of concrete.

In northern regions concrete structures deteriorate more rapidly due to the cold weather during the winter. Widespread use of de-icing salts during the winter is one of the major causes of the rapid degradation of concrete pavements, bridge decks, parking structures, and similar structures. De-icing salt is usually spread on the surface of concrete roads in the winter for driver safety because it lowers the water's freezing point. Concrete deterioration is more rapid in these cold weather environments because it is subjected to de-icing salts under freezing and thawing cycles [1].

When the temperature of the water saturated hardened concrete drops below zero, then the water contained in the capillary pores freezes, expanding in volume around 9%, and thus the concrete spreads. When the refreezing occurs, the additional expansion also occurs, so the cumulative effect is achieved by successive alternating freezing and thawing [2].

Another cause of the concrete spreading is in the diffusion of the water which is produced as a result of osmotic pressure.

When the stresses in concrete exceed its tensile strength, the concrete is destroyed.

Most vulnerable to destruction are curbs, and then roads.

While the concrete frost resistance depends on its strength, susceptibility of concrete to cold depends on the degree of humidity (dry concrete is completely resistant to cold) and pore structure of the cement stone.

The mechanisms connected to the scaling of concrete surfaces due to freezing in the presence of de-icing salts are still not fully understood [3,4].

The pores, which are often referred to as entrapped air, are mainly the result of incomplete compaction of concrete or subsequent allocations of water, and are usually larger than 1 mm, therefore their role in the freezing of concrete is insignificant.

Hydration products that create a quasi-layered structure are formed by the hydration of cement, and there are spaces between the layers representing the pores of the gel. The gel pores are too small, 1-5 nm, therefore ice is practically not produced in them, but the diffusion of the gel water to the capillary pores which already contain the ice due to the fall of temperature also contributes to the spread of concrete [2].

Capillary pores are in the places where are no products of hydration. The amount of the pores depends on the water when making the concrete mixture. They are connected, and the size of 5 nm to 5 μ m. Freezing begins with the large capillaries and progressively spreads towards the smaller ones [2].

Air entrained in concrete implies only the air that is deliberately drawn into the concrete using chemical additives-air entraining agents. These are spherical bubbles whose diameter ranges from 5 μ m to 1250 μ m, arranged in the cement paste, which are not connected, but they neither fill up with hydration products of cement since the gel can be formed only in the presence of water [2].

What is common for all types of pores is that in concrete they are formed in the cement stone and at the junction of the cement stone and aggregate.

In addition to the minimum volume of air bubbles that provides resistance to cold, it is important that they are evenly distributed in the cement paste. The indicator of which that

depends is the spacing factor of the micropores of entrained air (the average maximum distance of any point in the cement stone to the edge of the nearest bubble).

The entrained air content is higher in a more workable mixture and with the use of crushed aggregate instead of the river aggregate, and it decreases with an excess of fine particles of fine fraction and fineness of the cement grind.

The content of entrained air also depends on the way and length of concrete mixing (if the mixing is short the additive will not be evenly distributed throughout the concrete mass, and if the mixing is too long the air that has already been entrained in the concrete will be drawn out).

Higher temperature causes a greater loss of air.

Excessive vibration of concrete causes a certain amount of entrained air to be drawn out.

While drawing the air in, average loss of concrete strength of 5.5% should be calculated for each present percent of air, but the cone settlement is increased and workability is improved.

By the presence of entrained air the risk of segregation is reduced i.e. bleeding is reduced.

Air bubbles act like very fine aggregate, so that the percentage of participation of fine fraction can be reduced, and this gives the possibility to reduce the amount of water in the mixture which makes up for the loss in strength due to the space occupied by the entrained air.

If the concrete mixtures are prepared with such a low water-cement ratio which will ensure that sufficiently small capillaries form in the paste, the concrete will have very little permeability so it will not absorb water. In addition, the length of concrete curing in water (more intense hydration decreases the amount of water subject to freezing), and drying the concrete before freezing also have an influence.

The chemical composition of cement, or cement type, as well as the fineness of grind, do not have any influence on its resistance to frost except for the very young concrete.

To prevent the risk of devastating frost action, it is essential that the concrete is well compacted and impermeable, so the techniques that would not guarantee good compaction should be avoided.

Aggregates with too big largest grain of aggregate, as well as with a multitude of flat grains are not recommended for use because they increase the possibility of creating "pockets" filled with water under these grains.

For concrete exposed to frost and/or frost and de-icing salts during exploitation, special features can also be set as the conditions for the class of concrete: resistance to frost, that is, a certain number of cycles of alternating freezing and thawing which concrete has to satisfy according to SRPS U.M1.16 (M-50, M-100, M-150, M-200) and/or the resistance to frost and de-icing salts as the degree of damage 0-without scaling or 1-light scaling measured by loss of mass mg/mm^2 or depth of damage in accordance with SRPS U.M1.055.

According to the BAB '87 Regulations, concrete exposed to the frost or frost and de-icing salts must be protected by aeration and air content in fresh concrete is prescribed depending on the size of the largest aggregate fractions [2].

According to EN 206-1 the effects of environmental conditions are classified as the classes of exposure. When the concrete is in the wet state, exposed to significant cycles of freezing and thawing, with or without de-icing agents, exposure classes XF1-XF4 are defined. For each of these classes limit values are recommended for the composition and properties of concrete in terms of the maximum water-cement ratio, lowest strength class, the least amount of cement, the minimum air content in fresh concrete, and aggregate should be in accordance with EN 12620 with sufficient resistance to freezing and thawing [2].

According to the Serbian standard SRPS U.M1.206 concrete manufacturer may, by contract with the authorized body for conformity assessment of concrete production, replace the testing of concrete resistance to frost and/or frost and de-icing salts with the examination of the spacing factors of pores according to SRPS U.M1.056. In addition, the spacing factor in the initial tests must not be greater than 0.18, and later in control tests than 0.20. In the initial tests for every concrete composition it should be shown that the concrete meets the criteria for frost resistance and criteria for spacing factors of pores [5].

2 EXPERIMENTAL WORK

Two preliminary tests for concrete were conducted.

Samples were made from each concrete mixture, cubes with 150 mm edge, for testing the resistance to frost M-250 (the maximum value specified in the standard SRPS U.M1.016 and PBAB 87) and 1200 cycles of freezing and thawing, testing resistance of concrete to frost and de-icing salts according to SRPS U.M1.055, as well as the determination of the spacing factor of micropores of entrained air according to the SRPS U.M1.056 [6].

According to SRPS U.M1.016 the results of testing compressive strengths of the samples exposed to frost and appropriate samples-standards of the equivalent age were compared. According to SRPS U.M1.055 the depth of damage in mm and weight loss in mg/mm^2 were measured and a visual inspection of samples after every 5 cycles was carried out. Samples were photographed before and after tests.

Concrete exposed to frost changes its characteristics, both in terms of strength and in terms of structure. The aim of this study is also to determine how the concrete structure changes after frost action. Changing the structure of concrete is estimated by qualitative and quantitative analysis of the pores in concrete using the RapidAir 457 device.

According to SRPS U.M1.056 cubes were cut in order to obtain the required samples that were prepared adequately. By recording with the RapiAir 457 device the amount of pores according to their shape and size, as well as the arrangement of pores was obtained, and then the spacing factor of pores was determined [7].

For the requirements of PBAB to be satisfied, the amount of entrained air for $D_{\text{max}}=16$ mm of 5-7% was designed.

For the strictest level of exposure XF4 (XF4 are the environmental conditions that include a large water saturation with de-icing agents in the case of application for the elements of the roads and bridges and other concrete surfaces exposed to de-icing agents and frost) according to SRPS EN 206-1, water-cement ratio less than $W/C=0.45$, the minimum strength class C 30/37, the amount of cement higher than 340 kg/m^3 , air content in fresh concrete greater than 4% were designed and the aggregate of sufficient resistance to freezing and thawing was used in accordance with EN 12620 [2].

The consistency was designed by the slump flow of 200 ± 20 mm (class S4).

2.1 COMPONENT MATERIJALS

Both preliminary tests were performed with the same component materials:

- Cement: PC 20M (S-L) 42,5R BFC Lafarge,
- Aggregate: river, washed, granulated, $D_{\text{max}} = 16\text{mm}$
- Water: water from the water supply

- Additive: Admixture 1-superplasticizer "Superfluid 21 M1M" Admixture 2- air entraining agent "Poročinitelj", Ading

2.2. COMPOSITION OF CONCRETE

Mixture composition is shown in Table 1.

Table 1 - Concrete Mixture composition

Components materials	PP-1	PP-2
Cement, (kg/m ³)	430	400
Aggregate (kg/m ³)	1673	1725
Water (kg/m ³)	167	168
Admixture 1 (kg/m ³), 0.70%	3.010	2.800
Admixture 2 (kg/m ³), 0,06%	0.258	0.240
Density, (kg/m ³)	2273	2296
W/C ratio	0.388	0.420

2.3 EXPERIMENTAL RESULTS

Experimental results are shown in Table 2 and Figure 1.

All values are presented as the mean value of three test results.

Table 2 - Experimental results

Concrete testing	PP-1	PP-2
Air content (%)	6.0	5.2
Consistency (mm)	200	180
Scaling degree	0-without scaling	0- without scaling
Compressive strength (N/mm ²), 28 days	48.4	48.0
Compressive strength (N/mm ²)		
E-I	53.3	57.2
M-250 (% compared to standard)	52.8 (99)	57.4 (100)
E-II	59.7	63.6
1200 cycles (% compared to standard)	60.00 (100)	62.1 (98)
Spacing factor		
E	0.091	0.108
250 cycles	0.096	0.112
1200 cycles	0.095	0.109

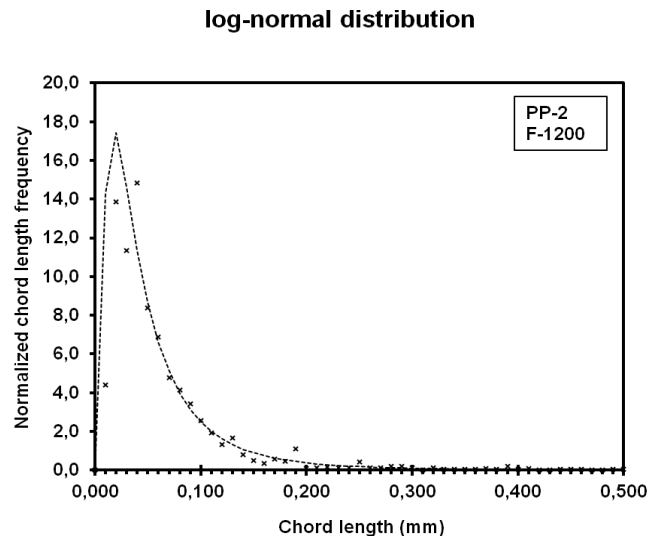
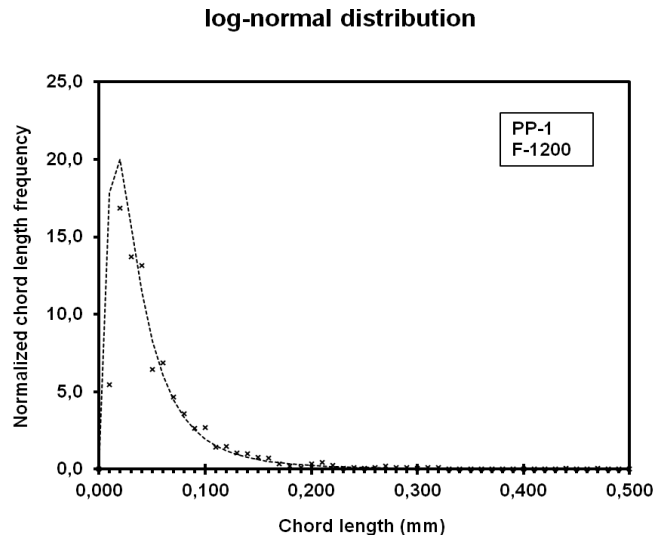


Figure 1 – Normalized chord length frequency for concrete PP-1 and PP-2 after 1200 cycles

In terms of air content both batches met the requirements of PBAB '87 and the recommendation of SRPS EN 206-1.

In both cases projected consistency was achieved.

With regard to the achieved compressive strengths MB 40 (equivalent to C 30/37) can be expected to be satisfied.

In terms of resistance to frost and de-icing salts in both batches of samples there have been no changes on the surface so that they are resistant, the degree of damage 0-without scaling. In terms of frost resistance samples of both series met M-250 (the condition is that the compressive strength of frozen samples must be at least 75% of the strength of the samples of equivalent age that were not frozen). The same criterion was applied to samples which were subjected to 1200 cycles of freezing and thawing, which have also met it. Spacing factor of pores demonstrates that the produced concretes are resistant to frost, and that there was no change in their structure in one year.

3 CONCLUSIONS

The aim of this study is to show that, if complied with all prescribed requirements and recommendations, concrete resistant to freezing and thawing with and without de-icing agents can be made. In addition, the accent is put on the durability of concrete, so the subject concretes were exposed to freezing and thawing for 1200 cycles. Criteria for evaluation were that according to SRPS U.M1.055 concrete should be resistant, according to SRPS U.M1.016 required resistances of M-250 and 1200 cycles of freezing and thawing should be met, and according to SRPS U.M1.056 spacing factor of pores in concrete the previous tests should be 0.18.

Test results showed that the criteria of all the regulations and standards were met, and that there was no difference in the values after one year.

ACKNOWLEDGMENTS

The work reported in this paper is a part of the investigation within the research project TR 36017 "Utilization of by-products and recycled waste materials in concrete composites in the scope of sustainable construction development in Serbia: investigation and environmental assessment of possible applications" supported by the Ministry of Education and Science, Republic of Serbia. This support is gratefully acknowledged.

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VEZA IZMEĐU DESTRUKTIVNIH I NEDESTRUKTIVNIH ISPITIVANJA BETONA SA NANOSILIKOM

Rezime:

Predmet ovog istraživanja su betoni sa i bez delimičnom zamenom potrebne količine cementa česticama nano-silike, kao i međusobna zavisnost rezultata njihovog ispitivanja destruktivnim i nedestruktivnim metodama. Na uzorcima betona sa 0%, 1%, 2% i 3% dodatka nano-silike određena je brzina prolaska ultrazvučnih talasa i visina odskoka čekića, a zatim ispitana čvrstoća pri pritisku. Uspostavljena je međusobna zavisnost merenih veličina dobijenih ovim ispitivanjima.

Ključne reči: nano-silika, pritisna čvrstoća, brzina ultrazvuka, visina odskoka čekića

RELATION OF DESTRUCTIVE AND NON- DESTRUCTIVE TESTING OF CONCRETE WITH NANOSILICA

Summary:

The subject of this research are concretes with and without partial replacement of the required amount of cement with the particles of nano-silica, as well as the interdependence of the results of their tests by using destructive and non-destructive methods. The ultrasonic pulse velocity and the rebound number was determined on the samples of concrete with 0%, 1%, 2% and 3% addition of nano-silica, and then compressive strength was tested. The interdependence of the measured values obtained in these trials was established.

Keywords: nano-silica, compressive strength, ultrasonic pulse velocity, the rebound number

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1 INTRODUCTION

Modern research prompted by the need to preserve natural resources and protection of the environment lead to increasingly frequent attempts to replace component materials in the production of concrete with raw materials which are secondary products of industrial production. One such attempt is the replacement of a part of the required amount of cement with the particles of nano-silica. In concretes with the addition of nano-silica, even in small amounts, the rapid increase in strength in the early ages, as well as improved durability were noticed [1]. The reason is the large reactive surface area of SiO_2 particles, which stimulate reactions of the hydration phase of the core on their surface [2, 3]. Here, the initial phase of high growth characterizes the increase in hydration heat, due to the degradation of free lime and aluminate phase and forming of entrigites. Then the phase of low hydration heat occurs, when the formation of C-S-H phases and portlandite cores begins. In the following three hours accelerated hydration of C_3A and the formation of C-S-H phases and portlandite, encourages the intense development of hydration heat, after what the heat increase begins to decline [4]. Due to the high specific surface and high reactivity of nano-particles, in concretes with the addition of nano-silica faster increase in strength is recorded. Yet research has shown that in the ages of over 90 days and in concretes with more than 2% of nanoparticles that increase declines [1,5].

The introduction of nanotechnology in the production of concrete, created new types of concrete, which requires testing of their behavior, properties and the establishment of functional dependencies between them. As tests by non-destructive methods are being increasingly used in the quality control of concrete and in assessment of the structure condition, it is required to approximately establish the functional dependence between the tested properties of concrete and sizes measured by these methods.

The dependence of compressive strength and ultrasonic pulse velocity, as well as the compressive strength and rebound number for concretes with and without the addition of nano-silica are analyzed in this paper. Tests were conducted at the age of 1, 2, 3, 7 and 28 days.

2 COMPONENT MATERIALS AND MIX DESIGN

For the purposes of this study, four mixtures were composed, one reference and three mixtures in which the proportion of nano-silica varied from 1 to 3%. The mixtures were made in the Laboratory for concrete, IMS Institute with the following component materials:

- Cement: CEM II A-M (S-L) 42.5 R "Lafarge BFC" Beocin, specific weight 3900 kg/m^3
- Aggregate: river, separation "Hidro base" Ostruznica, separated into fractions 0/4, 4/8, 8/16 mm, with percentage shares of 45%, 25%, 30%, bulk density of 2650 kg / m^3

- Water: water from the water supply
- Chemical admixture: superplasticizer "Sika Techno 20s" Sika, Serbia
- Secondary binder: nano SiO_2 , particle size 7 nm

Chemical compositions of cement and nano - silica are shown in Table1.

Mixtures with the addition of nano-silica were made by first adding superplasticizer and nano-silica in the water, and then mixing them with the dry cement and aggregate. The reason for this is the high surface energy of nano particles, which prevents their equal distribution in the concrete mix. The composition of concrete is shown in Table 2.

Table 1 - Chemical composition of cement and NS (%)

Oxide	Cement CEM II/ A-M (S-L) 42.5R	Nano - silica
SiO ₂	20.6	99.9
Al ₂ O ₃	5.11	-
Fe ₂ O ₃	2.61	-
CaO	60.48	-
MgO	2.45	-
Na ₂ O	0.32	-
SO ₃	3.90	-
Cl ⁻	0.04	-
K ₂ O	0.51	-

Table 2 - Composition of concrete mixtures

Concrete series	Components materials					
	Cement	Aggregate	Nano-silica	Techno 20S	Water	w/b
	kg/m ³	kg/m ³	kg/m ³	kg/m ³	kg/m ³	
S0	350.0	1876	-	3.85	155	0.443
S1	346.5	1876	3.5	3.85	155	0.443
S2	343.0	1876	7.0	3.85	155	0.443
S3	339.5	1876	10.5	3.85	155	0.443

3 EXPERIMENTAL WORK

3.1 APPLIED METHODS

All tests in this study were performed on cubes with dimensions 100x100x100 mm. Mixtures of S0 and S1, with and without the addition of 1% nano-silica were placed into molds using the vibro needle, while vibrating table was used for the mixtures S2 and S3, with 2 and 3% of nano-silica. After all the series were made, they were cured under the same conditions, 24 hours in air and then in water at the temperature 20°C until the moment of testing.

For all series, examination by non-destructive methods - determination of the ultrasonic pulse velocity and the rebound number was first conducted, and then compressive strength was tested on the same samples. Determination of ultrasonic pulse velocity was performed at the ages of 1, 2, 3, 7 and 28 days using an ultrasound device. Testing was conducted fully in accordance with SRPS EN 12504-4. The test results were obtained as mean values of the results of tests on the three cubes. Determination of the rebound number was carried out according to SRPS EN 12504-2. After these tests, determination of the compressive strength according to SRPS EN 12390-3 was performed, for which Controls digital press with the range of 4000 kN was used.

3.2 TEST RESULTS

Based on the measurements and the obtained results, a relation between the values measured by the non-destructive methods and compressive strength was established.

Figure 1 shows the dependence of the ultrasonic pulse velocity and compressive strength at the ages of 1,2,3,7 and 28 days, respectively for each of the four series.

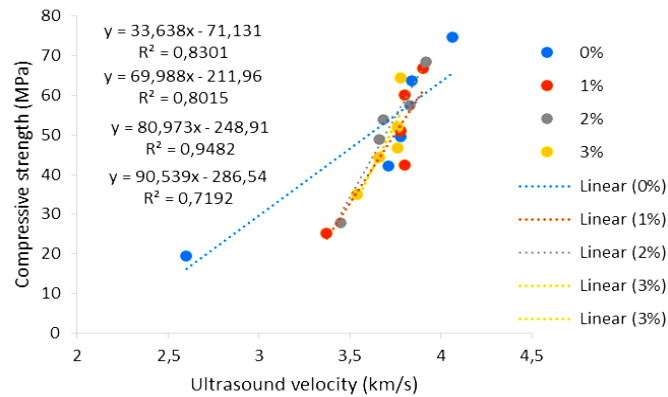


Figure 1 – Relation of the ultrasonic pulse velocity and compressive strength at the ages of 1,2,3,7 and 28 days for the series of concrete with 0%, 1%, 2% and 3% addition of nano-silica

In Figure 2, for easier observation of the results, the diagram of relation of the ultrasonic pulse velocity and compressive strength for all four series of concrete at the age of 28 days is shown. The relation of the observed properties is estimated by the coefficient of determination R^2 , which in this case is 0.8821.

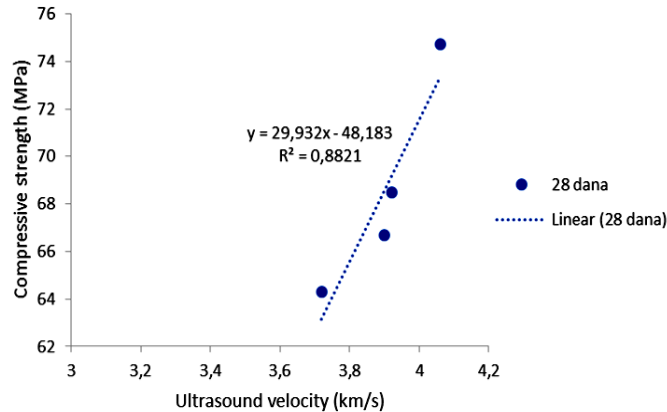


Figure 2 - Relation of the ultrasonic pulse velocity and compressive strengths for series of concrete with a of 0%, 1%, 2% and 3% addition of nano - silica in 28 days

In the same way, the connection between the rebound number and compressive strength was also established. Figure 3 presents the diagrams of relation of these values at the ages of 1,2,3,7 and 28 days for concrete series S0, S1, S2 and S3.

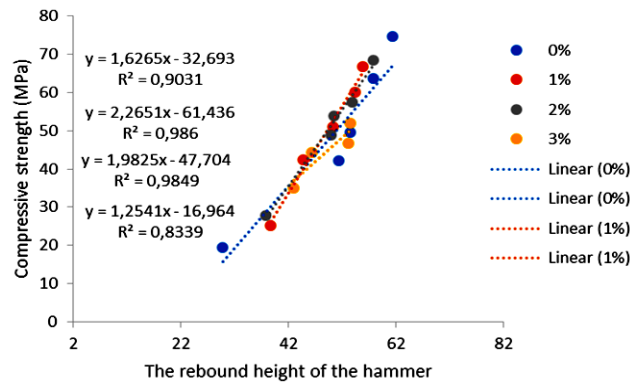


Figure 3 – Relation of the rebound number and compressive strength at the ages of 1,2,3,7 and 28 days for concrete series with 0%,1%,2% and 3% addition of nano-silica

Figure 4 shows a diagram of the relation of the height of the rebound number and compressive strength for concrete series S0, S1, S2 and S3 at the age of 28 days. The coefficient of determination R^2 is 0.9826.

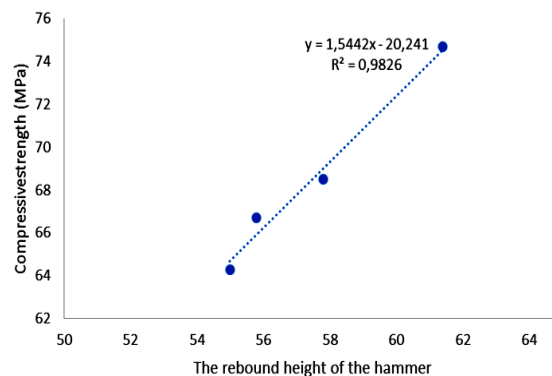


Figure 4 - Relation of the rebound number and compressive strength of concrete series with 0%, 1%, 2% and 3% addition of nano – silica, at the age of 28 days

Based on the coefficient of determination R^2 , correlation coefficient of linear dependence r is determined. If the correlation coefficient r is in the range of (0.8 – 1.0) it is considered that there is a very strong correlation.

4 CONCLUSION

By observing Figure 1 – the diagram of the relation of the ultrasonic pulse velocity and compressive strength at the ages of 1, 2, 3, 7 and 28 days, for four series of concrete and analyzing the obtained relation, it can be observed that the coefficients of determination R^2 of these linear relation range from 0.9482 to 0.7192, while the correlation coefficients r range from 0,974 to 0,848, which represents very strong correlations.

For the mutual relation of the rebound number and compressive strength at the ages of 1, 2, 3, 7 and 28 days of the observed series of concrete, which are shown in Figure 3, the coefficients R^2 range from 0.986 to 0.8339. The coefficients r , determined by the coefficients of determination, have a value of between 0.993 and 0.913 in this case.

In the case of relation of the rebound number and compressive strength at the ages of 1, 2, 3, 7 and 28 days for these concrete series, the correlation coefficient is closer to one, than in the case of the dependence of the ultrasonic pulse velocity and compressive strength. The same can be concluded for diagrams in Figure 2 and Figure 4, which show the linear relation of the ultrasonic pulse velocity and compressive strength and the rebound number and compressive strength at the age of 28 days. The correlation coefficient for the linear relation shown in Figure 2 is 0.939, while for the linear relation shown in Figure 4, the coefficient is 0.991.

