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BUILDING MATERIALS AND STRUCTURES

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NUMERIČKA ANALIZA NOSIVOSTI NEUNIFORMNIH PRITISNUTIH ELEMENATA NA FLEKSIONO IZVIJANJE

NUMERICAL ANALYSIS OF FLEXURAL BUCKLING RESISTANCE OF NON-UNIFORM COMPRESSION MEMBERS

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1 UVOD

Konstruktivni elementi sa neuniformnom promenom poprečnih preseka imaju značajnu primenu u zgradarstvu, najčešće kod stubova okvirnih nosača velikih raspona, ili kod stubova industrijskih hala koje su opremljene kranovima velike nosivosti. Promena poprečnog preseka, koja prati neuniformnu raspodelu presečnih sila duž elementa, može da bude linearna duž elementa, ili skokovita u određenom broju diskretnih tačaka elementa. Ovakvim konstruktivnim rešenjem postiže se značajna ušteda u količini čeličnog materijala i ceni konstrucije.

Proračun nosivosti neuniformnih elemenata na fleksiono izvijanje kompleksan je i zahtevan sa stanovišta svakodnevne inženjerske prakse.

Izraz za elastičnu kritičnu silu izvijanja nije obuhvaćen osnovnim Ojlerovim slučajevima izvijanja. Vrednost elastične kritične sile izvijanja treba da se odredi vodeći računa o tačnoj raspodeli geometrijskih

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1 INTRODUCTION

Structural members with non-uniform change in cross sections have a significant role in constructions, usually as columns of large-scale structural frames, or as columns in industrial halls that are equipped with cranes of high load carrying capacity. The cross-sectional change, which follows the non-uniform distribution of the internal forces along the member, can be linear along the member, or stepped in a certain number of discrete points. This structural solution achieves significant savings in the amount of steel material and in the costs of construction.

Analysis of flexural buckling resistance of non-uniform compression members is complex and demanding from a standpoint of everyday engineering practices.

The critical elastic buckling force is not included in the basic Eulerian cases of buckling. The value of the

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karakteristika po dužini elementa, graničnim uslovima oslanjanja i postojanju eventualnih ekscentriciteta u položaju sistemnih osa susednih segmenta kod elemenata sa stepenastom promenom preseka. Elastična kritična sila izvijanja može da se odredi koristeći teorijske analize koje zahtevaju nalaženje rešenja nelinearne diferencijalne jednačine izvijanja, ili primenom neke od metoda naprednih numeričkih analiza. U jednostavnijim slučajevima mogu se koristiti uprošćeni postupci proračuna u kojima se vrednost elastične kritične sile izvijanja određuje modifikacijom osnovnih Ojlerovih izraza za izvijanje uniformnih elemenata sa ekvivalentnim momentom inercije, ili ekvivalentnom dužinom izvijanja, kojima se uzima u obzir geometrijska neuniformnost analiziranog elementa.

Položaj kritičnog preseka u kojem je dostignuta vrednost granične nosivosti na fleksiono izvijanje nije jednoznačno određen. Ovo je posebno izraženo kod elemenata s linearnom promenom preseka, pa se pri nalaženju rešenja zahteva iterativan postupak proračuna. U opštem slučaju može se smatrati da je to presek u kome normalni napon pritiska dostiže maksimalnu vrednost.

Za praktičnu inženjersku primenu, proračun nosivosti neuniformnih elemenata na fleksiono izvijanje treba da bude zasnovan na primeni globalne analize drugog reda, uzimajući u obzir uticaj geometrijskih, materijalnih i strukturnih imperfekcija na ponašanje konstrukcije u stanju granične nosivosti. Uticaji drugog reda mogu se uzeti u proračunu na dva načina [1], [2]:

– **Indirektno, određivanjem vrednosti kritične sile izvijanja** primenom približnih metoda ili linearno-elastične analize za element bez imperfekcija, nakon čega treba primeniti opšti postupak proračuna nosivosti na fleksiono izvijanje koji važi za uniformne elemente, u saglasnosti sa SRPS EN 1993-1-1 [3].

– **Direktno, primenom proračuna po teoriji drugog reda** s početnim ekvivalentnim globalnim i lokalnim imperfekcijama elementa. Oblik imperfekcija treba da odgovara najnižem sopstvenom obliku elastičnog izvijanja u odgovarajućoj ravni izvijanja, a veličine imperfekcija treba da budu u skladu s preporukama datim u SRPS EN 1993-1-1/NA [4]. Ovim postupkom kontrola nosivosti elemenata na izvijanje svodi se na kontrolu nosivosti najopterećenijeg poprečnog preseka za dejstvo aksijalne sile pritiska i momenta savijanja drugog reda.

Treba pomenuti primenu Metode konačnih elemenata koja se danas efikasno koristi u analizi različitih problema stabilnosti koji uključuju geometrijsku i materijalnu nelinearnost. Osim što je osnova naprednih softverskih paketa (RSTAB, Sofistik, DIANA FEA...), ona je i osnova individualnih naučnih programa razvijenih primenom različitih programskih jezika. Kao primer, program ALIN [5], [6] rešava kompleksne probleme statičke i dinamičke analize, linerane analize izvijanja putem koje se kritično opterećenje može odrediti i u elastičnoj i u nelineranoj oblasti naprezanja.

Ovaj rad prikazuje rezultate parametarske numeričke analize neuniformnih, obostrano zglobno oslonjenih i konzolnih elemenata, koja je sprovedena u softveru Abaqus [7]. Neuniformnost se ogleda u promeni poprečnog preseka kroz dva segmenta i stepenastoj promeni aksijalne sile pritiska. Variran je odnos momenta inercije poprečnih preseka gornjeg i donjeg

critical elastic bucklingload should be determined taking into account the exact distribution of geometric characteristics along the length of the member, boundary conditions, and the existence of possible eccentricity in the position of the system axes of the adjacent segments in members with a stepped change in cross section. The critical elastic buckling load can be determined using theoretical analyses that require the solution of nonlinear differential equations, or using one of the advanced numerical analysis methods. In simpler cases, simplified calculation procedures can be used in which the value of the elastic buckling force is determined by modifying the basic Eulerian expressions for buckling of uniform elements with an equivalent moment of inertia or an equivalent effective length, both of which take into account the geometric non-uniformity of the analysed member.

The position of the critical cross-section in which the ultimate flexural buckling load is achieved is unclearly defined. This is especially pronounced in the case of members with linear change of cross-section, where finding the solution asks for iterative calculations. In general, it can be at the cross-section in which normal stress reaches the maximum value.

For practical engineering applications, the flexural buckling resistance of non-uniform members should be based on the use of global second-order analysis, taking into account the influence of geometric, material and structural imperfections on the behaviour of the structure in the state of ultimate load capacity. Second-order influences can be included in design in one of the two ways [1], [2]:

– **Indirectly, by determining the value of the critical buckling load** using approximate methods or linear-elastic analysis for members without imperfections, after which the general method of calculating the flexural buckling resistance, which is valid for uniform elements, should be applied in accordance with SRPS EN 1993-1-1 [3].

– **Directly, by applying a second order theory** with the equivalent initial global and local imperfections of members. The shape of imperfection should correspond to the fundamental form of elastic self-buckling in the appropriate plane of buckling, and the size of the imperfection should be in accordance with the recommendations given in SRPS EN 1993-1-1 / NA [4]. By this procedure, the calculation of the flexural buckling resistance of members is reduced to the calculation of the load bearing capacity of the critical cross-section for the compressive axial force and bending moment of second order.

Finite element method should be mentioned as an effective method in analysis of different stability problems that include geometric and material nonlinearities. Except for being the basis of advanced software packages (RSTAB, Sofistik, DIANA FEA...), it is also the basis of individual scientific programs developed using different programming languages. As an example, program ALIN [5], [6] solves complex problems of static and dynamic analysis, linear analysis of buckling through which the critical load can be determined both in the elastic and in the non-linear stress region.

This paper presents the results of parametric numerical analysis of non-uniform, hinged ends and cantilever members carried out in Abaqus software package [7].

segmenta i odnos aksijalnih sila u segmentima. Dužina elemenata je 10 m, visina gornjeg segmenta 4 m, visina donjeg segmenta 6 m. Za zglobno oslonjene elemente poprečni presek donjeg segmenta je HEA 300 dok je poprečni presek gornjeg segmenta variran u opsegu: HEA 160, HEA 180, HEA 200, HEA 220 i HEA 240. Za konzolne elemente poprečni presek donjeg segmenta stuba je HEB 450, dok je poprečni presek gornjeg segmenta variran u opsegu: HEB 220, HEB 240, HEB 260, HEB 280 i HEB 300. Analiziran opseg odnosa sila na krajevima FE modela, P1/(P1+P2) je od 0,05 do 0,50 s korakom od 0,05. Ukupan broj analiziranih FE modela je 100. Svrha ovog rada je da se na osnovu zadatih parametara i rezultata linerane analize izvijanja utvrdi opseg koeficijenata dužine izvijanja oko jače ose inercije pojedinačanih segmenata analiziranih neuniformih elemenata. Analiza fleksione stabilnosti elemenata oko slabije ose inercije nije analizirana u okviru ovog rada, imajući u vidu praktičan značaj utvrđivanja veličine koeficijenata izvijanja oko jače ose inercije koji ne mogu biti jednoznačno određeni kako je to u slučaju izvijanja oko slabije ose preseka. Takođe, u radu je izvršena procena tačnosti proračunskih vrednosti nosivosti na fleksiono izvijanje prema opštoj metodi proračuna [3] kroz njihovo poređenje s rezultatima nelinearne numeričke analize.

The non-uniformity is reflected in the change of cross-section through two segments and the stepped change in compressive axial force. The ratio of the moment of inertia of cross-sections of the upper and lower segments and the ratio of axial forces in the segments is varied. The length of the members is 10 m, the height of the upper segment is 4 m, and the height of the lower segment is 6 m. For the hinged elements, the cross-section of the lower segment is HEA 300 while the cross-section of the upper segment varies in the range: HEA 160, HEA 180, HEA 200, HEA 220 and HEA 240. For the cantilever members, the cross section of the lower segment of the column is HEB 450 while the cross-section of the upper segment varies in the range: HEB 220, HEB 240, HEB 260, HEB 280 and HEB 300. The analysed range of force ratio at the ends of the FE model, $P_1/(P_1+P_2)$ is from 0.05 to 0.50 with a step of 0.05. The total number of FE models analysed is 100. The purpose of this paper is to determine, on the basis of the given parameters and the results of the linear analysis of buckling, the range of effective length coefficients for the major axis of the individual segments of analysed non-uniform elements. The analysis of the flexural stability of the members for the minor axis of inertia has not been analysed in this paper, given the practical importance of determining the value of the buckling coefficient for the major axis, which cannot be uniquely determined as it is in the case of buckling for the minor axis of cross-section. Also, the paper assesses the accuracy of the design values for flexural buckling resistance according to the general method [3] by comparing them with the results of nonlinear numerical analysis.

2 OPŠTA METODA PRORAČUNA PREMA SRPS EN 1993-1-1

Opšta metoda proračuna nosivosti elemenata na izvijanje data u poglavlju 6.3.1, SRPS EN 1993-1-1 [3] odnosi se na pritisnute elemente konstantnog poprečnog preseka. Opšti format kontrole nosivosti na izvijanje prema ovoj metodi, podrazumeva zadovoljenje uslova da je odnos proračunske vrednosti sile pritiska N_{Ed} i proračunske nosivosti elementa na izvijanje $N_{b,Rd}$ manji ili jednak od jedan. Proračunska nosivost elementa na izvijanje $N_{b,Rd}$ predstavlja proizvod bezdimenzionalnog koeficijenta izvijanja χ i nosivosti poprečnog preseka koja odgovara naponu na granici razvlačenja Af_y, a koji je redukovan parcijalnim koeficijentom sigurnosti γ_{M1}. Bezdimenzionalni koeficijenat izvijanja χ zavisi od relativne vitkosti elementa $\bar{\lambda}$ $\bar{\lambda}$ i koeficijenta imperfekcije α kojim su obuhvaćeni uticaji nesavršenosti realnih elemenata. Relativna vitkost za fleksiono izvijanje u slučaju klase poprečnog preseka 1, 2 i 3 može da se odredi prema opštem izrazu:

2 GENERAL DESIGN METHOD ACCORDING TO SRPS EN 1993-1-1

The general design method for buckling resistance of members given in Chapter 6.3.1, SRPS EN 1993-1-1 [3] refers to compressed members with uniform cross-section. The general format of buckling resistance design according to this method implies that the ratio of the design value of compressive load N_{Ed} and design value of buckling resistance $N_{b,Rd}$is less than or equal to one. The design value of buckling resistance $N_{b,Rd}$ represents the product of non-dimensional buckling coefficient χ and the cross-sectional load carrying capacity correspondent to the yield stress Af_y, which is reduced by the partial safety factorγ_{M1}. The non-dimensional coefficient of buckling χ depends on the relative slenderness of the member $\bar{\lambda}$ and the coefficient of imperfection α, which includes the influence of imperfections of real members. The relative slenderness for flexural buckling in the case of the cross-section classes 1, 2 and 3 can be determined according to the general expression:

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} \tag{1}$$

$$N_{cr} = \frac{\pi^2 EI}{L_{cr}} \tag{2}$$

gde su: N_{cr} Ojlerova kritična sila za fleksiono izvijanje, E modul elastičnosti, I momenat inercije poprečnog

where: $N_{cr}$is Euler critical flexural buckling load, $E$is modulus of elasticity, $I$is moment of inertia of the cross-preseka u ravni izvijanja a L_{cr} dužina izvijanja elementa.

Nesavršenosti realnih elemenata smanjuju njihovu nosivost na izvijanje. Evrokod 3 [3] definiše analitičku zavisnost između relativne vitkosti elementa $\bar{\lambda}$ i bezdimenzionalnog koeficijenta izvijanja χ preko familije krivih izvijanja. Osnova za matematičku interpretaciju ovih krivih je Ajraton-Perijeva funkcija u kojoj je uticaj nesavršenosti realnih elemenata uzet preko koeficijenta imperfekcije α. Vrednost ovog koeficijenta zavisi od oblika poprečnog preseka, relevantne ravni izvijanja, vrste proizvodnog procesa (vruće valjanje, hladno oblikovanje ili zavarivanje), kao i kvaliteta osnovnog materijala.

Opšta metoda proračuna može se primeniti i pri proračunu nosivosti pritisnutih neuniformnih elemenata na fleksiono izvijanje uz modifikaciju koja se odnosi na određivanje vrednosti kritične sile izvijanja N_{cr}, za koju se ne može koristiti izraz (2). Savremeni postupci proračuna podrazumevaju određivanje elastične kritične sile izvijanja primenom linearno-elastične analize izvijanja u odgovarajućem softveru, ili primenom približnih, pojednostavljenih metoda koje su date u relevantnoj literaturi [1], na primer u funkciji ekvivalentnog momenta inercije.

3 OPIS NUMERIČKIH MODELA

Dve različite vrste analize su urađene za svaki numerički model: linearno-elastična analiza i nelinearna statička analiza fleksionog izvijanja oko jače ose inercije, uz korišćenje Riksovog solvera. Linearno-elastična analiza, zasnovana na problemu birfukacione stabilnosti, daje elastičnu kritičnu silu izvijanja idealno pravog elementa bez imperfekcija. Odgovor elementa je praćen zanemarljivo malim bočnim deformacijama, a kada sila dostigne graničnu, kritičnu vrednost, dolazi do naglog izvijanja koje je praćeno velikim deformacijama. Kako je modul elastičnosti u linearno-elastičnoj oblasti konstantan, vrednost kritične sile izvijanja isključivo zavisi od vitkosti elemenata i graničnih uslova na krajevima elementa. S druge strane, materijalna i geometrijska nelinearnost kao i strukturne imperfekcije realnih konstruktivnih elemenata ograničavaju njihovu nosivost na fleksiono izvijanje. U takvim slučajevima linearno-elastična analiza koristi se za određivanje osnovnih oblika izvijanja, a njeni rezultati za interpretaciju početnih geometrijskih imperfekcija u kasnijim fazama analize stabilitetnih problema realnih elemenata. U Abaqusu [7] postoji nekoliko različitih numeričkih metoda za rešavanje nelineranih statičkih problema. Metoda Riksa, ili metoda kružnog luka osnovna je i najviše korišćena metoda za analizu ponašanja pritisnutih elemenata na različite oblike izvijanja. Uslov za njenu primenu jeste da kriva sila-pomeranje bude glatka i bez grananja. Tačnost vrednosti granične sile značajno zavisi od veličine početnih geometrijskih imperfekcija realnog elementa. Ukoliko su ova odstupanja idealizovana, odnosno zanemarljivo mala, početni, uzlazni deo krive je strm s naglim prelaskom u nestabilno ravnotežno stanje, kada primena Riksove metode može dovesti do divergencije rešenja.

FE modeli su realizovani sa *solid* elementima i *wedge* mrežom konačnih elemenata dimenzija 15 mm. U

section in the buckling plane and L_{cr} is effective length of member.

The imperfections of real elements reduce their buckling resistance. Eurocode 3 [3] defines the analytical relation between the relative slenderness of the member $\bar{\lambda}$ and the non-dimensional coefficient of buckling φ with a family of buckling curves. The basis for the mathematical interpretation of these curves is the Ayrton-Perry formula, in which the influence of the imperfections of real elements is taken into account using the coefficient of imperfection α. The value of this coefficient depends on the shape of the cross-section, the relevant plane of buckling, the type of the production process (hot, cold rolled or welded), as well as the quality of the steel material.

The general design method can also be applied in the flexural buckling resistance design of the compressed non-uniform members with a modification that refers to the determination of the critical buckling force value $N_{cr}$for which the expression (2) cannot be used. Modern design methods involve determining the critical elastic buckling loadusing a linear-elastic analysis in the appropriate software, or using approximate, simplified methods, given in the relevant literature [1], for example in the function of the equivalent moment of inertia.

3 NUMERICAL MODELS DESCRIPTION

Two different types of analysis were undertaken for each numerical model: linear-elastic analysis and non-linear static analysis of flexural buckling for major axis, using the Riks solver. Linear-elastic analysis, based on the problem of bifurcation stability, gives the critical elastic buckling load of an ideal real member without imperfection. The response of the member is accompanied by negligibly small lateral deformations, and when the load reaches the ultimate critical value, a sudden buckling followed by large deformations occurs. Since the modulus of elasticity in the linear-elastic region is constant, the value of the critical buckling load depends exclusively on the slenderness ofmembers and the boundary conditions at the ends of the members. On the other hand, material and geometric non-linearity as well as structural imperfections of real structural members limit their flexural buckling resistance. In these cases, the linear-elastic analysis is used to determine the basic modes of buckling, and its results are used for the interpretation of initial geometric imperfections in later stages of analysis of the stability problems of real members. In Abaqus [7] there are several different numerical methods for solving nonlinear static problems. The Riks method, or the arc-length method, is the basic and most widely used method for analysing the behaviour of compressed membersin different modes of buckling. The requirementfor its application is that load-displacement curve is smooth and without branching. The accuracy of the critical load value depends greatly on the size of the initial geometric imperfections of the real member. If these deviations are idealised, or negligible, the initial, ascending part of the curve is steep with a sudden transition to an unstable equilibrium state, when the application of the Riks method can lead to a divergence incalculations.

FE models are developed using "solid" elements and težištima poprečnih preseka, na oba kraja FE modela, definisane su referentne tačke koje su povezane s površinama krajnjih poprečnih preseka preko opcije *kinematic copuling type*. U zavisnosti od analizranog statičkog sistema FE modela, referentnim tačkama su dodeljeni odgovarajući granični uslovi koji simuliraju idealni zglobni oslonac u slučaju proste grede, odnosno uklještenje u slučaju konzole. Koncentrisane sile pritiska u pravcu podužne ose FE modela zadate su u težištu krajnjeg gornjeg poprečnog preseka, odnosno na mestu stepenaste promene poprečnog preseka, respektivno. Nominalne vrednosti krive napon-dilatacija koja je dobijena eksperimentalnim ispitivanjem epruveta uzetih iz finalnog vruće-valjanog profila HEB 260 s kvalitetom čeličnog materijala S275 [8] usvojena je za opisivanje mehaničkih svojstava materijala svih FE modela. Eksperimentalne vrednosti su transformisane u stvarne vrednosti napon-dilatacija. Pri analizi su usvojene uobičajene vrednosti elastičnih konstanti materijala za čelik E = 210000 N/mm^2 i v = 0,3. Zaostali naponi nastali kao rezultat proizvodnog procesa nisu modelirani. Standard SRPS EN 1993-1-5 [9] u Prilogu C sugeriše da se geometrijske imperfekcije mogu bazirati na obliku kritičnog oblika izvijanja u relevantnoj ravni i dopušta redukciju geometrijskih tolerancija fabričke izrade u iznosu od 80% u interpretaciji početnih nesavršenosti. Međutim, budući da zaostali naponi nisu uključeni u analizi statičkog odgovora elementa, za amplitudu početne imperfekcije usvojena je dopuštena proizvodna tolerancija koja uključuje odstupanje ose elementa od vertikalnosti u punom iznosu od L /750, u skladu sa standardom SRPS EN 1090-2 [10]. Normalizovane vrednosti koordinata deformisanog modela za kritični oblik izvijanja su skalirane i učitane su naredbom *Imperfection* u opciji-*Edit keyword* za svaki pojedinačni model. Uticaj lokalnih imperfekcija poprečnog preseka zanemaren je u okviru analize.

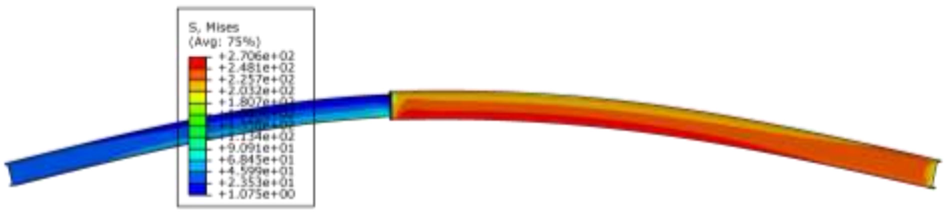
"wedge" finite element mesh with a size of 15 mm. In centroids of cross-sections, at both ends of the FE model, reference points are defined and they are coupledwiththe cross-sections at the endsusing the "kinematic coupling type" option. Depending on the type of the analysed FE model, the reference points are assigned appropriate boundary conditions that simulate an idle hinge in the case of simply supported beam, or a clamp in the case of cantilever. Concentrated compressive forces in the direction of the longitudinal axis of the FE model are applied at the centroid of the upper cross section, and at the stepped change in the cross section, respectively. The nominal values of the stress-strain curve obtained by experimental testing of the specimens taken from the final hot-rolled profile HEB 260 with the quality of steel material S275 [8] were adopted to describe the mechanical properties of materials for all FE models. The experimental values were transformed into the actual values of the stress-strain. In the analysis, the usual values of the elastic constants of steel material E = 210000 N/mm^{2}and$v$ = 0,3were adopted. The residual stresses resulting from the manufacturing process were not modelled. Design code SRPS EN 1993-1-5 [9] in Appendix C suggests that geometric imperfections can be based on the buckling modein the relevant plane and allows for the reduction of manufacturing geometric tolerances of 80% in the interpretation of initial imperfections. However, since the residual stresses are not included in the analysis of the static response of the member, the allowable manufacturing tolerance has been adopted for the amplitude of the initial imperfection, and it includes deviation of the member axis from the vertical plane in the full amount of L/750 in accordance with SRPS EN 1090-2 [10]. The normalized coordinate values of the deformed model for the critical buckling mode are scaled and assigned with the "Imperfection" command in the "Edit keyword" option for each individual model. The influences of local imperfections of the cross-section are disregarded in this analysis.

4 PRIKAZ REZULTATA NUMERIČKE ANALIZE

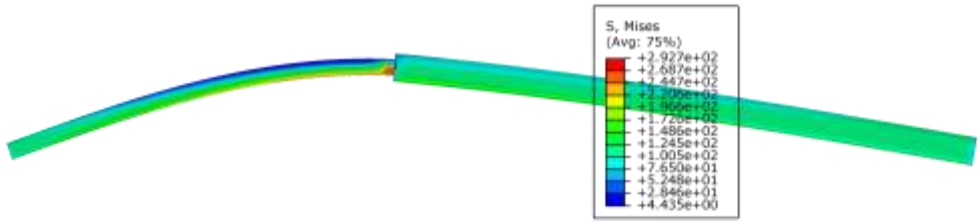
Slike 1 i 2 prikazuju raspodelu Misesovih napona pri graničnom stanju nosivosti analiziranog neuniformnog elementa statičkog sistema proste grede usled fleksionog izvijanja oko jače ose inercije. Lom elementa određen je plastifikacijom kritičnog preseka u donjem, odnosno gornjem segmentu, respektivno.

4 THE RESULTS OF THE NUMERICAL ANALYSIS

Figures 1 and 2 show the distribution of Mises stresses at the ultimate load state of the analysed non-uniform simply supported member due to the flexural buckling for major axis. The failure of the member is determined by the plasticization of the critical cross-section in the lower and upper segment, respectively.



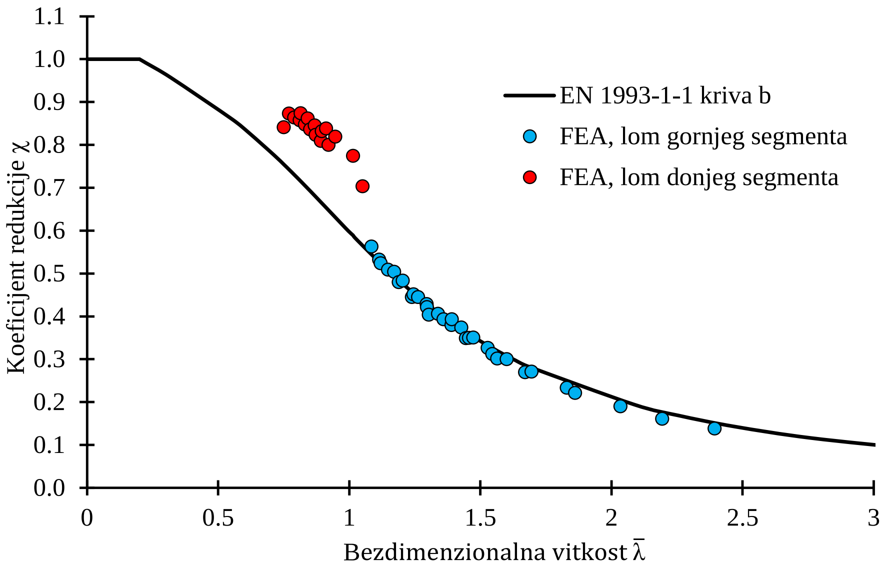
Slika 1. Lom donjeg segmenta FE neuniformnog modela
Figure 1. Failure of the lower segment of the non-uniform FE model



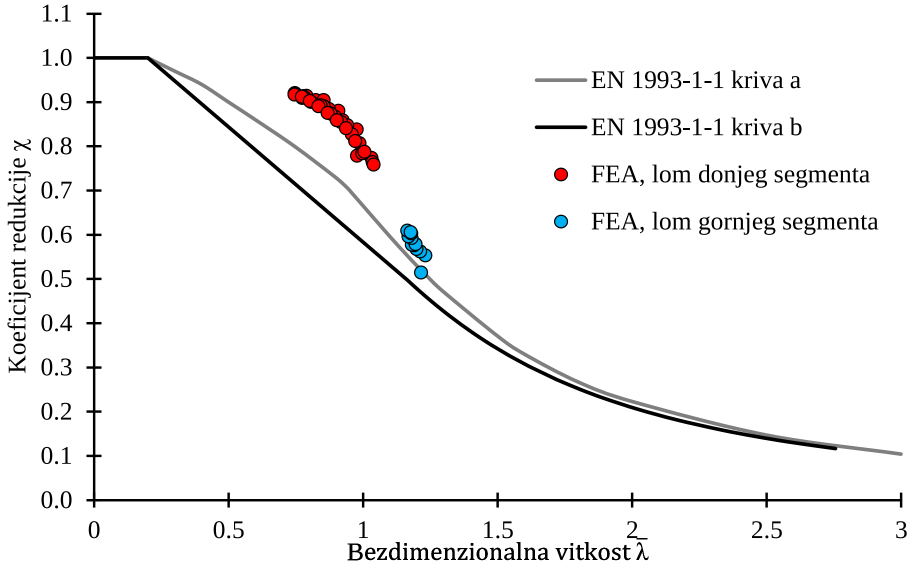
Slika 2. Lom gornjeg segmenta FE neuniformnog modela
Figure 2. Failure of the upper segment of the non-uniform FE model

Numeričke vrednosti graničnih sila fleksionog izvijanja oko jače ose inercije $N_{b,FEA}$ normalizovane su s vrednošću sile pri kojoj dolazi do plastifikacije poprečnog preseka Af_y za svaki pojedinačni segment, upoređene s rezultatima opšte metode proračuna prema SRPS EN 1993-1-1 [3] izborom relevantne krive za odgovarajući poprečni presek i grafički prezentovane na slici 3 u slučaju FE elemenata sistema proste grede, odnosno slici 4 u slučaju elemenata konzolnog sistema. Uzimajući u obzir oblik poprečnog preseka segmenta, proračunske nosivosti obostrano zglobno oslonjenih elemenata određene su izborom krive *b*. U slučaju konzolnih elemenata, proračunske nosivosti donjih segmenata sračunate su za krivu izvijanja *a*, odnosno za krivu *b* kod gornjih segmenata.

The numerical values of the ultimate flexural buckling for the major axis $N_{b,FEA}$ are normalized with the value of the force in which the cross-section is plasticized Af_y for each individual segment and compared with the results of the general design method according to SRPS EN 1993-1-1 [3] through adequate choosing of the curves for the corresponding cross-sections.These values are graphically presented in Figure 3 in the case of simply supported FE member, that is, in Figure 4 in the case of cantilevermembers. Taking into account the shape of the cross section of the segment, the design ultimate strength of simply supported members is determined by choosing the curve b. In the case of cantilever members, the design ultimate strength of the lower segments are calculated for the curve a, that is, for the curve b in the case of the upper segments.



Slika 3. Poređenje numeričkih i proračunskih vrednosti nosivosti na fleksiono izvijanje u slučaju obostrano zglobno oslonjenog elementa
Figure 3. Comparison of numerical and design values of flexural buckling resistance in the case of simply supported members



Slika 4. Poređenje numeričkih i proračunskih vrednosti nosivosti na fleksiono izvijanje u slučaju konzolnog elementa
Figure 4. Comparison of numerical and design values of flexural buckling resistance in the case of cantilever members

Tabela 1. Vrednosti koeficijenata dužine izvijanja β za zglobno oslonjene elemente
Table 1. The values of the effective length coefficients β for simply supported members

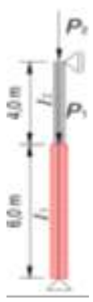
				$P_2/(P_1+P_2)$									
				0,05	0,1	0,15	0,2	0,25	0,3	0,35	0,4	0,45	0,5
	Donji segment *Lower segment*	I_2/I_1	0,09	2,295	2,398	2,497	2,593	2,687	2,777	2,866	2,951	3,035	3,117
			0,14	1,952	2,034	2,114	2,192	2,267	2,341	2,412	2,482	2,550	2,617
			0,20	1,692	1,757	1,821	1,883	1,944	2,004	2,062	2,118	2,174	2,229
			0,30	1,513	1,563	1,613	1,662	1,710	1,758	1,804	1,850	1,896	1,940
			0,42	1,394	1,430	1,468	1,506	1,544	1,581	1,618	1,655	1,692	1,727
	Gornji segment *Upper segment*		0,09	4,656	3,440	2,925	2,631	2,438	2,300	2,197	2,117	2,052	1,999
			0,14	4,854	3,577	3,035	2,725	2,522	2,377	2,268	2,183	2,114	2,058
			0,20	5,104	3,747	3,170	2,840	2,622	2,467	2,350	2,259	2,185	2,125
			0,30	5,523	4,035	3,399	3,034	2,792	2,620	2,490	2,389	2,307	2,240
			0,42	6,097	4,421	3,706	3,293	3,019	2,823	2,675	2,559	2,466	2,389

Tabela 2. Vrednosti koeficijenata dužine izvijanja β za konzolne elemente
Table 2. The values of the effective length coefficients β for cantilever members

				$P_2/(P_1+P_2)$									
				0,05	0,1	0,15	0,2	0,25	0,3	0,35	0,4	0,45	0,5
	Donji segment *Lower segment*	I_2/I_1	0,10	2,093	2,208	2,361	2,540	2,730	2,919	3,104	3,282	3,453	3,618
			0,14	2,089	2,186	2,304	2,440	2,586	2,736	2,885	3,032	3,175	3,314
			0,19	2,086	2,173	2,275	2,388	2,509	2,634	2,760	2,885	3,008	3,129
			0,24	2,084	2,166	2,257	2,357	2,462	2,569	2,679	2,788	2,896	3,003
			0,32	2,084	2,161	2,245	2,334	2,428	2,523	2,619	2,716	2,812	2,906
	Gornji segment *Upper segment*		0,10	4,469	3,332	2,910	2,711	2,606	2,544	2,504	2,477	2,457	2,442
			0,14	5,260	3,892	3,350	3,073	2,912	2,813	2,746	2,699	2,666	2,639
			0,19	6,048	4,455	3,808	3,462	3,253	3,117	3,024	2,957	2,907	2,868
			0,24	6,867	5,046	4,294	3,882	3,627	3,456	3,336	3,247	3,181	3,128
			0,32	7,847	5,753	4,880	4,395	4,088	3,878	3,727	3,615	3,529	3,461

Na osnovu zadatih ulaznih parametara koji se ogledaju u odnosu momenata inercije gornjeg i donjeg segmenta I_2/I_1 i odnosa aksijalnih sila u segmentima analiziranih FE modela $P_1/(P_1+P_2)$ s jedne strane i dobijenih vrednosti kritičnih sila izvijanja N_{cr} s druge strane, definisane su vrednosti koeficijenata izvijanja β pojedinačnih segmenata koje su prikazane u tabelama 1 i 2. Nomogrami prezentovani na slici 5a i 5b za slučaj obostrano zglobno oslonjenih elemenata, odnosno konzolnih elemenata omogućavaju grafičko određivanje koeficijenta izvijanja β. Na apscisi nomograma dat je odnos aksijalnih sila u segmentima. Crne linije na nomogramu definišu vrednosti koeficijenta dužine izvijanja za gornji segment koje se očitavaju na levoj ordinati, dok crvene linije na nomogramu definišu vrednosti koeficijenta dužine izvijanja za donji segment koje se očitavaju na desnoj ordinati.

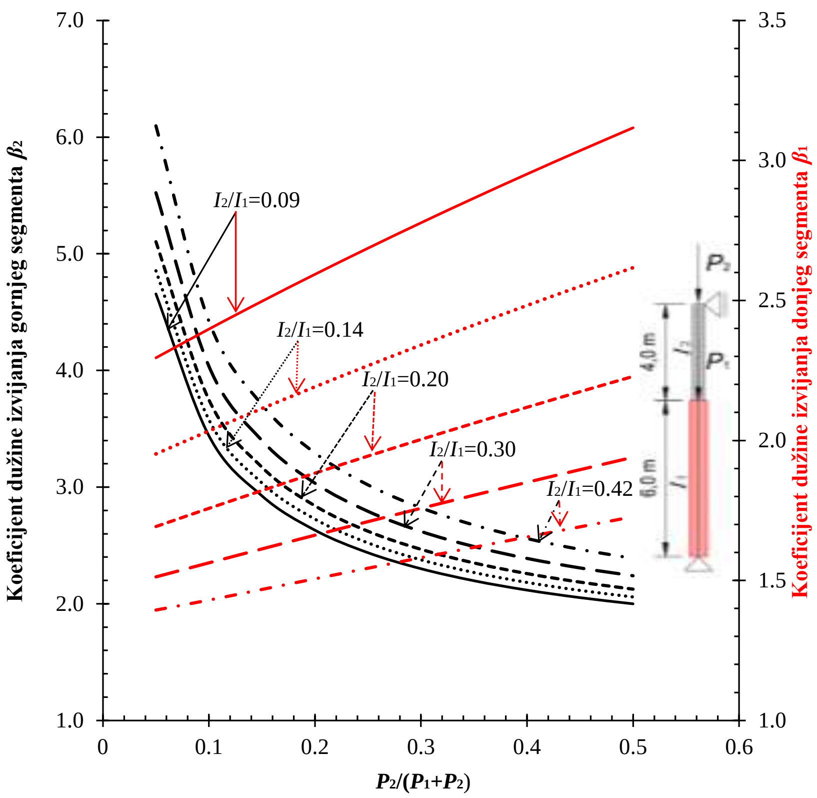
Vrednost koeficijenta dužine izvijanja za donji segment obostrano oslonjenog elementa je u opsegu od 1,4 do 3,1, a za konzolni element od 2,1 do 3,6. Koeficijenti dužine izvijanja imaju veću vrednost pri većim vrednostima odnosa sile u vrhu i sile na kontaktu segmenta. Vrednost koeficijenta smanjuje se povećanjem odnosa krutosti gornjeg i donjeg segmenta.

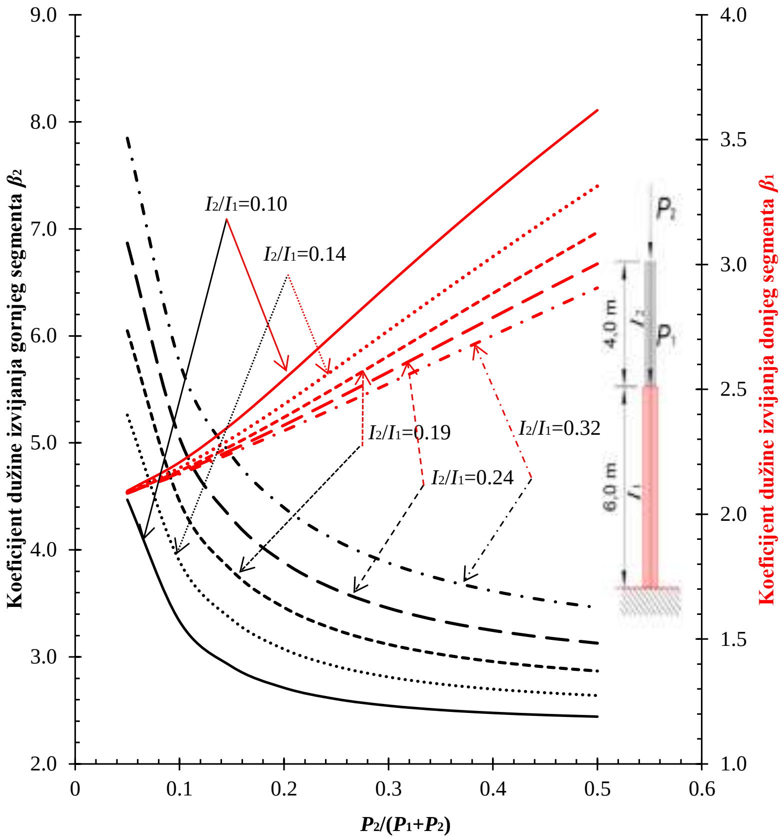
Vrednost koeficijenta dužine izvijanja za gornji segment obostrano oslonjenog elementa je u opsegu od 2 do 6,1, a za konzolni element od 2,4 do 7,8. Koeficijent dužine izvijanja opada s povećanjem odnosa sile u vrhu i sile na spoju segmenata, a raste s povećanjem odnosa krutosti gornjeg i donjeg segmenta.

Based on the given input parameters that represent the relation between the moment of inertia of the upper and lower segment I_2/I_1 and the ratio of axial forces in the segments of the analysed FE models $P_1/(P_1+P_2)$ on one hand, and the obtained values of the critical buckling loads N_{cr} on the other, the values of the buckling coefficients β of single segments are defined and shown in Tables 1 and 2. The nomograms presented in Figures 5a and 5b in the case of simply supported or cantilever members allow the graphical determination of the bucklingcoefficient β. The abscissa of the nomogram gives the ratio of axial forces in the segments. The black lines in the nomogram define the values of the effective length coefficients for the upper segment andare read on the left ordinate, while the red lines on the nomogram definethe values of theeffective length coefficient for the lower segment and are read on the right ordinate.

The effective length coefficient for the lower segment of the simply supported members is in the range of 1.4 to 3.1, and for the cantilever member from 2.1 to 3.6. The effective length coefficients have a higher value for greater values of the force at the top to the force at the segment change ratio. The value of the coefficient reduces with the increase of the ratio of the stiffness of upper and lower segments.

The value of the effective length coefficient for the upper segment of simply supported members is in the range of 2 to 6.1, and for the cantilever members from 2.4 to 7.8. The effective length coefficient decreases with the increase in the ratio of the force at the top and force at the segment change, and increases with the increase in the ratio of the stiffness of the upper and lower segments.





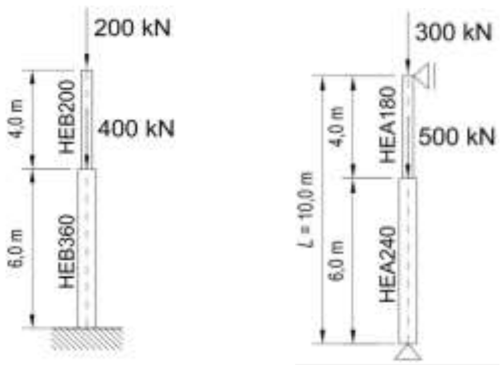
Slika 5. Koeficijent dužine izvijanja gornjeg i donjeg segmenta u funkciji odnosa momenta inercije I_2/I_1 i odnosa sila na krajevima elementa $P_1/(P_1+P_2)$
Figure 5. Effective length coefficient of upper and lower segments in function of moments of inertia ratio I_2/I_1 and axial forces ratio$P_1/(P_1+P_2)$

5 NUMERIČKE ANALIZE

Na primerima obostrano zglobno oslonjenog i konzolnog stuba, koji su prikazani na slici 6, pokazana je verifikacija rezultata koji su dobijeni u numeričkoj parametarskoj analizi.

5 NUMERICAL ANALYSES

Verification of the results of numerical parametric analysis is done through comparison with the examples of simply supported and cantilever members, shown in Figure 6.



Slika 6. Numerički primeri
Figure 6. Numerical examples

Verifikacija je sprovedena poređenjem vrednosti kritične sile fleksionog izvijanja dobijene u komercijalnom softveru Autodesk Robot 2012 i primenom nomograma koji su prikazani u ovom radu. Da bi se odredila vrednost kritične sile fleksionog izvijanja koristeći nomograme koji su dati na slici 5, potrebno je odrediti odnos momenata inercije gornjeg i donjeg segmenta i odnos vrednosti aksijalnih sila u gornjem i donjem segmentu.

The verification was carried out by comparison of the values of the flexural buckling critical force obtained in the commercial software Autodesk Robot 2012 and those obtained from the nomograms shown in this paper. In order to determine the value of the flexural buckling critical force using the nomograms given in Figure 5 it is necessary to determine the ratio of the moment of inertia of the upper and lower segments and the ratio of the axial forces in the upper and lower segments.

Konzolni stub

Cantilever column

$$\frac{I_2}{I_1} = \frac{I_{HEB200}}{I_{HEB360}} = \frac{5696\,\text{cm}^4}{43190\,\text{cm}^4} = 0{,}1319$$

$$\frac{P_2}{P_1 + P_2} = \frac{200}{200 + 400} = 0{,}333$$

Potrebno je uraditi linearnu interpolaciju za odnos momenata inercije 0,10 i 0,14.

