ISSN 2217-8139 (Print)
ISSN 2334-0229 (Online)

UDK: 06.055.2:62-03+620.1+624.001.5(497.1)=861

DIMK

2018.
GODINA
LXI

GRAĐEVINSKI MATERIJALI I KONSTRUKCIJE

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BUILDING MATERIALS AND STRUCTURES

ČASOPIS ZA ISTRAŽIVANJA U OBLASTI MATERIJALA I KONSTRUKCIJA
JOURNAL FOR RESEARCH OF MATERIALS AND STRUCTURES

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INTERNATIONAL EDITORIAL BOARD

PUBLISHER

Society for Materials and Structures Testing of Serbia, 11000 Belgrade, Kneza Milosa 9
Telephone: 381 11/3242-589; e-mail:dimk@ptt.rs, veb sajt: www.dimk.rs

REVIEWERS: All papers were reviewed
KORICE: AB zgrada bez zidova u prvoj etaži
COVER: RC building with open first floor

Financial supports: Ministry of Scientific and Technological Development of the Republic of Serbia

ISSN 2217-8139 (Print)
ISSN 2334-0229 (Online)

GODINA LXI - 2018.

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SADRŽAJ

CIP - Каталогизација у публикацији
Народна библиотека Србије, Београд

620.1

GRAĐEVINSKI materijali i konstrukcije : časopis za istraživanja u oblasti materijala i konstrukcija = Building Materials and Structures : journal for research of materials and structures / editor-in-chief Radomir Folić. - God. 54, br. 1 (2011)- - Beograd (Kneza Miloša 9) : Društvo za ispitivanje i istraživanje materijala i konstrukcija Srbije, 2011- (Novi Beograd : Hektor print). - 30 cm

Tromesečno. - Je nastavak: Materijali i konstrukcije = ISSN 0543-0798
ISSN 2217-8139 = Građevinski materijali i konstrukcije
COBISS.SR-ID 188695820

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Koce TODOROV
Ljupco LAZAROV

ORIGINALNI NAUČNI RAD
ORIGINAL SCIENTIFIC PAPER
UDK:624.072.33:692.2
doi:10.5937/GRMK1804003T

1 INTRODUCTION

The evaluation of the seismic performance of existing and newly designed structures is a relatively complex and multidisciplinary process. It includes elements of engineering seismology and soil dynamics, necessary to define the level of seismic hazard and the expected characteristics of input ground motion, elements of the dynamics of structures, for determination of the structural response, as well as elements of the structural mechanics, necessary for the accurately inclusion of the effects of material nonlinearity in the response of the systems under consideration, [2], [14], [22], [25].

Having in mind the expected structural behaviour to the permanent or seismic action, it would be logical to select appropriate analysis methods that can predict the structural behaviour with a high degree of confidentiality. If linear behaviour can be successfully predicted using linear methods for analysis, then it is expected that a nonlinear response should be determined with nonlinear analysis methods. Despite these expectations and considerable efforts for the conceptual transformation of earthquake engineering that have been made in the last twenty years, the generally accepted methods for seismic analysis are based on linear approximations. And these approximations are acceptable, if a certain number of effects are included in the process of analysis or design. Masonry infill, the influence of cracks in reinforced concrete elements, the contribution of effective slab width, soil - structure interaction, etc. are some of the parameters which can significantly change the desired seismic response of the analysed structures.

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2 ROLE OF MASONRY INFILL ON THE SEISMIC BEHAVIOUR OF STRUCTURES

Reinforced concrete frames with masonry infill are often used structural systems in construction practice worldwide. In this type of structures, the external and internal walls made of different materials, usually of ceramic blocks and bricks, are built as infill panels between reinforced concrete structural elements. Masonry infill is characterized with significant strength and stiffness and it can greatly alter the response of structures exposed to dynamic loads. The infill panels increase the structural stiffness, strength and damping and act as a first line of defence in seismic activity, reducing the ductility demand and consequent damage of structural elements. However irregular distribution of infill in plane and along building height can lead to series of unfavourable effects (torsion effects, dangerous collapse mechanisms, soft or weak storey, variations in the vibration period, etc.). That is why different opinions about the influence of the infill on the seismic behaviour of the structures can be found in the scientific and expert community [10], [15], [21], [29].

The presence of infill significantly changes the mechanism for lateral load redistribution [24]. Thus, the predominant frame system, in which the elements are exposed to bending, is transformed into a predominant truss system whose elements are generally exposed to axial action, Fig. 1.

Although this is an undeniable fact, according to the usual design practice, the interaction between the infill and the frame structure is most often neglected. This can lead to significant errors in determining the stiffness, bearing capacity and ductility of the analysed structure, which can especially be emphasized in reinforced concrete frames with discontinuity in the distribution of the masonry infill to the height of the building. Such structures in the literature are known as building with a weak floor (the strength on the lower floor is less than the strength of the upper floors), or structures with flexible or soft floor (the horizontal stiffness on the considered floor is lower than the stiffness of the upper floors). Although from a structural point of view these systems are unfavourable, for architectural or commercial reasons, they are quite attractive and exploited, especially in the central city cores and in densely populated urban areas. The open free space of such facilities, which is usually located on the first floor, is used as a corridor or as a space for accommodation in parking lots, shops, administration, etc., Fig. 2.

The influence of the infill on the seismic performance of reinforced concrete structures designed according to different national regulations has been the subject of a significant number of researches available in the literature, [6], [7], [9], [13], [16], [17], [18], [19], [21], [27].

3 DESCRIPTION OF ANALYSED STRUCTURES

In order to analyse the influence of the masonry infill on the behaviour of reinforced concrete frames that have been designed without taking into account the influence of the infill, a series of nonlinear static and dynamic analyses was conducted in this research. All analysed frames have been designed without taking into account the influence of the infill. In the phase of nonlinear assessment, all frames were additionally upgraded with the presence of masonry infill. Nonlinear static analyses were performed for bare and infilled frames. Two different distributions of infill panels were analysed: fully infill frames (FI) and infilled frames with open first floor (OS). Nonlinear dynamic analyses were performed for bare and infilled 2D frames with an open floor exposed to earthquakes with different intensities and with different frequency content. All numerical analyses were performed by the computer program SeismoStruct v6.0 [26].