From this analysis we can conclude that there is a very strong correlation between the values measured by non-destructive methods and compressive strength.

As the correlation coefficients for the linear relation of the rebound number and compressive strength are somewhat closer to one, than correlation coefficients of the ultrasonic pulse velocity and compressive strength, it is concluded that the rebound number follows the compressive strength slightly better than the ultrasonic pulse velocity.

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ОТПОРНОСТ НА SMRZAVANJE - ODMRZAVANJE MLAZNOG BETONA S RECIKLIRANIM POLIMERNIM VLAKNIMA IZ OTPADNIH GUMA

Rezime:

66-S

Glavni cilj ovog istraživanja je procijeniti učinkovitost recikliranih polimernih vlakana iz otpadnih guma (RTPF) kao ojačanja u betonu. U okviru ovog istraživanja, mlazni beton s RTPF i PP vlakna zajedno s kontrolnom mješavinom bez vlakana su pripremljeni i testirani. Sljedeća svojstva mlaznog betona su testirana: gustoća, konzistencija svježeg betona, tlačna čvrstoća i otpornost na smrzavanje - odmrzavanje. Analiza rezultata pokazuje da se RTPF vlakna mogu koristiti kao zamjena PP vlakana, budući da ne otežavaju prskanje ili pumpabilnost mlaznog betona a istovremeno osiguravaju poboljšanje svojstva betona.

Ključne reči: mlazni beton, polipropilenska vlakna, reciklirana polimerna vlakna

FREEZE - THAW RESISTANCE OF SPRAYED CONCRETE WITH RECYLED TYRE POLYMER FIBERS

Summary:

The main aim of this study is to assess effectiveness of recycled tyre polymer fibres (RTPF) as reinforcement in concrete. In the framework of this study, sprayed concrete with RTPF and PP fibres together with the control mix without fibres were manufactured and tested. Following properties of sprayed concrete were tested: density, fresh concrete consistency, compressive strength and freeze - thaw resistance. Analysis of the results indicates that RTPF can be used as substitution of PP fibres, since they do not impair sprayability or pumpability of sprayed concrete at the same time assuring beneficial properties of concrete.

Key words: sprayed concrete, polypropylene fibres, recycled tyre polymer fibres

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1 INTRODUCTION

Sprayed concrete can be applied for many different applications, from tunnelling and mining, slope stabilizations and concrete repairs. Good characteristics and various advantages, like a good adhesion to the substrate, concreting without formwork, high strengths with rapid strength gain, high speed output, ease application to restricted areas etc., assure its application. When higher compressive strength, flexural strength or durability is required, admixtures like water reducers, curing agents as well as fibres of different origin are more than welcome. In structures susceptible to shrinkage and/or exposed to specific load conditions, used concrete is improved with polypropylene fibres. Polypropylene fibres dispensed in small doses from 0.6 to 1.0 kg/m³, can have positive effect on cohesion of the mix, reduced both plastic settlement and shrinkage cracking, improved impact and abrasion resistance, reduced permeability and increased spalling resistance [1] - [4].

For the design of more eco-friendly sprayed concrete, it is of interest to see if recycled polymer fibres obtained from end-of-life tyres can replace polypropylene fibres in concrete. Main challenge regarding recycled tyre polymer fibres (RTPF) is storage; due to their low weight they are easily carried by the wind and are extremely flammable. Currently, RTPF are mainly landfilled or valorised as an alternative fuel during cement production. Research presented hereafter is part of a FP7 project - Anagennisi [5, 6], where the aim of the project is to identify suitable applications for RTPF in concrete and to put an end to the current practice of landfilling this material. Based on the limited literature data [5] - [8], RTPF do not induce negative effects on concrete mechanical properties and may have beneficial effect on early age deformation of concrete [5, 6, 8]. Due to the difference in compaction processes, fibre reinforced concrete and fibre reinforced sprayed concrete are expected to have different properties [9].

In the framework of this initial study, RTPF are used for the first time for production of sprayed concrete mixes for slope protection. Four mixes were prepared and tested, one without fibres, mix with 0.9 kg of polypropylene fibres per m³ and two with 0.9 and 1.8 kg of RTPF per m³. Since the analysis of RTPF showed that their size (length and diameter) is very small, polypropylene fibres with the maximum length of 6 mm were chosen for this investigation and were replaced by RTPF with ratios 1:1 and 1:2.

2 MATERIALS AND METHODS

2.1 CONSTITUTING MATERIALS

Concrete mixes were prepared with CEM II/B-M (S, V) 42.5 N, crushed limestone as aggregate (0/4 mm and 4/8 mm) and air entrainment. Two types of fibres were used as reinforcement: multifilament polypropylene (PP) fibres with properties presented in Table 1 and recycled tyre polymer fibres (RTPF) as shown in Table 2. Length distribution is from 2 to 20 mm, but more than 80% of fibres have length shorter than 12 mm. Reused tyre polymer fibres were used as received without further cleaning/sorting. Taking into account the high contamination of RTPF with rubber particles, an investigation was followed to determine the mass of each constituent in RTPF. Statistical analysis showed that fine rubber with very short RTPF occupied more than 65 % of the mass in each RTPF sample, with particles of a diameter less than 0.5 mm. Hereafter presented results are obtained on as received RTPF, but

information gather during cleaning of RTPF is taken into consideration during analysis of the results.

Table 1. Properties of polypropylene fibres

Length, mm	Diameter, μm	Density, g/cm^3	Tensile strength, N/mm^2	Melting point, $^{\circ}\text{C}$
6 mm	approx. 32 μm	0.91	> 270 N/mm^2	approx. 160 $^{\circ}$ C

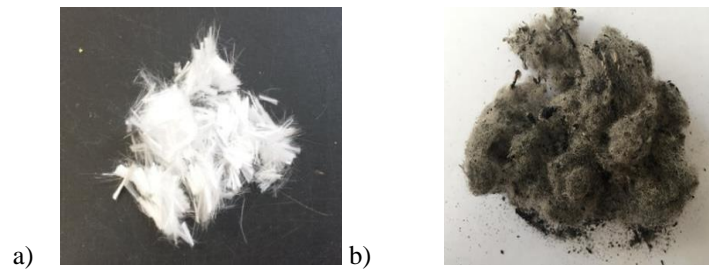


Figure 1. a) Polypropylene fibres b) Recycled tyre polymer fibres

Table 2. Properties of RTPF [8]

Average of length, mm	Diameter, μm		Composition	Melting point, $^{\circ}\text{C}$
8.4 \pm 3.8	type 1	30.93 \pm 2.46	Approx. 60% PET, 25% PA 66 and 15% of PBT, small contribution of steel fibres and high contribution of rubber	210 to 260
	type 2	20.67 \pm 1.75		
	type 3	13.15 \pm 1.82		

2.2 MIX DESIGN, CURING AND TEST METHODS

Experimental programme consisted from four mixes with mix design as shown in Table 3. All mixes are designed to satisfy consistency class S3 (100 - 150 mm) in fresh state.

Table 3. Concrete mix designs

ID	Cement (kg)	Water (l)	Air entrainment (kg)	w/c	Fibres (kg)		Aggregate (kg)		
					PP	RT PF	0-0.125	0-4	4-8
SC-A	470	215	6.91	0.46	-	-	126	1178	201
0.9 PP-A	470	215	6.91	0.46	0.90	-	126	1176	201
0.9 RTPF-A	470	215	6.91	0.46		0.90	126	1181	202
1.8 RTPF-A	470	215	6.91	0.46	-	1.80	126	1179	201

The mixing procedure was as follows: the aggregates and the recycled tyre polymer fibres were mixed together to ensure a good dispersion of fibres. The cement was then added together with water, and mixing started again with continuous addition of the residual water and air entrainment. After the insertion of all materials, the mixing continued for another two minutes. Mixes were transported with mixers and then placed in concrete pump and further sprayed in test panels. For each hardened property test, concrete is sprayed into a test panels made from plywood with dimension 600x600x100 mm (Figure 2).

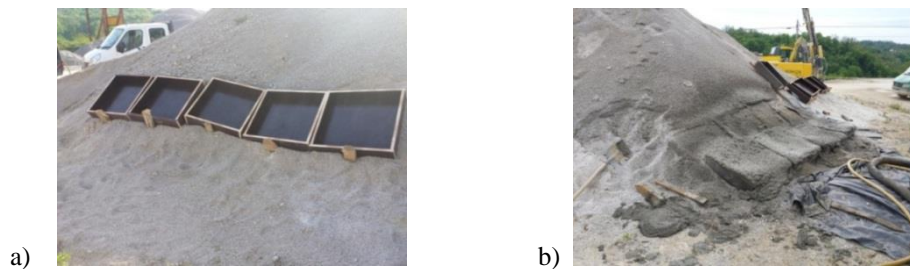


Figure 2. Preparation of in-situ specimens: a) modulus, b) specimens after spraying

After the preparation, specimens were kept in controlled conditions before testing. Due to the large size, specimens were kept outside the humid chamber and after demoulding were continuously cured for 24 days. Test panels were splashed with water every day of curing and covered by plastic foil. At the age of 28 days, specimens for tests were cut by sawing or cored from a test panels. Material close to the side of the formwork could not be well compacted in the spraying process, therefore materials within 50 mm from each side were cut off and discarded before specimens were obtained from the panels. Fresh concrete properties were obtained immediately after mixing and before spraying. Testing of concrete properties in fresh and hardened state was performed according to the standards listed in Table 4. All properties were obtained on 3 specimens from the same mix and the results presented below show the statistical average of these specimens as well as the standard deviation.

Table 4. Test methods for fresh and hardened concrete [10] - [13]

Property	Standard
Density	HRN EN 12350-6:2009
Slump-test	HRN EN 12350-2:2009
Compressive strength	HRN EN 12390-3:2009
Freeze-thaw resistance - scaling	CEN/TS 12390-9 :2006

3 RESULTS

3.1 FRESH STATE PROPERTIES

Table 5 shows the results of fresh concrete testing. All mixes were designed to achieve S3 consistency class (slump values 100 – 150 mm). All fiber reinforced mixes were within the required consistency class, but the reference mix showed a significantly higher consistency implying that fibers had a negative effect on workability (Table 5, Figure 3). Presented results are in line with preliminary results obtained within [5, 8]. Fresh state density of studied mixes was between 2.24 kg/dm³ and 2.27 kg/dm³ (Table 5). The differences are below 2%, indicating that RTPF do not have great influence on concrete density in fresh state. Results are in line with preliminary results obtained within [5, 8].

Table 5. Fresh concrete properties

Concrete mix	Slump (mm)	Density (kg/dm ³)
SC-A	190	2.24
0.9 PP-A	150	2.24
0.9 RTPF-A	140	2.27
1.8 RTPF-A	110	2.24

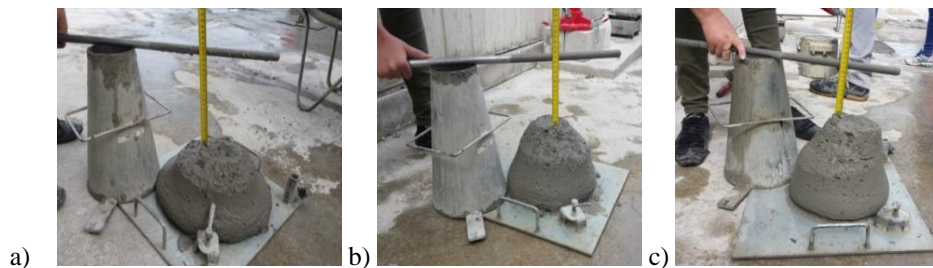


Figure 3 Slump test for concrete with air entrainment: a) without fibres, b) with 0.9 kg/m³ of RTPF c) with 1.8 kg/m³ of RTPF

3.2 COMPRESSIVE STRENGTH

Results of compressive strength testing at the age of 28 days are presented in Figure 4. Obtained compressive strength for all tested mixes ranging from: 26.9 MPa (PC mix) to 38.9 MPa (0.9 RTPF-A).

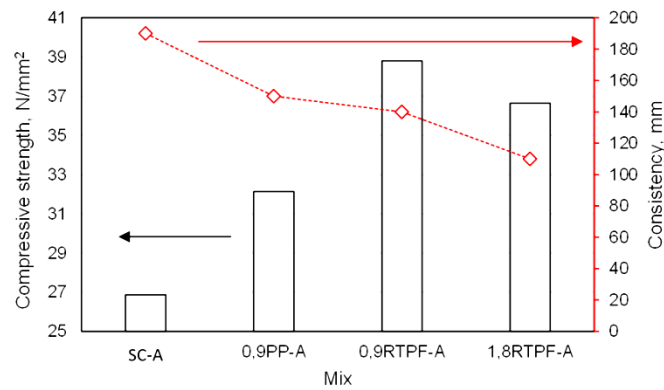


Figure 4. Compressive strength of tested concrete mixes

The difference ranging from 20 % to 40 % implies that the presence of both fibre types had a certain influence on the compressive strength. Compressive strength increased with increase of fibres in the mixes. Regarding fresh state results, fibres had a positive influence on the cohesiveness of the concrete mixes, which was followed by better compaction and consequently higher compressive strengths. These results were in accordance with previous research work [9] which showed that micro-synthetic fibres at low dosages ($< 0.9 \text{ kg/m}^3$) can have a positive effect on cohesion of a mix. Consistency of the fresh concrete mixes should also be considered where, as shown in figure 4, increase of compressive strength is followed by decrease in consistency, i.e. reduced amount of the water inside the mix. When two fibre contents are compared, a small decrease of compressive strength values was obtained at higher fibre content ($> 0.9 \text{ kg/m}^3$). In the case of these mixes, the same trend in compressive strength was observed, with mixes reinforced with fibres (both PP and RTPF) exhibited higher compressive strength compared to PC.

3.3 FREEZE-THAW RESISTANCE - SCALING

Cumulative mass loss obtained for sprayed concrete with air entraining admixture after exposure to 56 cycles of freeze-thaw cycles is presented in figure 5. Results clearly indicated a positive influence of RTPF fibres on reduced amount of scaled material. After 56 cycles, the total amount of cumulative mass loss was 4 times lower for mixes with RTPF. From the literature, it is known that improvement of concrete resistance to freezing and thawing can be achieved by using air entraining admixtures: Air voids act as empty chambers relieving hydraulic pressure and preventing degradation of concrete microstructure [14]. The positive effect of rubber can be attributed to rubber capability to entrap air on their jagged surfaces [15]–[17], and its role as absorber of stresses during freezing of water in the cement matrix. Nevertheless, all tested mixes fulfilled the criteria as described in the standards (the amount of scaled materials $< 0.5 \text{ kg/m}^2$ after 28 and 56 cycles) for the use in both XF2 and XF4 environments.

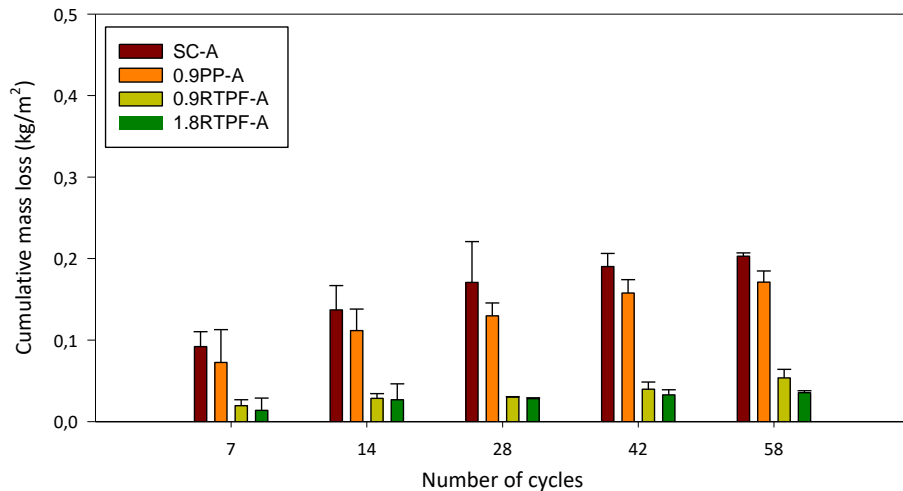


Figure 5 Cumulative mass loss of sprayed concrete mixes with air entrainment due to scaling

4 CONCLUSIONS

The main aim of this experimental study was to investigate whether PP can be substituted with RTPF in sprayed concrete and whether this substitution can have the same influence on concrete freeze-thaw resistance. For this reason, freeze-thaw resistance of mix with lower amount of PP fibres were compared to freeze-thaw resistance of two mixes with the same amount of RTPF and plain sprayed concrete. All mixes were prepared and samples were taken according to standards and procedures concerning sprayed concrete technology.

The first observation concerning application of RTPF in sprayed concrete was that the addition of fibres influenced cohesion and workability of the concrete mixes. With the addition of fibres, mixes tended to have lower consistency, but they were still suitable for pumping and spraying. It was also observed that the procedure of mixing, pumping and spraying of concrete favours shorter fibres such as RTPF, since they potentially get better distributed within the mix. It is interesting to note that compressive strength increases by increasing fibre content. Considering the fresh state properties, it can be assumed that fibres have had a positive influence on cohesiveness of the mix, which was followed by better compaction and consequently higher compressive strengths. The addition of RTPF affected positively the resistance of concrete to scaling during freeze-thaw cycles. This significant positive influence on resistance to freezing and thawing surely has to be attributed also to the rubber found in RTPF, which acts as absorber of stresses during freezing of water inside of pores. This indicates that fine rubber particles could be used instead of the air entraining admixture and may come as an additional benefit when unsorted RTPF are used. All stated, presents a platform for further investigation of RTPF as reinforcement in sprayed concrete.

ACKNOWLEDGMENTS

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ALTERNATIVNI MATERIJALI U SUVREMENIM KONSTRUKCIJAMA

Rezime:

Zaštita okoliša i ušteda energije postaju svjetski problemi u svim poljima tehnologije pa tako i u graditeljstvu, u kojem se traže nova rješenja kroz projektiranje, gradnju i sve više kroz uporabu alternativnih materijala. Alternativni materijali primjenjuju se kao zamjena cementu i/ili dodatak betonu te kao zamjenska ili nova komponenta sastava betona. Kao zamjena dijela agregata najviše se upotrebljavaju frakcije agregata nastale recikliranjem betona, ali i materijali koji su sekundarne sirovine metalurške industrije. U radu je dan pregled primjene letećeg pepela kao alternativnog materijala za zamjenu cementa i/ili dodatka betonu. Ujedno su prikazane građevine u svijetu kod čije je gradnje primijenjen leteći pepeo.

Ključne reči: alternativni materijali, građevinarstvo, leteći pepeo, beton

ALTERNATIVE MATERIALS IN CONTEMPORARY CONSTRUCTIONS

Summary:

Environmental protection and energy saving are becoming the world's problems in all fields of technology and therefore in construction, where new solutions are finding through design, construction and more and more through use of alternative materials. Alternative materials are applied as a cement replacement and/ or concrete admixture and as a replacement or a new component of the concrete composition. As a replacement of the part of aggregate are mostly used aggregate fractions obtained by concrete recycling, but also secondary raw materials from metallurgical industry. Overview of fly ash as an alternative material for cement replacement and / or concrete admixture is presented in the paper. Buildings in the world constructed by using alternative materials are also presented in the paper.

Key words: alternative materials, construction, fly ash, concrete

S-100

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1 UVOD

Razvijene zemlje Europske unije još krajem prošlog stoljeća razvijaju strategije razvoja u graditeljstvu (slika 1) [1] u kojima je prioritet održivi razvoj. Osnovni princip održivog razvoja u graditeljstvu je korištenje što manje prirodnih resursa i energije te stvaranje što manje po Zemlju štetnog otpada. Zaštita okoliša i ušteda energije postaju svjetski problemi u svim poljima tehnologije pa tako i u graditeljstvu u kojem se razvijaju novi koncepti projektiranja i izvedbe građevina. U području industrije betona koja je najveći potrošač prirodnih resursa, građevni je otpad prepoznat i određen od strane Europske komisije kao jedan od prioritarnih smjerova za rješavanje problema prekomjernog trošenja prirodnih neobnovljivih izvora. Odgovarajuće upravljanje građevinskim otpadom rezultiralo bi učinkovitim korištenjem prirodnih resursa i ublažavanjem utjecaja graditeljstva na okoliš.



Slika 1 - Ciljevi održivog razvoja u građevinarstvu [1]

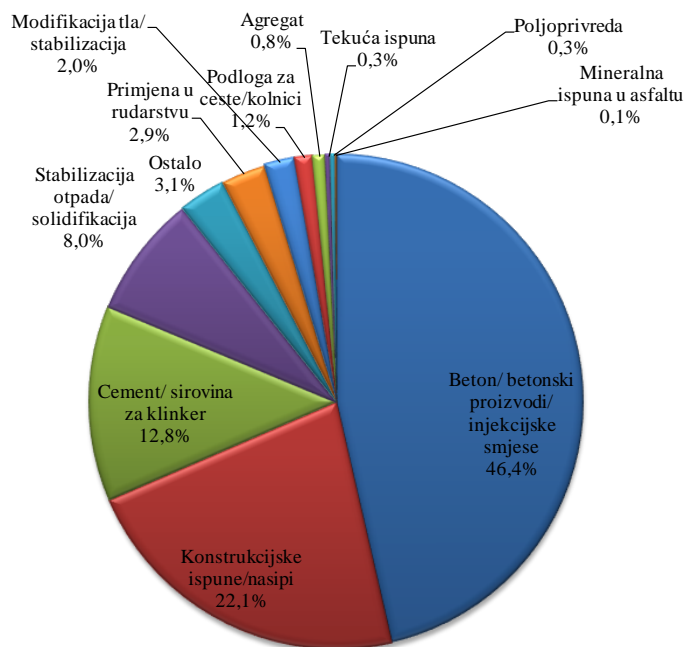
U istraživanjima u području tehnologije betona napravljen je iskorak u smjeru uporabe recikliranog agregata te se on može koristiti kao zadovoljavajuća zamjena za agregat iz prirodnih izvora [2]. Poznavanjem mehaničkih i trajnosnih svojstava betona izrađenih od recikliranog agregata mogu se pronaći prikladni načini njegove primjene, a time bi se povećala vrijednost materijalu koji se primjenjuje za nasipavanje ili se odlaže na odlagališta otpada. Jedan od načina primjene recikliranog betonskog loma i reciklirane opeke ili pak miješanog recikliranog agregata u betonu jest zasigurno u zgradarstvu za koje s povećanjem populacije postaje sve veća potreba. Građevinarstvo te u sklopu njega industrija betona mora odgovoriti izazovima rasta populacije (150.000 novih stanovnika na dan/globalno) i shodno tome potrebama za građevinama i građevnim materijalima. Sve to ima utjecaja na globalno zatopljenje, tako da zahtjevi održivog razvoja [3] na razini samog materijala i proizvodnje te njegovih svojstava u konstrukciji, postaju primarni zahtjevi za građevinu.

Alternativni materijali u betonu su pretežito industrijski nusproizvodi u obliku fino usitnjenog materijala koji se dodaju kao djelomična zamjena cementu radi poboljšanja određenih svojstava i/ili za dobivanje nekih posebnih svojstava betona. Pojedini od tih materijala već se godinama primjenjuju ili kao dodatak cementu ili kao zamjena cementu u sastavu betona, kao npr. leteći pepeo, čeličanska zgura, silicijska prašina i metakaolin. Neke od alternativnih materijala počelo se primjenjivati nedavno, kao što je primjerice mulj iz otpadnih voda i pepeo rižinih ljuskica, a neki su još u fazi istraživanja kao npr. pepeo iz biomase, zgura iz proizvodnje bakra ili cinka, crveni mulj, otpad iz proizvodnje feronikla, mulj iz proizvodnje papira, fosfogips, ljevački pijesak te mulj iz rezališta kamena [4 – 22].

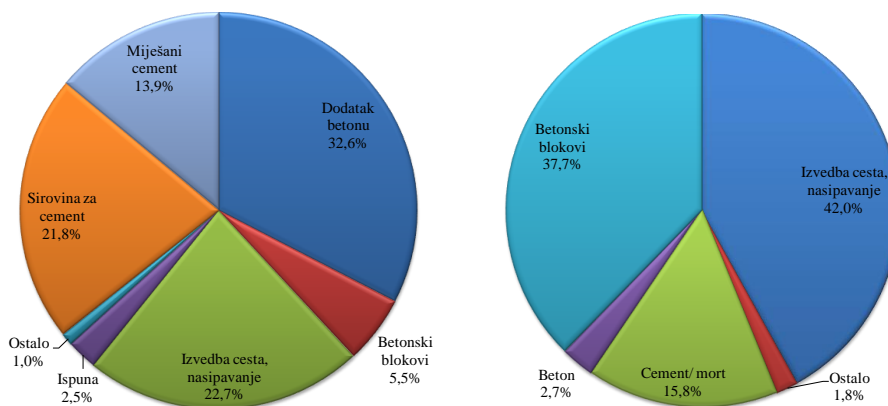
U ovom radu dan je pregled primjene letećeg pepela kao alternativnog materijala za zamjenu cementa i/ili dodatka betonu. Ujedno su prikazane građevine u svijetu kod čije je gradnje leteći pepeo kao alternativni materijal našao svoju primjenu.

2 LETEĆI PEPEO KAO ZAMJENA ZA CEMENT I/ILI DODATAK BETONU

Leteći pepeo je fini prah dobiven izgaranjem ugljene prašine u termoelektranama loženim na ugljen, koji je pomoću elektrostatskih filtarskih uređaja separiran iz otpadnih plinova. Leteći pepeo predstavlja materijal kojeg je potrebno zbrinuti. U 2006. godini je u SAD-u upotrijebljeno samo 45 % proizvedene količine letećeg pepela, približno 32,4 milijuna tona po područjima kako je prikazano na slici 2 [23], a u Europi (EU 15) je u 2008. godini upotrijebljeno približno 20 milijuna tona kako je prikazano na slici 3 [24].