Linear interpolation for the ratio of the moments of inertia between 0.10 and 0.14 is required.

$$\frac{I_2}{I_1} = 0{,}10 \Rightarrow \beta_1 = 3{,}04241; \beta_2 = 2{,}51763$$
$$\frac{I_2}{I_1} = 0{,}14 \Rightarrow \beta_1 = 2{,}83522; \beta_2 = 2{,}76832$$
$$\Rightarrow \frac{I_2}{I_1} = 0{,}1319 \Rightarrow \beta_1 = 2{,}87717; \beta_2 = 2{,}7156$$

Konačno, vrednost kritične sile je:

Finally, the value of the critical force is:

$$N_{cr} = \frac{E \cdot I_i \cdot \pi^2}{(\beta_i \cdot l_i)^2} \Rightarrow N_{cr,HEB360} = \frac{21000 \cdot 43190 \cdot \pi^2}{(2{,}87717 \cdot 600)^2} = 3003{,}78\,\text{kN}$$

$$N_{cr,HEB200} = \frac{21000 \cdot 5696 \cdot \pi^2}{(2{,}7156 \cdot 400)^2} = 1000{,}55\,\text{kN}$$

Obostrano zglobno oslonjen stub

Simply supported column

$$\frac{I_2}{I_1} = \frac{I_{HEA180}}{I_{HEA240}} = \frac{2510\,\text{cm}^4}{7763\,\text{cm}^4} = 0{,}3233$$

$$\frac{P_2}{P_1 + P_2} = \frac{300}{300 + 500} = 0{,}375$$

Potrebno je uraditi linearnu interpolaciju za odnos momenata inercije 0,30 i 0,42.

Linear interpolation for the ratio of the moments of inertia between 0.30 and 0.42 is required.

$$\frac{I_2}{I_1} = 0{,}30 \Rightarrow \beta_1 = 1{,}8274; \beta_2 = 2{,}4395$$
$$\frac{I_2}{I_1} = 0{,}42 \Rightarrow \beta_1 = 1{,}6368; \beta_2 = 2{,}6170$$
$$\Rightarrow \frac{I_2}{I_1} = 0{,}3233 \Rightarrow \beta_1 = 1{,}7904; \beta_2 = 2{,}4740$$

Konačno, vrednost kritične sile je:

Finally, the value of the critical force is:

$$N_{cr} = \frac{E \cdot I_i \cdot \pi^2}{(\beta_i \cdot l_i)^2} \Rightarrow N_{cr,HEA240} = \frac{21000 \cdot 7763 \cdot \pi^2}{(1{,}7904 \cdot 600)^2} = 1394{,}34\,\text{kN}$$

$$N_{cr,HEA180} = \frac{21000 \cdot 2510 \cdot \pi^2}{(2{,}4740 \cdot 400)^2} = 531{,}21\,\text{kN}$$

U tabeli 3 date su vrednosti kritičnh sila fleksionog izvijanja određenih u softveru i grafičkim putem preko nomograma i njihovo poređenje.

Table 3 gives values of flexural buckling critical force values obtained from software modelling and graphically determined using nomograms and their comparison.

Tabela 3. Poređenje vrednosti kritičnih sila fleksionog izvijanja
Table 3. Comparison of flexural buckling critical force values

		Kritična sila dobijena primenom softvera *Software critical force* $N_{cr,FEA}$	Kritična sila dobijena primenom nomograma *Nomogram critical force* $N_{cr,cal}$	Odstupanja *Differences*
		[kN]	[kN]	[%]
Konzolni stub *Cantilever*	Donji segment *Lower segment*	3022,7	3003,8	0,63
	Gornji segment *Upper segment*	1007,6	1000,6	0,70
Obostrano zglobno oslonjen stub *Simply supported column*	Donji segment *Lower segment*	1472,0	1394,3	5,57
	Gornji segment *Upper segment*	552,0	531,2	3,91

6 ZAKLJUČCI

Metoda koja indirektno uzima u obzir uticaje drugog reda uz prethodno određivanje kritične sile fleksionog izvijanja neuniformnog elementa daje dobru predikciju njegove granične nosivosti na fleksiono izvijanje.

Na osnovu rezultata parametarske numeričke analize koja je obuhvatila 100 razvijenih FE modela, definisani su grafički nomogrami i tabelarni moduli za određivanje koeficijenata dužine izvijanja segmenata neuniformnih elemenata. Analizom su obuhvaćeni najčešći statički sistemi ovakvog tipa stubova: uklješteni i obostrano zglobno oslonjeni.

Razvijeni nomogrami i tabelarni moduli jednostavni su za primenu u svakodnevnoj inženjerskoj praksi, što je pokazano u numeričkim primerima. Odstupanja između vrednosti kritične sile dobijene Linerano-elastičnom analizom i primenom nomograma ili tabela prihvatljiva su sa stanovišta tačnosti i pouzdanosti.

Parametarska analiza koja je prikazana u radu treba da bude proširena uzimajući u obzir različite visine segmenata s ciljem definisanja nomograma koji bi imali široku primenu u inženjerskoj građevinskoj praksi.

6 CONCLUSIONS

The method that indirectly takes into account the effects of the second order with the predetermination of the flexural buckling critical force of a non-uniform members provides a good prediction of its flexural buckling resistance.

Based on the results of the parametric numerical analysis that included 100 FE models, graphic nomograms and tabular modules were defined for determining effective buckling length coefficients of non-uniform members' segments. The analysis deals with the most common types of columns: simply supported and cantilever.

Developed nomograms and tabular modules are easy to apply in everyday engineering practice, as shown in numerical examples. The deviations between the critical force values obtained by linear-elastic analysis and from the nomograms or tables are acceptable from accuracy and reliability standpoint.

The parametric analysis shown in this paper should be further developed taking into account different heights of segments in order to define the nomograms that would have extensive application in the structural engineering practices.

7 LITERATURA
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REZIME

NUMERIČKA ANALIZA NOSIVOSTI NEUNIFORMNIH PRITISNUTIH ELEMENATA NA FLEKSIONO IZVIJANJE

Aljoša FILIPOVIĆ
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Rad prikazuje parametarsku linearno-elastičnu analizu fleksionog izvijanja idealizovanog neuniformnog elementa i nelineranu analizu fleksionog izvijanja sa ekvivalentnim geometrijskim imperfekcijama, urađenu u programu Abaqus. Analizom su obuhvaćeni zglobno oslonjeni i konzolni elementi sa stepenastom promenom poprečnog preseka, kod kojih je variran odnos krutosti gornjeg i donjeg segmenta i odnos vrednosti aksijalnih sila na vrhu i na mestu promene preseka. Rezultat rada je definisanje grafičkih i tabelarnih modela za određivanje koeficijenata dužine izvijanja segmenta neuniformnih elemenata na osnovu relevantne i pouzdane baze podataka koja je dobijena primenom Metode konačnih elemenata. Kao dodatak, ocenjena je pouzdanost metode proračuna fleksione stabilnosti pritisnutih stubova prema EC3 u kojoj je vrednost kritične sile izvijanja određena u prethodnom koraku primenom Elastične analize izvijanja idealizovanog elementa.

Ključne reči: neuniformni element, fleksiono izvijanje, FEA, opšta metoda, Evrokod 3

SUMMARY

NUMERICAL ANALYSIS OF FLEXURAL BUCKLING RESISTANCE OF NON-UNIFORM COMPRESSION MEMBERS

Aljosa FILIPOVIC
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Milan SPREMIC
Zlatko MARKOVIC
Nina GLUHOVIC

This paper presents parametric linear-elastic analysis of flexural buckling of idealised non-uniform member and nonlinear analysis of flexural buckling with equivalent imperfections, using software package Abaqus. The analysis includes hinged and cantileverstepped members, where the stiffness ratio of the upper and lower segments and the ratio of the values of axial forces at the top and at the change in the cross-section are varied. The aim of this paper is to define graphic and table models for determining effective lengths coefficients of non-uniform members' segments based on a relevant and reliable database that was obtained using the Finite element method. In addition, the reliability of the method for calculating the flexural stability of the compressed columns according to EC3 was evaluated in which the critical load value was determined in the previous step using the Elastic buckling analysis of idealised elements.

Key words: non-uniform members, flexural buckling, FEA, general method, Eurocode3

ANALIZA PRIMENE SEKUNDARNIH SEIZMIČKIH ELEMENATA U PRORAČUNU PREMA EVROKODU 8

THE ANALYSIS OF APPLICATION OF SECONDARY SEISMIC ELEMENTS IN DESIGN ACCORDING TO EUROCODE 8

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1 UVOD

Projektovanje seizmički otpornih konstrukcija s ciljem zaštite ljudskih života, iako jeste najvažniji, nije i jedini cilj analize ponašanja i projektovanja objekata u seizmičkim područjima. Osim obezbeđivanja prostorne stabilnosti, predmet istraživanja su i performanse objekta tokom i nakon zemljotresa, naročito nekonstruktivnih delova - fasade, pregrada, opreme, i uopšte povredljivost objekata [1]. Ipak, imajući u vidu potrebu za predstavljanjem zahteva i mogućnosti tehničkog propisa koji reguliše ovu oblast, Evrokoda 8 [2], u svetlu predstojećeg usvajanja ovog dokumenta kao nacionalnog standarda, fokus rada biće na rasvetljavanju jednog od aspekata primene Evrokoda 8 [2] u projektovanju objekata visokogradnje.

Savremeni seizmički propisi, među kojima je i Evrokod 8 [2], nude mogućnost da se doprinos pojedinih konstruktivnih elemenata u obezbeđivanju prostorne stabilnosti objekta za dejstvo zemljotresa zanemari. Takvi delovi konstrukcije nazivaju se „sekundarnim” seizmičkim elementima [2] za koje nije neophodno ispuniti sve zahteve Evrokoda 8 [2], već je moguće primeniti samo odredbe Evrokoda 2 [3]. Nekoliko je razloga za uvođenje mogućnosti podele konstruktivnih elemenata na „primarne” i „sekundarne” u aseizmičkom projektovanju. Pre svega, na ovaj način proširene su mogućnosti utvrđivanja osnovnog nosećeg sistema konstrukcije, jasnom definicijom elemenata koji su ključni

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1 INTRODUCTION

Design of structures for earthquake resistance with the purpose to protect human lives, although the most important, is not the only aim of behaviour analysis and design of structures in seismic regions. Apart from ensuring overall stability, the subjects of research are also building performances during and after earthquakes, particularly of non-structural elements – facades, partition walls, mechanical and electrical equipment and resiliency in general [1]. From the perspective of structural engineering society, there is a necessity to present requirements and possibilities of technical code that covers this field – Eurocode 8 [2], in light of the upcoming adoption of this document as a national standard. Therefore, the focus of this paper is on the presentation of one of the aspects of Eurocode 8 [2] implementation in seismic design of building structures.

Contemporary seismic codes, including Eurocode 8 [2], allow neglecting the contribution of some of the structural elements in assuring building's lateral stability during the earthquake action. Those structural elements are called "secondary seismic elements" [2] and they do not need to conform to all requirements of Eurocode 8 [2] but only to those of Eurocode 2 [3]. There are several reasons for introducing the distinction of structural elements between "primary" and "secondary" in aseismic design. First of all, it expands the possibilities for deter-

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za prijem uticaja zemljotresa i onih „sekundarnih”, kojima se prihvata isključivo gravitaciono opterećenje. Pored toga, čest je slučaj da neke odredbe Evrokoda 8 kao što su geometrijski uslovi, uslovi duktilnosti i zahtevi za oblikovanje detalja ili uslovi kapaciteta nosivosti, nije moguće ispuniti poštujući zahteve koji se odnose na položaj i dimenzije konstruktivnih elemenata. Ukoliko nije moguće promeniti dispoziciju ili bar dimenzije preseka, vodeći računa o dobro koncipiranom nosećem sistemu, označavanje tih elemenata kao sekundarnih može rešiti problem. Ovo je takođe i opcija za prevazilaženje problema da neki od konstruktivnih sistema, kao što su prethodno napregnuti sistemi ili sistemi ramova sastavljenih od stubova i delova ploča oslonjenih na njih (eng. *Flat slab frames*), nisu obuhvaćeni Evrokodom 8 [2]. Naime, postojeći eksperimentalni podaci i teorijska razmatranja nisu dovoljni da bi se sa adekvatnim stepenom sigurnosti objasnilo njihovo ponašanje pri dejstvu zemljotresa i da bi se na osnovu njih formirala pouzdana pravila za primenu u praksi. Dakle, jedna od opcija je svrstavanje ovakvih sistema u sekundarne seizmičke elemente, po principu - ako problem nije moguće rešiti na zadovoljavajući način, možda ga je moguće eliminisati [4]. Konačno, čest slučaj je da se konstrukcija visokogradnje dominantno sastoji od armiranobetonskih zidova, ali da iz konstruktivnih razloga (npr. prihvatanja teškog fasadnog zida) dođe do formiranja relativno malog broja ramova. Strogo i formalno gledano, prema aktuelnim domaćim propisima [5] ovakav sistem bi se klasifikovao kao mešovit i značajni deo seizmičkog opterećenja od čak 25% bi morao biti „dodeljen” ramovima. Potpuno suprotno osnovnoj ideji projektanta - zidovima se prihvata seizmičko opterećenje a stubovima samo gravitaciono, značajno se povećavaju dimenzije stubova i greda. Takođe, poštovanjem pravila za obezbeđivanje duktilnosti preseka povećavaju se količine armature u ovim elementima. Zato, svrstavanje pojedinih elemenata, u ovom slučaju fasadnih ramova, u grupu sekundarnih seizmičkih elemenata deluje kao primamljiva mogućnost u okviru savremenih seizmičkih propisa [2]. Ipak, iako opcija ovakve klasifikacije elemenata na prvi pogled izgleda kao jedno od najjednostavnijih rešenja, primena u proračunu konstrukcije nije trivijalna zbog niza uslova i zahteva koje treba ispuniti.

Objašnjenje koncepta, uslova i zahteva koje treba ispuniti, kao i posledica klasifikacije elemenata u grupu sekundarnih seizmičkih elemenata prema EC8 [2] osnovni je cilj ovog rada. Kako bi se detaljno objasnili svi koraci prilikom projektovanja seizmički otporne konstrukcije sa sekundarnim seizmičkim elementima, osmišljen je adekvatan numerički primer. Na bazi razmatranja rezultata analize konkretnog objekta, sprovedeno je tumačenje odredaba propisa [2] i donošenje odgovarajućih zaključaka.

2 KONCEPT PRIMARNIH I SEKUNDARNIH ELEMENATA

Osnovni koncept rada sa sekundarnim seizmičkim elementima zasniva se na zanemarenju krutosti sekundarnih elemenata pri analizi odgovora sistema u seizmičkoj proračunskoj situaciji. Da bi ovakav pristup

mining the basic lateral-force-resisting system of the building, by clearly defining the elements which are essential for resisting seismic action – primary seismic elements and those used only for supporting gravity loads – secondary seismic elements. Furthermore, there are some provisions of Eurocode 8 such as geometrical constrains, ductility requirements and detailing rules or capacity design conditions, which commonly cannot be satisfied as a result of architectural constrains regarding structural layout and dimensions of structural elements. If it is impossible to change the layout or at least cross-sectional dimensions, designation of those elements as secondary can solve the problem, while ensuring good and clear structural concept. It is also an option to overcome the problem concerning the structural systems that are not covered by Eurocode 8 [2], such as prestressed concrete structures or systems of flat slab frames. The reason is that the existing experimental data and theoretical analyses are insufficient to explain their behaviour during earthquakes with adequate certainty and to establish reliable recommendations and requirements for design practice. Therefore, one option is to classify these systems as secondary seismic elements, guided by the principle - when the problem is impossible to solve in a satisfactory manner, maybe it can be eliminated [4]. Finally, concrete building structures often consist of structural walls with only a few RC frames used for the purpose of bearing gravity loads (e.g. for supporting heavy facades). Strictly speaking, this structural system would be classified as a dual system according to current Serbian seismic design code [5] and a large portion of seismic load would be assigned to RC frames (at least 25%). As a result, column’s and beam’s dimensions are heavily increased which is contrary to the original designer’s intention – only structural walls resist seismic force and frames are used as gravity load-carrying elements. Furthermore, satisfying ductility demands would lead to an increase of required reinforcement area in those members. For this reason, classification of some elements as secondary seismic elements is certainly an appealing possibility in the framework of modern seismic design codes [2]. Although this option seems to be the simplest solution, its application in structural design is unlikely trivial since a number of conditions and requirements should be met.

The aim of this paper is to describe the concept of secondary seismic elements considering EC8 [2] demands and requirements and to present the consequences of classification of some structural elements as secondary. In order to explain all steps in the seismic design of building structures with secondary seismic elements in detail, an appropriate application example of the reinforced concrete building is designed. Eurocode 8 [2] provisions are commented based on the analysis of design results which led to the important conclusions.

2 THE CONCEPT OF PRIMARY AND SECONDARY ELEMENTS

The concept of secondary seismic elements is based on neglecting their lateral stiffness in the analysis of building structure’s seismic response. This approach is permitted only if the total contribution to building’s lateral bio moguć, doprinos krutosti sekundarnih elemenata u ukupnoj krutosti sistema je ograničen na 15% s ciljem da se globalni odgovor konstrukcije ne promeni značajno. Iz istog razloga, označavanje nekih elemenata kao sekundarnih nije dozvoljeno s namerom da se promeni klasifikacija konstrukcije iz neregularne u regularnu [2]. Ova odredba ima pre svega preventivni karakter i treba da suzbije mogućnost da se neregularnosti značajnog dela konstruktivnog sistema „prikriju" plaštom sekundarnih elemenata - npr. zidovi postoje na svim spratovima po visini „samo" ih nema u prizemlju.

Uz uvažavanje činjenice da svi konstruktivni elementi moraju da prihvate i prenesu sva gravitaciona opterećenja u seizmičkoj proračunskoj situaciji, suštinska razlika u proračunu primarnih i sekundarnih seizmičkih elemenata leži u pretpostavci ponašanja tih elemenata pri dostizanju istih maksimalnih pomeranja konstrukcije. Poznato je da duktilnost pomeranja konstruktivnog sistema zavisi od duktilnosti krivine preseka njegovih primarnih elemenata, koja odgovara faktoru redukcije seizmičkog opterećenja odnosno faktoru ponašanja *q*. Kako bi se postigla adekvatna duktilnost krivine, za takve elemente u Evrokodu 8 [2] propisani su zahtevi u pogledu armiranja preseka koji se odnose na geometrijske uslove, minimalne i maksimalne procente armiranja podužnom armaturom, kao i osiguranja od smicanja i načina utezanja preseka u kritičnim oblastima. S druge strane, svi elementi koji su klasifikovani kao sekundarni moraju da izdrže pomeranja uslovljena krutošću primarnih elemenata, ali bez jasno definisanog kapaciteta duktilnosti prema Evrokodu 8 [2]. Prva opcija zasniva se na obezbeđivanju adekvatne nosivosti koja bi odgovarala njihovom elastičnom ponašanju pri dejstvu zemljotresa, s ciljem sprečavanja krtog loma pri očekivanim, realnim pomeranjima konstrukcije. Ovakvi zahtevi rezultuju znatno većim statičkim uticajima u njima od onih koji se dobijaju uobičajenim proračunom konstrukcije - kada su svi elementi označeni kao primarni, ali ih istovremeno oslobađaju svih ograničenja i zahteva Evrokoda 8 [2] koji važe za primarne elemente. Vodeći računa o tome da svi konstruktivni elementi poseduju izvesnu duktilnost, druga mogućnost je da se sekundarni elementi dimenzionišu prema statičkim uticajima određenim na osnovu usvojenog faktora ponašanja, koji je niži od onog koji je usvojen za primarne seizmičke elemente. Time bi uticaji u sekundarnim elementima bili manji od onih koji su određeni primenom prve opcije tj. na osnovu pretpostavke elastičnog ponašanja pri zemljotresu. Međutim, sam standard [2] ne daje uputstvo za ovakav način proračuna. Treća opcija za određivanje uticaja u sekundarnim seizmičkim elementima je svakako i neka od nelinearnih metoda analize, npr. *pushover* analiza, kojom bi se realnije uzeo u obzir kapacitet duktiliteta ovih elemenata. Kako se nelinearne metode analize zasnivaju na prethodno usvojenim karakteristikama poprečnih preseka (u pogledu poprečne i podužne armature), proračun je iterativan, a za prvu iteraciju bi mogla da se primeni prva metoda proračuna. U ovom radu detaljno je objašnjena primena prve metode proračuna analizom konstrukcije u numeričkom primeru, koja daje najkonzervativnije rešenje.

Sigmund i ost. [6] pokazali su, primenom *pushover* analize na primeru kombinovanog sistema ramova i zidova (gde su ramovi klasifikovani kao sekundarni) da čak i pri zadovoljenju propisanih uslova, globalni

stiffness of all secondary seismic members is unlikely to exceed 15%, which precludes the global response of the structure to change significantly. For the same reason, designation of some structural elements as secondary members is not allowed if it changes the classification of the structure from non-regular to regular [2]. This provision serves as a preventive measure and it should suppress the possibility to conceal irregularities of building structures by designating them as secondary (e.g. structural walls that are continuous along the full height of the building except at the ground level).

On the basis of the fact that all structural elements should support and transfer gravity loads during earthquakes, the substantial difference in the design of primary and secondary seismic elements lies in the assumption of their behaviour when the structure is subjected to the same maximal displacements. It is well known that global ductility of a structural system depends on curvature ductility of its primary elements, which corresponds to the reduction factor of seismic action called the behavior factor *q*. Eurocode 8 [2] specifies the requirements for these elements in terms of design and detailing which refer to geometrical constrains, minimum and maximum values of longitudinal reinforcement ratios, as well as shear reinforcement ratio and confinement measures of boundary elements, in order to provide the sufficient curvature ductility. On the other hand, all elements classified as secondary members should withstand displacements governed by a primary system without clearly defined curvature capacity according to Eurocode 8 [2]. The first option is to provide adequate design resistance of secondary elements corresponding to the assumption of their elastic behaviour during an earthquake, in order to prevent brittle failure modes when subjected to the expected displacements induces by the seismic action. As a result of applying these demands, internal forces in secondary elements are much higher than those obtained from the usual seismic design – in which all elements are designated as primary, but they do not need to conform to the requirements of Eurocode 8 [2] specified for primary elements. Based on the fact that all structural elements with certain ductility, the other option is to design secondary elements for internal forces calculated from the analysis with adopted behaviour factor, which is lower than the one adopted for primary seismic elements. This would lead to lower internal forces in secondary elements than those obtained from the former option, i.e., based on the assumption of their elastic behaviour in the seismic design situation. However, the code [2] fails to provide the guidance for this type of analysis. Ultimately, the option for the analysis of secondary seismic elements is certainly some of the non-linear methods, e.g. *pushover* analysis, which takes into account ductility capacity of those members more realistically. Since the non-linear methods use predefined cross-sectional characteristics (in terms of longitudinal and transverse reinforcement), the analysis is iterative and the first option for the analysis of secondary elements can be used for the first iteration. Because of its simplicity and conservatism, this paper presents the application of the first method in the analysis of RC building structure considered in the numerical example.

odgovor konstrukcije može značajno da se razlikuje u zavisnosti od toga da li su ramovi označeni kao primarni ili kao sekundarni elementi. Takođe, uočeno je otvaranje plastičnih zglobova i na stubovima, kada su označeni kao sekundarni. Kako su ti elementi dimenzionisani samo prema EC2 [3], jako je važno sekundarne elemente dimenzionisati za uticaje koji se javljaju pri maksimalnim očekivanim pomeranjima konstrukcije u kojoj je krutost sekundarnih elemenata zanemarena. Fardis [7] je predložio postupak kojim je moguće proceniti ove uticaje, na osnovu odnosa relativnih međuspratnih pomeranja sistema u kome je krutost sekundarnih elemenata zanemarena i sistema u kome je krutost ovih elemenata uzeta u obzir.

There are only a few analyses of secondary seismic elements in relevant literature. Sigmund et al [6] conducted *pushover* analysis of the dual system of RC frames and shear walls, with frames taken as secondary elements. The results showed that global building response may significantly differ, depending on whether the frames are classified as primary or secondary even if the code requirements are met. Furthermore, plastic hinges development at ends of secondary columns was noticed. Since those elements are designed only in accordance with Eurocode 2 [3], it is crucial that the design of secondary elements is carried out by internal forces determined from maximal deformations of a primary seismic structure. Fardis [7] proposed a procedure for estimating these forces based on the ratio of inter-storey drifts: one in which the stiffness of secondary elements is not considered and another in which it is.

3 NUMERIČKI PRIMER

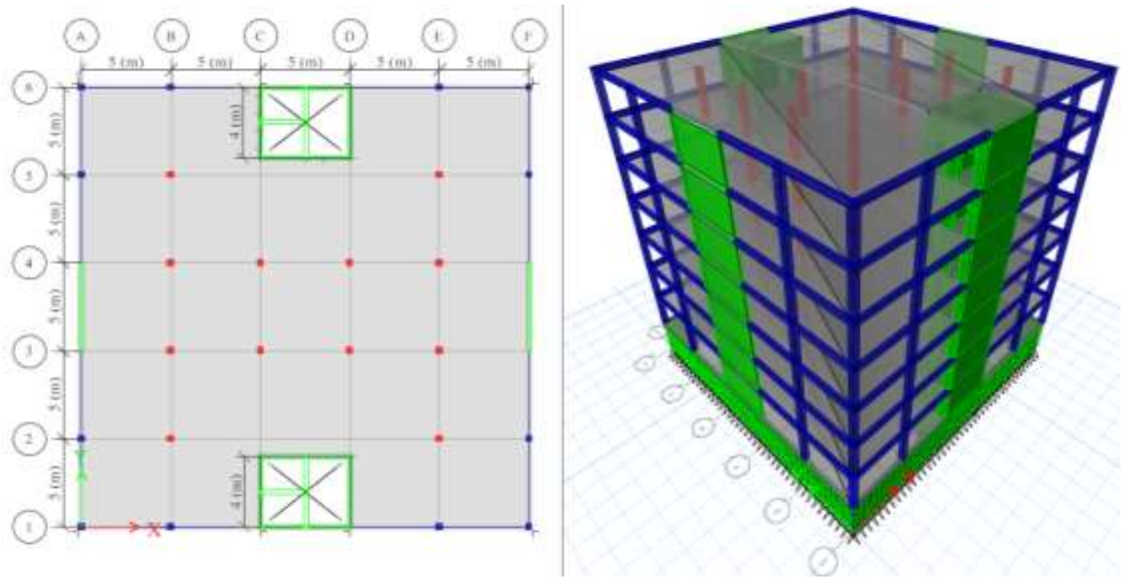
3.1 Ulazni podaci

Postupak klasifikacije primarnih i sekundarnih seizmičkih elemenata, njihova analiza i dimenzionisanje opisani su na primeru simetrične, osmoetažne armirano-betonske konstrukcije spratne visine h_s = 3,5 m, prikazane na slici 1.

3 NUMERICAL EXAMPLE

3.1 Geometry and design parameters

The classification procedure as well as the analysis, and design of primary and secondary seismic elements are described for symmetric reinforced concrete building, presented in Figure 1. The building has eight storeys and the story height of h_s = 3,5 m.



Slika 1. Numerički model razmatrane AB konstrukcije, Etabs 2015 (CSI)
Figure 1. Numerical model of RC building, Etabs 2015 (CSI)

Elementi konstrukcije koji učestvuju u prijemu horizontalnog opterećenja su armiranobetonski zidovi, fasadni ramovi koje čine stubovi sa gredama po obimu konstrukcije i ramovi koje čine unutrašnji stubovi sa delovima ploče koja je direktno oslonjena na njih (eng. *Flat slab* frames). Dimenzije elemenata konstrukcije su: debljina ploče h_p = 20 cm, debljine zidova d_z = 25 cm, dimenzije greda b_g/h_g = 25/40 cm a dimenzije stubova b_s/h_s = 40/40 cm.

Pored sopstvene težine, u nivou tavanice deluje

The lateral-force resisting system comprises reinforced concrete walls, columns and beams of the perimeter frames and flat slab directly supported on the columns – flat slab frames. Cross-sectional dimensions of structural elements are: slab thickness h_p = 20 cm, wall thickness d_z = 25 cm, beam dimensions b_g/h_g = 25/40 cm, and column dimensions b_s/h_s = 40/40 cm.

Design loads include apart from self-weight, additional dead load and live load of 2.5 kN/m^2 and 3.0 kN/m^2, respectively, as a uniform area load. The gravitaciono, jednakoraspodeljeno dodatno stalno i povremeno opterećenje intenziteta 2,5 kN/m^2 i 3,0 kN/m^2, respektivno. Usvojena je klasa čvrstoće betona C 30/37, i armatura kvaliteta B 500 (klase duktilnosti B) [3].

Projektno ubrzanje tla na osnovnoj steni a_g = 0,2g. Usvojen je projektni spektar tipa 1 za tlo kategorije B, prema EN 1998-1 [2]. Proračunom smičućih sila u zidovima utvrđeno je da sistem duktilnih zidova prihvata preko 65% ukupne seizmičke sile u oba ortogonalna pravca (približno 92%) što konstrukciju definiše kao sistem nepovezanih zidova [2]. Zahvaljujući regularnosti konstrukcije u osnovi i po visini, proračun seizmičkih uticaja izvršen je metodom Ekvivalentnih bočnih sila, sa usvojenim faktorom ponašanja q = 3,0 za klasu DCM [2].

Proračun stubova B1 i B2, grede u preseku ose 1 sa osama B i C, kao i njihovih veza, izvršen je primenom linearno-elastične analize prema EN 1992-1-1 [3] i EN 1998-1[2].

3.2 Analiza sekundarnih seizmičkih elemenata

Projektovanje i oblikovanje detalja sekundarnih elemenata i njihovih veza potrebno je izvršiti za uticaje koji nastaju pri maksimalnim deformacijama koje se javljaju usled dejstva zemljotresa, kako bi imali dovoljni kapacitet nosivosti da prihvate i prenesu gravitaciono opterećenje uključeno u seizmičku proračunsku situaciju [2]. Maksimalne deformacije sistema moguće je odrediti iz analize modela u kome je doprinos bočne krutosti svih sekundarnih elemenata zanemaren, dok se fleksiona i smičuća krutost primarnih elemenata modelira sa isprskalim presecima, pri čemu se moraju uključiti i *P-Δ* efekti.

Prethodni zahtevi Evrokoda 8 [2] podrazumevaju da je potrebno izvršiti dve analize razmatrane konstrukcije za svaki pravac seizmičkog dejstva: jednu, u kojoj se uzima u obzir horizontalna krutost svih elemenata i, drugu, u kojoj je krutost svih sekundarnih elemenata zanemarena. Da bi ovakva analiza bila moguća potrebno je formirati dva numerička modela konstrukcije [7]:

− model koji obuhvata krutost primarnih i sekundarnih elemenata - SP model, i

− model koji obuhvata krutost samo primarnih elemenata - P model.

Formiranje P modela zasniva se na zanemarenju bočne krutosti elemenata konstrukcije koje projektant želi proglasiti sekundarnim. To se postiže njihovim modeliranjem bez fleksione krutosti (redukcijom momenta inercije ili modula elastičnosti) ili postavljanjem momentnih zglobova na njihovim krajevima. Na osnovu maksimalnih deformacija dobijenih iz P modela, određuju se uticaji u sekundarnim elementima u SP modelu, postupkom koji je opisan u 3.2.2.

Osim za potrebe određivanja maksimalnih deformacija sistema, P model koristi se još i za klasifikaciju sekundarnih elemenata kao i za proračun primarnih elemenata pri dejstvu seizmičkog opterećenja (slika 3a). S druge strane, SP model koristi se za proračun sekundarnih elemenata u seizmičkoj proračunskoj situaciji, ali i za proračun cele konstrukcije u svim ostalim proračunskim situacijama.

S ciljem da se u što većoj meri pokažu specifičnosti analize nakon izbora pojedinih elemenata kao sekun-

concrete class C 30/37 and reinforcement B500 Class B are used as per Eurocode 2 [3].

The design ground acceleration of a_g= 0,2g is adopted as a design parameter. The Type 1 design spectrum applied for Ground type B is used, according to EN 1998-1 [2]. A preliminary static analysis is conducted in order to determine the fraction of seismic base shear taken by the walls. It was concluded that vertical structural walls resistance exceeds 65% of the total shear resistance of the whole structural system in both directions (approximately 92%). Therefore, the system is classified as a “wall system” [2]. The structure is regular both in plan and in elevation, which enables Lateral force method of analysis. The behaviour factor is adopted as q = 3.0 for ductility class DCM [2].

The columns B1 and B2, perimeter beams at an intersection of axis 1 and axes B and C, as well as their connections, are designed by linear-elastic analysis in accordance with EN 1992-1-1 [3] and EN 1998-1[2].

3.2 The analysis of secondary seismic elements

Secondary seismic elements and their connections should be designed and detailed for internal forces which occur at the maximum displacements during earthquakes, in order to have sufficient bearing capacity to support and transfer gravity loads included in seismic design condition [2]. Maximum deformations should be calculated in the analysis which neglects the contribution of secondary elements to the lateral stiffness of the structure while primary elements are modelled with their cracked flexural and shear stiffness. The analysis should also include *P-Δ* effects.

The Eurocode 8 [2] requirements mentioned above imply that it is necessary to conduct two separate analyses of the building structure, for each of two horizontal directions: one, in which the stiffness of all structural elements is considered and, another in which the lateral stiffness of all secondary elements is neglected. For this reason, it is necessary to build two separate numerical models of structure [7]:

− a model which includes the stiffness of primary and secondary elements - SP model, and

− a model which includes only the stiffness of primary elements - P model.

The P model is built based on neglecting lateral stiffness of those structural elements which are intended to be classified as secondary by the designer. This could be accomplished by modelling secondary elements without flexural stiffness (by reducing the moment of inertia or modulus of elasticity) or by modelling them with moment releases on their ends. Maximum displacements calculated from the P model are used for estimation of internal forces in the secondary elements in the SP model, by the procedure described in 3.2.2.

Besides being used for determination of maximum displacements of the structure, P model is also used for the purpose of classification of secondary elements as well as for the design of primary elements in seismic design situation (Figure 3a). On the other hand, SP model is used for the design of secondary elements in the seismic design situation, and for analysis and design of whole structure in all other design situations.

The aim of the paper is to highlight, as much as darnih, u ovom numeričkom primeru kao sekundarni elementi razmatrani su fasadni ramovi i ramovi koje čine unutrašnji stubovi s pločom.

possible, all the specifics of analysis which arise when certain structural elements are classified as secondary. For this purpose, in the current numerical analysis perimeter frames and flat slab frames (i.e. interior columns) are designated as secondary members.

3.2.1 Klasifikacija sekundarnih seizmičkih elemenata

Prema odredbi 4.2.2 (4) Evrokoda 8 [2], ukupan doprinos bočne krutosti svih sekundarnih seizmičkih elemenata ne sme da pređe 15% od doprinosa primarnih elemenata. Međutim, način određivanja doprinosa krutosti sekundarnih elemenata nije definisan, što omogućava dva pristupa analizi. Prva, i jednostavnija metoda bazira se na određivanju udela seizmičkih sila u posmatranom pravcu koje ovi elementi prihvataju u nivou osnove [7]. Druga metoda podrazumeva određivanje odnosa relativnih međuspratnih pomeranja konstrukcije $\delta_{r,P}/\delta_{r,SP}$ u P i SP modelu u nivou posmatrane etaže, sračunatih prema EN 1998-1: 4.3.4 [2], za isti sistem horizontalnih sila u razmatranom pravcu [7], gde su:

$\delta_{r,P}$ relativna spratna pomeranja u P modelu, a

$\delta_{r,SP}$ relativna spratna pomeranja u SP modelu.

Ovakav način klasifikacije razmatra odnos krutosti sistema preko fleksibilnosti, što je jednostavniji pristup u praktičnoj primeni, pri korišćenju softvera za analizu konstrukcija. Metoda se zasniva na analizi dva sistema sa jednim stepenom slobode, koji odgovaraju P i SP modelima definisanim u delu 3.2. Zahtev ograničenja doprinosa krutosti sekundarnih elemenata prema Evrokodu 8 [2] može se prikazati izrazom (1):

3.2.1 The classification of secondary seismic elements

The total contribution to the lateral stiffness of all secondary seismic elements should not exceed 15% of that of all primary elements, according to the requirement 4.2.2 (4) of Eurocode 8 [2]. However, the procedure for estimating the contribution of the stiffness of secondary elements is not defined, which allows two approaches to be used. The first method, and a simpler one, is based on the estimation of the fraction of base shear taken by secondary elements [7]. The second method uses inter-storey drift ratios of building structure $\delta_{r,P}/\delta_{r,SP}$ obtained from the analysis of P model and SP model at each building level. Interstorey drifts are calculated in accordance with EN 1998-1: 4.3.4 [2], for the same system of horizontal forces in each of the two horizontal directions [7], where:

$\delta_{r,P}$ is the design inter-storey drift obtained from P model, and

$\delta_{r,SP}$ is the design inter-storey drift obtained from SP model.

This method analyzes the contribution to the lateral stiffness through flexibility, which is a simpler approach in practical application when using software for structural analysis. The method is based on the analysis of two systems with a single degree of freedom (SDOF), corresponding to P model and SP model defined in Section 3.2. The requirement that limits the contribution of the stiffness of secondary elements, according to Eurocode 8 [2], can be presented by the Expression (1):

$$\frac{K_S}{K_P} \leq 0{,}15, \tag{1}$$

gde su:

K_S krutost sekundarnih seizmičkih elemenata,

K_P krutost primarnih seizmičkih elemenata koja odgovara P modelu.

Kako se klasifikacija sekundarnih seizmičkih elemenata sprovodi na osnovu pomeranja P i SP modela, uslov za klasifikaciju se može prikazati preko odgovarajućih fleksibilnosti:

where:

K_S is the lateral stiffness of all secondary seismic elements,

K_P is the lateral stiffness of all primary seismic elements which corresponds to P model.

Since the classification of secondary seismic elements is conducted for the displacements of P and SP models, the condition for the classification can be presented by using the corresponding flexibility:

$$\frac{\delta_P}{\delta_{SP}} = \frac{K_{SP}}{K_P} = \frac{K_S + K_P}{K_P} = \frac{K_S}{K_P} + 1 \leq 0{,}15 + 1 = 1{,}15 \tag{2}$$

gde je:

K_{SP} ukupna krutost sistema, koja obuhvata krutost primarnih i sekundarnih seizmičkih elemenata, $K_{SP} = K_P + K_S$.

Konačno, doprinos krutosti sekundarnih elemenata u ukupnoj krutosti sistema, izražen preko odnosa fleksibilnosti δ_P/δ_{SP}, glasi:

where:

K_{SP} is the total lateral stiffness of the system, which comprises the stiffness of both primary and secondary seismic elements, $K_{SP} = K_P + K_S$.

Finally, the contribution to the global lateral stiffness of secondary elements can be expressed in terms of the ratio of corresponding flexibilities δ_P/δ_{SP}:

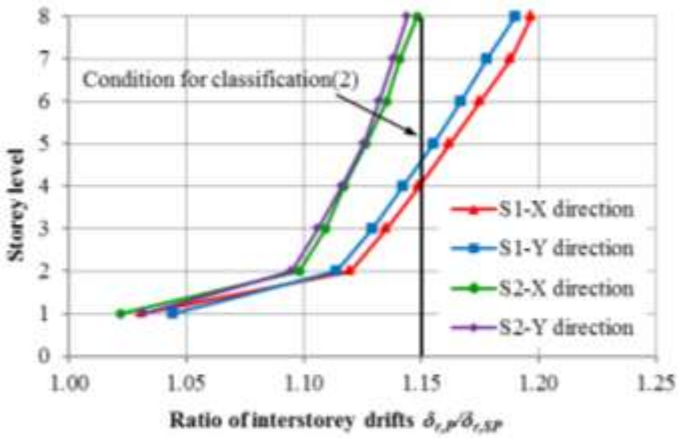
$$\frac{K_S}{K_{SP}} = \frac{K_{SP} - K_P}{K_{SP}} = 1 - \frac{K_P}{K_{SP}} = 1 - \frac{\delta_{SP}}{\delta_P} \tag{3}$$

Imajući u vidu definiciju krutosti konstrukcije, akcenat je na istom sistemu horizontalnih sila - iste raspodele po visini, ali i istog intenziteta. Prema preporuci nekih autora, raspodela opterećenja po visini treba da odgovara seizmičkom opterećenju [7]. Međutim, vrlo često se pri aproksimaciji krutosti sistema koristi i jednako raspodeljeno opterećenje po visini, što može biti jednostavnije za unos u proračunski model. Ghali i Gayed [8] pokazali su, na primeru konstrukcije od 12, 25 i 50 spratova, da je uticaj primene ove dve raspodele na odnos međuspratnih pomeranja manji od 1,0%. U ovom numeričkom primeru, razlike su manje od 1,7%, pri čemu raspodela koja odgovara seizmičkom opterećenju daje konzervativnije rezutate.

Odstupanja rezultata analize primenom ove dve metode mogu biti značajna, a posledica su različitih oblika deformisanja pojedinih konstruktivnih elemenata za prijem horizontalnog opterećenja po visini konstrukcije, koje druga metoda uzima u obzir. Razlika u obliku deformisanja elemenata posebno je naglašena u ovom numeričkom primeru (i to na višim etažama), imajući u vidu izbor elemenata koji se razmatraju kao sekundarni (fasadni ramovi i unutrašnji stubovi). Analizom relativnih spratnih pomeranja u oba modela (dijagrami S1 na slici 2), koja su sračunata za isti sistem seizmičkog opterećenja, pokazano je da zbir doprinosa krutosti svih ramova ne zadovoljava propisani uslov u oba ortogonalna pravca - $\delta_{r,P}/\delta_{r,SP}$ > 1,15. Poređenja radi, u nivou osnove ovi elementi prihvataju (svega) 8,9% ukupne seizmičke sile u X pravcu odnosno 8,6% u Y pravcu, čime bi propisani zahtev bio ispunjen. Pored rešenja u kome bi se samo jedan sistem ramova klasifikovao kao sekundarni sistem (sistem fasadnih ramova ili ploče sa unutrašnjim stubovima), za zadovoljenje uslovljenog odnosa međuspratnih pomeranja pri klasifikaciji oba sistema ramova treba ili povećati doprinos krutosti primarnih elemenata ili smanjiti doprinos krutosti sekundarnih, ukoliko je to moguće. U ovom slučaju, smanjen je doprinos krutosti sekundarnih elemenata, smanjenjem dimenzija poprečnog preseka stubova u fasadi koje iznose b_s/h_s = 25/40 cm, a određene su iz uslova duktilnosti. Rezultati analize korigovanog konstruktivnog sistema, na koji deluje sistem seizmičkih sila primenjen u prvoj iteraciji, prikazani su dijagramima S2 na slici 2.

In order to compare the lateral stiffness of two structures (P and SP models), the same system of horizontal forces – with the same distribution along the height, but of the same intensity also should be applied. The distribution of horizontal forces should correspond to the seismic load i.e. to the height-wise linear one [7]. However, it is common practice to use uniformly distributed load along the height to estimate lateral stiffness of the system, which arises from its simple application in the numerical analysis. Ghali and Gayed [8] showed that the influence of application of these two load distributions on the inter-storey drifts is less than 1.0%, based on the analysis of building structures with 12, 25 and 50 storeys. In the current numerical analysis, the differences are less than 1.7%, and the load distribution which corresponds to seismic load gives slightly conservative results.

The discrepancies of the analysis results arising from the application of these two methods can be significant. They are the result of the different deformed shape of the certain structural elements that are a part of a lateral-force-resisting system, which the other method takes into account. The difference between the deformed shapes of the elements is especially noticeable (at higher levels), considering the selection of the elements which are analyzed as secondary (perimeter frames and interior columns). The analysis of inter-storey drifts of both models (curves S1 in Figure 2), calculated for the same system of seismic load, have shown that the stiffness contribution of all frames (perimeter frames and flat slab frames) fail to fulfill the code requirement in both horizontal directions - $\delta_{r,P}/\delta_{r,SP}$> 1,15. For the purpose of comparison, these elements resist only 8.9% of total seismic base shear in X direction and 8.6% in Y direction, which would satisfy the code requirements. There are few possible solutions that satisfy code requirements in terms of the ratio of inter-storey drifts: (1) to increase the contribution to the lateral stiffness of primary elements, (2) to decrease the contribution of secondary elements, or (3) to classify only one system of frames as secondary (system of perimeter frames or system of flat slab frames). In this particular case, the contribution to the lateral stiffness of secondary members is decreased, by decreasing the cross-sectional dimensions of perimeter column to b_s/h_s = 25/40 cm,



Slika 2. Doprinos krutosti sekundarnih seizmičkih elemenata
Figure 2. Contribution of secondary seismic elements

determined from the ductility condition. The curves S2 depicted in Figure 2 present the analysis results of modified structure, loaded with the same seismic force system as in the first iteration.