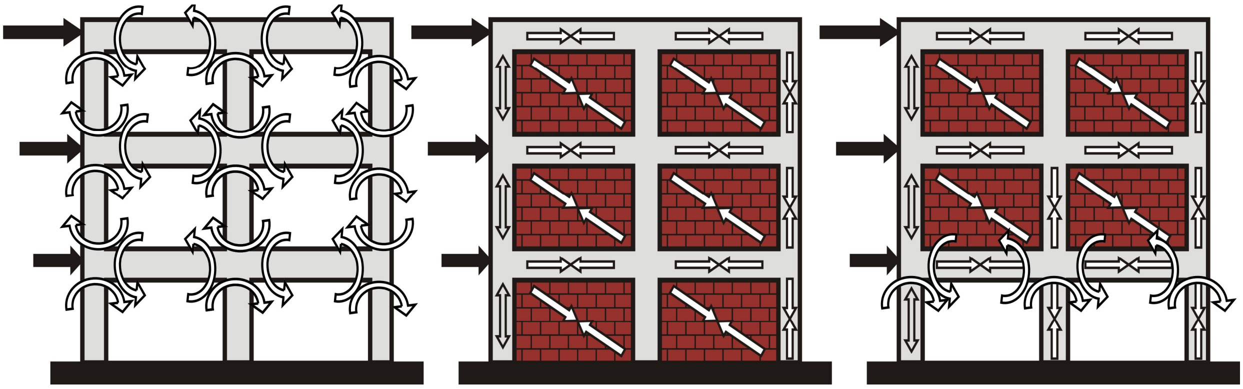

Fig. 1. Changes in the lateral load transfer mechanism owing to inclusion of masonry infill walls. (adopted from Murthy and Jain [24])

Fig. 2. Reinforced concrete buildings with open first floor

3.1 Design parameters

In order to cover structures with different dynamic characteristics, six reinforced concrete plane frames with different number of storeys (n=2, 3, 5, 7, 10 and 13), in the following text marked as frames R1, R2, R3, R4, R5 and R6, are generated in this investigation. All analysed structures are designed as three bay plane frames with a span of 5m and a constant storey height of 3m. The frames are designed to represent the exterior frame of a reinforced concrete spatial frame structure, Fig. 3. After the distribution of the surface load on the frames in both orthogonal directions, the beams of analysed frames were exposed to uniform load of 25kN/m' on the floors and 15kN/m' on the roof. At the beam - column joints additional concentrated forces, which represent the influence of the beams in the longitudinal direction, was applied. For the purpose of dynamic analysis and according to the defined loads, the distribution of the mass by floors, equal to 70.9t at each floor, and 38.2t at the level of the roof, was determined. A schematic representation of distribution of the gravity loads and the storey mass of the analysed frames is presented in Fig. 3.

The design of frames is done according to the actual Code of technical regulations for the design and construction of buildings in seismic regions, 1981 [31]. The total mass of the analysed frames (M_{tot}), the fundamental period of vibration T_1, the determined seismic design force (S), for the seismic intensity level IX (MCS), the ratio between the seismic design force and the weight of the frame (S/G), the maximum top displacement obtained from the action of the seismic forces (d_{max}) and the maximum stresses in the columns obtained from the action of the gravity loads are presented in Table 1.

Concrete with compressive cube strength of 30*MPa* (MB30), and reinforcement with yield strength of 400*MPa* (RA400/500) were used for the design of structural elements. In the columns of all frames, a minimum reinforcement ratio of 1% has been adopted, although this requirement is not explicitly set out in the Code. In order to increase the ductility of the beams, in the sections above the supports where plastic hinges are expected, a compression zone reinforcement at least 50% of the area of the tensile reinforcement is placed. The adopted dimensions of the structural elements and the adopted reinforcement in cross sections for all generated frames are shown in Fig. 4.

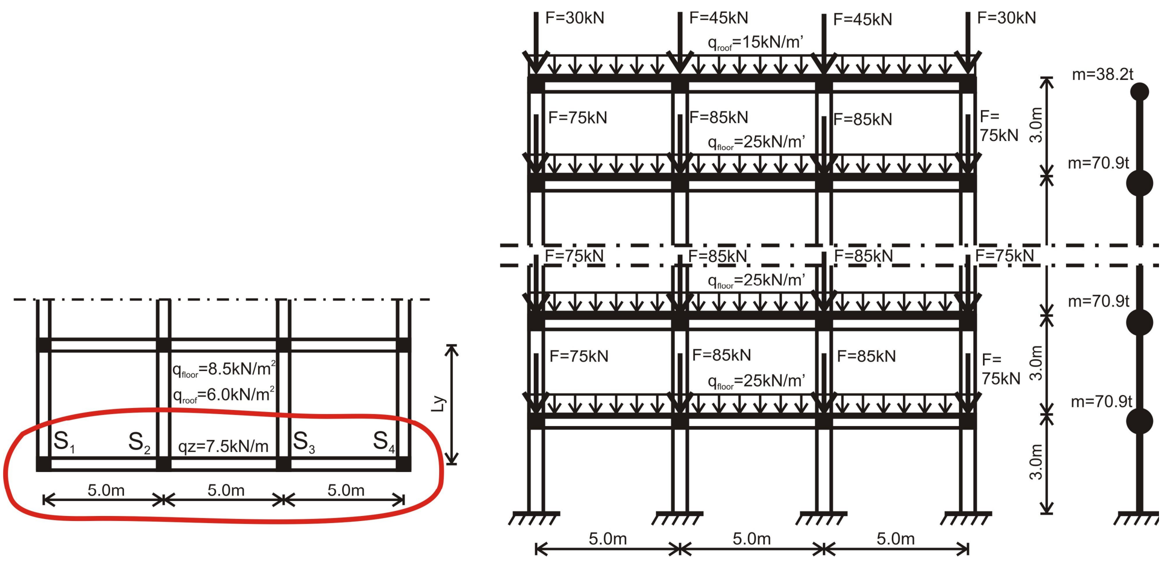

Fig. 3. Schematic representation of analysed frame structures

Table 1. Design characteristics of analysed frame

Frame	Num. of storeys	M_{tot} (t)	T_1 (sec)	S (kN)	S/G	δ_{max} (cm)	δ_{max}/ (H/600)	σ_0(g+p) (MPa)
R1	2	109.1	0.432	107.0	0.1	0.651	0.65	3.76
R2	3	180.0	0.517	176.6	0.1	1.075	0.72	4.57
R3	5	321.8	0.609	315.7	0.1	1.526	0.61	4.81
R4	7	463.6	0.820	388.2	0.085	2.668	0.76	5.70
R5	10	676.3	1.071	433.5	0.065	3.482	0.70	5.60
R6	13	889.0	1.386	440.2	0.05	4.586	0.71	6.06