Slika 2 - Primjena letećeg pepela u SAD-u 2006. [23]



Slika 3 - Primjena letećeg pepela i pepela s dna peći u industriji betona u EU 15 u 2008. godini [24]

Leteći pepeo iz termoelektrane Plomin u Republici Hrvatskoj koristi se kao sirovina u proizvodnji cementa u tvornici Holcim. Holcim je od TE Plomin u proteklih deset godina preuzeo čak milijun tona otpada, odnosno letećeg pepela i gipsa, što je korišteno u procesu proizvodnje u cementari u Koromačnom u RH. Prema [25], cilj TE Plomin je nula deponiranih nusproizvoda. Recesija i pad tržišta građevinske industrije ne pridonose dostizanju toga cilja te je potrebno pokrenuti nove kanale za stavljanje na tržište i uporabu navedenih materijala. Uz dobro definiranu zakonsku regulativu, kvalitetan proizvod, implementiranje sustava kontrole kvalitete u proizvodnji i transparentnim odnosima s kupcima moguće je u bliskoj budućnosti pronaći nove načine za zbrinjavanje ovih vrijednih materijala. Kao što je već navedeno, primjena letećeg pepela ima ekološke prednosti (smanjenje emisije CO₂) te ekonomske, jer cijena cementa i betona može biti manja, a također i tehnološke kroz niz poboljšanih svojstva betona. Sve to poznato je u inženjerskoj praksi već sredinom prošlog stoljeća kada su tehnolozi betona u suradnji s projektantima i izvođačima građevina počeli primjenjivati leteći pepeo u sastavu cementa i betona. Ako se projektira mješavina betona s letećim pepelom i superplastifikatorom bit će upotrijebljeno manje vode i samim time beton će biti manje sklon skupljanju, propusnosti i pojavi pukotina te će biti dugotrajniji te na taj način pogodan za građevine kao što su mostovi i brane.

Od 1950. do 1970. godine beton s letećim pepelom (i do 50 %) primijenjen je za gradnju čak 100 brana u svijetu. Jedna od prvih primjena letećeg pepela u betonu je za gradnju masivne lučne brane Hungry Horse u Montani između 1948. i 1953. godine gdje je upotrijebljeno 132.276 tona letećeg pepela (slika 4) [26]. Leteći pepeo također je primijenjen u betonu za izgradnju brane Lower Notch na jezeru Ontario u Kanadi (slika 5) [27] kako bi se smanjila alkalnoagregatna reakcija u betonu brane koja je izgrađena 1969 godine. Pregledom brane 1996. i 2006. godine ustanovljeno je da unatoč primijenjenom reaktivnom agregatu i visoko alkalnom cementu u betonu s letećim pepelom nije bilo značajnih oštećenja niti nakon 35 godina.

Pri gradnji riječnog tunela St. Clair od 1993. do 1994. godine između Sarnie na Ontariu, Kanada i luke Huron u Michiganu, SAD (slika 6a), upotrijebljen je leteći pepeo u betonu kako

bi se postigli specificirani zahtjevi za beton. Naime voda u tlu je sadržavala kloride (4000 ppm) i sulfate (155 ppm) i u kombinaciji s djelovanjem hidrostatskog tlaka projektirani su sljedeći zahtjevi za beton:

- sadržaj veziva 400 do 550 kg/m³
- tlačna čvrstoća nakon 28 dana ≥ 60 MPa
- koeficijent difuzije nakon 120 dana $D_a \geq 600 \times 10^{-15}$ m²/s
- vodopropusnost nakon 40 dana $k \leq 25 \times 10^{-15}$ m/s.

Propisane zahtjeve zadovoljio je sastav betona s 30 % letećeg pepela i 6 % silicijske prašine (SiO₂) i vodovezivnim omjerom 0,29 do 0,32.



Slika 4 - Brana Hungry Horse u Montani, SAD [26]



Slika 5 - Brana Lower Notch, Ontario, Kanada [27]

Leteći pepeo bio je korišten i za gradnju Euro tunela od 1987. do 1994. godine, tada drugog najdužeg tunela u svijetu i građenog predgotovljenim betonskim elementima (slika 6 b, c). Beton predgotovljenih elemenata sadržavao je 30 % letećeg pepela uz 440 kg/m³ veziva i primjenu superplastifikatora. Nakon 28 dana postignuta je čvrstoća od 80 MPa i koeficijent propusnosti od 10 m/s.



a)



b)



c)

Slika 6 - Tuneli, a) St. Clair riječni tunel [27], b) Eurotunel [28, 29]

Podmorski tunel Bjørvika duljine je 1100 m od čega je 650 m ispod razine mora u Oslu, Norveška, a pušten je u promet 2010. godine (slika 7) [30]. Za izradu tunelskih elemenata upotrijebljeno je 70 000 m³ betona s 26 600 tona cementa i 30 % letećeg pepela pri čemu je za 8000 tona smanjena emisija CO₂ u usporedbi s betonom bez letećeg pepela. Betoni za izgradnju brane na rijeci Temzi u Londonu (slika 8) [31], koja je puštena u rad 1984. godine sadržavali su 50 % letećeg pepela kako bi beton bio otporan na eventualna zagađenja vode, eroziju i eflorescenciju.

Pri ojačanju od potresa gradske vijećnice u Utahu, SAD 2006. godine, 4500 m³ betona napravljeno je s velikim udjelom letećeg pepela te je za 900 tona smanjena emisija CO₂ (slika 9) [32]. Kod izgradnje zgrade CITRIS, University of California, Berkeley, SAD, 2007. godine, (slika 10) [32] za izradu jako armiranih temelja, zidova i stupova upotrijebljeno je 7000 m³ betona koji je izrađen s cementom s 50 % klinkera i 50 % letećeg pepela.



Slika 7 - Bjørvika tunel u gradnji, Oslo, Norveška [30]



Slika 8 - Brana na rijeci Temzi, London Engleska [31]



Slika 9 - Izvedba ojačanja na potres na zgradi gradske vijećnice, Utah, SAD [32]



Slika 10 - Izgradnja CITRIS Building, University of California, Berkeley, SAD [32]

Ušteda energije kao jedan od postulata održive gradnje postignuta je na građevini kompjuterskog centra York sveučilišta u Torontu, Ontario, Kanada (slika 11) [27] kod koje su uz sve ostale mjere betoni projektirani i izvedeni kao betoni s velikim udjelom letećeg pepela od 50 %.



Slika 11 - Zgrada kompjuterskog centra York sveučilišta, Toronto [27]

Kod izgradnje vrlo visokih građevina također je korišten leteći pepeo u sastavu betona, kao npr. pri gradnji tornjeva Petronas u Kuala Lumpuru u Maleziji završenih 1998. godine (slika 12a), koji su izgrađeni s betonima od 60 i 80 MPa uz udio letećeg pepela od 37,5 % [27]. Sljedeći primjer je zgrada Burj Kalifa u Dubaiju, UAE (slika 12b) trenutno najviša građevina na svijetu koja je građena samozbijajućim betonom (*engl. self compacting concrete, SCC*) s mineralnim dodacima, 25 % letećeg pepela i 7 % silicijske prašine te kemijskim dodacima za povećanje viskoznosti [33, 34].



a)



b)

Slika 12 - Visoke zgrade, a) Tornjevi Petronas, Kuala Lumpur, Malezija [27] b) Burj Kalifa, Dubai, UAE [33]

Leteći pepeo upotrebljava se kao mineralni dodatak i za betone izložene agresivnom djelovanju morskog okoliša, kao npr. pri betoniranju mosta u Tampa zaljevu, Sunshine Skyway Bridge, Florida, SAD (slika 13) [35] kao i u betonima mosta New Liberty Memorial Bridge, (Slika 14) [36] preko rijeke Missouri, SAD izgrađenom na mjestu starog čeličnog mosta i puštenom u promet 2008. godine.



Slika 13 - Tampa Bay's Sunshine Skyway Bridge, Florida, USA [35]



Slika 14 - New Liberty Memorial Bridge, SAD [36]

3 ZAKLJUČAK

Kako su nove vrste i tehnologije betona uglavnom zasnivaju na primjeni alternativnih materijala bilo u cementu i/ili betonu, potrebno je dati osvrt na kojim dijelovima svijeta je to trenutno najbitnije. Analizirajući proizvodnju cementa, to su zemlje azijskog kontinenta, koje su u usponu po proizvodnji cementa, a shodno tome su i potencijalni zagađivači. Sretna okolnost je da se trenutno veliki dio istraživanja u području građevinskih materijala odvija vrlo intenzivno u tom dijelu svijeta. Razlog tome je da upravo to područje svijeta i raspolaže s velikim količinama alternativnih materijala koji se mogu upotrijebiti u industriji betona. Za nadati se da će budući prikaz proizvodnje cementa i zagađenja zraka biti optimističan, odnosno unatoč povećanoj proizvodnji cementa u tom dijelu svijeta, uz pridržavanje zahtjeva održivosti trebalo bi biti smanjenje zagađenja. U Hrvatskoj je u doba recesije proizvodnja cementa i betona smanjena, no upravo sada bi trebalo biti vrijeme za pripremu i usvajanje novih materijala i tehnologija koje bi vodile ekološki održivoj industriji betona.

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METODOLOGIJA ZA PRORAČUN TROŠKOVA NEUPOSLENE MEHANIZACIJE U OKVIRU ODŠTETNIH ZAHTEVA

Rezime:

Jedan od najznačajnijih troškova u odštetnim zahtevima je trošak građevinske mehanizacije na stajanju. Postoje različite metodologije za proračun ovih troškova koje se baziraju na empirijskim formulama, priručnicima proizvođača, iskustvenim podacima i naučnoj literaturi. S obzirom da rezultat proračuna troškova varira u odnosu 1:3 u zavisnosti od primenjene procedure, postoji potreba za formulisanjem nepristrasne metodologije čijom primenom bi se izbegli sporovi koji se vrlo često javljaju po ovom pitanju između izvođača i investitora. U ovom radu izvršena je analiza postojećih pristupa i predložena je metodologija za proračun troškova neuposlene građevinske mehanizacije u vidu algoritma sa formulama za svaki od bitnih elemenata troška.

Ključne reči: mehanizacija, amortizacija, kamata, osiguranje, odštetni zahtev

IDLE EQUIPMENT COST CALCULATION METHODOLOGY FOR DELAY CLAIMS

Summary:

One of the most significant costs within delay claims is idle equipment cost. There are different approaches to evaluating cost of unproductive/idle equipment based on empirical formulas, manuals, historical data and science literature. Since different approaches result with very different results (1:3 ratio), there is a need for formulating an unbiased methodology for evaluating such costs in order to avoid claim disputes. Based on analysis of existing approaches, this paper presents a methodology for calculating idle equipment costs in a form of an algorithm and formulas for every cost element.

Key words: construction equipment, depreciation, interest, insurance, claim

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1 UVOD

Na građevinskim projektima često se javljaju odštetni zahtevi izvođača zbog zastoja koji nastaju krivicom investitora. Jedna od najznačajnijih kategorija troškova koji se mogu nadoknaditi u okviru takvih odštetnih zahteva je trošak građevinske mehanizacije na stajanju. U praksi se mogu sresti različite metodologije za proračun ovih troškova koje se baziraju na empirijskim formulama, priručnicima proizvođača, iskustvenim podacima i naučnoj literaturi. U zavisnosti od primenjene metodologije, proračun ukupnih troškova mehanizacije na stajanju varira u odnosu 1:3. Ova činjenica ukazuje na potrebu za formulisanjem nepristrasne metodologije čijom primenom bi se izbegli sporovi koji se vrlo često javljaju po ovom pitanju između izvođača i investitora u okviru odštetnih zahteva. U ovom radu analiziraće se postojeći pristupi i predložiti objektivna metodologija za proračun troškova građevinske mehanizacije na stajanju.

2 PREGLED LITERATURE

Postoje različite kategorije literature u kojoj se obrađuje tema troška građevinske mehanizacije na stajanju: naučna literatura (udžbenici i naučni radovi) i publikacije koje objavljuju proizvođači opreme i strukovne organizacije.

2.1 NAUČNA LITERATURA

Što se tiče udžbenika iz oblasti građevinske mehanizacije, pristup je sličan. Troškovi građevinske mehanizacije na stajanju jednaki su troškovima vlasništva (*cost of ownership*) i nesporno obuhvataju sledeće kategorije:

- amortizacija,
- kamata,
- osiguranje.

Tabela 1. – Pregled formula za proračun amortizacije

METODA	FORMULA
„prava linija“ (<i>straight-line method</i>)	$D_n = \frac{V - SV - TC}{N}$
„suma godina“ (<i>sum-of-years method</i>)	$D_n = \frac{(\text{year}^n \text{ digit})}{1 + 2 + \dots + N} * (V - SV - TC)$
„dvostruko opadajuća“ (<i>double declining balance method</i>)	$D_n = \frac{2}{N} * (BV_{n-1} - TC)$
„potapajuća“ (<i>sinking fund method</i>)	$D_n = C * (1 + i)^{N-1}; C = (V - SV) * \left[\frac{i}{(1 + i)^N - 1} \right]$

Gde su: V-nabavna vrednost mašine; SV-rezidualna vrednost; TC-cena pneumatika; N-ekonomski vek u godinama;

U pojedinim udžbenicima postoje i drugi elementi troška. Na Građevinskom fakultetu u Beogradu predlaže se trošak investicionog održavanja (velike popravke, mimo redovnog

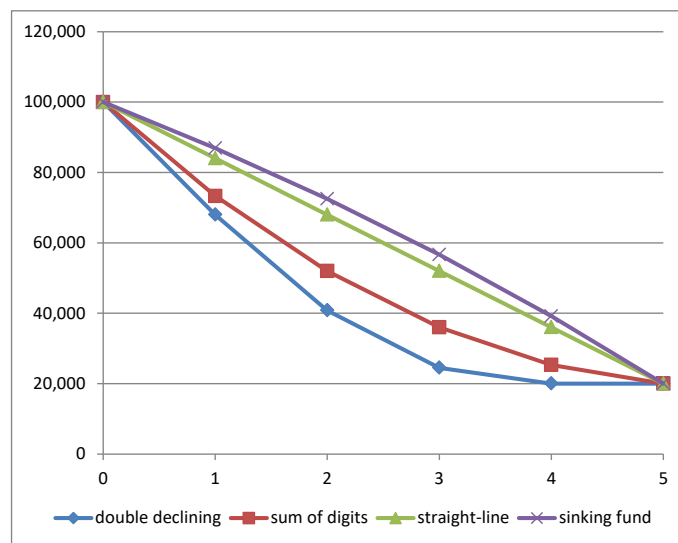
održavanja) [1]. Gransberg i ostali predlažu troškove skladištenja i drugih taksi (osim osiguranja) [2]. Osim vrste troška, u udžbenicima se daju formule koje se mogu koristiti za proračun pojedinih elementa.

Amortizacija

Amortizacija je trošak koji se, u okviru naplate radnog sata mašine, vremenom akumulira radi nabavke nove mašine. Amortizacija kao fizička kategorija nastaje zato što tokom vremena mašina postaje neupotrljiva ili zastarela. Amortizacija je u isto vreme i knjigovodstveni trošak koji se obračunava na nivou preduzeća vlasnika mehanizacije na godišnjem nivou. Postoje različite formule za obračun troška amortizacije [2]. Jedan od boljih pregleda formula daju Gransberg i ostali [3]:

Rezidualna vrednost predstavlja vrednost mašine na kraju ekonomskog veka. Vrednosti za rezidualnu vrednost mogu biti: 20% nabavne vrednosti, vrednost starog gvožđa, nula isl. Preporuke o tome koliko stvarno treba da bude rezidualna vrednost mogu se naći u literaturi (nešto više na kraju ovog poglavlja).

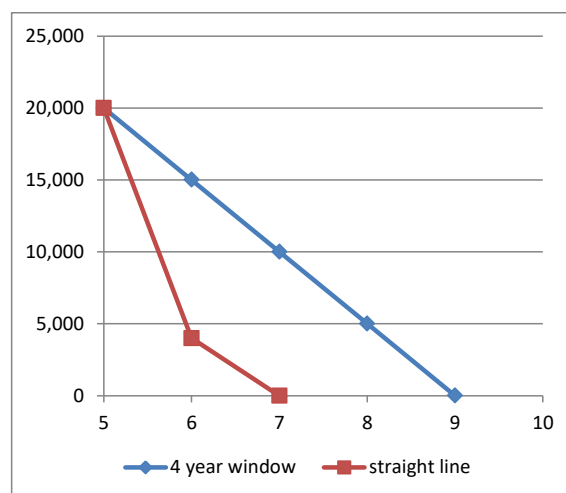
Na slici 1 prikazane se različite metode za amortizaciju na primeru mašine nabavne vrednosti 100,000€ rezidualne vrednosti 20%V i ekonomskog veka od 5 godina. Metode se razlikuju po trajektoriji amortizacije kojom se, kod svih metoda, polazi od nabavne vrednosti i na kraju ekonomskog veka stiže do rezidualne vrednosti.



Slika 1. – Ilustracija formula za amortizaciju

Najbitnije je uočiti da se proces amortizacije završava na kraju ekonomskog veka na rezidualnoj vrednosti. Preostala vrednost se može dodatno amortizovati u dodatnom periodu koji, u zavisnosti od primenjene metode proračuna amortizacije, može da traje fiksni ili proizvoljni broj godina.

Na slici 2 prikazane su dve metode za preostalu amortizaciju nakon rezidualne vrednosti za mašinu iz prethodnog primera: fiksni broj godina (u primeru 4 godine) i metoda prave linije po kojoj se amortizacija nastavlja istom tempom dok se potpuno ne završi.



Slika 2. – Amortizacija nakon rezidualne vrednosti

Može se zaključiti da je, po bilo kojoj metodologiji, amortizacija konačan proces koji je posle određenog broja godina u potpunosti završen što znači da su prikupljena sredstva za nabavku nove mašine. U analiziranoj literaturi nije posebno istaknut slučaj o novčanoj vrednosti troška amortizacije za starije mašine kod kojih je proces amortizacije završen.

Kamata

Ovaj trošak predstavlja kamatu koju preduzeće plaća prilikom kupovine mašine putem kredita. Može se definisati i kao trošak vezanog kapitala u visini vrednosti nove mašine [3].

Slično kao za amortizaciju, u literaturi se mogu naći različite formule. U tabeli 2 prikazane su neke od češće korišćenih formula.

Sve formule koje su navedene u tabeli 2 su korektne i daju relativno slične rezultate. Međutim, iz prirode formula se može uočiti da se odnose na period od nabavke nove mašine (V) do amortizacije na rezidualnu vrednost (SV). Isto kao kod formula za amortizaciju, postavlja se pitanje vrednosti ovog troška posle završene amortizacije. U analiziranoj literaturi nisu pronađene eksplicitne formule za ovaj slučaj.

Osiguranje, takse

Osiguranje je trošak koji se odnosi na slučaj požara, krađe i nezgoda i obračunava se kao premija na godišnjem nivou [3]. U ovu vrstu troška treba obračunati sve takse koje su vezane za vlasništvo mašine, uključujući i troškove skladištenja (koji se obično ne priznaju u okviru odštetnih zahteva ali realno postoje).

Trošak osiguranja je najegzaktniji trošak vezan za vlasništvo mašine, zato što se može dokazati konkretnim uplatama koje su bile izvršene u ovu svrhu. Drugi način obračuna troška osiguranja je korišćene sličnih formula kao za kamatu, s tim što se procenat za kamatu u formulama iz tabele 2 (i) zamenjuje procentom za osiguranje. Ovaj procenat obično iznosi približno 2% na godišnjem nivou.

Tabela 2. – Pregled formula za proračun kamate

METHOD	FORMULA – average annual cost of interest
Gransberg, Popescu & Ryan	$In = \frac{V(N+1) + SV(N+1)}{2N} * i$
US Army Corps of Engineers (jednako prethodnoj)	$In = \frac{(N-1)(1+SV/V) + 2}{2N} * V * i$
Caterpillar (jednako prethodnoj ako je SV=0)	$In = \frac{N+1}{2N} * V * i$
Peurifoy & Schexnayder (interest = total investment cost – depreciation)	$In = \left[V \left(\frac{(1+i)^N}{(1+i)^N - 1} \right) - \frac{V}{N} \right] - \left[SV \left(\frac{i}{(1+i)^N - 1} \right) - \frac{SV}{N} \right]$

U naučnim radovima u priznatim časopisima ova tematika se retko obrađuje. Mogu se pronaći radovi koji se odnose na ispitivanje uticaja rezidualne vrednosti na troškove mehanizacije [4] ili na upotrebu statističkih metoda (linearna regresija i *data mining*) za preciznije određivanje rezidualne vrednosti [5], [6]. Istraživanje se vrše i u oblasti određivanja optimalnog trenutka za zamenu mašine novom tj. odnosa troškova amortizacije i troškova popravki tokom životnog veka mašine [7], [8], [9]. Još jedna od oblasti naučnog istraživanja je u oblasti izbora optimalne kombinacije građevinske mehanizacije na konkretnom projektu [10], [11].

Ali nisu pronađena istraživanja u vezi proračuna troškova mehanizacije na stajanju, verovatno zato što se smatra da ta tema predstavlja opšte znanje i da je dovoljno obrađena u okviru udžbenika.

2.2 PUBLIKACIJE PROIZVOĐAČA I STRUKOVNIH ORGANIZACIJA

Što se tiče publikacija proizvođača i strukovnih organizacija ova tematika se obrađuje na nešto drugačiji način. Osim formula koje su slične kao u udžbenicima (Caterpillar, Volvo itd.) navode se i empirijske vrednosti za različite elemente troška. Na primer, U.S. Army Corps of Engineers [12]. definiše sopstvenu metodologiju proračuna sa tabelarnim pregledom vrednosti za ekonomski vek, rezidualnu vrednost itd. Volvo je razvio kalkulator troškova (*Volvo Owning and Operating Cost Template*) za proračun troškova sa sopstvenim iskustvenim vrednostima za pojedine elemente proračuna [13]. Caterpillar je razvio sopstveni Caterpillar online univerzitet sa kursovima na razne teme, pa i u vezi troškova mehanizacije [14]. Kao i u slučaju udžbenika, nema posebnih formula ili uputstava za proračun pojedinih elemenata troška u zavisnosti od starosti mašine.

2.3 BGL PROCEDURA

BGL je nemački nacionalni servis (baza podataka) za građevinsku mehanizaciju sa tehničkim i finansijskim podacima o pojedinim tipovima mašina (*“BauGeräteListe”*) [15]. Između ostalog BGL se koristi za: interno knjigovodstvo, proračun koštanja mehanizacije po

času ili jedinici mere, procenu učinka, planiranje i upravljanje mehanizacijom, procenu vrednosti vezano za osiguranje i sudske procese. Mašine su podeljene u 24 kategorije sa svojim pod-kategorijama [16].

U okviru BGL-a formulisana je metodologija za proračun troška građevinske mehanizacije na stajanju. Ova procedura je izdvojena u posebno poglavlje zato što se vrlo često može sresti u upotrebi na realnim projektima u okviru odštetnih zahteva. Razlog za raširenu upotrebu je veoma jednostavan – BGL procedura daje najveće vrednosti za koštanje građevinske mehanizacije na stajanju pa je veoma popularna kod izvođača koji formiraju odštetne zahteve.

BGL procedura proračuna podrazumeva upotrebu empirijske formule:

$$K = \frac{100}{v} + \frac{p * n * 100}{2 * v} = \frac{100}{v} \left(1 + \frac{p * n}{2}\right) \quad (1)$$

Gde je K-mesečni procenat; v-ekonomski vek u mesecima; p-procenat za kamatu na godišnjem nivou (6,5%); n-starost mašine u mesecima. Ukupan mesečni trošak izražen u novcu je $K * V$.

Prvi sabirak formule se odnosi na proračun amortizacije i predstavlja procentaulnu vrednost amortizacije na mesečnom nivou. Problem je što ovaj procenat ne zavisi od starosti mašine pa važi i za mašine kod kojih je proces amortizacije već završen tj. amortizacija je naplaćena u potpunosti.

Drugi deo formule se odnosi na empirijsku vrednost za kombinovanu vrednost kamate i osiguranja. Problem je što se u formuli starost mašine n nalazi u gornjem delu razlomka, pa vrednost troška raste sa starošću mašine. Ovo rezultuje nelogičnim rezultatima: što je mašina starija, trošak za kamatu i osiguranje je veći. Ovo je u suprotnosti sa logikom i sa priznatim formulama iz udžbenika.

Može se zaključiti da BGL procedura proračuna nije odgovarajuća jer favorizuje izvođače. Posredno to potvrđuje i činjenica da postoje presude sa arbitraža na kojim je izvođačima priznavan deo proračunatog troška po BGL proceduri (oko 70%), s tim da arbitraža nije utvrdila razloge već samo posredno priznala da nešto nije u redu. I sami autori su posredno priznali da postoji nelogičnost tako što su u novijoj verziji BGL procedure modifikovali formule. U delu 8.4 uveden je koeficijent redukcije na sledeći način:

Od prvog do desetog dana zastoja naplaćuju se puni troškovi po navednoj formuli (1)

Od jedanaestog dana na dalje naplaćuje se 75% amortizacije i kamate i 8% amortizacije i kamate za troškove održavanja (ukupno 83% amortizacije i kamate).

Može se zaključiti da je navedena korekcija paušalne i empirijske prirode i da nije u skladu sa formulama iz udžbenika a nije ni u skladu sa pomenutim umanjenjem (~70%) sa održanih arbitraža. Primenom ove korekcije i dalje se dobijaju znatno veći rezultati u odnosu na formule iz poglavlja 2.1. Imajući to u vidu, postavlja se pitanje zašto uopšte koristiti korigovanu empirijsku formulu kada u udžbenicima postoje korektne i priznate formule za proračun pojedinih elemenata troška mehanizacije na stajanju? U svakom slučaju upotreba BGL procedure je diskutabilna sa naučne i stručne tačke gledišta i, radi izbegavanja skupih i dugotrajnih procesa arbitraže, razumnije je u praksi koristiti naučno dokazane i priznate formule (iz poglavlja 2.1). Iz ugla investitora ovo je lako sprovesti u praksi definisanjem odgovarajućeg člana ugovora, čime bi pravila u vezi obračuna troškova mehanizacije na stajanju postala obavezujuća za obe strane.