3.2.2 Uticaji u sekundarnim seizmičkim elementima

Zahtev koji Evrokod 8 primenjuje za dimenzionisanje sekundarnih elemenata zasniva se na principu „jednakih pomeranja" koji vodi računa o različitom (smanjenom) kapacitetu duktilnosti sekundarnih elemenata (i njihovih veza) u odnosu na duktilnost primarnih elemenata. Ukoliko nije preciznije utvrđen kapacitet duktilnosti svih sekundarnih elemenata, potrebno je obezbediti onu nosivost koja bi odgovarala njihovom elastičnom ponašanju pri dejstvu zemljotresa. Štaviše, uticaje u ovim elementima treba odrediti na osnovu maksimalnih pomeranja u fleksibilnijem sistemu (P model), sa ciljem da se obuhvati najnepovoljniji mogući slučaj njihovog naprezanja (slika 3.a). To praktično znači da će uticaji u sekundarnim elementima biti veći od onih koji bi se javili kada bi ponašanje cele konstrukcije bilo elastično pri seizmičkom dejstvu, i to srazmerno odnosu pomeranja P i SP modela. Sličan princip proračuna uticaja u sekundarnim elementima prikazao je Milev [9]. Dobra procena ovih uticaja po visini konstrukcije može se dobiti pomoću odnosa relativnih spratnih pomeranja $d_{r,P,m}/d_{r,SP,m}$ (slika 3.b), određenih za seizmičko opterećenje koje je sračunato prema dinamičkim karakteristikama odgovarajućeg modela [7], za razliku od slučaja analize njihovog doprinosa krutosti pri klasifikaciji. Koristeći definisane odnose, uticaji na *m*-tom spratu u svim sekundarnim elementima u SP modelu (slika 3a), dobijaju se modifikacijom kombinacije opterećenja u seizmičkoj proračunskoj situaciji [10], koeficijentom α, tako da je:

3.2.2 Internal forces in secondary seismic elements

The Eurocode 8 requirement for the design of secondary elements is determined on the basis of "equal displacement" rule which considers different (reduced) ductility capacity of secondary elements (and their connections) in comparison with ductility capacity of primary elements. If the ductility capacity of all secondary members is not determined precisely, it is crucial to provide adequate resistance corresponding to the assumption of their elastic behaviour during the earthquake action. Moreover, the internal forces in these elements are determined from seismic displacements of the system which is more flexible (P model), in order to take into account the most unfavourable design condition (Figure 3.a). In other words, the internal forces in secondary elements are higher than those obtained from the analysis of the whole structure under seismic actions with the assumed elastic behaviour, proportionally to the inter-storey drift ratio of P and SP models. Milev [9] presented similar approach for calculation of internal forces in secondary members. A relatively accurate estimation of internal forces in secondary elements throughout the structure can be obtained by using the inter-storey drift ratios $d_{r,P,m}/d_{r,SP,m}$ (Figure 3.b). Unlike the case of the interstorey drift analysis for the classification of secondary elements, the interstorey drifts $d_{r,P,m}$ and $d_{r,SP,m}$ are calculated under the actual design seismic load acting on corresponding structural model [7]. Finally, the internal forces in secondary members at the floor level *m*, are computed in the SP model for the seismic combination [10], with introducing the coefficient α as a multiplier of the seismic action:

$$\sum_i G_{ki} + \alpha \cdot A_{Ed} + \sum_i \psi_{2,i} Q_{ki} \tag{4}$$

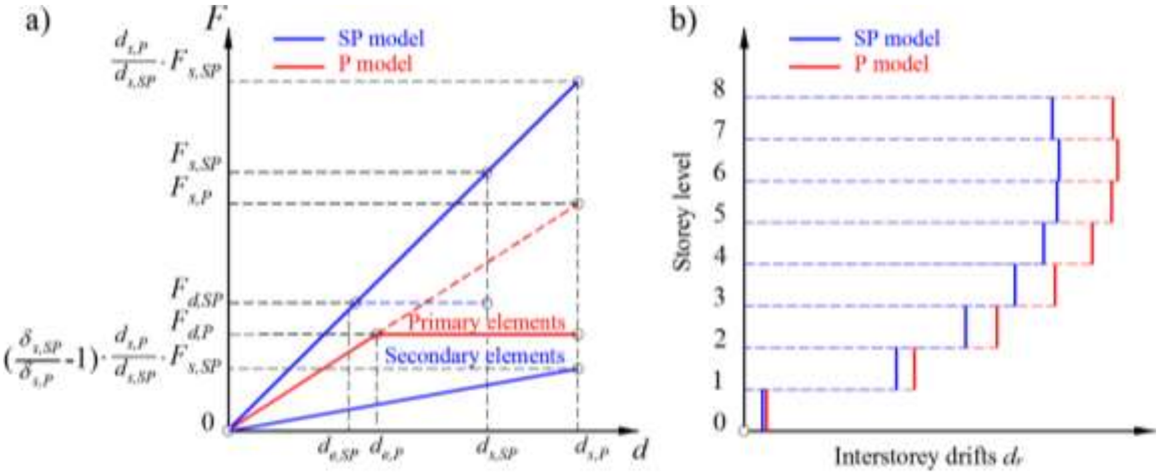
$$\alpha = q \cdot \frac{d_{r,P,m}}{d_{r,SP,m}} \cdot \frac{1}{(1-\theta_m)} \tag{5}$$

gde je:

- A_{Ed} seizmičko opterećenje;
- q faktor ponašanja konstrukcije u posmatranom pravcu i za usvojenu klasu duktilnosti;
- $d_{r,P,m}$ relativno spratno pomeranje u P modelu na *m*-tom spratu;
- $d_{r,SP,m}$ relativno spratno pomeranje u SP modelu na *m*-tom spratu, a
- θ_m koeficijent kojim se definišu *P-Δ* efekti, sračunat prema 4.4.2.2 (2) i (3) [2].

where:

- A_{Ed} is the design value of seismic action;
- q is the behaviour factor of the building, determined for each horizontal direction and for adopted Ductility Class;
- $d_{r,P,m}$ is the interstorey drift of the P model at level *m*;
- $d_{r,SP,m}$ is the interstorey drift of the SP model at level *m*, and
- θ_m is the interstorey drift sensitivity coefficient, which takes into account *P-Δ* effects, calculated in accordance with 4.4.2.2 (2) and (3) [2].



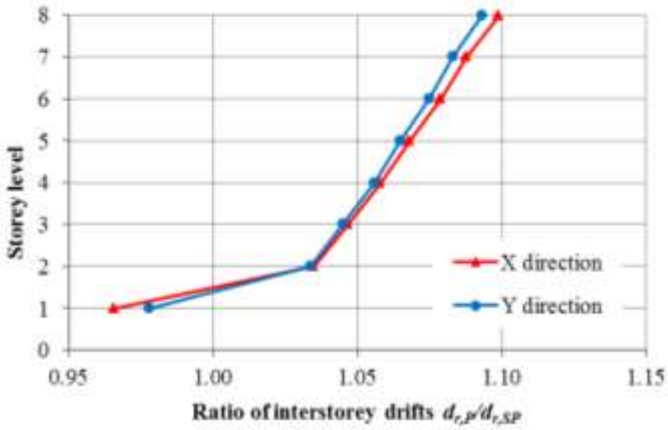
Slika 3. Primena principa „jednakih pomeranja" na proračun sekundarnih elemenata
Figure 3. Application of "equal displacement" rule in the design of secondary seismic elements

Opisani postupak može da se zakomplikuje pri analizi relativno krutih konstrukcija, sa osnovnim periodom oscilovanja manjim od T_c [2], gde princip „jednakih pomeranja" ne važi već princip „jednakih energija deformacija". Smatra se da je dovoljno tačno sračunati relativna međuspratna pomeranja koristeći izraz za duktilnost pomeranja μ_δ koji je dat u 5.2.3.4 (3) [2] i pomoću njih odrediti koeficijent α.

Uticaji u sekundarnim elementima razmatrane konstrukcije određeni su principom „jednakih pomeranja", imajući u vidu da su osnovni periodi oscilovanja u oba pravca približno jednaki 0,80 s i 0,85 s za SP model i P model, respektivno, što je veće od propisane vrednosti perioda T_c za kategoriju tla B (T_c = 0,5 s). Odgovarajuće vrednosti seizmičkih sila, određene metodom Ekvivalentnih bočnih sila, približno su jednake 4860 kN odnosno 4575 kN.

The procedure described above is inadequate for the analysis of rigid, short-period structures (with fundamental period of vibration smaller than T_c [2]), and instead of the "equal displacement" rule, the so-called "equal energy approximation" is used. In this case, the calculation of interstorey drifts on the basis of the displacement ductility factor μ_δ (given in 5.2.3.4 (3) [2]), for the purpose of determining the coefficient α, is considered as a reasonably accurate.

In the current numerical analysis, the fundamental periods of SP model and P model are approximately equal to 0.80 s and 0.85 s, respectively, which are larger than T_c = 0.5 s (Ground type B). The corresponding seismic forces, determined by Lateral force method of analysis, are about 4860 kN and 4575 kN. Therefore, the internal forces in secondary members are computed by using "equal displacement" rule.



Slika 4. Odnos relativnih spratnih pomeranja
Figure 4. Interstorey drift ratios

S obzirom na to što se P-Δ efekti mogu zanemariti (vrednost koeficijenta $\theta_{max} \approx$ 0,03), uticaji u sekundarnim elementima dobijaju se množenjem odnosa relativnih pomeranja prikazanim na slici 4, faktorom ponašanja q = 3,0, što povećava uticaje od seizmičkog opterećenja od

Since the value of sensitivity coefficient is low ($\theta_{max} \approx$ 0.03), the P-Δ effects can be neglected. As a result, the internal forces in secondary elements are obtained by multiplying the interstorey drift ratios, presented in Figure 4, with a behaviour factor q equal to 3.0. This increases

3,0 do 3,3 puta u odnosu na uticaje dobijene za primarne elemente, izraz (5). U nastavku su analizirani rezultati proračuna pojedinih konstruktivnih elemenata, koji su razmatrani kao: (1) primarni i (2) sekundarni elementi.

3.3 Analiza rezultata proračuna

Kako je to ranije istaknuto, pri proračunu primarnih elemenata od ključnog značaja je obezbediti njihovo duktilno ponašanje pri dejstvu zemljostresa oblikovanjem detalja kako bi izdržali nelinearne deformacije koje se tom prilikom javljaju. Ne vodeći računa o kapacitetu duktilnosti, sekundarni elementi svojom nosivošću treba da izdrže ista pomeranja konstrukcije.

Na primeru fasadnog stuba B1 i grede BC-1, unutrašnjeg stuba B2 i ploče koja se direktno oslanja na taj stub izvršena je uporedna analiza rezultata proračuna i istaknute su razlike u zahtevima koje ovi elementi moraju da ispune u slučaju kada su deo primarnog, odnosno sekundarnog sistema sa prihvatanje seizmičkog opterećenja.

3.3.1 Rezultati proračuna stuba B2 i njegove veze sa direktno oslonjenom pločom

Prikaz rezultata proračuna stuba B2 na slici 5, na osnovu merodavnih uticaja na pojedinim etažama, jasno pokazuje posledice njegove klasifikacije kao primarnog (PSE) odnosno sekundarnog (SSE) elementa. Kada je on razmatran kao primarni, zahvaljujući malom doprinosu unutrašnjeg stuba ukupnoj krutosti razmatrane konstrukcije, potrebne površine podužne armature su značajno manje od minimalno propisane za klasu DCM (slika 5a). Ista (minimalna) armatura dovoljna je da obezbedi zahtevanu nosivost stuba kada je razmatran i kao sekundarni, osim na poslednjoj etaži koja je merodavna za dimenzionisanje preseka. Pored toga, moguće je i smanjiti dimenzije preseka, imajući u vidu da uslov propisani duktilnosti ne važi za sekundarne elemente tj. da se dimenzije preseka mogu odrediti iz uslova maksimalnog dozvoljenog napona u betonu [3], što u konkretnom primeru znači smanjenje dimenzije sa 40 cm na 35 cm (tabela 1). Poređenja radi, za konstrukciju od 11 etaža sa istom dispozicijom, ovo smanjenje bi iznosilo oko 45% površine stuba.

the seismic action effects from 3.0 to 3.3 times in relation to the effects obtained for the primary elements, as per Equation (5). The following sections are focused on the differences between the analysis results of certain structural elements when considered as (1) primary and (2) secondary seismic elements.

3.3 The analysis of the design results

As pointed out before, the foundation for the design of primary elements is their capability of developing ductile behaviour which enables them to sustain large deformations in inelastic range under the seismic action. This is achieved by proper detailing of those members, especially in certain (“dissipative”) zones i.e. “critical regions” [2]. Unlike, the secondary members rely upon their strength, instead of ductility, to support gravity loads when subjected to the same displacements as primary members.

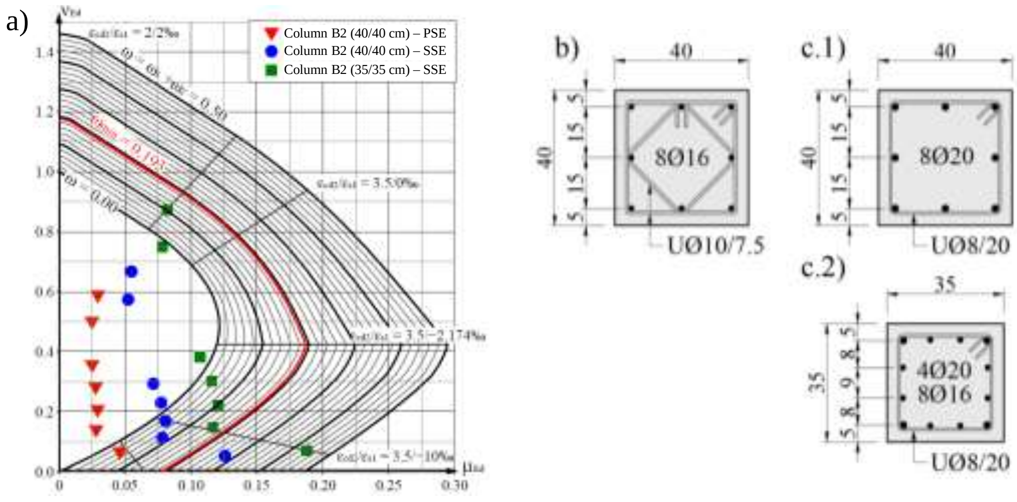
For the purpose of comparative study, the perimeter column B1 and beam BC-1, interior column B2 and its connection to flat slab are analyzed and discussed in cases of their classification as part of (1) primary and (2) secondary system for resisting seismic action.

3.3.1 The design results of column B2 and corresponding slab-to-column connection

The design results for column B2 under the extreme design combination of actions at each floor level are presented in Figure 5. It clearly shows the influence of the classification of column as primary (PSE) and secondary (SSE) element, in terms of required longitudinal reinforcement ratio. In case of its designation as primary element, the required reinforcement ratio is clearly lower than a minimum value required for Ductility class DCM (Figure 5.a), which arises from its small contribution to the global lateral stiffness. The same (minimum) amount of reinforcement is sufficient to ensure the required resistance of the column when it is designated as secondary, except at the top storey which is critical for section design. Furthermore, it is possible to decrease its cross-sectional dimensions, considering the fact that they are not governed by ductility requirements in terms of the maximum value of normalized axial force. Instead, the cross-sectional dimensions can be determined by limiting the compressive stresses in serviceability limit states [3]. In this case, the cross-sectional side length is decreased from 40 cm to 35 cm. For comparison, the cross-sectional area of the same column of 11-storey building with the same structural layout can be decreased by 45 %.

Tabela 1. Rezultati proračuna stuba B2
Table 1. Design results of column B2

Klasifikacija *Classification*	*b/h [cm]*	$\rho_{sl,max}$ [%]	$\omega_{wd,1}$	$\omega_{wd,2\text{-}7}$
PSE	40/40	1,00	0,187	0,113
SSE	40/40	1,57	0,106	0,106
SSE	35/35	2,34	0,125	0,125



Slika 5. Rezultati proračuna stuba B2: a) dijagram interakcije, b) poprečni presek stuba B2 kao PSE, c) poprečni preseci stuba B2 kao SSE

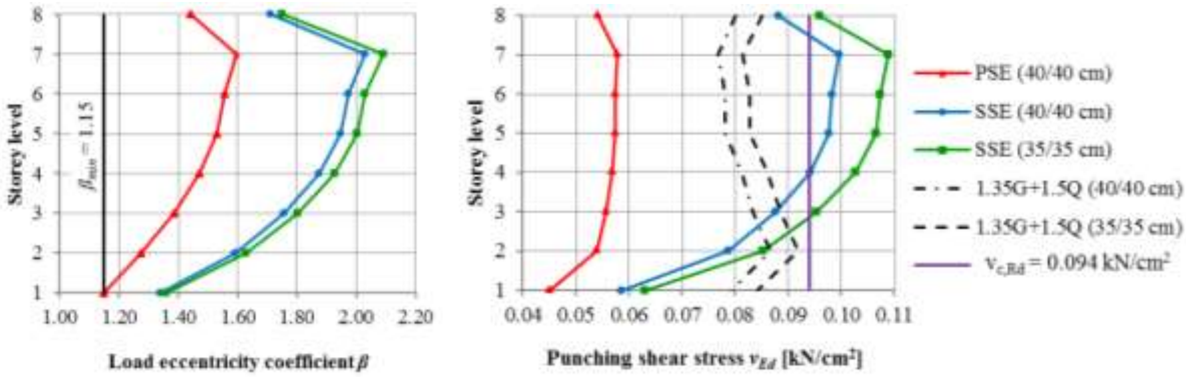
Figure 5. Design results of column B2: a) interaction diagram, b) cross section of the column B2 (PSE), c) cross section of column B2 (SSE)

Razlike u potrebnoj količini uzengija prikazane su na slikama 5b i 5c, kao i u tabeli 1 pomoću mehaničkog zapreminskog procenta armiranja $\omega_{wd,m}$ određenog za ceo *m*-ti sprat. Na osnovu prikazanih rezultata može se zaključiti da zahtevi za armiranje uzengijama primarnog seizmičkog stuba u kritičnim oblastima imaju rezultat u značajnom povećanju količine uzengija, i do 75% u nivou osnove gde je potrebno obezbediti adekvatno utezanje preseka. Jasno je, takođe, da klasifikacija u sekundarne rezultuje većom podužnom armaturom, ali kada se povede bitka za svaki kvadratni centimetar (skupog) slobodnog prostora, verovatno će opcija jače armiranih stubova manjih dimenzija dobiti prednost nad stubovima većih dimenzija s minimalnom armaturom.

Pored dokaza nosivosti stuba, neophodno je dokazati i nosivost veze stuba s pločom pri maksimalnim pomeranjima usled dejstva zemljotresa, kao što je navedeno u 3.2, što pre svega podrazumeva kontrolu smičućih napona od probijanja. Poznato je da ovi naponi zavise od gravitacionog opterećenja, ali se njihova vrednost značajno povećava pri dejstvu zemljotresa, što je posledica povećanja ekscentriciteta opterećenja obuhvaćenog koeficijentom β [3]. Na slici 6 prikazane su vrednosti koeficijenta β kao i smičućih napona u kritičnom preseku po visini konstrukcije, za stalnu proračunsku situaciju i seizmičku, pri različitoj klasifikaciji stuba B2. Može se zaključiti da ploča ima dovoljnu nosivost na probijanje bez armature za smicanje ($v_{c,Rd}$ = 0,094 kN/cm^2) pri dejstvu gravitacionog opterećenja u stalnoj proračunskoj situaciji. Kao rezultat povećanja momenata savijanja u sekundarnim elementima, rastu vrednosti koeficijenta β (gotovo dva puta više od minimalne propisane vrednosti od 1,15 [3]) i smičućih napona, koje prevazilaze vrednosti sračunate u stalnoj proračunskoj situaciji kao i nosivost ploče bez smičuće armature, što rezultuje potrebom za osiguranjem ploče armaturom za smicanje od probijanja.

The differences in required shear reinforcement area are shown in Figure 5b and 5c as well as in Table 1, in terms of mechanical volumetric ratio $\omega_{wd,m}$ calculated for the whole storey *m*. Based on presented results, it can be concluded that the detailing requirements of primary seismic column in critical areas give an increase of the shear reinforcement, up to 75 % at the column base where adequate degree of the confinement is needed. It is also clear that an increase of longitudinal reinforcement is a consequence of classification of the column as secondary. However, in the discussion for each square centimetre of (expensive) available space, the option of more heavily reinforced columns with smaller cross-sectional dimensions will probably prevail over the option of column with larger cross-sectional dimensions and minimum reinforcement ratios.

Apart from the column, the slab-to-column connections should also be designed and detailed when subjected to the maximum displacements due to earthquakes, as mentioned in Section 3.2. This implies, above all, that punching shear stresses need to be checked. It is well known that these stresses are a function of the intensity of gravity loads, but they also increase during earthquakes, as a consequence of an increase of the load eccentricity presented with the coefficient β [3]. Figure 6 presents the values of coefficient β as well as the shear stresses at the basic control perimeter along the height of the building as a function of different classification of column B2, for persistent and seismic design situation. It can be concluded that the slab has sufficient punching shear resistance ($v_{c,Rd}$ = 0,094 kN/cm^2) under gravity loads in the persistent design situation. The increase of bending moments in secondary columns leads to increase of coefficient β (almost two times than minimum prescribed value of 1.15 [3]) and punching shear stresses, which are higher than corresponding values calculated in persistent design situation. Moreover, the punching shear resistance is exceeded and, therefore, punching shear reinforcement is required.



Slika 6. Vrednosti koeficijenta β i napona smicanja od probijanja u funkciji klasifikacije stuba B2
Figure 6. Values of coefficient β and punching shear stress as a function of classification of column B2

Kako bi veza stuba i ploče u sekundarnom sistemu imala dovoljni kapacitet nosivosti da u elastičnoj oblasti prenese gravitaciono opterećenje pri dejstvu zemljotresa, od suštinske je važnosti obezbediti i odgovarajuću armaturu za savijanje na mestima oslonaca ploče, tj. na vezi ploča-stub. Imajući u vidu da se momenti velikog intenziteta na krajevima stubova uravnotežuju s momentima u ploči, javlja se potreba za armiranjem obe zone ploče nad osloncem usled momenata alternativnog znaka. U konkretnom slučaju, ovi momenti dostižu i do 80% negativnih oslonačkih momenata od gravitacionog opterećenja. Rešavanje detalja armiranja ploče direktno oslonjene na stubove s ciljem obezbeđivanja adekvatnog kapaciteta duktilnosti umesto nosivosti, kao što je ranije naglašeno, nije obuhvaćeno Evrokodom 8 [2].

3.3.2 Rezultati proračuna rama u osi 1 - stub B1 i greda BC-1

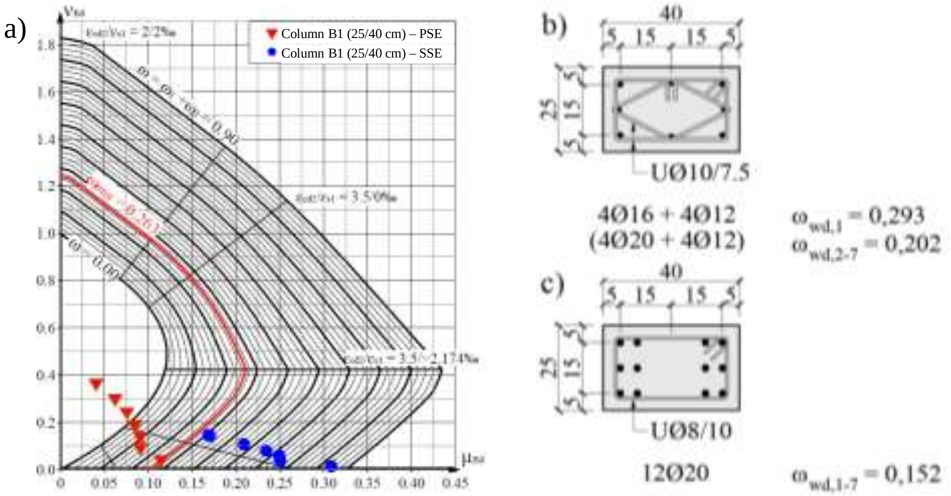
Činjenica da je uticaj krutosti ramova na veličinu i oblik deformacije čitave konstrukcije po visini dominantan, obrazložena je u delu 3.2.1. Do istog zaključka dolazi se analizom rezultata proračuna elemenata rama u osi 1, prikazanih na slici 7 i u tabeli 2. Kao posledica ramovskog dejstva u kome je izražen uticaj aksijalnih sila u stubovima, primena izraza (1) na proračun fasadnih stubova kao sekundarnih elemenata dovodi do smanjenja aksijalnih sila uz povećanje momenata što dodatno utiče na povećanje potrebne površine armature, posebno na donjim etažama (slika 7a). Zbog smanjene širine preseka stuba (određene iz uslova duktilnosti), normalizovane aksijalne sile u kritičnoj oblasti u osnovi su visoke ($\nu_{d,max}$ = 0,53), što rezultuje izraženom potrebom za utezanjem stubova kao primarnih elemenata. Vrednosti mehaničkog zapreminskog procenta armiranja $\omega_{wd,m}$ su za 33% do 93% veće od vrednosti koje odgovaraju stubovima kada su razmatrani kao sekundarni. Međutim, to nije dovoljno dobar razlog da bi se opravdala klasifikacija ovog stuba kao sekundarnog, pre svega iz ekonomskog aspekta, imajući u vidu znatno veće količine potrebne podužne armature. Smanjenje dimenzija poprečnog preseka, u ovom slučaju, nije opcija jer dovodi do prekoračenja maksimalnog koeficijenta armiranja od 4% [3]. Očigledno je da klasifikacijom ovih stubova kao sekundarnih nije

In order to provide sufficient bearing capacity of slab-to-column connection to support gravity loads during earthquakes in the elastic range, it is crucial to provide adequate amount of slab reinforcement at the supports i.e. at the slab-to-column connection. Considering the fact that the large bending moments at column ends are in equilibrium with slab bending moments, it is necessary to provide sufficient reinforcement area both at the top and the bottom since bending moments change sings under seismic action. In this particular case, the values of these moments are close to 80% of negative (hogging) bending moments due to gravity loads. As mentioned above, design and detailing of flat slabs with aim to provide sufficient ductility capacity instead of strength is not covered by Eurocode 8 [2].

3.3.2 The design results of perimeter frame in axis 1-column B1 and beam BC-1

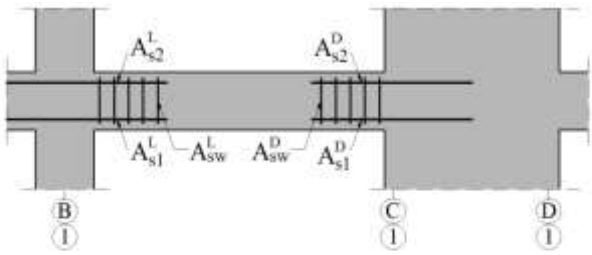
The influence of the perimeter frame stiffness on the magnitude of deformations and the shape of deformed structure is explained in Section 3.2.1. The same conclusion can be drawn from the analysis of design results of perimeter frame in axis 1, presented in Figure 7 and Table 2. As a result of the frame action and high axial forces in perimeter columns under lateral loads, the implementation of Equation (1) in design of perimeter columns, classified as secondary, leads to reduction of axial forces followed by an increase of bending moments. The design for such internal forces gives a large amount of reinforcement area, especially in lower storeys (Figure 7a). On the other hand, the normalized axial forces in primary columns are high ($\nu_{d,max}$ = 0,53) due to narrowed cross-sectional width (determined from ductility demands) which subjects them to strict rules for detailing and confinement of the concrete core. The values of mechanical volumetric ratio $\omega_{wd,m}$ are from 33% to 93% higher than those determined for secondary perimeter columns. However, this is insufficient reason for their classification as secondary, mainly for economic reasons, because of a large amount of longitudinal reinforcement. In this case, the reduction of cross-sectional dimensions is unlikely an option since it would further increase the reinforcement ratio, above the maximum value of 4% [3]. It is evident that the desired moguće postići željene rezultate i da ih je najbolje razmatrati kao deo primarnog sistema.

results could not be achieved by classification of the column B1 as secondary, and that it is reasonable to treat them as a part of primary system for supporting seismic loads.



Slika 7. Rezultati proračuna stuba B1: a) dijagram interakcije, b) poprečni presek stuba B1 kao PSE, c) poprečni presek stuba B1 kao SSE

Figure 7. Design results of the column B1: a) interaction diagram, b) cross-section of the column B1 (PSE), c) cross section of the column B1 (SSE)



Slika 8. Armatura grede BC–1

Figure 8. Reinforcement layout for the beam BC-1

Tabela 2. Rezultati proračuna grede BC-1 na etaži 5
Table 2. Design results of the beam BC-1 at floor level 5

Klasifikacija *Classification*	A_{s1}^L	A_{s2}^L	A_{s1}^D	A_{s2}^D	A_{sw}^L	A_{sw}^D	$\omega_{wd,5}$
PSE	3Ø16	2Ø20	3Ø16	3Ø20	UØ8/10	UØ8/10	0,227
SSE	6Ø20	3Ø25	5Ø20	5Ø25	UØ8/7,5	UØ8/10	0,265

Slični zaključci mogu se primeniti i na grede koje su deo fasadnih ramova. Rezultati proračuna grede BC-1 pokazuju očigledan uticaj povećanja momenata savijanja u sekundarnim seizmičkim gredama, dobijenih primenom izraza (1), koji rezultuje povećanjem armature i do tri puta. U ovom primeru, uzengije u primarnim gredama određene iz uslova kapaciteta nosivosti praktično su iste kao uzengije sekundarnih greda određene iz elastičnih uticaja (tabela 2).

Similar conclusions apply for the perimeter beams. The design results of the beam BC-1 indicate that the bending moments are significantly increased by application of Equation (1) for secondary seismic beams, which increases the required reinforcement area up to three times. Table 2 shows that, in this case, the amount of transverse reinforcement is similar to the different classification of the beam i.e. the capacity design shear forces in primary beams are almost equal to the elastic shear forces in secondary ones, calculated by an implementation of Equation (1).

4 ZAKLJUČCI

Analiza proračuna armiranobetonske konstrukcije sa sekundarnim seizmičkim elementima predstavljena u ovom radu ukazala je na prednosti i nedostatke primene ovog zanimljivog koncepta u aseizmičkom projektovanju objekata visokogradnje. Iako projektantski primamljiv, jer je dimenzionisanje i oblikovanje detalja definisano „samo” Evrokodom 2 [3], sprovođenje koncepta sekundarnih seizmičkih elemenata je zametan posao s prilično neizvesnim ishodom. Očekivana korist u vidu lakšeg proračuna kompromitovana je postupkom klasifikacije i proračuna statičkih uticaja na bazi uporedne analize dva modela. U radu je pokazano da se, u nekim slučajevima, zahtevi za armiranje sekundarnih elemenata ne razlikuju značajno od zahteva Evrokoda 8 [2] koji važe za primarne (duktilne) elemente. Takođe, pokazano je da postoje značajne posledice na ponašanje čvora stub-ploča i osiguranje ploče od proboja. Potencijalno se može očekivati smanjenje dimenzija poprečnih preseka vertikalnih elemenata ukoliko je njihov doprinos krutosti sistema relativno mali, uz „naplatu” kroz veću količinu podužne armature. Uvođenjem ovog koncepta Evrokod 8 [2] otvara mogućnosti za kompleksno tretiranje pojedinih delova konstruktivnog sistema, a tumačenje zahteva propisa svakako predstavlja istraživački i projektantski izazov.

4 CONCLUSIONS

The analysis of reinforced concrete structure with secondary seismic elements presented in this paper highlighted the advantages and disadvantages of application of this interesting concept in aseismic design of building structures. Although this concept may seem appealing to the designer because secondary members can be designed and detailed according to Eurocode 2 [3] “only”, its application in design practice is rather demanding with uncertain outcome. The expected benefit in terms of the simple design procedure is compromised by classification procedure and methods for calculation of internal forces which are based on comparative analysis of two numerical models of the same structure. The results of the analysis showed that, in some cases, design requirements for secondary elements are almost the same as Eurocode 8 requirements [2], which apply for primary (ductile) elements. Further, it is shown that the choice of this classification has significant influence on the behaviour of slab-to-column connection and on the values of punching shear stresses. The decrease in cross-sectional dimensions of secondary members can be expected if their contribution to the lateral stiffness is low. However, this will increase the amount of longitudinal reinforcement. The implementation of this concept in Eurocode 8 [2] provides the possibility for complex analysis of certain structural elements, and the interpretation of the code rules is certainly a challenge for both designers and researchers.

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REZIME

KONCEPT PRORAČUNA SEKUNDARNIH SEIZMIČKIH ELEMENATA PREMA EVROKODU 8

Ivan MILIĆEVIĆ
Ivan IGNJATOVIĆ

U radu je analiziran koncept proračuna armiranobetonske konstrukcije sa sekundarnim seizmičkim elementima prema zahtevima Evrokoda 8. Iako se doprinos krutosti ovih elemenata zanemaruje prilikom seizmičkog odgovora konstrukcije, primena ovog koncepta je kompleksna zbog niza zahteva u pogledu klasifikacije i načina proračuna statičkih uticaja. S ciljem da se istaknu i objasne specifičnosti primene, pokažu prednosti, ali i kritički razmotri upotreba opcije sekundarnih elemenata, izvršen je proračun osmoetažne armiranobetonske konstrukcije. Prikazane su dve metode za klasifikaciju sekundarnih elemenata, način proračuna uticaja u njima, kao i rezultati uporedne analize u kojoj su određeni elementi konstrukcije razmatrani kao primarni i kao sekundarni.

Ključne reči: aseizmičko projektovanje, sekundarni seizmički elementi, beton, duktilnost, Evrokod 8

SUMMARY

DESIGN CONCEPT OF SECONDARY SEISMIC ELEMENTS ACCORDING TO EUROCODE 8

Ivan MILICEVIC
Ivan IGNJATOVIC

Analysis of conceptual design of reinforced concrete (RC) structure with secondary seismic elements in compliance with Eurocode 8 is presented in this paper. The application of this concept is complex due to the requirements regarding the classification and calculation of design internal forces, although the contribution of these elements in the total structural stiffness is neglected. The basic calculations of 8-story RC structure are performed with the main goal to emphasize and explain the problems of utilization of this concept. Its advantages are clearly presented and critical analysis of application in aseismic structural design is performed. The results of comparative analysis of structural design in which some structural elements are treated as a primary or as a secondary are presented.

Key words: seismic design, secondary seismic elements, concrete, ductility, Eurocode 8

OCENA STANJA I PERSPEKTIVE ODRŽAVANJA MOSTOVSKIH KONSTRUKCIJA U GRADU NIŠU

CONDITION ASSESSMENT AND MAINTENANCE PERSPECTIVES OF BRIDGE STRUCTURES IN THE CITY OF NIS

Milan GLIGORIJEVIĆ

PREGLEDNI RAD
REVIEW PAPER
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1 UVOD

Potrebu za mostovima čovek je osećao od svog postanka. Mostovi su građevinski objekti koji saobraćajnicu prevode preko neke prepreke, što je utilitarna definicija mosta. Pri tome se podrazumeva zadovoljenje određenih ljudskih potreba, stvaranje novih oblika na zemljinoj površini, kao i objektivna težnja ka funkcionalnosti, stabilnosti, racionalnosti, unutrašnjem skladu i skladu sa okolinom. Takođe, mostovske konstrukcije istovremeno moraju biti sigurne i trajne, ali – neretko – most može biti i svojevrsno umetničko delo. Tehnički besprekorno rešen zadatak, a što podrazumeva optimalnu funkcionalnost i pouzdanost uz najmanje moguće troškove, neće biti potpun ako rezultat nije ujedno i lep most.

Mostovi su građevinski objekti koji svojom veličinom, izgledom, pojavom u prostoru, pa čak i simbolikom, vrlo često dominiraju ambijentom ili krajolikom u kojem se nalaze. Stoga, uz osnovne principe veštine projektovanja i građenja mostova (objektivnost, funkcionalnost, stabilnost, racionalnost i originalnost) – koje mora zadovoljiti svaki most, dolazi i estetika.

Može se reći da mostovi nisu konstrukcije, jer mostovi sadrže konstrukcije.

Vekovna tradicija mostogradnje i njen razvoj putem svojevrsnih oblika građenja različitih mostovskih konstrukcija, predstavlja težnju ka zadovoljenju mnogih čovekovih potreba i zahteva. U tom procesu, čovek je koristio razne materijale i sredstva koja su mu u pojedinim periodima stajala na raspolaganju, a najstariji primitivni mostovi su raznovrsne forme od srušenih

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1 INTRODUCTION

People always need bridges. A utilitarian definition of bridges is that they are structures intended to carry a road across some obstacle. This entails meeting of certain human needs, creation of new forms on the face of the earth and an objective aspiration to functionality, stability, rationality, interior harmony and harmony with the environment. In addition, bridge structures should be safe and durable, but often, it is a work of art. A technically impeccably performed design, which means optimum functionality and reliability with the least possible cost, is unlikely to be complete if it has not resulted in a beautiful bridge as well.

Bridges are civil engineering structures which very often dominate the environment or landscape where they are situated by their size, appearance in space and even by symbolism. Therefore, fundamental principles of designing and constructing bridges (objectivity, functionality, stability, rationality and originality), which should be met by any bridge, are accompanied by esthetics.

It can be said that bridges are not structures, since bridges contain structures.

Centuries long tradition of bridge-building and its development through various forms of construction of different bridge structures, represents an aspiration to satisfy various human needs and demands. In this process, man used materials and resources at his disposal in specific periods, and the oldest primitive bridges are certainly various forms ranging from felled trees to stone slabs, plant fibers and timber beams. Bridge-building history is a sound basis and instruction

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stabala ili od kamenih ploča, te od biljnih vlakana i drvenih greda. Graditeljska istorija mostogradnje dobra je pouka i podloga za stvaranje novih savremenih dela. Industrijska revolucija i njene posledice pred mostogradnju su postavljali nove zadatke, stvarajući preduslove za razvijanje novih konstrukcija i ostvarivanje još većih raspona.

Uspeh u projektovanju i građenju mostova zasniva se na dobrom poznavanju teorije konstrukcija i materijala, mašti i hrabrosti konstruktera u razvijanju novih ideja, kao i volji da se uči na tuđim greškama i sopstvenim iskustvima. Savremene metode analize sve više se primenjuju pri proučavanju i konstruisanju mostovskih konstrukcija, što daje ne samo lakše i ekonomičnije mostove, već se adekvatnim oblikovanjem konstrukcijskih detalja omogućuje mostovskim konstrukcijama pravilniji rad, a samim tim i postizanje većih raspona, te dalje domete savremenih mostova. Nova saznanja doprinose tome da – uz novu tehnologiju i tehniku – čovekova mašta postaje stvarnost.

Mostovske konstrukcije – zbog svoje individualnosti, složenosti i funkcionalnosti – imaju značajan uticaj na društvo, ukupnu razvijenost neke zemlje i sveukupni napredak čovečanstva. Mostovi imaju izuzetno veliki značaj – kako u funkcionalnom, tako i u ekonomskom pogledu. Kao specifični objekti u prostoru, koji pre svega „spajaju ljude", u sastavu su saobraćajnog sistema i planiraju se, projektuju, grade i održavaju radi obezbeđenja društvene i ekonomske dobiti, te predstavljaju objekte izuzetno velike kapitalne vrednosti. Potpuni ili privremeni prekid saobraćaja usled oštećenosti mostovskih konstrukcija, može da izazove poremećaj sa ozbiljnim posledicama za normalno funkcionisanje privrednih i drugih tokova. Generalno, mostovi su suštinski važni i za obezbeđenje i očuvanje kvaliteta života uopšte. Dakle, održivo funkcionisanje ove infrastrukturne imovine od ključne je važnosti za celokupnu državu i samu društvenu zajednicu.

2 EKSPLOATACIONI VEK MOSTOVA

Mostovske konstrukcije predstavljaju i najosetljiviji deo saobraćajne mreže. Kao objekti u spoljnoj sredini, direktno i u potpunosti su izloženi agresivnom dejstvu neposredne okoline (npr. uticaj temperature, soli, razna aero zagađenja), kao i sve oštrijim uslovima eksploatacije stalnim povećanjem osovinskog opterećenja, intenziteta i frekvencije saobraćaja. U toku eksploatacije, usled permanentnog starenja materijala i uticaja drugih raznovrsnih parametara i procesa, neminovne posledice jesu oštećenja i progresivno pogoršanje stanja, što povećava stepen dotrajalosti[1] mostovskih konstrukcija. Sve ove pojave negativno utiču na nosivost, upotrebljivost, trajnost, kao i na stepen pouzdanosti mostovskih konstrukcija, a samim tim – i na njihovu sigurnost.

[1] *Dotrajalost*: prirodan i neizbežan proces gubljenja početnih karakteristika prilikom razvoja oštećenja i pogoršanja stanja mostova, promenom svojstava konstitutivnih materijala usled starenja i uticaja svih delovanja tokom eksploatacije. *Stepen dotrajalosti* (*EC1 – Degree of deterioration*): mera napredovanja procesa koji ugrožavaju konstrukcije mostova tokom eksploatacionog veka.

for making new, contemporary structures. The Industrial revolution and its consequences, set new tasks for bridge-building engineering, creating preconditions for development of new structures and bridging larger spans.

Success in designing and building bridges is based on the theory of structures and materials, imagination and courage of designers in developing new ideas, and in the will to learn from numerous mistakes and their own experiences. Contemporary analysis methods are being increasingly used in studying and designing bridge structures, which results not only in more lightweight and more cost-effective bridges, but also in adequate formation of structural details which enables bridge structures to perform better, and thus enables achieving larger spans of contemporary bridges, which so reach further. New findings with new technology help human imagination become reality.

Bridge structures, because of their individuality, complexity and functionality have a significant impact on the society, total level of development of a country and overall progress of humanity. Bridges have an extremely high importance both in functional and economic terms. As specific structures in space which primarily "connect people" they are integral part of transportation systems and they are planed, designed, built and maintained in order to provide social and economic benefit, and represent structures with a huge capital value. Full or temporary interruption of traffic due to the damage of bridge structures can cause a disruption with severe consequences for normal functioning of economic and other activities. In general, bridges are essentially important for provision and preservation of the quality of life. Therefore, sustainable functioning of this infrastructural property is of key importance for the entire country and social community.

2 SERVICE LIFE OF BRIDGES

Bridge structures represent the most sensitive part of transportation network. As outdoor structures, they are directly and entirely exposed to the aggressive effects of the environment (temperature, salts, air pollution etc.), as well as to the increasingly severe service conditions, such as increasing axle loads, intensity and frequency of traffic. During service, due to the permanent ageing of material and impact of various parameters and processes, there is an inexorable accumulation of damage and progressive deterioration of the structural condition, which increases the dilapidation degree[1] of bridge structures. All these phenomena have a negative impact on bearing capacity, serviceability, durability and safety degree of bridge structures, and therefore on their overall safety.

[1] *Deterioration*: a natural and unavoidable process of losing the initial characteristics in the process of damage development and deterioration of bridge condition, meaning the change of the properties of constitutive materials due to aging, and effects of all impacts during service. D*egree of deterioration* (EC1 - Degree of deterioration): is the measure of the progress of processes endangering bridge structures during service life.