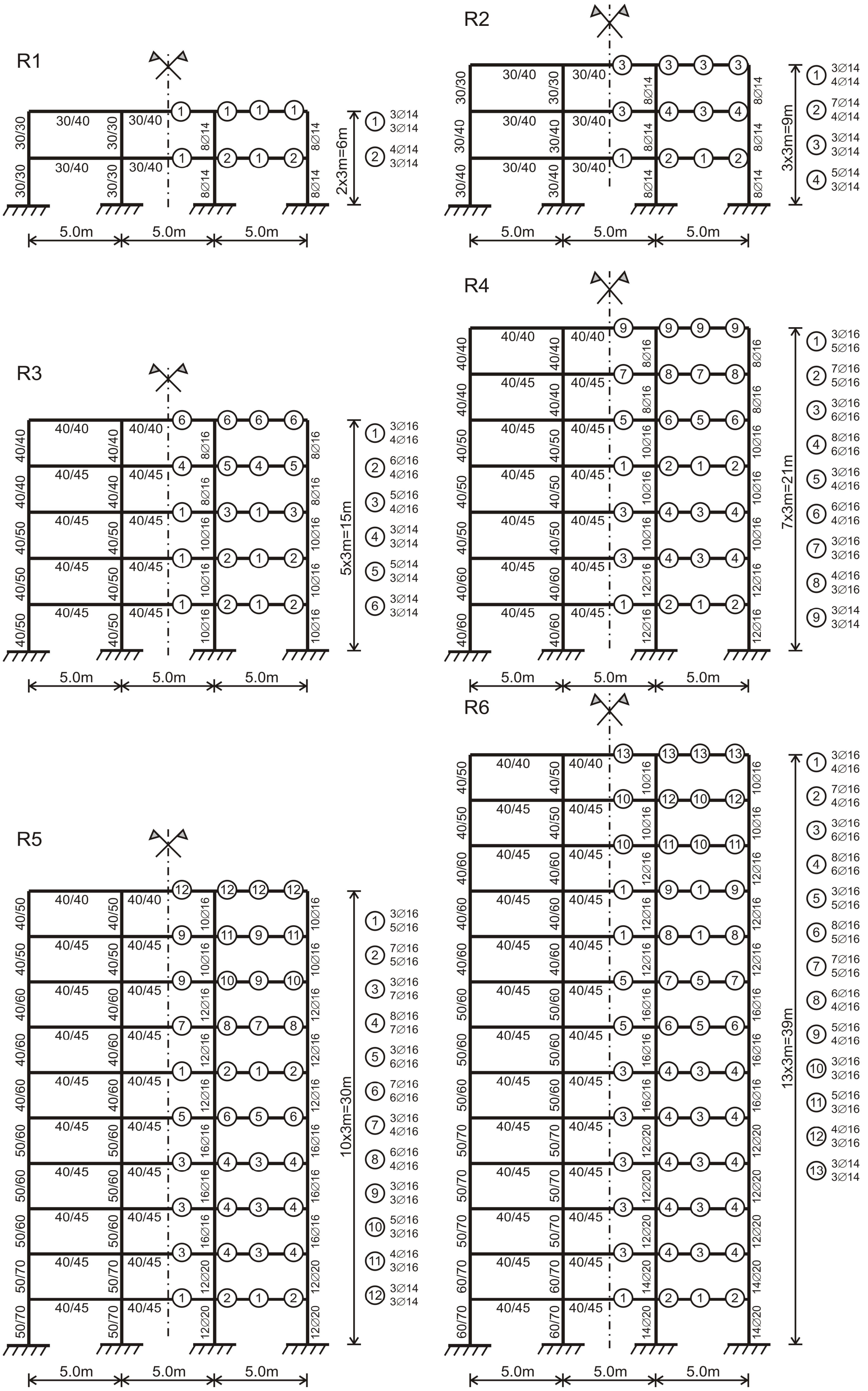

Fig. 4. Adopted dimensions and reinforcement in the cross sections of the analysed frames

3.2 Modelling assumptions

For the modelling of reinforced concrete elements, the model of distributed plasticity with the displacement based formulation of finite elements is used. Each structural element is divided into four subsections with different lengths. For the purpose of numerical integration, the cross sections in Gauss integration points are discretised with a triangular mesh of elements – fibres, with an optimal mesh density of about 100 fibres per cross-section. During the analysis, each fibre of cross-section is exposed to a uniaxial state of stress. Internal forces are obtained by numerical integration of stresses in the individual fibres. Mander's RC model [20] for confined and unconfined concrete and Menegoto - Pinto steel model with the Monti - Nuti buckling rules [23] for the longitudinal reinforcement was used to describe the stress - strain relationship in individual fibres. The nonlinear behaviour of masonry infill was modelled with the equivalent diagonal strut, using the model developed by Crisafulli [5], [28].

3.3 Adapted features of masonry infill

Masonry infill was defined with two different strength and stiffness characteristics, namely weak infill (WI) and strong infill (SI). The weak infill is characterized by the compression strength in the diagonal direction f_m=0.8MPa and a thickness of 15 cm., while the strong infill has a compression strength of 1.2MPa and a thickness of 25cm. An initial modulus of elasticity equal to $1500f_m$ was adopted for both types of infill. The compression strength is reached at strains of 0.002, while complete degradation of the strength and stiffness of the infill occurs at ultimate dilatation of 1%. The secant modulus of elasticity, at the point of achievement of compression strength is equal to one third of the initial modulus of elasticity, $E_{sec}=500f_m$. For defining the characteristics of the shear spring in the used material model, the initial shear bond strength of the infill τ_o=0.3MPa and coefficient of friction μ=0.5 were adopted. The maximum shear strength τ_{max} was limited to 0.6 MPa.

