**Univerzitet Crne Gore**
**GRAĐEVINSKI FAKULTET**
81000 Podgorica
Džordža Vašingtona b.b.

Tel: 020 245 014, 244 905
Fax: 020 241 903
Website: *www.gf.ac.me*
E-mail: *gf@ac.me*
Žiro račun: **510-278-79**
**530-13649-97**

Podgorica, 10.12.2018.godine
Broj: 1667

**UNIVERZITET CRNE GORE**
**- Centar za doktorske studije -**
**PODGORICA**

U prilogu vam dostavljamo predlog Vijeća Građevinskog fakulteta Univerziteta Crne Gore, za imenovanje Komisije za ocjenu doktorske disertacije.

S poštovanjem,

UNIVERZITET CRNE GORE
Građevinski fakultet

SEKRETAR FAKULTETA,
Miro Božović,dipl.prav.

# ISPUNJENOST USLOVA DOKTORANDA

| **OPŠTI PODACI O DOKTORANDU** | | | |
|---|---|---|---|
| Titula, ime, ime roditelja, prezime | Mr Nikola Milan Baša | | |
| Fakultet | Građevinski fakultet Univerziteta Crne Gore | | |
| Studijski program | Građevinarstvo - Konstrukcije | | |
| Broj indeksa | 1/10 | | |
| **NAZIV DOKTORSKE DISERTACIJE** | | | |
| Na službenom jeziku | Efekti preraspodjele uticaja na granična stanja kontinualnih greda armiranih FRP armaturom | | |
| Na engleskom jeziku | Effects of Redistribution of Internal Forces on Limit States of Continuous Beams Reinforced with FRP Reinforcement | | |
| Naučna oblast | Građevinarstvo - Konstrukcije | | |
| **MENTOR/MENTORI** | | | |
| Mentor | Prof.dr Radomir Zejak, dipl.inž.građ, redovni profesor | Univerzitet CrneGore, Građevinski fakultet, Crna Gora | Građevinarstvo – Konstrukcije |
| **KOMISIJA ZA PREGLED I OCJENU DOKTORSKE DISERTACIJE** | | | |
| Prof.dr Mladen Ulićević, dipl.inž.građ, redovni profesor | | Univerzitet CrneGore, Građevinski fakultet, Crna Gora | Građevinarstvo – Konstrukcije |
| Prof.dr Snežana Marinković, dipl.inž.građ, redovni profesor | | Univerzitet u Beogradu, Građevinski fakultet, Srbija | Građevinarstvo – Konstrukcije |
| Prof.dr Radomir Zejak, dipl.inž.građ., redovni profesor | | Univerzitet CrneGore, Građevinski fakultet, Crna Gora | Građevinarstvo – Konstrukcije |

| **Datum značajni za ocjenu doktorske disertacije** | |
|---|---|
| Sjednica Senata na kojoj je data saglasnost na ocjenu teme i kandidata | 23.06.2016. godine<br>Odluka br. 03-1634/2 |
| Dostavljanje doktorske disertacije organizacionoj jedinici i saglasanost mentora | 03.12.2018. godine |
| Sjednica Vijeća organizacione jedinice na kojoj je dat prijedlog za imenovanje komisije za pregled i ocjenu doktorske disertacije | 07.12.2018. godine |

| **ISPUNJENOST USLOVA DOKTORANDA** |
|---|
| U skladu sa članom 38 pravila doktorskih studija kandidat **je** dio sopstvenih istraživanja vezanih za doktorsku disertaciju publikovao u časopisu sa **SCI/SCIE** liste kao prvi autor. |

**Spisak radova doktoranda iz oblasti doktorskih studija koje je publikovao u časopisima sa SCI/SCIE liste:**

**Baša Nikola, Ulićević Mladen, Zejak Radomir :** Experimental Research of Continuous Concrete Beams with GFRP Reinforcement, Advances in Civil Engineering, Article ID 6532723, 2018., 16 pages. ISSN: 1687-8086 (Print), ISSN: 1687-8094 (Online), doi.org/10.1155/2018/6532723
**Link ka radu**: http://downloads.hindawi.com/journals/ace/2018/6532723.pdf

**Konferencijski radovi koji sadrže djelove disertacije:**

**Baša Nikola, Ulićević Mladen, Zejak Radomir:** Preraspodjela uticaja u kontinualnim gredama armiranim FRP armaturom, Simpozijum društva građevinskih konstruktera Srbije, Zlatibor, 15-17. septembar 2016.god, str. 328-335.

**Baša Nikola, Ulićević Mladen, Zejak Radomir:** The Response Analysis of Continuous Beams with FRP Reinforcement, Proceedings of the 1st International Conference on Construction Materials for Sustainable Future, Zadar, Croatia, 19-21 April 2017, pp. 768-776.

**Obrazloženje mentora o korišćenju doktorske disertacije u publikovanim radovima**

Mr Nikola Baša je, kao prvi autor, dio rezultata sopstvenih istraživanja vezanih za doktorsku disertaciju objavio kroz jedan rad koji je publikovan u časopisu sa SCI/SCIE liste i kroz dva rada publikovana na međunarodnim/regionalnim konferencijama. U nastavku je dat osvrt na rad objavljen u časopisu *Advances in Civil Engineering*, sa dijelom rezultata sopstvenih eksperimentalnih istraživanja na kontinualnim gredama armiranim GFRP armaturom, sprovedenih u okviru izrade doktorske disertacije.
Naslov objavljenog rada je "Experimental Research of Continuous Concrete Beams with GFRP Reinforcement". Koautori u radu su prof. dr Mladen Ulićević i prof. dr Radomir Zejak. U radu je prikazan eksperimentalni program, u okviru kog su navedeni osnovni parametri koji su razmatrani: korišćena vrsta GFRP armature i različiti odnosi GFRP armatura u kritičnim presjecima. Detaljno je opisano ponašanje razmatranih greda koje su ispitivane do loma, a koje je definisno različitim stanjima: graničnom nosivošću pri savijanju, modalitetom loma, načinom prostiranja i veličinom širina prslina, veličinama ugiba u sredinama raspona, dilatacijama u zategnutoj GFRP armaturi i pritisnutom betonu i preraspodjelom momenata savijanja u kritičnim zonama. Detaljan opis eksperimentalnog programa je prikazan u okviru poglavlja 3 doktorske disertacije, a rezultati, koji su dijelom objavljeni u radu, detaljno su prikazani u poglavlju 4 doktorske disertacije.
U radu je ukazano na sposobnost kontinualnih greda sa GFRP armaturom, da bez gubitka nosivosti vrše preraspodjelu uticaja između kritičnih presjeka, koja se prvenstveno bazira na elastičnoj preraspodjeli. Takođe, pokazano je da širina prslina uveliko zavisi od vrste GFRP armature koja se primjenjuje, odnosno od uslova prianjanja između GFRP armature i betona. Rezultati eksperimentalnih istraživanja su upoređeni sa aktuelnim propisima iz predmetne oblasti. Pokazano je da američki standard ACI 440 1R-15 veoma dobro predviđa opterećenje pri lomu kontinualnih greda sa GFRP armaturom. Takođe, pokazano je da svi aktuelni propisi potcjenjuju ugibe kontinualnih greda, posebno za više nivoe opterećenja. Iz tog razloga, kao rezultat eksperimentalnih istraživanja, u radu je od strane autora predložen modifikovani izraz za proračun ugiba kontinualnih greda sa GFRP armaturom, koji je pokazao veoma dobra

| |
|---|
| poklapanja sa eksperimentalnim rezultatima. Analize koje se tiču poređenja eksperimentalnih rezultata sa modelima proračuna aktuelnih propisa, prikazani su u tačkama 5.2 i 5.3 poglavlja 5 doktorske disertacije. |
| **Datum i ovjera (pečat i potpis odgovorne osobe)** |
| U Podgorici,<br>10.12.2018. god.<br>MP<br>DEKAN |

**Prilog dokumenta sadrži:**

1. Potvrdu o predaji doktorske disertacije organizacionoj jedinici
2. Odluku o imenovanju komisije za pregled i ocjenu doktorske disertacije
3. Kopiju rada publikovanog u časopisu sa odgovarajuće liste
4. Biografiju i bibliografiju kandidata
5. Biografiju i bibliografiju članova komisije za pregled i ocjenu doktorske disertacije sa potvrdom o izboru u odgovarajuće akademsko zvanje i potvrdom da barem jedan član komisije nije u radnom odnosu na Univerzitetu Crne Gore

Na osnovu člana 64. Statuta Univerziteta Crne Gore i člana 41. Pravila doktorskih studija Univerziteta Crne Gore, Vijeće Građevinskog fakulteta u Podgorici na sjednici održanoj 07.12.2018.godine, utvrdilo je

# PREDLOG

Predlaže se Senatu Univerziteta Crne Gore da imenuje Komisiju za ocjenu doktorske disertacije mr Nikole Baše,dipl.inž.građ., pod naslovom „Efekti preraspodjele uticaja na granična stanja kontinualnih greda armiranih FRP armaturom", u sastavu:

1. Prof. dr Mladen Ulićević,dipl.inž.građ., redovni profesor Građevinskog fakulteta Univerziteta Crne Gore.
2. Prof. dr Radomir Zejak,dipl.inž.građ., redovni profesor Građevinskog fakulteta Univerziteta Crne Gore.
3. Prof. dr Snežana Marinković,dipl.inž.građ., redovni profesor Građevinskog fakulteta Univerziteta u Beogradu.

Komisija je dužna da Vijeću Građevinskog fakulteta i Senatu Univerziteta Crne Gore, podnese izvještaj koji sadrži ocjenu doktorske disertacije, u roku od 45 dana od dana imenovanja Komisije.

- VIJEĆE GRAĐEVINSKOG FAKULTETA U PODGORICI -

DEKAN,

Prof. dr Srđa Aleksić

**VIJEĆU GRAĐEVINSKOG FAKULTETA**
**UNIVERZITETA CRNE GORE**
**PODGORICA**

***PREDMET:*** *Zahtjev za ocjenu doktorske disertacije mr Nikole Baše, dipl.inž. građ.*

Poštovani,

U skladu sa pravilima studiranja na doktorskim studijama Univerziteta Crne Gore podnosim zahtjev za ocjenu doktorske disertacije pod nazivom:

**Efekti preraspodjele uticaja na granična stanja kontinualnih greda armiranih FRP armaturom**

Završetkom doktorske disertacije i objavom rada u časopisu sa SCI/SCIE liste koji sadrži rezultate dijela sopstvenih istraživanja sprovedenih u toku izrade doktorske disertacije, ispunio sam uslove za njenu predaju. Ovim putem se obraćam se Komisiji za doktorske studije Građevinskog fakulteta da inicira predlog Komisije za ocjenu doktorske disertacije.

Uz zahtjev prilažem:

- pismenu saglasnost mentora da rad zadovoljava kriterijume doktorske disrtacije;
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- fotokopije objavljenih radova tematski vezanih za doktorsku disertaciju;
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- potpisanu Izjavu o autorstvu (prilog 1 iz Uputstva o oblikovanju doktorske disertacije).

S poštovanjem,

U Podgorici,
03. 12. 2018. godine

PODNOSILAC
Баша Никола
mr Nikola Baša, dipl.inž.građ.

УНИВЕРЗИТЕТ ЦРНЕ ГОРЕ
ГРАЂЕВИНСКИ ФАКУЛТЕТ - ПОДГОРИЦА

| Примљено | 03. 12. 2018. | | |
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| | 1631 | | |

**UNIVERZITET CRNE GORE**
**GRAĐEVINSKI FAKULTET**
**PODGORICA**

Na osnovu člana 37. Pravila doktorskih studija Univerziteta Crne Gore dajem sljedeću

SAGLASNOST

Rad pod nazivom: "Efekti preraspodjele uticaja na granična stanja kontinualnih greda armiranih FRP armaturom", autora mr Nikole Baše, dipl.inž.građ., stručnog saradnika Građevinskog fakuleta Univerziteta Crne Gore, zadovoljava kriterijume doktorske disertacije, propisane Statutom Univerziteta Crne Gore i Pravilima doktorskih studija.

U Podgorici,
29. 11. 2018. godine

MENTOR

Prof.dr Radomir Zejak, dipl.inž.građ.

## Spisak radova koji sadrže rezultate i djelove doktorske disertacije:

- radovima objavljeni u časopisima sa SCI/SCIE liste

1. **Baša N., Ulićević M., Zejak R.:** *Experimental Research of Continuous Concrete Beams with GFRP Reinforcement*, Advances in Civil Engineering, Article ID 6532723, 2018., 16 pages.

- radovima objavljeni na međunarodnim/regionalnim konferencijama

2. **Baša N., Ulićević M., Zejak R.:** *Preraspodjela uticaja u kontinualnim gredama armiranim FRP armaturom*, Simpozijum društva građevinskih konstruktera Srbije, Zlatibor, 15-17. septembar 2016., str. 328-335.

3. **Baša N., Ulićević M., Zejak R.:** *The Response Analysis of Continuous Beams with FRP Reinforcement*, Proceedings of the 1st International Conference on Construction Materials for Sustainable Future, Zadar, Croatia, 19-21 April 2017., pp. 768-776.

U Podgorici,
03. 12. 2018. godine

MENTOR

Prof.dr Radomir Zejak, dipl.inž.građ.

# Izjava o autorstvu

Potpisani: Mr Nikola Baša, dipl. inž. građ.

Broj indeksa/upisa: 1/10

**Izjavljujem**

da je doktorska disertacija pod naslovom:

EFEKTI PRERASPODJELE UTICAJA NA GRANIČNA STANJA KONTINUALNIH GREDA ARMIRANIH FRP ARMATUROM

- rezultat sopstvenog istraživačkog rada,
- da predložena disertacija ni u cjelini ni u djelovima nije bila predložena za dobijanje bilo koje diplome prema studijskim programima drugih ustanova visokog obrazovanja,
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U Podgorici

03. 12. 2018. godine

Potpis doktoranda

Баша Никола

**Univerzitet Crne Gore**
**GRAĐEVINSKI FAKULTET**
81000 Podgorica
Džordža Vašingtona b.b.

Tel: 020 245 014, 244 905
Fax: 020 241 903
Website: ***www.gf.ac.me***
E-mail: ***gf@ac.me***
Žiro račun: **510-278-79**
**530-13649-97**



**CENTAR ZA STUDIJE I KONTROLU KVALITETA**
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UNIVERZITET CRNE GORE GRAĐEVINSKI FAKULTET Nikola M. Baša EFEKTI PRERASPODJELE UTICAJA NA GRANIČNA STANJA KONTINUALNIH GREDA ARMIRANIH FRP ARMATUROM Doktorska disertacija _ Podgorica, decembar 2018. godine UNIVERSITY OF MONTENEGRO FACULTY OF CIVIL ENGINEERING Nikola M.

Baša EFFECTS OF REDISTRIBUTION OF INTERNAL FORCES ON LIMIT STATES OF CONTINUOUS BEAMS REINFORCED WITH FRP REINFORCEMENT Doctoral dissertation _ Podgorica, December 2018. godine Doktorand: Ime i prezime: Mr Nikola Baša, dipl.inž.građ. Datum i mjesto rođenja: 30.10.1980, Trstenik, Srbija Postdiplomske studije: Građevinski fakultet, Univerzitet Crne Gore, konstruktivni smjer, godina završetka: 2009. godine Mentor: Prof.

dr Radomir Zejak, dipl. inž. građ, redovni profesor na Građevinskom fakultetu Univerziteta Crne Gore Komisija za ocjenu podobnosti doktorske teze i kandidata: Prof. dr Mladen Ulićević, dipl.inž.građ, redovni profesor na Građevinskom fakultetu Univerziteta Crne Gore Prof. dr Radomir Zejak, dipl.inž.građ, redovni profesor na Građevinskom fakultetu Univerziteta Crne Gore Prof. dr Snežana Marinković, dipl.inž.građ, redovni profesor na Građevinskom fakultetu Univerziteta u Beogradu Komisija za ocjenu doktorske disertacije: Komisija za odbranu doktorske disertacije: Lektor: Milena Bajčeta Datum odbrane: Datum promocije: ZAHVALNOST Ova doktorska disertacija rađena je pod rukovodstvom dr Radomira Zejaka i dr Mladena Ulićevića, redovnih profesora Građevinskog fakulteta Univerziteta Crne Gore. Na korisnim savjetima i sugestijama prilikom izrade disertacije najsrdačnije im zahvaljujem.

Posebno zahvaljujem profesoru Mladenu Ulićeviću na dugogodišnjoj nastavnoj, naučnoj i stručnoj saradnji, tokom koje je autor stekao značajna iskustva koja su uticala na

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*Research Article*

# Experimental Research of Continuous Concrete Beams with GFRP Reinforcement

**Nikola Baša, Mladen Ulićević, and Radomir Zejak**

*University of Montenegro, Faculty of Civil Engineering, Cetinjski put bb, 81000 Podgorica, Montenegro*

Correspondence should be addressed to Nikola Baša; nik.ma@t-com.me

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Continuous beams are often used within RC structures, which are exposed to aggressive environmental impact. The use of the fiber-reinforced polymer (FRP) reinforcement in these objects and environments has a big significance, taking into account tendency of steel reinforcement to corrode. The main aim of these research studies is to estimate ability of continuous beams with glass FRP (GFRP) reinforcement to redistribute internal forces, as a certain way of ductility and desirable behaviour of RC structures. This paper gives the results of experimental research of seven continuous beams, over two spans of 1850 mm length, cross-section of $150 \times 250$ mm, that are imposed to concentrated forces in the middle of spans until failure. Six beams were reinforced with different longitudinal GFRP and same transverse GFRP reinforcements, and one steel-reinforced beam was adopted as a control beam. The main varied parameters represent the type of GFRP reinforcement and ratio of longitudinal reinforcement at the midspan and at the middle support, i.e., design moment redistribution. The results of the research have shown that moment redistribution in continuous beams of GFRP reinforcement is possible, without decreasing the load-carrying capacity, compared to elastic analysis. The test results have also been compared to current code provisions, and they have shown that the American Concrete Institute (ACI) 440.1R-15 well predicted the failure load for continuous beams with GFRP reinforcement. On the contrary, current design codes underestimate deflection of continuous beams with GFRP reinforcement, especially for higher load levels. Consequently, a modified model for calculation of deflection is proposed.

## 1. Introduction

For RC structures, elements reinforced with steel reinforcement are still used nowadays. As preventing of steel reinforcement to corrode in RC structures could be expensive and very often without significant effects, FRP internal reinforcement is lately used as a replacement of steel reinforcement in RC structures, especially in aggressive environments. Nowadays, there is a significant number of structures such as garages, bridges, retaining walls, reservoirs, and marine objects, within which FRP reinforcement is successfully applied at RC structural elements. Continuous concrete beams are commonly used in some of these structures, especially in bridges, overpasses, marine structures, and parking garages. Additionally, continuous beams with FRP reinforcement can also find their application in facilities with magnetic scanning equipment, laboratories, airport towers, and MRI rooms in hospitals and other facilities with equipment requiring electrical and magnetic neutrality, where the presence of steel reinforcement can have an adverse effect on the usability of devices in these facilities.