Predviđeni period u kome treba da budu obezbeđena navedena potrebna svojstva naziva se projektovani eksploatacioni vek objekta, a u praksi to predstavlja period od puštanja objekta u saobraćaj do njegovog zatvaranja. Osnovni razlozi koji dovode do zatvaranja mosta za saobraćaj [OECD 1992] jesu:

− Konstrukcijska (strukturalna) neadekvatnost – uzrokovana nedostacima koji dovode do opadanja stabilnosti mostovskih konstrukcija, kao i smanjenja sigurnosti objekta – najčešći je razlog zatvaranja starijih mostova. Konstrukcijski vek mosta u ovom slučaju odgovara „životnom” veku objekta koji može biti veoma dug (duži od 100 godina).

− Funkcionalna neadekvatnost – kada njegova geometrija ili nosivost ne zadovoljavaju aktuelne zahteve, te se most zatvara za saobraćaj, iako su njegove konstrukcije u dobrom stanju.

Funkcionalni vek mosta manji je od konstrukcijskog (25-50 godina), naročito u zemljama sa znatnim porastom saobraćajnog opterećenja.

Adekvatnom primenom odgovarajućih mera održavanja, popravki, sanacija i rekonstrukcija, mostovske konstrukcije mogu se održavati u dobrom stanju, sve dok ne dostignu neki optimalni vek, tzv. ekonomski vek, posle koga se stavljaju van upotrebe. Ekonomski vek definiše se kao period posle koga intervencije na održavanju, popravkama ili rekonstrukciji nisu isplative u poređenju s cenom novog mosta. Ekonomski vek jeste i cilj kome se teži u toku eksploatacionog veka nekog objekta.

Na slici 1 predstavljene su dužine trajanja pojedinih ciklusa u životu nekog mosta [OECD 1992], gde je:

The anticipated time period within the mentioned properties that should be provided is called design service life of the structure, and in practice it is the time span since commissioning of a structure (opening it for traffic) to closing down. The most common reasons necessitating closing down of a bridge for traffic [OECD 1992] are:

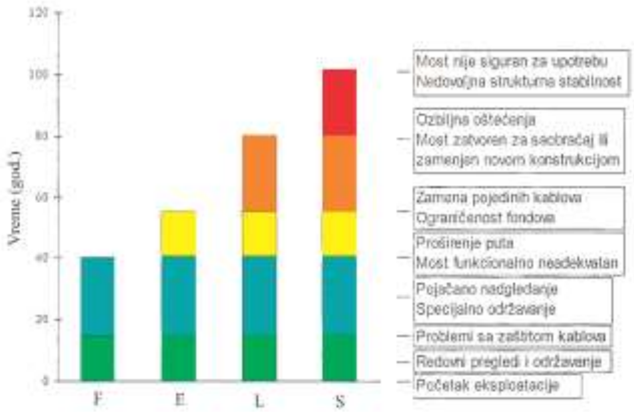
− Structural inadequacy of old bridges, caused by the deficiencies which lead to the decline of bridge structure stability, and reduced safety. The structural life of a bridge, which corresponds with the service life of the structure, can be very long (over 100 years).

− Functional inadequacy, when the bridge geometry or bearing capacity fails to meet current requirements, so it is closed down even though its structure is in good condition.

Functional life of bridge is shorter than the structural life (25 - 50 years), especially in the countries with a significant increase of traffic load.

Adequate implementation of appropriate maintenance, repairs, restoration and reconstruction, may keep the bridge structures in good conditions, until they reach an optimum life, the so called, economic life, after which they are decommissioned. The economic life is defined as a period after which the maintenance, repair and reconstruction interventions cease to be cost-effective in comparison with the cost of a new bridge. The economic life is also a goal to be reached during service life of a structure.

In figure 1, durations of certain cycles in the life of the bridge [OECD 1992] are represented, where:



F – funkcionalni vek mosta
functional life of bridge

E – ekonomski (optimalan)
economical life (optimal)

L – eksploatacioni vek
service life

S – konstrukcijski vek
structural life

Slika 1. Dužine trajanja pojedinih ciklusa eksploatacije mosta
Figure 1. Durations of individual cycles of bridge service

Eksploatacioni vek zavisi od konstrukcijske adekvatnosti (50-100 godina) i funkcionalne adekvatnosti (25-50 godina). To znači da optimalni vek zavisi i od konstrukcijskog i od funkcionalnog veka i najčešće je kraći od prvog a duži od drugog.

Konstrukcijski i funkcionalni vek nisu direktno međusobno zavisni, jer vitkije mostovske konstrukcije imaju brže pogoršanje stanja od masivnih konstrukcija. S druge strane, konstrukcijski vek može se produžiti blagovremenim i kvalitetnim održavanjem i popravkama, a takođe se može privremeno prihvatiti funkcionalna neadekvatnost u određenim situacijama. Isto tako, za

Service life depends on the structural adequacy (50 - 100 years) and functional adequacy (25 - 50 years). This means that optimum life depends both on structural and functional lives and most often it is shorter than the former and longer than the latter.

Structural and functional lives are not directly mutually dependent, because slender bridge structures deteriorate faster than massive structures. On the other hand, structural life can be prolonged with timely and quality maintenance and repairs, and also, functional inadequacy can be temporarily acceptable in certain situations. Also, for the proposes of normal traffic and normalno odvijanje saobraćaja i potrebnu sigurnost korisnika, podjednako je značajna adekvatnost – kako u konstrukcijskom, tako i u funkcionalnom pogledu. U tom smislu, treba naglasiti njihovu međusobnu povezanost i uslovljenost, te odrediti neophodno potrebne mere za istovremeno obezbeđenje i konstrukcijske i funkcionalne adekvatnosti.

Sigurnost i upotrebljivost jesu osnovna svojstva mostovskih konstrukcija da neprekidno očuvaju upotrebnu vrednost, odnosno radnu sposobnost, u toku nekog perioda (eksploatacionog veka). Uz obezbeđenje potrebne trajnosti za odgovarajući nivo pouzdanosti, ona daju osnovni imperativ korektnog oblikovanja i konstruisanja savremene populacije mostova u svetu. Međutim, nemoguće je postići apsolutnu sigurnost, večitu trajnost ili savršenu upotrebljivost, ali zato moramo težiti stanju optimuma.

Izuzetno je značajno da se uvede efikasnije održavanje postojećih mostova – kako bi se izašlo u susret javnom interesu, te je zbog toga razvijen sistem upravljanja mostovima.

3 SISTEM UPRAVLJANJA MOSTOVIMA

Danas širom sveta postoje mnogobrojne mostovske konstrukcije, različitih namena, koje su izgrađene u raznim sredinama, različitim vremenima, od raznorodnih materijala, raznovrsnih oblika, sistema, raspona i dimenzija. S druge strane, preventivnom održavanju ovih objekata – kao optimalnom rešenju iz tehničkog, organizacionog i ekonomskog aspekta – nije se posvećivala neophodna pažnja onoliko koliko je to potrebno, naročito kada je reč o našem području. Uočen je permanentni porast oštećenja na mostovima, uz enormni pad nosivosti i bezbednosti mostova, pa su čak zabeležena i rušenja nekih vrlo značajnih mostova u svetu [Pakvor, Bajić et al, 2000].

Mostovske konstrukcije dostižu svoj eksploatacioni vek kada troškovi njihovog daljeg održavanja u stanju potrebne sigurnosti, upotrebljivosti i trajnosti postanu veći od troškova održavanja koji su smatrani prihvatljivim u toku tog eksploatacionog veka.

Sadašnje okolnosti ne omogućavaju neograničena ulaganja s bilo koje tačke gledišta, pa je zajednički problem svih zemalja taj da u svoj saobraćajni sistem ulažu na najkorisniji i najefikasniji način. To je bio povod da se preduzme niz koraka radi definisanja problema, utvrđivanja postojećeg stanja, postavljanja cilja i iznalaženja načina za postizanje optimalnih rezultata putem odgovarajućih istraživanja i tehno- ekonomskih analiza. Ispravnim ulaganjem, smanjuju se ukupni troškovi i omogućuje se očuvanje investicija za duži period. Stoga, veoma je značajna potreba da se uvede efikasnije održavanje postojećih mostova, kako bi se izašlo u susret javnom interesu.

S jedne strane, mostovi su podložni pogoršanju stanja usled starenja konstrukcijskih elemenata i konstitutivnih materijala; s druge strane, njihovo duže isključivanje iz funkcije, radi sanacije, rekonstrukcije ili zamene, može da izazove veoma neprijatne poremećaje saobraćaja na putevima i prugama. Lošem stanju mostova znatno doprinose i veoma loša organizacija i iskorišćenost postojećih kapaciteta, kao i nedovoljno iskustvo u upravljanju.

U skladu s tim, potrebno je raspolagati efikasnim

necessary safety of users, structural and functional adequacies are equally important. In this sense, one must emphasize their mutual interdependencies and specify necessary measures for simultaneous insurance of both structural and functional adequacies.

Safety and serviceability are fundamental properties of bridge structures to be preserved, so that the bridges would be serviceable, that is, operable within a certain time period (service life). They provide the necessary durability at a corresponding level of safety and they are the goal of proper designing of contemporary bridges in the world. However, it is impossible to achieve absolute safety, eternal durability or perfect serviceability, so an optimum solution must be sought.

It is vitally important to introduce a more efficient maintenance of the existing bridges in order to satisfy the public interest, and for this reason, the bridge management system was developed.

3 BRIDGE MANAGEMENT SYSTEM

There is a large number of bridge structures worldwide, having various uses, built in most diverse environments, times, materials, forms, systems, spans and dimensions. On the other hand, preventative maintenance of these structures, as an optimum solution from the technical, organizational and economical aspects, was not paid due attention, especially in our parts. A permanent increase of damage on the bridges was detected, followed by the enormous decrease of bearing capacity and safety, and there were even collapses of some very important bridges in the world [Pakvor, Bajić et al, 2000].

Bridge structures reach their service life when the cost of their further maintenance in the condition of required safety, serviceability and durability exceed the maintenance cost considered acceptable in the course of the service life.

Present day circumstances hinder unlimited investments of any kind, so the common problem off all the countries is how to invest in their transportation system in the most useful and efficient way. It was a motivation to take a number of steps in the direction of defining the problem, determining the current condition, setting a goal and finding ways to obtain optimum results using appropriate research and techno-economical analyses. Correct investment results in reduction of total costs and provides preservation of investments for a long period of time. For this reason, it is crucial to introduce a more efficient maintenance of existing bridges, in order to satisfy the public interest.

On one hand, bridges are susceptible to deterioration due to aging of structural elements and constitutive materials, and on the other hand, their long decommission for the purposes of restoration, reconstruction or replacement can cause very adverse disturbances of traffic on the roads and railways. Poor condition of bridges is considerably contributed by very poor organization and efficiency of the existing capacity, as well as the inexperience in management.

Accordingly, it is necessary to possess an efficient bridge management system, and to use it to reach sistemom upravljanja mostovima i na osnovu njega doneti razumne i utemeljene odluke o raspodeli sredstava, s tačke gledišta očuvanja saobraćajnih pravaca i okoline, uz poštovanje postojećih okolnosti. Na značaj primene sistema upravljanja mostovima direktno ukazuju nedostaci i oštećenja mostovskih konstrukcija, što u prvom redu jesu posledice neblagovremenog i neadekvatnog održavanja. Nedostaci nastaju prilikom planiranja, projektovanja i građenja, a oštećenja nastaju tokom upotrebe mostovskih konstrukcija. Ovakav problem zahtevao je efikasan sistem upravljanja mostovima koji će dati razumne i utemeljene odluke o raspodeli sredstava u uslovima veoma ograničenih fondova i budžeta.

Savremeni procesi planiranja, projektovanja, izgradnje i eksploatacije mostova danas se ne mogu zamisliti bez adekvatnog sistema upravljanja mostovima.

U takvoj situaciji neophodno je primeniti globalni pristup upravljanja mostovima, te planiranjem i koordinacijom relevantnih aktivnosti povećati efikasnost upravljanja tokom celokupnog eksploatacionog veka.

Upravljanje mostovima jeste proces kojim se nadgledaju, prate, održavaju i popravljaju uočena pogoršanja stanja mostovskih konstrukcija, s raspoloživim sredstvima u toku proračunskog upotrebnog veka. Proračunski upotrebni vek je pretpostavljeno razdoblje korišćenja mostovskih konstrukcija, uz redovno održavanje, ali bez velikih popravki.

Problematika upravljanja mostovima uključuje celokupni eksploatacioni (životni) vek mostovskih konstrukcija, počev od koncepta i osnovnih prethodnih studija, preko procesa projektovanja, građenja, eksploatacije i gazdovanja, tj. održavanja, tokom adaptacije, sanacije, rekonstrukcije i na kraju njihove zamene ili uklanjanja. Stoga, upravljanje kao poslovni proces zahteva multidisciplinarni pristup i poznavanje svih tehničkih i drugih netehničkih disciplina. Krajnji cilj jeste optimalno zadovoljenje filozofije trajnosti, tj. postići maksimum učinka s minimumom uloženih sredstava. Upravo zato, upravljanje mostovima i njihovo adekvatno održavanje jeste perspektivan posao u savremenom građevinarstvu.

Strategija razvoja sistema upravljanja mostovima u svetu bazirana je na metodologiji za razvoj sistema optimizacije i korišćenja resursa u procesu upravljanja i održavanja mostova. To uključuje stanje mostovskih konstrukcija, njihov kapacitet nosivosti, stepen oštećenja odnosno dotrajalost konstitutivnih elemenata mostovskih konstrukcija, saobraćajne efekte, kao i popravke, sanacije i rekonstrukcije.

Koncept upravljanja mostovima počeo je da se razvija u svetu ne tako davno, da bi se izašlo u susret svim tim rastućim potrebama. Prvi sistemi upravljanja mostovima u svetu počeli su da se razvijaju od 1970. godine [Gligorijević, 2016].

Jedna od prvih zemalja koja je uvela sistematsko, dobro isplanirano i organizovano istraživanje u sferi upravljanja mostovima jeste Amerika. U SAD, od početka izgradnje sistema mreže međudržavnih autoputeva, sredinom pedesetih godina prošlog veka, federalna sredstva izdvajana su samo za novogradnju, proširenje i jačanje infrastrukture. Shodno tome, aktivnosti na održavanju, revitalizaciji i obnavljanju postojeće infrastrukture bile su prilično ograničene ili odlagane od strane državnih organa. Ovakve zaostale potrebe ulaganja u postojeću infrastrukturu SAD, a koje

sensible and well-grounded decisions of allocation of resources, from the standpoint of preservation of traffic communications and the environment, with consideration of the existing circumstances. The importance of implementation of bridge management system is directly reflected through the deficiencies and damage of bridge structures, primarily due to untimely and inadequate maintenance. Deficiencies are created during planning, designing and construction, and damage occurs during the usage of bridge structures. Such problem requires an efficient bridge management system which assists in making sensible and well-grounded decisions about resources allocation in the conditions of very limited funds and budgets.

Contemporary processes of planning, designing, construction and service of bridges cannot be conceived nowadays without an adequate bridge management system. Proper investment, leads to reduction of total cost and ensures preservation of investments for a longer time period.

In such situation, it is necessary to implement a global approach to bridge management, and by planning and coordinating relevant activities, to increase management efficiency during entire service life.

Bridge management is a process, used to supervise, monitor, maintain and repair detected deterioration of bridge structures, using available resources during design service life. Design service life is a predicted period of usage of bridge structures, with regular maintenance but without any considerable repair.

The subject of bridge management includes entire service life of bridge structures, starting from the concept and basic preliminary studies, through designing, construction, service and maintenance processes (adaptation, restoration, reconstruction) to the final replacement or removal of bridge structures. For these reasons, management as a business process requires a multidisciplinary approach and knowledge of all technical and other non-technical disciplines. The ultimate goal is optimum satisfaction of durability philosophy, i.e. achieving maximum effects with a minimum of invested resources. For this reason, bridge management and adequate maintenance is a perspective business in contemporary civil engineering.

Strategy of development of bridge management system in the world is based on methodology for development of optimization system and usage of resources in the process of management and maintenance of bridges. This includes condition of bridge structures, their bearing capacity, damage degree, that is, deterioration of constitutive elements of bridge structures, traffic effects, as well as repairs, restoration and reconstructions.

The bridge management concept started to develop recently in the world, in order to meet the growing demand. The first bridge management systems in the world started to develop since 1970 [Gligorijević, 2016].

One of the first countries to introduce a systemic, well-planned and organized research in bridge management domain is the USA. In the USA, since the beginning of construction of interstate highway network by the middle of 1950s, the Federal budget was allocated only for new construction, expansion and strengthening of infrastructure. Accordingly, the activities on maintenance, revitalization and renewal of the nisu dobile dovoljnu pažnju, doprinele su permanentnom pogoršanju stanja svih konstrukcijskih elemenata objekata postojeće infrastrukture SAD.

Prvi programi za upravljanje mostovima u SAD datiraju još iz ranih sedamdesetih godina XX veka. Posledice rušenja više mostova u SAD, prvo *Silver* mosta 1967. godine [15], a potom i drugih kapitalnih mostova, kao i sve veći jaz između raspoloživih sredstava i potreba nacionalne mreže mostova Amerike, uticali su na stimulisanje povećanog obima istraživanja ove problematike i na razvoj sistema upravljanja mostovima sredinom osamdesetih godina. Ubrzo nakon toga, 1991. godine, intermodalni zakon o efikasnosti transporta u SAD nalaže državama potrebu da razvijaju i implementiraju sisteme upravljanja mostovima. Sistemi upravljanja mostovima u većini država SAD razvijeni su sredinom devedesetih godina XX veka [Small,1999].

Danas, američke državne transportne agencije uspostavile su programe inspekcija mostova i većina njih je implementirana u savremen sistem upravljanja mostovima *AASHTOWare Bridge Management* (ranije *Pontis*). U svetu, poslednjih godina znatno se povećava broj država koje su razvile i koje koriste sistem upravljanja mostovima.

Savremen sistem upravljanja mostovima sadrži procenu stanja mostova, modeliranje budućeg pogoršanja stanja i ponašanja, kao i module za donošenje odluka u pogledu toga kako najekonomičnije održavati, popravljati i obnavljati mostovske konstrukcije.

Očuvanje bitnih svojstava mostovskih konstrukcija u toku njihovog životnog veka predstavlja permanentan zadatak sistema upravljanja mostovima. Prema podacima iz bogate prakse upravljanja razvijenih zemalja, plansko održavanje kroz eksploatacioni (životni) vek konstrukcija mosta zahteva ulaganja približno 2% do 3% investicione vrednosti godišnje.

U Srbiji, generalno posmatrano, usled dugogodišnjeg nedovoljnog ulaganja u održavanje i rekonstrukciju mostova, stanje mostova može se oceniti kao neprihvatljivo, naročito kada je u pitanju njihova starost. Redovno održavanje uglavnom je primitivno, tako da se ubrzava starenje konstrukcijskih elemenata i pogoršava stanje mostova, a veće popravke i sanacije gotovo su jedini vid aktivnosti i obavljaju se u bezizlaznim situacijama.

Osnovni savremeni Sistem upravljanja mostovima u Srbiji uveden je 1986. godine, kao originalan i za to vreme izuzetno moderan sistem [Bebić, 1986]. Za potrebe kvalitetnog upravljanja mostovima i primene ovog sistema na teritoriji Republike Srbije formirana je elektronska baza podataka o mostovima (BPM), koja u svakom trenutku pruža sve potrebne informacije o traženim mostovskim konstrukcijama, na osnovu urađenih inspekcijskih pregleda. Cilj formiranja baze podataka o mostovima bio je da se prikupe raspoložive informacije o mostovskim konstrukcijama radi ustanovljavanja prioriteta u održavanju mostova i razvoja sistema upravljanja mostovima u Srbiji.

Uspostavljanje prioriteta u aktivnostima održavanja mostovskih konstrukcija treba shvatiti kao odgovor na neadekvatna finansijska sredstva koja su izdvajana za održavanje mostova u uslovima opšte političke i ekonomske situacije u Srbiji poslednjih decenija. Od 1991. godine, primenjivana je verzija SR - 02, koja sadrži inventarske podatke i podatke o stanju mostova u trenutku pregleda. Da bi se olakšao rad na unošenju

existing infrastructure were quite limited or postponed by the state authorities. Such absence of investment into the existing infrastructure of the USA, lacking adequate attention, led to permanent deterioration of all the structural elements of the existing infrastructure in the USA.

The fist bridge management programs in the USA date back to the early 1970’s. Collapse of several bridges in the USA, first the "*Silver*" bridge in 1967 [15], and then of other capital bridges, and the growing gap between the available resources and needs of national network of the USA bridges stimulated increased research of this issue and gave rise to the development of bridge management system by the mid 1980’s. Soon after that, in 1991, the intermodal law on efficiency of transport in the USA obliges the states to develop and implement the bridge management systems. Bridge management systems in the majority of the USA states were developed in the mid 1990’s [Small,1999].

Nowadays, state transport agencies in the USA, established bridge inspection programs, and most of them are implemented into the contemporary bridge management system *AASHTOWare Bridge Management* (earlier *Pontis*). In the world, recently, the number of states which developed or are developing bridge management system is increasing considerably.

Contemporary bridge management system contains bridge condition assessment, modelling of the future deterioration and behaviour and modules for decision making about most cost-efficient ways of maintenance, repair and renewal of bridge structures.

Preservation of the important properties of bridge structures during their service life represents a permanent task of bridge management systems. According to the data from the extensive experience of management in the developed countries, planned maintenance during the service life of bridge structures requires investments of approximately 2% to 3% of the investment value annually.

In Serbia, generally speaking, due to the long lasting lack of investment into maintenance and reconstruction of bridges, the bridge condition can be evaluated as unacceptable, especially in terms of their age. Regular maintenance is mostly primitive, which accelerates ageing of structural elements and deteriorates the bridge condition, and large repairs and restorations are almost the only form of activities, and they are performed in the situations when they remain the only alternative to closing down the bridge.

The basic contemporary Bridge management system in Serbia was introduced in 1986 as original, and it was a modern system for the time [Bebić, 1986]. For the needs of quality management of bridges and implementation of this system in the territory of the Republic of Serbia, an electronic database of bridges was formed (BPM), which at any moment provided all necessary information about the researched bridge structures on the basis of performed inspections. Formation of the data base had a goal to collect the available information about bridge structures in order to establish a priority in bridge maintenance and development of bridge management system in Serbia.

Establishing priorities in the bridge structure maintenance activities should be understood as a response to inadequate finances which were allocated for bridge maintenance in the general political and podataka, urađena su detaljna korisnička uputstva.

Prateći dalji razvoj računarske tehnike, baza podataka o mostovima prošla je više faza razvoja i od 1999. godine u upotrebi je verzija SR - 03, a od 2003. godine koristi se verzija koja radi pod MS ACCESS-om.

Međutim, praktična primena Sistema upravljanja mostovima u Srbiji, tokom niza godina, pokazala je određene nelogičnosti u dobijenim listama prioriteta.

U svojoj doktorskoj disertaciji [Gligorijević, 2016], autor ovoga rada dao je novi predlog, na osnovu optimizovanog kriterijuma vrednovanja prioriteta. Time su otklonjene uočene nelogičnosti našeg aktuelnog sistema upravljanja mostovima i ostvareno je značajno poboljšanje efikasnosti određivanja liste prioriteta.

Predloženom metodologijom, a na osnovu rezultata sopstvenog višedecenijskog monitoringa mostova, ocenjeno je trenutno stanje i data je prognoza budućeg stanja mostovskih konstrukcija na gradskim saobraćajnicama Niša, prezentovana u ovom radu.

economic conditions in Serbia in the last several decades. Since 1991, the version SR - 02, was used, which contains inventory data and data on the condition of bridges at the moment of inspection. In order to make the work on data input easier, detailed user instructions were made.

Following the further development of computer technology, the database on bridges passed through several development phases, and since 1999, the version SR – 03 was used, and since 2003, there has been a version working under MS ACCESS.

However, practical application of the Bridge management system in Serbia during a long time exhibited certain illogical issues in the obtained lists of priority activities.

In his doctoral dissertation [Gligorijević, 2016], the author of this paper provided a new proposal based on the optimized criterion of priority evaluation. This removed the detected illogical issues of our current bridge management system and provided a considerable improvement of efficiency in determining the priority list.

The proposed methodology, based on the results of his own bridge monitoring lasting for several decades, is used to provide bridge condition assessment and maintenance perspectives of bridges in the city of Nis, that are presented in this paper.

4 OCENA STANJA MOSTOVA U GRADU NIŠU

Za potrebe istraživanja u doktorskoj disertaciji, autor ovoga rada 1997. godine, nakon izrade Elaborata o stanju mostova [Gligorijević et al, 1997], formirao je bazu podataka za mostovske konstrukcije na gradskim saobraćajnicama Niša. Od 1998. godine prati pogoršanje stanja ovih mostova na osnovu izvršenih periodičnih kontrolnih, redovnih i glavnih inspekcijskih pregleda mostovskih konstrukcija u gradu Nišu. Značaj ovih mostova zahtevao je takav pristup koji podjednako obezbeđuje i blagovremeno, ali i racionalno održavanje i popravke, odnosno ojačanje i/ili zamenu ovih objekata.

Posle svakog inspekcijskog pregleda, metodologijom predloženom u doktorskoj disertaciji [Gligorijević, 2016], sračunata je ocena stanja svih elemenata mostova iz baze podataka, tj. njihov „rejting", na osnovu kojeg su formirane rang-liste prioritetnih aktivnosti neophodnih intervencija. Analizom dobijenih rang-lista prioriteta, uočeno je da u njima nema bitnijih promena između dva redovna inspekcijska pregleda (interval od dve godine), ako u tom periodu nije bilo aktivnosti održavanja. Kako su sanacije i popravke nekih ugroženih mostova na gradskim saobraćajnicama Niša izvršene u periodu od 1999. godine do 2002. godine, odnosno pre drugog glavnog pregleda mostova iz 2003. godine, a kasnije nisu urađene, u ovom radu je prikazan rejting mostova nakon glavnih inspekcijskih pregleda (interval od šest godina).

Globalno stanje mostovkih konstrukcija, nakon inspekcijskog pregleda 1997. godine, prikazano je na slici 2.

Prvi na rang-listi, most „Mramor", imao je izuzetno visok rejting i zahtevao je hitnu sanaciju, jer je bila ugrožena njegova stabilnost usled erozije rečnog korita u zoni srednjeg stuba S2, na šta je već ukazano u elaboratu iz 1997. godine. Takođe, elementi gornjeg

4 ASSESSMENT OF BRIDGE CONDITION IN THE CITY OF NIŠ

For the needs of research in the doctoral dissertation, the author of this paper formed a database for bridge structures in the city transport of Niš in 1997, after producing the Analysis of bridge condition [Gligorijević et al, 1997]. Since 1998, he has been monitoring deterioration of the condition of those bridges on the basis of performed periodical control, as well as regular and main inspections of bridge structures in the city of Niš. Importance of these bridges demanded an approach which provided an equally timely and cost-effective maintenance and repair, that was strengthening and/or replacing these structures.

After every inspection, using methodology proposed in the doctoral dissertation [Gligorijević, 2016], condition of all the bridge elements in the data base was assessed, i.e. their rating was made, which was used for making rank-lists of priority activities and necessary interventions. The analysis of the obtained rank-lists of priorities showed that there were no considerable changes between two regular inspections (2 year interval), if there were no maintenance activities in that period. Since restorations and repairs of some affected bridges on the main traffic routes of Niš were performed in the 1999-2002 period, that is, before the second main inspection of 2003 and they were not performed later; this paper shows the rating of the bridges after the main inspections. (6 years interval).

Global condition of bridge structures after inspection of 1997 is presented in figure 2.

The first on the rank list, the "Mramor", bridge had an extremely high rating, and demanded urgent restoration, because its stability was at risk due to the erosion of the riverbed in the zone of the medium pier S2, which was indicated in the Analysis of 1997. Also, the superstructure elements were badly damaged [Gligorijević et stroja mosta bili su jako oštećeni [Gligorijević i dr, 2002], te je most hitno saniran 2002. godine.

al, 2002], so the bridge underwent emergency restoration in 2002.

Slika 2. Broj i procenat mostova za svaki tip održavanja nakon pregleda 1997. godine
Figure 2. Number and percentage of bridges for each type of maintenance after 1997 inspection

Most „Mladosti”, nakon NATO agresije 1999. godine i rušenja mostova na autoputu u okolini Niša, primio je celokupni saobraćaj koridora X, što je dodatno pogoršalo stanje mosta [Gligorijević, 2002], pa je zbog ugrožene stabilnosti urgentno rekonstruisan i ojačan prethodno napregnutim karbonskim trakama 2001. godine [Gligorijević, 2007]. Most u ulici „12. Februar”, bombardovan je 1999. godine i obnovljen iste godine [Gligorijević, 2002].

Na taj način, najugroženiji mostovi s vrha ove rang-liste bili su popravljeni do sledećeg glavnog pregleda mostova na gradskim saobraćajnicama Niša. Može se reći da je splet okolnosti značajno uticao na to da se najviše oštećeni mostovi poprave i da se ispoštuje data rang-lista prioriteta.

Most kod Vrežinskog bazena, koji se nalazi na perifernoj gradskoj saobraćajnici s manjim intenzitetom saobraćaja, ostavljen je da čeka svoju popravku u narednom periodu, dok Tvrđavski most – kao četvrti na ovoj rang-listi – svojim rejtingom skreće na sebe pažnju, jer je u samom centru grada, neposredno ispred ulaza u Nišku tvrđavu.

Globalno stanje mostovkih konstrukcija, nakon inspekcijskog pregleda 2003. godine, s novoizgrađenim mostovima iz 2005. godine, prikazano je na slici 3.

After NATO aggression of 1999 and destruction of bridges on the highway near Niš, the “Mladosti” bridge received the entire corridor X traffic, which additionally aggravated the its condition [Gligorijević, 2002]. Therefore, due to the endangered stability it was urgently reconstructed and strengthened using pre-stressed carbon strips in 2001 [Gligorijević, 2007]. The bridge in “12. Februar” street was bombed in 1999 and renewed in the same year [Gligorijević, 2002].

In this way, the most critical bridges from the top of this rank-list were repaired until the next main inspection of bridges on the main transport routes of Niš. It can be said that owing to a turn in events the bridges that were most damaged were repaired, and the rank-list of priorities was observed.

The bridge next to the Vrežina swimming pool, which is on the peripheral city route with a low intensity of traffic, was left to wait for its repair in the next period, while the Fort bridge, as the fourth on this rank-list draws attention to itself by its rating, since it is located in the centre of the city, immediately in front of the gate of the Fort.

Global condition of bridge structures after inspection of 2003 with newly constructed bridges of 2005 is presented in figure 3.

Slika 3. Broj i procenat mostova za svaki tip održavanja nakon pregleda 2003. godine
Figure 3. Number and percentage of bridges for each type of maintenance after 2003 inspection

Na vrhu rang-liste iz 2003. godine nalaze se dva izuzetno značajna mosta – već spomenuti Tvrđavski most u centru grada i most kod Ćele-kule. Međutim, u narednom periodu nisu preduzete nikakve aktivnosti radi popravke i održavanja mostova u Nišu.

Globalno stanje mostovskih konstrukcija nakon inspekcijskog pregleda 2009. godine, prikazano je na slici 4.

On top of the rank-list of 2003, there are two extremely important bridges, the already mentioned Fort bridge in the centre of the city and the bridge near the “Skull tower“. However, in the following period, no activities including repair and maintenance of bridges in Niš were undertaken.

Global condition of bridge structures after inspection in 2009 is presented in figure 4.

Slika 4. Broj i procenat mostova za svaki tip održavanja nakon pregleda 2009. godine
Figure 4. Number and percentage of bridges for each type of maintenance after 2009 inspection

Napredovanje procesa pogoršanja stanja Tvrđavskog mosta, naročito montažnih adheziono prethodno napregnutih betonskih talpi pešačkih staza, umnogome je povećalo rejting čeličnog mosta ispred Tvrđave u Nišu, tako da ovaj most ostaje na prvom mestu rang liste prioriteta 2009. godine. Prvo incidentno urušavanje dela pešačke staze desilo se juna 2008. godine [Gligorijević, 2009], a koje je „sanirano” zamenom oštećene adheziono prethodno napregnute talpe armiranobetonskom.

I pored ove iznuđene intervencije, čelični most u centru Niša svojim izuzetno visokim rejtingom i prvim mestom na rang-listi prioriteta, nakon izvršenog glavnog pregleda mostova 2009. godine, zahtevao je hitnu i ozbiljnu popravku. Nažalost, to se nije desilo, te 2014. godine dolazi do novog, znatno većeg urušavanja uzvodne pešačke staze. Krajem 2014. godine i početkom 2015. godine montažne talpe zamenjene su livenim betonom armiranim čeličnim vlaknima. Već u proleće 2015. godine, novoizgrađena pešačka staza pokazala je brojne prsline. Iako su uočena oštećenja, glavnim inspekcijskim pregledom 2015. godine nove pešačke staze ocenjene su najboljom ocenom („dobro”), ali je pogoršanje stanja ostalih nosećih elemenata ovog mosta uticalo da njegov rejting 2015. godine bude izuzetno visok i da most ostane na prvom mestu rang-liste prioriteta.

U periodu od 2009. godine do 2015. godine, nije bilo nikakvih drugih intervencija koje bi popravile stanje na ostalim mostovskim konstrukcijama iz baze podataka mostova u gradu Nišu.

Globalno stanje mostovkih konstrukcija, nakon inspekcijskog pregleda 2015. godine, prikazano je na slici 5.

I pored izvršenih radova na zameni betonskih ploča pešačkih staza na mostu ispred ulaza u Nišku tvrđavu, on ostaje na prvom mestu rang-liste prioriteta nakon pregleda 2015. godine, jer ima izuzetno veliki rejting (u klasi rejtinga stanja mosta 6), što zahteva hitnu sanaciju nosećih konstrukcijskih elemenata, odnosno tip održavanja 6.

Advance of deterioration of the Fort bridge, and especially of pre-fabricated pre-stressed concrete structures of pedestrian sidewalks, significantly increased the rating of the steel bridge in front of the Fort in Niš, so this bridge remained on the top of the rank-list of priorities of 2009. The first incidental collapse of a part of pedestrian sidewalk occurred in June 2008. [Gligorijević, 2009], which was “repaired” by replacing the adhesion prestressed element with an reinforced-concrete one.

Notwithstanding this forced intervention, the steel bridge in the centre of Niš, with its extremely high rating and the first place on the rank-list of priorities after the main inspection of bridges of 2009 required an urgent and serious repair. Unfortunately this did not happen, and in 2014 there was a new, considerably larger destruction of the upstream pedestrian sidewalk. By the end of 2014 and beginning of 2015, prefabricated elements were replaced with steel fibber reinforced cast concrete. As early as in spring 2015, newly built sidewalk exhibited numerous cracks. Even though the damage was detected, the main inspection of 2015 evaluated the new sidewalks with the best mark – “good”, but the deterioration of other bearing elements of this bridge made its rating of 2015 extremely high, and the bridge remained at the first place of the rank-list of priorities.

In the from 2009 to 2015 there were no other interventions which would improve the condition of the remaining bridge structures listed in the database of bridges in the city of Niš.

Global condition of bridge structures after inspection of 2015 is presented in figure 5.

Notwithstanding the performed works on replacement of concrete slabs of pedestrian sidewalks on the bridge opposite the entrance to the Niš fortress, it remains on the first place of the rank-list of priorities after the inspection of 2015 because it has an extremely high rating (in the class of bridge condition rating 6), which calls for an urgent repair of bearing structural elements, i.e. maintenance type 6.

Slika 5. Broj i procenat mostova za svaki tip održavanja nakon pregleda 2015. godine
Figure 5. Number and percentage of bridges for each type of maintenance after 2015 inspection

Drugo mesto na rang-listi prioriteta u 2015. godini zadržao je most kod Ćele-kule, ali sa znatno većim rejtingom, čime ulazi u klasu rejtinga stanja mosta 6 i takođe zahteva „hitnu sanaciju" nosećih konstrukcijskih elemenata, odnosno tip održavanja 6. Treba naglasiti i to da je ovaj most na rang-listama prioriteta bio na drugom mestu još nakon pregleda 2009. godine, ali od tada nisu preduzete nikakve aktivnosti da se most popravi.

Na trećem mestu, na rang-listama prioriteta 1997. godine, 2009. godine i 2015. godine, nalazi se most kod Vrežinskog bazena, koji je sagrađen u „Karpoš" sistemu i ima ugroženu stabilnost zbog velikog oštećenja donjeg stroja mosta.

Često se govori da je upravljanje mostovima veština iznalaženja najboljeg odgovora na pitanja: šta? (*what*?), gde? (*where*?), kada? (*when*?) i pošto? (*how much*?). Odgovore na prva dva pitanja daje baza podataka inventara mostova i zapisnik o pregledu mosta, ako je registrovao štete. Odgovor na druga dva pitanja daje sistem upravljanja, primenom inženjerskog prosuđivanja (*engineering judgment*) i ekonomske analize (*economic considerations*).

Formirane rang-liste prioriteta daju najbolje odgovore na prva tri osnovna pitanja sistema upravljanja mostovima: šta? (*what*?), gde? (*where*?) i kada? (*when*?), jer su pregledi svih mostovskih konstrukcija sa ovih rang-lista vršeni redovno, na osnovu kojih su određene klase rejtinga stanja za iste vremenske preseke.

5 MODELI POGORŠANJA STANJA MOSTOVSKIH KONSTRUKCIJA

Pogoršanje stanja elemenata mostovskih konstrukcija jeste proces smanjenja njihovih svojstava pri normalnim uslovima rada. Pogoršanje stanja pokazuje složene fenomene fizičkih i hemijskih promena koje se dešavaju u različitim komponentama mostovskih konstrukcija. Svaki element mosta ima svoju jedinstvenu stopu pogoršanja, što čini problem komplikovanijim. Pouzdano i precizno prognoziranje brzine napredovanja procesa pogoršanja stanja, za svaki element mostovskih konstrukcija, od presudnog je značaja za uspeh bilo kog sistema upravljanja mostovima.

Za prognoziranje i predviđanje stanja konstitutivnih elemenata mostova u nekom budućem trenutku, neophodan je teorijski model procesa pogoršanja stanja

The second place on the rank-list of priorities in 2015 was kept by the bridge near the "Skull tower" but with a considerably higher rating which was also bridge condition class 6 and also required "urgent restoration" of bearing structural elements, maintenance type 6. It should be pointed out that this bridge was on the second place after inspection of 2009, but no activities were undertaken since then with the purpose of repairing the bridge.

In the third position, on the rank-lists of priorities of 1997, 2009 and 2015 was the bridge near the Vrežina swimming pool, which was built in "Karpoš" system and whose stability is endangered because of the extensive damage of the bridge substructure.

It is often said that the bridge management is a skill of finding the best answers to the following questions: *what*, *where*, *when*? and *how much*? The answers to the first two questions are provided by the database of bridge inventory and the bridge inspection report, if damage was detected. The answer to other two questions is provided by the management system, by implementing *engineering judgment* and *economic considerations*.

The formed rank-lists of priorities provide the best answers to three basic questions of the bridge management system: *what*? *where*? and *when*?, because inspections of all bridge structures from these rank-lists were performed regularly, on which basis the condition rating classes for the same time points were determined.

5 MODELS OF BRIDGE STRUCTURE CONDITION DETERIORATION

Deterioration of bridge structure element condition is a process of decline of their properties under normal operational conditions. The condition deterioration process shows complex phenomena of physical and chemical changes which occur in various components of bridge structures. Every bridge element has its characteristic deterioration rate which makes the problem even more complicated. Reliable and precise forecast of the deterioration process progress, for each element of bridge structures shows critical importance for success of any bridge management system.

Forecast and prediction of condition of constitutive elements of bridges in some future moment requires a theoretical model of deterioration process – aging of the

- starenja mosta. Modeli pogoršanja stanja elemenata mostovskih konstrukcija uvedeni su krajem osamdesetih godina XX veka, kako bi se predvidelo buduće stanje infrastrukturne imovine u funkciji očekivanog nivoa usluge. U studiji sprovedenoj u centru transportnih sistema (*Transportation Systems Center - TSC*) [Busa et al, 1985] u Kembridžu, ispitivani su faktori koji utiču na pogoršanje stanja elemenata jednog mosta. Studija je zaključila da najuticajniji faktori koji utiču na pogoršanje stanja mostovskih konstrukcija jesu starost, intenzitet saobraćajnog opterećenja, uslovi okolne sredine, parametri korišćeni pri projektovanju i proračunu konstrukcijskih elemenata mosta, kao i kvalitet korišćenih materijala i kvalitet same izgradnje. Prema izveštaju FHWA [USDOT/FHWA, 1989], većina istraživanja ukazala je na to da indeksi pogoršanja stanja pokazuju značajne promene u prvih nekoliko godina eksploatacije, a da kasnije imaju tendenciju da predvide sporiji pad rejtinga stanja mostovskih konstrukcija.

Mogućnost prognoziranja procesa pogoršanja stanja osnovnih tehničkih i funkcionalnih karakteristika mostova, kao i procena preostalog servisnog veka, izuzetno su važni ulazni podaci za sistem upravljanja mostovima. Modeli kojima se prognozira pogoršanje stanja mostovskih konstrukcija tokom vremena od ključnog su značaja za efikasno planiranje održavanja. To naročito dolazi do izražaja u procesu optimizacije i planiranja potrebnih aktivnosti i odgovarajućih finansijskih sredstava.

Modeliranje procesa pogoršanja stanja veoma je kompleksno i složeno, jer je mnogo faktora koji utiču na ovu pojavu, zbog čega se u mnogim zemljama posvećuje velika pažnja tom problemu. Različite tehnike primenjuju se za prognoziranje pogoršanja stanja mostovskih konstrukcija. U principu, modeli za predviđanje pogoršanja stanja mostova mogu se svrstati u četiri kategorije: modeli fizičko- hemijskih procesa pogoršanja stanja, deterministički modeli, stohastički modeli i modeli veštačke inteligencije.

6 PROGNOZA BUDUĆEG STANJA

Kada postoji ažurna baza podataka sa inspekcijskih pregleda mostovskih konstrukcija, može se definisati vreme zadržavanja mostova u pojedinom stanju. Determinističkim modelom, na osnovu vremena zadržavanja mostova u određenoj klasi rejtinga stanja, može se odrediti trajektorija pogoršanja stanja mostovskih konstrukcija i ustanoviti klasa rejtinga stanja mostova u budućnosti. Nove mostovske konstrukcije polaze od klase rejtinga stanja „1" i sukcesivno prolaze kroz svaku narednu klasu rejtinga stanja, dok ne dostignu najgore stanje „6". Na osnovu redovnih inspekcijskih pregleda promene stanja mostova u Nišu tokom vremena, ustanovljeno je vreme zadržavanja mostova u pojedinoj klasi rejtinga i dobijeno najkraće vreme potrebno da nov most stigne do nedopustivog rejtinga stanja „6", koje iznosi 42 godine, ukoliko se nikakva intervencija ne preduzima i ne ulaže u održavanje i popravke. Podrazumeva se da mostovske konstrukcije ne smeju da borave u klasi rejtinga stanja „6".

Postavlja se ključno pitanje: koja društvena zajednica može sebi da dopusti da „zamenjuje" mostove svakih četrdesetak godina?

Pogoršanje stanja mostovskih konstrukcija na gradskim saobraćajnicama Niša analizirano je i prime-

bridge. The models of deterioration of condition of bridge structures were introduced by the end of 1980's in order to predict the future condition of infrastructural property in the function of the expected level of service. In the study conducted in the *Transportation Systems Centre - TSC* [Busa et al., 1985] in Cambridge, the factors aggravating the condition of the elements of a bridge were examined. The study concluded that the most influential factors of bridge structure deterioration condition are age, intensity of traffic load, environmental conditions, parameters used in design of structural elements of the bridge, and quality of the used material and quality of construction process itself. According to the report of FHWA [USDOT/FHWA, 1989], most of the research indicated that the indices of condition deterioration showed considerable changes in the first several years of service, while later on they tend to predict a decelerated decline of rating of bridge structures condition.

The potential for forecast of the condition deterioration process of basic technical and functional characteristics of bridges, as well as the evaluation of the remaining service life are invaluable input data for the bridge management system. The models which forecast deterioration of bride structures condition in time are crucial for efficient planning of maintenance. This is particularly prominent in the process of optimization and planning of necessary activities and corresponding finances.