According to the adopted concept of modelling masonry infill, a compression strut is characterised with variable area which reduction is a function of the reached axial strains. The initial area of compression strut is a product of the thickness of the infill and the width of the equivalent diagonal, for which a value of 20% of the length of the diagonal is adopted. When strains in infill reach the value ε_1=0.0005, the area of the diagonal begins to decrease linearly. At strain equal to 0.0063, its area is 50% of the initial. Schematic diagrams of stress - strain relationship for masonry infill, reduction of strut area and defined diagram lateral force - inter-storey drift for masonry infill panel are presented in Fig. 5.

The limit values of the adopted strains correspond to a certain level of damage in masonry infill and according to the (Eq.1), can be directly connected with the achieved inter-storey drifts.

$$e = \frac{d}{2}\sin 2q \tag{1}$$

Strains at the beginning of the strut area reduction correspond to the state of cracks in the infill, which occur at inter-storey drift in range from 0.05% to 0.15%. These range of inter-storey drifts correspond to the operational seismic performance level.

Strain at maximum strength in the infill correspond to inter-storey drift from 0.4% to 0.5% and according to the performance level represent the end of the immediate occupancy state. The strains at the end of the diagonal area reduction correspond to inter-storey drift of 1% to 2% and are the limit values of the life safety performance level. The ultimate strain in the infill which corresponds to inter-storey drift in the range from 2-3% according to the level of seismic performance represents the collapse prevention limit state. The boundary ranges and the adopted values of strains for this research and corresponding inter-storey drifts for different performance levels are given in Table 2.

4 NONLINEAR STATIC ANALYSIS

Nonlinear static analysis is performed for the action of constant gravity load, which represents the initial state of stress and deformation in the structural elements and for the action of a monotonically increasing lateral load, which represents the inertial seismic forces initiated by the action of the seismic impact. It is well known that results of the nonlinear static analysis are strongly dependent on the pattern of distribution of the lateral force. Taking into account that in the frames with an open first floor, the effective modal mass in the first mode shape is over 80% of the total mass for all frames, i.e. over 90% for frames R1 to R4 with a strong infill and for frames R1 to R3 with a weak infill, in the further analyses assumed distribution of the lateral load according to the first mode shape was used. This assumption is

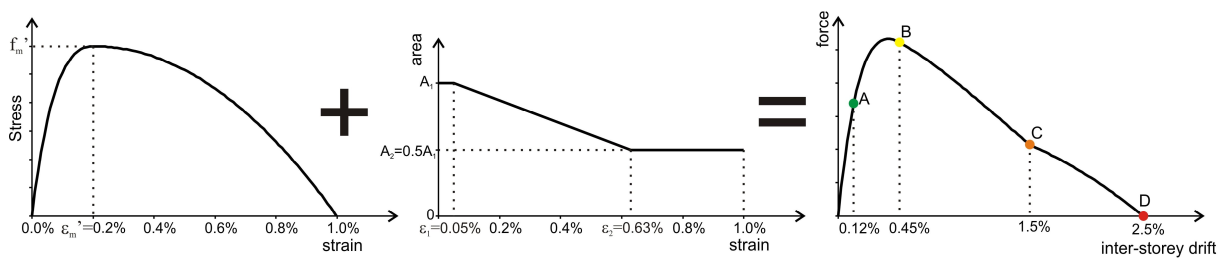

Fig. 5. Stress – strain relationship for masonry infill, reduction of strut area and defined diagram lateral force – inter-storey drift for masonry infill panel

Table 2. Performance level and corresponding boundary values of inter-storey drifts and strain in masonry infill

Performance level	Range	Boundary values of strain	Adopted strain limit	Boundary values of inter-storey drifts	Adopted values of inter-storey drifts
Operational	0-A	ε_A 0.02-0.07%	0.05%	δ_A 0.05-0.15%	0.12%
Immediate occupancy	A-B	ε_B 0.1-0.3%	0.2%	δ_B 0.25-0.7%	0.45%
Life safety	B-C	ε_C 0.4-0.85%	0.63%	δ_C 1-2%	1.5%
Collapse prevention	C-D	ε_C 0.75-1.25%	1%	δ_D 2-3%	2.5%

based on the fact that with the development of nonlinearities in the structural elements the participation of the effective modal mass of the dominant mode shape is usually increased. It should be noted that the distribution according to the first mode shape, determined for an undamaged structure, assumes the invariance of the mode shapes in function of the damage, which is usually false. However, changes in mode shapes, due to the influence of material nonlinearity, are usually small and do not have a major impact on the results of nonlinear static analysis, although the changes in vibration periods due to a reduction in the horizontal stiffness can be significant.

4.1 Capacity curves

The capacity curves for the analysed frames without and with masonry infill, determined for the distribution of the lateral load according to the first mode shape, are presented in Fig. 6.

On the vertical axis, the total base shear, expressed as a percentage of the total weight of the frame is displayed, while the horizontal axis shows the top displacements as a percentage of the total height of the frame (top drift). The curves are also marked by the points of appearance of the first plastic hinge in the structure, the points of formation of the mechanism, as well as the level of the design seismic force increased by 30%. The appearance of the first plastic hinge is defined by determination of the step in the analysis when the strain of the longitudinal reinforcement in any section of the frame elements reaches yield strains, which is 0.2%. The state of plastic mechanism formation is reached in the moment when plastic hinges occurs in at least two cross sections of all four columns of the considered frame.