Due to different mechanical and deformation characteristics of FRP reinforcement, as high tensile strength and low modulus of elasticity, the behavior of RC elements is considerably different compared to RC elements with steel reinforcement. Concerning the fact that FRP reinforcement demonstrates linear elastic behavior up to failure, meaning demonstrating lack of material nonlinearity, there is a question of ability of this material, in conjunction with concrete, to realize load redistribution in statically indeterminate structures [1]. Regarding the significant contribution of the elastic redistribution in continuous RC beams with steel reinforcement [2], it is expected that continuous beams with FRP reinforcement give certain ability to redistribute the internal force. Redistribution of internal forces is expected as the result of cracks development and adopted reinforcement within them [1, 3]. In other words, one of the basic characteristics of ductility is considered, i.e., variation of stiffness without loss of capacity of the section [4].

## 2. Background

So far, thorough theoretical and experimental research studies have been carried out on simple supported beams with FRP reinforcement in order to evaluate behavior regarding failure modes, load-carrying capacity, deflection, and cracks [5–13]. Therefore, provisions of certain codes are based on conclusions given on simple supported beams. A great number of formulas and equations are suggested to determine response of elements with FRP reinforcement at service load conditions, especially when deflection is concerned (Toutanji and Saafi [7], Yost et al. [8], Bischoff and Gross [9], Mousavi and Esfahani [10], and Ju et al. [11]).

Certain experimental and theoretical research studies were also carried out on continuous beams with FRP reinforcement [1, 3, 4, 14–22], but not as much as they were carried out on simple beams. Mostofinejad [4] carried out research studies on two continuous beams with steel reinforcement and eight underreinforced and overreinforced continuous beams with CFRP reinforcement. These research studies showed that moment redistribution in continuous beams with FRP reinforcement is possible, although in lower degree than in steel-reinforced beams. Overreinforced beams with FRP reinforcement fulfill requirements of serviceability, while underreinforced beams, designed to experience failure by FRP bars, usually do not fulfill these conditions. Moreover, overreinforced continuous beams show significant deformations before failure, which was determined as a distinctive way of ductility. Grace et al. [14] researched behavior and ductility of continuous beams, T cross-section, reinforced by different types of longitudinal and transverse FRP reinforcements (GFRP and CFRP). It was indicated on different failure modes and beam ductility with FRP reinforcement, in relation to beams with steel reinforcement. It was also concluded that the use of GFRP stirrups increases shear deformations. As a result of that, total deformations increase in the midspan of continuous beams. El-Mogy et al. [1] conducted the research on four continuous beams of rectangular cross-section with GFRP, CFRP, and steel reinforcement by varying the ratio of longitudinal reinforcement in the midspan and at the middle support. It was concluded that continuous beams with FRP reinforcement are capable of redistributing the moments from middle support towards the midspan of 23% in relation to elastic analysis, similar to the fact that they do not cause adversely effects concerning the beam characteristics, neither at service loads, nor at failure loads. Moreover, it was concluded that the increase of reinforcement in the midspan of continuous beams in relation to section at the middle support has positive effects on the increase of load capacity of beams, decrease of deflections, and postponing of propagation of cracks in the beams' midspan. Habeeb and Ashour [15] noticed signs of moment redistribution within overreinforced beams in the bottom zone in the midspan, during the experimental research studies on three continuous beams with different combinations of GFRP longitudinal reinforcement in the midspan and at the middle support. As a key factor of increasing the load capacity and limitation of deflection and propagation of cracks, the increase of the reinforcement in the bottom zone of the midspan was noticed. In the scope of same experimental research studies, Ashour and Habeeb [16] conducted the research on three continuous beams with CFRP reinforcement, designed with different configurations of reinforcement along the beam, so as to experience the failure by CFRP bars. Intolerant wide cracks at the middle support were noticed in all beams as a result of debonding of CFRP bars from concrete. It was concluded that the basic parameter of increasing the load capacity of continuous beams was the amount of CFRP reinforcement in the bottom zone in the midspan. Other recent research studies on continuous beams [3, 17, 18, 20] also pointed out to the importance of increasing the reinforcement in the bottom zone of the beam midspan, as a result of redistribution of moment over the middle support.

The approach that in continuous beams reinforced with FRP reinforcement, moment redistribution in critical sections is not allowed could be conservative [21]. Therefore, it needs additional research studies. For this purpose, it is necessary to define more clearly and precisely the influence of ratio of longitudinal reinforcement at the midspan and at the middle support on behavior of continuous beams with FRP reinforcement. The main aim of these research studies is consideration of behavior of continuous beams reinforced with GFRP reinforcement during loading until failure, with different configurations of reinforcement along the beam. Thus, in these experimental research studies for the same design failure load, for every type of GFRP reinforcement, three models with different configurations of reinforcement along the beam were used. Experimental results are discussed and evaluated based on failure modes, cracking, deflection, moment redistribution, and strains in concrete and reinforcement and compared to code predictions regarding the load-carrying capacity and deflection. Based on experimental results, a modified model for better prediction of deflection of continuous beams with GFRP reinforcement is proposed.

After a literature review, it is concluded that, in very few number of experimental research studies on continuous beams with longitudinal FRP reinforcement, FRP reinforcement for stirrups was used [3, 14, 22]. In the research studies carried out on the beams with FRP reinforcement, steel reinforcement for stirrups was mainly used. In that case, a problem with corrosion of reinforced concrete elements is still present, especially in aggressive environments. The problem increases when it is known that the effects of FRP stirrups and steel stirrups on behavior of the beams are different [14]. Regarding the previously mentioned, in these experimental research studies, besides the longitudinal GFRP reinforcement, it was also used for stirrups.

## 3. Experimental Program

The experimental program consisted of six continuous beams of total length of 3940 mm, at two equal spans of 1850 mm length, with a rectangular cross-section of $150 \times 250$ mm and with longitudinal and transverse GFRP reinforcement. In addition, a single beam with steel reinforcement was adopted as a control beam. All beams were examined up to failure, loaded by concentrated forces in the middle of both spans. The beams were divided into two series, with different GFRP longitudinal bars, and all were designed for a similar failure load. Dimensions and geometry of continuous beams and load disposition are given in Figure 1.

Considering beams of Series 1, longitudinal reinforcement of beam G1-0 was designed for the elastic bending moments along the beam, while reinforcement of beams G1-15 and G1-25 was obtained for assumed moment redistribution at the middle support of 15% and 25%, respectively. For the beams G1-15 and G1-25, this meant smaller amount of reinforcement at the middle support and higher amount of reinforcement at the midspan compared to beam G1-0. In this way, for designed failure load, models with 0% (G1-0), 15% (G1-15), and 25% (G1-25) of designed moment redistribution from the middle support to the midspan were obtained. The reinforcement ratio of the beam G1-0 at the middle support has been chosen so as to be approximately 3 times higher than the balanced reinforcement ratio, which corresponds to recommendations of codes that beams with FRP reinforcement should be designed to experience concrete compression failure. In this way, it was provided, after the moment redistribution was made, that all cross-sections, in all beams of Series 1, were designed to have reinforcement ratio above the balanced reinforcement ratio. The control beam with steel reinforcement (S1-15) was designed to achieve the moment redistribution of 15% from the middle support to the midspan. The beams of Series 2 were designed in the same way as the beams of Series 1, only with different types of longitudinal GFRP reinforcement. Models with 0% (G2-0), 15% (G2-15), and 25% (G2-25) of designed moment redistribution from the middle support to the beam midspan were also adopted.

All beams were designed in accordance with ACI 440.1R-15 [23], while CSA S806-12 [24], CNR-DT-203 [25], and EC2-04 [26] were used as control. For shear reinforcement, GFRP stirrups were adopted for beams with GFRP longitudinal reinforcement, similar to steel stirrups for the beam S1-15 with longitudinal steel reinforcement. Stirrups with a diameter of 8 mm were adopted at the space of 60 mm for interior shear span and at the space of 120 mm for exterior shear span for all beams, in order to prevent beams to experience failure due to shear. Details of reinforcing for experimental models are given in Table 1.

### 3.1. Materials

*3.1.1. Reinforcement.* In these experimental research studies, two types of GFRP reinforcement were used: wrapped GFRP bars with 70% of longitudinal glass fibers (E-glass) in total volume, impregnated in the unsaturated polyester matrix for Series 1 (marked G1), and GFRP reinforcement with 75% of longitudinal glass fibers (E-glass), impregnated in an epoxy matrix for Series 2 (marked G2). GFRP reinforcement with polyester was wrapped in glass fibers, while GFRP reinforcement with epoxy resin was with rebars (Figure 2). In all beams, GFRP stirrups with polyester were used. According to the prospect of the manufacturer, GFRP reinforcement with the polyester matrix has a nominal tensile strength of $f = 700$ MPa and a modulus of elasticity of $E = 40000$ MPa, while GFRP reinforcement with the epoxy matrix has a nominal tensile strength of $f = 1100$ MPa and a modulus of elasticity of $E = 50000$ MPa. In order to define design moment redistribution more precisely and to provide identical failure load for beams of the same series, different diameters of GFRP and steel reinforcements were used along the beam. For each diameter of GFRP reinforcement, real cross-sectional areas of the bar, similar to the equivalent diameter, on at least five samples of 200 mm length were determined. Also, for each diameter of the bars, five samples were examined to tension until failure in order to define mechanical and deformation characteristics of GFRP reinforcement, all in accordance with ACI 440.3R-12 [27]. Average values of the test results are given in Table 2.

*3.1.2. Concrete.* Two designed classes of concrete of 40 MPa and 45 MPa were used in experimental research studies, for the beams of Series 1 and Series 2, respectively, in order to provide a similar failure load for all beams. For each series of beams, concrete compressive strength after 28 days was obtained according to the investigation of 8 cubes of 150 mm edge, 8 cubes of 200 mm edge, and 17 cylinders of 150/300 mm dimension. Average values of test results of concrete compressive strength on cylinders 150/300 mm are given in Table 1.

*3.2. Test Setup and Instrumentation.* Continuous beams consisted of two equal spans placed on three supports over steel bearings. End supports were designed as horizontally movable, while the middle support was designed to prevent horizontal movement. Experimental models were examined in a closed frame which consisted of a unique system of horizontal beams and vertical ties. The load was placed over two hydraulic presses, capacity of 200 kN, in the middle of both spans.

Twelve electrical strain gauges were placed on longitudinal tension reinforcement in both bottom and upper zones of each beam. Also, three strain gauges were placed on compression reinforcement according to the scheme shown in Figure 3. Two strain gauges were placed on the compression zone of continuous beams in critical sections at 1 cm from the bottom one (at the middle support), similar to that at the upper edge of concrete (in the midspan), so as

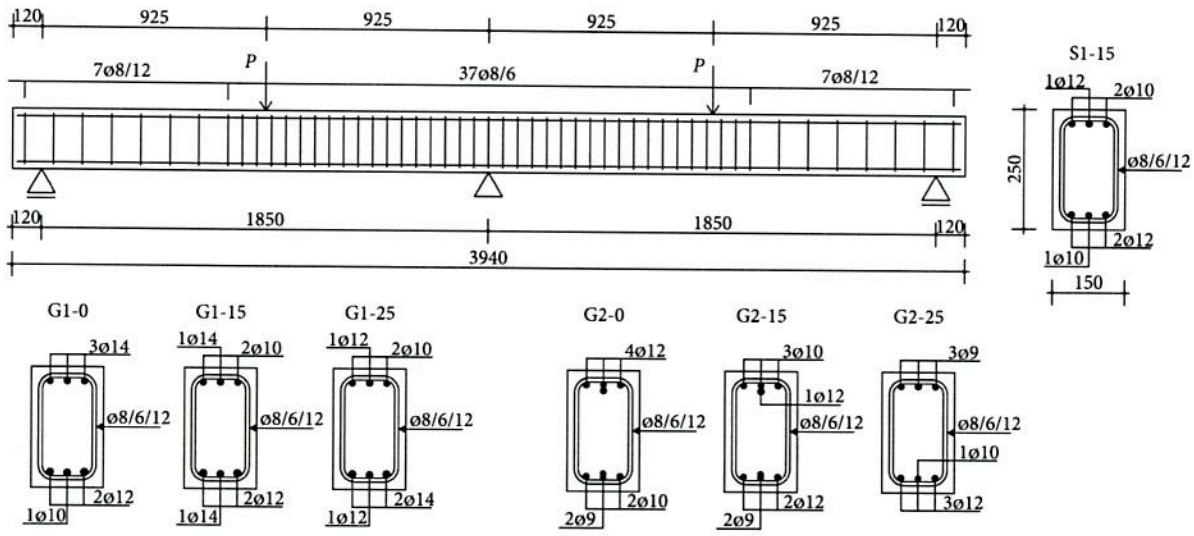


FIGURE 1: Geometry and reinforcement details for tested beams (all dimensions are in mm).

TABLE 1: Reinforcement details and compressive strength of concrete for tested beams.

| Beam | Day of testing | Middle support-top reinforcement: Longitudinal reinforcement | EA (kN) | Reinforcement ratio (ACI): $\rho_f$ (%) | $\rho_{fb}$ (%) | $\rho_f/\rho_{fb}$ | Midspan-bottom reinforcement: Longitudinal reinforcement | EA (kN) | Reinforcement ratio (ACI): $\rho_f$ (%) | $\rho_{fb}$ (%) | $\rho_f/\rho_{fb}$ | Concrete compressive strength $f_c$ (MPa) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| S1-15 | 28 | 2Ø10 + 1Ø12 | 49526 | 0.82 | 1.26 | 0.65 | 2Ø12 + 1Ø10 | 54763 | 0.92 | 1.31 | 0.71 | 42.2 |
| G1-0 | 29 | 3Ø14 | 20318 | 1.40 | 0.46 | 3.01 | 2Ø12 + 1Ø10 | 13558 | 1.00 | 0.57 | 1.75 | 42.2 |
| G1-15 | 30 | 2Ø10 + 1Ø14 | 12604 | 0.86 | 0.53 | 1.63 | 2Ø12 + 1Ø14 | 17415 | 1.19 | 0.46 | 2.58 | 42.2 |
| G1-25 | 30 | 2Ø10 + 1Ø12 | 11153 | 0.74 | 0.51 | 1.44 | 2Ø14 + 1Ø12 | 18867 | 1.25 | 0.45 | 2.80 | 42.2 |
| G2-0 | 28 | 4Ø12 | 17654 | 1.11 | 0.33 | 3.35 | 2Ø10 + 2Ø9 | 10734 | 0.65 | 0.29 | 2.27 | 50.2 |
| G2-15 | 29 | 3Ø10 + 1Ø12 | 12482 | 0.79 | 0.33 | 2.37 | 2Ø12 + 2Ø9 | 14182 | 0.86 | 0.32 | 2.69 | 50.2 |
| G2-25 | 30 | 3Ø9 | 8033 | 0.48 | 0.29 | 1.70 | 3Ø12 + 1Ø10 | 15930 | 1.10 | 0.37 | 3.02 | 50.2 |

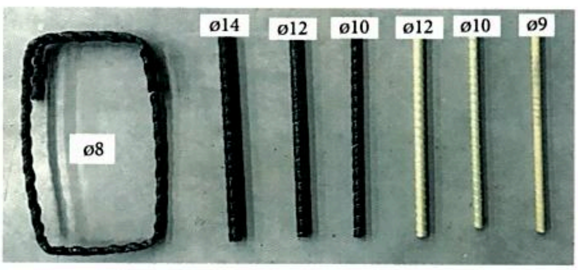


FIGURE 2: Samples of GFRP longitudinal and transverse reinforcement.

strains could be measured in compression concrete. Deflections of continuous beams along the span were registered by LVDT transducers 1/100 mm and 1/50 mm of accuracy, which were placed at the level of the bottom edge of the beam. Three LVDT transducers were attached on every span, in the middle, in the quarter, and three quarters of the beam span. For each increment, i.e., load level, appearance and development of vertical and possibly shear cracks along the beam were registered. Maximal width of a few cracks was measured in critical sections, in the span and at the middle support by a microscopic magnifier (Zeiss) with 0.025 mm of accuracy. Load cells were placed below the end supports, capacity of 100 kN, due to measurement of end reactions. The scheme of the measuring equipment of continuous beams is given in Figures 3 and 4.

*3.3. Test Procedure.* The load was applied to the beams as monotonically static growing load in increments from zero up to the failure. At the beginning of the tests, load was applied in increments of approximately 2-3 kN and after development of first cracks in increments of 5 kN. When approximately 80% of the estimated failure load was reached, again the load was applied in increments of 2-3 kN.

Table 2: Mechanical and deformation characteristics of GFRP reinforcement.

| Diameter | Real area of bar A ($mm^2$) | Tensile strength $f_u$ (MPa) | Yield strength $f_y$ (MPa) | Modulus of elasticity $E$ (MPa) | Ultimate strain $\varepsilon_u$ (‰) |
|---|---|---|---|---|---|
| GFRP-1-Ø8 | 39.9 | 714.8 | — | 42640 | 16.8 |
| GFRP-1-Ø10 | 70.6 | 703.1 | — | 41300 | 17.0 |
| GFRP-1-Ø12 | 116.1 | 865.9 | — | 45832 | 18.9 |
| GFRP-1-Ø14 | 152.8 | 813.5 | — | 44324 | 18.4 |
| GFRP-2-Ø9 | 53.3 | 1170.4 | — | 50235 | 23.3 |
| GFRP-2-Ø10 | 61.5 | 1059.3 | — | 43734 | 24.2 |
| GFRP-2-Ø12 | 91.6 | 1060.4 | — | 48182 | 22.0 |
| Steel-Ø10 | 78.5 | 639.5 | 509.6 | 188064 | 2.7[a] |
| Steel-Ø12 | 113.1 | 622.5 | 452.7 | 176835 | 2.6[a] |

[a]$\varepsilon_y$ = yield strain of steel reinforcement.

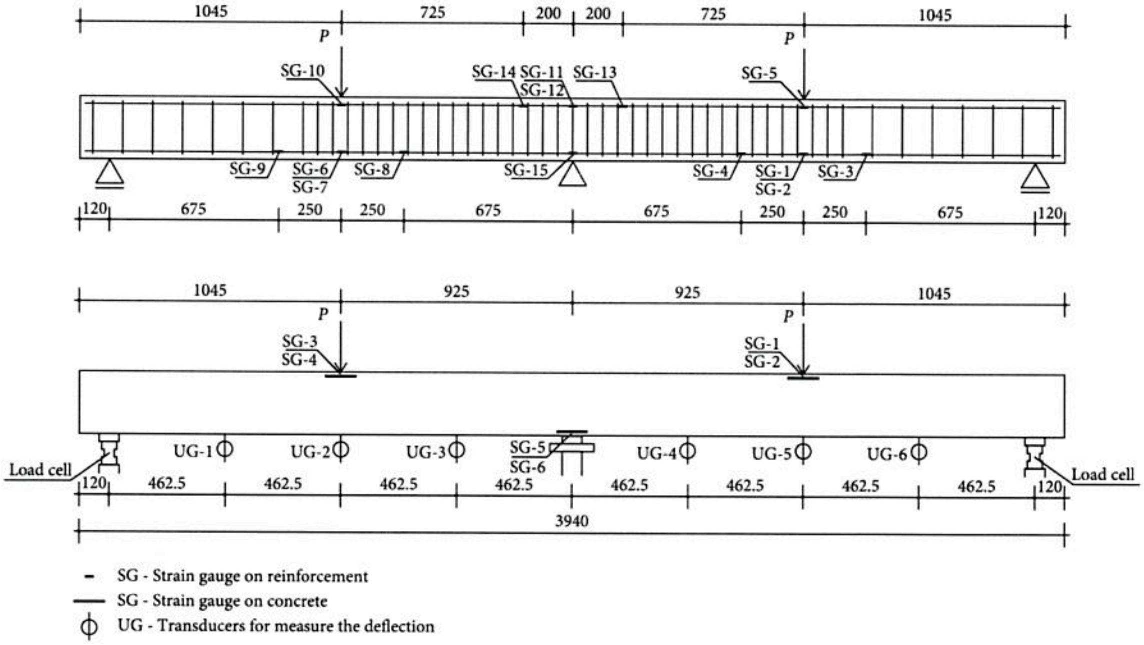


Figure 3: Experimental setup and instrumentation for tested beams (all dimensions are in mm).