3 PREDLOŽENA METODOLOGIJA

Na osnovu prethodne analize, u ovom radu se predlaže metodologija za obračun troškova građevinske mehanizacije na stajanju koja se može objektivno primenjivati i u okviru odštetnih zahteva. Predložena metodologija predstavlja dopunu postojećih pristupa koji su opisani u priznatim udžbenicima i objašnjeni u poglavlju 2.1. Dopuna se odnosi na uvođenje posebnih setova formula koje se primenjuju u zavisnosti od starosti mašine. Ova dopuna ima određeni značaj u naučnom pogledu ali se najveći značaj ogleda u praktičnom smislu primene na projektima. Primenom unapred propisanih setova formula izbegli bi se brojni sporovi oko kvantifikacije troška mehanizacije na stajanju u odštetnim zahtevima koji u današnjoj praksi iznose 1:3 između viđenja izvođača i investitora.

Setovi formula se razlikuju za tri karakteristična perioda u životnom ciklusu mašine:

- Mašina je u okviru ekonomskog veka
- Mašina je starija od ekonomskog veka ali nije još uvek u potpunosti amortizovana
- Mašina je starija od ekonomskog veka i u potpunosti amortizovana

Amortizacija

Ukoliko je mašina još uvek u okviru ekonomskog veka, amortizacija postoji kao realan trošak i može se računati po nekoj od priznatih formula koje su prikazane u tabeli 1. Preporučuje se metoda prave linije kao najjednostavnija a istovremeno dovoljno dobra za potrebe odštetnih zahteva. Ukoliko je prošao ekonomski vek a mašina još uvek nije u potpunosti amortizovana, za obračun troška može se primeniti metoda prave linije ili konačnog broja godina, koje su prikazane na slici 2. Najvažnija komponenta metodologija se odnosi na mašine koje su u potpunosti amortizovane. U tom slučaju trošak za amortizaciju ne postoji i ne treba da se naplaćuje. Navedena dopuna ima veliki značaj kod obračuna troška mehanizacije na stajanju jer se upravo tu javlja najveća razlika u obračunima.

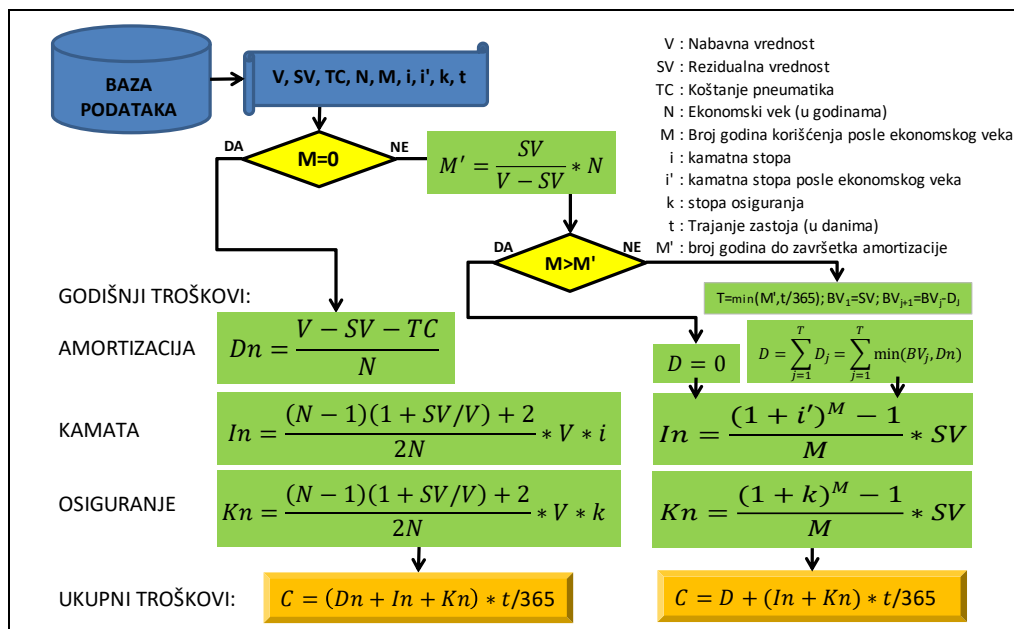
Kamata

Ukoliko je mašina u okviru ekonomskog veka, troškovi kamata mogu se računati po nekoj od formula koje su prikazane u tabeli 2. Ukoliko je prošao ekonomski vek a mašina još uvek nije u potpunosti amortizovana onda se kamata računa po istim formulama ali se umesto nabavne vrednosti mašine V u formuli koristi rezidualna vrednost SV . Ako je amortizacija završena, kapital koji je vezan u mašini jednak je nuli, pa je i trošak za kamate jednak nuli. Razumna je pretpostavka da je i slučaju nabavke mašine na kredit u trenutku završene amortizacije mašina već otplaćena, što dovodi do istog zaključka (trošak za kamatu jednak je nuli).

Osiguranje

Osiguranje predstavlja egzaktni trošak. Može se dokazati plaćenim računima. Ukoliko se proračunava može se koristiti slična formula kao za proračun kamate, sa odgovarajućom stopom za obračun osiguranja.

Navedene dopune su prikazane na slici 3 u vidu preporučenih formula. Ulazni podaci su osnovni podaci o mašini (nabavna vrednost V , rezidualna vrednost SV , cena pneumatika TC , ekonomski vek u godinama N , godina proizvodnje na osnovu koje se sračunava broj godina proteklih nakon ekonomskog veka M , procenti za godišnje naknade za kamatu i osiguranje i i k). Ulazna veličina je i i broj dana zastoja t .



Slika 3. – Predložena metodologija

Može se uočiti da postoje dve grane za proračun, koje zavise od starosti mašine. Leva grana se odnosi na mašine koje se još uvek nalaze u okviru ekonomskog veka ($M=0$), sa odgovarajućim formulama za proračun amortizacije, kamate i osiguranje na godišnjem nivou. Formule za kamatu i osiguranje se razlikuju samo u procentu za godišnju naknadu (i za kamatu; k za osiguranje).

Desna grana se odnosi na mašine starije od ekonomskog veka. U okviru amortizacije se razlikuje period do potpune amortizacije ($M > M'$) i posle njega ($M < M'$). U prvom slučaju amortizacije nema ($D=0$), dok se u drugom slučaju amortizacija nastavlja metodom prave linije i proračunava se u zavisnosti od trajanja zastoja ($t/365$) i rezidualne vrednosti koja je preostala za amortizaciju.

Za proračun kamate i osiguranja na godišnjem nivou predložena je identična formula sa odgovarajućim procentom godišnje maknade (i' i k).

4 STUDIJA SLUČAJA

Studija se odnosi na jedan od odštetnih zahteva izvođača sa projekta izgradnje autoputa. Izvođač je angažovao konsultantsku firmu za izradu odštetnog zahteva. Za proračun koštanja mehanizacije na stajanju korišćena je BGL verzija iz 1996. godine, upravo iz razloga što ta metoda daje najveći rezultat.

Trošak koji je proračunao izvođač za 27 mašina koje su stajale promenljiv broj dana (76-240), iznosio je 866.744€. Ukoliko se trošak preračuna po predloženoj proceduri dobija se znatno manja vrednost: 245.863€ za iste mašine i isti broj dana zastoja.

Obračun je prikazan na slici 4. U prvom delu tabelu su osnovni podaci o mašinama: nabavna vrednost, rezidualna vrednost, godina proizvodnje i ekonomski vek u godinama. U narednim kolonama je proračun broja godina nakon završene amortizacije (negativna vrednost za novije mašine) i godišnjeg troška za amortizaciju, kamatu i osiguranje. U sledećoj koloni je broj dana zastoja a zatim proračun ukupnog troška za amortizaciju, kamatu i osiguranje. Na kraju je dat ukupan trošak za stajanje za svaku mašinu (kolona „ukupno“). U poslednje dve kolone dat je završni obračun izvođača po odštetnom zahtevu sa dnevnim i ukupnim koštanjem stajanja za svaku mašinu.

BR.	NABAVNA VREDNOST	REZIDUAL. VREDNOST (20%)	GODINA PROIZV.	EKONOMSKI VEK	BR.GOD. POSLE EK. VEKA	AMORTIZA CIA GOD.	KAMATA GOD.	OSIGURAN JE GOD.	DANI	AMORTIZA CIA	KAMATA	OSIGURAN JE	UKUPNO	DNEVNO KOŠTANJE	ODŠTETNI ZAHTEV
1	320,000	64,000	2007	12	-7	21,333	13,173	4,053	120	7,014	4,331	1,333	12,677	368	44,200
2	195,000	39,000	2007	11	-6	14,182	8,066	2,482	97	3,769	2,144	660	6,572	220	21,364
3	195,000	39,000	2007	11	-6	14,182	8,066	2,482	120	4,663	2,652	816	8,130	220	26,430
4	210,000	42,000	2006	11	-5	15,273	8,686	2,673	140	5,858	3,332	1,025	10,215	237	33,207
5	125,000	25,000	2009	11	-8	9,091	5,170	1,591	120	2,989	1,700	523	5,212	141	16,943
6	26,964	5,393	2002	12	-2	1,798	1,110	342	120	591	365	112	1,068	29	3,474
7	345,000	69,000	2006	12	-6	23,000	14,203	4,370	240	15,123	9,339	2,873	27,335	370	88,896
8	88,000	17,600	2003	10	-1	7,040	3,661	1,126	120	2,315	1,204	370	3,888	83	9,943
9	144,182	28,836	1995	12	5	0	1,593	600	120	0	197	197	395	148	17,754
10	39,618	7,924	1999	12	1	2,641	396	158	131	948	142	57	1,147	41	5,325
11	41,826	8,365	1985	10	17	0	636	197	120	0	209	65	274	38	4,510
12	88,250	17,650	2007	10	-5	7,060	3,671	1,130	120	2,321	1,207	371	3,899	79	9,515
13	104,550	20,910	2007	10	-5	8,364	4,349	1,338	120	2,750	1,430	440	4,620	94	11,273
14	104,550	20,910	2007	10	-5	8,364	4,349	1,338	120	2,750	1,430	440	4,620	94	11,273
15	70,250	14,050	2004	10	-2	5,620	2,922	899	120	1,848	961	296	3,104	63	7,574
16	108,011	21,602	2009	10	-7	8,641	4,493	1,383	120	2,841	1,477	455	4,773	97	11,646
17	108,011	21,602	2009	10	-7	8,641	4,493	1,383	120	2,841	1,477	455	4,773	97	11,646
18	116,300	23,260	2008	4	0	23,260	5,292	1,628	120	7,647	1,740	535	9,922	276	33,125
19	116,300	23,260	2008	4	0	23,260	5,292	1,628	120	7,647	1,740	535	9,922	276	33,125
20	93,774	18,755	2007	4	1	18,755	938	375	76.5	3,931	197	79	4,206	223	17,027
21	240,000	48,000	2007	8	-3	24,000	10,140	3,120	240	15,781	6,667	2,052	24,500	357	85,738
22	284,745	56,949	2008	8	-4	28,475	12,030	3,702	98.5	7,684	3,247	999	11,930	424	41,748
23	201,656	40,331	1991	12	9	0	2,471	874	240	0	1,625	575	2,199	222	53,376
24	242,000	48,400	2006	12	-6	16,133	9,962	3,065	240	10,608	6,551	2,016	19,174	267	64,056
25	726,003	145,201	2007	12	-7	48,400	29,887	9,196	240	31,825	19,652	6,047	57,523	801	192,168
26	40,000	8,000	2007	12	-7	2,667	1,647	507	240	1,753	1,083	333	3,169	44	10,589
27	6,200	1,240	2004	12	-4	413	255	79	120	136	84	26	246	7	821
										145,631	76,179	23,683	245,493		866,744

Slika 4. – Odštetni zahtev proračunat po predloženoj metodologiji i po BGL proceduri

Narandžastom bojom su obeležene mašine koje su još uvek u ekonomskom veku a zelenom pet starijih mašina. Za njih se može uočiti da je amortizacija jednaki nuli za tri mašine dok su za dve priznati troškovi zbog činjenice da su samo godinu dana starije od ekonomskog veka. S obzirom da je rezidualna vrednost 20%V, trošak je opravdan jer amortizacija nije u potpunosti završena (slučaj sa slike 2). Troškovi kamate i osiguranja su takođe priznati, ali su računati na osnovu rezidualne vrednosti (ne nabavne vrednosti).

Ako se posmatraju pojedinačne mašine koje su još uvek u ekonomskom veku, predložena procedura rezultuje troškovima koji prosečno čine 33% od potraživanja izvođača. Treba istaći da je izvođač u ovom slučaju nekorektno sproveo samu BGL proceduru pa su razlike veće nego što bi zaista trebale da budu. Ako se posmatraju samo starije mašine onda predložena procedura rezultuje troškovima koji prosečno čine samo 12% od potraživanja izvođača. Razlika je u direktnoj zavisnosti od starosti konkretnih mašina koje se pojavljuju u okviru odštetnog zahteva. Što su mašine starije, razlika je veća.

Koristeći predloženu metodologiju investitor može osporiti izvođačevu nameru da naplati neopravdano visok trošak za stajanje već amortizovanih mašina. U ovom slučaju, investitor je priznao samo 28% traženog iznosa, čime su se očekivano stvorili uslovi za spor i arbitražu oko vrednosti odštetnog zahteva, što nepobitno ukazuje za potrebu za formulisanjem nepristrasne metodologije za proračun troškova neupolnene građevinske mehanizacije.

5 ZAKLJUČAK

Značaj proračuna troškova neuposlene građevinske mehanizacije ogleda se u velikim vrednostima koji ovaj trošak može imati na građevinskim projektima. U relativnom smislu, trošak mehanizacije na stajanju je obično najveći pojedinačni trošak u okviru odštetnih zahteva izvođača zbog zastoja izazvanih krivicom investitora. U slučaju dugačkih zastoja ukupan trošak za neuposlenu građevinsku mehanizaciju može značajno uticati na uspeh ili neuspeh na projektu.

Analize literature je pokazala da ne postoji jedinstvena metodologija za obračun. Praksa je pokazala da se proračun može razlikovati i do 1:3 u zavisnosti od primenjene metodologije proračuna. Zbog toga je veoma značajno da se uspostavi jedinstvena i nepristrasna metodologija koji bi bila priznata na svim projektima.

U ovom radu analizirana je naučna literatura, kao i posebni pristupi proizvođača mehanizacije i strukovnih organizacija. Uočen je nedostatak u pogledu proračuna koštanja mašina koje su starije od svog ekonomskog veka, a koje su itekako prisutne na građevinskim projektima. Posebno je analizirana nemačka BGL procedura proračuna zbog njene česte upotrebe na projektima. Nakon analize nametnulo se pitanje opravdanosti upotrebe BGL procedure na projektima zbog nelogičnosti u okviru empirijske formule koja se koristi i zbog mnogo većih rezultata koji se dobijaju u odnosu na poznate formule, što favorizuje izvođača u odštetnim zahtevima i često dovodi do skupih i dugotrajnih procesa arbitraže.

Zatim je, na bazi priznatih pristupa iz udžbeničke literature iz oblasti građevinske mehanizacije, predložena metodologija koja predstavlja dopunu postojećih znanja u segmentu definisanja posebnih setova formula koje se primenjuju u zavisnosti od starosti mašine. Na predloženi način mogu se objektivno i nepristrasno proračunati troškovi mehanizacije na stajanju i izbeći nepotrebni arbitražni procesi i dodatni troškovi koji idu uz njih.

Studijom slučaja potvrđeno je na konkretnom primeru sa izgradnje autoputa da je razlika između (nekorektno primenjene) BGL procedure i predložene metodologije velika. Razlika između potraživanja izvođača i investitorovog viđenja istog odštetnog zahteva je 1:3,5 što ukazuje na veliku verovatnoću spora i arbitražnog postupka. Primenom predložene nepristrasne metodologije na projektima, što se lako može postići definisanjem odgovarajućeg člana u ugovoru, sporovi oko troška neuposlene mehanizacije mogu se izbeći i tako olakšati realizacija projekata.

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Snežana Mašović¹, Nikola Tanasić²

RIZIK U GRAĐEVINSKOM KONSTRUKTERSTVU

Rezime:

Savremeni propisi za građevinske konstrukcije su bazirani na teoriji pouzdanosti, odnosno verovatnoći „otkaza“ konstrukcije za posmatrani životni vek. Postoji tesna veza između pojmova pouzdanost konstrukcija i rizik. Koncept rizika pored verovatnoće „otkaza“ uvodi i pojam posledica „otkaza“. Rizik postoji u svakoj fazi života konstrukcije počev od faze projektovanja preko faze eksploatacije pa zaključno sa uklanjanjem (programiranim rušenjem). Zadatak savernih projekatana je da sagledaju sve ove faze života konstrukcije te da na osnovu procene rizika donesu odluku o njenoj koncepciji. U radu su izloženi osnovni principi procene rizika u životnom veku konstrukcije.

Ključne reči: pouzdanost, posledice, rizik

RISK IN STRUCTURAL ENGINEERING

Summary:

Contemporary building codes are introducing an explicit consideration of reliability in structural design. There is a close connection between reliability and risk. In addition to the probability of failure, the concept of risk includes the consequences of failure. Risk emerges throughout the whole lifecycle of the structure: design stage, construction stage, utilization stage (including maintenance, assessment, refurbishment or repair) and finally to demolition. The task of conscientious designers is to consider entire lifecycle of a structure, and based on risk assessment decide on the design concept. An overview of the basic principles of risk assessment in structural engineering is presented in the paper.

Key words: reliability, consequences, risk

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1 POJAM RIZIKA I UPRAVLJANJA RIZIKOM

U najširem smislu problematika rizika se razmatra u SRPS ISO 31000:2015 (identičnim sa ISO 31000:2009 [1]). Definicija pojma rizika, zastupljena u ovom standardu, često deluje neupotrebljivo građevinskim konstrukterima jer doslovce glasi: ***rizik je uticaj neizvestosti na ciljeve***. Pri tome uticaj, ne mora biti obavezno loš (negativan), tako da bi se rizik sa pozitivnim efektima može shvatiti kao "šansa".

Kvantifikacija prikazane definicije je veoma jednostavna u matematičkoj interpretaciji kao:

$$R = p(H) \cdot C(L|H) \quad (1)$$

gde su:

R rizik,
 $p(H)$ verovatnoća događaja H ,

$C(L|H)$ posledica – gubitak L u slučaju da se realizovao događaj H .

Događaj H se naziva hazard, odnosno proces ugrožavanja, npr, zemljotres, korozija armature, požar i sl. Naravno, sama verovatnoća je bezdimenziona veličina i to obavezno manja ili jednaka 1 (što bi predstavljalo siguran događaj), što znači da se rizik izražava u jedinicama posledica događaja H . Međutim, posledice mogu biti raznovrsne, od manjih oštećenja konstrukcije preko umanjene funkcionalnosti pa sve do kolapsa, eventualno praćenim gubicima ljudskih života. To zahteva razgraničenje posledica na merljive (u novčanim jedinicama) i nemerljive (gubitak ugleda, povrede, ljudski životi). Primer merljive posledice je oštećenje ili kolaps same konstrukcije, dok je smanjenje funkcionalnosti teže proceniti u monetarnim jedinicama. Što se tiče nemerljivih posledica, koliko god to zvučalo nemoralno, i one se mogu monetarizovati, pri čemu je stalna debata oko „vrednosti ljudskog života“. U vezi ovoga koriste se različiti modeli kao što je npr. LQI (indeks kvaliteta života), vrednosti isplata osiguravajućih društava za životna osiguranja i sl.

Posledice mogu biti direktne (koje su neposredno vezane za nepovoljni događaj) ili indirektno (nastupaju u dužem periodu nakon samog događaja i u vezi su sa umanjenom funkcionalnošću objekta).

Značajno je istaći i sledeće zablude koje se često javljaju u vezi pojma rizika. To su:

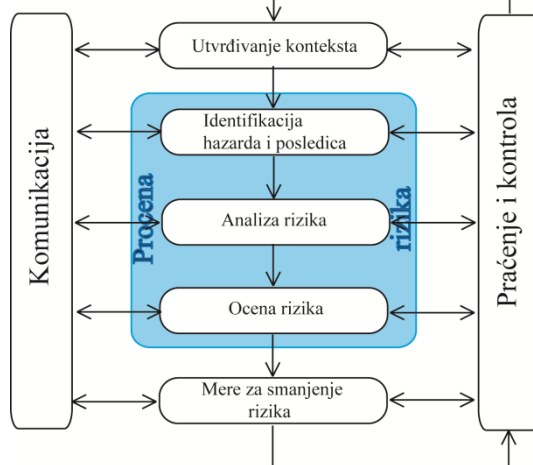
- ***rizik je hazard*** (verovatnoća pojave hazarda bez eventualnih posledica ni u kom slučaju nije rizik),
- ***rizik su posledice*** (ako nema verovatnoće da se određeni hazard realizuje, takođe nema rizika)
- ***verovatnoća pojave hazarda i njegove posledice su međusobno zavisne*** (povećanje verovatnoće hazarda ni u kom slučaju ne povećava posledice izazvane njime, odnosno posledice pojedinog hazarda zavise od same magnitude hazarda a ne od toga koliko je učestao).

Samo saznanje o riziku je bezvredno ako ne podrazumeva neku akciju u smislu njegovog smanjenja ili čak prihvatanja što je zapravo proces upravljanja rizikom tj. donošenje odluka uzimajući u obzir neizvesnosti. U tom smislu se građevinski konstrukteri svakodnevno u praksi suočavaju sa problemom rizika: donose odluke o koncepciji konstrukcije, načinu izvođenja pod neizvesnim opterećenjima, neizvesnim načinima upotrebe ili neizvesnim uslovima okoline. U

upravljanju rizikom u građevinskoj praksi posebno mesto zauzimaju odluke o načinima popravki i sanacija postojećih konstrukcija.

1.1 UPRAVLJANJE RIZIKOM

Opšti okvir za upravljanje rizikom je sadržan u ISO 31000:2009 i prikazan na slici 1.



Slika 1 – Proces upravljanja rizikom prema ISO 3100

Početni korak predstavlja utvrđivanje konteksta čime se postavljaju granice sistema za koji se vrši analiza. Primera radi kontekst može obuhvatati celokupan životni vek konstrukcije ili samo pojedine faze: projektovanje, ugovaranje, izvođenje, održavanje i slično. Širi kontekst povlači kompleksniju analizu pa se vrši razlaganje problema na manje segmente, npr. faza projektovanja. Uspostavljanje konteksta takođe podrazumeva i definisane ciljeva.

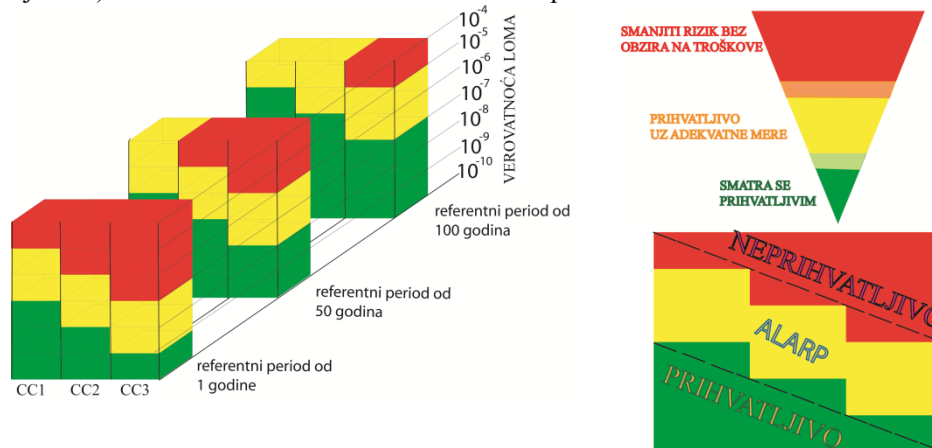
Bez sumnje ciljevi svakog racionalnog investitora (klijenta) je konstrukcija koja zadovoljava:

- sigurnost (nosivost),
- funkcionalnost (upotrebljivost),
- održivost (trajnost),
- ekonomičnost i
- estetiku.

Prva tri zahteva su eksplicitno sadržana i u tehničkoj regulativi za projektovanje konstrukcije dok su poslednja dva često konfliktni, kako međusobno tako i sa setom od prva tri zahteva. Optimalno rešenje konstrukcije treba da u najvećoj meri zadovolji svih pet zahteva.

Procena rizika je ključan segment u upravljanju rizikom i sledi pošto je utvrđen kontekst. Sastoji se iz identifikacije hazarda (procesa ugrožavanja) i procene posledica pod uslovom da se taj hazard ostvari. Sama analiza rizika se sprovodi povezivanjem uzroka i posledica u formu Bajesovih mreža uz određivanje, po mogućstvu, kvantitativne vrednosti rizika. Treba istaći da je određivanje kvantitativne vrednosti rizika veoma otežano usled problema oko kvantifikacije posledica. Upravo je to razlog što se rizik najčešće kvalitativno izražava u diskretnom broju kategorija (obično 3 do 5) u okviru takozvane matrice rizika – slika 2. Ovde prikazana matrica

obuhvata i vremenski period uzimajući u obzir da se verovatnoća nekog događaja (npr. zemljotresa) može razmatrati u različitim referentnim periodima.