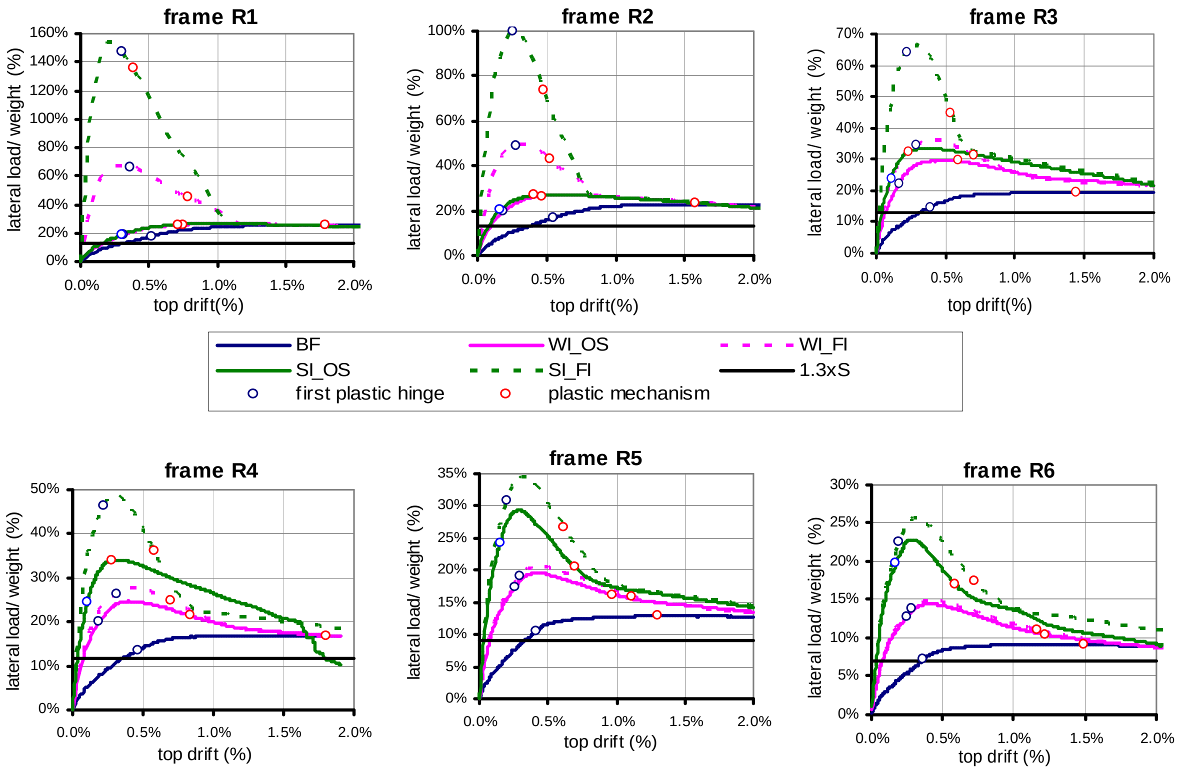

Fig. 6. Capacity curves of analysed frames

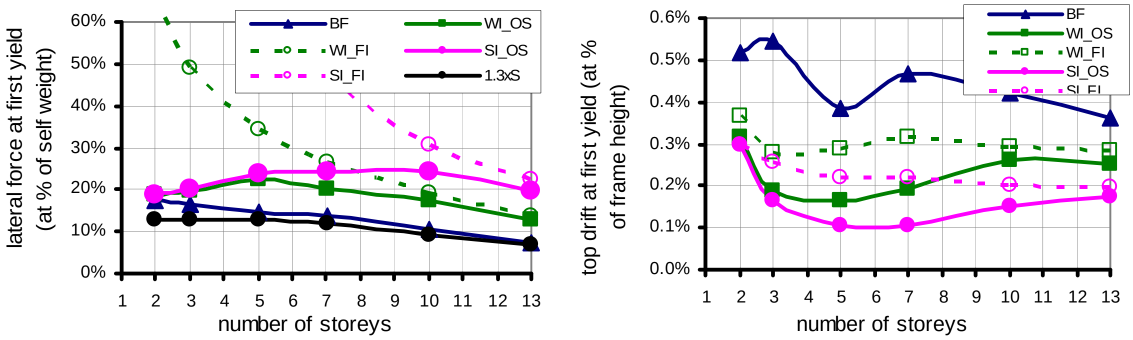

Fig. 7. Lateral force and top drift at occurrence of first plastic hinge for infilled and bare frames

From the diagrams presented in Fig. 6, it can be noticed that the masonry infill has favourable effects on the bearing capacity of the analysed frames, by increasing the level of the lateral force that causes the appearance of the first plastic hinge, as well as, by increasing the maximum capacity of the frames. The level of lateral force, as well as, the top drift at moment of first plastic hinge appearance for the infilled and bare frames is presented in Fig. 7.

Increasing the level of the lateral load at first plastic hinge in the infilled frames, compared to the bare frames, is a result of the redistribution of the internal forces due to the influence of the infill. Unlike the bare frames, where the first plastic hinge occurs in the beam elements of the lower storeys, in the frames with an open first floor, the first plastic hinge usually occurs in the support of one of the columns, Fig. 9.

The ratio between the lateral load at appearance of the first plastic hinge in the frame with the open floor and the bare frame, rises while increasing the number of storeys. This ratio is greater in the high rise frames with strong infill compared to the infilled frames with weak infill. Unlike open storey frames, in frames with a continuous infill, the ratio between the yield force for the infilled and bare frames decreases with increasing a number of storeys. At the highest frame the lateral load at the first plastic hinge formation for frames with open ground floor and frames with continuous infill is almost the same, indicating that the impact of the infill absence in the first floor is reduced by increasing the number of storeys, making the frames with the open floor to behave approximately in the same way as the frames with a continuous infill. Unlike the level of lateral load, the top drift at yield decreases due to the presence of the infill. The infilled frames reach the yield strength at smaller top displacement compared to the bare frames, Fig.6. The yield displacements in frames with weak infill are greater than the displacements in the appropriate frames with strong infill. This ratio is quite small in the frames R1 and R2, and rises with increasing a number of storeys. Top drift and the level of the lateral force, expressed as a weight ratio, obtained at the moment of plastic mechanism formation for the analysed frames are presented in Fig. 8.