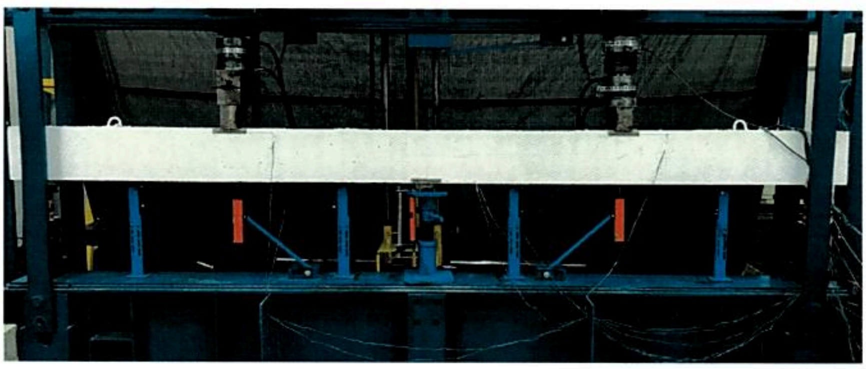

Figure 4: Equipped continuous beam before testing.

Speed of load exposure of each increment was approximately 5 kN/min. All electronic data were collected in the computer using the data logger.

## 4. Test Results

*4.1. Modes of Failure.* All beams of Series 1 and Series 2 were designed to experience concrete compression failure. Beam S1-15 demonstrated typical ductile flexural behavior, with high values of strains and deflections before failure. At the middle support, existing cracks experienced significant widths at failure. First, the tensile reinforcement yielded at the middle support, and after that, tensile reinforcement in the beam midspan. The failure of the beam G1-0 was initiated by concrete compression failure in the midspan in combination with shear, when one crack in the span diagonally propagated toward the load location. This leads to rupture of GFRP bars in the compressed zone by the dowel effect and one GFRP stirrup at the location of its bending. Within the beam G1-15, concrete compression failure took place at the middle support, when one crack near the support diagonally propagated toward the support. The failure of the beam G1-25 appeared at the same time, at the middle support, where concrete crushing was followed by the shear, and at the midspan, where concrete crushing was manifested by spalling of the cover in the extension of the diagonal crack that occurred in the exterior shear span.

The failure mode of the beams G2-0 and G2-25 was similar, initiated by concrete compression failure in the midspan in combination with shear, when one crack in the interior shear span diagonally propagated toward the load location. The failure at the midspan was followed by concrete crushing in the compression zone at the middle support within both beams. Also, in beam G2-25 at the same time, concrete crushing was manifested by spalling of the cover in the midspan in the extension of the diagonal crack that occurred in the exterior shear span. Within the beam G2-15, concrete compression failure took place at the middle support in combination with shear, with the characteristic bang. All longitudinal compressed and tensioned GFRP bars and stirrups ruptured at that section by the dowel effect. The failure modes of all beams are given in Figure 5. It can be concluded that all continuous beams with GFRP reinforcement experienced concrete compression failure combined with shear, while beam with steel reinforcement experienced ductile flexural tension failure.

*4.2. Crack Patterns.* In Table 3, failure loads are given, similar to the first crack loads in the midspan and at the middle support for all beams. It is evident that the first crack load was significantly higher in the beam S1-15, than in the beams with GFRP reinforcement, without exception. This could be attributed to high modulus of elasticity of steel reinforcement compared to GFRP reinforcement (3.8–4.5 times higher), which leads to conclusion that the cracking moment does not only depend on the concrete tension strength but also on the modulus of elasticity of reinforcement. Concerning all beams with GFRP reinforcement, the first cracks at the midspan and at the middle support were vertical, and they appeared almost simultaneously, at very similar loads. Moreover, especially for the beams of Series 1, right after the appearance, cracks significantly propagated along the height and entered the last quarter of section height, which additionally affected the decrease of stiffness of this section.

Appearance of new cracks and propagation of existing cracks in the beams of Series 1 with GFRP reinforcement stabilized at load that corresponded to approximately 60% of the failure load. The space between cracks, in average, was 120 to 180 mm, and it did not match the space between stirrups. It is evident that, in beams with GFRP reinforcement, a smaller number of cracks occurred than in the beam S1-15 with steel reinforcement, where the space between cracks was from 60 to 100 mm in the interior span, which basically matched the space between stirrups. This indicated poor bond strength between GFRP reinforcement and surrounding concrete, which caused great wideness on already developed cracks in critical sections. This phenomenon was also recorded by a few researchers who examined beams with FRP reinforcement [16]. Concerning the beams G1-15 and G1-25, it was a visible appearance of long horizontal cracks in the tension zone at loads near failure, which implies, due to large deflections, slipping of reinforcement from concrete in that part of the beam (Figure 5).

Concerning beams of Series 2, with GFRP reinforcement with rebars, the number of cracks was greater with less widths, compared to the beams of Series 1. Cracks appeared in the sagging and in the hogging moment region until failure, regarding the fact that an utmost number of them formed until the load that corresponded to approximately 60% of failure load. The development of the cracks with increasing load corresponded fully to the adopted reinforcement in critical sections; i.e., the higher amount of reinforcement corresponded to a higher number of formed cracks in the section. The largest number of cracks at the middle support formed in the beam G2-0, with an extremely wide hogging zone where cracks appeared, due to the largest axial stiffness of the reinforcement compared to the beams G2-15 and G2-25. In the midspan, the most number of cracks, with the widest sagging zone, appeared in the beam G2-25, with the largest amount of reinforcement in the midspan. The development of the cracks in the beams of Series 2 was similar to that in the beam S1-15 and corresponded to the space between stirrups, indicating the good bond strength between GFRP reinforcement and surrounding concrete. The more pronounced diagonal cracks at higher load levels for the beams of Series 2, especially in the interior shear span, compared to the beam S1-15, indicated an increase of the shear stresses in the beams with GFRP reinforcement, which can be directly attributed to the use of GFRP stirrups, instead of steel stirrups. This was particularly pronounced in the beam G2-0, in which, due to the higher axial stiffness of reinforcement at the middle support and achieved "opposite" redistribution of internal forces (Section 4.5), higher shear stresses in the interior shear span appeared, compared to the beams G2-15 and G2-25, causing

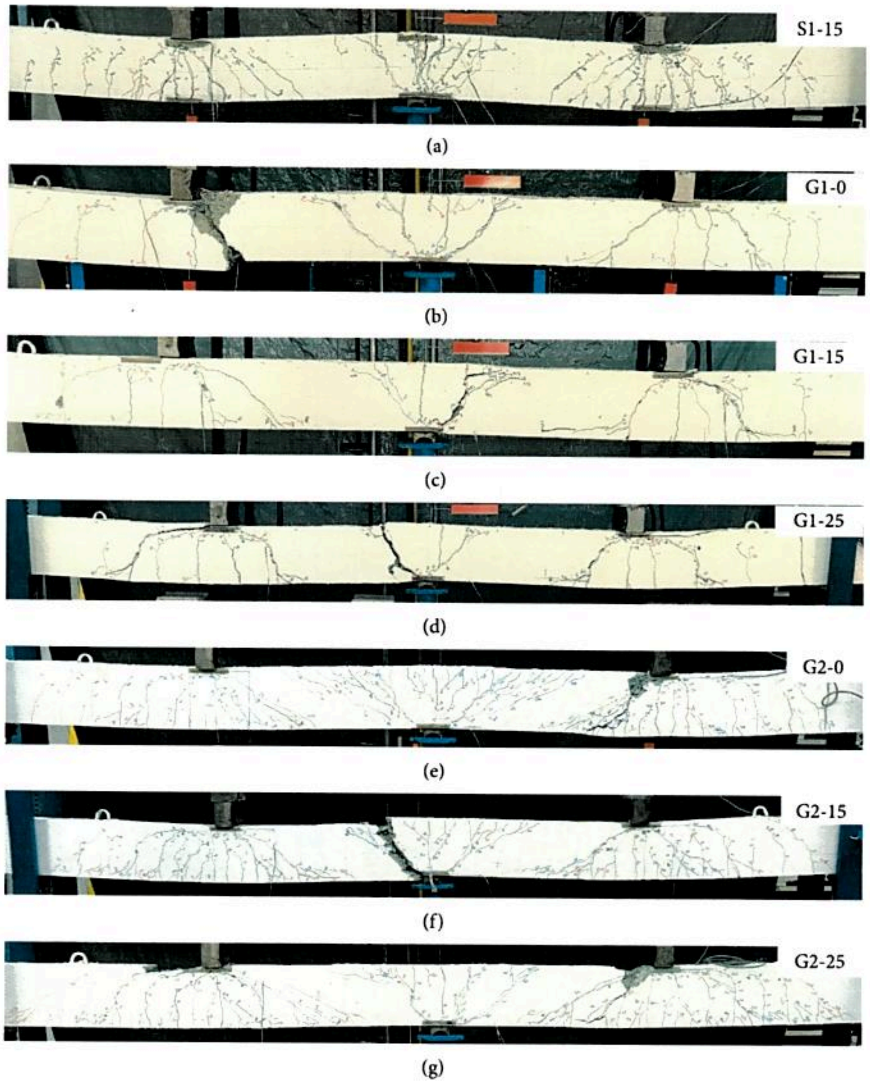


Figure 5: Failure modes of tested beams. (a) S1-15. (b) G1-0. (c) G1-15. (d) G1-25. (e) G2-0. (f) G2-15. (g) G2-25.

Table 3: First crack loads and failure loads.

| Beam | Load at first crack $P_{cr}$ (kN) | | | Failure load $P_u$ (kN) | $P_{cr}/P_u$ | |
|---|---|---|---|---|---|---|
| | Left midspan | Right midspan | Middle support | | Midspan | Middle support |
| S1-15 | 32 | 32 | 25 | 134.3 | 0.238 | 0.186 |
| G1-0 | 15 | 13 | 13 | 115.6 | 0.112 | 0.112 |
| G1-15 | 11 | 13 | 13 | 115.2 | 0.095 | 0.113 |
| G1-25 | 13 | 13 | 13 | 119.6 | 0.109 | 0.109 |
| G2-0 | 20 | 20 | 17 | 125.2 | 0.160 | 0.136 |
| G2-15 | 17 | 17 | 17 | 124.9 | 0.136 | 0.136 |
| G2-25 | 17 | 17 | 15 | 137.8 | 0.123 | 0.109 |

a large number of shear cracks at the hogging moment region (Figure 5).

*4.3. Crack Width.* In Figures 6 and 7, developments of maximal flexural crack widths depending on loads are given for all beams, in the midspan and at the middle support, respectively. It can be seen that beams of Series 1 had larger maximum flexural crack widths than the beams of Series 2. This is not only the consequence of a bit lower modulus of elasticity of the GFRP reinforcement used in beams of Series 1 but also the consequence of poor bond strength of this

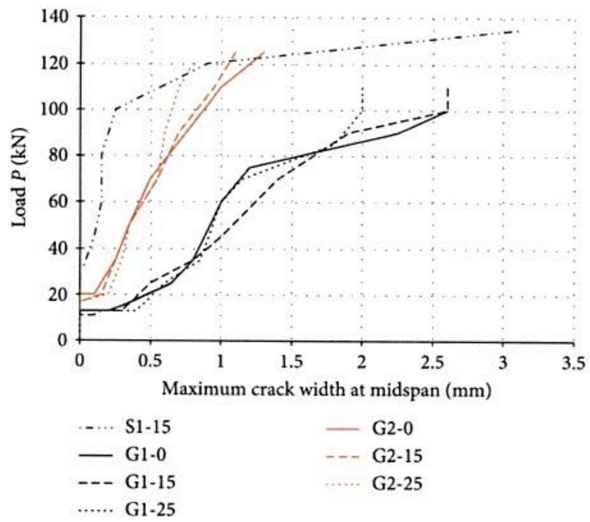


Figure 6: Load-maximum crack width relationship at the midspan for tested beams.

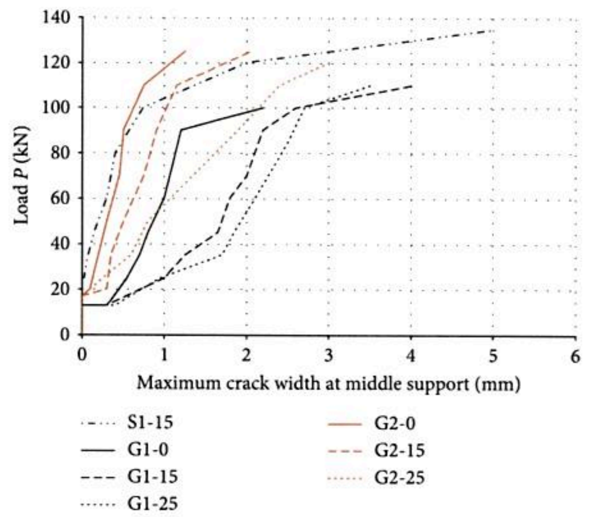


Figure 7: Load-maximum crack width relationship at the middle support for tested beams.

GFRP reinforcement with concrete. Also, beam S1-15 had the smallest crack width in relation to the beams with GFRP reinforcement because of the significantly larger modulus of elasticity of the steel reinforcement compared to the GFRP reinforcement. However, at higher load levels, when the steel reinforcement yielded, the maximum crack widths were greater in the beam S1-15 than in the beams with GFRP reinforcement.

Within the midspan of the beams of Series 1 with GFRP reinforcement, the development of the maximum crack width was much equalized, until loads that corresponded to 40% of the failure load, regardless of the fact that reinforcement in the beam G1-0 had less stiffness (EA = 13558 kN) in relation to reinforcement of the beams G1-15 (EA = 17415 kN) and G1-25 (EA = 18867 kN). At the failure, the smallest maximum crack width in the midspan was in the beam G1-25 with the highest amount of reinforcement in the midspan. At the support, the influence of stiffness of reinforcement was evident, meaning that less axial stiffness of reinforcement at the middle support generated larger crack widths in the beams. Therefore, for the most of different load levels, the largest crack width was in the beam G1-25 (EA = 11153 kN) and the smallest was in the beam G1-0 (EA = 20318 kN).

Within the beams of Series 2, the axial stiffness of the GFRP reinforcement was clearly expressed on the maximum cracks width, both in the midspan and at the middle support. For initial load levels, the maximum crack width in the midspan was almost uniform for all beams of Series 2. For higher load levels, the beam G2-25 exhibited the smallest maximum crack width with the largest reinforcement axial stiffness in the midspan (EA = 15930 kN), and the beam G2-0 exhibited the largest maximum crack width, with the smallest reinforcement axial stiffness in the midspan (EA = 10734 kN). At the middle support, beam G2-25 exhibited the largest maximum crack width with the smallest reinforcement axial stiffness at the support (EA = 8033 kN), compared to the beams G2-0 (EA = 12482 kN) and G2-15 (EA = 17654 kN).

4.4. *Deflection Response.* In Figure 8, the diagrams of load-average deflection for both spans are given for all beams. It is distinctive for all beams that they showed linear behavior of load-deflection before cracking. Right after the first crack appeared in the beams with GFRP reinforcement, a significant decrease of section stiffness occurred, which is the result of low modulus of elasticity of GFRP reinforcement, and it was manifested by a sudden change in the rate of load-deflection curves. The beams of Series 1 recorded a higher increase of deflection right before the appearance of cracks, compared to the beams of Series 2.

Within the beams of Series 1, for the same load level, as expected, deflections in beams with GFRP reinforcement were much higher than in the beam S1-15 with steel reinforcement. This is a result of larger crack widths in the beams with GFRP reinforcement, i.e., lower stiffness of the section. At higher load levels, the largest deflection exhibited the beam G1-0, with the smallest stiffness of the reinforcement in the midspan. The beams G1-15 and G1-25 practically had the same average values of deflection during the loading, which is expected due to a small difference in the stiffness of GFRP reinforcement in the beam midspan.

For the beams of Series 2, it could be seen that deflections were fairly uniform during loading, regardless of the different values of the axial stiffness of reinforcement in the critical sections. At higher load levels, the largest deflection was exhibited by beam G2-15 (EA = 14182 kN) and the smallest deflection was exhibited by beam G2-25 (EA = 15390 kN). Nonetheless, the beam G2-0 had significantly lower stiffness in the midspan (EA = 10734 kN), compared to beam G2-15, and also had a lower deflection. This is a consequence of a significant "opposite" moment redistribution from the midspan to the middle support

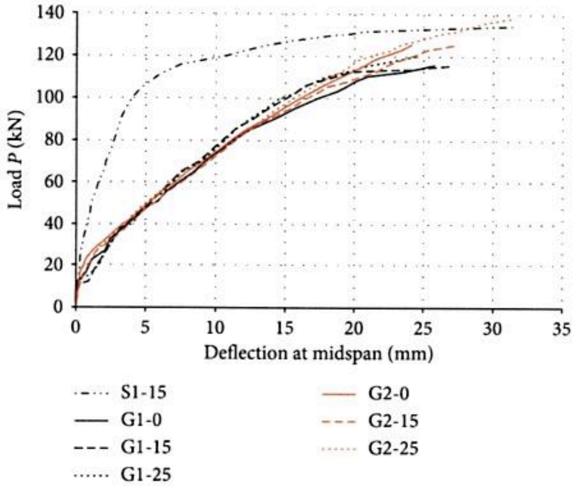


FIGURE 8: Load-deflection relationship at the midspan for tested beams.

(Section 4.5), i.e., significantly lower moment in the midspan of the beam G2-0. It can therefore be concluded that the axial stiffness of the reinforcement in the midspan is of crucial importance not only for the deflection of the beam but also for the axial stiffness ratio of the tensile reinforcement at the middle support and in the midspan.

*4.5. Moment Redistribution.* Measuring of reactions served for determining internal forces, i.e., bending moments, along the continuous beam, based on which the process of moment redistribution was analyzed. Moment redistribution was obtained by comparing actual bending moments and those obtained by elastic analysis. In Table 4, bending moments at failure and bending moments obtained by elastic analysis are given, similar to the percentage of achieved moment redistribution at failure for all beams.