Slika 2 – Primer matrica rizika za 3 klase posledica i referentne periode; ALRAP princip

Nakon procene rizika vrši se njegovo ocenjivanje – kategorisanje. Uopšteno postoje 3 područja čije je granice neophodno uspostaviti u okviru samog konteksta. Ova se područja jasno uočavaju na slici 2 kroz 3 boje: zelena – zanemarljivi rizik, žuta – podnošljivi rizik sa mogućnošću racionalnog smanjenja - ALARP (As Low As Reasonably Practicable) odnosno crvena – neprihvatljivi rizik. Ocena je ulazni podatak za donošenje odluke o odgovarajućim merama za njegovo smanjenje. Treba dakle imati „spisak“ potencijalnih mera među kojima su i slučajevi da nisu potrebne nikakve mere i odustajanje od projekta.

Zona neprihvatljivog rizika je imperativ gde se ili odustaje od projektne koncepcije ili se, bez obzira na troškove, rizik mora smanjiti. U ALARP zoni postoji čitav dijapazon mera koje se koriste: od dodatnih istraživanja vezanih za izloženost konstrukcije, ojačanja ili zaštita konstrukcije i njenih segmenata pa sve do promene koncepta konstrukcije. Naravno da je svaka od ovih mera karakterisana odgovarajućim troškovima koje određuju njenu racionalnost tj. mera je racionalna ukoliko je smanjenje rizika koje se njome postiže veće od njene cene.

Na levoj strani slike 1 istaknut je segment koji je od izuzetnog značaja u procesu upravljanja rizikom: **komunikacija** između zainteresovanih strana (klijent – investitor, projektant, potencijalni izvođači). Kroz ovaj proces, koji treba da bude sveprisutan u svakoj fazi, treba jasno odrediti ko snosi koji rizik (odnosno troškove jer se isti meri u monetarnim jedinicama).

Na desnoj strani slike 1 prikazano je da je su redovne provere kroz praćenje i kontrolu sastavni deo upravljanja rizikom. Odgovornost za praćenje i ocenu treba biti jasno definisana već pri uspostavljanju konteksta.

2 POUZDANOST KONSTRUKCIJA I RIZIK

Koncept savremenih propisa, kao što su Evrokodovi, je baziran na teoriji pouzdanosti. U praksi se, međutim, koristi semiprobabilistički pristup zasnovan na parcijalnim koeficijentima

sigurnosti. Parcijalni koeficijenti sigurnosti se kalibrišu kako bi se ispunio kriterijum postavljen u odnosu na verovatnoću „otkaza“ konstrukcije.

Tako posmatrano, teorija pouzdanosti se pre svega bavi verovatnoćom „otkaza“, dok je za ocenu rizika neophodan i drugi član jednačine (1) koji uvodi i posledice posmatranog „otkaza“.

2.1 TEHNIČKA REGULATIVA ZA PROJEKTOVANJE KONSTRUKCIJA

Parcijalni koeficijenti sigurnosti zastupljeni u Evokodu [2] su kalibrisani prema ciljanim indeksu pouzdanosti β , u kome se krije verovatnoća „otkaza“. Pozadina parcijalnih koeficijenata sigurnosti ([3], [4]) koji su predloženi je sadržana u Probabilističkom modelu propisa izrađenom od strane JCSS [5]. Parcijalni koeficijenti sigurnosti zavise od projektovanog životnog veka konstrukcije ali i od značaja konstrukcije. U terminu „značaj konstrukcije“ sadržana je takozvana klasa posledica čime se indirektno uvodi razmatranje rizika. Ovde je pristup za proračun sa parcijalnim koeficijentima sigurnosti semiprobabilistički, a pristup kod razmatranja rizika kvalitativan – zastupljene su tri klase posledica CC1-CC3 (kao što je prikazano na slici 2).

Za projektante je od izuzetnog značaja napomena data u informativnom Aneksu B EN 1990: 2002, koja upućuje na to da se parcijalni koeficijenti sigurnosti odnose na referentni period od 50 godina sa srednjom vrednošću posledica po živote ljudi ali i značajne ekonomske i socijalne posledice. Drugim rečima, prihvatanje parcijalnih koeficijenata sigurnosti prema Evrokodu prihvata se i verovatnoća „otkaza“ od oko 10^{-4} (za period od 50 godina), za granično stanje loma kao i oko 10^{-1} za granično stanje upotrebljivosti.

Može se diskutovati na osnovu čega su izabrane navedene ciljane verovatnoće „otkaza“ imajući u vidu društvenu percepciju povreda i smrtnih slučajeva pri „otkazu“ konstrukcije. Smanjenjem referentnog perioda, na bazi koga su određivani intenziteti dejstava na konstrukcije data u odgovarajućim delovima Evrokoda 1, menja se i prihvatljiva verovatnoća otkaza. Tako da se sa istim koeficijentima sigurnosti postiže značajno manja verovatnoća „otkaza“ na godišnjem nivou: 10^{-6} za granično stanje loma odnosno oko 10^{-3} za granično stanje upotrebljivosti. Do ovih vrednosti se došlo na osnovu takozvanog „prihvatljivog“ rizika za godišnje stope smrtnosti pri obavljanju svakodnevnih aktivnosti.

Treba naglasiti da se mogu koristiti i niži nivoi ciljane pouzdanosti (posledično i parcijalnih koeficijenata sigurnosti) u slučaju procene postojećih konstrukcija (ISO 13822 2010. baziran na ISO 2394, 1998) ako su opravdani na osnovu socijalnih, kulturnih, privrednih i uslova održivosti. To ukazuje da ciljani nivo pouzdanosti zavisi od balansa između posledica „otkaza“ i troškova sigurnosnih mera sa ciljem smanjenja ukupnih troškova životnog veka konstrukcije.

Semiprobabilistički pristup se smatra sasvim zadovoljavajućim pri projektovanju uobičajenih konstrukcija za normalne uslove eksploatacije. Pri projektovanju kompleksnih konstrukcija na koje mogu delovati nestandardna opterećenja sve više se primenjuju probabilističke metode i logičke zavisnosti uzroka i posledica (npr. Bajesove mreže) za procenu rizika i postupanje u skadu sa time. Rizik je definisan u informativnom aneksu B u EN 1991-1-7 [6] kao:

$$R = \sum_{i=1}^{N_H} p(H_i) \sum_{j=1}^{N_D} \sum_{k=1}^{N_S} p(D_j | H_i) p(S_k | D_j) C(S_k) \quad (2)$$

gde su:

R rizik,

$p(H_i)$ verovatnoća pojave neke magnitude hazarda H_i ,

N_H ukupan broj različitih hazarda koji se razmatra

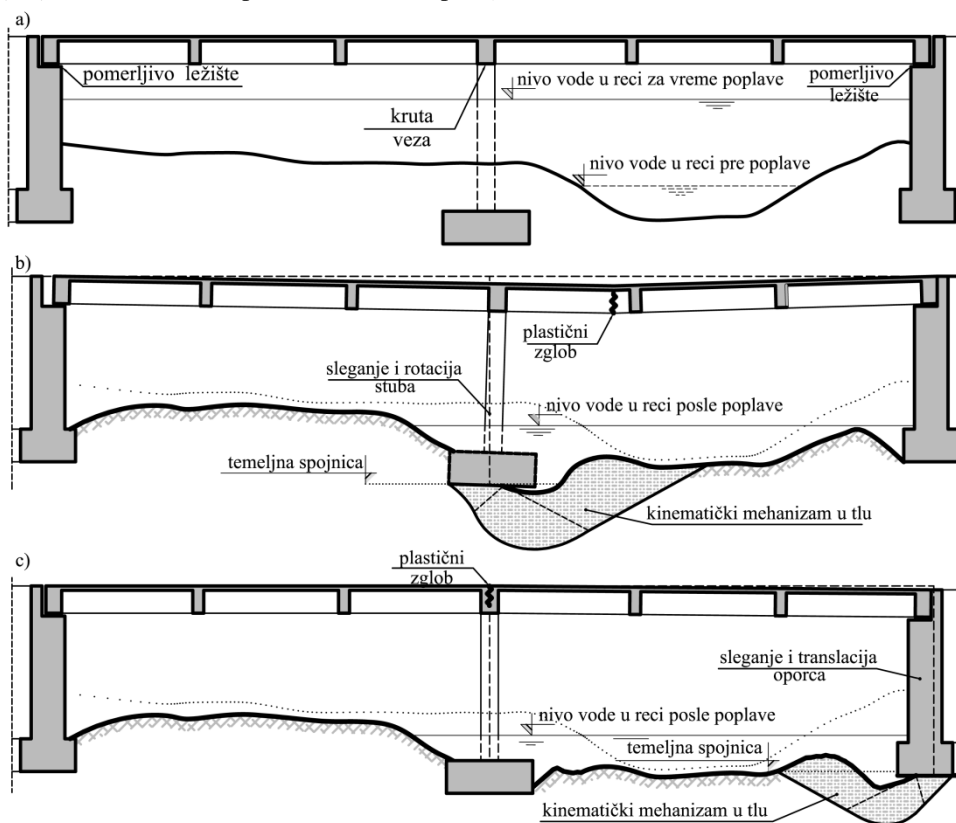
$p(D_j|H_i)$ uslovna verovatnoća za oštećenje konstrukcije tipa D_j pod uslovom realizacije hazarda sa magnitudom H_i

N_D ukupan broj mogućih tipova oštećenja konstrukcije

$p(S_k|D_j)$ uslovna verovatnoća za otkaz tipa S_k pod uslovom oštećenja D_j

N_S ukupan broj mogućih tipova otkaza konstrukcije

$C(S_k)$ monetarizovane posledice otkaza tipa S_k (direktne i indirektne).



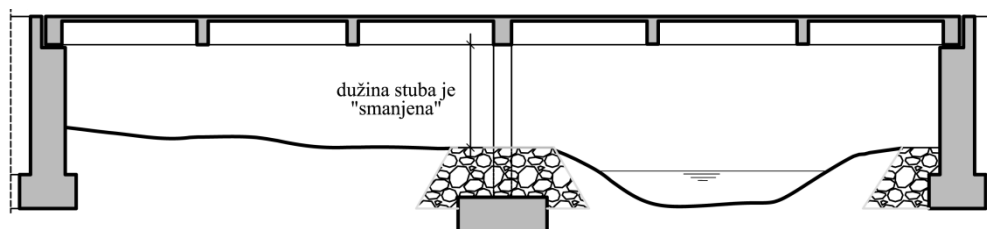
Slika 4 – a) Kontinualni AB most na koji deluje poplava b) Mogući mehanizam loma usled lokalne erozije rečnog dna kod srednjeg stuba c) Mogući mehanizam loma usled lokalne erozije rečnog dna kod oporca

Primer delovanja nestandardnog opterećenja od poplave na jedan kontinualni armiranobetonski most je ilustrovan na Slici 4. Prema formuli 2, proračun rizika bi u ovom slučaju podrazumevao određivanje verovatnoće pojave neke magnitude poplave a zatim

određivanje uslovne verovatnoće da se pri njoj desi oštećenje osloničkog tla kod stuba ili oporca (lokalna erozija rečnog dna).

Zatim sledi određivanje uslovne verovatnoće mehanizma otkaza (sleganje, rotacija, translacija) usled nastalog oštećenja i konačno evaluacija posledica kod takve vrste otkaza.

Kad se razmatra jedan tip hazarda logično je da se već u projektovanju razmišlja o merama za smanjenje rizika usled njega. Treba međutim imati u vidu da je konstrukcija u svom životnom veku izložena različitim hazardima sa različitim verovatnoćama pojave. U tom smislu, mere preduzete za smanjenje rizika usled jednog hazarda ne smeju da povećavaju rizik od drugog hazarda. Na primer, primena kamenog nabačaja radi smanjenja rizika od lokalne erozije rečnog dna kod stubova mosta može da poveća rizik na dejstvo zemljotresa (slika 5).



Slika 5 – Zaštita srednjeg stuba i oporca od lokalne erozije rečnog dna utiče na povećanje rizika od dejstva zemljotresa

2.2 RIZIK U ŽIVOTNOM VEKU KONSTRUKCIJE

Ukoliko se životni vek konstrukcije grubo podeli u tri faze: projektovanje, izvođenje i eksploatacija (uz održavanje), proces upravljanja rizikom započinje identifikacijom hazarda koji se mogu javiti u svakoj od ovih faza [7], [8].

2.2.1 Rizik u fazi projektovanja

Treba istaći da je projektovanje ključna aktivnost građevinskih konstruktera znatno šira od statičkog proračuna i izrade detaljnih planova u skladu sa projektnim zadatkom. Zato projektanti stalnih i privremenih konstrukcija imaju obavezu da izrade projekat konstrukcije koja će pored uobičajenih kriterijuma nosivosti, upotrebljivosti i trajnosti zadovoljiti i kriterijume za izvodljivost, održavanje i uklanjanje.

Upravo je faza projektovanja pravi trenutak za upravljanje rizikom i ne treba propustiti tu priliku. Projektanti treba da su upoznati sa potencijalnim hazardima i neizvesnostima u vezi sa njima. Lista potencijalnih hazarda (svakako ne potpuna) bi mogla da izgleda ovako:

- neadekvatne ili nepotpune podloge (geodetske, geomehničke, hidrološke, seizmološke i dr.),
- nerazumevanje konstrukcije (uključujući zloupotrebu i nepromišljeno korišćenje računara),
- nedovoljno poznavanje materijala konstrukcije,
- neadekvatno razmatranje pojedinih slučajeva opterećenja (potpuno isključivanje određenog opterećenja ili pogrešna procena njegovog intenziteta),
- greške u proračunu ili grafičkoj dokumentaciji,

- greške u detaljima veza,
- nedovoljno poznavanje načina izvođenja, održavanja ili uklanjanja,

U stručnoj literaturi se mogu naći mnogi primeri grešaka (od manjih do katasrofalnih), čiji su uzroci i posledice detaljno analizirani. Neki tipični propusti mogu da posluže kao uputstvo pri razmatranju potencijalnih hazarda. Veliki broj nesreća je posledica loše komunikacije, bilo sa stručnjacima iz drugih oblasti npr. arhitektama, mašinskim inženjerima, stručnjacima za hidrologiju ili meteorologiju, ali i u komunikaciji između projektanta i na početku nepoznatog, izvođača.

S obzirom na složenost zadatka i potencijalne posledice, razvijen je sistem nezavisne kontrole projekta - revizija. Tako Evrokod 1990 predviđa 3 nivoa revizije proračuna koji generalno odgovaraju klasama posledica, kao i tri nivoa nadzora u toku izvođenja. Međutim, treba imati u vidu da je data mogućnost smanjenja parcijalnih koeficijenata sigurnosti ukoliko je klasa nadzora veća od zahtevane za određeni tip posledica. Ovu odredbu treba shvatiti krajnje oprezno u našim uslovima. Moderni propisi, ugovorni modeli i proračun preko računarskih programa uveli su veću kompleksnost u čitav proces projektovanja i izvođenja i samim tim otvorili mogućnosti za pravljenje greški.

Projektanti treba da su svesni da projektuju konstrukciju koja treba da funkcioniše u svom predviđenom životnom veku, i da u vezi sa tim načine pretpostavke o tome kako će konstrukcija stariti pod normalnim okolnostima, kada je treba pregledati i na koji način. Tako da se pored zahteva o trajnosti sve češće sasvim opravdano postavljaju zahtevi u pogledu mogućnosti inspekcije tokom eksploatacije.

2.2.2 Rizik u fazi izvođenja

Iako je neželjeno produženje roka izgradnje prepoznato od strane izvođača kao ozbiljan hazard, ono je posledica okolnosti kao što su: neusklađenosti projekta sa uslovima na terenu, nemogućnosti nabavke materijala i opreme, vremenskih prilika ili propusta pri ugovaranju.

Međutim, ovde najznačajniji hazard predstavlja neadekvatna priprema koja se ogleda u analizi privremenih konstrukcija kao što su skele ili podgrade, čiji kolaps može imati značajne posledice.

Pored toga kritična opterećenja se mogu javiti u fazi izvođenja i/ili biti diktirana načinom izvođenja [9]. Skladištenje materijala ili opreme na izvedeni deo konstrukcije često nije predviđeno kao opterećenje u projektu konstrukcije te je svrsishodno da izvođači konsultuju projektante.

2.2.3 Rizik u fazi eksploatacije

Listu najznačajnijih hazarda tokom eksploatacije konstrukcije je potrebno neprekidno revidovati i dopunjavati u skladu sa pojavama novih, nepredvidivih i nepovoljnih okolnosti, a ovde se navode neki primeri:

- klimatske promene,
- teroristički napadi,
- neadekvatna upotreba,
- neadekvatno održavanje,
- neadekvatna rekonstrukcija ili sanacija

U svom radnom veku, konstrukcija može biti izložena različitim modifikacijama od strane korisnika ili biti izložena dejtvima koja nisu predviđena u projektu (nisu prepoznata u dosadašnjoj praksi tzv. „black swan“).

Izmena uslova okoline, opterećenja ili čak propisa u odnosu na one kada je konstrukcija projektovana može je, makar prividno, učiniti neprikladnom [10]. To ne znači da sama konstrukcija nije više bezbedna već da ne odgovara propisima za nove konstrukcije. Stvarne promene opterećenja (npr. osovinska opterećenja vozila) mogu da postave pitanje da li npr. postojeći drumski mostovi imaju zadovoljavajuću nosivost.

Racionalno je vršiti procenu postojećih konstrukcija na bazi rizika za period preostalog životnog veka a u skladu sa novim saznanjima kako o opterećenjima tako i o stanju same konstrukcije.

Najsloženije metode za procenu konstrukcija, bazirane na probablističkom pristupu, nisu naravno primenjive na sve konstrukcije pošto zahtevaju veoma specijalizovano znanje kao i multidisciplinarni pristup.

3 MERE ZA SMANJENJE RIZIKA

Mere za smanjenja rizika su usmerene na smanjenje verovatnoće otkaza konstrukcije i/ili ublažavanje nastalih posledica otkaza.

Pri projektovanju novih konstrukcija, Evrokod upućuje projektante da vode računa o klasi posledica pri dostizanju pojedinih graničnih stanja. Takođe je predložen niz mera i njihove kombinacije kojima se postiže zahtevana pouzdanost konstrukcije tj. ograničava se verovatnoća „otkaza“. Ove mere obuhvataju: preventivne (zaštitne) mere radi smanjenja izloženosti konstrukcije, mere upravljanja kvalitetom, povećanje robusnosti (preraspodela opterećenja u konstrukciji), zahteve u pogledu duktilnost i sl.

Kod postojećih konstrukcija često se primenjuje ALARP princip pri donošenju odluka za izbor mera kod smanjenja rizika. Ove mere se kreću od ograničenja upotrebe, preko zaštite izloženih elementa, ojačanja delova konstrukcije, povećanja robusnosti konstrukcije pa sve do zamene čitave konstrukcije novom. Najpre se procenjuje inicijalni rizik (bez primenjene mere) a potom i nova vrednost rizika nakon što bi se predložena mera sprovela. Nakon toga se troškovi sprovođenja mere porede sa postignutom razlikom inicijalne i nove vrednosti rizika. Takva cost-benefit analiza je veoma zahtevna a često i neprimenjiva usled nedostatka podataka o monetarnoj vrednosti posledica pri izostanku primene bilo kakvih mera. Ovakva analiza je međutim nezaobilazna kod značajnih i složenih konstrukcija.

4 ZAKLJUČAK

U domaćem građevinskom konstrukterstvu se ne posvećuje dovoljna pažnja značaju upravljanja rizikom tokom životnog veka konstrukcije. Najčešće se prihvatanjem principa sakupljenih u prethodnom kolektivnom iskustvu pod nazivom „dobra praksa“ obezbeđuje da se rizici drže u razumnim granicama. Savremeni propisi za projektovanje, ukoliko se savesno primenjuju, obezbeđuju da se upravlja rizikom već u fazi projektovanja kada je to najsvishodnije. Dobar projekat podrazumeva omogućavanje kasnijih pregleda radi održavanja i/ili procene. Održivost nalaže procenu postojećih konstrukcija za izmenjenu upotrebu, rađe no gradnju novih. Za očekivati je da će dalji razvoj sistema Evrokodova, koji predviđa izradu upstava za procenu postojećih konstrukcija, biti baziran na principima upravljanja rizikom.

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VIZUELIZACIJA U DINAMIČKOM PLANU IZGRADNJE

Rezime:

Ovim radom je pokazana dodatna mogućnost koju pruža MS Project prilikom praćenja realizacije projekta. Vizuelizacija dinamičkog plana izgradnje podrazumeva da se za određenu vrstu radova vrši povezivanje pomoću opcije „hyperlink“ sa određenom fotografijom tj. fotografijom faze radova, što korisniku umnogome olakšava praćenje toka realizacije projekta.

Ključne reči: upravljanje građenjem, dinamički plan, vizuelizacija

VISULIZATION IN CONSTRUCTION WORK PLAN

Summary:

Study has demonstrated the additional opportunity provided by MS Project during the monitoring of the project implementation. Visualization of the construction work plan means that for a certain type of work the connection is done using the "hyperlink" option with a particular photo, i.e. photographic phase of works, which greatly facilitates the user's monitoring of the project implementation.

Key words: construction management, work plan, visualization

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1 INTRODUCTION

Software development for the design of construction structures has the aspiration to reach a higher level of visualization, i.e. 3D graphical presentation. Today's level of software development enables integrated 2D and 3D design (architecture, construction, installation) with all the elements and forms of visualization. Modern approach to design, using the BIM (building information models) technology, allows the presentation of all physical and functional characteristics of the structure, the coordination of designers and design solutions.

For processes of construction and construction management the very important segment is a visualization of individual construction phases of works. By linking 3D BIM and 3D models, created in any of the software (Revit, Tekla, ArchiCAD, CAD-Duct, IFC files, even SketchUp and 3D DWG files) and time scales, 4D models are formed, which show the status of the construction process in certain time intervals. In addition to the level of development over time, construction organization can also be displayed, i.e. key machinery and equipment positions (Figure 1), as well as videos that follow the course of construction (Figure 2). This approach to BIM has the great potential to improve the project management of any size or complexity.

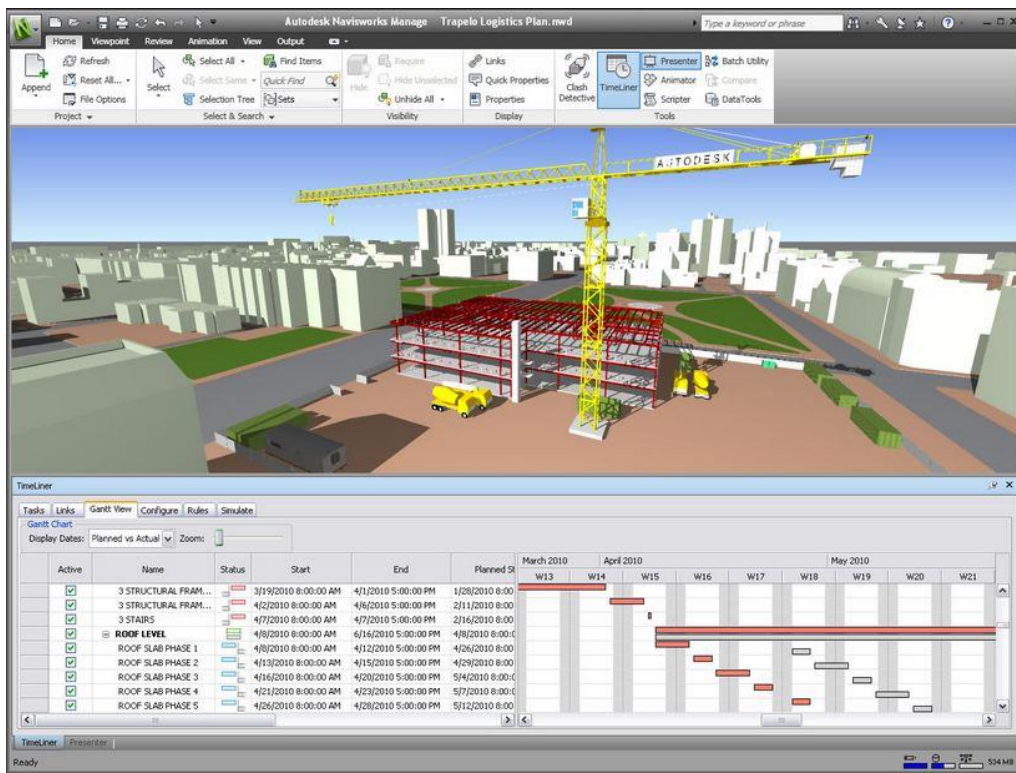


Figure 1 – Construction process as 4D model (acc. to [7])

The next step in the development of software within the BIM technology, significant for project and construction management, was in the line of 5D model development. These models are based on the analysis of construction costs associated with 4D models. Visualization of design solutions changes and fast analysis of changes in costs, as well as a visualization of a progress of activities in the construction and related costs over time, all of which improve the management and implementation of projects of any size or complexity.

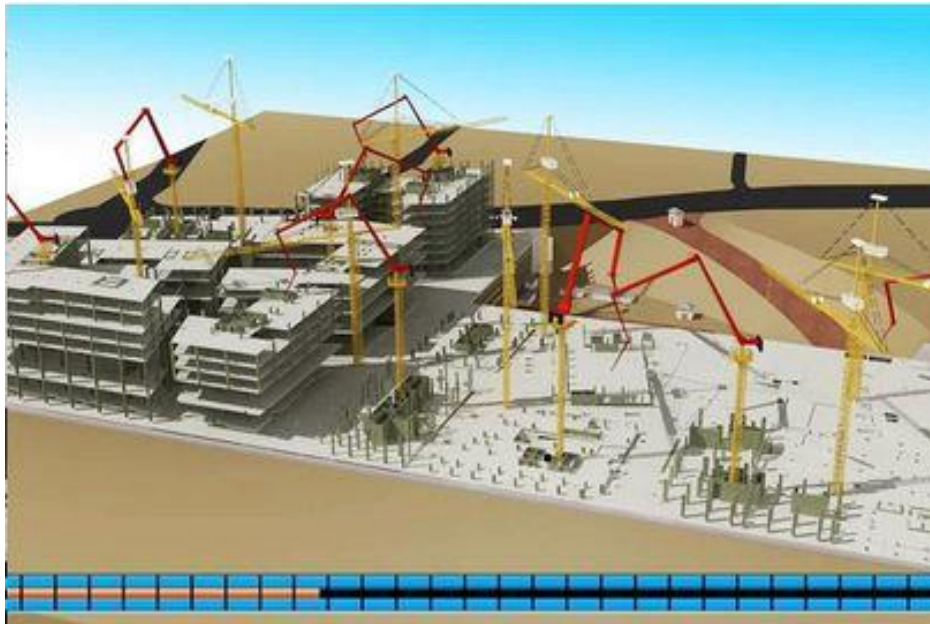


Figure 2 – Construction process as 4D model - video record (acc. to [5])

Next step are 6D BIM technology models which are arising as as-built design complemented by all relevant data about the structure (information on the equipment, instruction manual, certificate of quality, warranty, web links of equipment manufacturers and contacts, photos, etc.), with the aim of quality maintenance management.