Modelling of the condition deterioration process is very complex and intricate, since there are multiple factors affecting this phenomenon, because of which due attention is paid to that in many countries. Various techniques are implemented for the forecast of bridge structure condition deterioration. In principle, the models for forecast of bridge condition deterioration can be classified in four categories: models of physical - chemical processes, deterministic models, stochastic models and artificial intelligence models.

6 FUTURE CONDITION FORECAST

The length of time that bridges spend in certain condition can be defined according to the updated database of the bridge structure inspection. In addition, trajectory of bridge structure deterioration can be determined along with the future rating class of bridge condition by using deterministic model based on the length of time the bridge spends in certain class of condition rating. New bridge structures start from the condition rating class "1" and they successively pass through every following class of condition rating, until the worst condition "6" is reached. On the basis of regular inspections of bridge condition change in time in Niš, the length of time bridges spend in certain rating classes was established, and the shortest time required for a new bridge to reach impermissible condition rating "6" was obtained; it amounts to 42 years, if no interventions are undertaken and no investments are made in maintenance and repair. It is stated that bridge structure should not dwell in the condition rating class "6".

The key question is: what social community can afford to "replace" the bridges every 40 years?

Bridge structure condition deterioration on the traffic routes of Niš was also analyzed using stochastic models. Usage of stochastic models considerably contrinom stohastičkih modela. Upotreba stohastičkih modela značajno doprinosi na polju modeliranja pogoršanja stanja infrastrukture, zbog izuzetno visoke neizvesnosti i slučajnosti koje karakterišu proces pogoršanja stanja konstrukcija. Najčešće korišćena tehnika za prognoziranje pogoršanja stanja infrastrukture je model Markovljevih[2] lanaca. Markovljevi modeli pogoršanja stanja mostovskih konstrukcija zasnivaju se na konceptu definisanja stanja u smislu ocene stanja konstitutivnih elemenata mosta i dobijanja verovatnoće prelaza iz jednog stanja u drugo stanje. Pomoću Markovljevog lanca računa se verovatnoća da se element mosta ili mostovska konstrukcija u određenom trenutku vremena nađe u određenom stanju. Stanja su diskretne kategorije. Broj stanja (stepena dotrajalosti) u kojima se Markovljev proces može naći je konačan (u ovom slučaju 6). Između dva sukcesivna inspekcijska pregleda moguć je prelazak iz boljeg stanja u gore stanje najviše za jedan nivo ocene stanja. Markovljevi procesi bi trebalo da ispunjavaju sledeće uslove [Collines, 1972]:

– sistem je definisan nizom konačnih stanja i može biti u jednom jedinom stanju u datom trenutku;

– poznato je početno stanje sistema i raspodela verovatnoće početnog stanja;

– pretpostavlja se da su verovatnoće prelaza stacionarne tokom vremena i nezavisne od načina kako je samo stanje bilo postignuto.

Verovatnoća prelaza

butes to the modelling of infrastructure condition deterioration, because of extremely high uncertainty and randomness which characterize the process of structure condition deterioration. Most frequently used technique for forecasting the infrastructure condition deterioration is the Markov[2] chains model. Markov's models of bridge structure condition deterioration are based on the concept of definition of the condition in terms of assessing the condition of constitutive elements and obtaining the probability of transition from one condition to another. By using the Markov's chain, one calculates the probability of a bridge element or bridge structure being in a certain condition in a certain moment in time. Conditions are discrete categories. The number of conditions (deterioration degrees) in which the Markov process can be found is finite (in this case it is 6). Between two successive inspections, a transition from a better to a worse condition is possible, but not for more than one degree of condition assessment. The Markov's processes should satisfy the following conditions [Collines, 1972]:

– a system is defined by a series of finite states and it can be only in one state in a given time,

– the initial condition of the system and distribution of initial condition probability are known,

– it is assumed that the probabilities of transitions are stationary in time, an independent of the way how the state was achieved.

Transition probability

$$P_{ij} = P[X_t = i, X_{t+1} = j] \tag{1}$$

iz jednog stanja u drugo stanje, predstavljena je matricom (n x n) koja se naziva matrica verovatnoća prelaza **P**, gde je n broj stanja. Oblik matrice verovatnoća prelaza jeste:

from one state into another is represented by the matrix (n x n) which is called the matrix of transition probability **P**, where n is the number of the condition. The form of the transition probability matrix is:

$$\mathbf{P} = \begin{bmatrix} p_{11} & p_{12} & 0 & 0 & \cdot & 0 \\ 0 & p_{22} & p_{23} & 0 & \cdot & 0 \\ 0 & 0 & p_{33} & p_{34} & \cdot & 0 \\ 0 & 0 & \cdot & \cdot & \cdot & 0 \\ 0 & 0 & \cdot & \cdot & p_{n-1\,n-1} & p_{n-1\,n} \\ 0 & 0 & \cdot & \cdot & 0 & 1 \end{bmatrix} \tag{2}$$

U ovom slučaju, matrica verovatnoća prelaza **P** jeste kvadratna matrica 6. reda sa elementima p_{ij}, gde je:

In this case transition probability matrix **P** is a square matrix of 6th order with the elements p_{ij}, where:

$$0 \le p_{ij} \le 1 \tag{3}$$

Svaki element u ovoj matrici p_{ij} predstavlja verovatnoću da će komponenta sistema napraviti prelazak iz stanja „i" u trenutku t_n u stanje „j" u trenutku $t_{n+1} > t_n$ (tokom određenog prelaznog perioda).

Pretpostavka da tokom jednog diskretnog vremenskog razdoblja (od t_n do t_{n+1}), proces može ili ostati u istom stanju ili preći u prvo naredno više stanje, daje konačan oblik matrice verovatnoća prelaza.

Ako je sadašnje ili početno stanje poznato, tj. **p(0)** = [$p_1(0)$ $p_2(0)$ $p_3(0)$ … $p_n(0)$], onda se buduće stanje može predvideti u svakom trenutku t.

Each element in this matrix p_{ij}, represents a probability that a system component will transit from state "i" at the moment t_n into state "j" at the moment $t_{n+1} > t_n$ (in the course of certain transition time period).

The assumption that during one discrete time period (from t_n to t_{n+1}), the process can either remain in the same state or transit into the first successive higher state provides the final form of the transition probability matrix.

If the present or initial state is known, i.e. **p(0)** = [$p_1(0)$ $p_2(0)$ $p_3(0)$ … $p_n(0)$], then the future state can be forecast at any given moment t.

[2] Андрей Андреевич Марков (14. 06. 1856-20. 07. 1922) bio je ruski matematičar i član Ruske akademije nauka. Najpoznatiji je po svojim istraživanjima u teoriji stohastičkih procesa, koja su posle postala poznata kao Markovljevi procesi.

[2] Андрей Андреевич Марков (June 14th 1856 - July 20th 1922) was a Russian mathematician and a member of the Russian academy of sciences. He was the most famous for his research of the theory of stochastic processes, which later became known under the name of Markov's processes.

Vektor verovatnoća početnog stanja formira se tokom prvog niza pregleda mostovskih konstrukcija. Vektor verovatnoća budućeg stanja dobija se množenjem vektora verovatnoća početnog stanja **p(0)** s matricom verovatnoća prelaza **P** na *m*-ti stepen, kao što je dato matričnom jednačinom (4).

Initial state probability vector is formed during the first series of bridge structures inspections. Future state probability vector is obtained by multiplying the vector of initial state probability **p(0)** with the transition probability matrix **P** to *m*-th power, as provided by the matrix equation (4).

$$\boldsymbol{p}(t_n) = \mathbf{p}(0) \times \boldsymbol{P}^m \tag{4}$$

gde je:

Where is:

$$\boldsymbol{p}(t_n) = [\ p_1(t_n)\ \ p_2(t_n)\ \ p_3(t_n)\ \dots\ p_n(t_n)] \tag{5}$$

vektor verovatnoća budućeg stanja nakon *m* vremenskih intervala (godina).

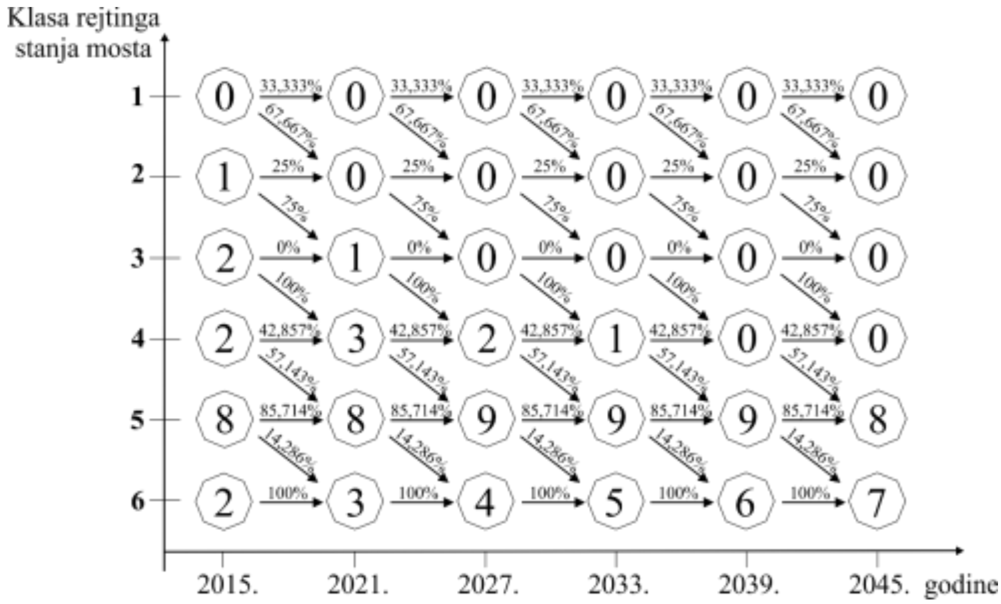
Na osnovu jednačine (4), sračunate su verovatnoće da određeni broj mostova boravi u svakoj klasi rejtinga stanja za period do 2045. godine, ako bi se nastavila primenjivana „jeftina" strategija „ne preduzimati ništa" (*do nothing*), odnosno „čekati".

Pogoršanje stanja mostovskih konstrukcija na gradskim saobraćajnicama Niša, dobijeno stohastičkim modelom Markovljevog lanca za prognozirani period, predstavljeno je na slici 6.

the future state probability vector after *m* time intervals (years).

On the basis of the equation (4), are calculated probabilities that certain number of bridges is dwelling in each condition rating class for the period until 2045 if the implemented "cheap" *do nothing or wait* strategies persist.

Bridge structure condition deterioration on the city traffic routes of Niš, obtained using the stochastic model of the Markov's chain for the forecast time period, is displayed in figure 6.



Slika 6. Grafički prikaz Markovljevog lanca pogoršanja stanja mostova u Nišu
Figure 6. Graphic preview of the Markov chain bridges deterioration in Nis

Dobijeni broj mostova u klasi rejtinga stanja 6 treba uzeti uslovno (podrazumeva se da mostovske konstrukcije ne mogu da borave u klasi rejtinga stanja „6"), jer dosledna primena ovakve „jeftine" strategije „ne preduzimati ništa", odnosno „čekati" neminovno dovodi do njihovog urušavanja. Dakle, vođenjem ovakve „jeftine" strategije održavanja „ne radi ništa" (*do nothing*), koja ne proizvodi nikakve direktne troškove, dobija se najkraći eksploatacioni vek mosta. Stoga, nameće se potreba za planiranjem aktivnosti održavanja mostovskih konstrukcija, čime bi se produžio eksploatacioni vek postojećih mostova uz razumne troškove. Zbog toga, dalja istraživanja i aktivnosti autora ovoga rada usmerena su na program preventivnog održavanja – kako na individualnom, tako i na mrežnom nivou,

The obtained number of bridges in the condition rating class 6 is only provisional (it is comprised that bridge structures cannot dwell in the condition rating class "6", because "consistent implementation" of such "cheap" strategy – "do nothing" or "wait", will lead to their certain collapse. Therefore, using such "cheap" maintenance strategy "do nothing" which fails to incur any direct costs, will result in the shortest service life of the bridge. For these reasons, it is imperative to plan the bridge structure maintenance activities, which would extend the service life of the existing bridges with the reasonable costs. For this reason, further research and the activities of the author of this paper are directed towards the program of preventative maintenance, both at the individual and network level, selection and choice selekciju i izbor strategija i programa za održavanje i rekonstrukciju, odnosno zamenu objekta na osnovu metode analize koštanja životnog ciklusa (*life-cycle cost*) i optimizacije radova, kao i na selekciju i manipulaciju ogromnog broja neophodnih podataka za sve mostove u mreži. Fokus ovih istraživanja je da se nađe odgovor na osnovno pitanje „kako se može uraditi bolje?", a u skladu s raspoloživim sredstvima, umesto dosadašnjeg menadžment pristupa „prvi najgori", s obzirom na to što je nivo finansiranja daleko ispod potreba za rekonstrukcijom i revitalizacijom svih neadekvatnih mostovskih konstrukcija u zemlji, kod kojih su utvrđeni određeni konstrukcijski i funkcionalni nedostaci.

7 ZAVRŠNE NAPOMENE

Sistem upravljanja mostovima obezbeđuje racionalan i sistematičan pristup svim aktivnostima koje se odnose na upravljanje mostovima kako na individualnom, tako i na mrežnom nivou. Najekonomičnije obezbeđenje nosivosti, upotrebljivosti i trajnosti, uz zahtevani nivo pouzdanosti i sigurnosti postojećih mostova, veoma je značajna tema savremene projektantske prakse i naučnih istraživanja. Ovaj problem znatno se uvećava kada je reč o celovitom saobraćajnom sistemu - mreži. Za razliku od tradicionalnog pristupa da se mostovi tretiraju ponaosob, a problemi rešavaju u trenutku kada već nastanu, sistemi za upravljanje mostovima baziraju se na bankama podataka i imaju planski i organizovan pristup rešavanju problema na nivou mreže objekata.

Za obezbeđenje planskog i kvalitetnog optimalnog upravljanja mostovima, izuzetno je važno da odgovorna ličnost koja upravlja bazom podataka i rezultatima pregleda mostova, mora biti sertifikovani inženjer sa znanjem i iskustvom u projektovanju i građenju mostovskih konstrukcija.

Upravljanje mostovima i drugim objektima u sklopu saobraćajne mreže, veoma je kompleksan sistem s velikim brojem izuzetno raznovrsnih, ali međusobno usko povezanih i zavisnih aktivnosti.

Problematika se povećava shodno veličini mreže, rastu obima saobraćaja, promeni transportnih sredstava, zatim različitoj osetljivosti konstrukcija i okoline, kao i istorijskom nasleđu koje se prvenstveno ogleda u prevaziđenim metodama projektovanja i građenja, sanacije, rekonstrukcije i održavanja. Na primer, korišćenje dilatacionih sprava za sprečavanje prekomernih podužnih pomeranja i sila na dugačkim železničkim mostovima skupo je i loše rešenje u pogledu bezbednosti saobraćaja, udobnosti, kao i troškova održavanja [16]. Zbog toga se primenjuju druga moguća rešenja – u skladu s projektom konstrukcije. U ovakvim slučajevima, upravljačke odluke ne mogu biti zasnovane na intuitivnom procenjivanju i prosuđivanju. One nužno moraju biti zasnovane na rezultatima ključnih elemenata celovitog upravljačkog sistema, sposobnog da iz aspekta široke društvene zajednice dugoročno proceni sve posledice odlaganja ili nepreduzimanja potrebnih mera održavanja mostovskih konstrukcija. Dosadašnji rezultati ukazuju na potrebu novih istraživanja i usvajanja adekvatne strategije održavanja, odnosno planskog i sistematskog pristupa u oblasti upravljanja mostovima. Način na koji se taj pristup ostvaruje određen je

of strategies and programmes for maintenance and reconstruction, i.e. replacement of structures based on the life-cycle cost analysis method and on the optimization of works and on selection and manipulation of immense number of necessary data for all the bridges in the network. The focus of these research has a goal of finding an answer to the fundamental question "how to work better?", according to the available resources, instead of the management approach of tackling the "first and worst", regarding that the level of finances is far below the requirements for reconstruction and revitalization of all inadequate bridge structures in the country where certain structural and functional deficiencies are detected.

7 CLOSING REMARKS

Bridge management system provides a rational and systematic approach to all activities which relate to bridge management, both on an individual and the network level. The most cost-effective provision of bearing capacity, serviceability and durability, with the required level of reliability and safety of the existing bridges is a very important topic of contemporary designer practice and scientific research. This problem considerably rises the question about the integral transportation system network. As opposed to the traditional approach that bridges are treated individually, and problems are solved when they occur, the bridge management systems are based on databases and have a well planned and organized approach to problem solving at the level of structure network.

For provision of the planned and good quality optimum bridge management, it is very important to have a certified engineer with knowledge and experience in designing and building bridge structures as a responsible person who manages the database and bridge inspection results.

Management of bridges and other structures within a transportation network is very complex system with a high number of extremely diverse but mutually closely connected and dependent activities.

The complexity of the problem increases according to the size of network, rise of traffic frequency, change of transport vehicles, different sensibility of structures and environment and to the historical legacy which primarily means obsolete methods of designing and building, restoration, reconstruction and maintenance. For instance, usage of expansion devices prevention of excessive longitudinal displacements and forces on the long railway bridges is a costly and poor solution in terms of traffic safety, comfort and maintenance cost [16]. For this reason, alternative solutions in accordance with the structural design are implemented. In such cases, the management decisions cannot be based on the intuitive assessment and judgment. They should be necessarily based on the results of the key elements of comprehensive management system able to provide a forecast to the wide social community of all the consequences of delaying and doing nothing regarding the bridge structure maintenance. Current results indicate the need for a new research and adoption of adequate maintenance strategy, that is, a planned and systematic approach in the field of bridge management. The way in which this approach is realized is determined razvojem sistema upravljanja mostovima, a uspešnost upravljanja bitno zavisi od izbora i doslednog sprovođenja svih relevantnih aktivnosti koje čine taj sistem.

by the development of the bridge management system, and the success of management greatly depends on the choice and consistent performance of all the relevant activities comprising this system.

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REZIME

OCENA STANJA I PERSPEKTIVE ODRŽAVANJA MOSTOVSKIH KONSTRUKCIJA U GRADU NIŠU

Milan GLIGORIJEVIĆ

Mostovi su građevinski objekti koji svojom veličinom, izgledom, pojavom u prostoru, pa čak i simbolikom, vrlo često dominiraju ambijentom ili krajolikom u kojem se nalaze. S druge strane, preventivnom održavanju ovih objekata – kao optimalnom rešenju – nije se posvećivala neophodna pažnja koliko je to bilo potrebno, naročito kada je reč o našem području. Uočen je permanentni porast oštećenja na mostovima, uz enormni pad nosivosti i bezbednosti mostova, te su od ogromnog značaja razvoj i uvođenje adekvatnog sistema upravljanja mostovima s ciljem efikasnijeg održavanja i očuvanja postojećih mostovskih konstrukcija.

Sistem upravljanja mostovima u Srbiji uveden je 1986. godine, kao originalan i za to vreme izuzetno moderan. Međutim, njegova praktična primena pokazala je određene nelogičnosti u dobijenim listama prioritetnih aktivnosti.

U svojoj doktorskoj disertaciji, autor ovoga rada dao je novi predlog – na osnovu optimizovanog kriterijuma vrednovanja prioriteta. Time su otklonjene uočene nelogičnosti našeg aktuelnog sistema upravljanja mostovima i ostvareno je značajno poboljšanje efikasnosti određivanja liste prioriteta.

Predloženom metodologijom, a na osnovu rezultata sopstvenog višedecenijskog monitoringa mostova, urađena je ocena stanja i perspektiva održavanja mostova u gradu Nišu, koja je prezentovana u ovom radu.

Ključne reči: Upravljanje mostovima, vrednovanje prioriteta, analiza, ocena stanja, prognoza.

SUMMARY

CONDITION ASSESSMENT AND MAINTENANCE PERSPECTIVES OF BRIDGE STRUCTURES IN THE CITY OF NIS

Milan GLIGORIJEVIC

Bridges are civil engineering structures which very often dominate the environment or landscape where they are situated by their size, appearance in space and even by symbolism. On the other hand, preventive maintenance of these structures, as an optimum solution, is paid insufficient deal of attention, especially in our country. A permanent increase of damage on the bridges is observed, followed by an enormous decrease of bearing capacity and safety, which means that it is critical to develop and introduce an adequate system of bridge management with an aim of maintenance and preservation of existing bridge structures.

Bridge management system in Serbia was introduced in 1986, as an original, and extremely contemporary system for the time. However, its practical application exhibited certain illogical issues in the obtained lists of priority activities.

The author of this paper, in his doctoral dissertation, offered a new proposition based on the optimized criterion of priority evaluation. This removed certain illogical points of the current bridge management system, and achieved a considerable increase of efficiency when making priority lists.

The paper presents the proposed methodology based on the results of bridge monitoring over a period of several decades which was used to provide bridge condition assessment and its maintenance perspectives in the city of Nis.

Key words: Bridge management, priority evaluation, analysis, assessment, forecast.

PODZEMNE STAMBENE ZGRADE U KONTEKSTU ENERGETSKI EFIKASNIH GRAĐEVINA

EARTH-SHELTERED HOUSING BUILDINGS IN THE ENERGY EFFICIENT STRUCTURES CONTEXT

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1 UVOD

Razvoj podzemne stambene arhitekture u velikoj meri zavisi od okruženja u kojem su objekti bili izgrađeni (klima, geografsko područje...) ali i od materijala i tehnologije koji su primenjeni prilikom građenja. Većina podzemnih stambenih objekta podignuta je u područjima u kojima su visoka temperatura i retke kiše klimatske karakteristike, a retko su prisutni u hladnijim, planinskim krajevima sa snegom. Istraživanja pokazuju da je podzemno stanovanje veoma zastupljeno u mediteranskim zemljama, Aziji i Africi, Severnoj Americi i delovima Evrope, i to u većem ili manjem obimu u svim istorijskim periodima.

Od praistorije do danas, čovek se prilagođava uslovima i okruženju u kojem obitava, postepeno podižući nivo kvaliteta življenja i funkcije zajednice u kojoj učestvuje kao jedinka. Pećine, planine i pustinjski ambijent nude mogućnosti za različite oblike podzemnih habitata. Od čovekovog napuštanja pećine, on se bori s problemima da savlada prirodu i bira najfunkcionalnije i najprikladnije oblike fizičke strukture da bi zadovoljio svoje potrebe. Podzemni objekti, kao specifičan oblik građevine koja zadovoljava potrebe stanovanja, pojavljuju se na svim delovima naše planete.[1] Njihova tipologija uglavnom najviše zavisi od klimatskih karakteristika područja na kojem čovek živi. [2] Podzemni stambeni objekti suštinski se mogu istraživati putem njihove dve najčešće pojave:

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1 INTRODUCTION

Evolution of underground residential architecture is largely dependent on the environment in which the buildings were constructed (climate, geographical area ...), but also of the materials and technologies used during the construction. Most residential underground facilities were built in areas where the climatic characteristics are high heat and rare rainfall, while they are rarely present in colder, mountainous areas with snow. Researches indicate that the underground dwellings were built in many Mediterranean countries, Asia and Africa, North America and Europe, with a greater or lesser extent, through all historical periods.

From prehistory to nowadays, man adapts to the conditions and environment in which he resides, gradually raising the quality of life and the function of the community he acts as an individual. Caves, mountains, and desert ambients offered opportunities for different forms of underground habitats. From the man's exodus from the cave, he is struggling with the problems of overcoming nature and choosing the most functional and most suitable form of physical structure for the fulfillment of his own needs. Underground objects as a specific form of structure for meeting the needs of housing, appear in all parts of our planet. [1] Their typology was mostly dependent on the climatic characteristics of the climate that man lived. [2] Earth-sheltered housing objects can essentially be viewed through their two most commonly occurring forms;

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• podzemni stambeni objekti smešteni u prirodnom okruženju s minimalnom intervencijom ljudi;

• podzemni stambeni objekti koje ljudi stvaraju, menjajući karakterisitike prirodnog okruženja pri čemu nove strukture prilagođavaju prirodnom okruženju.

Za razliku od praistorijskog čoveka, čovek XXI veka ima veće tehnološke mogućnosti da prilagodi život u podzemnim građevinama svojim potrebama, bez obzira na to da li se radi o jednom ili drugom načinu navedenog oblika njihovog stvaranja.[3]

Podzemni objekti predstavljaju kombinovani održivi model koji prvenstveno funkcioniše na bazi recikliranja atmosferske vode s površine zelenih krovova, koristeći energiju koja dolazi iz zemlje ili podzemnih voda za potrebe zagrevanja ili hlađenja. Usled ograničenih prostornih kapaciteta u gradovima za primenu principa bioklimatskog dizajna, u razvoju podzemnih građevina izvan naselja koja nisu infrastrukturno obezbeđena, moguće je postići maksimalnu energetsku efikasnost uvođenjem solarnih sistema, kao trećeg dodatnog oblika snabdevanja energijom. Neposredni rezultat primene modela podzemnih objekata u urbanizovanim okruženjima jeste zadržavanje ukupne površine tla pod vegetacijom, čime se postiže maksimalni efekat povoljnog uticaja na kvalitet i kvantitet ekosistema koji se može sačuvati ovim specifičnim tipom građenja. [4]

Evoluirajući od pećina i drugih sličnih oblika, danas u podzemnim stambenim objektima živi preko 50 miliona ljudi. Nova podzemna stambena arhitektura stvara se sa izraženom brigom zbog povećanja energetske potrošnje i gradi se na osnovu iskustva anonimnih graditelja podzemnih objekata, čiji su koncepti danas inovativna rešenja za probleme ljudi XXI stoleća. [5]

U ovom članku mi ukazujemo na značajno povoljnije energetske aspekte ukoliko podzemna gradnja postane učestalija.

2 PODZEMNE STAMBENE ZGRADE U KONTEKSTU ENERGETSKI EFIKASNIH GRAĐEVINA

S ciljem da se smanji uticaj atmosferskih padavina i taloženja, posebnu pažnju tokom gradnje podzemnih stambenih objekata treba obratiti na:

• izbor mesta za građenje;

• pravljenje drenaže u skladu s lokacijom i vodootpornim slojevima terena u svim aspektima dodira objekta sa zemljom.

Vegetacija na mestu građenja može biti od višestruke koristi za buduće ponašanje objekta. Zeleni fond se može posmatrati iz više aspekata u odnosu na uticaj koji ima na podzemno stanovanje. Jedan je, svakako, značajna štednja energije koja se može postići postojanjem ili sađenjem drveća na mestu planiranom za gradnju podzemnog stanovanja. Testovi vezani za količinu uštede energije sa zelenim zasadima još nisu urađeni, ali iz prakse je poznato da zasadi na južnoj strani podzemne stambene zgrade mogu tokom leta znatno smanjiti temperaturu unutar zgrade.

U Americi se istraživanje o energetskoj efikasnosti podzemnih stambenih objekata sprovodi od osamdesetih godina prošlog veka. Parametri koji konkretno ukazuju na potrebu da se definišu propisi za izgradnju ovih objekata, na osnovu činjenica koje su merljive, potvrđuju da podzemni stanovi predstavljaju kuće s vrlo

• Underground residential buildings located in a natural environment with minimal human intervention;

• underground residential buildings that man creates by changing the characteristics of the natural environment and the new structure adapts to the natural environment.

Unlike a prehistoric man, a man of the 21st century has far greater technological abilities to compare life in underground facilities to his needs, whether it is one or the other mentioned approach to the use of these facilities. [3]

Underground structures represent a combined sustainable model, which functions primarily on the recycling of atmospheric waters from the surface of green roofs, using energy derived from earth or groundwater for heating and cooling purposes. Due to limited spatial capabilities in cities for the application of bioclimatic design principles, by affirming the development of subterranean buildings outside settlements that are not infrastructural supplied, it is possible to achieve maximum energy efficiency by introducing solar systems as the third supplementary form of energy supply. The direct result of applying the model of underground facilities in urbanized environments is to maintain the total area of the soil under vegetation, thus achieving the maximum effect of achieving favourable impacts on the quality and the quantity of ecosystems that can be preserved using this form of specific construction type. [4]

Evolving from the caves and other different types, over 50 million people live today in earth-sheltered housing. New underground residential architecture is created with great awareness for increased energy consumption and built on the experiences of anonymous builders of underground facilities, whose concepts today are innovative solutions to the problems of a man of the 21st century. [5]

In this article, we suggest significantly more favourable energy aspects if underground structures becomes more common.

2 EARTH-SHELTERED HOUSING BUILDINGS IN THE ENERGY EFFICIENT STRUCTURES CONTEXT

Special attention during the construction of underground residential facilities with the aim of reducing the impact of atmospheric precipitation should be paid to:

• Selection of the site for construction

• Making drainage in accordance with the location and terrain waterproofing layers in all aspects of contact with the ground facility.

The vegetation on the site for construction can have multiple benefits for future operation of the facility. Green Fund can be viewed from several aspects in terms of its impact on the underground dwellings. One, of course, is a significant energy saving that can be achieved with the existence or planting trees at the site planned for the construction of an underground dwelling. The tests related to the amount of energy saving with green plantations are not yet done, but it is known from practice that the plantations on the southern front of the underground residential building can significantly reduce the temperature inside the building in summer.

In America, research on energy efficiency underdobrim energetskim performansama. Istraživanje energetske efikasnosti prvenstveno podrazumeva određivanje parametara koji dovode do procene koristi od izgradnje podzemnog objekta u odnosu na nadzemne strukture. Isti rezultati se mogu dobiti u smanjenju emisija CO_2 korišćenjem energetski efikasnih modela kao što su podzemne konstrukcije.[6]

Prema bazi podataka koja je sastavljena tokom pomenutog istraživanja, „može se reći da kuće pokrivene zemljom pokazuju znatno bolje energetske performanse nego kuće standardnih fasada”. Tvrdnja da se potrošnja energije za ovu vrstu kuća može smanjiti do 75%, može se dokazati korišćenjem podataka koji se odnose na praćenje parametra faktora termičkog integriteta, koji u slučaju kuća pokrivenih zemljom iznosi 28.40 KV / m^2 dnevno, dok isti parametar u slučaju tradicionalnih nadzemnih kuća iznosi 113.56 KV / m^2”.[7]

Prilikom istraživanja koje je sprovedeno na dve južno orijentisane strukture: jednoj podzemnoj stambenoj kući i standardnoj nadzemnoj kući na istom području, Stanjec i Novak iz Odeljenja za građevinarstvo na Tehnološkom univerzitetu u Vroclavu, zaključili su da gubici i dobici stoje u određenim razmerama, s vrlo sličnim pokazateljima. Utvrdili su, takođe, da se, ako se parametri vrednosti za hlađenje i grejanje analiziraju zasebno, primećuju značajne razlike. [8]

Ova pojava uzrokovana je činjenicom da je toplotni gubitak u podzemnom objektu znatno manji nego u nadzemnim zgradama, ali samo u toku grejne sezone, te da je tokom letnje sezone toplotni gubitak u pozemnim objektima veći. Pošto je temperatura tla u letnjem periodu niža od temperature vazduha, posledica je hlađenje objekta. Ovo je jedan od glavnih razloga zašto podzemne konstrukcije zahtevaju znatno manje energije od površinskih. Primetili su, takođe, da su u toku grejne sezone „toplotni gubici u podzemnim stambenim zgradama od 14%, 8% i 5% manji za 5 cm, 10 cm i 20 cm debljine toplotne izolacije. Povećanje debljine slojeva zemlje iznad krovnog pokrivača smanjuje gubitke toplote za 20-25%, 10-15% i 5% primenom toplotne izolacije od 5 cm, 10 cm i 20 cm u poređenju s toplotnom izolacijom od 0.5 m”. U ovom smislu može se zaključiti da su toplotni dobici u podzemnim stambenim zgradama do 40% veći nego u površinskim strukturama.[9]

Da bi dokazali teze iz njihovog istraživanja s jasnim uporednim odnosima prosečnih vrednosti, Stanjec i Novak su razvili „dijagram gubitaka toplote” koji predstavlja uporednu analizu za nadzemne i podzemne objekte, u zavisnosti od debljine izolacije i debljine slojeva zemlje iznad podzemnih objekata, koji su predstavljeni kao prosečne vrednosti, vrednosti tokom grejne sezone i sezone hlađenja.

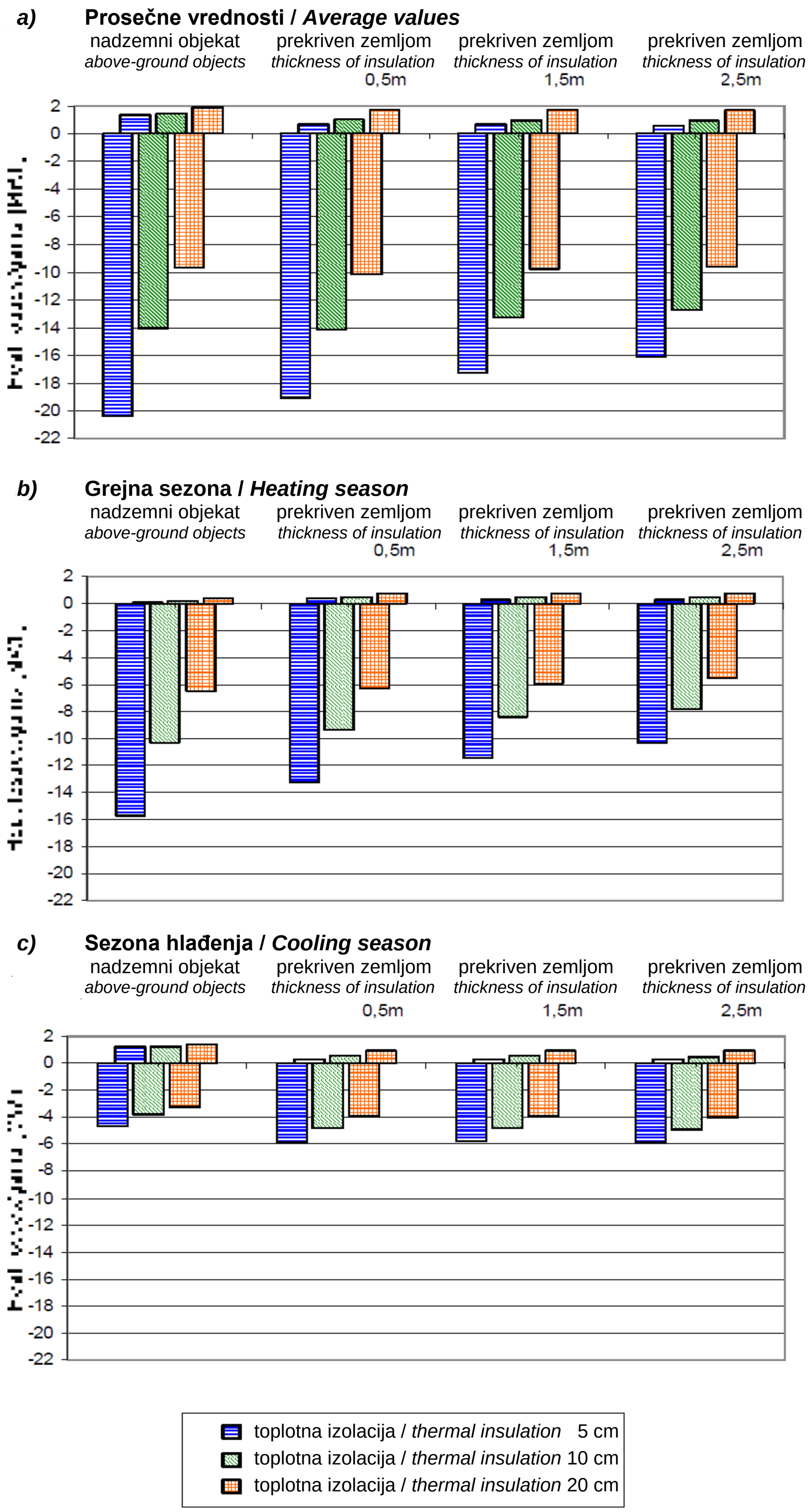
Istraživanja sprovedena na Državnom univerzitetu Oklahoma [10] i Vendt-ova istraživanja [7] potvrđuju da su podzemne stambene zgrade energetski efikasne zgrade i da su bile od velike važnosti za odnos prema proceni energetske efikasnosti kao jedne od najvažnijih komponenti u vrednovanju opšte efikasnosti podzemnih zgrada.

ground residential facilities is conducted since the 80th of the last century. Parameters that specifically point to the need to define regulations for the construction of these facilities, based on the facts that are measurable, and which indicate that the underground dwellings are houses with very good energy performance. Research on energy efficiency; primarily implies to determine the parameters which lead to the assessment of the benefits of building underground in relation to the above-ground structures. The same results could be obtained in reducing CO_2 emissions by using more energy-efficient models such as underground structures. [6]

According to the database compiled during the mentioned research "it can be said that the houses covered with earth exhibited considerably better energy performances than the standard surface houses”. The claim that the energy consumption for this type of houses can be reduced up to 75%, can be proved using the data related to monitoring the thermal integrity factor parameter, which in case of the houses covered with soil averages 28.40 KW/m^2 daily, while the same parameter in case of traditional surface houses averages 113.56 KW/m^2 ". [7]

On the event of the research conducted on two south-oriented structures: one being earth-sheltered elevated housing and the standard surface house of the same area the Staniec and Nowak from the Department of Civil Engineering at the University of Technology of Wroclaw, concluded that the losses and gains stand in certain proportions, with very similar indicators. They also determined that if value parameters for cooling and heating are separately analyzed, considerable differences are observed. [8]

They found that they arise due to the fact that the thermal loss in an earth-sheltered housing is considerably less than in the surface housing, but only during the heating season and that during the summer season, thermal loss in the earth-sheltered housing is higher. Since the temperature of soil in the summer is lower than the air temperature, this causes cooling of the structure. it is one of the main reasons why underground structures require considerably less energy than the surface ones. They also noticed that during the heating season "thermal losses in underground residential buildings by 14%, 8%, and 5% are less by the 5cm, 10cm and 20cm thickness of thermal insulation. Increasing the thickness of the layers of the earth above the roof panel reduces heat losses by 20-25%, 10-15% and 5% for 5cm, 10cm and 20cm thermal insulation compared to 0.5m thermal insulation". The [9] in this sense, it can be concluded that heat gains in the underground housing structures are up to 40% higher than in the surface structures.



Slika 1. Uporedna analiza nadzemnih i podzemnih objekata, u zavisnosti od debljine izolacije i debljine zemljanog sloja iznad podzemnog objekta (prema [8])

Figure 1. Comparative analysis of the above-ground and underground objects, depending on the thickness of the insulation and the thickness of the ground layers in underground objects (after [8])

Kada se statistika kao što je ova posmatra u svetlu kvalitativnih polaznih razloga za promenu čitavog skupa dokumenata vezanih za područja planiranja i izgradnje, može se očekivati promocija široko prihvaćene podzemne arhitekture stanova, s jednim od primarnih ciljeva današnjice, a to je planiranje i izgradnja podzemnih pojedinačnih zgrada, ali i čitavih naselja, pri čemu se ova tipologija stambenog prostora može odlično implementirati radi stvaranja budućih energetski efikasnih naselja u Republici Srbiji. [11] [12]

In order to show the theses from their research with clear comparative relationships of average values Stanjec and Novak developed a "heat loss diagram" which represents a comparative analysis of the above-ground and underground objects, depending on the thickness of the insulation and the thickness of the ground layers in underground objects, presented as average values, values during the heating season and the cooling season.

The research conducted by Oklahoma State University [10], and Wendt [7] confirm underground residential buildings as energy-efficient buildings and were of great importance for the relationship to the assessment of the energy efficiency as one of the most important components of general efficiency of underground buildings.

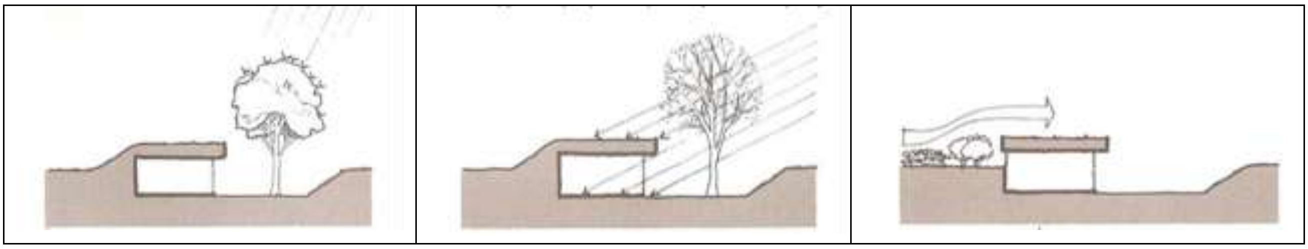
When the statistics such as this is viewed in the light of qualitative initial reasons for the change of an entire set of documents related to planning and construction areas, one can expect promotion of a more widely present underground housing architecture, with one of the primary goals of the present times, which is planning and construction of individual buildings, but also of entire settlements, where this housing typology can be implemented with an aim of creation of future energy efficient settlements in the Republic of Serbia. [11] [12]

3 UTICAJ BIODIVERZITETA NA PODZEMNO STANOVANJE

Prisustvo vegetacije na mestu građenja može pružiti višestruke pogodnosti za buduće funkcionisanje objekta.[13] Zelenilo se može posmatrati s nekoliko stanovišta, s obzirom na njegov uticaj na podzemne stambene zgrade. Jedan od nesumnjivo važnih aspekata jeste ušteda energije, koja se može postići postojećim stablima ili njihovim postavljanjem na lokaciji koja je planirana za izgradnju podzemne stambene zgrade zaštićene zemljom. Testovi koji se odnose na količinu energije ušteđene sađenjem zelenila još nisu dovoljno zastupljeni, ali iz prakse je poznato da biljke na južnoj strani zgrade zaštićene zemljom mogu pružiti značajno smanjenje temperature unutar zgrade tokom letnje sezone.

3 IMPACT OF BIODIVERSITY ON EARTH-SHELTERED HOUSING

The presence of vegetation at the construction site can provide multiple benefits for the future functioning of the structure. [13] The greenery can be observed from several viewpoints, regarding its impact on the earth-sheltered housing buildings. One certainly important aspect is energy saving which can be achieved by having or planting trees on the location planned for construction of earth-sheltered housing building. The tests related to the amount of energy saved by planting the greenery have not been conducted yet, but it is known from the practice that plants on the south side of the earth-sheltered building can provide the significant reduction of the temperature inside the building in the summer season.



Slika 2. Primeri uticaja biodiverziteta na podzemne građevine: a. vegetacija štiti od sunčanih zraka tokom letnje sezone; b. vegetacija propušta sunčane zrake tokom zimske sezone; c. vegetacija štiti od vetra. (prema [14])

Figure 2. Examples of biodiversity impacts on the earth sheltered buildings: a. vegetation blocks solar radiation in summer season; b. vegetation lets through solar radiation in the winter season; c. vegetation as a protection from wind. (after [14])

Ako se sade nove biljke, potrebno je da se na južnoj strani objekta zasade listopadne biljke, kako bi se omogućilo prodiranje sunčeve svetlosti kroz ogoljene krune drveća u zimskom periodu. Vetar, takođe, može

If new plants are planted, it is necessary to plant deciduous trees on the south side of the structure, so as to allow sunlight penetration through bare tree crowns in winter. The wind can also impact the building temperauticati na temperaturu zgrade, tako da se, u zavisnosti od smera duvanja vetra, može saditi nova vegetacija ili se može koristiti postojeća, sprečavajući tako direktan udar vetra na objekat.

U izveštaju Agencije za energetiku države Minesote u vezi sa istraživanjem u Južnoj Dakoti, navodi se: „kada se uporede parametri za dva identična objekta (jednog sa zelenilom u neposrednom okruženju, a drugog bez zelenila), zaključuje se sledeće:

− kuća koja je imala zelenilo u neposrednom okruženju zgrade, imala je uštedu do 40% energije u odnosu na kuću bez zelenila".[15]

Istraživanje je takođe ukazalo na to da postoji uticaj vegetacije na štednju prilikom upotrebe podzemnog stanovanja, tako da je prilikom izgradnje takvih struktura neophodno planirati vegetaciju u skladu s građevinskom orijentacijom i lokalnim zahtevima.