Development of bending moments and moment redistribution at the middle support for all beams depending on the applied load are given in Figures 9 and 10, respectively. Upturns, i.e., changes in trends, are evident in diagrams of moments and moment redistribution at lower load levels, especially for the beams of Series 1 at appearance of first cracks in the midspan and at the middle support. This could be explained by a sudden change in stiffness of critical sections—from uncracked into cracked section (significant width and height of the cracks). After stabilization of the crack pattern, at higher levels of loads, upturns in diagrams of moment redistribution are less expressed.

Within the beam G1-0, that was designed based on elastic analysis, "opposite" moment redistribution is noticed with a highest value of 23% after appearance of first cracks. At failure, this value is significantly decreased, and it was 0.5%, which totally corresponded to designed values. "Opposite" redistribution caused the ratio between axial stiffness of tensile GFRP reinforcement between the middle support and midspan that was numbered 1.5. The beam G1-15 was designed to achieve redistribution of 15%, and it had the relation of axial stiffness of tensile reinforcement in the midspan and at the middle support of 1.38. At failure, moment redistribution at the middle support was significantly increased, and it was numbered 27%. Within the beam G1-25, designed to achieve moment redistribution of 25%, a failure moment redistribution of 18.5% was achieved, even though the relation between axial stiffness of reinforcement in critical sections was numbered 1.69. For the most levels during loading, the beam G1-25 had moment redistribution higher than 20% (Figure 10).

Within the beam S1-15 with steel reinforcement, moment redistribution was expected after yielding of reinforcement at the support. Nevertheless, in the diagram of Figure 9, it could be seen that after appearance of cracks, growth of moment at the support was higher than the growth of moment in the midspan. The reason for this fact is that, after yielding of reinforcement at the support, yielding of reinforcement in the midspan also appeared. Then, much higher strains at the middle support than those in the midspan probably lead to the strengthening of reinforcement at the support, which provided acceptance of additional moment. Because of that, moment redistribution was achieved by only 3%. In the diagram of Figure 10, it could be noticed that moment redistribution in the beam S1-15 was almost always lower than that in the beam G1-15. The reason for this behaviour of the beam S1-15, compared to the beam G1-15, was the relation between stiffness of critical sections. Moreover, a much lower modulus of elasticity of GFRP reinforcement compared to steel provides much wider cracks in beams with GFRP reinforcement, similar to a more dominant influence of reinforcement on the stiffness of section along the continuous beam. Therefore, the ratio between stiffness of critical sections mainly depends on axial stiffness of reinforcement, which provides the large ratio between stiffness of critical sections. In this way, easily could be seen the emphasis of elastic redistribution of internal forces in the beams with GFRP reinforcement in relation to the beams with steel reinforcement.

Within the beam G2-0, designed based on the internal forces obtained by the elastic analysis, at initial load levels,

TABLE 4: Moments at failure, moments based on elastic analysis, and achieved percentage of moment redistribution.

| Beam | Moments at failure (kNm) | | | Moments based on elastic analysis (kNm) | | Achieved percentage of moment redistribution at failure (%) |
|---|---|---|---|---|---|---|
| | Middle support | Left midspan | Right midspan | Middle support | Midspan | |
| S1-15 | 45.1 | 39.2 | 39.9 | 46.6 | 38.8 | 3.1 |
| G1-0 | 40.3 | 32.7 | 33.9 | 40.1 | 33.4 | −0.5 |
| G1-15 | 29.2 | 38.8 | 38.5 | 40.0 | 33.3 | 26.9 |
| G1-25 | 33.8 | 38.2 | 38.7 | 41.5 | 34.6 | 18.5 |
| G2-0 | 50.6 | 31.2 | 34.0 | 43.4 | 36.2 | −16.4 |
| G2-15 | 35.3 | 41.9 | 38.3 | 43.3 | 36.1 | 18.5 |
| G2-25 | 35.0 | 46.8 | 45.7 | 47.8 | 39.8 | 26.7 |

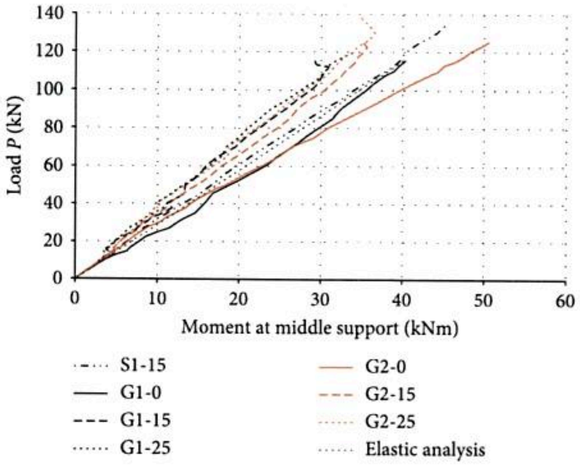


FIGURE 9: Load-moment relationship at the middle support for tested beams.

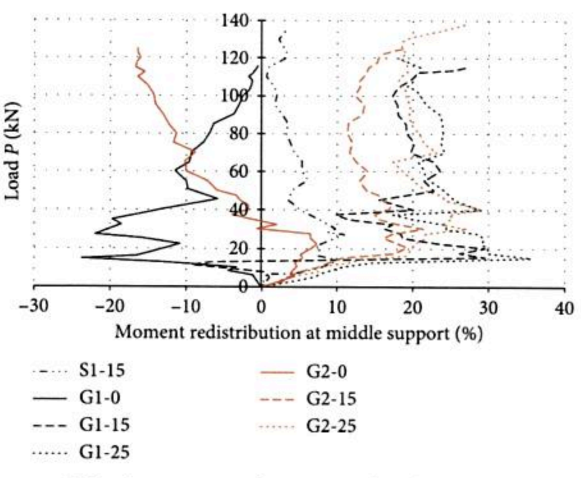


FIGURE 10: Load-percentage of moment redistribution relationship at the middle support for tested beams.

the increase of moment in the midspan was noticed, in relation to the moment at the middle support. After the appearance of the first cracks along the beam, a trend of moment redistribution was changed, i.e., a significant increase of the hogging moment in relation to the moment obtained by the elastic analysis. This trend of moment growth was kept to the failure of the beam. Therefore, "opposite" moment redistribution happened at failure of 16.4%, which was greatly influenced by the axial stiffness ratio of reinforcement at the middle support and in the midspan, which was numbered 1.65. Beams G2-15 and G2-25, designed to achieve moment redistribution, from the middle support into the midspan of 15% and 25%, achieved higher percentage of redistribution at failure of 18.5% and 26.7%, with ratio of axial stiffness of GFRP reinforcement between critical sections of 1.14 and 1.98, respectively. During the complete process of loading, beams had positive moment redistribution, which was the result of "set up" of the beams by means of adopted reinforcement, i.e., axial stiffness of reinforcement, which in the cracked section had a great contribution in the stiffness of critical sections, as previously discussed. Within the beams G2-15 and G2-25, an increase in the moment redistribution at failure was observed, probably as a result of the development of full nonlinearity of the compressed concrete.

*4.6. The Strains in Reinforcement and Concrete.* In Figures 11 and 12, developments of the strains in tensile reinforcement and compressed concrete in the midspan and at the middle support against the applied load for tested beams are given, respectively. It is an evident characteristic upturn in values of the strains in tensile reinforcement after appearance of first cracks in every critical section, which is especially noticed in beams of Series 1.

For the beams of Series 1, at load levels after appearance of first cracks, the strains in reinforcement in the midspan and at the support were higher in beams with GFRP reinforcement than in beam S1-15 with steel reinforcement. However, at higher load levels, after yielding of steel reinforcement, strain in the beam S1-15 significantly increased and overcame values of strains in beams with GFRP reinforcement. Comparing strains in reinforcement of the beams with strains in GFRP reinforcement, it is evident that, in the midspan, strains were highest in the beam G1-0 as a result of lowest stiffness of this reinforcement, while at the support, strains were highest in the beam G1-25, especially at higher load levels.

Within the beams of Series 2, quite uniform development of strains in the tensile reinforcement in the midspan, regardless of the significant differences in amount, i.e., the axial stiffness of adopted reinforcement of beams, could be noticed. At higher load levels, before failure, strains in the

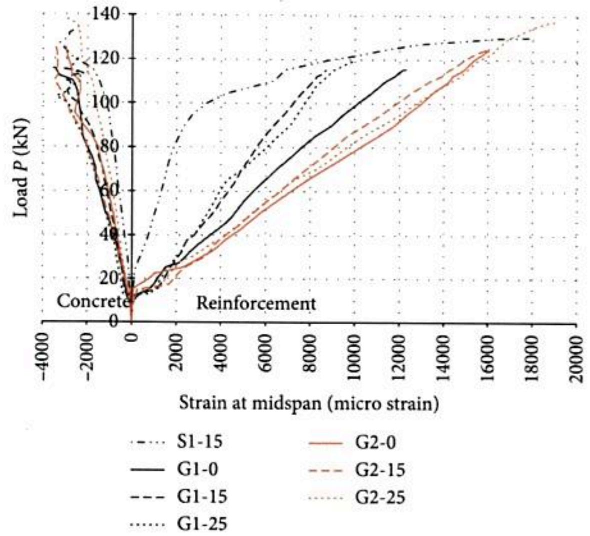


FIGURE 11: Load-strain relationship at the midspan for tested beams.

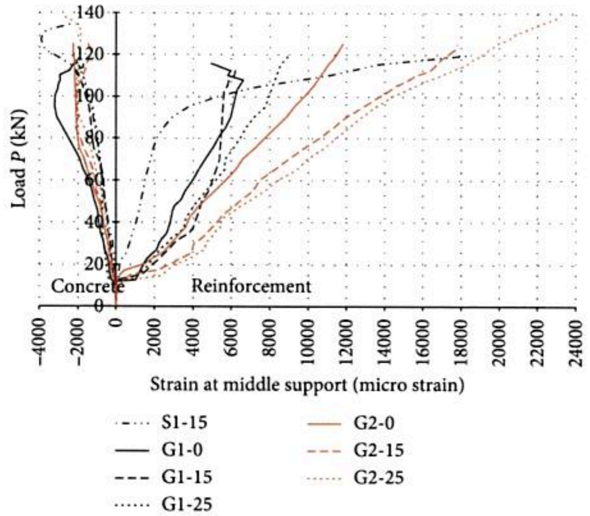


FIGURE 12: Load-strain relationship at the middle support for tested beams.

midspan of the beam G2-0 were about 10% higher than those in the beam G2-15 and in an average 5% higher compared to the beam G2-25. At the failure, strains in the beams G2-0 and G2-15 were practically identical, while the largest strains were in the beam G2-25, which reached the highest load-carrying capacity. As it has been already mentioned, this could be explained by achieved moment redistribution, which contributed that stiffness of reinforcement corresponded to higher bending moments. Because of that, lower strains did not correspond to higher amount of reinforcement, and vice versa. Comparing the strains in the tensile reinforcement at the middle support, the difference in strain values was significantly more pronounced than at the midspan. The higher strains were within the beam G2-25, and the smallest strains were within the beam G2-0, which fully corresponded to the adopted reinforcement at the middle support. The maximum tensile strains did not reach neither the ultimate values nor a single beam of Series 1 and Series 2, which responded to the fact that beams were designed to experience concrete compression failure. The largest measured strains at the middle support were in the beam G2-25 and amounted to 23‰, which was very close to the ultimate value of 23.3‰ shown in Table 2.

From Figures 11 and 12, it is noticed that in certain critical sections at the midspan, strain values in the compressed concrete of 3‰, which are defined as ultimate by ACI 440.1R-15 [23], have been reached and exceeded, both in the beams of Series 1 and in the beams of Series 2. For some beams, at the middle support, it is also noticed that slightly lower strain values were measured in compressed concrete because at higher load levels, higher than 70% of failure loads, strains start to decrease, which can be explained by appearance of diagonal cracks near the location of strain gauge.

## 5. Comparison of Experimental Results with Code-Predicted Results

*5.1. Load Capacity.* All beams were designed in accordance with the ACI 440.1R-15 [23], while CSA S806-12 [24] and EC2-04 [26] were used as control. In designing, data from the manufacturer for GFRP reinforcement were used, so as designed concrete compression strength. Regarding the difference in actual and designed characteristics of materials, different experimental values of failure load were obtained. Calculated failure load was obtained as load at which one of the critical sections reached flexural capacity, i.e., as the lower value of load capacity in the midspan and at the middle support. In determining the calculated failure load, designed moment redistribution from the middle support into the midspan was taken into consideration. In Table 5, experimental failure loads are given in comparison to calculated failure loads, in accordance with the current codes for elements with FRP reinforcement ACI 440.1R-15 [23], CSA S806-12 [24], and EC2-04 [26] for all experimental models.

From Table 5, it is clear that ACI 440.1R-15 [23] provides a very good prediction of failure load for continuous beams with GFRP reinforcement. CSA S806-12 [24] and EC2-04 [26] predict higher values of failure loads than those obtained by experimental tests. This happens because the ACI 440.1R-15 [23] suggests an ultimate strain of 3.0‰, and the CSA S806-12 [24] and EC2-04 [26] suggest an ultimate strain of 3.5‰. Measured values of strains in concrete, near failure, are closer to values of 3‰, which is the reason that obtained experimental failure loads are in agreement with calculated values in accordance with ACI 440.1R-15 [23]. Every beam, in accordance with ACI 440.1R-15 [23], reached calculated load capacity, where for beams of Series 1 relation $P_{exp}/P_{cal}$ is 1.0–1.03 and for beams of Series 2 it is 1.07–1.24.

Although the beams of Series 1 and Series 2 with GFRP reinforcement were designed to achieve similar failure loads, some higher values of failure loads were obtained for beams

Table 5: Experimental and calculated failure loads for tested beams.

| Beam | Failure load-load-carrying capacity (kN) | | | | $P_{exp}/P_{cal}$ | | |
|---|---|---|---|---|---|---|---|
| | Experiment | ACI | CSA | EC2 | Exp./ACI | Exp./CSA | Exp./EC2 |
| S1-15 | 134.3 | 90.1 | 88.8 | 90.1 | 1.49 | 1.51 | 1.49 |
| G1-0 | 115.6 | 115.4 | 126.3 | 141.3 | 1.00 | 0.92 | 0.82 |
| G1-15 | 115.2 | 111.8 | 123.0 | 137.0 | 1.03 | 0.94 | 0.84 |
| G1-25 | 119.6 | 117.4 | 128.6 | 143.8 | 1.02 | 0.93 | 0.83 |
| G2-0 | 125.2 | 113.6 | 128.0 | 145.2 | 1.10 | 0.98 | 0.86 |
| G2-15 | 124.9 | 117.2 | 131.7 | 149.8 | 1.07 | 0.95 | 0.83 |
| G2-25 | 137.8 | 110.8 | 125.2 | 141.7 | 1.24 | 1.10 | 0.97 |

of Series 2. The reason for this phenomenon could be the sliding of GFRP reinforcement and surrounding concrete in the beams of Series 1. Reducing the amount of GFRP reinforcement at the middle support and increase in the midspan of continuous beams, as a result of designed moment redistribution, did not have influence on decrease of load capacity of continuous beams. Moreover, the beams G1-25 and G2-25, with the designed moment redistribution of 25%, achieved higher load capacity, compared to the beams with a designed moment redistribution of 0% and 15%, for 5% and 10%, respectively.

*5.2. Load-Deflection Response.* In the background of this paper, it is stated that, until now, as a result of a number of research studies on investigation of behavior of simple beams with FRP reinforcement, a number of expressions for determining deflection-load response is suggested. For calculation of deflection of continuous beams, loaded by concentrated forces at the middle of the span, the following equation obtained by elastic analysis is used:

$$\Delta = \frac{7}{768} \cdot \frac{PL^3}{E_c I_e}, \quad (1)$$

where stiffness $E_c I_e$ is used and $I_e$ represents the effective moment of inertia of the considered section. Effective moment of inertia $I_e$ is calculated at both critical sections as follows:

$$I_e = 0.85 I_{em} + 0.15 I_{ec}, \quad (2)$$

where $I_{em}$ and $I_{ec}$ are the effective moment of inertia at the midspan and at the middle support, respectively.

ACI 440.1R-15 [23] suggests an equation for determining effective moment of inertia based on research studies made by Bischoff and Gross [9] with a remark that it could also be used, with a great degree of reliability, for elements with steel reinforcement and for elements with FRP reinforcement, without empirical parameters:

$$I_e = \frac{I_{cr}}{1 - \gamma \cdot (M_{cr}/M_a)^2 \cdot (1 - I_{cr}/I_g)}, \quad (3)$$

where $\gamma$ is the integration factor that influences stiffness variation along elements, and for the beams loaded by concentrated forces, it is calculated from the following expression:

$$\gamma = 3 - 2 \cdot \left(\frac{M_{cr}}{M_a}\right). \quad (4)$$

Calculation of deflection, in accordance with CSA S806-12 [24], is based on relation to the moment-curvature along the span. For calculation of deflection of continuous beams on two spans loaded by concentrated forces in the middle of the spans, the following expression can be used:

$$\Delta_{max} = \frac{PL^3}{48 E_c I_{cr}} \left(\frac{5}{16} - \frac{15}{8}\left(1 - \frac{I_{cr}}{I_g}\right) \cdot \left(\frac{L_g}{L}\right)^3\right). \quad (5)$$

Habeeb and Ashour [15] investigated behavior of continuous beams with GFRP reinforcement, and they suggested modification of expression for calculation of deflection of continuous beams, i.e., effective moment of inertia that is given in ACI 440.1R-06 [28]:

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \cdot \beta_d \cdot I_g + \left(1 - \left(\frac{M_{cr}}{M_a}\right)^3\right) \cdot I_{cr} \cdot \gamma_G \le I_g, \quad (6)$$

where the factor $\beta_d$ reduces tension stiffening for elements with FRP reinforcement, and it is given as follows:

$$\beta_d = \frac{1}{5} \frac{\rho_f}{\rho_{fb}} \le 1.0, \quad (7)$$

and where $\gamma_G = 0.6$ is a reduction factor, and it is introduced into calculation for the condition after the appearance of cracks because it is concluded that modified Branson's equation underestimates deflections for higher load levels.