Until the procurement of modern software and transition to BIM technology visualization of construction process can be carried out in a simple way by using 3D models, images that arise from it and connect to the work plan delivered in MS Project. Example of this type of simple visualization within the work plan is shown in this study.

2 VISUALIZATION EXAMPLE IN WORK PLAN

In the process of project management, i.e. construction process management, an important segment is the visual perception of the achieved level of construction. In addition, model calculation and observation of the construction process is possible, as well as the layout of the structure after the certain activities' completion.

Most often, preparation of work plans is done by using software applications, such as MS Project and Primavera.

Work plan as Gant chart table presentation of the flow of the construction process can be complemented by appropriate graphical presentation of progress of works.

Since activities of work plan include development of part of the building it is possible to connect the appearance of the building as an image (3D) and the actual activity. Intensity of construction of high-rise buildings and other structures, for example, is in making the actual structure. Construction management in part of structure construction process is the most important segment for execution of other activities. Visualization of activities related to the construction process significantly contributes in an overall management construction process.

Based on the above mentioned, one example of visualization of certain activities on the development of prefabricated concrete construction hall is shown in this study.

Furthermore, based on the construction requirements and adopted technology model of execution process works (network plan) was formed and by processing in MS Project software, plan for execution of works on the construction of the entire building is defined.

During the design stage the building is drawn in Auto Cad. Thereafter model is exported into SketchUp where the possibility to show only part of 3D model is given, i.e. using the options „hide“ only certain parts of 3D model will be shown, which enables the formation of 3D presentation of certain construction phases, i.e. appearance of the building after the completion of certain work plan activities. Following the above mentioned, set of images is formed as a presentation of individual work plan activities within a prefabricated construction development. Appearance of individual images – presentation of structure completion level after the realization of certain activities of work plan is shown in Table 1.

After the previous stage finalization connection of formed image (3D presentation) and the individual activities within the work plan is carried out, made in MS Project via “hyperlink” option. Minor imperfection of this study is that there is no possibility to perform the activity simply by mousing over certain positions in order to open picture but the presentation must be activated.

Figure 1 shows the work plan for the construction of prefabricated halls (done in MS Project) with visualization activities "Assembling of pillars I tact".

Presentation of work plan activities makes monitoring and project management easier to track.

There is a possibility to add into a presentation of certain activities completion, from the part of adopted technology and construction organization, position of key machinery (self-propelled crane) and equipment during the construction phase.

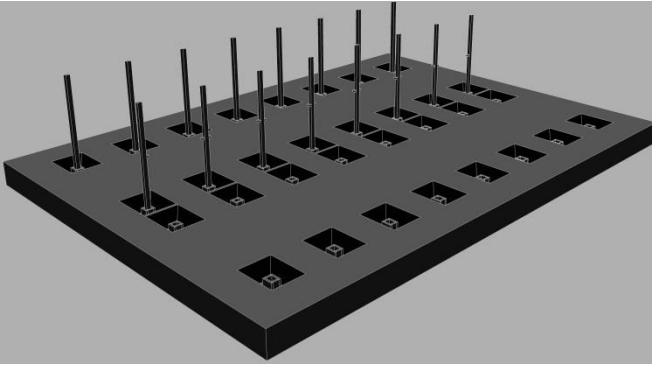
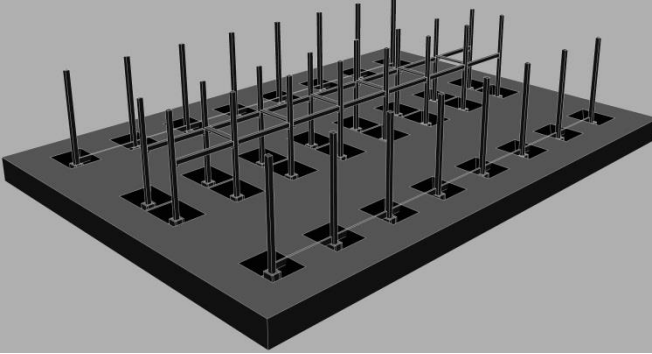
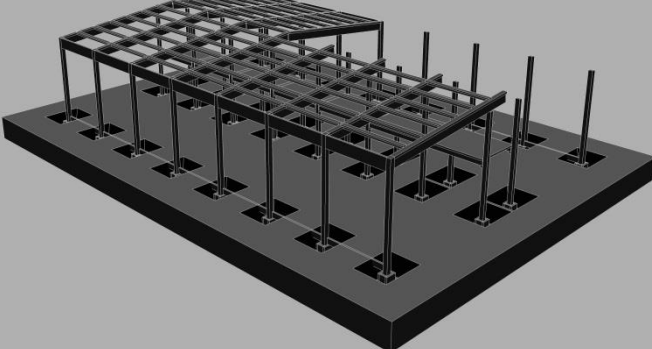
3 CONCLUSIONS

Potential exists in a simple visualization of construction process using 3D structure model (or part of a structure) as images that arise from it and connection to a work plan performed in MS Project.

Such a simple visualization of construction process associated with a work plan receives the time component and can significantly help in the construction process.

Transition to BIM technologies and procurement of appropriate software will enable even better project management support by using 4D and 5D models.

Table 1 – Work Plan Activities and Adequate 3D Presentation

Work plan activity	3D presentation of structure layout after work plan activity completion
<p><i>Assembling of pillars I tact</i></p>	
<p><i>Assembling of beams technical floors</i></p>	
<p><i>Assembling of main girders and intermediate beams 5th field</i></p>	

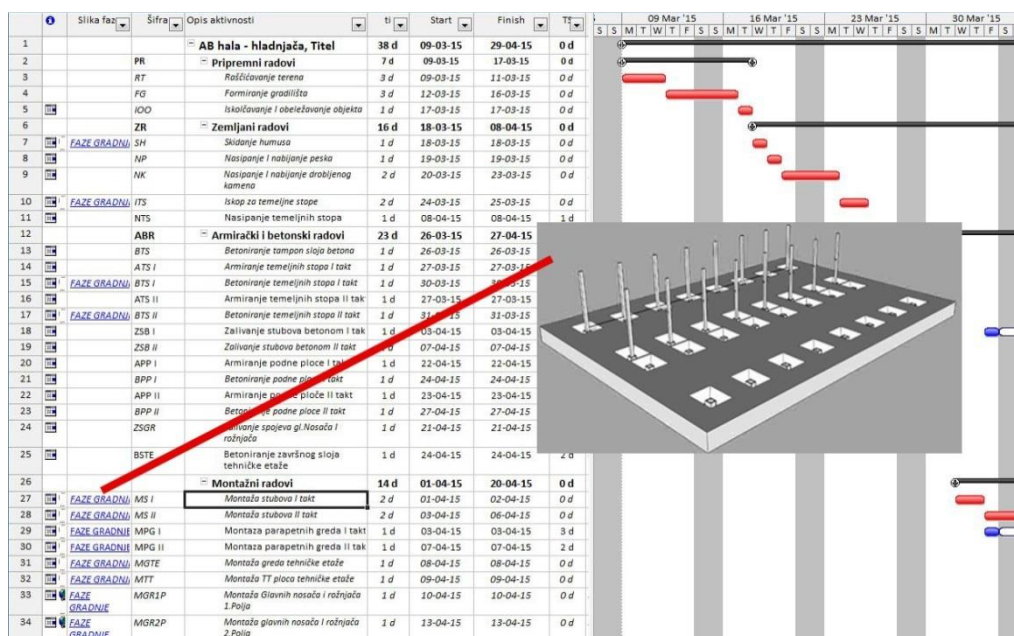


Figure 3 – Work plan in MS Project and 3D prefabricated construction model

ACKNOWLEDGEMENTS

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Predrag Petronijević¹, Dragan Arizanović², Nenad Ivanišević³, Igor Milošević⁴

UTICAJ STAROSTI GRAĐEVINSKE MAŠINE NA TROŠKOVE RADA

Rezime:

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Troškovi rada građevinske mašine menjaju se tokom vremena. Pri tome, troškovi vlasništva nad mašinom opadaju, troškovi eksploatacije rastu, a ukupni troškovi najčešće imaju pad prvih godina rada, zatim doživljavaju svoj minimum, i nakon toga konstantno rastu do kraja radnog veka mašine. Da bi se utvrdili troškovi rada mašine tokom vremena, potrebno je definisati radni vek mašine, angažovanje po godinama i pretpostaviti sve direktne troškove tokom radnog veka mašine. Osnovna primena ovako dobijenih podataka je u sagledavanju realnih troškova i optimalnom planiranju eksploatacije i zamene građevinskih mašina.

Ključne reči: građevinske mašine, troškovi rada, zamena mašina

IMPACT OF THE CONSTRUCTION EQUIPMENT AGE ON THE COSTS OF WORK

Summary:

The costs of work for the construction machinery change with time and in such a way that the machinery ownership costs decrease, the exploitation costs increase and the overall costs most often show a decrease in the first years of operation and, after reaching their minimum, increase all the way to the end of a machine's working life. In order to determine the costs of machinery operating in time it is necessary to define a working life of a machine, exploitation hours by year and assume all the direct costs. The primary use of the paper is to provide an insight into the real costs and the optimal planning for the construction machinery replacement.

Key words: construction equipment, work costs, machinery replacement

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1 UVOD

“Koliko me zaista košta građevinska mašina” i “koliko dugo treba da je zadržim” su dva pitanja na koja najčešće, u praksi, nema preciznog odgovora, a koja su od ključnog značaja za ekonomsku opravdanost vlasništva nad građevinskom mašinom. Ako je mašina u vlasništvu i ako se koristi u kratkom vremenskom periodu onda troškovi vlasništva nastali zbog visokih početnih troškova amortizacije dovode do visokih troškova po satu. Sa druge strane, ako mašina ostaje u vlasništvu previše dugo, onda visoki troškovi njene eksploatacije takođe dovode do povećanja troškova po satu. Praksa poznaje ekonomični period vlasništva ili „sweet spot“ (idealan period) kada je suma troškova po satu za korišćenje ili posedovanje minimalna, usled činjenice da je mašina radila dovoljno dugo da smanji troškove vlasništva, ali ne dovoljno dugo da bi se doživeli nepotrebni i neuobičajni visoki troškovi eksploatacije.

Proračun visine troškova i perioda kada su troškovi rada minimalni („sweet spot“), zahteva detaljno razumevanje troškova koje mašina pravi tokom vremena i kako oni variraju u odnosu na radni vek mašine. Nekoliko činjenica treba istaći kao posebno važne:

1. Troškovi vlasništva i troškovi eksploatacije su veoma različiti;
2. Troškovi sata vlasništva opadaju tokom vremena;
3. Troškovi sata eksploatacije rastu tokom vremena;
4. Proračun koštanja sata je veoma osetljiv na promene ulaznih parametara.

Odnos između koštanja po času i starosti mašine je veoma kompleksan i bazira na većem broju pretpostavki koje uvek treba proveravati u praksi. Dobijanje tačnog rezultata proračuna koštanja radnog časa tokom vremena rada mašine je moguće samo ako se ispravno pretpostave radni vek mašine i očekivani period angažovanja mašine tokom svake godine eksploatacije.

2 RADNI VEK MAŠINA

Radni vek mašina može biti definisan na tri načina: kao fizički vek, kao profitni vek i kao ekonomski vek. Na slici 1. prikazano je kako se definišu odnosi ove tri vremenske faze. Na kraju radnog veka se obično vrši zamena mašine. Na vlasniku mašine je da sagleda kako želi da posmatra kao radni vek mašine i da definiše kriterijume za odluku da je potrebno da se oslobodi mašine. Da bi se ova odluka ispravno donela, neophodno je da se definiše i primeni kriterijum završetka radnog veka građevinske mašine.

2.1 FIZIČKI VEK

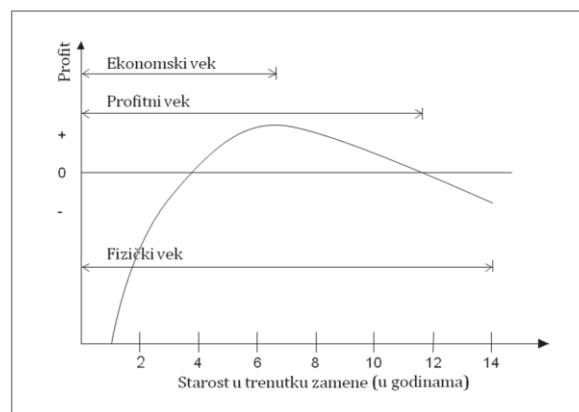
Fizički vek mašine predstavlja doba mašine nakog koga je ona istrošena i više ne može pouzdano da se koristi. Kada se istroši, najčešće biva prodana ili pretopljena u staro gvožđe. U tom dobu održavanje i operativni troškovi značajno rastu. Koliko će troškovi eksploatacije mašine rasti zavisi od načina upravljanja tom mašinom, prirode posla i kvaliteta održavanja i nege koji se primenjuju. Smatra se da se uvođenjem malih troškova za redovno održavanje ukida potreba za trošenjem velikih količina novca za zamenu glavnih komponenti mašine. Dakle, kod dve potpuno iste mašine, sa potpuno identičnim delovima, fizički vek se može znatno razlikovati u zavisnosti od načina održavanja i težine radnih uslova u kojima su korišćene.

2.2 PROFITNI VEK

Profitni vek je vreme u toku kog mašina ostvaruje profit. Korišćenje mašine van te faze dovodi do stvaranja troškova većih od profita koje je mašina ostvarila. U suštini, to je period kada mašina zapravo provodi više vremena u servisu nego na gradilištu, povećavaju se skupe popravke i smanjuje profit, jer se glavne komponente sukcesivno troše i moraju da se zamene. Stoga se mora ustanoviti kada se određena mašina približava toj tački, ili ju je već dostigla, i planirati zamenu novom mašinom, dok su glavne komponente još uvek u funkciji.

2.3 EKONOMSKI VEK

Ekonomski vek predstavlja period u kome je postignuta maksimalna produktivnost, kada mašina dostiže tačku najvećeg profita. Vlasnici mašina konstantno teže tome da maksimizuju učinak uz minimalne troškove. Slika 1 ilustruje praksu u kojoj je ekonomski život mašine kraći od fizičkog života i završava se kada profit koji ta mašina stvara dostigne najvišu vrednost. Stoga, ukoliko se mašina na vreme zameni, sprečava se pad profitabilnosti usled povećanih troškova održavanja. Vlasnici mogu, ukoliko precizno prate troškove održavanja i troškove popravki, odrediti tačno vreme za zamenu mašine. Određivanje optimalnog vremena zamene mašine zahteva da vlasnik mašine u analizu ne uključuje samo troškove vlasništva i troškove eksploatacije, već i sve druge troškove koji su u vezi sa posedovanjem mašine. Ti troškovi uključuju troškove amortizacije, inflaciju, investicione troškove, troškove održavanja i popravki, gubitke usled zastoja i troškove usled zastarelosti.



Slika 1 – ekonomski, profitni i fizički vek građevinske mašine

3 PROMENA TROŠKOVA RADA MAŠINE TOKOM VREMENA

Da bi se sagledala promena troškova rada mašina tokom njenog radnog veka, potrebno je analizirati kako se menjaju pojedini troškovi tokom radnog veka mašine. U ovom radu tretiraće se ukupni direktni troškovi kao suma troškova osnovnog sredstva (Eos) i troškova eksploatacije (Eex) uvećana za koeficijent neoperativnosti D, odnosno, za troškove koje mašina ima kada je u stanju otkaza (Down time), prema sledećoj formuli:

$$Ch = (Eos + Eex) * (1 + Down) \quad (1)$$

gde je:

Ch ukupni trošak rada mašine po času,
Eos troškovi osnovnog sredstva
Eex troškovi eksploatacije
Dow procenat neraspoloživosti mašine

3.1 TROŠKOVI OSNOVNOG SREDSTVA

Troškovi osnovnog sredstva predstavljaju troškove vlasništva nad mašinom. Oni predstavljaju zbir troškova depresijacije mašine (*Eam*), troškova investicionog održavanja (*Eio*) i troškova kamata i osiguranja (*Ekios*).

3.1.1 Troškovi depresijacije

Depresijacija predstavlja gubitak vrednosti građevinske mašine tokom vremena, usled habanja, oštećenja i zastarevanja. Depresijacija nastaje usled dva glavna razloga:

fizičkog oštećenja usled habanja i oštećenja mašine usled rada i ekonomskog pada vrednosti usled protoka vremena.

Depresiona vrednost predstavlja razliku tržišne vrednosti građevinske mašine u trenutku njene nabavke i tržišne vrednosti mašine u trenutku njene prodaje ili otpisa sa liste sredstava. Period depresijacije je vreme između nabavke mašine i njene prodaje ili otpisa.

Trošak depresijacije predstavlja količnik depresione vrednosti i perioda depresijacije.

Za proračun troškova depresijacije tokom vremena postoji nekoliko metoda, opisanih u radu [3]. Razvijane su i metode da se troškovi depresijacije predvide na osnovu statističkih podataka sa ostvarenih aukcijskih prodaja [8]. Analiza u radu [3] pokazuje da Metoda silazne ravnoteže (*Declining Balance Method*) daje rezultate koji su najpribližniji realnom kretanju cena građevinskih mašina na tržištu.

U ovom radu, u daljem proračunu troškova depresijacije, koristiće se ova metoda.

3.1.2 Troškovi investicionog održavanja

Investiciono održavanje obuhvata srednje i velike (*generalne*) opravke. U nedostatku sopstvenih podataka o izvršenom investicionom održavanju, ovi troškovi se najčešće prikazuju kao procenat od nabavne vrednosti mašine.

U skladu sa uobičajenim vrednostima koje se koriste u praksi, investicioni troškovi se pretpostavljaju na godišnjem nivou od 10% nabavne vrednosti mašine.

Radi realnijeg proračuna, prihvaćeno je da se iznos od 10% koristi u sredini radnog veka, da se za svaku raniju godinu ovaj iznos umanjuje za 2% a da se za kasnije godine ovaj iznos uvećava za 2%.

3.1.3 Troškovi kamata i osiguranja

Troškovi kamata i osiguranja predstavljaju trošak kapitala angažovanog ulaganjem u građevinsku mašinu, trošak osiguranja i registracije mašine. Obično se ovaj trošak obračunava u iznosu od 10% trenutne tržišne vrednosti mašine (dobijene metodom silazne ravnoteže).

3.2 TROŠKOVİ EKSPLOATACIJE

Troškovi eksploatacije predstavljaju troškove rada građevinskih mašina. Čini ih zbir troškova energenata i maziva (E_e , E_{maz}), troškova radne snage (E_{rs}), troškova habajućih delova (E_h) i troškova tekućeg održavanja (E_{to}). Način proračuna ovih troškova opisan je u radovima [7] i [2].

3.2.1 Troškovi energenata i maziva

Većina metodologija vrši proračun troškova energenata i maziva na osnovu snage motora, specifičnog utroška goriva (maziva), koeficijenta opterećenja motora u radu i cene goriva (maziva). Ovakav pristup, svakako lakši za proračun, je prihvaćen i u radu [7]. Mana pristupa je da se pretpostavlja da u toku rada i nove i zastarele mašine imaju istu potrošnju goriva, što ne odgovara stvarnosti.

U radu [2] uvodi se koeficijent tehničke dotrajalosti kojim se potrošnja goriva, posmatrana u zavisnosti od radnog veka mašine povećava i do 50% za sasvim zastarele mašine (koje se često koriste na našim gradilištima). Ovaj pristup je korišćen i u ovom radu, uz pretpostavku da se povećanje potrošnje energenata i maziva progresivno povećava tokom radnog veka mašine.

3.2.2 Troškovi radne snage

Trošak radne snage predstavlja trošak rukovalaca građevinske mašine izražen po času rada mašine. Ovaj trošak se najčešće ne menja tokom vremena, ili je njegova promena bez velikog značaja za ukupne troškove eksploatacije građevinske mašine.

3.2.3 Troškovi habajućih delova

Trošak habajućih delova predstavlja trošak zamene delova koji su podložni habanju i koji su predviđeni za zamenu nakon isteka radnog veka dela. Ni ovaj trošak se ne menja tokom vremena, ili je, ako se menja, njegova promena zanemariva u odnosu na promenu drugih troškova.

3.2.4 Troškovi tekućeg održavanja

Troškovi tekućeg održavanja i popravki su rezultat troškova rada i delova koji se koriste za redovnu popravku i održavanje mašine. Ovi troškovi zavise od sledećih faktora:

- tip mašine;
- starosti;
- uslova rada;
- radne veštine rukovaoca;
- dnevne nege od strane rukovaoca;
- postojanja odeljenja za održavanje u kompaniji;
- učestalosti i stepena preventivnog održavanja.

Za sagledavanje ovih troškova, veoma je važno voditi tačnu evidenciju troškova. Metodologija proračuna ovih troškova opisana je u radu [2]. Ipak, troškovi dobijeni na ovaj način važe za novu mašinu. U ovom radu je usvojeno da se ovi troškovi svake godine povećavaju za 10% u odnosu na prethodnu godinu.

3.3 PRORAČUN UKUPNIH TROŠKOVA

Ukupni troškovi obračunavaju se na osnovu formule (1). Troškovi koji se u proračunu veoma često ne obračunavaju, a imaju veliki uticaj na ukupno koštanje radnog sata mašine, su troškovi nastali usled kvarova mašine, odnosno njene neraspoloživosti za angažovanje na gradilištu. Praktični problem može predstavljati koeficijent neraspoloživosti, odnosno, procenat vremena koji je mašina u otkazu. Vreme zastoja predstavlja vreme kada mašina ne radi jer je u stanju otkaza i ono ima tendenciju da se povećava sa povećanjem vremena korišćenja mašine. Vreme dostupnosti, deo vremena kada je mašina na raspolaganju, je suprotno vremenu zastoja.

Troškovi usled otkaza mašine obuhvataju sve troškove nastale usled gubitka produktivnosti izazvanog gubitkom dostupnosti mašine. U radu [5] date su neke preporuke za koeficijent neraspoloživosti koji su korišćeni i u ovom radu.

3.4 PRIMER PRORAČUNA PROMENE UKUPNIH TROŠKOVA TOKOM VREMENA I ANALIZA DOBIJENIH REZULTATA

Kao ilustracija prethodno iznetih stavova, u ovom radu je izvršen proračun troškova rada bagera CAT 320 tokom njegovog radnog veka. Svi parametri za proračun troškova nove mašine preuzeti su iz referentne literature [7] i [3]. Rezultati proračuna su prikazani u tabeli 1. U skladu sa podacima iz [7], usvojen je radni vek mašine od 12.000 radnih sati, sa godišnjim fondom radnih sati od 1.500h, što znači da će radni vek mašine iznositi 8 godina.

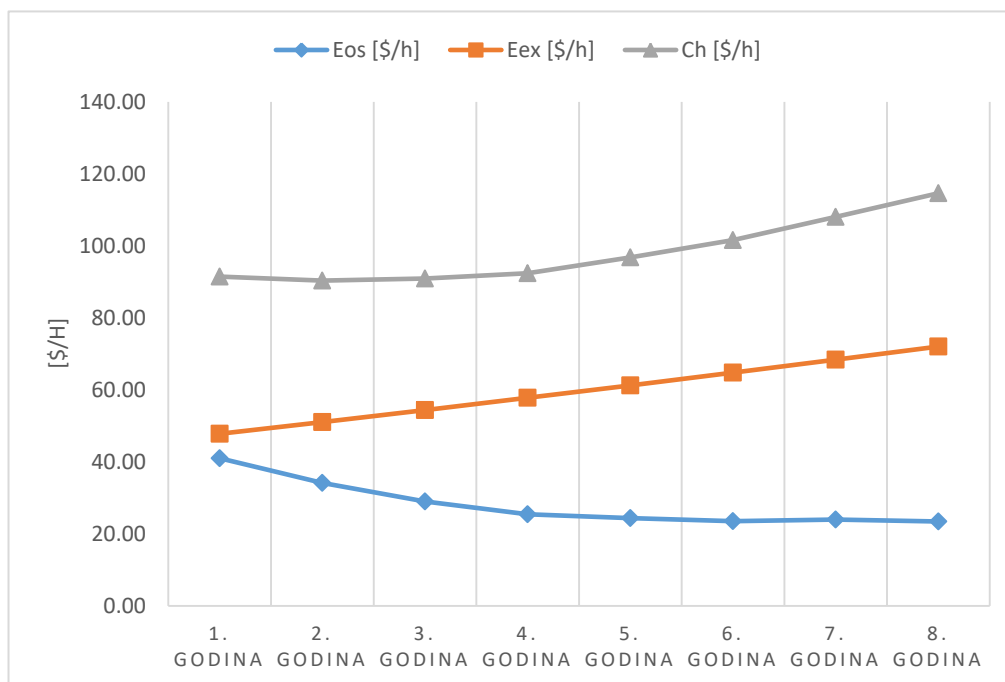
Tabela 1 – Prikaz proračuna troškova bagera CAT 320 tokom vremena:

	1. godina	2. godina	3. godina	4. godina	5. godina	6. godina	7. godina	8. godina
Eam [\$h]	13,15	9,86	7,36	5,52	4,21	3,16	2,37	1,71
Ekios [\$h]	10,52	7,89	5,89	4,42	3,37	2,52	1,89	1,37
Einv [\$h]	4,21	6,31	8,41	10,52	12,62	14,72	16,83	18,93
Eos [\$h]	41,01	34,18	29,02	25,45	24,40	23,56	23,98	23,45
Ers [\$h]	4,00	4,00	4,00	4,00	4,00	4,00	4,00	4,00
Een [\$h]	35,18	37,69	40,20	42,72	45,23	47,74	50,25	52,77
Emaz [\$h]	3,36	3,60	3,84	4,08	4,32	4,56	4,80	5,04
Eto [\$h]	5,26	5,78	6,36	7,00	7,70	8,47	9,31	10,25
Eex [\$h]	47,80	51,08	54,41	57,80	61,25	64,77	68,37	72,06
Down time [%]	3%	6%	9%	11%	13%	15%	17%	20%
C [\$h]	91,47	90,37	90,94	92,40	96,78	101,58	108,05	114,61

3.4.1 Analiza dobijenih rezultata

Kako je i navedeno u uvodnom poglavlju, troškovi osnovnog sredstva (troškovi vlasništva nad mašinom) opadaju tokom vremena, a ukupni troškovi eksploatacije rastu. Njihov zbir, kombinovan sa troškovima neraspoloživosti mašine tokom vremena, pojednostavljeno prikazan, ima oblik slova „U“, jer prvih godina ukupni toškovi padaju, dostižući minimum, da

bi nakon toga počeli da rasti, i taj trend zadržavaju do kraja radnog veka mašine. Grafički prikaz promene troškova tokom vremena dat je na slici 2.