The lateral force at the moment of plastic mechanism formation in low rise open storey frames is approximately the same as the lateral force in the respective bare frames, and is significantly less than the lateral force in the frames with continuous infill. With increasing a number of storeys, the differences between open storey frames and frames with continuous infill are reduced, while the differences between bare frames and open storey frames increases. In all frames with continuous infill, and in the frames R5 and R6 with open ground floor, the level of lateral force at mechanism formation is less than the level of force at the beginning of the yield. At the bare frames, the top drift at the moment of plastic mechanism formation ranges from 1.45% to 1.8%, while at the infilled frames these displacements are 2 to 6 times smaller. On the bare frames the plastic mechanism is manifested through the formation of plastic hinges at the ends of the beams almost along the entire height of the frames, while in the infilled frames a plastic mechanism is expressed through a soft story mechanism formed by the appearance of plastic hinges at both ends of the columns on the ground floor, or through a combination of the formation of plastic hinges at the ends of the beams and columns on the lower storeys, Fig. 9.

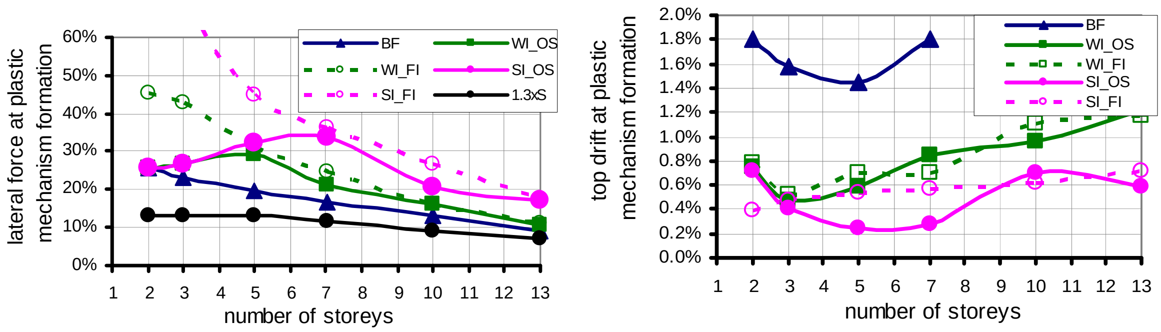

Fig. 8. Lateral force and top drift at the moment of plastic mechanism formation

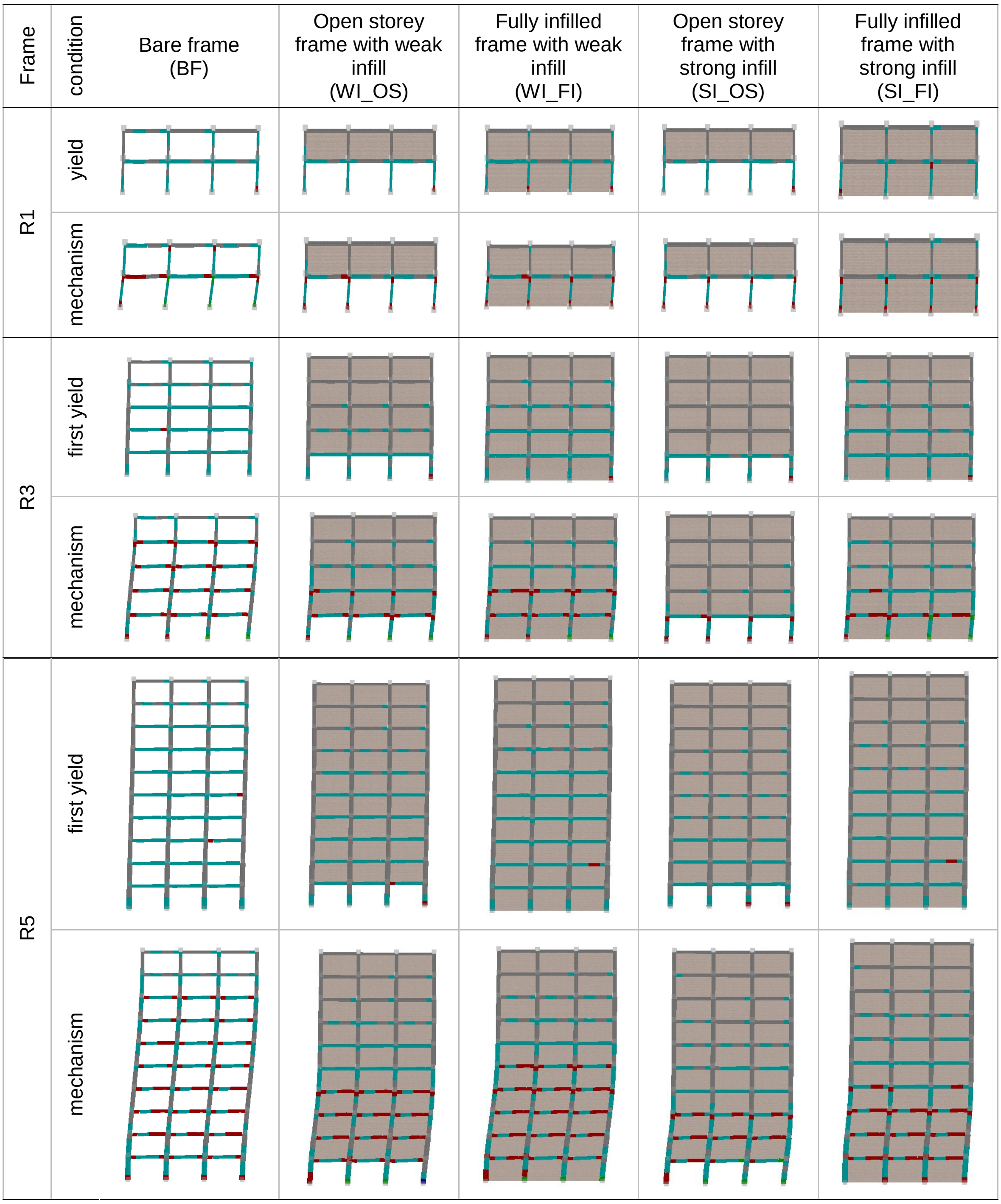

Fig. 9. Occurrence of the first plastic hinge and plastic mechanism formation for analysed frames

The state of occurrence of the first plastic hinge and the state of mechanism formation for the analysed frames is shown in Fig. 9. The blue colour indicates an element in which the condition of the appearance of first crack is reached; red colour represents the sections in which the yield strain in tensile reinforcement are reached; green colour marks a cross section at which the ultimate strain in the concrete cover is achieved, while the dark blue colour marks sections in which strains reach values 0.8% in the concrete core, i.e. in the confined concrete.