Kara and Ashour [19] concluded, based on previous research studies on continuous beams with FRP reinforcement [1, 15, 16], that current codes underestimate deflections of continuous beams due to appearance of wide cracks above the middle support, that influence a significant decrease of effective stiffness of the section. Modified stiffness of the section in the midspan of continuous beams through effective moment of inertia is suggested:

$$I_e = I_{cr} \cdot \frac{\alpha}{1 - 0.5 \cdot (1 - \alpha) \cdot (M_{cr}/M_{ser})}, \quad (8)$$

where $\alpha$ is the reduction factor for beams with GFRP and AFRP reinforcement given by the following expression:

$$\alpha = 0.65 \cdot \left(0.7 + 0.36 \cdot \frac{E_f}{E_s} \cdot \frac{\rho_f}{\rho_{fb}}\right) \le 0.65. \quad (9)$$

Ju et al. [11] proposed a semiempirical model for determining the effective moment of inertia, which is based on the modification of the Branson's equation, following the approach of Toutanji and Saffi [7]. A nonlinear parameter $K$ is introduced which reduces the effective moment of inertia at higher load levels. The equations are proposed as follows:

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^m \cdot I_g + \left(1 - \left(\frac{M_{cr}}{M_a}\right)^m - K\right) \cdot I_{cr} \le I_g,$$
$$m = 6 - 13 \cdot \rho_f \frac{E_f}{E_s}, \tag{10}$$
$$K = \left(\frac{1}{11} \cdot \left(\frac{M_{cr}}{M_a}\right)\right)^4.$$

Load-deflection diagrams obtained by calculation according to expression (1), using effective moment of inertia according to ACI 440.1R-15 [23], CSA S806-12 [24], Ju et al. [11], Habeeb and Ashour [15], and Kara and Ashour [19], are compared to experimental results in Figure 13 for all beams. Since the cracking moment $M_{cr}$ is key to the accuracy of the deflection calculation, the experimental values were used for all models in order to eliminate its influence on the characteristics of the curve. In accordance with ACI 440.1R-15 [23], CSA S806-12 [24], and Ju et al. [11], for the beams of Series 1, already at initial load levels, deflections obtained by the experiment are higher than calculated deflections, and for the beams of Series 2, there is a matching for experimental and calculated diagrams only for the loads that correspond to 35–40% of the failure load. For higher load levels, the values obtained by the experiment are higher than calculated. The suggested model for deflection calculation according to Habeeb and Ashour [15] shows better agreement with experimental results. At loads near failure, exceptions occur, primarily due to additional fall of the deflection-load curve obtained by the experiment, when it comes to full development of nonlinearity of concrete. Kara and Ashour's model for deflection calculation [19] for continuous beams with GFRP reinforcement overestimates deflections especially for lower load levels for all beams.

In order to overcome stated shortcomings of the previous models, a modified model for calculation of deflection is proposed. The model is based on Branson's equation used in ACI 440.1R-06 [28] introducing a coefficient with a value of 0.7, which reduces the effective moment of inertia after the appearance of cracks, in analogy with Habeeb and Ashour's model [15], and a nonlinear parameter $K$, proposed by Ju et al. [11], which additionally reduces the effective moment of inertia at higher load levels:

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \cdot \beta_d \cdot I_g + \left(1 - \left(\frac{M_{cr}}{M_a}\right)^3 - K\right) \cdot I_{cr} \cdot 0.7 \le I_g. \tag{11}$$

The very good match of the proposed model and experimental results was shown, both at lower and at higher load levels. The exception occurs in the beams G1-15 and G1-25 at loads immediately after cracking where a large increase in deflection happened, which indicates poor bond strength between GFRP bars and concrete in these beams. In particular, it is noted that the proposed model, using the coefficient $K$ defined by Ju et al. [11], describes good development of deflection for higher load levels close to failure, when it comes to full development of nonlinearity of concrete and an additional drop in the slope of the deflection-load curve. Further experimental testing would be required to verify the proposed model.

## 6. Conclusions

The subject of the experimental research shown in this paper is consideration of six continuous beams reinforced with GFRP reinforcement loaded by concentrated forces in the middle of the span, until failure, for different arrangements of reinforcement along the beam. Results of experimental research studies are compared to provisions of current regulations and codes by means of load capacity and load-deflection response. Based on these research results, following conclusions could be made:

(i) Continuous beams with GFRP reinforcement have the ability of moment redistribution in relation to moments obtained by linear elastic analysis, after appearance of first cracks in concrete. Values of moment redistribution dominantly depend on stiffness of critical sections at the support and in the midspan, which, primarily, due to wide and deep cracks, come to relation of axial stiffness of GFRP reinforcement in critical sections. Elastic redistribution of internal forces is based on this.

(ii) Continuous beams with GFRP reinforcement show significant warnings before failure, in terms of large deflections and wide and deep cracks. It is specially defined additional curving of the deflection diagram at loads close to failure, as a result of the development of full nonlinearity of the compressed concrete.

(iii) Reducing the amount of GFRP reinforcement at the middle support and increase of amount of GFRP reinforcement in the midspan of continuous beams, as a result of designed moment redistribution, in relation to the moments obtained by elastic analysis, do not have negative influence on load capacity of continuous beams and mainly influence the decrease of deflection. By increasing the designed moment redistribution to 25%, the load-carrying capacity increases by 5% to 10% in the beams with GFRP reinforcement.

(iv) Wide and deep cracks that are formed in critical sections of continuous beams with wrapped GFRP bars with the unsaturated polyester matrix, in a smaller number compared to beams reinforced by steel reinforcement or GFRP bars with rebars and epoxy matrix, point out at poor bond strength

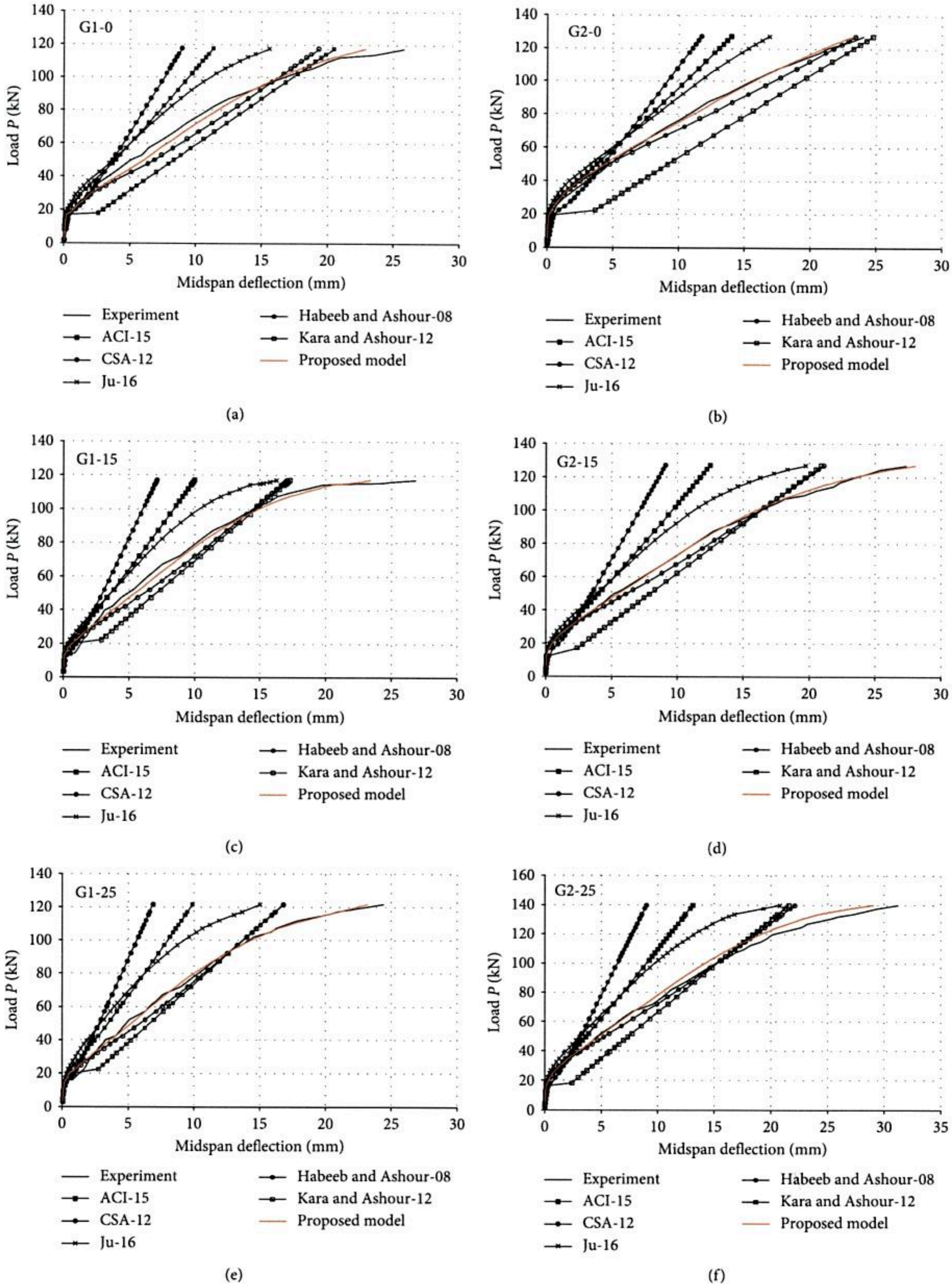


FIGURE 13: Experimental and predicted deflection for tested beams. (a) G1-0. (b) G2-0. (c) G1-15. (d) G2-15. (e) G1-25. (f) G2-25.

between GFRP reinforcement and surrounding concrete. On the contrary, beams with GFRP reinforcement with rebars and epoxy matrix, based on development of the cracks of the beams, indicate very good bond strength between GFRP reinforcement and concrete.

(v) ACI 440.1R-15 [23] reasonably predicts failure load for continuous beams with GFRP reinforcement, for overreinforced sections. CSA S806-12 [24] and EC2-04 [26] predict quite higher values of failure loads than those obtained by the experiment.

(vi) Current codes ACI 440.1R-15 [23] and CSA S806-12 [24] underestimate deflection of continuous beams with GFRP reinforcement for higher load levels. Habeeb and Ashour's suggested model [15] shows better agreement to experimental results. The proposed model for calculation of deflection of continuous beams with GFRP reinforcement shows very good prediction to experimental results, during the entire loading process.

## Nomenclature

$P$: Applied load at the midpoint of each span (kN)
$P_u$: Load at failure (kN)
$P_{cr}$: Load at the first crack (kN)
$P_{exp}$: Experimental failure load (kN)
$P_{cal}$: Calculated failure load (kN)
$L$: Beam span (mm)
$L_g$: Uncracked length at half of the beam (mm)
$I_g$: Moment of inertia of the gross section ($mm^4$)
$I_{cr}$: Cracked moment of inertia ($mm^4$)
$I_e$: Effective moment of inertia ($mm^4$)
$M_{cr}$: Cracking moment (kNm)
$M_a$: Applied moment (kNm)
$A_f$: Cross-sectional area of tensile GFRP reinforcement ($mm^2$)
$E_f$: Modulus of elasticity of GFRP reinforcement (MPa)
$E_s$: Modulus of elasticity of steel reinforcement (MPa)
$E_c$: Modulus of elasticity of concrete (MPa)
$f_f$: Tensile strength of GFRP reinforcement (MPa)
$f_y$: Yield strength of steel reinforcement (MPa)
$f_c$: Concrete compressive strength of cylinder (MPa)
$\varepsilon_{fu}$: Ultimate strain of GFRP reinforcement
$\varepsilon_y$: Yield strain of steel reinforcement
$\varepsilon_{cu}$: Ultimate strain of concrete
$\rho_f$: GFRP reinforcement ratio
$\rho_{fb}$: Balanced GFRP reinforcement ratio
$\gamma_G$: Proposed reduction factor used in the calculation of the effective moment of inertia for the continuous beam with GFRP reinforcement
$\alpha$: Proposed reduction factor used in the calculation of the effective moment of inertia for the continuous beam with FRP reinforcement
$\beta_d$: Reduction factor used in the calculation of the effective moment of inertia
$\Delta$: Deflection at the midspan of the beam (mm).

## Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

## Conflicts of Interest

The authors declare that there are no conflicts of interest regarding the publication of this paper.

## Acknowledgments

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*Nikola Baša*[1], *Mladen Ulićević*[2], *Radomir Zejak*[3]

S-20

# PRERASPODJELA UTICAJA U KONTINUALNIM GREDAMA ARMIRANIM FRP ARMATUROM

***Rezime:***

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*

[1] Mr. saradnik u nastavi, Građevinski fakultet, Podgorica, nik.ma@t-com.me
[2] Prof. dr, redovni profesor, Građevinski fakultet, Podgorica, mladenu@t-com.me
[3] Prof. dr, redovni profesor, Građevinski fakultet, Podgorica, rzejak@t-com.me

## 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.

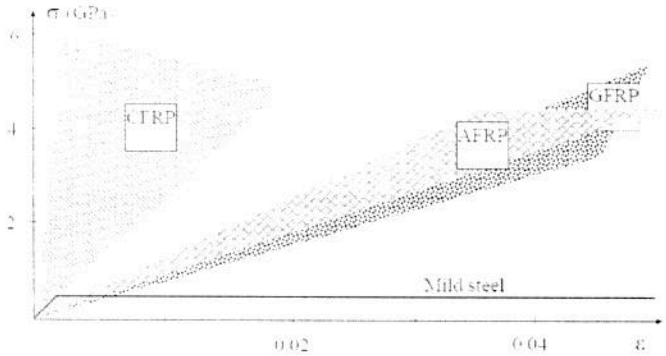
Preraspodjela momenata je itekako korisna za svakodnevnu inženjersku praksu jer omogućava različite aranžmane armature u AB elementima konstrukcije. Koristi se kada je iz zona u kojima se očekuje veća količina armature (veze greda i stubova), poželjno izmjestiti momenat u zone u kojima se može smjestiti 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 uglavno 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 širi 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 krti 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.

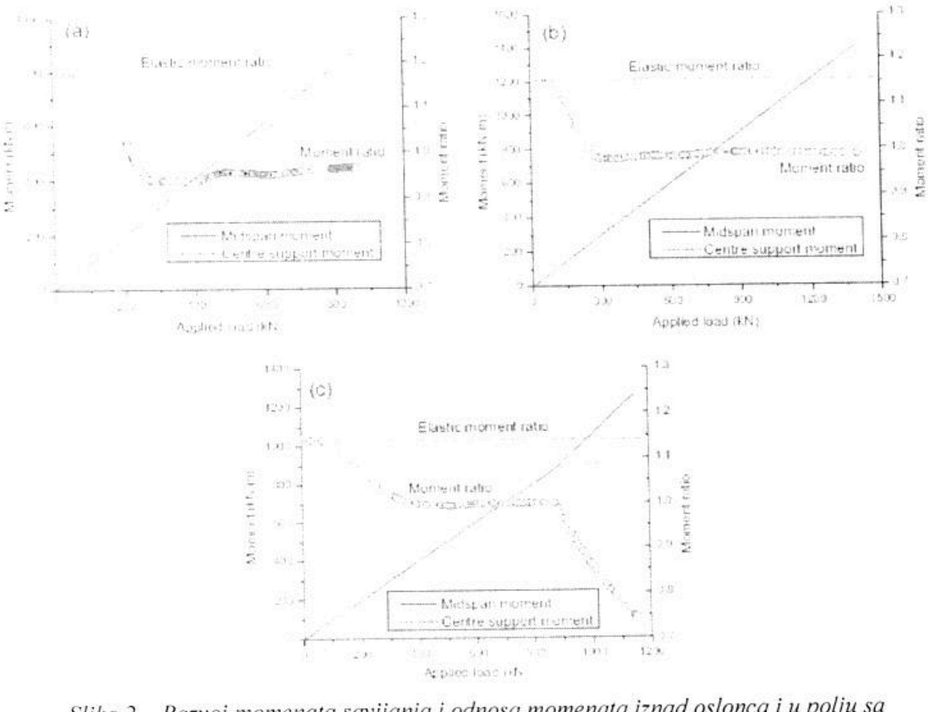
Simultani (balans) lom se u praksi veoma teško ostvaruje, jer ustvari predstavlja granicu između loma po armaturi i loma po betonu. Uslov balans loma je dostizanje granične dilatacije u pritisnutom betonu zajedno sa kidanjem FRP armature na zatezanje. Ukoliko je procenat armiranja AB presjeka FRP armaturom manji od procenta armiranja koji odgovara balans lomu, dolazi do loma po FRP armaturi. Lom koji nastaje usljed kidanja FRP armature je krt 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 ograničena 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 GREDAMA ARMIRANIM FRP ARMATUROM

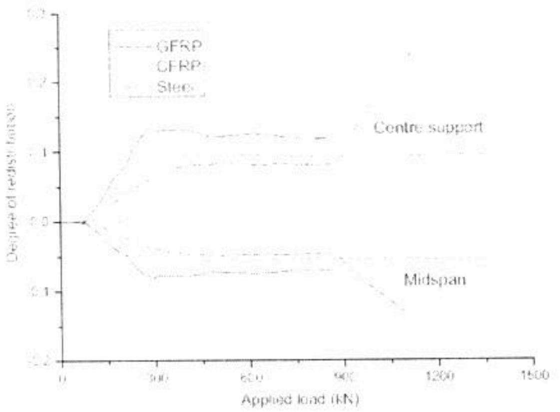
U ovom dijelu rada je prikazana osnovna razlika u ponašanju kontinualnih AB greda sa FRP i čeličnom armaturom u uslovima preraspodjele momenata između kritičnih presjeka.

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 prsline 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 prsline. 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, odrediće 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. Eksperimetalno 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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# THE RESPONSE ANALYSIS OF CONTINUOUS BEAMS WITH FRP REINFORCEMENT

Nikola Basa[1], Mladen Ulicevic[1], Radomir Zejak[1]

[1] University of Montenegro, Faculty of Civil Engineering, Cetinjski put bb, 81000 Podgorica, Montenegro, email: nik.ma@t-com.me, mladenu@t-com.me, rzejak@t-com.me

**SUMMARY:** Continuous beams are often used in reinforced concrete structures which are exposed to aggressive effects of the environment, such as marine structures, bridges, overpasses, garages, reservoirs, culverts, retaining walls, foundations and others. In aggresive environments there is a reduction of alkalinity of concrete, which often results in corrosion of the steel reinforcement, and causes further damage to the concrete, which endangers the functionality and serviceability of the RC structures. Having in mind the aforementioned, in recent years FRP composites find its wider application in building construction around the world, as internal reinforcement in RC elements. Although there is currently little reliable information that describes the behaviour of continuous beams reinforced with FRP reinforcement, the necessity of research work is emphasized, especially experimental, in regard to the small database that is currently available. Since FRP reinforcement shows a linear elastic behaviour up to failure, there is a question of the ability of material, that in conjunction with concrete, redistribute internal forces in statically indeterminate structures. In this paper, the main issue associated with the redistribution of the internal forces of continuous beams with a description of the main characteristics of FRP materials and the reinforcement is stressed, and the basic characteristics of the behaviour of RC elements with the FRP reinforcement are listed. This paper presents a short review of experimental research planned on the 12 RC continuous beams with two equal span length of 1.9 m, the cross-sectional dimensions 15/25 cm, with longitudinal and transverse GFRP reinforcement. In order to determine the possibilities of the redistribution of internal forces, and the behavior of RC elements in terms of the redistribution of the internal forces, the parameters of which it depends are varied: longitudinal reinforcement ratio at the midspan and at the middle support, the percentage of the longitudinal tensioned reinforcement and concrete compressive strength. It is expected that the results of the experiment show that is, the redistribution of the internal forces at statically indeterminate structures with FRP reinforcement possible, regardless of the linear elastic behaviour of FRP reinforcement up to failure, and that has a positive effect on the load-carrying capacity and fulfill the requirements of serviceability.