Slika 2 – grafički prikaz promene troškova osnovnom sredstva, eksploatacije i ukupnih troškova tokom radnog veka mašine

Pod pretpostavkom da je konstantna vrednost valute u kojoj se proračun radi (u konkretnom slučaju - dolar), troškovi za svaku komponentu pomenutu u prethodnom tekstu se mogu sumirati i period ekonomskog veka mašine može se izmeriti kroz identifikaciju godine u kojoj se javlja minimalna cena po satu.

U podacima datim u tabeli 1, može se zaključiti da je minimalna cena 90.37 dolara po času u drugoj godini rada mašine, a u trećoj je neznatno veća (90,94\$/h), što je i ekonomski vek mašine. Dakle, nabavku nove opreme treba razmatrati u trećoj godini.

Prilikom analize podataka ove vrste, odnosno, pri analizi ukupnih troškova rada mašine tokom vremena, prema Voresteru [5], potrebno je obratiti pažnju na sledeće podatke:

1. Veličinu minimalne tačke – Potrebno je utvrditi koji su minimalni ukupni troškovi rada građevinske mašine;
2. Vreme minimalne tačke - Važan podatak koji pokazuje koliko dugo mašina treba biti zadržana u vlasništvu da bi suma opadajućih troškova vlasništva i rastućih troškova korišćenja bila minimalna;
3. Oblik krive – Ova informacija slikovito potvrđuje i kvantifikuje osetljivost obračuna troškova na skraćenje i produženje životnog veka mašine i vlasniku mehanizacije daje mogućnost stvaranja seta vrednosti ili zone koje služe kao osnova za planiranje zamene masina.

Osnovna primena podataka dobijenih na opisani način je u sagledavanju realnih troškova i planiranju optimalnog momenta za zamenu građevinskih mašina. Planiranje zamene je za sve građevinske kompanije veoma bitno jer je deo strateških pogleda u budućnost i služi kao baza za planiranje finansijskih sredstava i donošenje važnih budžetskih odluka.

4 ZAKLJUČAK

Značaj proračuna troškova građevinske mašine tokom vremena postaje očigledan kada se koristi da bi se prikazali kako troškovi po satu rastu tokom vremena i ako se to koristi da bi se postavila racionalna i dosledna politika zamene mašina koja bi starost flote održala balansiranu i što više moguće približila optimalnoj ceni. Broj različitih mašina koje firma poseduje jedan je od esencijalnih uslova da kompanija konkuriše za neki posao i dobije mogućnost da učestvuje na projektu. Niska prosečna starost građevinskih mašina i pravilno, blagovremeno investiranje u njihovu zamenu omogućavaju kompaniji da ostane produktivna i konkurentna u budućnosti. Veličina i starost seta mašina koje su u vlasništvu kompanije su dve veoma različite stvari. Jednom se utiče na sadašnjost, a drugom na budućnost. Obe su važne i obe zaslužuju da im se posveti pažnja. Ovaj rad upravo tome ide u prilog.

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FUNDIRANJE ZIDOVA PALATE TVRDJAVE „GOLUBAČKI GRAD“ U GOLUPCU, METODOM „JET GROUTING“

Rezime:

Zbog brojnih okolnosti u kojima treba da se nalaze temelji za zidove buduće palate, kao i uslova za izvodjenje temelja, došlo se do rešenja izvodjenja metodologijom „jet grouting“. Pored stalnog prisustva vode i muljevitog tla (loših karakteristika), a u nivou su u kome bi se nalazila temeljna spojnica, nosivost i stabilnost takvog temelja se dovodi u pitanje. Stoga je odlučeno da se komplet tlo od nivoa dna budućih zidova palate zameni „jet grouting“ kolonama do same krečnjačke mase. Time bi tako zamenjeno tlo preuzelo ulogu temelja. Na delu gde postoji zid pristupilo se „poduhvatanju“ sa „J.G.“ kolonama do same krečnjačke mase.

Ključne reči: jet grouting, ojačanje temelja, izrada temelja, zid palate

PALACE WALLS FOUNDATION OF FORTRESS „GOLUBAČKI GRAD“ IN GOLUBAC, USING THE „JET GROUTING“ METHOD

Summary:

Due to the numerous circumstances in which should be based palace walls, as well as the conditions for the execution of the foundation, a solution that has been chosen is jet grouting method. With the constant presence of water and silty soils (bad characteristics), and the level in which fundamental hinge would be located, load capacity and stability of that foundation are a problem. Therefore, it was decided to replace the set ground level from future palace wall bottom to the limestone mass with "jet grouting" columns. This replaced ground would be represent of foundation. At the part where wall exists, the „underpinning“ of „J.G.“ columns to limestone has been applied..

Key words: jet grouting, foundation reinforcing, making foundation, palace wall

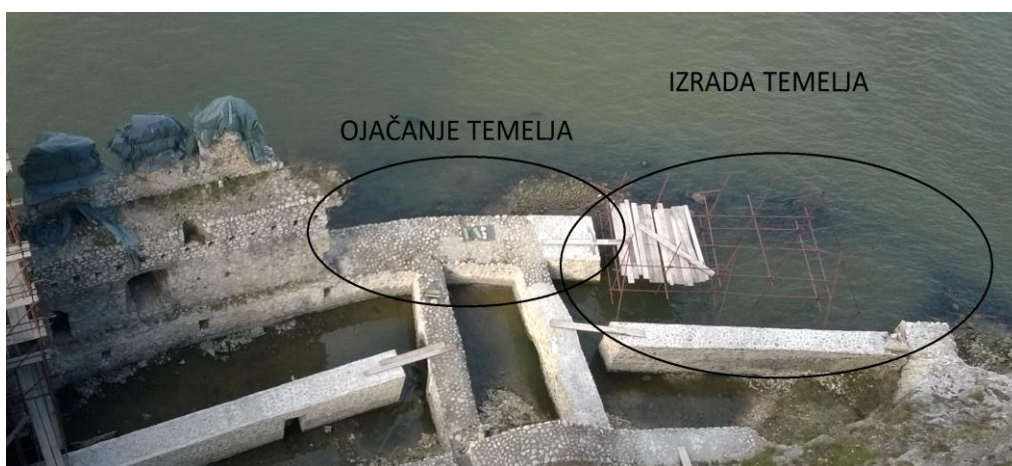
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1 UVOD U PROBLEMATIKU

Po projektu rekonstrukcije i konzervacije tvrđave „Golubački grad“ na delu palate potrebno je dozidati fasadni zid ka Dunavu. Na nizvodnom delu palate prvobitno je potrebno oformiti (izraditi) temelj kako bi se sa njega moglo nastaviti zidanje, dok i temelj postojećeg zida koji nije fundiran na stenskoj masi treba ojačati (slika 1). Na delu izrade temelja trebalo je oformiti temelj koji prati zidove palate iz dva pravca. Svi ovi radovi odvijaju se u okviru toka reke Dunav, te je neophodno bilo primeniti tehničko rešenje koje je izvodljivo u datim uslovima. Nakon sagledavanja problematike, usvojena je metoda izrade temelja pomoću tehnologije „jet grouting“. U okviru dopune projekta revitalizacije tvrđave „Golubački grad“-rekonstrukcija palate, priložena je usvojena tehnologija izvođenja radova na izradi i ojačanju temelja ispod zidova palate, koja će se izvesti metodom mlaznog injektiranja – **jet grouting**.

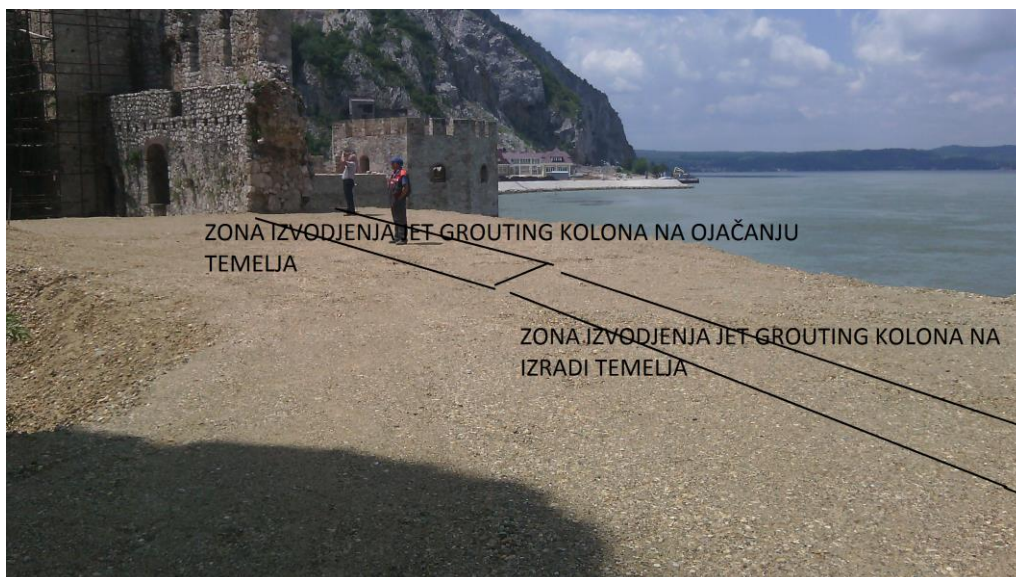


Slika 1 - Položaj zidova nekadašnje i buduće palate

2 OPIS TEHNIČKOG REŠENJA

2.1 PRIPREMNI RADOVI

Da bi se radovi mogli izvoditi, potrebno je bilo prethodno nasuti lokaciju i obezbediti pristup mašini koja izvodi radove. Nasip se izvodio od kamenog nabačaja ka toku reke Dunav, dok je u zoni formiranja kolona potreban nasip od mešavine šljunka. Ispod nasutog materijala je muljevito tlo, ispod koga zaleže stenska krečnjačka masa. Jet grouting kolone se izvode do gornje ivice stenske mase. Izvodjenje jet grouting kolona je vršeno sa kote terena tj. kote vrha nasipa(slika 2). Zbog nepristupačnosti za rad bagera i ostalih mašina za nasipanje sa strane drumskog saobraćaja, celokupno izvodjenje nasipa (doprema materijala,opreme..) je radjeno sa barže na Dunavu.



Slika 2 - Radni plato za izvodjenje jet grouting kolona od šljunčanog nasipa

2.2 OPŠTE O „JET GROUTING“ METODI

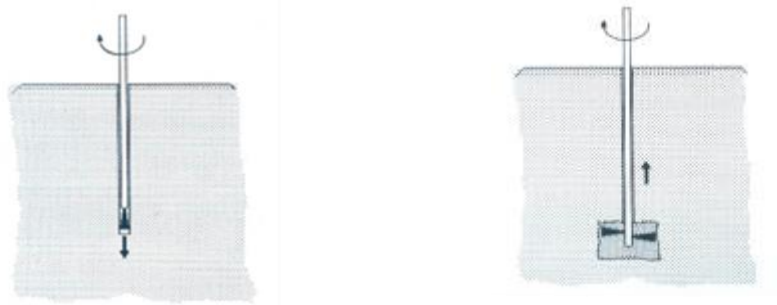
Jet grouting je savremena tehnologija geotehnike. U samom početku razvoja koristio se isključivo u funkciji poboljšanja mehaničkih karakteristika tla, tj. ojačanja tla. Jet grouting se može primenjivati u svim vrstama tla, od glina do krupnozrnih šljunkova nezavisno od granulometrijskog sastava i poroznosti. Osnovna karakteristika jet grouting-a je da se bušenjem do potrebne dubine pod visokim pritiskom u tlo injektira cementno mleko, koje se meša sa degradiranim tlom (usled velikog pritiska) i sa česticama tla (agregat) formira kolonu čije su čvrstoća i/ili propustljivost poboljšane u odnosu na postojeće tlo (slika 3).



Slika 3 - „Jet grouting“ kolone

Faze izvođenja jet grouting-a:

- Bušenje do potrebne dubine. Najčešće se već i u fazi bušenja kroz alat za bušenje pušta voda pod pritiskom da bi se u izvesnoj meri degradiralo tlo, što olakšava bušenje i poboljšava uslove za krajnje injektiranje.



Slika 4 - prva faza levo: bušenje, druga faza desno: injektiranje

-Pri dostizanju zadate dubine bušenja, kroz alat za bušenje se injektira cementno mleko i simultano rotira i podiže. Ovaj proces je unapred programiran, i vrši se automatski. Pritisak injektiranja je oko 400 bara. Kroz horizontalne mlaznice koje se nalaze iznad bušaće krune čestice jeta ističu brzinom od oko 200m/s. Kako se bušaća šipka i kruna tokom izvlačenja i injektiranja rotiraju, formira se homogeno cilindrično telo sastavljeno od injektiranog cementnog mleka i čestica tla. Oblik i veličina tela kolone zavise od zahteva projekta i mogu biti različiti u zavisnosti od pritiska injektiranja, brzine izvlačenja i rotiranja alata, vodocementnog faktora, kao i granulometrijskog sastava tla. Kolone se mogu izvoditi vertikalno, ili pod uglom. Sav materijal koji se koristi u procesu izrade jet grouting kolona (voda, cement i tlo) su prirodni materijali, koji nemaju negativan uticaj na okolinu i na nivo podzemne vode. **Radijus dejstva** jet kolone je u funkciji više faktora :

radnog pritiska,
vremena injektiranja,
smičuće čvrstoće tla,
prečnika mlaznica,
specifične težine cementnog mleka.

Tri osnovne metode mlaznog injektiranja:

monofluidna tehnologija (jet – 1),

dvofluidna tehnologija (jet – 2),

trofluidna tehnologija (jet – 3).

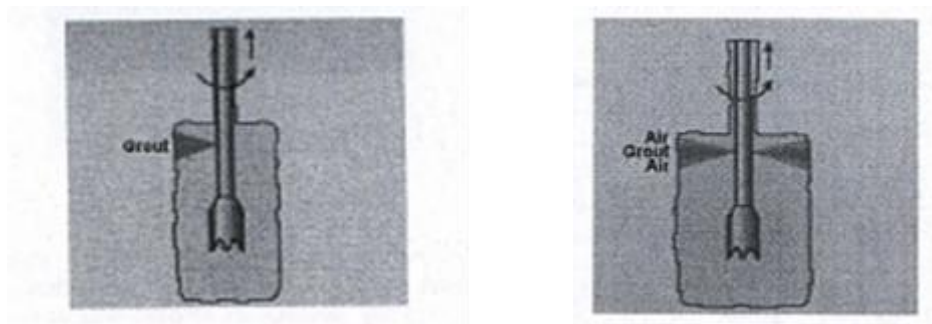
Jet 1 :

U ovom postupku koristi se samo jedan fluid za razaranje i mešanje sa tlom. Najčešće je to cementno mleko koje se kroz mlaznice na šipki za bušenje pod velikim pritiskom (oko 400 bara) injektira u tlo, razara ga, a potom se meša sa in-situ tlom. Prečnik tela dobijenih ovom tehnologijom kreću se od 40 do 120 cm. Za čiste peskove dobijaju se veći prečnici, dok za gline najmanji. Jet grouting kolone izvedene na ovaj način su najhomogenijeg sastava, tj. imaju najveću čvrstoću. Oprema potrebna za izvođenje kolona monofluidnom tehnologijom:

pumpa visokog pritiska,

hidraulička bušača garnitura opremljena odgovarajućim tajmerom koji precizno kontroliše korak podizanje monitora
 mikser i agitator,
 silos za cement.

Monofluidna tehnologija je najbrža, jer koristi jedan fluid, pa je samim tim i oprema nešto jednostavnija, i cena izrade niža. Po pravilu, ovaj postupak se koristi u nevezanom tlu.



Slika 5 - jet 1, sl. levo(cementna emulzija); jet 2, sl. desno(cementna emulzija i vazduh)

Jet 2 :

Korišćenjem specijalnih šipki sa dvostukim mlaznicama (jedan kanal za ubrizgavanje cementnog mleka, a drugi za vazdušnu struju) dolazi do udruživanja dve kinetičke energije : cementnog mleka i vazdušne struje. Ovim postupkom radi se na većim dubinama, i dostižu se veći prečnici nego monofluidnom tehnologijom. Obično prečnik kolona izvedenih dvofluidnim postupkom iznosi od 100 – 180 cm, ali su manjih čvrstoća, zbog prisustva vazduha koji stvara šupljine u telu. Oprema za izvođenje ovom tehnologijom obimnija je nego oprema kod jednofluidnog postupka. Između ostalog potreban je i kompresor koji bi davao potreban pritisak vazduha.

Jet 3:

U ovom slučaju potrebna je burgija sa tri nezavisna kanala. Pored cementne emulzije i vazduha koji se ubrizgavaju dvofluidnom tehnologijom, postoji i treći fluid – voda pod pritiskom od 200-500 bara. Postoji odvojen set mlaznica za vodu i vazduh, a ispod njih set mlaznica za upumpavanje injeksione mase. Molekuli vazduha obavijaju vodeni mlaz, i dolazi do “airlifting” efekta – vazduh seče tlo. Zatim se cementno mleko pod manjim pritiscima (30-70 bara) injektira u stvorene šupljine i popunjava ih. I ovde dolazi do superpozicije sve tri kinetičke energije. Ovim postupkom dobijaju se veći prečnici nego kod dvofluidnog postupka, za peskovita tla 150-250 cm. Ova tehnologija je najkompleksnija i zahteva dodatnu opremu.

Jet grouting parametri koji se određuju, zavise od zahtevanih karakteristika gotovih kolona, karakteristika tla i izabrane tehnologije izvođenja.

Glavni jet grouting parametri su:

- brzina povlačenja garniture ka površini,
- brzina rotacije garniture.
- pritisak injektiranja,
- količina cementa,
- vodocementni factor,

broj i prečnik mlaznica,
pritisak vazduha

Čvrstoća elemenata zavisi od sledećih faktora:

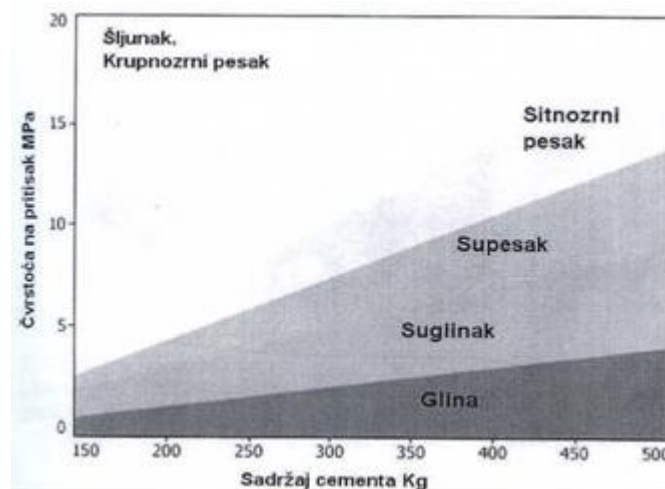
vodocementnog faktora ($W/C \uparrow$, čvrstoća \downarrow),

sadržaja cementa ($C \uparrow$, čvrstoća \uparrow),

vrste postupka (monofluidni, dvofluidni, trofluidni),

vrste i granulometrijskog sastava tla,

Na sledećoj slici su prikazane procenjene očekivane vrednosti čvrstoće tla stubova za odgovarajuće vrste tla:

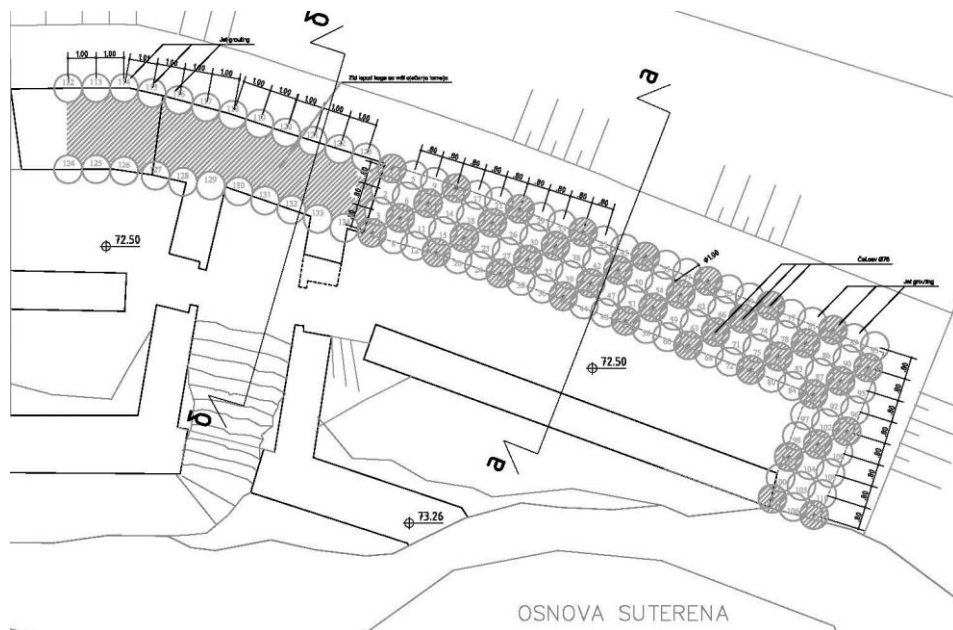


Slika 6 - čvrstoće „J.G.“ kolona u zavisnosti od vrste tla u kojem se radi i sadržaja cementa

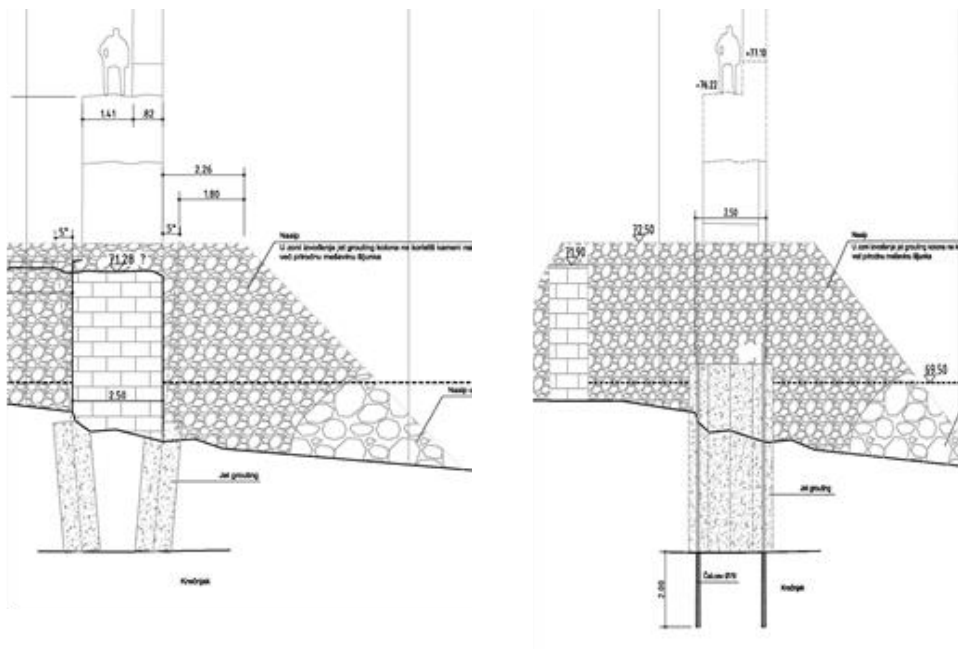
2.3 FUNDIRANJE ZIDOVA PALATE „JET GROUTING“ METODOM

Iz razloga ekonomičnosti, dinamike, kao i geoloških uslova, za temeljenje zidova palate Golubačke tvrđave usvojena je metodologija dvofluidnog „jet groutinga“, koja će stvarati prečnike kolona minimum od 1m (slika 7). Tlo u kome se vrši jet grouting se sastoji od aluviona muljevitog peska, peskovitog sljunka i šljunkovitog peska. Na delu „ojačanja“ temelja ispod postojećeg zida jet grouting metodom se odlučilo na takav pristup u vidu blago zakošenih jet grouting kolona zbog pretpostavke da taj zid nije oslonjen na stensku masu kao ni temeljen. Tačnije, prvi graditelji zidova palate su zid oslanjali na kamene nabačaje koji su davali dobar vertikalni oslonac. Pretpostavlja se da je upravo iz razloga takvog „temeljenja“ došlo do kolapsa zidova, odnosno cele palate. Buševići neposredno ispod postojećeg zida pod uglom od 5° upravo se nailazilo na kamenje kroz i pored koga se moralo proći „krunom“ za bušenje, sve do kontakta sa krečnjačkom masom. Kada se došlo do krečnjačke mase, podizanjem na gore i istovremenim injektiranjem cementne emulzije (jet grouting) vršio se poduhvat postojećeg zida.