4 PRIKAZ OSNOVNIH UTICAJA NA FUNKCIONISANJE PODZEMNIH STAMBENIH OBJEKATA

4.1 Uticaj padavina na podzemno stanovanje

Prilikom izgradnje podzemnih stambenih zgrada posebnu pažnju treba obratiti na:

- izbor mesta za izgradnju građevine;
- postavljanje drenažnog sistema, u odnosu na mesto i konfiguraciju zemljišta;
- postavljanje vodootpornih slojeva na mestu dodira objekta sa zemljom kako bi se otklonio uticaj padavina i vode uopšte.

Uticaj atmosferskih padavina na podzemnu stambenu arhitekturu jeste jedan od najvećih rizika za njihovu eksploataciju. Tipologija podzemnih objekata predstavlja mogućnost za manje ili veće uticaje atmosferskih voda. Prema Reju Skotu, autoru knjige „Kako izgraditi svoj podzemni objekat" (*How to build your own underground home*) najveću opasnost po podzemne objekte predstavljaju ukopani objekti, jer ovaj tip objekata omogućava pojavu nekontrolisane količine vode pri obilnim padavinama. [16] [17]

ture, so depending on the wind blowing direction, vegetation can be planted or the existing vegetation can be used, preventing direct impact of the wind on the structure

In the report of the Agency for Energy of Minnesota, related to the research in South Dakota, it is stated that "when comparing the parameters for two identical objects (one with greenery in the immediate environment and the other without greenery), the following conclusions were reached:

− House that had greenery in the immediate surroundings of the building, had savings of up to 40% energy in relation to a house without greenery". [15]

That research also indicated that there is an impact of the vegetation on the saving in the usage of earth-sheltered housing so that when constructing such structures, it is necessary to plan the vegetation in accordance with the building orientation and local requirements.

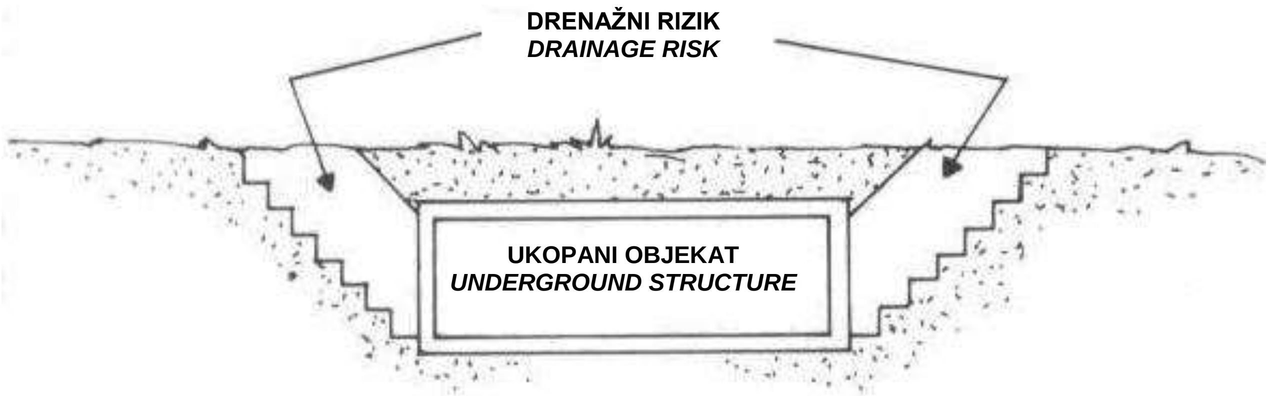
4 PRESENTATION OF THE BASIC IMPACTS ON ENERGY FUNCTIONING OF EARTH - SHELTERED STRUCTURES

4.1 Impact of rainfall on underground housing

When constructing underground housing, a special attention must be paid to:

- The choice of location of construction of the structure
- Construction of a drainage system, regarding the location and land configuration
- Construction of waterproofing layers at the contact of the structure with soil In order to reduce the rainfall impacts.

The impact of rainfall on an underground housing structure is one of the greatest risks for the operation of such buildings. The typology of underground structures represents the potential for a weak and the strong impact of rainfall. According to Ray G. Scott, author of the book "How to build your own underground home" "The greatest danger to underground objects is buried objects because this type of objects allows the appearance of uncontrolled water in abundant precipitation". [16] [17]



Slika 3. Detalj preseka kroz ukopani-prodirući tip podzemnog objekta, s prikazom rizika prilikom dreniranja (prema [18]).
Figure 3. A cross section detail of an envelope/true underground structure, with the presented drainage risks (after [18]).

Poseban problem koji prouzrokuje potencijalnu štetu zbog pritiska kiše koji prolazi kroz slojeve zemlje do objekta javlja se u podzemnim zgradama u onim slučajevima kada nema kanalizacionih kanala i mrežnog sistema s drenažnim cevima, kojima bi se odvodila voda nakupljena na zidu prema nagibu. [18]

Pored rizika od bočnih uliva atmosferske vode, osnovni problem koji su podzemni objekti imali s velikom količinom vode i vlage u objektu, nastajao bi kada bi se, pri projektovanju, odabrao pogrešan tip objekta u odnosu na teren. Jedan od takvih primera upravo je objekat atrijumske ukopane kuće u Arizoni, građene osamdesetih godina prošlog veka. Prema izjavama samih vlasnika, objekat je u najvećem broju slučajeva imao probleme s nekontrolisanim prodorom atmosferske vode, usled položaja pristupa objektu, koji se odvija preko atrijuma, formiranog ispod saobraćajnice, i to na nižoj koti u odnosu na put s kojeg se pristupa objektu. [19]

A particular problem causing potential damage due to the rainfall pressure flowing to the house through the earth layers is found in elevation – in-hill underground buildings, in those cases when no drainage canals and drainage pipe network system is provided, which would drain the water accumulated on the wall towards the slope. [18]

Apart from the risk form the lateral penetration of rainfall, the basic problem the earth sheltered buildings had in terms of excess water and dampness in the structure was caused by the improper choice of structure for the land configuration in the design phase. [One of such examples is the atrium envelope house in Arizona, built in the 80's of the previous century. According to the owners themselves, the building in most of the case had a problem with the uncontrolled penetration of rainfall, which is caused by the position of the access to the house, which is organized via an atrium created below a road, at a lower level than the road used for accessing the house. [19]

Slika 4. Podzemna kuća sa atrijumom u Arizoni: a. pozicija pristupa objektu; b. pogled na objekat ukopanog tipa sa atrijumom (prema [19])

Figure 4. The atrium envelope house in Arizona: a. Building access position, b. View of the envelope/true underground building (after [19])

Pojava privremenih podzemnih voda pri prelasku iz jednog u drugo godišnje doba, „može stvoriti problem usled velikog nagomilavanja vode u situacijama kada je nepripremljen objekat izložen pritisku vode" [20], koji se može znatno smanjiti preduzimanjem svih potrebnih koraka prilikom same izgradnje objekta.

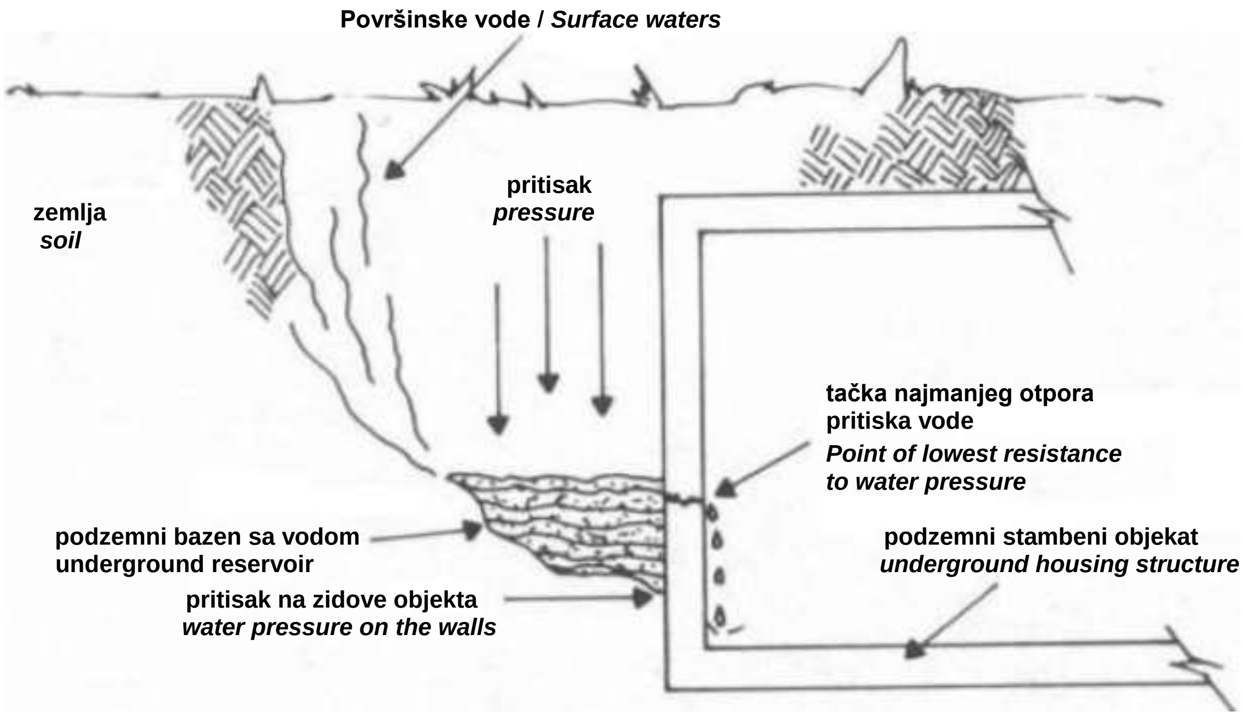
Pored standardnih pristupa izradi hidroizolacionih slojeva kod podzemnih stambenih objekata, negativni uticaji se, danas, svakako značajno mogu umanjiti postavljanjem gumiranih membrana, koje ne samo što utiču na nemogućnost prodiranja vode ka objektu, već istovremeno služe za zadržavanje vode radi navodnjavanja vegetabilnog sistema iznad krovne konstrukcije objekta.

Provera geoloških slojeva zemlje, uz informacije o stabilnosti i slojevima podzemnih voda, može znatno sprečiti moguće negativne pojave prilikom eksploatacije objekta.

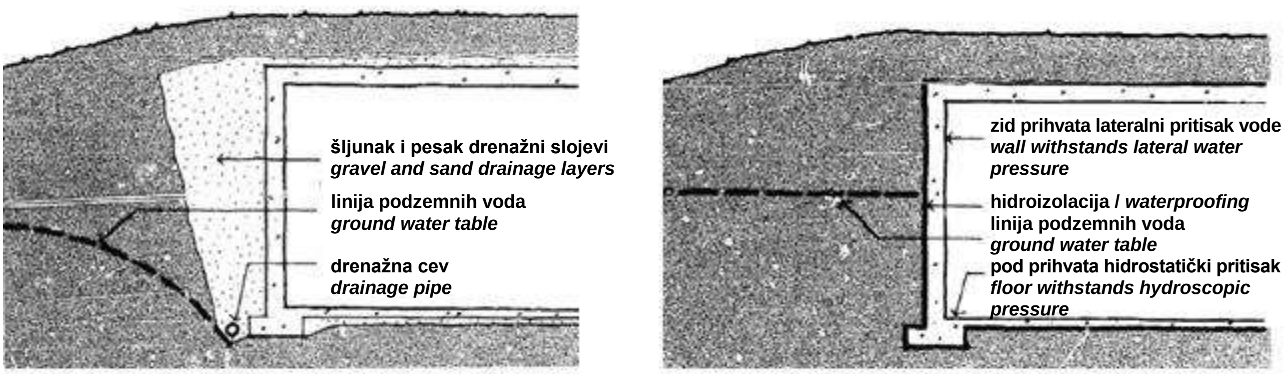
The occurrence of temporary underground waters when the season's change, "can create a problem due to the large accumulation of water in situations where the unprepared object is exposed to water pressure" [20], which can be significantly reduced when all necessary steps when constructing the building are taken.

By creating a drainage layer made of sand and gravel and by placing a drainage pipe around the building potential for damage to the structure and penetration of rainfall water is considerably reduced.

In addition to the standard approaches to the building of waterproofing layers of the underground structures roofs, the negative impacts can be considerably reduced nowadays by installing rubber membranes, which not only prevent water from penetrating the structure but retain water so that vegetation system above the roof structure can use it.



Slika 5. Detalj zone pritiska atmosferske vode (prema [20])
Figure. 5. A detail of the rainfall water pressure zone (after [20])

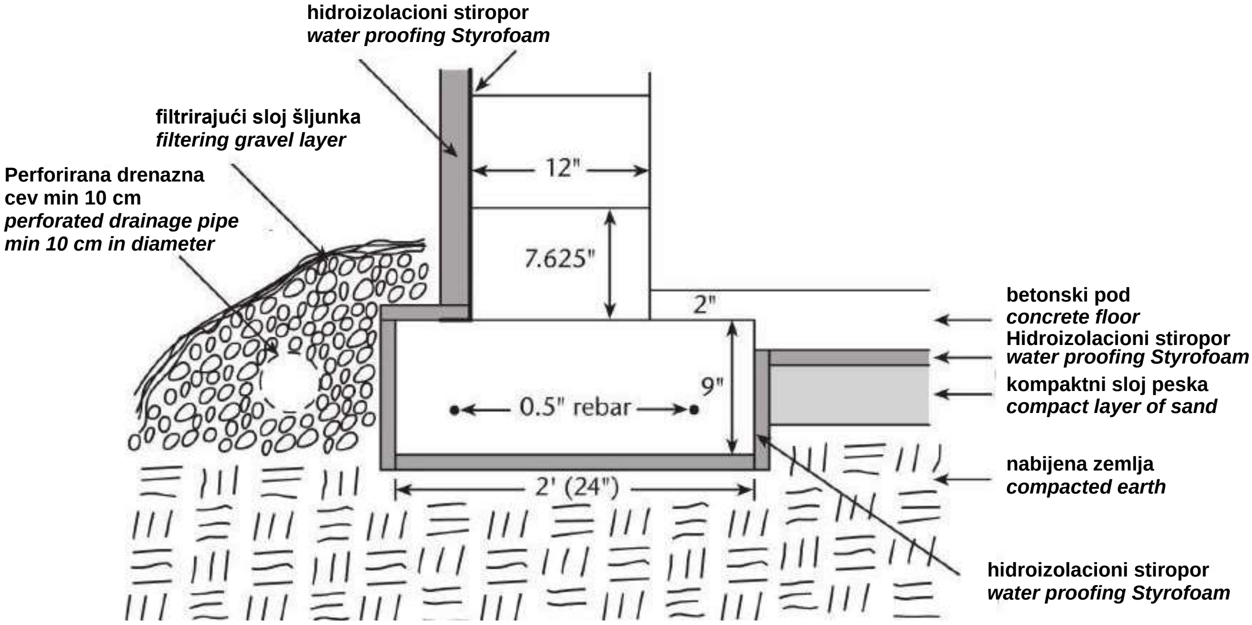


Slika 6. a. Detalj zgrade bez hidroizolacionog sloja; b. Detalj zgrade sa hidroizolacionim slojem
Figure 6. a. A detail of the building without the waterproofing layer; b. A detail of the building with the waterproofing

Istraživanja energetske efikasnosti podrazumevaju pre svega utvrđivanje parametara kojima se dolazi do ocene povoljnosti izgradnje podzemnih u odnosu na nadzemne objekte, ali i nasutih u odnosu na ukopane objekte u terenu. U zavisnosti od unetih parametara i karakteristika objekata koji se istražuju, mogu se dobiti i različiti rezultati. Ipak, značajno je istaći da su podaci koji se dobijaju egzaktni i dobijeni računskim putem, jer se koriste specijalizovani softveri koji su upravo namenjeni takvim proračunima. Pored podataka o toplotnoj sprovodljivosti, energetska efikasnost zavisi i od niza drugih faktora, koji zbirno utiču na ocenu o energetskoj kategoriji podzemnog stambenog objekta, kao što se vidi iz Tabele 1.

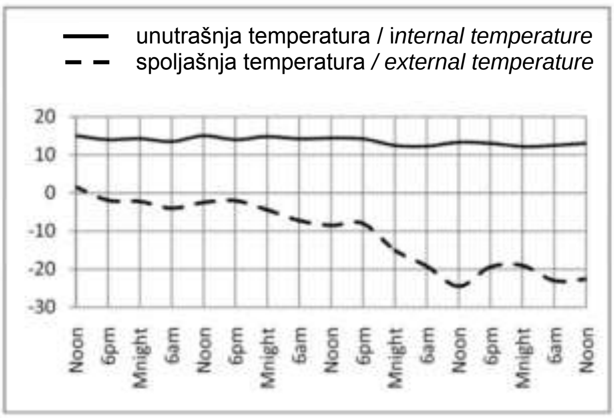
A survey of the geological layers of earth, along with information on the stability and layers of ground waters can prevent potential negative situations during the building service life.

Research of energy efficiency, comprise the primarily identification of parameters used to assess advantages of construction of earth sheltered structures over the surface structure, but also of buried structures in comparison with envelope/true underground structures. Depending on the input parameters and characteristics of researched structures, different results can be obtained. Yet, it is interesting to point out that the obtained data are exact and computational, obtained using specialized software. In addition to data about thermal conductivity, energy efficiency depends on a number of other factors, which cumulatively affect the assessment of energy category of an underground housing building as could be seen in Table 1.



Slika 7. Detalj preseka kroz temeljnu zonu zida podzemnog objekta
Figure 7. A detail of the cross section through the foundation zone of the underground structure wall

Slika 8. a. Gumirana hidroizolaciona membrana; b. Primer termičke izolacije
Figure 8. a. Waterproofing rubberized membrane; b. Thermal insulation example



Slika 9. Odnos promena spoljašnje i unutrašnje temperature na primeru podzemnog objekta u Misuriju, SAD
Figure 9. Relation of variation of external and internal temperature for an underground structure in Missouri, USA

Tabela 1. Stepen energetske efikasnosti u odnosu na tipologiju objekta
Table 1. Degree of efficiency regarding the structure typology

Faktor / *Factor*	Tip podzemnog stambenog objekta / *Type of earth-sheltered dwelling building*			
	nasuti / *covered with the earth*		u terenu / *buried in to the soil*	
Pasivni solarni potencijal *Passive solar potential*	odlican / *excellent*		manje efikasan / *less efficient*	
Termalna stabilnost *Thermal stability*	manje efikasan / *less efficient*		odlican / *excellent*	
Prirodni osvetlja potencijala *Potential of natural light*	efikasan / *efficient*		manje efikasan / *less efficient*	
Zastita od vetra *Wind protection*	manje efikasan / *less efficient*		odlican / *excellent*	
Zastita od buke *Protection against noise*	manje efikasan / *less efficient*		odlican / *excellent*	
Vizuelizacija / *Visualization*	odlican / *excellent*		manje efikasan / *less efficient*	
Klimatski uslovi *Climate conditions*	efikasan za umerenu klimu *efficient for moderate climate*		efikasan za tropsku klimu *efficient for tropical climate*	
Strukturni troskovi *Structural costs*	moderan dizajn *modern design*	tradicionalna *traditional*	moderan dizajn *modern design*	tradicionalna *traditional*
	prosecan *average*	manji troskovi *lower costs*	visoki troskovi *high* *costs*	manji troskovi *lower costs*

Istraživanja termalnih pokazatelja, obavljena na nasutoj podzemnoj stambenoj kući u Misuriju, predstavljaju „parametre praćenja temperature u tom objektu u periodu od četiri dana sa intervalima merenja od šest sati, prilikom kojih je uočena konstantna unutrašnja temperatura, u odnosu na promene spoljne temperature”. [21] Ti podaci konkretno ukazuju na prednosti izgradnje podzemnog nasutog stambenog objekta, posebno uzimajući u obzir činjenicu da za održavanje konstatne temperature prilikom merenja nisu postojali niti su korišćeni sistemi za zagrevanje objekta.

The research of thermal indicators, conducted on the bermed earth sheltered house in Missouri represent "parameters for monitoring the temperature of this object for a period of 4 days with 6-hour measurement intervals, during which a constant internal temperature is detected, in relation to the changes in the outdoor temperature". [21] Those data indicate advantages of construction of bermed housing buildings, especially taking into consideration the fact that for retaining of constant temperature during measuring, no building heating systems were used.

4.2 Podzemni stambeni objekti u kontekstu smanjenja negativnih ekoloških uticaja na životnu sredinu

Od 1900. godine i francuskog arhitekte Ežena Enara [21], koji je objavio manifest u kojem zastupa sistem podzemnog saobraćaja na nekoliko nivoa, preko Frenka Lojda Rajta, koji je 1930. godine uspostavio sistem perifernih gradova [22], do današnjih gradova s neboderima, kao karakterističnim tipovima visokih zgrada XXI veka, došlo se do tačke razvoja zajednice kada je neophodno smanjivanje negativnih uticaja na životnu sredinu, putem modela povratka prirodi i životu izvan zagađene urbane centralne zone, što, najverovatnije, može biti izvedeno neposrednom interakcijom s prirodnim okruženjem. [23] Podzemne stambene strukture u ovom kontekstu mogu se smatrati strukturalnim posrednicima u epicentru takvih interakcija.

Statistički podaci ukazuju na konkretne pokazatelje, prema kojima u gradu danas živi više ljudi, nego u njegovom okruženju ili samoj prirodi. Poslednjih sto godina zabeležen je protok stanovništva i migracija sa imanja na sela, od sela ka gradovima i od gradova ka metropolama. Poznato je da gradovi koriste do 75 posto globalnih izvora energije i proizvode većinu otpadnih materijala. [24] S takvom stopom potrošnje energije,

4.2 Earth sheltered housing buildings in the context of reduction of negative environmental impacts

Since 1900 and the French architect Eugene Hénard [21], who had a manifest advocating the system of underground traffic at several levels, through Frank Lloyd Wright who in 1930 established a system of peripheral cities [22], up to the present day cities with towers, as a characteristic typology of tall building of 21st century we have come to the point of community development where the necessity to reduce negative environmental impacts, through the model of return to nature and life outside polluted urban central zone, can most likely be effected by a direct interaction with natural environment. [23] Underground housing structures in this context can be considered as structural mediators in the epicenter of such interactions.

The statistics indicate concrete data, according to which there are more people nowadays living in the cities, then in their environment, or in nature. In the last one hundred years, the flow of population and migration from the countryside towards villages, and from the villages towards cities and from the cities towards metropolises was recorded. It is known that cities use up to 75 per cent of global energy resources and produce gradovi postaju generatori negativnog uticaja globalnog zagrevanja na planetu Zemlju.

"Posmatrajući statistiku koja upućuje na činjenicu da komercijalni i stambeni objekti emituju 39 posto CO_2 u atmosferu, a da u skoro 70% slučajeva ovi potrošači troše električnu energiju (prema analizi sprovedenoj u SAD), jasno je da su upotreba energije za hlađenje i grejanje od ključnog značaja za uticaj na životnu sredinu okruženja u kojoj se objekti nalaze." [25]

Da bi se uporedili troškovi energije za grejanje koje su imali prosečna nadzemna i podzemna stambena zgrada, napravljena je studija slučaja za zgradu od 135 m^2. Ova studija je potvrdila da arhitektura podzemnog stambenog prostora može ostvariti smanjenje od 72% potrošnje energije u poređenju sa standardnom nadzemnom kućom.

most of the waste materials. [24] With such energy consumption rate, the cities become generators of the negative impact of the global warming on the planet Earth.

"Observing statistics that point to the fact that commercial and residential objects in CO_2 emissions in the atmosphere account for 39 percent, and that in almost 70% of cases these consumers are consumers of electricity (taking an example of analysis conducted in the US), it is clear that the use of energy for cooling and heating are of key importance for the environmental impacts that objects perform in their environment." [25]

In order to compare heating energy costs incurred by an average surface and an underground housing building, a case study for the building of 135m^2 was made. This study confirmed that underground housing architecture can result in the reduction of 72% of energy consumption in comparison with a standard surface house.

Tabla 2. Odnos potrošnje energije nadzemnih u odnosu na podzemne objekte
Table 2. Ratio of energy consumption, standard surface house in comparison with underground structures

merna jedinica ***measurement unit***	**nadzemni stambeni objekti** ***above ground residential buildings***	**podzemni stabeni objekti** ***earth sheltered residential buildings***
zima / *winter* :		
gas / *gas*	2,656.9 m^3 ($65.80)	871.5 m^3 ($27.60)
nafta / *oil*	710 ($129.90)	233 ($42.60)
elektricna energija / electricity	23,157 ($428.80)	7,596 ($191.10)
leto / *summer*:		
elektr. ener. / *electricity*	3,962 ($98.40)	0

Posmatrajući ove podatke, može se zaključiti da je potrošnja energije podzemnog objekta u letnjem periodu za trećinu niža od one za nadzemnu zgradu, dok je potrošnja energije tokom leta nula, što znači veoma mnogo u smislu zaštite životne sredine, jer su uticaji znatno smanjeni. [26] U slučajevima kada su korišćene druge vrste energije za grejanje, kao što su gas ili ulje, može se primetiti da su korišteni tri do četiri puta manje, tako da se ukupna količina energije tokom zime može smatrati finansijskom uštedom, ali predstavlja i ekološki prihvatljiv metod, u pogledu ukupnog uticaja na životnu sredinu. Mogućnost formiranja veštačkih zelenih krovova primenjiva je za obe tipološke grupe stambenih zgrada, tako da se kontekst ekološki prihvatljivog oblika zgrade može pozitivno posmatrati i na ovaj način, uzimajući u obzir činjenicu da zelena krovna struktura utiče na povećanje površine ispod vegetacije ili zadržavanje postojeće vegetacije u građevinskoj zoni.

By observing these data, it can be concluded that the energy consumption in the summer period is for third lower than that of a surface housing building, while the energy consumption in the summer is zero, which means a lot in environmental terms, as impacts are considerably reduced. [26] In cases where other kinds of heating energy were used, such as gas or oil, it can be noticed that they were used 3 to 4 times less, so the total amount of energy during winter can be considered financial saving but also the environmentally acceptable method, regarding the total environmental impact. The potential for formation of artificial green roofs is a possibility of both typological groups of housing buildings, so the context of environmentally acceptable form of buildings can be positively observed separately, taking into account the fact that he green roof structure affects the increase of area under vegetation or retention of the existing vegetation in the construction zone.

5 ZAVRŠNE NAPOMENE

Podzemna arhitektura stanovanja predstavlja energetski efikasnu vrstu objekata. Ovaj specifičan tip stambene izgradnje preporučuje se umesto standardnih nadzemnih zgrada zbog svojih energetski efikasnih svojstava i kvaliteta.

Svi tipološki oblici arhitekture podzemne stambene kuće do sada su bili uglavnom pojedinačni, eksperimentalni napori. Nisu postojali, takođe, ni propisi za efikasan nadzor nad izgradnjom ovih objekata. Za srpsku teoriju i praksu međunarodno iskustvo je od ključnog značaja za dalji razvoj i usklađivanje s proce-

5 CONCLUDING REMARKS

Underground housing architecture represents an energy efficient type of structures. This specific type of housing construction is recommended for the standard surface buildings because of its more energy efficient properties and its quality.

All the typological forms of underground housing architecture up to date were mostly individual, experimental endeavors. Construction had no regulations for efficient monitoring of construction of these structures. For Serbian theory and practice international experience is crucial for further durama prilikom projektovanja i izgradnje podzemnih stambenih objekata, iz sledećih razloga:

- u većini proučenih primera, podzemni pristup izgradnji ukazao je na značajne uštede koje su ostvarili takvi građevinski modeli;
- kombinovanje zelenih krovova sa strukturama nasutog elevacionog tipa najčešći je građevinski model u Srbiji;
- izgradnja ovih objekata umnogome je izražena u Vojvodini koja već ima tradiciju stvaranja takvog tipa zgrada;
- prosečne temperature u podzemnim stambenim zgradama kreću se od 16 do 20 stepeni Celzijusove skale;
- zeleni krovovi, sa srednjim zahtevima, poluintenzivni, procenjuju se kao tip koji može obezbediti dobre efekte tokom celog leta [27] [28], tako da se ovaj tip zelenog krova može smatrati adekvatnim za objekte nasutog elevacionog tipa;
- maksimalne uštede ostvaruju se sa zelenim slojem krovnog vrta, pri čemu je sloj tla debeo od 100 do 900 mm; - optimalna vrsta krovnog vrta ima bujnu vegetaciju - sloj tla debljine od 300 mm sa grmovima iznad može uštedeti do 15% godišnje potrošnje energije, odnosno 79% uštede energije za hlađenje zgrade;
- krovne bašte smanjuju toplotni tok za 52–57% u odnosu na keramičke ili metalne krovove; [29] [30]

– za potrebe održavanja konstantne temperature tokom zimskog perioda neophodno je projektovati i alternativne hibridne sisteme za snabdevanje objekata potrebnom količinom energije i to u kombinaciji vetro-generatora i solarnih kolektora.

Prednosti izgradnje podzemne stambene arhitekture jesu smanjenje efekata toplotnih ostrva i ušteda energije u zgradama, jer podzemni stambeni objekti smanjuju godišnju potrebu za grejanjem i hlađenjem. [31] S druge strane, s obzirom na činjenicu da je sunčevo zračenje daleko većeg intenziteta leti, podzemni stambeni objekti igraju važnu ulogu u redukovanju energetske potrošnje za hlađenje u odnosu na standardne nadzemne termički izolovane objekte.[32]

Na osnovu opisanih istraživanja, mogu se formulisati preporuke za projektovanje objekata nasutog elevacionog tipa. Ovu preporuku treba povezati s pravilnim izborom korišćenih materijala i tehnikama gradnje, čime se generišu rezultati koji se primenjuju prilikom procene izbora novih dizajnerskih kreacija, kojima se obogaćuje tipologija podzemnih stambenih objekata.

Prikazana istraživanja nesumnjivo dokazuju da podzemni stambeni objekti predstavljaju adekvatan i veoma kvalitetan model stanovanja, koji se dobro prilagođava i može se primenjivati na teritoriji cele Republike Srbije.

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development and harmonization with procedures during designing and construction of underground housing structures for the following reasons:

- In almost all presented examples, the underground construction approach indicated considerable savings accomplished by such construction models;
- Combining of green roofs with the bermed elevation structures is the most common construction model in Serbia;
- Construction of these structures is to a great extent present in Vojvodina, which already has a tradition of construction of such buildings;
- Average temperatures in underground housing buildings range between 16 -20 degrees Celsius;
- Green roofs, with medium requirements, semi-intensive, are assessed as the type which can provide good effects during the entire summer [27] [28] so this type of the green roof can be observed as an adequate for bermed underground housing;.
- Maximum savings are accomplished with a green layer of the roof garden, where the soil layer is from 100 to 900 mm thick; - the optimal type of roof garden has bush vegetations – 300 mm thick layer of soil with shrubbery can save up to 15% of yearly energy consumption, that is, 79% saving of building cooling energy;
- Roof gardens reduce the heat flux for 52–57% in comparison to ceramic or metal roofs. [29] [30]

In order to maintain constant temperature during winter period, it is necessary to design alternative hybrid systems for supplying structures with the necessary amount of energy, as a combination of wind power generators and solar panels;

The advantages of implementation of underground housing architecture are the reduction of heat islands effects and saving energy in buildings (underground housing buildings reduce yearly heating and cooling energy demands), [31]. On the other hand, regarding the fact that solar radiation has considerably higher intensity in the summer, underground housing buildings play an important role in the reduction of the cooling energy consumption in comparison to the standard surface thermally insulated buildings. [32]

On the basis of the described research, the recommendations for the design of the bermed elevation type buildings can be formulated. This recommendation should be related to the proper choice of the used materials and construction techniques, which generate the results which are implemented on the assessment of new designs which enrich typology of underground housing structures.

The conducted research prove that underground housing structures represent an adequate and very good quality housing model which is very adaptable and can be implemented across the entire territory of the Republic of Serbia.

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REZIME

PODZEMNE STAMBENE ZGRADE U KONTEKSTU ENERGETSKI EFIKASNIH GRAĐEVINA

Aleksandar RUDNIK MILANOVIĆ
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Podzemna stambena arhitektura predstavlja energetski efikasnu vrstu struktura. Ova specifična vrsta stambene izgradnje preporučuje se više od standardnih nadzemnih zgrada zbog svojih energetski efikasnih svojstava i kvaliteta.

Svi tipološki oblici arhitekture podzemnih stambenih zgrada do sada su uglavnom bili pojedinačni, eksperimentalni napori. Izgradnja podzemnih stambenih objekata uglavnom nema propise za efikasan nadzor nad izgradnjom ovih specifičnih tipova objekata. Za srpsku teoriju i praksu, međunarodno iskustvo, prikazano i obrazlagano u članku, presudno je za dalji razvoj i usklađivanje s procedurama prilikom projektovanja i izgradnje podzemnih stambenih objekata u nas.

Kada se međunarodno iskustvo posmatra u svetlu kvalitativnih inicijalnih razloga za promenu čitavog skupa dokumenata vezanih za planiranje i područja izgradnje, može se očekivati promocija široko prisutne podzemne stambene arhitekture, s jednim od primarnih ciljeva sadašnjeg vremena, koji se odnosi na planiranje i izgradnju pojedinačnih zgrada, ali i čitavih naselja, u kojima se ova specifična tipologija stambenog prostora može realizovati radi stvaranja budućih energetski efikasnih naselja u Republici Srbiji.

Ključne reči: podzemno stanovanje, energetski efikasni objekti, uticaj biodiversiteta, osnovni uticaji na podzemne objekte, smanjenje negativnih uticaja okruženja.

SUMMARY

EARTH-SHELTERED HOUSING BUILDINGS IN THE ENERGY EFFICIENT STRUCTURES CONTEXT

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Underground housing architecture represents an energy efficient type of structures. This specific type of housing construction is recommended for the standard surface buildings because of its more energy efficient properties and its quality.

All the typological forms of underground housing architecture up to date were mostly individual, experimental endeavors. Construction had no regulations for efficient monitoring of construction of these structures. For Serbian theory and practice international experience reviewed in the article is crucial for further development and harmonization with procedures during designing and construction of underground housing structures.

When the international experience is viewed in the light of qualitative initial reasons for the change of an entire set of documents related to planning and construction areas, one can expect promotion of a more widely present underground housing architecture, with one of the primary goals of the present times, which is planning and construction of individual buildings, but also of entire settlements, where this housing typology can be implemented with an aim of creation of future energy efficient settlements in the Republic of Serbia.

Key words: earth-sheltered housing, energy efficient structures, impact of biodiversity, basic impacts on earth-sheltered structures, reduction of negative environmental impacts.

SAKRALNI OBJEKTI OD DRVETA U BOSNI I HERCEGOVINI

WOODEN SACRAL OBJECTS IN BOSNIA AND HERZEGOVINA

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1 UVOD

Bosna i Hercegovina bogata je šumom s različitim listopadnim i crnogoričnim drvećem koje je pogodno za izgradnju konstrukcija, a drvo kao građevinski materijal našlo je svoju masovnu upotrebu u najstarijim periodima. Brvnara[1] je tipični oblik objekta koji je građen ne samo u brdskim područjima, nego i u Sloveniji te u zapadnoj i centralnoj Srbiji. Nije iznenađujuće da je mnogo crkava brvnara izgrađeno od drveta te da džamije imaju drvene munare.

Istorijski značaj sakralnih drvenih konstrukcija tek je u skorije vrijeme prepoznat u Evropi, a u Bosni i Hercegovini u posljednjoj deceniji. U tom smislu, početna tačka jeste prepoznavanje vrijednosti i značaja objekta, te njihovog odgovarajućeg konstrukcijskog ponašanja koji će biti osnova prilikom razmatranja različitih postupaka konzervacije i restauracije. Drveni sakralni objekti dugi niz godina bili su zapostavljeni i bili su stavljeni na marginu da budu zaboravljeni. Značaj ovih objekata prepoznat je tek u posljednjim decenijama što nije bilo slučaj prije nekoliko decenija, kada su ovi objekti ili rekonstruisani na neadekvatan način (što se još uvijek dešava u pojedinim slučajevima), ili u potpunosti uništeni, ili su kao rezultat neodržavanja doživjeli značajno propadanje. Nepotrebne popravke veoma često dovode do gubljenja istorijskog materijala i kao posljedica gubi se istorijski karakter i autentična vrijednost naslijeđa.

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[1] Tipična kuća u Bosni i Slavoniji, zapadnoj i centralnoj Srbiji.

1 INTRODUCTION

Bosnia and Herzegovina is rich in forests with different deciduous and coniferous trees which are suitable for construction, and timber as a construction material found its massive usage in the oldest periods of time. Brvnara[1] (Log cabin) is a typical building form which was built not only in the hilly areas but as well in Slovenia and in West and Central Serbia. It is of no surprise that many of the log cabin churches are made of wood and that mosques are with wooden minarets.

The historical significance of sacral wooden structures has only recently been recognized in Europe and in Bosnia and Herzegovina only in the last decade. In that respect, the starting point is recognizing the value and significance of structures and then their proper structural behavior which will be the basis for considering different conservation and restoration approaches. Wooden sacral objects for a long time have been abandoned and were put on a margin to be forgotten. The significance of these structures has been recognized in the last decades which was not the case several decades ago, when these structures were either reconstructed in an inappropriate manner (which is still happening in some occasions), or completely destroyed, or as a result of non-maintenance they have significantly deteriorated. Unnecessary repairs very often lead to the loss of historic material and consequently the loss of

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[1] Typical house in Bosnia and Slavonia, West and Central Serbia.

Dakle, može se konstatovati da nekada neznanje ljudi dovodi do gubitka autentičnosti objekata kulturno-istorijskog naslijeđa. Nakon utvrđivanja stanja konstrukcije na licu mjesta neophodno je izvršiti modeliranje i proračun ovih konstrukcija uzimajući u obzir karakteristike poprečnog presjeka, uslove veze, te veliki broj nepoznanica. S obzirom na to što se radi uglavnom o linijskim elementima drvenih konstrukcija, to može dovesti do netačne ideje da se radi o jednostavnim elementima što kao posljedicu može imati neadekvatno modeliranje i analize. Ovo je nešto što treba imati na umu.

Drveni sakralni objekti koji su preživjeli nisu prepoznati kao prenos spoznaja o razvoju konstruktivnog projektovanja i prakse zanatskih radnika. Ali, ovo je u nekim slučajevima dovelo do drugog ekstrema a to je „lažiranje" originalnosti konstrukcija. Sakralni objekti kao što su džamije s drvenim munarama i crkve brvnare, bilo da su pravoslavne ili katoličke, jesu reprezenti drvenih objekata u Bosni i Hercegovini, koji nisu samo međusobno prepleteni nego su čvrsto povezani stambenom arhitekturom što ih čini još interesantnijim. Ove konstrukcije imaju zajedničku arhitektonsku stambenu bazu a razlikuju se u nekim specifičnim svojstvima. Zajedničke elemente predstavlja tehnika izvođenja temelja od drvenih greda, zatim dva načina izvođenja zidova od dugačkih brvana, kao i ista konstrukcija krovova, a sve je preuzeto iz domaće tradicije gradnje drvetom. Razlika je jedino u tome što se kod drvenih džamija u Bosni kao pokrov primjenjuje samo šindra, a kod crkava sitni drveni klis. Ove zajedničke karakteristike mogu se objasniti radom domaćih majstora. U XIX stoljeću domaći majstori grade jednostavne crkve bez ukrasa i u tlocrtnom smislu su jako slične seoskim kućama.

historic character and loss of authentic value of the heritage. So, it can be stated that sometimes peoples' ignorance leads to the loss of authenticity of cultural heritage structures. Once the state of the structure is determined on the site it is necessary to make adequate modeling and calculation of these structures taking into account section properties, support conditions, and a vast number of uncertainties has to be taken into account. Due to the fact that here one is dealing mainly with linear elements of the timber structures, it can lead to a false idea of their simplicity, resulting in inadequate modelling and analysis. This is something that should be kept in mind.

The surviving wooden sacral objects are not recognized as passing on evidence of the development of structural design and craftsman practice. However, this on some occasions has led to the other extreme and that is "faking" the originality of the structures. Sacral objects such as mosques with wooden minarets and log cabin churches either orthodox or catholic are representatives of wooden structures in Bosnia and Herzegovina, which are not only interconnected but connected by the residential architecture which makes them even more interesting. These structures have a common architectural residential basis on one hand and they still differ is some specific features. The construction technique of foundation made out of wooden beams represent a common feature; two possible construction techniques of building walls of long logs, construction of the roof are the same, and all of this is actually taken from the local domestic traditional wood building. The only difference is that mosques with wooden minarets in Bosnia use only shingles and churches use tinny wooden plagues. These common characteristics can be explained by the works of the domestic carpentry workers. In the XIX century domestic workers build simple churches without any decorations and regarding the layout, they are very similar to the rural houses.

2 DŽAMIJE S DRVENIM MUNARAMA NA TLU BIH

Sama lokacija Bosne i Hercegovine, koja je bila smještena na perifernom području Osmanskog carstva, igrala je značajnu ulogu i imala je veliki uticaj na arhitekturu džamija. Dodatno sveobilje drveta i kamena, kao i konfiguracija zemljišta i tradicije građenja, imali su značajan uticaj. Džamije u Bosni i Hercegovini mogu se podijeliti u dvije osnovne grupe. Džamije s krovom na četiri vode s drvenim ili kamenim munarama predstavljaju prvu grupu. Najmanji broj istraživanja posvećen je ovim konstrukcijama mada one odražavaju tipičnu bosansku arhitekturu i manje se mogu povezati sa osmanskom a više s lokalnom tradicionalnom arhitekturom. Zasvođene džamije predstavljaju drugu grupu, pri čemu one nisu mnogobrojne (iz registra iz 1933. godine samo 3.2% predstavlja ovu vrstu konstrukcija), međutim, propraćene su značajnim istraživanjima zbog svoje veličine ili monumentalnog izgleda. Ove objekte karakterizira tzv. osmanska arhitektura [1], [2], koja za objekte u BiH podrazumijeva pojednostavljenu matematičku

2 MOSQUES WITH WOODEN MINARETS IN BOSNIA AND HERZEGOVINA

It is the position of Bosnia and Herzegovina, located in the peripheral region of the Ottoman Empire, that played an important role and had a great influence on the architecture of mosques. Additionally, the abundance of wood and stone, as well as the land configuration and the building traditions had a great impact. The mosques in Bosnia and Herzegovina can be divided into two basic groups. The mosques with four-sided roofs (broached roofs) with wooden or stone minarets represent the first group. The smallest amount of research was conducted on these structures. Even though they reflect the typical Bosnian architecture and cannot be entirely connected to the Ottoman architecture but to the local traditional architecture. The domed mosques form the second group, which are not so numerous (from 1933 registry only 3.2% were these type of structures); however major research has been done due to their size or monumental appearance. These structures are characterized by the so-called Ottoman architecture [1], [2], which for the kompleksnost geometrijskog sklada osmanske imperije, ali i zadržanu tradicionalnost zasnovanu na stambenoj arhitekturi bosanskog prostora. Interesantno je primjetiti da su najpriznatije džamije, kojima su posvećena značajna istraživanja, one zasvođene, tj. s kupolama [1], [2], iako brojčano nisu zastupljene u velikom broju. Donekle se može razumijeti da su ove vrste džamija bile najinteresantnije za istraživanja sa arhitektonskog gledišta; međutim, nezainteresiranost za džamije s drvenim munarama ne može se prihvatiti jer iz istorijskog aspekta, aspekta zaštite, kao i stručno-građevinskog gledišta, ove konstrukcije predstavljaju integralni dio urbanog i ruralnog okruženja Bosne i Hercegovine već više od pet stoljeća.

Bosna i Hercegovina specifična je po broju džamija s drvenim munarama, preostalih iz doba osmanske imperije u odnosu na ostala područja zapadnog Balkana.