# ODZIV KONTINUIRANIH GREDA ARMIRANIH POLIMERNIM VLAKNIMA

**SAŽETAK:** Kontinuirane grede često se upotrebljavaju u armiranobetonskim konstrukcijama izloženim agresivnim djelovanjima okoliša kao što su konstrukcije na moru, mostovi, nadvožnjaci, garaže, spremnici, odvodni kanali, potporni zidovi, temelji i drugi. U agresivnom okolišu dolazi do smanjenja alkalnosti betona što često rezultira korozijom čelične armature i uzrokuje daljnja oštećenja betona čime se ugrožavaju funkcionalnost i uporabljivost armiranobetonskih konstrukcija. Imajući u vidu navedeno, posljednjih godina kompoziti od polimera armiranih vlaknima kao unutarnja armatura armiranobetonskih elemenata imaju sve širu primjenu u gradnji u svijetu. Kako danas ima malo pouzdanih podataka, a baza dostupnih podataka koji opisuju ponašanje kontinuiranih greda armiranih polimerima armiranim vlaknima je mala, naglašena je nužnost eksperimentalnih istraživanja. Kako je ponašanje armature od polimera armiranih vlaknima linearno-elastično sve do sloma upitna je sposobnost tog materijala koji u spoju s betonom raspodjeljuje unutarnje sile u statički neodređenim konstrukcijama. U radu je stavljen naglasak na preraspodjelu unutarnjih sila kontinuiranih greda s opisom glavnih značajki polimera armiranih vlaknima i armature, a popisane su i temeljne značajke ponašanja armiranobetonskih elemenata armiranih polimerom armiranim vlaknima. Dan je kratak pregled eksperimentalnih ispitivanja planiranih za dvanaest kontinuiranih greda s dva jednaka raspona od 1,9 m, dimenzija poprečnog presjeka 15/25 cm i s uzdužnom i poprečnom armaturom od polimera armiranog staklenim vlaknima. Varirani su parametri o kojima ovise mogućnosti preraspodjele unutarnjih sila i ponašanje armiranobetonskih elemenata: omjer uzdužne armature u sredini raspona i na srednjem osloncu, postotak uzdužne vlačne armature i tlačne čvrstoće betona. Očekuje se da će rezultati eksperimenata pokazati da je moguća preraspodjela unutarnjih sila statički neodređenih konstrukcija armiranih polimerima armiranim vlaknima neovisno o linearno-elastičnom ponašanju polimera armiranog vlaknima do sloma i da ona ima pozitivan učinak na nosivost i ispunjenje zahtjeva uporabljivosti.

## 1. INTRODUCTION

Today, concrete reinforced with steel reinforcement is still predominantly used for the construction of the civil engineering structures, which is in aggressive environments exposed to moisture, temperature, chloride, that leads to corrosion of steel reinforcement. Corrosive process causes damages to the concrete and endangers the functionality and serviceability of reinforced concrete structures. In recent years, there are more and more researches of materials that could replace steel in reinforced concrete structures, particularly in aggressive environments. Thus, FRP composite materials (Fiber Reinforced Polymer) find their application in civil engineering structures as well as internal and external reinforcement in RC elements. Since the FRP materials are not subjected to corrosion, their use in the structure can represent significant savings in maintenance, recovery and strengthening of structures, in particular when they are in the course of its exploitation life exposed to various corrosive and destructive influences, which certainly leads to economic benefits. Today we have a significant number of structures such as bridges, retaining walls, marine structures and others, on which FRP reinforcement in structural elements is succsessfully applied. Due to different mechanical and deformation characteristics of FRP reinforcement, the behaviour of RC elements greatly vary in relation to the elements of the steel reinforcement. The behaviour of reinforced concrete structures with steel reinforcement under load is well established and parameters that relate to behavior are clearly defined. This is primarily due to the significant and extensive research conducted in recent decades on the elements with steel reinforcement. In case of structures with FRP reinforcement, which is relatively new material, despite a significant increase in research work, it is evident that a limited number of experimental results, which are certainly reflected in the lack of appropriate regulations and standards for engineering applications. The regulations and standards for the structure with the FRP reinforcement, which currently exist in the world, are largely based on the proposed models and equations that are used for structures with steel reinforcement, with the variations of the parameters and the coefficients on which depend the characteristics of the FRP reinforcements and their interaction with the concrete. All this leads to that, the civil engineers are neither familiar with the properties and characteristics of FRP reinforcement, nor with its application in reinforced concrete structures.

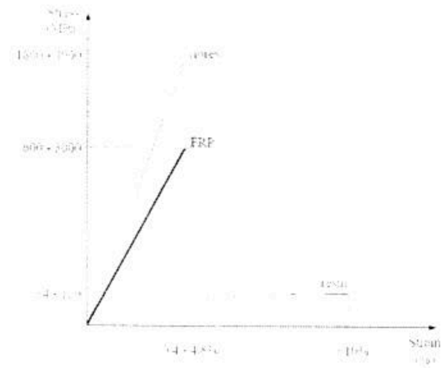
Research work that is currently being conducted, represents an attempt to contribute to further clarification of the behaviour of beams with FRP reinforcement. Most researchers have focused on examining the behaviour of simple supported beams with FRP reinforcement till now, whereas a very small number of tests were carried out on continuous beams. For this reason, the authors of the paper opted to study continuous beams with FRP reinforcement, taking into account the possibly significant breakthrough in the closer determination of answer of these structures to the action of the load up to failure. As a part of research an experimental program is currently being conducted, in order to provide a more reliable testing of the effect of the critical parameters on the behaviour of the continuous beams with FRP reinforcement.

As it is already mentioned, researches of FRP reinforcement have been mostly conducted on simple supported beams, so that are the provisions of certain regulations based on the conclusions reached at these elements. Although poor, database of experimental results on continuous beams with FRP reinforcement will be in any case enriched with the conducted experimental researches, that will greatly assist in the verification of accuracy and improving the results of existing experimental researches. Also, engineers and future researchers will have better insight into use and application of FRP reinforcement, a relatively new material, as internal reinforcement in elements of RC structures. In this way, contribution is given to the improvement of the guideliness and provisions of regulations and standards, applicable to designing of everyday engineering practice, especially in the area of statically indeterminate RC structures reinforced with FRP reinforcement.

## 2. FRP REINFORCEMENT

Development of FRP materials is intensive over the last 50 years, especially in the aero and electro industries. A major use in the building construction FRP materials have as systems for strengthening of existing RC structures with steel reinforcement, in the form of strips, and as well as an internal reinforcement in reinforced concrete structures. Such materials, as composites, may significantly vary, and their characteristics greatly depend on the constituent materials of which are composed.

FRP material is a composite material consisting a fine continuous fibers, bonded with the polymeric resins - a matrix. FRP composites are anisotropic, with mechanical characteristics and properties which are the best in the direction of the fibers, i.e. in the direction of the applied load. The polymer matrix is the material of low elastic modulus and low strength, which carries and distributes the load on the fibers which have a high modulus of elasticity and high strength. In this way, the result is a composite material of high tensile strength and a relatively high modulus of elasticity (Figure 1).




FRP reinforcement, as well as a steel reinforcement, may be produced in different profiles and different shapes (straight, curved, circular, spiral, etc.). Bond stresses between reinforcement and concrete depend on the workability of the surface of the bars which may be smooth, ribbed, spiral, and sand coated bars (see Figure 2). As an internal reinforcement in reinforced concrete structures, three types are mostly used: carbon (CFRP), glass (GFRP), and aramid (AFRP), and in recent years there is also basalt (BFRP) reinforcement. Mechanical and physical properties of the composite can vary depending on the type of the fiber that is used. So far the glass FRP reinforcement (GFRP) has the widest application as the cheapest, with a minimum tensile strength and the lowest modulus of elasticity in comparison to other FRP reinforcement. The most frequently used mechanical characteristics of the FRP reinforcement are shown in Table 1.

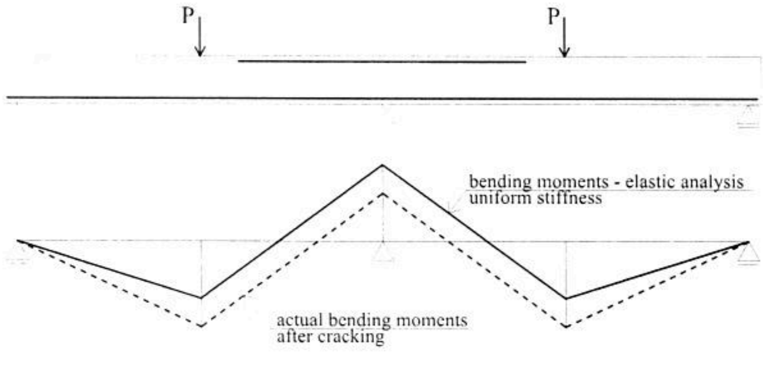

The main advantage of FRP reinforcement is a high tensile strength in comparison to steel reinforcement. They are not subjected to corrosion, that makes them very highly recommended in aggresive environments, but they also show the complete electrical and magnetic neutrality. They are lighter than steel, which can simplify their transport and speed building construction. In case of very reinforced sections, lower bulk density of FRP reinforcement can reduce the overall weight of the building, including static and dynamic internal forces that can negatively influence the structure itself.

| Type of reinforcement | Tensile strength (MPa) | Modulus of elasticity (GPa) | Ultimate strain (%) |
|---|---|---|---|
| GFRP | 450-1600 | 35-60 | 1.2-3.7 |
| CFRP | 600-3500 | 100-580 | 0.5-1.7 |
| AFRP | 1000-2500 | 40-125 | 1.9-4.4 |

FRP reinforcement has certain disadvantages in comparison to steel reinforcement. FRP composites show a linear elastic behaviour in tension up to failure, and they are brittle. They are characterized by a relatively low modulus of elasticity, especially for GFRP reinforcement, which implies greater strains of reinforcement in RC elements, in comparison to RC elements with steel reinforcement. Larger strains cause wider cracks and larger deformations (deflections). However, due to the absence of possible corrosion in the RC elements with FRP reinforcement, larger width of cracks is tolerated, while the deformations are limited as in RC steel elements. Compressive strength and shear strength are significantly lower than the tensile strength.

## 3. MOMENT REDISTRIBUTION IN CONTINUOUS BEAMS WITH FRP REINFORCEMENT

Redistribution of internal forces in RC statically indeterminate structures throughout the literature is defined as the difference between the actual cross section forces and those resulted from the linear elastic theory for the state without cracks. Usually, this phenomenon manifests itself in two phases. The first stage is caused by the difference of uniform bending stiffness along the element, which is assumed by elastic analysis, and the actual stiffness that occurs by variation of the reinforcement along the element and the occurrence of cracks in the concrete. Redistribution of internal forces made in this way is often in literature known as an elastic redistribution (Figure 3). The second stage is the result of plastic deformations in steel reinforcement, i.e. starts after reaching the yield point of the steel, and is manifested further by changing the value of bending stiffness. This redistribution is often called plastic redistribution. Previous researches bring to the conclusion that elastic redistribution can have a significant share in the total redistribution of internal forces along the element.

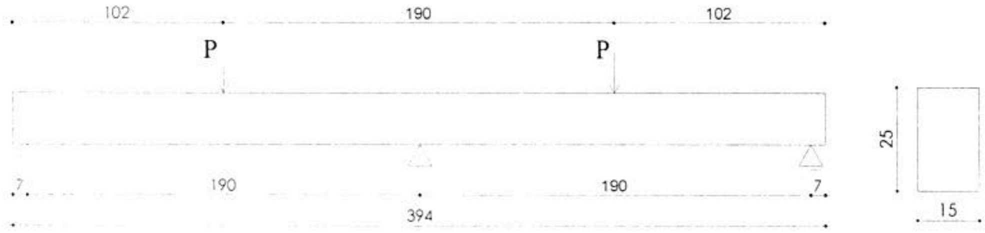



Since FRP reinforcement shows a linear elastic behaviour up to the failure, and shows a lack of material nonlinearity, there is a question of the ability of such material that in conjunction with concrete, redistribute the load in statically indeterminate structures. Taking into account the significant contribution of elastic redistribution in the total redistribution of moments in RC beams with steel reinforcement, it is expected that the continuous beam with FRP reinforcement demonstrates a certain ability to redistribution of moments, regardless of the lack of ductility at FRP reinforcement up to the failure. The redistribution of the internal forces is expected as a consequence of the difference of stiffness between the critical cross-sections, which is directly dependent on the relationship of the degree of development of cracks, and adopted reinforcement therein. In other words, one of the basic features of ductility is counted on, and that is the change of stiffness without the loss of capacity of section.

Previous research works on the continuous beams with FRP reinforcement [2], on a modest number of samples, show that the redistribution of the internal forces between the critical cross-sections is possible, especially if the reinforcement along the corresponding element is determined properly. RC continuous beam with FRP reinforcement showed significant warnings, regarding the large deformation and wide and deep cracks before failure. Increasing the bottom reinforcement at the midspan of continuous beam reinforced with reinforcement related to the cross-section at the middle support, had a positive effect on increasing the loading capacity of the beam, reducing deformation and delaying propagation of cracks in the areas of the beams, while the increase of top reinforcement over the middle support is not found as a significant contribution to increasing the load capacity and reducing deformation. This statement may be a good basis for the introduction to redistribution of moments (primarily from the middle support to the midspan of the continuous beam), to provide improved behaviour of continuous beams with FRP reinforcement for the serviceability limit state as well as for the ultimate limit state. The justification of this thinking lies in the fact, that in relation to the elastic analysis, a significant portion of bending moments from middle support displaces at the midspan, already for service load. A width of cracks that occur at the middle support of continuous beams should be controlled, and the same can be significantly increased due to the relocation of moments. However, the fact that, due to the absence of corrosion in the FRP reinforcement, with the current regulations that allow greater width of cracks in concrete beams with FRP reinforcement (even up to 0.7 mm) than it is the case with beams with steel reinforcement, encourages with a view to implementing the moment redistribution with a continuous beam with FRP reinforcement, and that current regulations are still not allowed. The ratio of bottom reinforcement at the midspan and top reinforcement at the middle support of the continuous beam with FRP reinforcement, has a major effect on the redistribution of the available moments. Percentage of longitudinal tensioned reinforcement represents also one of the important factors which directly affects the degree of redistribution moments. When it comes to over-reinforced sections with FRP reinforcement, due to inelastic deformation of the compressive concrete, there are significant deformations and wider cracks before failure, as a form of pseudo-ductile behaviour, and is expected to come to a significant redistribution of internal forces between the critical sections of the element. For this reason, most of the regulations for structures with FRP reinforcement, recommend that the sections should be over-reinforced, i.e. with the percentage of reinforcement higher than balance reinforcement ratio.

## 4. DESCRIPTION OF EXPERIMENTAL MODELS AND MEASURING TECHNICS

Experimental researches are being conducted currently at the Faculty of Civil Engineering University of Montenegro, within PhD theses comprising behaviour of continuous beams, primarily to bending, in the conditions of redistribution of internal forces. Experimental research comprises 12 continuous beams in total, on 2 spans of length of 1.9 m, of cross-section being 15/25 cm, with longitudinal and transverse GFRP reinforcement and one control beam with steel reinforcement. All the beams are to be examined up to failure, being loaded by concentrated forces at the middle of both spans. Dimensions and geometry of continuous beams and load disposition have been presented in figure 4. All the beams are dimensioned in line with American standard ACI 440.1R-15 which is being used for design of elements with FRP reinforcement, while the other regulations (CSA S806-02, CSA S806-12, CNR-DT-203) have been used as control.




Representative models should have enabled realisation of stated aims, therefore the following parameters have been combined: longitudinal reinforcement ratio at the midspan and at the middle support, the percentage of the longitudinal tensioned reinforcement and concrete compressive strength. All the details related to selected models with beam mark, designed failure load, designed moment redistribution from the middle support to the midspan of the beam, chosen ratio of longitudinal reinforcement ratio and balance reinforcement ratio and designed class of concrete compressive strength have been presented in table 2. Continuous beams are reinforced by GFRP reinforcement, that having tension strength of f=1100 MPa and modulus of elasticity of E=55000 MPa.

| Beam | Design failure load (kN) | Design moment redistribution (%) | Reinforcement ratio / balance reinforcement ratio | | Concrete strength (MPa) |
|---|---|---|---|---|---|
| | | | middle support | midspan | |
| G-A-0-N | 100 | 0 | 4.9 | 3.0 | 28 |
| G-A-15-N | 100 | 15 | 3.42 | 3.86 | 28 |
| G-A-30-N | 100 | 30 | 2.1 | 4.9 | 28 |
| G-B-0-N | 80 | 0 | 2.8 | 1.76 | 28 |
| G-B-15-N | 80 | 15 | 1.91 | 2.25 | 28 |
| G-B-30-N | 80 | 30 | 1.46 | 2.8 | 28 |
| G-C-0-N | 60 | 0 | 1.0 | 0.75 | 28 |
| G-C-15-N | 60 | 15 | 0.75 | 1.0 | 28 |
| G-A-15-H | 130 | 15 | 2.36 | 2.66 | 50 |
| G-A-30-H | 130 | 30 | 1.45 | 3.38 | 50 |
| G-B-15-H | 110 | 15 | 1.32 | 1.55 | 50 |
| G-B-30-H | 110 | 30 | 1.01 | 1.93 | 50 |

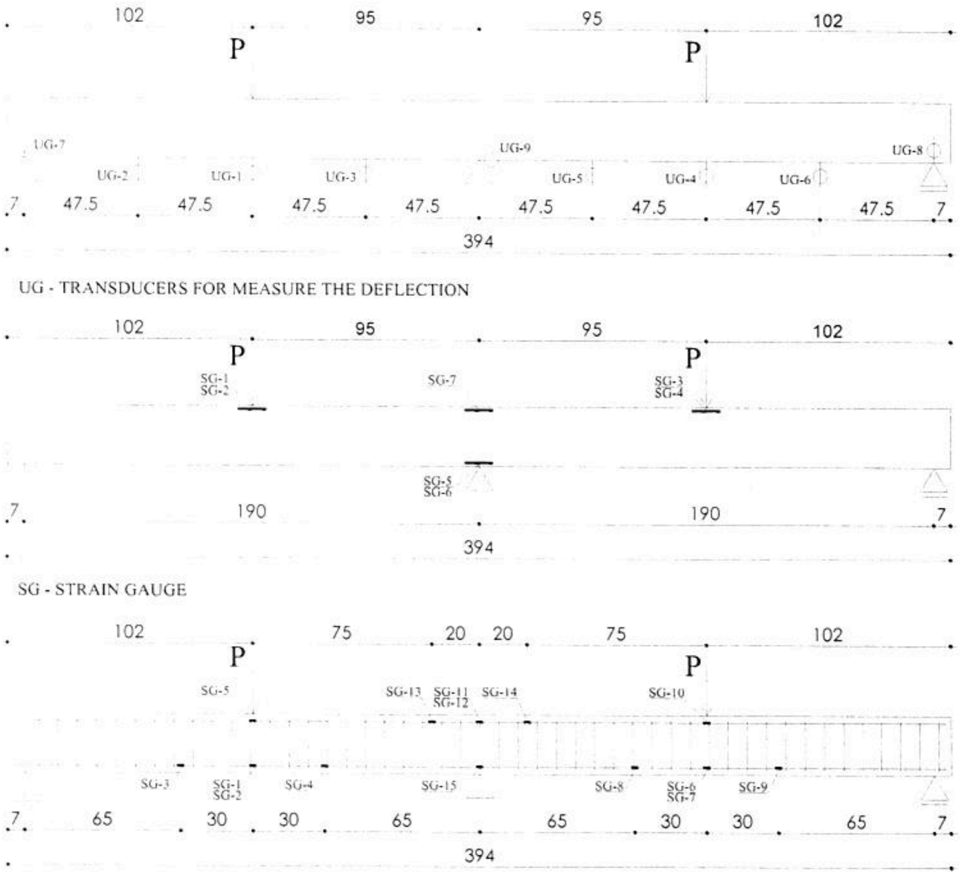
For the first series of beams, having A and N marks, identical failure load has been designed, hence one beam has been dimensioned to the internal forces similar to elastic analysis, while for 2 beams moment reduction above the middle support has been conducted, by 15% and 30%, and adequate moment increase at the midspan. Therefore, for the designed fixed failure load, models with 0%, 15% and 30% of designed moment redistribution from the middle support to the midspan have been obtained. Percentage of beam reinforcement at the middle support with mark 0 has been selected of cca 4.9 times higher than balance reinforcement ratio, which is in line with references of the regulations that beams with FRP reinforcement should be designed to have concrete compression failure. Even after conducted redistribution, all the cross-sections have been designed to have reinforcement ratio being higher than balance reinforcement ratio. Different profiles of FRP reinforcement in the cross-section have been selected, in order to define as precisely as possible designed moment redistribution and enable identical failure load for the beams of the same series. Mark N is adequate for the designed concrete compressive strength upon cylinder of 28 MPa. All the beams from the same series are to be constructed in the same day, in order to reduce influence of concrete strength.