4.2 Stiffness degradation

The linear analysis theory is based on the constant stiffness of the structure, regardless of the degree of deformation of the structural elements. However, with the cracks appearance in the reinforced concrete elements and in the masonry infill, as well as, with the achievement of the yield strains in the reinforcement, the stiffness of the structure continuously degrades. The degree of degradation depends on the achieved deformation in the structural elements, which at global level are in function of the achieved lateral displacements. In Fig. 10 are presented the changes in the secant lateral stiffness, obtained as the ratio between the lateral force and the top displacements of the analysed frames. At low rise frames (2 and 3 storey), the initial lateral stiffness of the frames with open ground floor infilled with weak and strong infill is almost identical and is 2 to 3 times larger compared with the stiffness of the bare frames, but several times smaller than the rigidity of the frames with a continuous infill. With increasing the number of storeys, the differences in the lateral stiffness between the frames with the open first floor and the bare frames increases, unlike the differences in secant stiffness between the frames with open ground floor and continuous infill. The differences between the stiffness of the frames infilled with weak and strong infill are increased by increasing the number of storeys.

Fig. 11 shows the ratio between the secant and the initial lateral stiffness in function of the achieved lateral

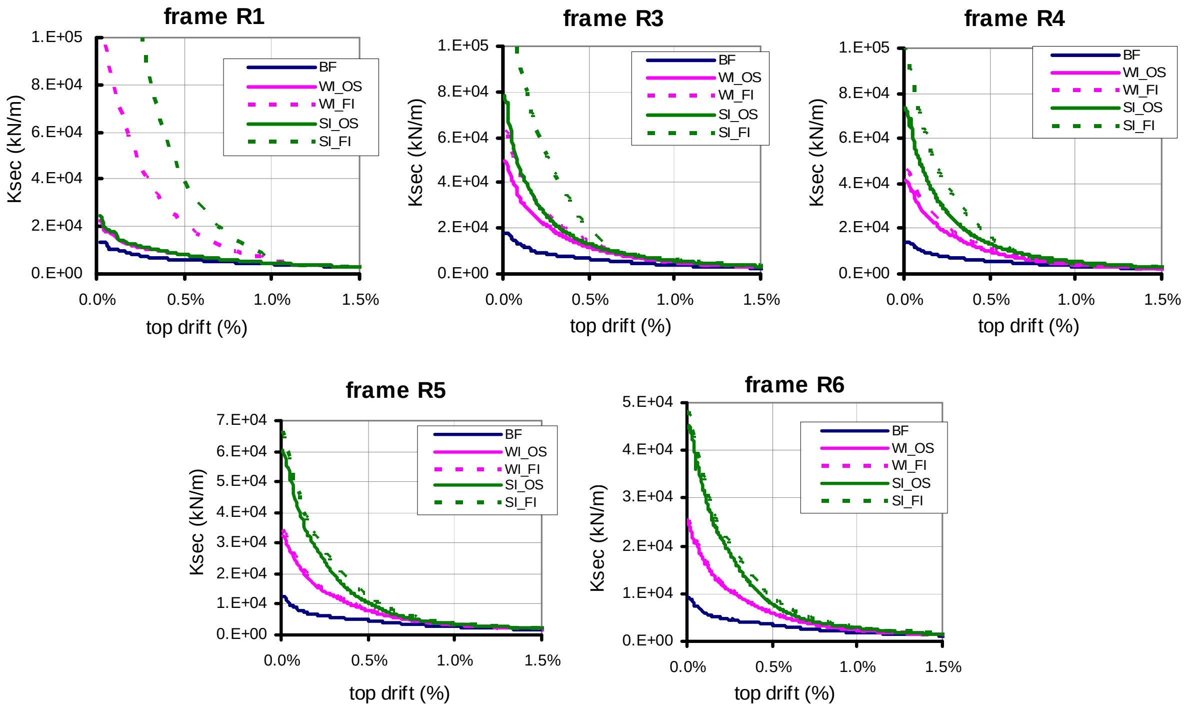

Fig. 10. Change on the secant stiffness of analysed frames

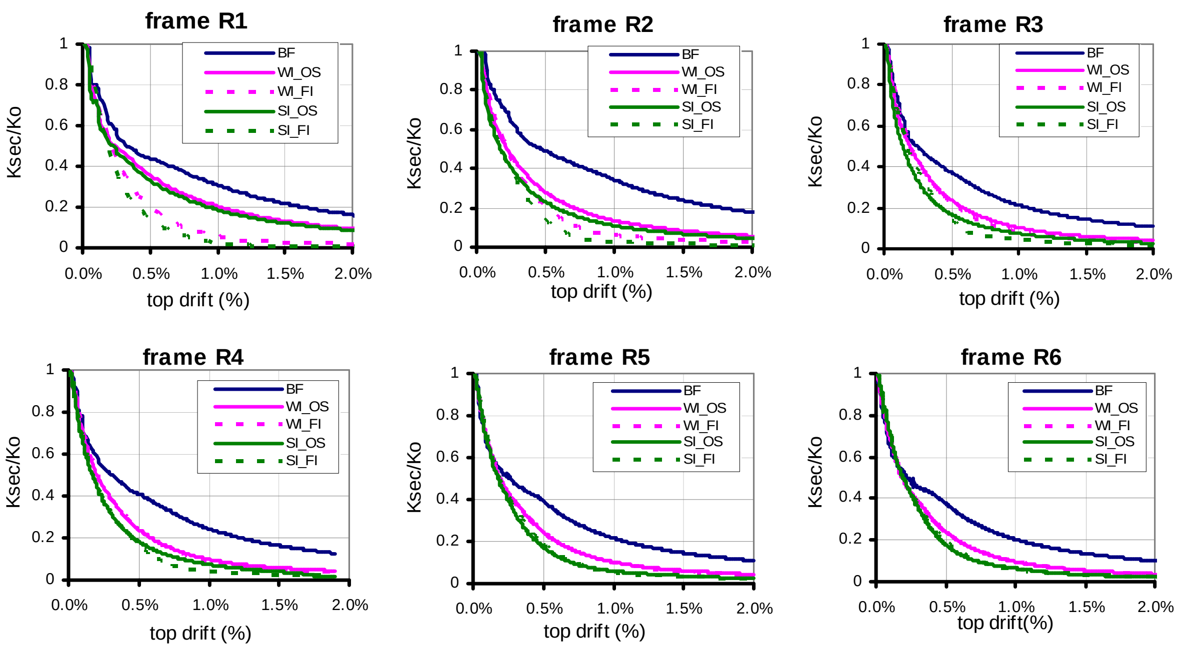

Fig. 11. Degree of stiffness degradation of analysed frames

top drift. The degree of degradation of the lateral stiffness of the bare and infilled frames is almost identical with the top drift in range from 0.1% to 0.15%, which is especially expressed at the higher frames. With the further increase of displacements, bare frames have a lower degree of degradation, compared with the infilled frames. The highest degree of degradation occurs in frames with a continuous infill over the entire height. Degree of degradation, almost does not depend on the number of storeys. At top drift equal to 0.5% of the total height, the secant stiffness, with respect to the initial ranges from 38% to 50% in the bare frames, is about 20% to 25% in the frames with an open ground floor and about 15% to 25% in frames with continuous infill.

At the moment of the appearance of the first plastic hinge, and due to the effects of cracks in reinforced concrete elements, the secant stiffness of the bare frames ranges from 42% to 47% in relation to the initial stiffness. These values are 3% to 8% lower than the recommendations for stiffness reduction according to Eurocode 8 (EN 1998-1:2004).

According to the obtained distribution of cracks through the structural elements, it can be noticed that the usual reduction of the stiffness is due to the appearance of cracks in the beam elements, in which the degree of cracks at the moment of first yield is significantly greater compared to the columns. This is especially expressed in the higher frames. The concept of equal reduction of stiffness in all elements is without greater impact on the global characteristics of the analysed structures, but it may have a significant impact on the redistribution of internal forces in structural elements with different degree of stiffness degradation.

5 INCREMENTAL DYNAMIC ANALISIS