Jet grouting kolone ispod postojećeg zida palate (na delu **ojačanja** temelja) imaju funkciju poboljšanja mehaničkih karakteristika tla ispod temelja. Izvodile su se na razmaku od 1,0m sa obe strane zida pod blagim zakošenjem od 5° (slika 7). Kolone, kao što je rečeno, formiraju se do gornje ivice stenske mase (slika 8 levo).



Slika 7 - Dispozicija „jet grouting“ kolona (šrafirane predstavljaju kolone u koje se ubacuju čelične cevi)

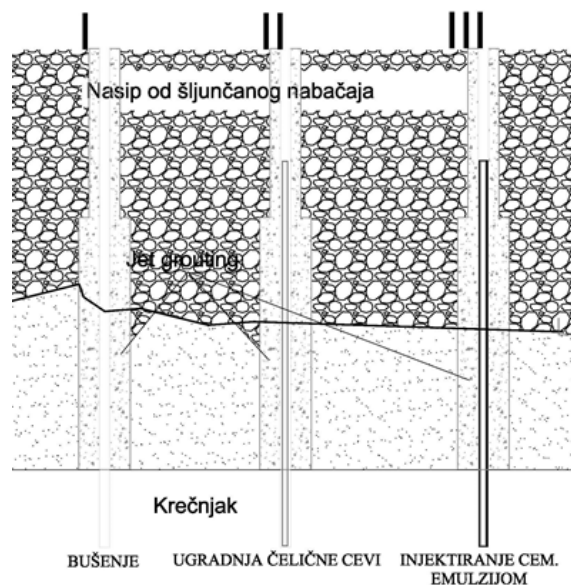


Slika 8- Presek b-b, sl. levo (ojačanje temelja); Presek a-a, sl. desno (izrada temelja)

Na delu izrade temelja izvelo se bušenje kolona u više redova, sa preklopom, kako bi oformili podlogu za budući zid minimalne širine 2,5m. Opet se sa bušenjem išlo do stenske mase i odatle na gore sa samom izradom jet grouting kolona (slika 8 desno). Kao i na delu ojačanja temelja i ovde se na većim dubinama tokom bušenja nailazilo na odlomke kamenja. Ove kolone izvodile su se vertikalno, na razmaku od 80cm, u prečniku od 100cm (Slika 7).

2.4 OSTVARIVANJE VEZE SA ZIDOM PALATE

Kako bi temelj ostvario vezu sa zidom iznad, u pojedinim kolonama na delu izrade temelja se nakon same izrade ugrađivala čelična cev (Slika 7). Ove cevi zaležu u stensku masu 2,0m, i prepuštene su sa gornje strane 0,5m, za vezu sa budućim zidom (Slika 8, desno). Bušenje pre same ugradnje cevi se vršilo takozvanim dubinskim čekićem koji ostvaruje bušotinu u prečniku od 90 mm. Da bi dubinski čekić mogao da dejstvuje duž bušotine treba da ima određeni otpor podloge. S tim u vezi kolone u kojima su se ugrađivale čelične cevi su radjene u potpunosti (do vrha nasipa) kao jet grouting kolone, pošto se i bušenje, kao i ugradnja cevi vršila sa kote radnog platoa. Kao dokaz uspešno izvedenih jet grouting kolona je i njihova čvrstoća koja je omogućila nesmetan rad dubinskog čekića duž bušenja u vidu dovoljnog otpora. Nakon bušenja kroz jet grouting kolone i krečnjak ubacivale su se cevi i vršilo injektiranje cementnom emulzijom. Ceo gore navedeni postupak ostvarivanja veze temelja sa budućim zidom se može prestaviti u tri faze (slika 9).



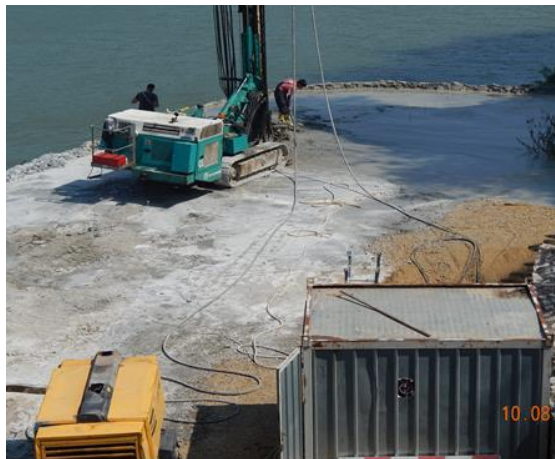
Slika 9- Tri faze opisivanja postupka ostvarivanja veze temelja sa budućim zidom



Slika 10- Bušenje sa dubinskim čekićem

3 ZAKLJUČAK

Pretpostavka o uspješnom i efikasnom izvršenju posla izrade i ojačanja temelja za zidove palate tvrđave Golubački grad u datim (otežanim) uslovima sa jet grouting metodom je bila opravdana. Naime celokupno izvodjenje je vršeno sa kote terena, tj. kote vrha nasipa, bez kakvog podgradjivanja zbog dotoka vode. Vremenski period za koji su svi radovi izvršeni je daleko manji od onog kojeg bi zahtevala druga potencijalna rešenja. Sam odabir dvofluidnog sistema jet grouting tehnologije se pored efikasnosti ispostavio i ekonomičniji (manje bušotina, manje vremena).



Slika 11 - Bušaća garnitura Casagrande „C6“ (gore levo na sl.) na koju se „kače“ creva sa dotokom fluida cementne emulzije i vazduha(od kompresora: dole levo na sl.)

LITERATURA

- [1] Arhiva Novkol,
- [2] Projekat revitalizacije tvrđave „Golubaki grad“



SPONZORI SIMPOZIJUMA

Shandong Hi-Speed Group је групација чија је основна делатност улагање, изградња и управљање ауто-путевима, мостовима, железничким пругама, лукама и аеродромима, као и пословање у области логистике. Поред тога, бавимо се и следећим областима: банкарство, некретнине, информације, грађевинарство, грађевински материјал. Ми смо компанија која обједињује све набројане делатности и на тржишту смо познати као савремена, интернационална и високоефикасна државна мегакомпанија.



Основни капитал компаније износи 3,01 милијарду долара, вредност укупне aktive је 75,2 милијарде долара, а у компанији је запослено више од 70 хиљада радника. Према укупној активи, Shandong Hi-Speed Group се налази испред свих кинеских компанија у истој бранши, а у последњих девет година налазимо се у групи "500 најбољих кинеских предузећа".

Групација Shandong Hi-Speed Group је до сада пословала у 106 држава и подручја света, а укупна вредност улагања износи 15 милијарди долара. Изградили смо и управљамо 2800 км ауто-путева, у овом тренутку градимо 1089 км ауто-путева; само у провинцији Шандонг смо изградили 586 км железничке пруге, а тренутно градимо 743 км

пруге, међу којима је и деоница пруге Пекинг-Шангај у провинцији Шандонг. У изградњи је 308 км брзе пруге Ђинан-Ћингдао, у тај пројекат је уложена 9,1 милијарда долара.



17. јуна 2016. године кинески председник Сји Ђинпинг (Xi Jinping) и председник Републике Србије Томислав Николић су положили камен-темељац за Кинески културни центар, чију ће градњу финансирати и који ће изградити Shandong Hi-Speed Group. Ово ће бити први Кинески културни центар на Балкану.



Shandong Hi-Speed Group у Србији гради деоницу ауто-пута Е763, од Обреновца до Љига укупне дужине 50,2 км.



Најдужи прекоморски мост на свету дугачак је 42 км, то је мост Shandong Hi-Speed залив Ђиаоџу, чију изградњу је финансирала, изградила и сада управља њиме наша компанија. Shandong Hi-Speed Group је за изградњу тога моста 2013. године награђена највишим светским признањем у области мостоградње „Медаљом Џорџа Ричардсона“, која је сматра "Нобеловом наградом" у области мостоградње.



Shandong Hi-Speed Group је 2015. године купила 49,99% акција аеродрома у Тулузу, који је четврти аеродром по количини путничког, а други по количини робног саобраћаја у Француској. Аеродром у Тулузу је први инострани аеродром који је купила кинеска компанија.



ENERGOPROJEKT



ULOG ZA NAREDNIH 65 GODINA USPEHA!

ULOG ZA NAREDNIH 65 GODINA USPEHA!



Energoprojekt se od 1951. godine kada je osnovan, razvio u složen poslovni sistem koji obuhvata 10 kompanija sa sedištem u Beogradu i preko 30 kompanija u svetu.

Kao pouzdan, efikasan i kreativan partner, Energoprojekt uspešno povezuje tehničko-tehnološko znanje i razumevanje specifičnosti lokalnih tržišta sa bogatim međunarodnim iskustvom. Sve ovo čini osnovu za širok i sveobuhvatan spektar projektantsko-konsultantskih, izvođačkih i inženjering usluga koje pruža svojim klijentima.

Energoprojekt je aktivan u oblastima proizvodnje, transporta i distribucije energije, infrastrukture, visokogradnje, vodoprivrede i zaštite životne sredine, industrije, informacionih tehnologija i osiguranja.

U sedam decenija postojanja Energoprojekt je projektovao i/ili izgradio projekte u preko 70 zemalja sveta (50 hidroelektrana, 34 termoelektrane, 15 energana, 20 toplana, 34 sistema danjinskog grejanja, preko 200 trafostanica, više hiljada kilometara dalekovoda, 50 postrojenja za preradu voda, 40 hidromelioracionih sistema, veliki broj objekata putne infrastrukture, preko 1.300 složenih projekata u oblasti industrije, 100 javnih objekata, 10 konferencijskih centara, preko milion m² komercijalnih prostora i preko milion m² prostora različite namene).

Sistem Energoprojekt danas ima ugovorenih projekata u vrednosti od preko 450 miliona evra i prosečan godišnji prihod od 300 miliona evra.

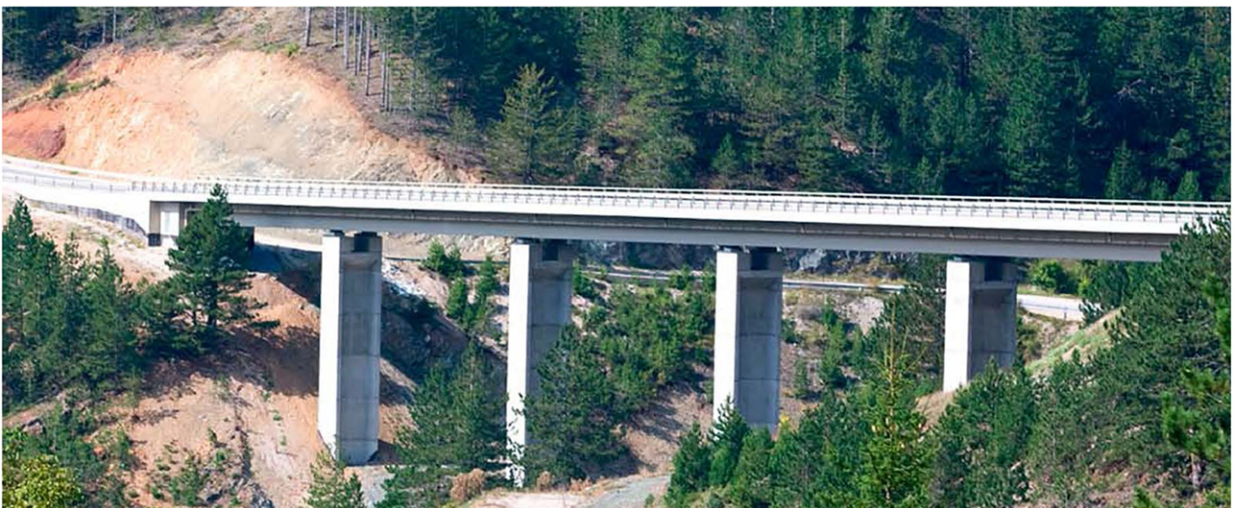
U uslovima stalno rastuće konkurencije - sinergija znanja, poslovne kompetencije i iskustva, potkrepljena impresivnom listom realizovanih projekata i klijenata u zemlji i inostranstvu, glavna su prednost i ključ uspeha SISTEMA ENERGOPROJEKT.





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- 6 CENTRALNI MAGACIN
- 7 UPRAVA
- 8 TRANSPORT
- 9 POGON SEPARACIJE

ŠIRBEGOVIĆ betonske konstrukcije za cijelu Regiju

U proteklih 25 godina kompanija **Širbegović** je izgradila preko 7 miliona m² prostora i time stekla imidž vodeće građevinske kompanije u Regiji kada su u pitanju betonske konstrukcije. Sjedište kompanije **Širbegović** nalazi se u Gračanici, Bosna i Hercegovina, sa podružnicama u Srbiji i Hrvatskoj. Osnovna djelatnost kompanije je projektovanje, inženjering te proizvodnja, transport i montaža prefabrikovanih armirano-betonskih i prednapregnutih konstrukcija koje imaju široku primjenu u izgradnji komercijalnih objekata, infrastrukture kao i stambenih objekata.

Kompanija **Širbegović** preko 25 godina nudi tehnički razvijena rješenja visokog kvaliteta, uz maksimalno uvažavanje estetsko-funkcionalnog aspekta objekata nastojeći da pronađe ravnotežu između arhitekture i građevinarstva i da uvijek bude u koraku sa vremenom odgovarajući na najraznovrsnije zahtjeve kupaca.

Kvalitet, brzina izvođenja radova, mogućnost gradnje kompleksnih objekata, iskustvo u pronalaženju tehničkih rješenja, maksimalno poštivanje rokova i cjelovita ponuda aduti su koji stvaraju ključnu konkurentnu prednost kompanije **Širbegović**.

Tokom dosadašnjeg poslovanja kompanija **Širbegović** je izgradila preko 4000 objekata, odnosno preko 7 miliona m² prostora i time zasluženost postala jedna od vodećih građevinskih kompanija na regionalnom tržištu kada su u pitanju betonske konstrukcije. Instalirani proizvodni kapaciteti se prostiru na 150.000 m² zemljišta i više od 35.000 m² objekata te omogućavaju izgradnju cca 800.000 m² betonskih konstrukcija godišnje.

Bitno je naglasiti da je kompanija Širbegović prva u Bosni i Hercegovini ispunila zahtjeve evropskih normi kojim se obezbjeđuje i dokazuje kvalitet betonskih konstrukcija.

Standardi o kvaliteti koji važe u EU su integralni dio proizvodnje, te sa njima rade i na tržištima na kojima nije obavezno da se primjenjuju.

To se prije svega odnosi na tržište Bosne i Hercegovine, Srbije i Crne Gore gdje ovi standardi još uvijek nisu stupili na snagu.

Osim toga kompanija **Širbegović** je od instituta IGH dobila certifikat kojim se potvrđuje vrhunska kvaliteta prefabrikovane konstrukcije i posebno visoki razredi čvrstoće, što je i potvrđeno razredom tlačne čvrstoće betona MB70 (C55/67) koji je postao uobičajena proizvodnja.

Ovo je prvi certifikat u regiji sa ovako visokim čvrstoćama betona, čime je kompanija Širbegović postavila nove standarde kvaliteta.

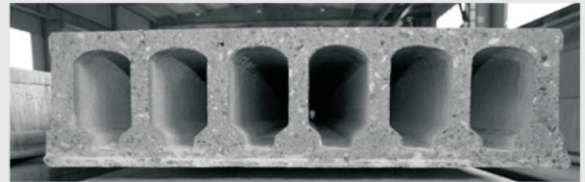
KVALITET JE NAŠ PRIORITET

- NORMA EN 13369:2013
- NORMA EN 206:2013
- NORMA EN 1168:2005+A3:2012
- NORMA EN 13225:2013
- NORMA EN 14992:2007+A1:2012
- NORMA EN 14991:2007
- DBV/ BDZ UPUSTVO ZA VIDNI BETON

- BETONSKE KONSTRUKCIJE
- EU NORME
- IZGRAĐENIH 7 MILIONA m²



Prefabrikacija je savremeni sistem gradnje kod kojeg se konstruktivni elementi objekta proizvode serijski u fabrici, a potom transportuju do mjesta gradnje gdje se vrši montiranje objekta. Prednapregnute olakšane ploče (PPS) su montažni konstruktivni elementi sa kontinuiranim unutrašnjim šupljinama koje umanjuju težinu elemenata i stvaraju efikasan nosivi presjek. Riječ je o jedinstvenom proizvodu, raznolike primjene, sa garancijom velike vrijednosti u odnosu na ulaganje. Ove ploče mogu bez oslonca da premoste raspone do 19 metara.



VIZIJA:

"Brzo i sigurno uz primjenu savremenih tehnologija."

Namjera kompanije **Širbegović** je ostati lider na tržištima na kojima djeluje i dalje se širiti na tržištu EU, te sve više jačati brend **Širbegović**. Tržišni udio planira povećavati ulaganjem u kvalitet, širenjem ponuda i usluga te jačanjem odnosa sa kupcima, čije će zadovoljstvo ostati primarni cilj poslovne politike.

Kompanija **Širbegović** planira i dalje razvijati transparentnost komuniciranja i saradnje sa projektantima i generalnim izvođačima, te stalno povećavati produktivnost putem inovacija i moderniziranja cjelokupne organizacije i korporativnog imidža.



MONTAŽNE ARMIRANO BETONSKE KONSTRUKCIJE

Beton je jedan od najrasprostranjenijih građevinskih materijala. Osim što ima veliku oblikovnu moć, veliku nosivost na pritisak i krutost, beton je takođe veoma isplativ materijal. Prefabrikovane armirano betonske konstrukcije imaju izrazito dobre protivpožarne osobine. Proizvode se u zatvorenom prostoru i nude čitav niz različitih tehničkih rešenja visokih estetskih i funkcionalnih vrednosti.

U pogonima **MARTINI GRADNJE** odvija se i proces prednaprezanja konstruktivnih elemenata, kojim se pre svega postiže bolja nosivost i stabilnost svakog pojedinačnog elementa. Prefabrikovana betonska konstrukcija sastoji se od temelja, stubova, greda, međuspratnih ploča, krovnih nosača i zidnih panela. Ovim se omogućava građenje različitih objekata upotrebom relativno malog broja tipskih elemenata, proizvedenih u fabrici u velikim serijama. Proizvodnja ne zavisi od vremenskih uslova i može se odvijati tokom cele godine.

U niz objekata koji se mogu izraditi prefabrikovanim betonskim elementima spadaju: industrijski objekti, logistički centri, višespratni tržni centri i garaže, hoteli, stambene i upravne zgrade, kao i mnogi drugi.

Martini Gradnja je u mogućnosti da sa svojim stručnim timom, a na osnovu projektnog zadatka, klijentu ponudi svoje tehničko rešenje objekta. Shodno zahtevima investitora, prefabrikovani elementi se mogu prilagoditi već postojećem projektu.



 **MARTINI**
GRADNJA D.O.O.

Sa prometom od preko milijardu evra u 2014. godini PERI je najveći svetski proizvođač i isporučilac sistemskih oplata i skela za sve vrste betonskih radova. Ovaj koncern zapošljava preko 7.000 ljudi u 56 zemalja na svim kontinentima.



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SISTEMSKA REŠENJA KONSTRUKCIJE U SVAKOM DETALJU

Ankeri za dizanje

Sigurna manipulacija montažnim betonskim elementima.



Ivični elementi

Zaštita ivice betonskih elemenata.



Anker ploče

Za zavarivanje na površini betona.

DELTABEAM greda

Savršena spregnuta greda za konstruisanje tanke međuspratne ploče u kombinaciji sa svim vrstama međuspratnih elemenata: šuplje ploče, filigran ili monolitne ploče.



PCs konzola

Skrivena alternativa betonskoj konzoli.



Spregnuti stubovi

Vitka alternativa AB stubovima. Kompletna nosiva konstrukcija u kombinaciji sa Deltabeam gredama prožarne otpornosti do 180 minuta bez dodatne zaštite.



Peikko vijčana veza AB stubova

Efikasna i brza veza AB stuba sa temeljnom konstrukcijom ili nastavljjanje stuba na stub primenom Peikko sidrenih vijaka i Peikko papuča stuba.



Sidreni vijci

Za svestrana rešenja prenosa sile zatezanja, pritiska i smicanja.



PSB armatura za osiguranje proboja

Omogućava tanke AB ploče oslonjene direktno na stubove. Najefikasnije rešenje kod međuspratnih i temeljnih ploča.

138 ГОДИНА ПОВЕРЕЊА.



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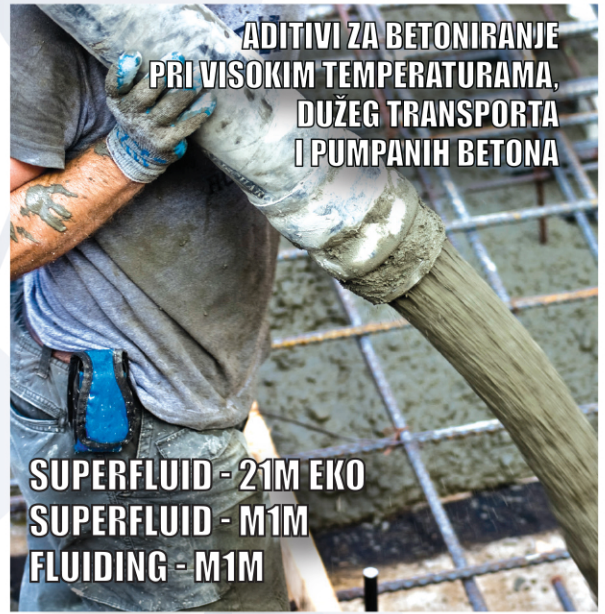
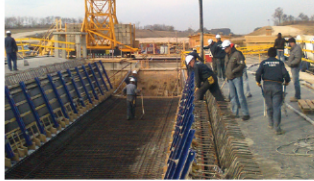


Кроз дугу традицију свог постојања, рада и развоја Саобраћајни институт ЦИП д.о.о. је израстао у једну од највећих истраживачко-пројектних компанија у региону. Основан за пројектовање прве железничке пруге у Кнежевини Србији, пројектовао је све пруге у Србији и бившој Југославији. Данас, Саобраћајни институт ЦИП д.о.о. покрива комплетне услуге од извођења геодетских радова, геолошких истраживања, лабораторијских испитивања из области заштите животне средине, израде студијске, планске и техничке документације, стручне и техничке контроле техничке документације, испитивања конструкција, стручног надзора у току изградње објеката, техничког прегледа објеката, инжењеринг - консалтинг услуга. Пројекти железничке и друмске инфраструктуре, градских саобраћајних система, објеката високоградње, спортских и специјалних објеката, сталне су активности на којима се доказује високим квалитетом и кратким роковима. Наведене активности се одвијају у оквиру 13 организационих јединица, а изводе у складу са одговарајућим документима Система менаџмента квалитетом ISO 9001 и ISO 17025, законом о планирању и изградњи, осталим референтним законима и подзаконским актима, као и општим актима Саобраћајног института ЦИП д.о.о.

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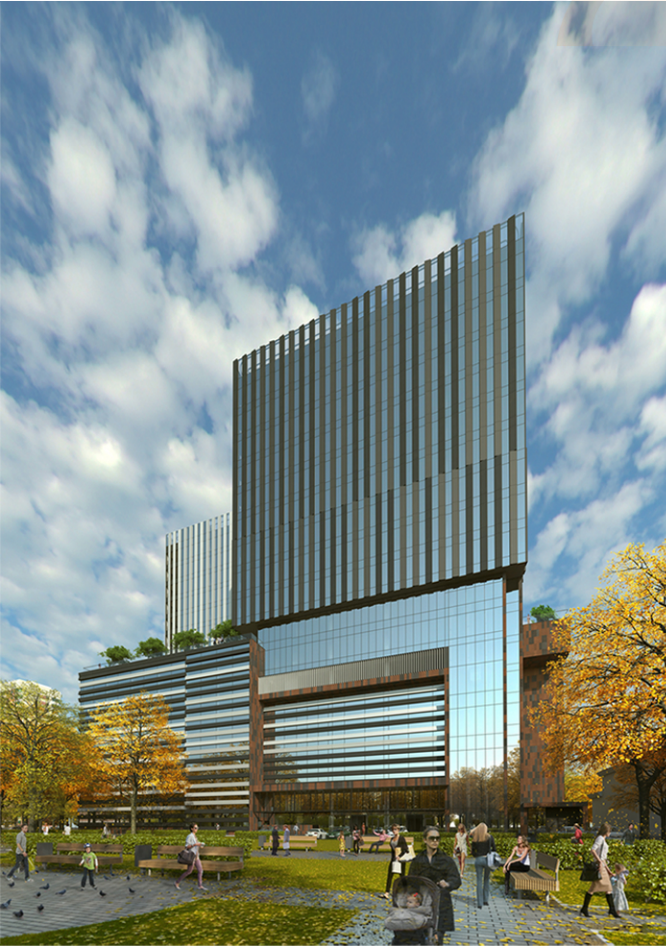
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Doka Serb je srpski ogranak austrijske kompanije **Doka GmbH**, jednog od svetskih lidera na polju razvoja, proizvodnje i distribucije oplatnih sistema za sve oblasti građevinarstva. Osnovna delatnost kompanije Doka Serb jeste isporuka oplatnih sistema i komponentni za primenu u visokogradnji i niskogradnji, kao i pružanje usluga konsaltinga, izrade tehničkih planova i asistencije na gradilištu od strane Doka instruktora, i to na teritoriji Srbije, Crne Gore i Makedonije. Zahvaljujući bogatoj ponudi inovativnih i visokokvalitetnih proizvoda i usluga, kao i efikasnoj globalnoj distributivnoj mreži, Doka omogućava građevinskim kompanijama da grade brže, bezbednije i ekonomičnije.

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