For the other series of beams, having B and N marks, lower percentage of reinforcement has been selected, which above the middle support for the beam with mark 0 is adequate to the value of cca 2.5 times higher than the balance reinforcement ratio, which still results in concrete compression failure. Models with 0%, 15% and 30% of moment redistribution from the middle support to the midspan have been also adopted. Concrete compressive strength has been adopted as for the beams in the first series.

For the third series of beams, having C and N marks, reinforcement ratio have been adopted that beams have failure by reinforcing bars, and with designed moment redistribution of 0% and 15%. As failure by reinforcing bars with FRP reinforcement presents brittle failure, it was considered as unrealistic that continuous beam achieves redistribution of 30% over the middle support without reduction of failure force, hence such a beam was not even taken into consideration.

In order to examine the concrete strength onto beam behaviour, for the beams with which we have designed moment redistribution from the middle support to the midspan of 15% and 30%, with their marks being A and B, for which it was planned to have concrete compression failure, new models have been made with designed concrete strength upon cylinder of 50 MPa (beams with mark H). It is evident that the beams with adopted higher concrete compressive strength will enable achievement of significantly higher failure load, i.e. ultimate load-carrying capacity. Namely, in the case concrete compression failure, upon increase of concrete strength, it comes to increase in strains in GFRP reinforcement, which enables higher tension stresses in GFRP reinforcement, and higher load-carrying capacity of cross-section, and higher failure force accordingly.

For shear reinforcement, GFRP stirrups have been adopted, of the diameter of 8 mm and at the distance of 8 cm for all the beams, in order to enable flexural failure, i.e. to avoid shear failure. In order to fulfil the requirements, the beams have been designed in line with regulations (ACI 440.1R-15, CSA S806-02, CSA S806-12, CNR-DT-203) which show significant differences in amount of GFRP stirrups, depending on the regulation it is use. For the beams having mark N, assurance from shear failure has been enabled in line with all the regulations. For the beams having mark H, by increase in concrete strength and therefore increase in designed failure load, dimensioned onto bending, there is a possibility of shear failure. Having in mind unreliability of obtained results of dimensioning onto shear capacity upon different regulations, which significantly vary, it has been decided that these beams are not to be insured additionally to the shear capacity, but identical quantity of shear reinforcement has been adopted. Possibility that beams have shear failure has been left in this manner, and therefore the effect of redistribution of internal forces onto ultimate shear capacity to be examined.




Upon selection of representative models, recommendations and references of actual regulations have been taken into consideration that both cross-sections at the midspan and at the middle support are to be over-reinforced, i.e. to achieve concrete compression failure, as semi-ductile, therefore more desirable in comparison to failure by reinforcing bars. Two beams have been designed to have failure by reinforcing bars in order to examine behaviour of continuous beams in the conditions of redistribution of internal forces and upon lower reinforcement percentages. While, on one hand, upon designing of beams with FRP reinforcement the quality of the same should be taken into consideration, which concrete compression failure certainly enables, on the other hand, economic aspect should be taken into account, meaning the best use of characteristics of FRP reinforcement. Namely, for reinforcement percentages being much higher than balance reinforcement ratio, strains in FRP reinforcement are low hence forces in the reinforcement remain unused (linear elastic behaviour up to failure). Therefore, such a design of FRP elements enables quality option, but is quite expensive. Failure by reinforcing bars is considered as highly undesirable, but it enables full use of tension stresses in the reinforcement, hence such a design of FRP elements is much cheaper. Therefore, reinforcement percentages for the models have been carefully selected, in order to enable their proper and quality response, but also stresses in the reinforcement not to be with low values. Such a problem is not present with the beams with steel reinforcement due to low strain at which reinforcement is yielding (higher modulus of elasticity) hence stresses in the reinforcement are used to its maximum.

The load is applied as a monotonically increasing static load in increments of zero state up to failure. At each loading level the following variables are measured: the intensity of the load, strain in the longitudinal tensioned GFRP reinforcement, strain in the concrete, deflection along the beam, the width of the cracks, and the reactions of the end supports. Measure of the reactions at the end supports are used for determining the internal forces along the beam and monitor the process of moment redistribution in which purpose two load cells are used. Transducers for deflection measuring and strain gauges for measuring the strains in the GFRP reinforcement and concrete are shown in Figure 5. For each increment photographing beam conditions is performed.

## 5. AIMS AND RESEARCH SIGNIFICANCE AND EXPECTED RESULTS

Main aim of this research is reviewing in behaviour of continuous beams reinforced with FRP reinforcement, loading up to failure, in the conditions of moment redistribution between critical cross-sections. Closer determination of behaviour of continuous reinforced concrete beams with FRP reinforcement is to be defined with different states as it follows: ultimate load-carrying capacity upon bending, ultimate load-carrying capacity upon shear, mode failure, state of cracks, deflection state, strains in the FRP reinforcement and compressive concrete, possibility of moment redistribution in the critical sections. Therefore, research aims have been set accordingly:

Own experimental program with the aim of examination of effect of critical parameters onto behaviour of reinforced concrete beams with FRP reinforcement, and moment redistribution alongside the beam and significant contribution in the database of experimental results.

Analysis of effect of certain parameters onto behaviour of reinforced concrete beams with FRP reinforcement, with the accent being on the effect of redistribution of internal forces onto limit state of continuous beams with FRP reinforcement.

Comparison of results of experimental researches in terms of load-carrying capacity, deflection and crack states with provisions of actual regulations for continuous beams with FRP reinforcement, and verification of correctness and reliability of certain provisions.

Research significance is especially seen in reviewing of the facts being stated below. Namely, upon literature overview it has been concluded that in the very few experiments on the beams with longitudinal FRP reinforcement, FRP reinforcement has been used for the stirrups. In the researches conducted on the beams with FRP reinforcement, steel reinforcement has been mostly used for the stirrups. Through the literature, as it was stated, corrosion was used as the main reason for implementation of FRP reinforcement instead of steel reinforcement in the RC elements, since RC elements with steel reinforcement are exposed to the same. It is therefore logical to ask the question of the purpose of use of FRP reinforcement for longitudinal reinforcement, and then use a steel one for the stirrups. Therefore, in the mentioned case, RC elements would still have the issue of corrosion, especially in the aggressive environments and bearing in mind width of cracks, being significantly wider when longitudinal FRP reinforcement is used in the beams. The problem increases with the fact that efficiency of FRP and steel stirrups is completely different, especially when the beam reaches ultimate shear load-carrying capacity. However, even when examinations with excluded reaching of ultimate shear load-carrying capacity are being done, the question still remains if the use of FRP reinforcement for the stirrups instead of steel reinforcement, would give the same effect onto behaviour of the beams where ultimate flexural load-carrying capacity is being reached. Having aforementioned in mind, it was decided that even in the experimental researches, FRP reinforcement is to be used for the stirrups, besides longitudinal FRP reinforcement.

Planned research should have shown that even statically indeterminate structures with FRP reinforcement are able to redistribute of internal forces and therefore have contributed to more quality structure response onto the action of load. It is expected that increase in ultimate load-carrying capacity is achieved by designed redistribution of internal forces in the continuous beams with FRP reinforcement, and even more important, that use conditions are achieved, i.e. reduction in deflection and cracks, which is regularly significant and relevant with these structures.

# BIOGRAFIJA i BIBLIOGRAFIJA DOKTORANDA

*mr Nikole Baše, dipl.inž.građ.*

Nikola Baša je rođen 30. oktobra 1980. godine u Trsteniku, Srbija, gdje je završio osnovnu i srednju školu. Na Građevinski fakultet Univerziteta Crne Gore u Podgorici upisao se školske 1999./2000. godine. Apsolvirao je u roku sa prosječnom ocjenom 8.83. Diplomski rad pod nazivom *Projekat konstrukcije tržnog centra u Podgorici* odbranio je septembra 2005. godine sa ocjenom 10. U toku studija je bio više puta nagrađivan za postignuti uspjeh u studiranju. Dobitnik je nagrade "19. decembar" u Podgorici 2000. godine, kao najbolji student na Građevinskom fakultetu.

Magistarski rad pod nazivom *Analiza uticaja popuštanja veza na seizmički odgovor montažne armirano betonske ramovske konstrukcije* odbranio je dana 09.04.2009. godine sa ocjenom "A", pod mentorstvom prof. dr. Mladena Ulićevića.

Od oktobra 2005. godine angažovan je na Građevinskom fakultetu u Podgorici, kao honorarni saradnik u nastavi na više predmeta iz oblasti Betonskih konstrukcija. Radni odnos na Građevinskom fakultetu Univerziteta Crne Gore zasnovao je septembra 2008. godine u svojstvu saradnika u nastavi na predmetima Betonske konstrukcije inženjerskih objekata i Fundiranje. U proteklom periodu, od septembra 2011. godine, u svojstvu saradnika u nastavi, izvodio je nastavu na osnovnim studijama studijskog programa Građevinarstvo na predmetima: Betonske konstrukcije I, Građevinski materijali i Zidane konstrukcije, i studijskog programa Menadžment u građevinarstvu na predmetima: Betonske konstrukcije i Projektovanje i građenje betonskih konstrukcija. Na specijalističkim studijama studijskog programa Građevinarstvo izvodio je nastavu na predmetima: Prethodno napregnute konstrukcije, Betonske konstrukcije inženjerskih objekata i Projektovanje i građenje betonskih konstrukcija. Na Arhitektonskom fakultetu izvodi je nastavu na osnovnim studijama iz predmeta Zidane i betonske konstrukcije.

Na doktorskim studijama na Građevinskom fakultetu upisao se 2010/2011 godine, položio sve ispite sa prosječnom ocjenom A, odbranio polazna istraživanja 04.05.2016.

god. i odlukom Senata broj 03-1634/2 iz 23.06.2016, doktorske teza je prihvaćena kao podobna. U sklopu izrade doktorske disertacije pod nazivom *Efekti preraspodjele uticaja na granična stanja kontinualnih greda armiranih FRP armaturom*, sproveo je eksperiment u laboratoriji Građevinskog fakulteta u Podgorici, na gredama sa GFRP armaturom, novim materijalom koji se sve više koristi u svijetu kao zamjena za čeličnu armaturu.

U naučnom smislu bavi se problemima nelinearnog ponašanja AB konstrukcija, dok mu je stručni rad uglavnom vezan za projektovanje AB konstrukcija. Objavio je više naučnih i stručnih radova na međunarodnim i na domaćim naučnim konferencijama. Kao projektant saradnik, a kasnije kao odgovorni projektant, učestvovao je u izradi projektne dokumentacije, stručnom nadzoru i izvođenju velikog broja novih objekata, kao i sanaciji postojećih. Član je ekspertskog tima za izradu Eurokodova, novih propisa koji se već primjenjuju u Crnoj Gori.

Čita, piše i govori engleski jezik.

Mr Nikola Baša je učestvovao na više međunarodnih i domaćih naučnih skupova i tokom naučno-istraživačkog rada kao autor i koautor objavio više naučnih radova. Pojedini radovi se navode:

- radovima objavljeni na međunarodnim i domaćim skupovima i u časopisima

1. Ulićević M., **Baša N.**: *Prikaz konstrukcije otvorene nadzemne garaže tržnog centra «DELTA CITY» u Podgorici*, Internacionalni naučno-stručni skup Građevinarstvo - nauka i praksa, GNP 2008, Žabljak, 2008.
2. **Baša N.**, Ulićević M.: *Analiza uticaja popuštanja veza na seizmički odgovor montažne AB ramovske konstrukcije*, Internacionalni naučno-stručni skup Građevinarstvo - nauka i praksa, GNP 2008, Žabljak, 2010.
3. **Baša N.** : *Uporedna analiza metoda proračuna vremenskih deformacija po Evrokodu 2 i odabranih propisa*, Peti internacionalni naučno-stručni skup Građevinarstvo - nauka i praksa, GNP 2014, Žabljak, 2014.
4. **Baša N.**, Radovanović Ž., Sinđić Grebović R., Serdar N.: *Evropski standard EN 1991-1-1 - Opšta dejstva - Zapreminske težine, sopstvena težina, korisna opterećenja za zgrade*, časopis Inženjerske komore, Podgorica, jul 2014.

5. Radovanović Ž., **Baša N.**, Sinđić Grebović R., Serdar N.: *Evropski standard EN 1991-1-5 - Termička dejstva - Termička dejstva na zgrade*, časopis Inženjerske komore, Podgorica, jul 2014.

6. **Baša N.**, Ulićević M., Zejak R.: *Preraspodjela uticaja u kontinualnim gredama armiranim FRP armaturom*, Simpozijum društva građevinskih konstruktera Srbije, Zlatibor, septembar 2016.

7. **Baša N.**, Ulićević M., Zejak R.: *The response analysis of continuous beams with FRP reinforcement*, 1st International conference Construction materials for sustainable future, Zadar, Croatia, april 2017.

- radovima objavljeni u časopisima sa SCI/SCIE liste

1. **Baša N., Ulićević M., Zejak R.:** *Experimental Research of Continuous Concrete Beams with GFRP Reinforcement*, Advances in Civil Engineering, Article ID 6532723, 2018., 16 pages. https://doi.org/10.1155/2018/6532723

01-620
10. 07. 2003

Na osnovu člana 97. Zakona o Univerzitetu ("Sl.list RCG", br. 27/92 i 6/94) i člana 94. Statuta Univerziteta Crne Gore, Naučno-nastavno vijeće Univerziteta Crne Gore, na sjednici održanoj 09.07.2003.godine, donijelo je

# O D L U K U
# O IZBORU U ZVANJE

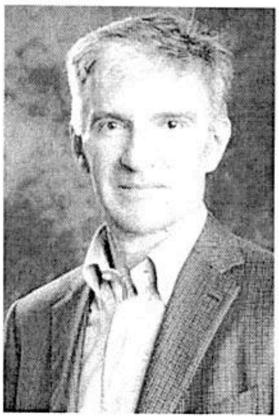
**Dr MLADEN ULIĆEVIĆ** bira se u zvanje **redovnog profesora** Univerziteta Crne Gore za predmet: Projektovanje i gradjenje betonskih konstrukcija i Betonske konstrukcije inženjerskih objekata **na Gradjevinskom fakultetu u Podgorici.**

**R E K T O R,**

**Prof.dr Predrag Obradović**

# BIOGRAFIJA

**Mladen Ulićević, Građevinski fakultet u Podgorici**
**Tel: 020 206 050, Mob: 069 013 317, e-mail: mladenu@t-com.me**



**Datum i mjesto rođenja:** 14.08.1957. Cetinje
**Osnovno obrazovanje:** 1964-1972. Osnovna škola, Cetinje
**Srednje obrazovanje:** 1972-1976. Gimnazija, Cetinje
**Fakultet-diploma:** 1976-1982. Građevinski fakultet, Beograd

**Magistarski rad:** Prilog nelinearnoj analizi armirano betonskih ramovskih konstrukcija, Građevinski fakultet, Univerzitet u Beogradu, 1989.

**Doktorska disertacija:** Eksperimentalna i teorijska analiza kontinualnih visokih greda od armiranog betona, Građevinski fakultet, Podgorica, 1992.

**Izborna zvanja:** 1982. g.- asistent pripravnik; 1989. g.- asistent; 1993. g.- docent; 1998. g.- vanredni profesor; 2003. g.- redovni profesor.

**Pedagoška aktivnost:**
Kao saradnik izvodio nastavu na predmetima Građevinski materijali, Betonske konstrukcije I, Betonske konstrukcije II i Mostovi.
Kao docent izvodio nastavu na predmetima Betonske konstrukcije I i Betonske konstrukcije II.
Kao vanredni profesor izvodio nastavu na predmetima Teorija betonskih konstrukcija i Projektovanje i građenje betonskih konstrukcija.
Kao redovni profesor izvodio nastavu na predmetima Zidane konstrukcije, Projektovanje i građenje betonskih konstrukcija i Betonske konstrukcije inženjerskih objekata.
Na master studijama izvodi nastavu iz predmeta Posebna poglavlja iz betonskih konstrukcija inženjerskih objekata.
Na doktorskim studijama izvodi nastavu iz predmeta Nelinearno ponašanje betonskih konstrukcija.
Rukovodio je izradom više desetina diplomskih i specijalističkih radova iz oblasti betonskih konstrukcija.
U školskoj 1997/98.g. učestvuje u izvođenju postdiplomske nastave na Građevinskom fakultetu Univerziteta u Beogradu koja je organizovana za strane studente, na predmetu Nelinearna analiza i teorija plastičnosti betonskih konstrukcija.

Bio je član komisija za ocjenu i odbranu tri doktorske disertacije i član komisija za ocjenu i odbranu osam magistarskih teza na Građevinskom fakultetu Univerziteta u Beogradu i na Građevinskom fakultetu Univerziteta Crne Gore, pri čemu je bio komentor za izradu jedne doktorske disertacije i dvije magistarske teze i mentor pri izradi jedne doktorske disertacije i tri magistarske teze na Univerzitetu Crne Gore.

**Naučno-istraživački i stručni rad**

Oblast naučnih istraživanja su granična stanja armiranobetonskih konstrukcija, sa posebnim razmatranjem naponskog stanja smicanja i specijalnih betona visokih performansi, kao i otpornost konstrukcija u uslovima dejstva zemljotresa. Autor ili koautor 58 naučno-istraživačkih radova iz oblasti građevinskog konstrukterstva koji su objavljeni u naučnim časopisima i na domaćim i međunarodnim naučnim skupovima. Koautor je četiri publikacije.