Međutim, ove vrste džamija bile su neprimjetne i na neki način ostale su u sjeni monumentalnih džamija, a nazivane su „drvenim" ili „ruralnim" džamijama. Ipak, postojalo je veoma mnogo ovih objekata koji su bili smješteni ne samo u malim mjestima nego i u gradovima kao što su Sarajevo, Tuzla, Banja Luka i tako dalje, te svakako zaslužuju pažnju kao kulturno-istorijski spomenici.

Češki arhitekt Josef Pospíšil bio je prvi koji je priznao arhitektonsku vrijednost ovih objekata, a radio je u Sarajevu od 1908. do 1918. godine i istraživao kulturno naslijeđe u Bosni. Do 1990. godine u Bosni su postojale 992 džamije, i od toga 770 (77,6%) bilo je s drvenim munarama, 186 (18,7%) s kamenim munarama i 36 (3,6%) bez munara, kao karakterističnog arhitektonskog obilježja. S druge strane, u Hercegovini je ukupno bilo 128 džamija, a od toga samo 16 (2,5%) s drvenim munarama, 73 (57%) s kamenim munarama i 39 (30%) bez munara [3]. Prema tome, u Bosni i Hercegovini 70% od svih džamija predstavljaju džamije s drvenim munarama. Najveći broj džamija s drvenim munarama bio je u Banjalučkom muftijstvu[2], 307 (40%), zatim u Sarajevskom muftijstvu, 254 (33%), dok ih je najmanje bilo u Tuzlanskom muftijstvu – 205 (27%).

2.1 Osnova konstrukcije objekta džamije s drvenim munarama

U konceptualnom smislu postoje dva tipa džamija s drvenim munarama u Bosni i Hercegovini. Prva vrsta džamija su džamije s jednim prostorom za molitvu, a drugi tip su džamije sa stubovima kao posebnim konstruktivnim elementima unutar džamije. Površina džamija je različita i kreće se od malih, sa 25 m^2 osnove do velikih koje dostižu 460 m^2.

Tipična konstrukcija džamije s jednim prostorom prikazana je na slici 1. Uobičajeno je da krov na četiri vode pokriva sofu[3] izgrađenu od drvenih stubova i

[2] Administrativno teritorijalno područje koje je pod nadzorom muftije.

[3] Istureni dio u vidu balkona.

buildings in B&H implies the simplified mathematical complexity having a geometrical harmony of the Ottoman Empire, but retained its traditionalism based on the residential architecture of the Bosnian territory. It is interesting to note that mostly recognized and investigated were domed mosques [1], [2] even though numerically they are non-numerous. From the architectural aspect, it is to a certain extent understandable that these types of mosques were the most interesting for research; however, the lack of interest for the mosques with wooden minarets cannot be accepted because from the historical, preservation and engineering aspect of view, these structures are an integral part of urban and rural environment of Bosnia and Herzegovina already for more than five centuries.

Bosnia and Herzegovina is specific regarding the number of mosques with wooden minarets remaining from the Ottoman Empire in respect to other areas of the Western Balkans.

However, these types of mosques were rather unnoticed and in a way stayed in the shadow of monumental mosques, and were referred to as "timber" or "rural" mosques. Yet, these types of structures were numerous, and placed not only in the small "rural" places but in cities like Sarajevo, Tuzla, Banja Luka etc. and they indeed deserve attention as cultural heritage monuments.

Czech Architect Josef Pospíšil was the first one who acknowledged the architectural value of these structures. He worked in Sarajevo from 1908 to 1918 and conducted research on cultural heritage in Bosnia. Until 1990 in Bosnia there were 992 mosques, and from that 770 (77,6%) were with wooden minarets, 186 (18,7%) with stone minarets and 36 (3,6%) without minarets, as a characteristic architectural feature. On the other hand, in Herzegovina, there were in total 128 mosques, and from that, only 16 (12,5%) with a wooden minaret, 73 (57%) with stone and 39 (30%) without minaret[3]. So, in Bosnia and Herzegovina, 70% of all the mosques were with wooden minarets. Most of the mosques with the wooden minarets were in Banja Luka muftiate[2] 307 (40%), then in Sarajevo muftiate 254 (33%), while the smallest was in Tuzla muftiate 205 (27%).

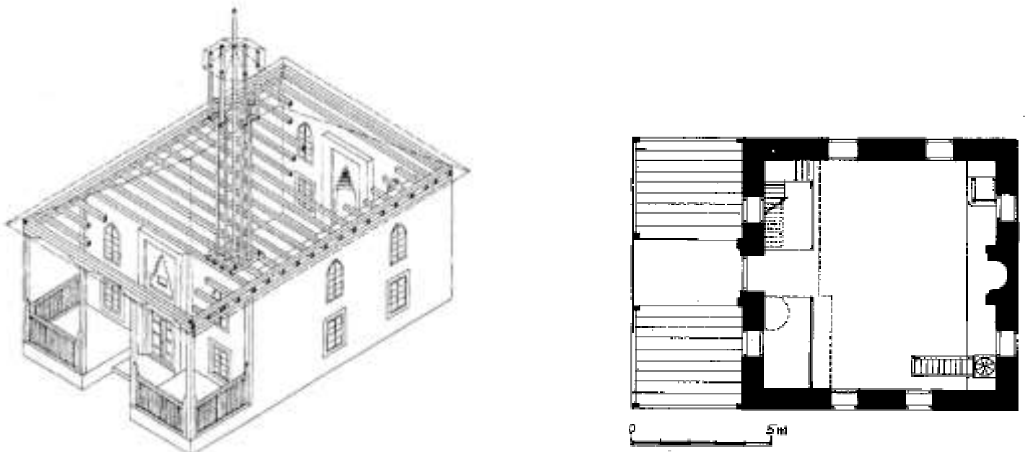
2.1 The layout of the mosque with wooden minarets

Conceptually mosques with wooden minarets in Bosnia and Herzegovina are of two types. The first type is mosques with a single-space room for prayer and the second type is mosques with the pillars as a specific construction element inside the mosque. The area of the mosques is various from very small ones, of 25 m^2in layouts, to the biggest ones reaching 460 m^2.

A typical construction of a mosque with a single space is shown in Figure 1. Usually, the four-sided roof covers the sofa[3] made of wooden pillars and the space

[2] Is an administrative territorial entity under the supervision of a mufti.

[3] put out a part like a balcony.



Slika 1. 3D izgled džamije s drvenom munarom i tlocrt tipične džamije s jednim prostorom [3]
Figure 1. 3D view of a mosque with a wooden minaret and layout of a typical mosque with single space[3]

mjesto za molitvu koje je ili kvadratno ili pravougaono u osnovi. Pored korištenja različitih materijala za izgradnju i arhitektonskih oblika ovih elemenata, razvio se veliki broj mogućnosti rasporeda različitih religioznih elemenata. Stari krovovi u Bosni su uglavnom pokriveni drvenom šindrom, kao dijelom tradicionalne arhitekture, a koji su kasnije u XIX stoljeću zamijenjeni crijepom. Međutim, u Hercegovini, krovni pokrivači su od kamenih ploča/šindri, što je rešenje direktno povezano s podnebljem i tradicijom.

Ovo je karakteristika bosanske islamske arhitekture koja je široko rasprostranjena u gradovima i selima. Interesantna je činjenica da je u Sarajevu i Tuzli više od 50% džamija ovog tipa, dok je u Travniku više od 66%, a u Banja Luci više od 75%. U Bosanskoj Krajini skoro sve džamije su s drvenim munarama.

Interesantno je naznačiti da je kod džamija s drvenim munarama naglasak bio na formiranju što većeg prostora za molitvu a ne na monumentalnosti konstrukcije. U tom smislu, položaj mujezinskog mahfila[4] određen je tako da se minimizira upotreba građe i da se obezbijedi što veći prostor za molitvu. U Sulejmaniji džamiji u Travniku, mujezinski mahfil je izgrađen duž tri zida (slika 2) ali ova džamija je s kamenom munarom, kao i džamija u Bužimu (2.2.4.).

Zidovi džamije s drvenim munarama napravljeni su od različitih materijala koji su bili korišteni prilikom izgradnje stambenih objekata, od drveta, čerpića do kamena. Ista filozofija je korištena i kod starih crkava brvnara (poglavlje 3). Kod džamija s drvenim munarama najzanimljiviji je način oslanjanja drvenih munara na drvene tavanice, tj. njihova veza s njima, kakva im je konstrukcija, ukrućenja i visina munare u odnosu na dio objekta koji je predviđen za molitvu. Drvene munare mogu biti oslonjene na mujezinski mahfil, prislonjene uz džamiju, s tim što je drvena munara postavljena na kameni postament koji ima visinu zidanog dijela objekta (npr. stara džamija sa haremom u Špionici Donjoj kod

[4] Posebno uzdignuti prostor/platforma u džamiji, obično smještena nasuprot položaja munare.

for prayer is either square or rectangular in plan. A large number of variations regarding the inner positioning of different religious elements has been developed in addition to the usage of different materials for construction and architectural features of these elements. The old roofs in Bosnia were usually covered with wooden shingles, as a part of the traditional architecture, and later on, in the 19^{th} century replaced by tiles. However, in Hercegovina, the roofs were covered with stone plates, which is directly connected to the geographical area and tradition.

This is a characteristic of Bosnian Islamic architecture which is widely spread in towns and villages. An interesting fact is that in Sarajevo and Tuzla, more than 50% of the mosques are of this type, while in Travnik more than 66% and in Banja Luka more than 75%. In Bosanska Krajina almost all mosques are with wooden minarets.

It is interesting to state that the emphasis lies in creating as large space as possible for the prayer and not on the monumentality of the structure. In this respect the position of the Müezzinmahfili[4], is formed in a way to use minimum construction elements and obtain maximum space for the prayer. In Sujelmanpashic mosque in Travnik, Müezzinmahfili is constructed along three walls (Figure 2) but it has a stone minaret, as well as the mosque in Bužim(section 2.2.4.).

The walls of the mosques with wooden minarets are made of different materials which have been used for the construction of residential buildings, from timber, adobe to stone. The same philosophy was used for the old log cabin churches (chapter 3). The most interesting feature for the mosques with wooden minarets is the way the wooden minarets are supported by the wooden ceilings, i.e. their connection to them, their structure type, stiffeners and the height of the minaret in relation to the part of the object intended for prayer. Wooden minarets can rest on the Müezzinmahfili, be aligned to the mosque,

[4] Is a special raised platform in a mosque, usually opposite the minaret.

Slika 2. Sulejmanija džamija u Travniku – poznata kao Šarena džamija [4]
Figure 2. Sulejmanpashic mosque in Travnik-known as "The colorful mosque"[4]

Srebrenika). Visina drvenih munara uglavnom se kreće od 7 do oko 20 metara. Tijelo drvene munare veoma često je dvanaestrostrano, što odgovara broju stavaka u pozivu na molitvu koje mujezin izgovara s munare. Konstrukcija drvenog krovišta koju čine rogovi (12x13 cm) postavljeni na međurazmacima od cca 90 cm i grebene grede, oslanja se na podrožnice (rubnu gredu vjenčanicu i središnju podrožnicu). Podrožnice se oslanjaju na višestruke uspravne stolice (stubove cca 13x13 cm). Stubovi stolice, na visini od cca 1,20-1,30 m (mjereno od poda tavana) ukrućeni su s kliještima 2x8/10 cm, a same stolice se oslanjaju na vezne grede (presjeka cca 16x16 cm), postavljene u podužnom pravcu. Unakrsno u odnosu na vezne grede, a s njihove donje strane, pričvršćene su na osovinskim međurazmacima od cca 90 cm, stropne grede. No, svaka džamija ima svoje specifičnosti. Materijali i tehnike gradnje isti su kao i za stambene objekte [5].

with a wooden minaret set on a stone construction that has the height of the masonry part of the building (e.g. the old mosque with a harem in ŠpionicaDonja near Srebrenik). The height of the wooden minarets varies from 7up to approximately 20 meters. The body of the wooden minaret is very often twelve-sized, which corresponds to the number of mosque-goers in the call for pray that the Müezzin pronounces from the minaret. The structure of the wooden roofing is made of rafters (12x13 cm) placed at a distance of cca 90 cm and the ridge beams which is placed on purlin (tie beam and central rafter).Rafters are placed on multiple upright constructions (pillars approximately 13x13 cm). The pillars are at a height of approx. 1,20-1,30 m (measured from the floor of the attic), stiffened with pins 2x8 / 10 cm, and the upright construction is supported by the connecting beams (cross section approximately 16x16 cm), placed in the longitudinal direction. In relation to the connecting beams, and from their bottom, they are attached to the axle spacing of approximately 90 cm, ceiling beams. But each mosque has its own specificities. The materials, construction techniques are the same as for residential architecture [5].

Slika 3. Karta rasporeda objekata od drveta, čerpića i kamena [5]
Figure 3. Map of wooden, adobe and stone houses [5]

Na karti na sjevernom dijelu (slika 3) kuće su izgrađene od čerpića [5]. Centralni dio zemlje je bio, i još uvijek jeste bogat drvetom, što je rezultiralo da su kuće u ovom području izgrađene od drveta, dok su u južnom dijelu, u Hercegovini, kuće uglavnom izgrađene od kamena. Čerpić se takođe nalazi u centralnom dijelu zemlje i u velikom broju slučajeva zamijenio je objekte od drveta u centralnom dijelu, ili je napravljena kombinacija u obliku bondruka. Ovo sve upućuje na karakteristike džamija u pojedinim područjima.

2.2 Primjeri džamija s drvenim munarama

2.2.1 Sarajevo: Golobrdica džamija

Mahala[5] gdje je smještena džamija prvi put se spominje u 1528. godini, što znači da je džamija izgrađena nekoliko godina ranije, jer su prvo izgrađivane džamije a onda su okolo formirana naselja. Radi se o jednoprostornoj džamiji unutarnjih dimenzija 9,80x7,40 m. Munara je napravljena bez stubova. Specifičnost ove munare je da se ona oslanja na mujezinski mahfil. Godine 1982. na samoinicijativu građana koji idu u ovu džamiju, munara je pomjerena na prethodno pripremljene temelje na sjeverni vanjski zid džamije, i na taj način poremećena je autentičnost džamije. Ova intervencija je ugrozila jednu od rijetko očuvanih specifičnih munara grada Sarajeva, munara koja je bila smještena unutar konstrukcije krova i koja se oslanjala na mujezinski mahfil. Ovo je jasan pokazatelj neadekvatnih intervencija na povijesnim spomenicima i u suprotnosti je s Venecijanskom konvencijom [6].

Tokom rekonstrukcije munare džamije, jedan od osnovnih zadataka bio je da se munara povrati u prvobitni položaj, tako da se džamiji vrati njen autentičan izgled [6]. Nakon demontaže jednog dijela tavanice krova kao i dijela konstrukcije mahfila, a na osnovu vidljivih ostataka, ustanovljen je autentičan položaj munare (slika 4). Munara (visine 7 metara) vraćena je na prvobitno mjesto, u unutrašnjost džamije, na mujezinski mahfil (slika 4). Ovaj posao je izvršen 2009. godine [7].

Jedna od karakteristika sarajevskih džamija s drvenim munarama jeste blago nagnuti krov kojem je kao pokrivač najpogodniji bio ćeremit. Za razliku od sarajevskog tipa, drugom tipu džamije s drvenom munarom pripadaju objekti strmih krovnih ravni, s drvenim pokrovom. Osim razlika u krovu, njihove munare imaju manje istaknute galerije s višim parapetom, prema tome s manjim otvorom. Ovoj grupi pripadaju džamije u Tuzli.

[5] Znači „susjedstvo” ili „četvrt”, dio ruralnog ili urbanog naselja, datira iz vremena Osmanskog carstva.

On the map shown in figure 3, the houses in the northern part are made of adobe [5]. The central part of the country was rich and still is, in the wood making, which resulted in numerous houses made of wood, while in the southern part, Herzegovina, houses were mainly made of stone. Adobe is as well seen in the central part of the country and in many occasions replaced the wooden building in the central part, or a combination has been made in the form of “bondruk” construction. This all points to the characteristics of the mosques in certain areas.

2.2 Examples of mosques with wooden minarets

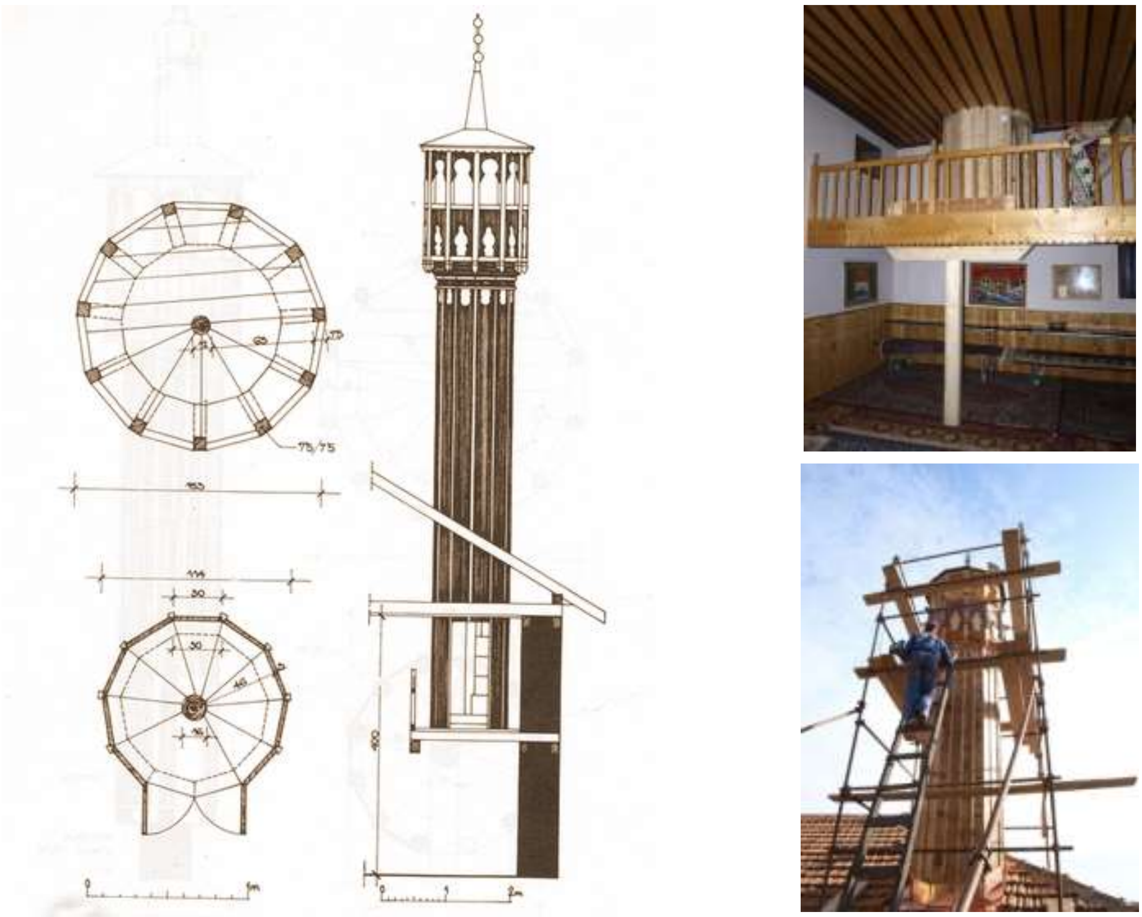
2.2.1 Sarajevo: Golobrdica Mosque

The mahala[5] where this mosque is located is mentioned for the first time in 1528, meaning that the mosque was built before that year, as the mosques were built initially and then the settlements were formed around it. It is one space mosque having inner dimensions of 9,80x7,40 m. The minaret is made without pillars. The specific feature of this minaret is that it is supported by the müezzinmahfili. In 1982, as a self-initiated intervention by the local people going to the mosque, the minaret was moved to a prepared foundation on the northern outer wall of the mosque, thus disrupting its authentic appearance of the mosque. This intervention jeopardized one of the rarely preserved specific minarets of the city of Sarajevo, a minaret that was placed inside the roof structure and supported on the müezzinmahfili. This is a clear indication of inadequate intervention on the historical monuments and is contradictory with the Venice-Charter [6].

During the reconstruction of the mosque's minaret, one of the basic tasks was to restore the minaret to the original position, so that the mosque would return its authentic appearance[6]. After disassembly of one part of the roof ceiling, as well as part of the mahfil structure, and on the basis of the visible remains, an authentic position of the minaret was established (Figure 4). The minaret (of 7 meters height) was returned to the original place, inside the mosque, on the müezzinmahfili (Figure 4). This work was done in 2009 [7].

One of the characteristics of Sarajevo’s mosques with wooden minarets is a slightly sloping roof, which was the most suitable roof cover. Contrary to the Sarajevo-type, the second type of the mosque with a wooden minaret belongs to the objects with steep roofs, with a wooden lid. Apart from the differences in the roof, their minarets have a less prominent gallery with a higher parapet therefore with a smaller opening. Mosques in Tuzla region belong to this group.

[5] Means "neighborhood" or "quarter", a section of a rural or urban settlement, dating to the times of the Ottoman Empire.



Slika 4. Džamija Golobrdica Sarajevo i oslanjanje drvene munare na mujezinski mahfil, izgradnja drvene munare 2009, te unutrašnjost objekta [7]

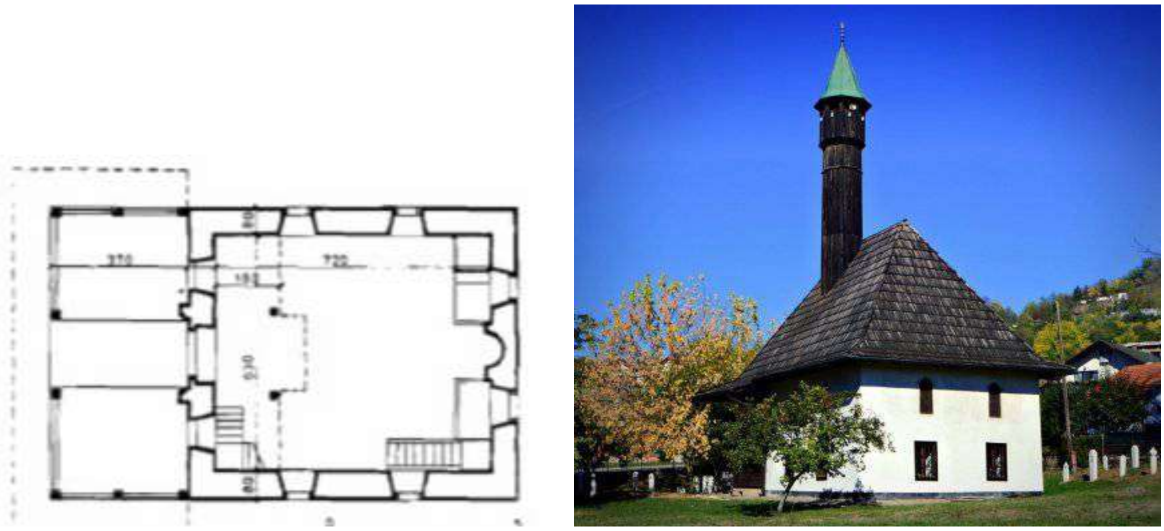
Figure 4. Golobrdica Mosque in Sarajevo and placement of the wooden minaret on the müezzinmahfili construction of the wooden minaret in 2009,–interior of the structure[7]

2.2.2 Tuzla: Džamija Husein Čauša

Jedna od najstarijih džamija u Tuzli je džamija Husein Čauša ili Džindijska džamija koja je prikazana na slici 5 [8]. Ona se prvi put spominje u 1701. godini. Iako je bila zatvorena u nekoliko navrata zbog slijeganja zemljišta i renoviranja (u periodu 1863–1864. godine, kao i 1961. godine), zadržala je svoj prvobitni oblik. Statistički podaci pokazuju da ova džamija predstavlja jedan od rijetkih sačuvanih primjera džamija sagrađenih od drveta u islamskom svijetu. Nalazi se u zoni s najintenzivnijim slijeganjem terena u Tuzli, koje je poznato po prirodnim izvorima soli – bunari soli. Ovo je tipična mahalska džamija, s jednom prostorijom za molitvu, dimenzija 6,3x7,2 m, s drvenom munarom i 16 prozora. Pravokutnog je oblika i s veoma dekorativnim drvenim vratima. Ona je građena od čerpića i pokrivena visokim krovom s pokrovom od šindre, dok je okvir oko vrata izgrađen od kamena krede (cretaceous). Zanimljivo je napomenuti da se gornji dio iznad sofe oslanja na osam drvenih stubova, čije rezbarenje predstavlja rad lokalnih majstora.

2.2.2. Tuzla region: Hussein Čauš's Mosque

One of the oldest mosques in Tuzla is Hussein Čauš mosques or Džindia mosque which is shown in Figure 5 [8]. It was mentioned for the first time in 1701. Even though it had been closed on several occasions because of soil settlement and renovations (in 1863-64 as well as in 1961), it has retained its original appearance. Statistics show that this mosque is one of the rare surviving examples of mosques built of wood in the Islamic world. It is located in the zone with the most intensive settlement of the terrain in Tuzla, which is well known by the natural salt wells. This is a typical mahala mosque, with one room for praying, dimensions 6,3x7,2m, with a wooden minaret and 16 windows. Rectangular in shape and with a highly decorative wooden door. It is built of adobe and covered with a steep wooden roof, while the frame around the door has been constructed by cretaceous stone. It is interesting to note that the upper part beyond the sofa rests on eight wooden posts, whose carving represents the work of the local craftsman.



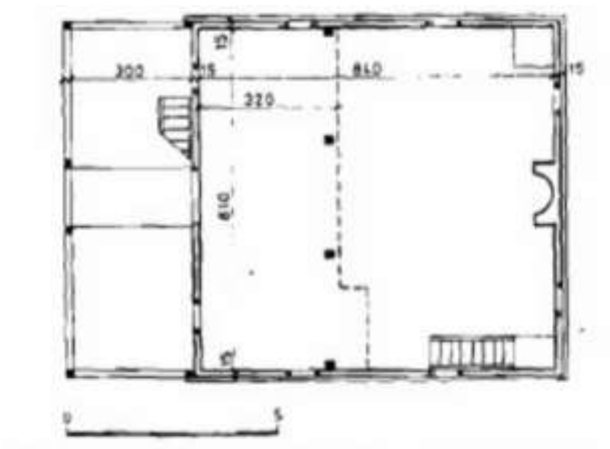
Slika 5. Tlocrt džamije i izgled džamije Husein Čauša u Tuzli [3], [9]
Figure 5. Layout and view of the Hussein Čauš's Mosque in Tuzla[3], [9]

2.2.3 Područje Tuzle: Atik džamija u Ćivi

Već sredinom druge polovine XX stoljeća lokalno stanovništvo Gornje Spreče u blizini Tuzle znalo je za postojanje ove džamije i da je već u to vrijeme bila stara 300 godina. Na turskom „atik" znači „stari" [3]. Prije četiri desetljeća stručnjaci sa Arhitektonskog fakulteta u Sarajevu sproveli su istraživanje o arhitektonskim karakteristikama Atik džamije. Pokazano je da je napravljena kao bondruk konstrukcija [10]. Tlocrtne dimenzije džamije su 8,1x8,4 m. Interesantno je da kao i džamija Husein Čauša ima sređenu kompoziciju sa 16 prozora. Karakteristika poligonalne munare (visine osam metara) ove džamije jeste da je opšivena drvenim daskama u „riblja kost" rasporedu, što doprinosi njenoj nosivosti i povećanju krutosti i stabilnosti (slika 6). Džamija je obnovljena u dva navrata: 1996. i 1997-1998. godine. Trenutno je u proceduri da bude proglašena nacionalnim spomenikom pod zaštitom na nivou Bosne i Hercegovine.

2.2.3. Tuzla region: Atik Mosque in Ćiva

It was already in the mid-second half of the 20th century that the local citizens of the Upper Spreča near Tuzla known about the existence of this mosque and at the time it was already 300 years old. In Turkish "Atik" means "old"[3].Four decades ago experts from the Faculty of Architecture in Sarajevo conducted research on the architectural characteristics of the Atik mosque. It was shown that it was made by bondruk construction [10]. Layout dimensions of the mosque are 8,1x8,4m. It is interesting that this mosque has a composition of 16 windows the same as Hussein Čaušmosque. The characteristic of the polygonal minaret is its coverage with "zig-zag" wooden planks, which contributes to its resistance and increase its stiffness and stability (Figure 6). The mosque was reconstructed in two occasions 1996 and in 1997-1998. Currently, it is in the process to become a national monument under the protection of Bosnia and Herzegovina.



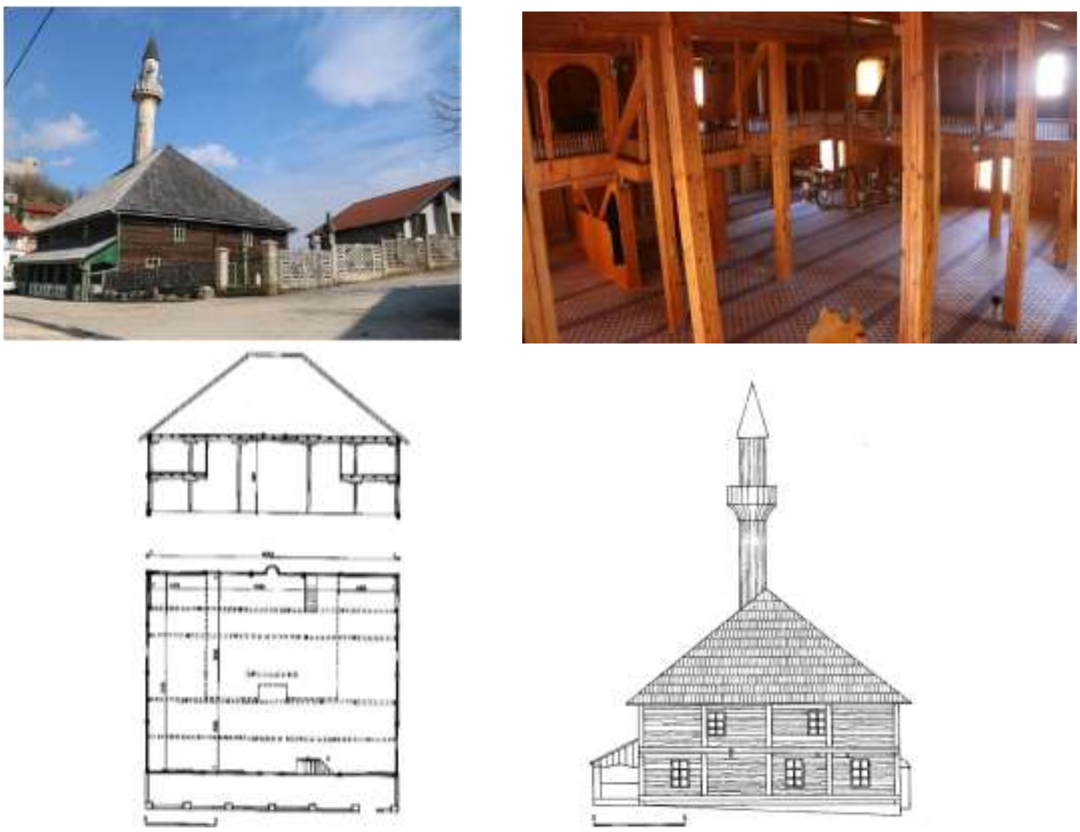
Slika 6. Atik džamija u Ćivi, detalj opšivanja munare [10] i tlocrt [3]
Figure 6. Atik mosque in Ćiva, detail of the diagonal wooden planks [10] and layout [3]

2.2.4 Područje Bihaća: Džamija u Bužimu

Džamija u Bužimu (slika 7) najveća je drvena džamija u Bosni i Hercegovini, a najvjerovatnije i u Evropi. Nalazi se u Bužimu, malom gradiću koji je nakon Bihaća bio najveća utvrda u Krajini. U stanju u kojem se nalazi danas, smatra se da je obnovljena 1838. godine [11]. Na osnovu ovoga može se zaključiti da je džamija još starija, međutim, iz natpisa se ne može zaključiti kakva vrsta restauratorskih radova je izvršena [12]. Na mezarju u džamiju najstariji grob datira iz 1856. godine [13].

2.2.4 Bihać region: Mosque in Bužim

The mosque in Bužim (Figure 7) is the biggest wooden mosque in Bosnia and Herzegovina, and most probably in Europe. It is located in Bužim, a small town which after Bihać was the largest fortification in Krajina. As it stands today, it is believed that it has been reconstructed in 1838[11]. On the basis of this, it can be concluded that the mosque is even older, however, from the inscriptions it cannot be concluded what kind of restoration works took place [12]. In the cemetery within the mosque, the oldest grave dates from 1856 [13].



Slika 7. Džamija u Bužimu [3]
Figure 7. Bužim mosque [3]

Ovo je najveća džamija ovog tipa, dimenzija 18x14 m i visine 5 m, a visina munare iznosi 17.2 metara. Ukupna površina iznosi 414 m^2. Zanimljivo je napomenuti da je džamija napravljena od vrlo rijetke vrste hrasta lužnjaka, koja raste u ovom području u blizini granice s Republikom Hrvatskom, a ono što je dodatno zanimljivo jeste da tokom izgradnje nisu korišteni ekseri niti bilo kakva druga vrsta metalnih spajala. Kruta veza je napravljena sa usjecima i drvenim klinovima (vidi sliku 7).

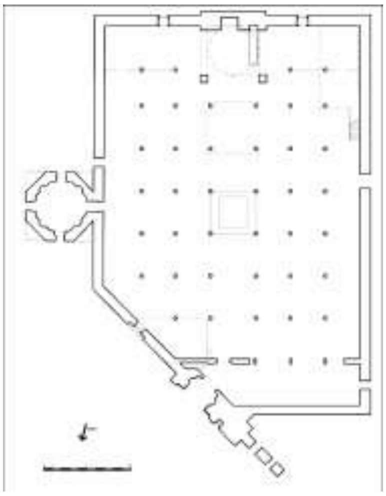
Dodatna karakteristika koja čini ovu džamiju jedinstvenom jesu redovi stubova (slika 7). Džamije s redovima drvenih stubova vrlo su rijetke i nastale su iz anadolskih seldžučkih velikih džamija (Ulu Cami), među kojima su najvažnije Eşrefoğlu džamija u Beyşehiru izgrađena 1296-99. godine sa sedam redova drvenih stubova i Ahi Elvan džamija u Ankari s tri reda drvenih stubaca [14], sagrađena 1382. godine na kosim stijenama s jednostavnom fasadom. Ovi stubovi pripadaju rimskom i vizantijskom periodu. Ahi Elvan džamija obnovljena je 1413. godine. Njena fino izrezbarena ograda od hrasta odražava karakteristike seldžučke tradicije gradnje drvetom.

Eşrefoğlu džamija je pravougaonog tlocrta sa oko 29.25 metara u pravcu istok-zapad i 43.9 metra u pravcu sjever-jug s bruto unutarnjim prostorom od 1187 m^2 u prizemlju. Ovo je čini najvećom drvenom džamijom. Krov džamije je podržan sa šest redova stubova koji su postavljeni okomito na zid kible, pri čemu se u svakom redu nalazi sedam redova drvenih stubova. Postoje ukupno četrdeset dva stuba [14], slika 8. Drveni stubovi su od kedrovine i prema usmenim predanjima bili su potopljeni u jezero Beyşehir šest mjeseci prije no što su upotrijebljeni za izgradnju. Dvadeset dva stuba su ortogonalnog oblika, dok je jedan stub dekagon a 19 je kružnog oblika. Visina stubova iznosi 7.5 metara dok poprečni presjek varira od 0.40 m do 0.45 m. Kaptoli stubova imaju tzv. kalemişi ručno izgravirane ornamente. Ovi ornamenti su crvene, plave i krem boje. Džamija stara preko 700 godina, kao drvena džamija, opstala je zahvaljujući „sniježnoj jami" koja se nalazi u sredini džamije. Prema predanjima snježna jama se popunjavala snijegom sa okolnih planina. Usljed postepenog topljenja snijega omogućavano je u toku toplih mjeseci

This is the biggest mosque of this type, having dimensions of 18 x 14 m and a height of 5 m, while the minaret's height is 17.2 meters. The total area is 414 m^2. It is interesting to note that the mosque is made of a very rare type of oak, "quercusrobur", which grows in this region near the border with the Republic of Croatia, and what is additionally interesting is that during the construction no nails or any type of metallic connections were used.A rigid connection is made with cuttings and wooden wedges (see Figure 7).

An additional feature which makes this mosque unique are rows of columns (Figure 7). Mosques with rows of wooden piles are very rare and are derived from the Anatolian Seljuk Great mosques (Ulu Cami), among which, the most important, theEşrefoğlu Cami in Beyşehir built1296-99 with seven rows of wooden poles and Ahi Elvan Cami (Ahi Elvan Mosque) in Ankara with three rows of wooden columns [14], built in 1382 on a sloping rock with a simple façade. These columns belong to the Roman and Byzantine periods. Ahi Elvan Mosque was restored in 1413. Its finely carved walnut-timber(pulpit) reflects the characteristics of the Seljuk wood tradition.

Eşrefoğlumosque has a rectangular plan of roughly 29.25 meters east to west and 43.9 meters north to south with a gross interior area of 1187 square meters on the ground floor. This makes it the largest of the remaining wooden hypostyle mosques. The mosque's roof is supported by six rows of columns perpendicular to the kibla wall, with seven ranks of columns in each row. There is a total of forty-two columns [14], picture 8. The wooden pillars are made of cedarwood and according to folk tales were immersed in the lake Beyşehir for six months before they were used for construction. Twenty-two pillars are orthogonal, while one pillar is decagonal, and 19 are of circular cross section. The height of the pillars is 7.5 meters, while the cross section varies from 0.40 m to 0.45 m. Pillar capitols have the so-called "kalemişi" hand-drawn ornaments. These ornaments are red, blue and cream colors. The mosque over 700 years old, as a wooden mosque, survived thanks to the "snow cave" which is located in the middle of the mosque. According to the



Slika 8. Eşrefoğlu džamija u Beyşehiru [14]
Figure 8. EşrefoğluMosque in Beyşehir [14]

da se unutrašnjost objekta hladi i da se obezbijedi potrebna vlažnost drvene konstrukcije. Nakon XIV stoljeća nema podataka o izgradnji ovakvih džamija. Džamije s redovima stubova pripadaju tzv. starijem tipu i prestale su da se grade kada su Turci usvojili tip džamije zasnovan na vizantijskim crkvama, naročito nešto kasnije kada je uzor postala Sveta Sofija.

Stubovi u džamiji u Bužimu protežu se od podruma kroz cijelu konstrukciju do krova [3], što se jasno može vidjeti na slici 7. Kada se uporede ove džamije, evidentno je da u džamiji u Bužimu stubovi nisu ravnomjerno raspoređeni kao u slučaju navedenih džamija u Turskoj. Džamija u Bužimu je proglašena nacionalnim spomenikom 2003. godine [12].

Konstrukcija džamije u Bužimu je sistem s nosivim zidovima, izrađen od drvenih talpi debljine 7 cm i visine 25 cm, vertikalno užljebljenih u drvene stubove dimenzija 21x21 cm i 24x24 cm. Zidovi u unutrašnjosti su fiksni sa spregovima dimenzija 12x12 cm. Temelj je izrađen od drvenih greda 21x21 cm, koji se nalaze na kamenoj podlozi [12]. Izgled džamije je prilično jedinstven i ne postoji sličan tip džamije u cijeloj Bosni i Hercegovini.

Džamija je nekoliko puta rekonstruisana [3,12]. Po prvi put 1937. godine, gotovo stotinu godina nakon njene izgradnje. Tada je krov uklonjen i promijenjen iz šindre u crijep. Osim toga, munara je skraćena, te je šerefet[6] iz zatvorene galerije pretvoren u otvoreni, u orijentalnom stilu, i na taj način podsjeća na stare kamene osmanske džamije. Sa stanovišta Venecijanske povelje [6], ovaj vid konstrukcijske realizacije je nešto što je uistinu promijenilo karakteristike džamije.

Najspecifičnija restauracija izvršena je 1960-ih godina, kada je čitava džamija podignuta iznad temelja. Ovo je bilo neophodno radi zamjene dotrajale grede – tzv. jastuci koji su bili u temeljima - i da se sazida zid ispod drvene konstrukcije. Kao što je izjavio lokalni imam, rekonstrukcija se dogodila tokom nekoliko mjeseci i za to vrijeme, što je prilično zanimljivo, molitve su obavljane u džamiji. Nakon što su novi temelji napravljeni, džamija je, bez izazivanja bilo kakvih oštećenja, ponovo vraćena na temelje.

Tokom 1980. godine, konstrukcija je ponovo obnovljena i kao unutrašnja obloga je postavljena lamperija unutar džamije. Kada se referira na Venecijansku povelju [6] i ICOMOS [15] ovo je prilično upitno, jer je to značajno promijenilo unutarnji izgled konstrukcije.

Generalno, nema pisanih podataka kada su izgrađene najstarije drvene džamije u Bosni i Hercegovini, već se znanje prenosi s generacije na generaciju i povezano je s početkom osmanske uprave. Općenito govoreći, džamije posjeduju sve arhitektonske karakteristike stambene kuće: od načina gradnje, građevinskih detalja do oblika i forme krova. Ova vrsta gradnje vidljiva je u nekim starim kućama u Krajini (sjevernoj Bosni), te na putu iz Sarajeva prema Vlasenici (istočna Bosna). U Karićima džamija je izgrađena kao brvnara, što se povezuje sa crkvama (poglavlje 3).

[6] Prošireni dio munare, vrsta galerije.

folk tales, the snow pit was filled with snow from the surrounding mountains. Due to gradual snow melting, it was possible during the warm months to cool the interior of the building and to provide the necessary humidity of the wooden structure. After the XIV century, there was no data on the construction of such mosques. The mosques with rows of columns belong to the so-called older type I, and they have ceased to be built when the Turks adopted a type of mosque based on Byzantine churches, especially a little later when Hagia Sophia became a role model.

Columns in Bužim mosque extend from the basement throughout the entire structure to the roof [3], which can be clearly seen in Figure 7.When comparing these mosques it is evident that in Bužim the pillars are not uniformly distributed as in the case of above-mentioned mosques in Turkey. The mosque in Bužim was declared a national monument in 2003 [12].

The structure of the Bužim mosques is a load bearing wall system, made of wood planks 7cm thick and 25 cm height, vertically grooved into wooden poles having dimensions 21x21 and 24x24 cm. The walls inside are fixed with braces of dimensions 12 x 12 cm. The foundation is made of wooden beams 21 x 21 cm, which are placed onto the stone backing[12]. The layout of the mosque is rather unique and there is no similar type of mosque in the entire Bosnia and Herzegovina.

The mosque was reconstructed several times [3,11]. For the first time in 1937, almost a hundred years after its construction. At this time the roof was removed and changed from shingle to tiles. In addition, the minaret was shortened, and Şeref[6] from the closed galleries transformed into an open, oriental style, recalling the old stone Ottoman mosque. Looking now at the Venice-Charter [6], this kind of structural changes is something that truly changed the features of the mosque.

The most specific restoration was done in the 1960s when the entire mosque was lifted from the ground up.This was needed because it was necessary to replace the worn-out beams, so-called pillows that were in the foundations, and to construct a wall under the wooden structure. As stated by the local imam, the reconstruction took place over several months and during that time, which is quite interesting, the prayers were performed in the mosque. After the new foundations were made, the mosque was restored to the foundations without causing any damage.

During 1980 the structure was again renovated and now wooden paneling was done inside the mosque. When referring to Venice-Charte [6] and ICOMOS [15] this is rather questionable, as this significantly changed the inner view of the structure.

Generally, there is no written data when were the oldest wooden mosques built in Bosnia and Herzegovina, but by the knowledge passed from generation to generation, it is connected to the beginning of the Ottoman administration. Generally speaking these mosques have all architectural characteristics of a residential house: from the construction method, construction details up to the shape and the form of the roof. This kind

[6] Enlarged part of the minaret, a kind of gallery.

of construction is seen in some old houses in Krajina (northern Bosnia), and along the road from Sarajevo to Vlasenica (eastern Bosnia). In Karići the mosque is built as a log cabin, which is a connection to the churches (Chapter 3).