Autor i odgovorni projektant konstrukcije je za preko 50 značajnih građevinskih objekata - mostova, stambenih i javnih zgrada i inženjerskih objekata koji su, po svojoj prirodi, zahtijevali primjenu savremenih, naučno zasnovanih metoda projektovanja, analize i ispitivanja armirano betonskih konstrukcija. Rukovodilac ekspertskih timova je za preko 20 ekspertiza iz oblasti ocjene kvaliteta ugrađenih materijala i ocjene seizmičke otpornosti postojećih objekata.
Ekspert Agencije Crne Gore za prestrukturiranje privrede i strana ulaganja i Fonda za razvoj Crne Gore za pitanja procjene vrijednosti građevinskih objekata i krupnih infrastrukturnih sistema za potrebe transformacije i privatizacije preduzeća.

**Profesionalne aktivnosti:**

Funkcije na Univerzitetu Crne Gore i na Građevinskom fakultetu:
- Rukovodilac Laboratorije za ispitivanje materijala i konstrukcija GF od 1983.-1998.
- Prodekan za materijalna pitanja GF od 24.10.1990. do 26.09.1991.g.
- Prodekan za nastavu GF od 1.10.1994. do 8.03.1996.g.
- Direktor Instituta za zemljotresno inženjerstvo GF od 1.10.1996. do 1.10.1998.g.
- Dekan Građevinskog fakulteta od 1.10.1998. do 1.10.2000.g.
- Član Vijeća Univerziteta Crne Gore za prirodne i tehničke nauke, 2006-2007.
- Član Senata Univerziteta Crne Gore, 2007-2013.
- Šef Katedre za Betonske i Zidane konstrukcije, 2010-2015.

Članstvo u profesionalnim i naučnim tijelima i organizacijama:
- Član Odbora za tehničke nauke CANU od 1997.
- Redovni član Akademije inženjerskih nauka Crne Gore od 2008.
- Crnogorska asocijacija za zemljotresno inženjerstvo, član Glavnog odbora
- Savez građevinskih inženjera i tehničara SCG, član Glavnog odbora
- Crnogorsko društvo građevinskih konstruktera, predsjednik
- Društvo građevinskih konstruktera Makedonije, počasni član
- Jugoslovensko društvo za istraživanje materijala i konstrukcija, član Upravnog odbora
- Evropski savjet inženjerskih komora, član Nadzornog odbora
- Glavni i odgovorni urednik publikacije "Građevinski kalendar", SGITSCG, Beograd
- član redakcionog odbora časopisa "Materijali i konstrukcije", JUDIMK, Beograd
- član redakcijskog odbora časopisa "Istraživanja", Građevinski fakultet, Podgorica
- član naučnih komiteta više međunarodnih i nacionalnih naučnih skupova

**Adresa stanovanja:**
Bjelopoljska 8, Podgorica

Dr **Mladen Ulićević**, dipl.inž.građ.
redovni profesor Građevinskog fakulteta Univerziteta Crne Gore

Odabrane reference iz oblasti doktorske disertacije:
***"Efekti preraspodjele uticaja na granična stanja kontinualnih greda armiranih FRP armaturom"***

1. NAUČNE MONOGRAFIJE OBJAVLJENE NA NEKOM OD SVJETSKIH JEZIKA

1.1. ISEE 2000 - International Symposium on Earthquake Engineering, Faculty of Civil Engineering, The Proceedings: Editor-in-chief Mladen Ulicevic, Podgorica, 2000, p 423

1.2. MODERN CONCRETE STRUCTURES, Monograph, Editor M. Aćić, Faculty of Civil Engineering, Belgrade, 1994, p 309, art. 3: Aćić, M., Vujović, A., Ulićević, M.: LIMIT STATES OF RC DEEP MEMBERS WITH SIGNIFICANT INCLINED CRACKS, pp 34-44

2. NAUČNE MONOGRAFIJE MEĐUNARODNOG ZNAČAJA

2.1. AKTUELNA PITANJA UPRAVLJANJA SEIZMIČKIM RIZIKOM U CRNOJ GORI I OKRUŽENJU, Zbornik radova sa Međunarodne naučne konferencije, Urednici M. Ulićević i M. Jaćimović, Crnogorska akademija nauka i umjetnosti, Univerzitet Crne Gore, Građevinski fakultet, Podgorica, 2004, str. 247

3. RADOVI OBJAVLJENI U MEĐUNARODNIM NAUČNIM ČASOPISIMA KOJI SE NALAZE NA SCIENCE CITATION INDEX EXPANDED

3.1. Baša, N., Ulićević, M., Zejak, R.: Experimental Research of Continuous Concrete Beams with GFRP Reinforcement, Advances in Civil Engineering, vol. 2018, Article ID 6532723, p. 16, 2018.

3.2. Serdar, N., Janković, S., Ulićević, M.: Influence of horizontal curvature radius and bent skew angle on seismic response of RC bridges, GRAĐEVINAR, 69 (2017) 2, pp. 83-92, doi: https://doi.org/10.14256/JCE.1508.2015

3.3. Janković S., Ulićević M.: PROBABILISTIC SEISMIC PERFORMANCE ANALYSIS OF REINFORCED CONCRETE FRAME BUILDING DESIGNED IN LINE WITH EC8, Građevinar 64 (2012)3, Zagreb, 207-215, ISSN 1333-9095

3.4. Radovanović, Ž., Ulićević, M.: DJELOVANJE TEMPERATURE U MOSTOVIMA SANDUČASTOG POPREČNOG PRESJEKA, *Građevinar 60 (2008) 2*, Zagreb, str. 109-121

4. RADOVI OBJAVLJENI U NAUČNIM ČASOPISIMA

4.1. Ulićević, M., Janković, S.: KAPACITET I ROTACIJA PLASTIČNIH ZGLOBOVA AB RAMOVA PRI DEJSTVU ZEMLJOTRESA RAZLIČITIH KARAKTERISTIKA, časopis Tehnika - Naše Građevinarstvo 55 (2001), broj 3, Beograd 2001.

4.2. Aćić, M., Ulićević, M.: PROJEKTOVANJE SEIZMIČKI OTPORNIH ZGRADA OD ARMIRANOG BETONA (II), Publikacija Građevinski kalendar 1999, Vol. 31, SGIT Jugoslavije, Beograd, novembar 1998., str. 233-288

5. RADOVI SAOPŠTENI NA SKUPU MEĐUNARODNOG ZNAČAJA, ŠTAMPANI U CJELINI

5.1. Janković, S., Stojadinović, B., Ulićević, M., Popović, J.: THE EFFECTS OF R/C FRAME STIFFNESS MODELING ON SEISMIC PERFORMANCE, fib Symposium: Concrete Structures in Seismic Regions, Athens, (2003): 254-256

5.2. Janković, S., Stojadinović, B., Ulićević M.: PROBABILISTIC SEISMIC DEMAND MODEL FOR REINFORCED CONCRETE FRAME BUILDINGS, International Conference in Earthquake Engineering to Mark 40 Years from Catastophic 1963 Skopje Earthquake and Succesfull City Reconstruction, Skopje, Ohrid, August 2003, Proceedings

5.3. Ulićević, M.: REHABILITATION AND STRENGTHENING OF ST. EUSTAHIUS CHURCH MASONRY STEEPLE, First European Conference on Earthquake Engineering and Seismology, Geneva, (2006)

УНИВЕРЗИТЕТ ЦРНЕ ГОРЕ
Ул. Цетињска бр. 2
П. фах 99
81000 ПОДГОРИЦА
ЦРНА ГОРА
Телефон: (020) 414-255
Факс: (020) 414-230
E-mail: rektor@ac.me

UNIVERSITY OF MONTENEGRO
Ul. Cetinjska br. 2
P.O. BOX 99
81 000 PODGORICA
MONTENEGRO
Phone: (+382) 20 414-255
Fax: (+382) 20 414-230
E-mail: rektor@ac.me

Број: 08-652
Датум, 15.04.2014 г.

Ref:
Date:

Na osnovu člana 75 stav 2 Zakona o visokom obrazovanju (Sl.list RCG, br. 60/03 i Sl.list CG, br. 45/10 i 47/11) i člana 18 stav 1 tačka 3 Statuta Univerziteta Crne Gore, Senat Univerziteta Crne Gore, na sjednici održanoj 14.aprila 2014. godine, donio je

**ODLUKU**
**O IZBORU U ZVANJE**

**Dr RADOMIR ZEJAK** bira se u akademsko zvanje **redovni profesor** Univerziteta Crne Gore za predmete: Građevinski materijali i Tehnologija betona, na osnovnom akademskom studijskom programu Građevinarstvo i Primjena računara, na postdiplomskom specijalističkom akademskom studijskom programu Građevinarstvo, **na Građevinskom fakultetu** i Građevinski materijali, na osnovnim akademskim studijama, na Arhitektonskom fakultetu.

REKTOR
Mиpaнoвић Предраг
**Prof.dr Predrag Miranović**

17.04.2014

## Kratka biografija

Rođen sam 11. 01. 1962. godine u Baricama, opština Bijelo Polje. Osnovnu i srednju školu sam završio u Bijelom Polju. Za postignute rezultate u toku školovanja sam nagrađen diplomom „Luča I". Na Građevinski fakultet Univerziteta "Veljko Vlahović" u Titogradu upisao sam se školske 1981/82. godine. Po upisu na fakultet, proveo sam godinu dana u JNA. Diplomirao sam 17. februara 1987. godine na Smjeru za konstrukcije, predmet Betonske konstrukcije, sa ocjenom 10 i prosječnom ocjenom u toku studija 8.54.

U februaru 1987. godine upisao sam postdiplomske studije na Građevinskom fakultetu Univerziteta u Beogradu na Odsjeku za Betonske konstrukcije. Magistarski rad sam odbranio 10. februara 1993. godine iz oblasti armiranobetonskih konstrukcija, pod naslovom: "Prilog rješenju problema granične nosivosti vitkih armiranobetonskih elemenata", (mentor prof. dr Mirko Aćić). Doktorsku disertaciju, čiji je naslov "Prilog analizi vitkih armiranobetonskih elemenata sa kosim savijanjem" (mentor prof. dr Mirko Aćić), odbranio sam 11. februara 2003. godine, takođe na Građevinskom fakultetu Univerziteta u Beogradu.

Dobitnik sam priznanja Jugoslovenskog društva građevinskih kostruktera (JDGK) za najbolje ostvarenje u oblasti građevinskog konstrukterstva - naučno djelo za 2003. godinuu u SRJ, za doktorsku disertaciju "Prilog analizi vitkih armiranobetonskih elemenata sa kosim savijanjem".

U okviru studijskih boravaka ili kao istraživač na Projektima boravio sam na nekoliko univerziteta i instituta među kojima su: TU Wien - Institut fur Stahlbetonbau, La Sapienza - Roma, University of Architecture, Civil Engineering and Geodesy – Sofia, Tsinghua University – Beijing.

Kao predsjednik Tehničkog komiteta: TK 002 – Eurokodovi, u okviru Implementacije jedinstvenih Evropskih propisa u građevinarstvu (EN), učestvovao sam na više skupova u organizaciji Evropske Komisije (CEN, TC–250, JRC), tj. na Workshopovima u Briselu, Lisabonu, Berlinu, Beču, Dablinu i Milanu.

Znanje stranih jezika: engleski, ruski.

## REFERENCE IZ OBLASTI DOKTORATA (do 10 najvažnijih)

1. N. Baša, M. Ulićević, **R. Zejak**: ***„Experimental Research of Continuous Concrete Beams with GFRP Reinforcement"***, Advances in Civil Engineering, Article ID 6532723, 2018., 16 pages.

2. **R. Zejak,** I. Nikolić, D. Blečić, V. Radmilović, V. R. Radmilović: ***„Mechanical and Microstructural Properties of the Fly–Ash–Based Geopolymer Paste and Mortar"***, Materials and Technology, Vol.47, No. 4, 2013, , p. 535 – 540, UDK: 678.86, ISSN 1580–2949, Ljubljana, Slovenia.

3. M. Krgović, **R. Zejak**, M. Ivanović, M. Vukčević, I. Bošković, M. Knežević, B. Zlatičanin: ***„Properties of the Sintered Product Based on Electrofilter Ash Depending on the Mineral Content of Binder"***, Research Journal of Chemistry and Environment, Vol. 15, No. 4, Decembar 2011, p. 52–56, ISSN 0972–0626, Indore, India. (vodeći autor).

4. I. Nikolić, **R. Zejak**, I. J. Častvan, Lj. Karanović, V. Radmilović, V. R. Radmilović: ***„Influence of Alkali Cation on the Mechanical Properties and Durability of Fly Ash Based Geopolymers"***, Acta Chimica Slovenica, No. 3, Vol. 60, 2013, p.636 - 643, ISSN 1318-0207.

5. I. Nikolić, D. Đurović, **R. Zejak**, Lj. Karanović, M. Tadić, D. Blečić, V. R. Radmilović: ***„Compressive Strength and Hydrolytic Stability of Fly Ash – Based Geopolymers"***, Journal of the Serbian Chemical Society, No. 6, Vol. 78, 2013, p.851 - 863, ISSN 0352-5139.

6. I. Nikolić, D. Đurović, D. Blečić, **R. Zejak**, Lj. Karanović, S. Mitsche, V. R. Radmilović: ***„Geopolymerization of Coal Fly Ash in the presence of Electric Arc Furnace Dust"***, Minerals Engineering, Vol. 49, 9. April 2013, p. 24 - 32, ISSN 0892-6875.

7. M. Krgović, M. Knežević, M. Ivanović, I. Bošković, M. Vukčević, **R.Zejak**, B. Zlatičanin, S. Đurković: ***„The Properties of Sintered Product Based on electrofilter ash"***, Materials and Technology, vol.43, No. 6, 2009, , p. 327 – 331, UDK: 669+666+678+53, ISSN 1580–2949, Ljubljana, Slovenia.

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9. M. Vukčević, D. Turović, M. Krgović, I. Bošković, M. Ivanović, **R.Zejak**: ***„Utilization of Geopolymerization for Obtaining Construction Materials Based on Red Mud"***, Materials and Technology, vol.47, No. 1, 2013, p. 99 – 104, UDK: 66.095.26:691 : 539.411, , Ljubljana, Slovenia.

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Dr Radomir Zejak, dipl.inž.građ. redovni profesor

УНИВЕРЗИТЕТ У БЕОГРАДУ

02-344/8-11

Адреса: Студентски трг 1, 11000 Београд, Република Србија
Тел.: 011 3207400; Факс: 011 2638818; E-mail: officebu@rect.bg.ac.rs

Београд, 29.02.2012. године
06 Број: 020-329/XXVIII-2.2.
JJ

На основу чл. 65. ст. 2. Закона о високом образовању ("Службени гласник РС", број 76/05, 97/08, 44/10 и 100/07-аутентично тумачење), чл. 42. ст. 1. тач. 23 и чл. 43. ст. 4. Статута Универзитета у Београду ("Гласник Универзитета у Београду", број 162/11 – пречишћени текст), чл. 25. ст. 1. и ст. 2. тач. 1. Правилника о начину и поступку стицања звања и заснивања радног односа наставника Универзитета у Београду ("Гласник Универзитета у Београду", број 142/08, 150/09 и 160/11) и Критеријума за стицање звања наставника на Универзитету у Београду ("Гласник Универзитета у Београду", број 140/08, 144/08, 160/11 и 161/11), а на предлог Изборног већа Грађевинског факултета, број: 344/6 од 22.12.2011.године, и мишљења Већа научних области грађевинско-урбанистичких наука број 02:06-55/11 од 07.02.2012.године, Сенат Универзитета, на седници одржаној 29. фебруара 2012. године, донео је

**О Д Л У К У**

**БИРА СЕ проф. др СНЕЖАНА МАРИНКОВИЋ** у звање редовног професора на Универзитету у Београду – Грађевински факултет, за ужу научну област Бетонске конструкције.

*О б р а з л о ж е њ е*

Грађевински факултет је дана 16. новембра 2011. године, преко листа Послови, објавио конкурс за избор у звање редовног професора, за ужу научну област Бетонске конструкције, због потреба факултета.

Извештај Комисије за припрему извештаја о пријављеним кандидатима стављен је на увид јавности дана 05. децембра 2011. године преко библиотеке Факултета.

На основу предлога Комисије за припрему извештаја о пријављеним кандидатима, Изборно веће Факултета, на седници одржаној дана 22. децембра 2011.године, донело је одлуку о утврђивању предлога да се кандидат др Снежана Маринковић изабере у звање редовног професора.

Грађевински факултет је дана 29.децембра 2011.године доставио Универзитету комплетан захтев за избор у звање на прописаним обрасцима.

Универзитет је комплетну документацију коју је доставио Факултет ставио на web страницу Универзитета дана 31. јануара 2012. године.

Веће научних области грађевинско-урбанистичких наука, на седници одржаној дана 07. фебруара 2012.године дало је мишљење да се проф. др Снежана Маринковић може изабрати у звање редовног професора.

Сенат Универзитета, на седници одржаној дана 29. фебруара 2012. године разматрао је захтев Грађевинског факултета и утврдио да кандидат испуњава услове прописане чл. 64. и 65. Закона о високом образовању и чланом 124. Статута Универзитета у Београду, па је донета одлука као у изреци.

ПРЕДСЕДНИК СЕНАТА
Ректор

Проф. др Бранко Ковачевић

Доставити:
- Факултету (2)
- Сектору 06
- архиви Универзитета

**BIOGRAPHY**

Snežana Marinković, PhD, MSc in Civil Engineering

Professor at the Chair for materials and structures
Faculty of Civil Engineering
University of Belgrade, Serbia.

Main activities: teaching, research and design of reinforced and prestressed concrete structures.
Courses: Theory of concrete structures, Concrete structures (BSc studies), Numerical modeling of non-linear behavior of concrete structures (PhD studies).
At the Chair for materials and structures (Concrete structures group):
Assistant from 1987 to 2002.
Docent from 2002 to 2007.
Assistant professor from 2007 to 2012.
Full professor from 2012.
Vice dean of the Faculty from 2006 to 2012.
Head of Serbian Delegation in General Assembly and member of Technical Committee of FIB-International Federation for Structural Concrete (Lausanne, Switzerland).
General secretary of SDGK-Serbian Association of Structural Engineers: 2006-2010, member of Steering Committee from 2010.
Head of the Chair for materials and structures from 2015.
Supervisor and co-supervisor of 9 PhD thesis (5 in progress) and large number of MSc dissertations.
Author and co-author of more than 95publications including journal articles, books and book chapters and conference papers.
Currently participant of several national and international research projects in the area of Sustainable construction and Life Cycle Assessment.
Member of a design team of a wide variety of civil and structural engineering projects, following being the most important ones: Universal Sports Hall - the Belgrade Arena for 20000 spectators (concrete roof structure with spans of 133/103 m) in New Belgrade, the Cooling Towers for Thermo Power Plant in Kolubara, near Belgrade, the Sports Hall in Herceg Novi in Monte Negro etc.

**Snežana Marinković**
**Chair for Materials and Structures**
**Faculty of Civil Engineering**
**University of Belgrade**

**Work address**
Bulevar kralja Aleksandra 73
11000 Belgrade, Serbia
**E-mail**
sneska@imk.grf.bg.ac.rs

## Journal articles

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