3 CRKVE BRVNARE NA TLU BOSNE I HERCEGOVINE

Pored džamija, sakralne konstrukcije napravljene od drva specifične za ovu regiju su tzv. crkve brvnare (talpare) jer brvnare upućuju na oblu građu, a talpare na tesanu pravougaonu. Porijeklo brvnara u Bosni i Hercegovini, koje se koriste kao kuće, ili bogomolje, u vezi je s prvobitnom domovinom Slovena, karpatskom regijom poznatom po svojim gustim šumama. Sloveni su vjerovatno donijeli ideju o brvnari u zemlje koje su naseljavali i koje su također bogate šumama, brvnare se nalaze u Rusiji i Poljskoj, kao i na čitavom Balkanu. Naravno, pojava ovih brvnara je vrlo različita, zbog dominantnih društveno-historijskih i kulturnih odnosa u određenim područjima [16].

Prvi pisani dokazi o kapelama, drvenim crkvama i zvonicima datiraju iz hronike kneževskog perioda X stoljeća u Ukrajini. Više od 800 crkava je izgrađeno u Norveškoj, Skandinaviji, Francuskoj, Njemačkoj i nekoliko u Engleskoj. Većina je izgrađena u periodu širenja hrišćanstva u cijeloj Evropi. Period izgradnje drvenih crkava počeo je u tim zemljama.

Drvene crkve u Bosni i Hercegovini su različitog tipa hramova i dijela narodnog stvaralaštva. U Bosni i Hercegovini danas postoji oko 30 crkava pravoslavne provenijencije [17], dok se spominje samo jedna katolička crkva ovakvog tipa - Sveti Josip na Palama [12].

One su zadržale bosanski tradicionalni način gradnje s jedne strane i imaju dosta sličnosti sa džamijama. Očuvani ostaci današnjih drvenih crkava vjerojatno su rezultat nekih transformacija, neke vrste evolucije iz originalnog koncepta drvenih objekata za stanovanje, što je vidljivo iz rasporeda i organizacije prostora unutar crkava brvnara, koje su sačuvane do danas. Čuvene drvene crkve u Bosni datiraju iz XVIII i XIX stoljeća.

Arhitektura drvenih crkava bila je na dobrom putu da se razvije u opsežnu arhitekturu s vlastitim tipičnim ukrasom kada je njena evolucija naglo prekinuta. Izgradnja crkava brvnara je poseban oblik gradnje koji proizlazi iz narodnog-stambenog objekta bez povezivanja za bilo koji kulturu. Dakle, arhitektura ovih objekata predstavlja prepoznatljive i autentične oblike radova izvedenih iz istorijskih okvira i uslova.

S ciljem očuvanja njihovog nacionalnog identiteta i religije od osmanskog carstva, stanovnici pravoslavne vjeroispovjesti gradili su svoje crkve od drveta, kako bi ih lako preko noći mogli prebaciti na druge lokacije. Pismeni dokazi i znakovi na konstrukcijama pokazuju da je u stvari došlo do pomicanja tih objekata za nekoliko kilometara te njihovog ponovnog sklapanja [18]. U produžetku vremena, veliki broj crkava brvnara je uništen. Dva faktora koji igraju dominantnu ulogu u uništavanju objekata jesu prirodni i ljudski faktor (neodržavanje).

3 LOG CABIN CHURCHES IN BOSNIA AND HERZEGOVINA

Besides mosques, sacral structures made out of wood which is specific for this region are so called log cabin churches. The origin of log cabin buildings in Bosnia and Herzegovina, being used as homes, or places of worship, is associated with the primordial homeland of Slavs, the Carpathian regions famous for their dense forests. Slavs probably brought the idea of a log cabin with them to the lands that they inhabited and which was also rich in forests, log cabins are found in Russia and Poland, and entire Balkans. Of course, the appearance of these log cabins is very different, due to socio-historical and cultural relations predominant in certain areas [16].

The first written evidence of chapels, wooden churches, and bell towers date back to the chronicles of the princely period of the 10th century in Ukraine. More than 800 churches were built in Norway, Scandinavia, France, Germany and several in England. Most of them were built during the period of spreading Christianity throughout Europe. A period of building wooden churches started in those countries.

Wooden churches in Bosnia and Herzegovina are a distinct type of temples and part of folk creativity. In Bosnia and Herzegovina today, there are about 30 churches of Orthodox provenance [17], while only one Catholic Church of this type is mentioned, Saint Joseph in Pale [12].

They retained the Bosnian traditional way of construction on one hand and have a lot of similarities with the mosques. Today's preserved remains of wooden churches probably are the result of some transformation, some kind of evolution from the original concept of wooden buildings for housing, which is evident from the layout and organization of the space inside the log cabin churches, which have been preserved until today.The famous wooden churches in Bosnia date from XVIII and XIX centuries.

Timber church architecture was on its way to developing an extensive architecture with its own typical decoration when its evolution was abruptly cut short. The construction of the church log cabin is a particular form of construction resulting from the folk building without binding to any culture. Thus, the architecture of these buildings represents a distinctive and authentic form of works derived from the historical framework and conditions.

In order to preserve their national identity and religion from the Ottoman Empire, the Orthodox residents built their churches from wood, so that they could easily overnight transfer them to other locations. Written evidence and signs on the structures indicate that in fact there has been a movement of these structure for several kilometers and their re-assembly [18].Over time, a large number of log cabin churches was destroyed. Two factors which play the dominant role in the destruction of these objects is a natural or human factor (non-maintenance).

3.1 Crkve brvnare u BiH pravoslavne vjeroispovijesti: tipovi, osobenosti i primjeri

Postoje dvije vrste drvenih crkava koje se mogu razlikovati u zavisnosti od načina izgradnje i opštih arhitektonskih karakteristika [3]. Do današnjeg dana, ukupno je samo tridesetak objekata sačuvano, a oni se uglavnom nalaze u selima oko Banja Luke i Prijedora.

Prvi tip je povezan sa XVIII stoljećem i odnosi se na prilično male, jednostavne konstrukcije koje su i bez ukrasa i bez apside[7]. Crkve u ovoj grupi su jednostavne, skromnih dimenzija, tlocrtnih dimenzija 9x4 metara i površine od oko 30 m^2. Konstrukcija se nalazi direktno na zemlji bez bilo kakvih temelja ili kamene konstrukcije preko koje bi se prenosilo opterećenje na tlo. Većina su skeletnog sklopa, sa zidovima od užljebljenih dasaka, a pojedine od pravih brvana četvorougaonog presjeka. Brvna su na uglovima povezana na preklop. Zidovi se razlikuju i rijetko dostižu visinu čovjeka. Krovovi su vrlo strmi za brže odvođenje oborinskih voda i redovno zaobljeni na uglovima (grebenima). Na ovaj način, izbjegava se ugao na krovu koji je neprikladan, a istovremeno se formira polueliptični krov nad frontalnim zidovima koji pruža bolju zaštitu od atmosferskih uticaja i vizuelno dijelimično oponaša apsidu koja se posebno ne izvodi, a što dodatno daje specifičan estetski izgled konstrukcije. Ove zgrade nemaju plafonsku/među-spratnu konstrukciju tako da je krovna konstrukcija u unutrašnjosti sasvim vidljiva. Otvori su jako mali, te je unutrašnjost crkve prilično mračna.

Ove crkve nalaze se u: Malom Blaškom (slika 9), Kolima (slika 10), Romanovcima (slika 11) i Javorinama.

3.1 Log cabins in Bosnia and Herzegovina of orthodox religion: types, characteristics, and examples

There are two different types of wooden churches that can be distinguished depending on the way they are constructed and general architectural characteristics [3]. To this day, only thirty of these facilities have been preserved, which are mostly located in villages around Banja Luka and Prijedor.

The first type is connected to the XVIII century and it refers to rather small structures of simple construction, without decoration and apses[7].The churches in this group are simple, modest in size, with layout dimensions of 9x4 meters and an area of about 30 m^2. The structure is placed on the bare ground without any foundations or stone construction that would transfer the load to ground. Most of them are of skeleton type, with walls made of grooved boards and some made of logs with rectangular cross section. On the corner, the logs are connected with a flattened flap. The walls vary and rarely reached the height of a man. The roofs are very steep for faster drainage of rain water. At the corner, the faces of the roof are regularly rounded. In this way, a corner on the roof is avoided which is unsuitable for the roof, and at the same time a semi-elliptical roof of the frontal walls is obtained which provides a better protection from atmospheric effects. This additionally gives a specific esthetical vision of the structure. These buildings do not have a ceiling so the roof structure in the interior is fully visible. Openings are very small so the inside of the church is rather dark.

These churches are located in Malo Blaško (Figure 9), Kola (Figure 10), Romanovci (Figure 11) and Javorine.

Slika 9. Crkva brvnara u Malom Blaškom [19]
Figure 9. Log cabin church in Malo Blaško [19]

[7] Zadnji, polukružni dijelovi crkava, naročito onih koje su građene u romanskom stilu.

[7] Semicircular part of the altar, especially the ones that are made in Roman style.

3.1.1 Crkva brvnara u Kolima

Drvena crkva u Kolima potiče s kraja XVIII vijeka. Osnova i konstrukcija crkve su najjednostavnijeg tipa, s drvenim okvirom koji je popunjen daskama na pero i utor, vidljivim rogovima u enterijeru i krovom koji je pokriven šindrom. Crkva je pravougaonog oblika cca 7,18x4,21 m. Kao konstruktivni sistem upotrijebljen je drveni skeletni sistem. Na crkvi postoji samo jedan ulazni otvor koji se nalazi na sjeverozapadnoj strani. Ulaz je pravougaonog oblika (svijetle dimenzije 69x126 cm), sa okvirom od drvenih greda. Crkva Brvnara posvećena Vaznesenju Hristovom u Kolima proglašena je nacionalnim spomenikom u januaru 2009. godine [20].

3.1.1 Log cabins in Kola

The wooden church in Kola dates from the end of the 18th century. The layout and construction of the church are of the simplest type, with a wooden frame filled with planks on a feather and groove system, visible rafters in the interior and a roof covered with a shingle. The church has a rectangular shape of approximately 7,18 x 4,21 m. A wooden skeleton system was used as a constructive system. There is only one entrance to the church and it is located on the northwest side. The entrance is of rectangular shape (bright dimensions 69 x 126 cm), with a frame made of wooden beams. The Log cabin is dedicated to the Worshiping of Christ in Kola and it was declared a national monument in January 2009 [20].

Slika 10. Crkva brvnara u Kolima [20]
Figure 10. Log cabin church in Kola [20]

3.1.2 Crkva Brvnara u Romanovcima

Drvena crkva posvećena Svetom Nikoli u Romanovcima (slika 11) proglašena je nacionalnim spomenikom Bosne i Hercegovine, odlukom Komisije za očuvanje nacionalnih spomenika 2004. godine. Podaci o vremenu izgradnje crkve u Romanovcima ne postoje i uglavnom se odnose na narodna predanja. Osnovu crkve predstavlja pravougaonik dimenzija 7,42 m dužine i 4,16 m širine, što je jako slično dimenzijama crkava u Kolima.

3.1.2 Log cabins in Romanovci

The wooden church dedicated to St. Nicola in Romanovci (Figure 11) was declared a national monument of Bosnia and Herzegovina by the Decision of the Commission to Preserve National Monuments on in the year of 2004. Data regarding the time of construction of the Church in Romanovci is not available and it is mainly related to the folk tales and traditions.The church's layout is a rectangle measuring 7.42 meters in length and 4.16 meters wide, which is very similar to the dimensions of the Log cabin in Kola.

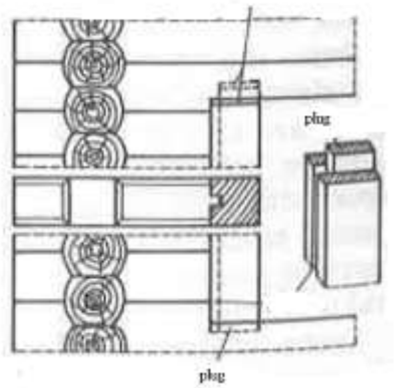
Slika 11. Crkve brvnare u Romanovcima [21]
Figure 11. Log cabin church in Romanovci [21]

Zidovi objekta su napravljeni od hrastovih talpi. Zidovi su ojačani podupiračima koji su raspoređeni samo iznutra. Drvena krovna konstrukcija vidljiva je iznutra, kao i kod crkve brvnare u Kolima. Crkva je pokrivena klisom[8] od drveta jele malih dimenzija i sa užljebljenjima. Nosiva konstrukcija je izrađena od horizontalno postavljenih greda poprečnih presjeka 15/15, 17/16 i 13/12 cm. Ova crkva ima dva otvora, glavni ulaz na zapadnoj strani, a drugi otvor na južnoj strani. U 2000. godini izvršeni su konzervatorski radovi i oni su se sastojali od sanacije krovne konstrukcije, oblaganja krovnog materijala novim – klisom (slika 11), te od pripremnih radova koji su napravljeni za postavljanje kamenih ploča na pripremljenu podlogu od pijeska, šljunka i nabijene zemlje, kao i pripreme za izradu kanala koji će se koristiti za odvod kiše i vode koja pada s krova crkve [21].

Drugi tip se odnosi na crkve iz XIX stoljeća koje su izgradili majstori iz Osata i oni su uključili neke specifične islamske uticaje [3]. Zidovi su napravljeni od debala i u slučaju kada je crkva bila veća, pri čemu je to premašilo dužinu debla, to bi bilo riješeno tako što bi se u sredini perifernog zida pravio jedan stub u koji se užljebljuju „na čep" debla (slika 12).

The walls of the building are made of oak planks. The reinforcement of the walls is done by struts arranged only on the inside. The wooden roof structure is visible from the inside, as is the case with the church in Kola. The covering of the church is made of "klis"[8] firwood having small dimensions and chamfered. The bearing structure is made of horizontally placed beams having sections 15/15, 17/16 and 13/12 cm. This church has two openings, the main entrance on the west side and the second opening on the south side. In 2000 conservation works took place and they consisted of: repairs of the roof structure, cladding of the church was done with new roofing material – "klis" (Figure 11), and preparatory works were done for the placement of stone slabs on the prepared surface of sand, gravel and packed earth as well as preparations for making channels that will be used for rain drainage and water that falls from the church's roof [21].

The second type refers to the churches from XIX century which were constructed by well-known, a craftsman from Osat and they incorporated some specific Islamic features [3]. The walls are made of chumps and in the case where the church was bigger and this exceeded the length of chumps, it was solved in a way that in the middle of the peripheral wall one pier was constructed in which the clamps were slammed "on the plug" (Figure 12).



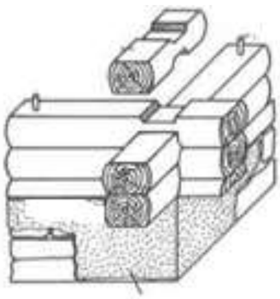
Slika 12. detalji ukrućenja zidova [22]
Figure 12. Connections of the chumps [22]

Na ovaj način postignuta je željena dužina i ujednačenost materijala kao i debljina zida. Crkve ove grupe uvijek su imale poligonalne apside, gdje je izvršeno pričvršćivanje debla na uglovima na kompleksan način s dvostruko kosim preklopom (slika 13) i blago savijenim krajevima. Otvori prozora su većih dimenzija, dok su lukovi, vrata i okovratnici dekorisani. Svod je zakrivljen i širi. Svod iznad ulaza i na vratima vrlo često ima islamska svojstva [3]. Uglavnom, ove konstrukcije su većih dimenzija i s više dekorativnih elemenata.

In this way, the desired length and the uniformity of the material and thickness of the wall were achieved. The churches of this group always had polygonal apses, where the fastening of the beavers to the corners was performed in a complex manner (Figure 13) with a double flattened flap and a slightly curved corner. Window openings are of larger dimensions, while arches, doors and door mirrors are decorated. The vault is curled and wider. The arch above the entrance and on the doors very often has Islamic features [3]. Generally, these structures are of larger dimension and with more decorative elements.

[8] Drveni crijep ili daščica u obliku crijepa.

[8] Wooden tiles or planks in the form of tiles.



Slika 13. Detalj veze na uglu [22]
Figure 13. Corner connection [22]

Zanimljivo je napomenuti da je unutrašnjost crkava dosta slična. Primjeri se mogu naći u nekoliko sela kao što su Palačkovac (slika 14), Dragivići, Rakelića i tako dalje [3, 19]. Smatra se da postojeća crkva u Palačkovcima predstavlja treću crkvu, i da 1843. godina jeste godina njene obnove. Izrađena je od vrlo kvalitetne hrastovine s vrlo preciznim vezama i spojevima. Proglašena je nacionalnim spomenikom 2005. godine [23].

3.1.3 Crkva brvnara u Palačkovcima

Urezana 1843. godina iznad ulaznih vrata crkve brvnare u Palačkovcima može navesti na možda pogrešan zaključak da je to godina njene izgradnje. Jedna od njenih karakteristika jeste da nema velike dimenzije (osnova dimenzija 9,70x4,80 m) kakve imaju druge crkve sagrađene u ovo doba. To navodi na zaključak da je morala nastati znatno ranije i da 1843. godina predstavlja godinu njene obnove. Prilikom restauracije carskih dveri, ispod postojećeg, otkriveni su stariji slikani slojevi daleko većih likovnih vrijednosti. Ovo slikarstvo, po svemu sudeći, pripada srpskom ikonopisu XVIII vijeka i to njegovim najboljim ostvarenjima. Otkriće ovog sloja, uz pronalazak starog poda od kamenih ploča ispod poda od opeke, kao i spomenute građevinske karakteristike objekta, bili bi pouzdan argument za tvrdnju da je objekat 1843. godine samo obnovljen [23]. Bez ikakvog pretjerivanja može se reći da je crkva brvnara u Palačkovcima – Prnjavorski srez – remek-delo ove vrste. Građevina spada u razvijeni, gotovo savršeni oblik crkava brvnara XIX veka.

It is interesting to note that the interior of churches is rather similar. Examples are found in several villages like Palačkovac (Figure 14), Dragivići, Rakelići etc. [3, 19]. It is believed that the existing church in Palačkovac is the third church and the year 1843, is the year of its last reconstruction. It is constructed of a very high-quality oak wood with very precise connections and junctions. It was declared a national monument in 2005 [23].

3.1.3. Log cabins in Palačkovac

The year 1843 engraved above the entrance of the church in Palačkovac may lead to perhaps an incorrect conclusion that it was the year of its construction. One of its characteristics is that it not of large dimensions (the base is 9.70x4.80 m), opposed to the other churches built at this time. This leads to the conclusion that it had to be constructed much earlier and 1843 it the reconstruction year. During the restoration of the imperial doors, under the existing one, older paintings were retrieved of far greater artistic values. This painting, most probably, belongs to the Serbian iconography of the 18th century, and to its best achievements. The discovery of this layer, with the discovery of an old stone floor beneath the brick one, as well as the aforementioned construction characteristics of the building, would be a reliable argument for the claim that the church was only renovated in 1843 [23]. Without any exaggeration, it can be that the Log cabin in Palačkovac - the Prnjavor region - is a masterpiece of this kind. The building belongs to the developed, almost perfect shape of the log cabin churches of the 19th century.

Slika 14. Crkva u Palačkovcima [12]
Figure 14. Church in Palačkovac [12]

3.2 Crkve brvnare u BiH katoličke vjeroispovijesti: osobenosti i primjer

Katoličke crkve brvnare u Bosni i Hercegovini potpuno su neistražene. Nema pouzdanih informacija o njihovoj izgradnji, obliku, niti se zna da li potiču iz autohtone arhitekture. Crkva Svetog Josipa se razlikuje svojim arhitektonskim karakteristikama od pravoslavnih crkava, kao i od katoličkih crkava u Republici Hrvatskoj.

Jedina katolička crkva u vidu brvnare je Rimokatolička crkva Svetog Josipa na Palama (slika 15). Crkva je izgrađena 1911. godine od strane austrijskih inženjera za njihove vjerske potrebe. Napravljena je isključivo od drveta, tako da je to jedina drvena crkva u Vrhbosanskoj nadbiskupiji [24], te je odlukom Komisije za očuvanje nacionalnih spomenika proglašena nacionalnim spomenikom Bosne i Hercegovine 2004. godine.

3.2 Log cabin churches in B&H of Catholic religion: features and example

Catholic churches of the log cabin type in Bosnia and Herzegovina are totally unexplored. There is no reliable information on their construction, form, and whether they originate from autochthonous architecture. The Church of St. Josip is distinguished by its architectural characteristics from Orthodox churches, as well as from Catholic churches in the Republic of Croatia.

The only Catholic church of the cabin log type is the Roman Catholic Church of St. Joseph in Pale (Figure 15). The church was built in 1911 by Austrian engineers for their religious needs. It is made exclusively from wood, so it is the only wooden church in the Vrhbosanska Archdiocese [24], and by the decision of the Commission to Preserve National Monuments declared a national monument of Bosnia and Herzegovina in 2004.

Slika 15. Rimokatolička crkva na Palama [24]
Figure 15. Roman Catholic church of Pale [24]

Spoljašnje dimenzije konstrukcije su 17x8 m, sa apsidom, ili 12x8 m, bez apside. Visina zidova crkve, mjereno od poda do početka svoda, iznosi oko 4 m, a visina do dna svoda je oko 7 m. Zidovi crkve su napravljeni od masivnih greda pri čemu je povezivanje izvršeno drvenim gredicama.

Brvna su postavljena na kamenoj platformi visine 40 cm. Kamena platforma je izgrađena od nepravilno klesanih i horizontalno postavljenih kamenih blokova. Zidovi crkve su malterisani sa unutarnje strane. Unutrašnji prostor je natkriven svodom, oslonjenim na podužne zidove i s natkrivenim dvovodnim krovom. Krov crkve je prekriven crijepom, dok je višedijelni krov apside i zvonik obložen limom [24].

Iznad ulaza u crkvu je postavljen zvonik s baroknim elementima. Zvonik u dvije razine, s kvadratnom osnovom prve i osmokutnom osnovom druge razine, završen je lukovičastom kupolom. Svojom obradom najistaknutije je ulazno pročelje (slika 16). Ulazni zid je obradom i upotrijebljenim materijalom podijeljen u horizontalnom smislu na tri polja. Prvo polje po svojoj visini odgovara visini posrednih stubaca, odnosno visine je oko 3 m. Građeno je kao i ostali zidovi crkve, od brvana s tankim drvenim gredicama između. Naredna dva polja, međusobno odijeljena horizontalnim, nazubljenim vijencem,

The outer dimensions of the structure are 17 x 8 m, with an apse, or 12 x 8 m, without an apse. The height of the walls of the church, measured from the floor to the beginning of the vault, is about 4 m and the height to the bottom of the vault is about 7 m. The walls of the church were made of massive beams (birch) with thin wooden planks between them.

The logs were placed on a stone platform 40 cm high. The stone platform is made of irregularly carved stone blocks placed horizontally. The walls of the church are internally lynched. The interior space is covered by a vault, lying on the longitudinal walls and with a covered gutter roof. The roof of the church is covered with tiles, while the multi-faceted roofs of the apse and bell-tower are covered with metal sheets [24].

Above the entrance to the church is a belfry with baroque elements. The two-level bell, with a square base of the first and octagonal bases of the second level, is completed with a bulbous dome. With its processing, the most important is the frontal facade (Figure 16). The entrance wall is divided into three fields in respect to the treatment and the used material in the horizontal direction. The first field corresponds to the height of the indirect columns, that is, the height is about 3 m. It was built like all other walls of the church, from urađena su od dasaka postavljenih okomito na brvna. Daske su ukrašene reljefno izvedenom arkadom s polukružnim lukovima (slika 16) [24].

the logs with thin wood plagues in between. The next two fields, separated by a horizontal beam, are made of planks mounted perpendicular to the logs. The boards are decorated with a relief arcade with semicircular arches (Figure 16) [24].

Slika 16. Zvonik i ulazno pročelje [24]
Figure 16. Bell and entrance [24]

Kako su dimenzije crkve veće, korišten je isti sistem povezivanja brvna kao i u pravoslavnim crkvama drugog tipa (slika 12). Izgradnja crkve se razlikuje od ostalih, budući da su ovdje korištene i talpe za gradnju. Enterijer crkve je prostran i pun svjetla, što je u suprotnosti s pravoslavnim crkvama, a slično s džamijama. Svi konstruktivni i dekorativni drveni elementi unutar crkve – stubovi, kosnici, ograde, vijenci i prozorski ramovi bogato su profilisani.

Stručnjaci Kantonalnog instituta za zaštitu kulturne, povijesne i prirodne baštine u Sarajevu tokom primarnog pregleda crkve 1985. godine utvrdili su prisustvo vlage u crkvi zbog blizine potoka Repašnice. Usljed neodržavanja tokom dugog razdoblja od tri godine (1992–1995) pogoršanje stanja konstrukcije bilo je više nego očigledno. Tokom 2004. godine u crkvi su izvršeni radovi obnove. Radovi nisu izvedeni pod nadzorom stručne službe za zaštitu spomenika [24].

As dimensions of the church are larger, the same connection system was used as in the Orthodox churches of the second type (figure 12). The construction of the church differs in respect to others as here boards were used for construction as well. The interior of the church is spacious and lightly which is in contacts with the Orthodox Churches and similar to the mosques. All constructive and decorative wooden elements within the church - columns, trusses, fences, cornice and window frames are richly profiled.

Experts of the Cantonal Institute for the Protection of the Cultural and Historical and Natural Heritage of Sarajevo during the primary inspection of the church in 1985 found the presence of moisture in the church caused by the vicinity of the Repašnica stream. Due to no maintenance during a long period of three years (1992-1995) deterioration of the structure was more than evident. During the year 2004, renovation works were carried out in the church. The works were not carried out under the supervision of the experts for the protection of the monument [24].

4 ZAKLJUČAK

Džamije s drvenim munarama u Bosni i Hercegovini variraju u tlocrtu, arhitektonskom obliku i načinu gradnje, primjeni raznih materijala za zidove i oblaganje krovova, kao i u fasadnim oblicima, te u načinu konstruisanja i izvođenja različitih krovnih sistema i sistema munara.

Objekti brvnara u BiH koji su korišćeni za stanovanje bili su direktna inspiracija za objekte za bogosluženje,

4 CONCLUSION

Mosques with wooden minarets in Bosnia and Herzegovina vary in ground plan, architectural form, and manner of construction, application of various materials for walls and roofs, as well as façade forms, roofs, and minarets.

The origin of log cabin buildings in Bosnia and Herzegovina, being used as homes, have been the bilo pravoslavne ili katoličke vjeroispovjesti. Sloveni su vjerojatno donijeli ideju brvnare na područja BiH iz područja koja su takođe bila bogata šumama, čemu su dokaz brvnare pronađene u Rusiji i Poljskoj, te na cijelom Balkanu. Naravno, izgled ovih brvnara veoma je različit, zbog društveno-istorijskih i kulturnih odnosa koji prevladavaju u nekim područjima. U Bosni i Hercegovini postoji samo jedna katolička crkva ovog tipa što je čini još vrijednijom.

Za sve vrste vjerskih građevina, građevinski materijal se koristi na isti način kao i za gradnju stambenih kuća. Vidljivo je da se sakralni objekti razlikuju regionalno na isti način kao i stambena arhitektura. Potrebno je stalno održavanje i popravak ovih vrsta konstrukcija. Nažalost, svijest o vrijednosti ovakvih objekata nije zadovoljavajuća. To je očigledno samom činjenicom da je sačuvan mali broj drvenih sakralnih objekata. Osim toga, intervencije koje su učinjene i koje se u nekoj mjeri vrše i danas, ne slijede pravila Venecijanske konvencije i pravila ICOMOS-a. Nedostatak finansija i osoblja predstavlja najveći problem za potpunu obnovu i konzervatorske radove na tradicionalnim sakralnim objektima od drveta. Sve ove konstrukcije moraju biti zaštićene, uključujući sve nematerijalne i materijalne elemente s ciljem očuvanja kulturno-istorijske baštine.

inspiration for places of worship, either Orthodox or Catholic. Slavs probably brought the idea of a log cabin with them to B&H from areas that were also rich in forests, the proof is log cabins in Russia and Poland, and entire Balkans. Of course, the appearance of these log cabins is very different, due to socio-historical and cultural relations predominant in certain areas. In Bosnia and Herzegovina, there is only one Catholic Church of this type which makes it even more valuable.

For all types of religious structures, the building material is used in the same manner as for local dwellings. It is seen that the sacral buildings differ regionally in the same way as the residential architecture. Permanent maintenance and repair of these types of structures are required. Unfortunately, awareness regarding the value of these types of structures is not satisfactory. This is evident by the fact that a small number of wooden sacral objects has been preserved. Additionally, interventions that have been done and that are even being conducted today do not follow the Venetian Charter and rules of ICOMOS and the maintenance of such structures is not satisfactory at all. Lack of finance and personnel are cumbersome for full restoration and conservation works on these types of structures. All of these structures need to be protected including all the intangible and tangible assets with the goal to preserve the cultural-historic heritage.

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REZIME

SAKRALNI OBJEKTI OD DRVETA U BOSNI I HERCEGOVINI

Naida ADEMOVIĆ
Azra KURTOVIĆ

Sakralni objekti, džamije s drvenim munarama i crkve brvnare, pravoslavne i katoličke, predstavljaju kulturno istorijsko naslijeđe, reprezente drvenih konstrukcija u Bosni i Hercegovini, koje nisu samo međusobno preplетene nego su povezane i sa stambenom arhitekturom što ih čini još interesantnijim. Ove konstrukcije su dugo vremena bile zapostavljene i stavljene su na margine zaborava. Sakralni objekti s drvenim obilježjima zadržali su zajedničke stambene osnove s jedne strane, dok se različitost ogledala u njihovim specifičnim karakteristikama. Neupitno je da ljudski faktor i prirodni uticaji na drvene konstrukcije zahtjevaju stalni monitoring i održavanje, što u nedostatku finansija i osoblja otežava konzervaciju i restauraciju tradicionalnih sakralnih drvenih građevina. U radu su prikazani osnovni podaci o ovoj vrsti građevina s najznačajnijim pojedinostima karakterističnih primjera.

SUMMARY

WOODEN SACRAL OBJECTS IN BOSNIA AND HERZEGOVINA

Naida ADEMOVIC
Azra KURTOVIC

Sacral buildings, mosques with wooden minarets and chapel churches, Orthodox and Catholic churches, represent cultural heritage, and are representatives of wood structures in Bosnia and Herzegovina, which are not only interconnected but also connected with residential architecture, making them even more interesting. These structures were for a long time neglected and placed on the margins to be forgotten. Sacral objects with wooden features retained common housing elements on one hand and the diversity is reflected in their specific characteristics. It is undoubtedly that natural influences and the human factor on wooden structures require constant monitoring and maintenance, which in the absence of finance and personnel makes it difficult to conserve and restore the traditional sacral wooden structures. This paper presents the basic data on this type of construction with the most significant details of characteristic examples.

IN MEMORIAM

Prof D-r eng **YORDAN MILEV**
(1960-2017)

On January 9, 2017, in Sofia Prof. Dr. Yordan Ivanov Milev suddenly left us at the age of 57. Born on February 15, 1960, in Sofia, he completed his secondary education in 1978 at the Sofia Mathematical High School. From 1980 to 1985 he studied "Industrial and Civil Engineering" at the Higher Institute of Architecture and Construction (HIAC). Over the period from 1985-86, he studied a post-graduate specialization at the Center for Applied Mathematics of the Higher Institute of Mechanical and Electrical Engineering. In 1986 he was promoted to Assistant Professor at the Department of Reinforced Concrete Structures and until 2004 he passed through Senior Assistant Professor and Chief Assistant Professor. In the years 1994, 1995 and 1996 he was consistently a postgraduate and a visiting researcher at the Tsukuba Science Research Institute, Yokohama University - Japan and the Institute of Seismic Engineering at the Tokyo University. In 1997 he was visiting researcher at the CITY University, London and the Technical University of Darmstadt, Germany. In 1998 he was a visiting researcher at the University of Naples, Italy.

In 1999 he defended his dissertation thesis for acquiring the educational and scientific degree "Doctor of Science" on the topic: "Modelling of reinforced concrete walls for seismic research of buildings with reinforced concrete structure" - under the guidance of Japanese professor Toshimi Kabeyasava. Over the period of time from 2000-2001, he was a visiting researcher and lecturer at Chalmers-Göteborg Technical University, Sweden. In 2001-2003 he conducted a postgraduate research at the Institute of Seismic Engineering at Tokyo University.

In 2004 Dr. Eng. Milev became Associate Professor and in 2013 he was promoted to Full Professor at the Department of Reinforced Concrete Structures at the University of Architecture, Civil Engineering and Geodesy (UACEG) - Sofia. He received three workshop certificates - fire safety in 2010, Eurocode 2 in Brussels, 2011 and Eurocode 8 in Lisbon, 2011.

When regarding his educational and pedagogical activities, Professor J. Milev delivered lectures, exercises, and course design in the Bulgarian and English language in a wide range of disciplines such as Reinforced Concrete, Design of structures for earthquake resistance, Seismic assessment, and restoration of existing buildings. He was a tutor of one Ph.D. student who successfully defended his doctoral dissertation. He was the author or co-author of 7 manuals and 54 scientific papers. Two of the manuals are the first manuals for the design of reinforced concrete multi-storey buildings according to EC2 and EC8, which is extremely useful in the application of Eurocodes for students and designers not only in Bulgaria but also in the Balkan region.

Professor Yordan Milev was acquainted in detail with the methods of modeling different structural objects as well as the capabilities of powerful computer programs for mechanical and mathematical modeling in structural design which helped him both in design practice and in student education.

In his science and science - applied activity, he wrote a monograph titled: Eurocode 8, Seismic Design of Reinforced Concrete Structures - Part One - Practical Guide, CEID, Sofia, 2012. The modeling of structures for seismic impacts, methods of analysis, application of modern software products were considered. He was convinced that according to EC8, large lengths of hidden columns were obtained compared to Japanese and US building codes, which resulted in non-economic solutions. The author proposed a methodology for adapting the three-stage Japanese assessment procedure for Bulgarian conditions, explaining all the elements of the procedure and their possible values. He was a co-author of several edited manuals and Guidance (7) related on Reinforced Concrete Structures with very wide application in education and design practice.

Prof. Y. Milev was engaged in active design activity. More than 50 industrial, residential, administrative, office buildings and facilities were constructed by his projects. A number of innovations were implemented. He was honored with 4 awards Designer of the Year in 2008 and 2009 in National Competition "Building of the Year" held under the patronage of Ministry of Regional Development and Public Works (MRDPW). He carried out research and developed and realized projects for strengthening more than 70 buildings. He applied modern methods, materials, and technologies with externally bonded FRP and embedded steel connections.

In the eyes of his teachers, as a student and a graduate, Prof. Milev will remain forever excellent as he was during the course of studies and when he developed and defended his diploma thesis with the excellent mark. In the minds of his colleagues, he will remain forever a very modest and hardworking man, talented researcher, erudite lecturer, and outstanding designer of modern structures. He conducted numerous specializations at world-renowned universities and research institutes, with great ambition and desire to share and deliver the lectures to specialists at the engineering college in Bulgaria and abroad, especially in our neighborhood, Serbia and Macedonia. At the invitation of colleagues from Serbia, he delivered two introductory lectures at scientific meetings. In this Journal, No. 3/2016, he published a valuable manuscript: *Problems and their solution in the practical application of Eurocodes in seismic design of RC structures*.

Doncho Partov
Ivan Ivanchev

UPUTSTVO AUTORIMA*

Prihvatanje radova i vrste priloga

U časopisu Materijli i konstrukcije štampaće se neobjavljeni radovi ili članci i konferencijska saopštenja sa određenim dopunama, iz oblasti građevinarstva I srodnih disciolina (geodezija I arhitektura). Vrste priloga autora i saradnika koji će se štampati su: originalni naučni radovi, prethodna saopštenja, pregledni radovi, stručni radovi, prikazi objekata i iskustava (studija slučaja), kao i diskusije povodom objavljenih radova.

Originalni naučni rad je primarni izvor naučnih informacija i novih ideja i saznanja kao rezultat izvornih istraživanja uz primenu adekvatnih naučnih metoda. Dobijeni rezultati se izlažu sažeto, ali tako da poznavalac problema može proceniti rezultate eksperimentalnih ili teorijsko numeričkih analiza, tako da se istraživanje može ponoviti i pri tome dobiti iste ili rezultate u okvirima dopuštenih odstupanja, kako se to u radu navodi.

Prethodno saopštenje sadrži prva kratka obaveštenja o rezultatima istraživanja ali bez podrobnih objašnjenja, tj. kraće je od originalnog naučnog rada.

Pregledni rad je naučni rad koji prikazuje stanje nauke u određenoj oblasti kao plod analize, kritike i komentara i zaključaka publikovanih radova o kojima se daju svi neophodni podaci pregledno i kritički uključujući i sopstvene radove. Navode se sve bibliografske jedinice korišćene u obradi tematike, kao i radovi koji mogu doprineti rezultatima daljih istraživanja. Ukoliko su bibliografski podaci metodski sistematizovani, ali ne i analizirani i raspravljeni, takvi pregledni radovi se klasifikuju kao stručni radovi.

Stručni rad predstavlja koristan prilog u kome se iznose poznate spoznaje koje doprinose širenju znanja i prilagođavanja rezultata izvornih istraživanja potrebama teorije i prakse.

Ostali prilozi su prikazi objekata, tj. njihove konstrukcije i iskustava-primeri u građenju i primeni različitih materijala (studije slučaja).

Da bi se ubrzao postupak prihvatanja radova za publikovanje, potrebno je da autori uvažavaju Uputstva za pripremu radova koja su navedena u daljem tekstu.

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Od broja 1/2010, prema odluci Upravnog odbora Društva i Redakcionog odbora, radovi sa pozitivnim recenzijama i prihvaćeni za štampu, publikovaće se na srpskom i engleskom jeziku, a za inostrane autore na engleskom (izuzev autora sa govornog područja srpskog I hrvatskog jezika).

Svaka stranica treba da bude numerisana, a optimalni obim članka na jednom jeziku, je oko 16 stranica (30000 slovnih mesta) uključujući slike, fotografije, tabele i popis literature. Za radove većeg obima potrebna je saglasnost Redakcionog odbora.

* **Uputstvo autorima je modifikovano i treba ga, u pripremi radova, slediti.**

GUIDELINES TO AUTHORS

Acceptance and types of contributions

The Building Materials and Structures journal will publish unpublished papers, articles and conference reports with modifications in the field of Civil Engineering and similar areas (Geodesy and Architecture).The following types of contributions will be published: original scientific papers, preliminary reports, review papers, professional papers, objects describe / presentations and experiences (case studies), as well as discussions on published papers.

Original scientific paper is the primary source of scientific information and new ideas and insights as a result of original research using appropriate scientific methods. The achieved results are presented briefly, but in a way to enable proficient readers to assess the results of experimental or theoretical numerical analyses, so that the research can be repeated and yield with the same or results within the limits of tolerable deviations, as stated in the paper.

Preliminary report contains the first short notifications on the results of research but without detailed explanation, i.e. it is shorter than the original scientific paper.

Review paper is a scientific work that presents the state of science in a particular area as a result of analysis, review and comments, and conclusions of published papers, on which the necessary data are presented clearly and critically, including the own papers. Any reference units used in the analysis of the topic are indicated, as well as papers that may contribute to the results of further research. If the reference data are methodically systematized, but not analyzed and discussed, such review papers are classified as technical papers.

Technical paper is a useful contribution which outlines the known insights that contribute to the dissemination of knowledge and adaptation of the results of original research to the needs of theory and practice.

Other contributions are presentations of objects, i.e. their structures and experiences (examples) in the construction and application of various materials (case studies).

In order to speed up the acceptance of papers for publication, authors need to take into account the Instructions for the preparation of papers which can be found in the text below.

Instructions for writing manuscripts

The manuscript should be typed one-sided on A-4 sheets with margins of 31 mm (top and bottom) and 20 mm (left and right) in Word, font Arial 12 pt. The entire paper should be submitted also in electronic format to e-mail address provided here, or on CD. The author is obliged to keep one copy of the manuscript.

As of issue 1/2010, in line with the decision of the Management Board of the Society and the Board of Editors, papers with positive reviews, accepted for publication, will be published in Serbian and English, and in English for foreign authors (except for authors coming from the Serbian and Croatian speaking area).

Each page should be numbered, and the optimal length of the paper in one language is about 16 pages (30.000 characters) including pictures, images, tables and references. Larger scale works require the approval of the Board of Editors.

Naslov rada treba sa što manje reči (poželjno osam, a najviše do jedanaeset) da opiše sadržaj članka. U naslovu ne koristiti skraćenice ni formule. U radu se iza naslova daju ime i prezime autora, a titule i zvanja, kao i ime institucije u podnožnoj napomeni. Autor za kontakt daje telephon, adresu elektronske pošte I poštansku adresu.

Uz sažetak (rezime) od oko 150-250 na srpskom i engleskom jeziku daju se ključne reči (do sedam). To je jezgrovit prikaz celog članka i čitaocima omogućuje uvid u njegove bitne elemente.

Rukopis se deli na poglavlja i potpoglovlja uz numeraciju, po hijerarhiji, arapskim brojevima. Svaki rad ima uvod, sadržinu rada sa rezultatima, analizom i zaključcima. Na kraju rada se daje popis literature.

Kod svih dimenzionalnih veličina obavezna je primena međunarodnih SI mernih jedinica.

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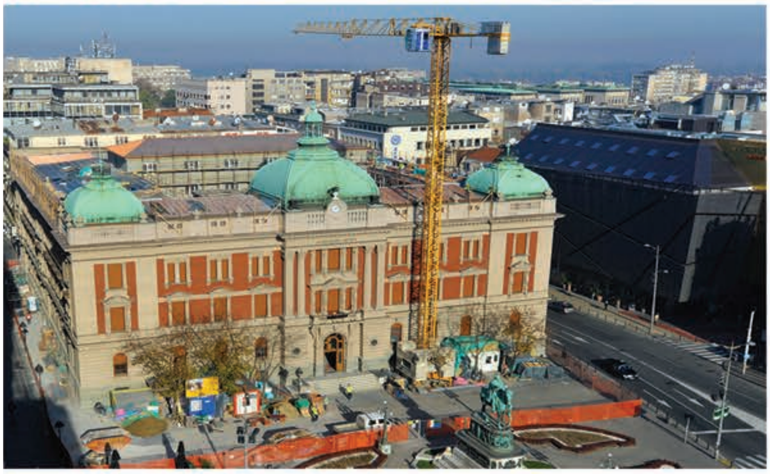


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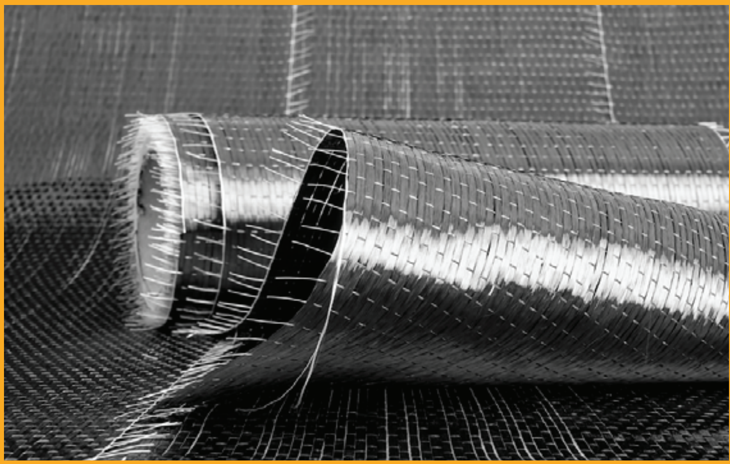

- Integrisani proizvodi i sistemi visokih performansi koji mogu da povećaju i poboljšaju kapacitet, efikasnost, trajnost i estetiku zgrada i drugih objekata – u korist naših klijenata i boljeg održivog razvoja
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