An incremental dynamic analysis (IDA) is one kind of parametric nonlinear dynamic analysis that provides a continuous display, from elastic behaviour via yielding to the state of collapse, of the considered structures exposed to seismic action. The concept of incremental dynamic analysis was first proposed by Bertero [1], but a detailed description of the method and development of a methodology for its practical application was provided by Vamvatsikos and Cornell [30]. In this analysis structural model is exposed to one or more acceleration records, each of them scaled at multiple intensity levels. Incremental dynamic analysis is a pushover analysis pendant, with the difference that in the pushover analysis results are obtained with the incremental increase of static load, while in the incremental dynamic analysis with the increases of the intensity of the input ground motion. The results of the incremental dynamic analysis are presented in the so-called IDA curves, which give the connection between a certain intensity measure (IM) and the behaviour of the structure expressed through a certain measure of damage (DM). In order to carry out this analysis, it is necessary to define confidential data for the behaviour of structural materials and elements exposed to cyclic loading over the limit of elasticity, to select and to scale the acceleration ground motion and to define a stable algorithm for solving the system of differential equations of motion.

5.1 Selection and scaling of earthquake ground motions

In the probabilistic approach for the assessment of seismic performance, the analysis of the structural behaviour usually is performed for the multiple earthquake scenarios, while the evaluation of seismic performance is carried out by statistical processing of the obtained results. This approach allows determination of the probability of damage for a certain earthquake scenario and also allows identification of the ground motion records that have the greatest influence on the behaviour of a certain structure.

Each ground motion is characterized by a few engineering parameters, among which the parameters that characterize the amplitudes and the frequency content of the earthquake are the most important. In order to select records with defined frequency content from the existing database of earthquake ground motions, a methodology for the identification of dominant frequency domain in the acceleration spectrum was developed [29]. The developed methodology is based on the transformation of the acceleration spectrum into a modified cumulative spectral intensity diagram, which enables easy identification of the boundary periods of the dominant frequency range (T_1 and T_2), the mean dominant period - T_m, and the mean amplification of the acceleration spectrum. The defined procedure was applied to 910 registrations from the PEER NGA database [3], of which 21 registrations, divided into three groups of 7 records, Fig. 12, were selected and used for further analyses.

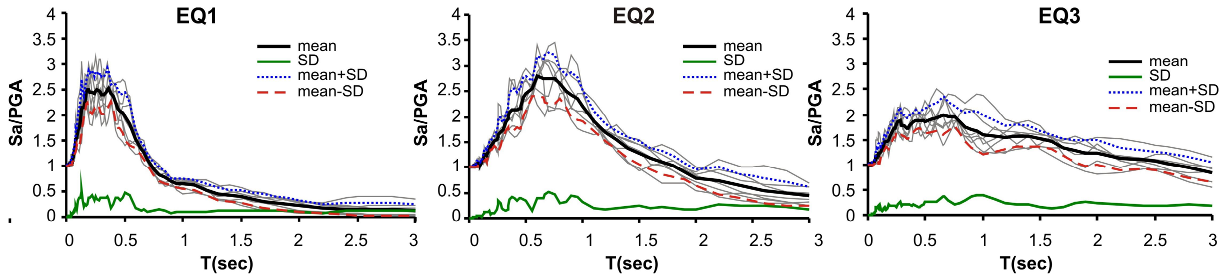

Fig. 12. Acceleration spectra of the selected records for analysis

The first group of earthquakes (EQ1) contains registrations of earthquakes with a predominantly frequency range of low periods. The boundary values of the dominant frequency for this group of earthquakes is in the range from (0.1sec. <T_1<0.15sec.) to (0.55sec.<T_2 <0.6sec.). Thus, the width of the dominant frequency band, B_T, is confined within the limits (0.4 <B_T <0.5 sec.), and the mean period T_m is defined in boundaries from 0.325sec to 0.375sec. This group of earthquakes should expose stiff structures to oscillate dominantly in the first mode, while more flexible structures to oscillate in higher mode shapes. The second group of earthquakes (EQ2) is defined by a predominantly frequency range of medium to high periods, (0.24sec.<T_1 <0.35sec.) and (1.1sec.<T_2<1.5sec.). The width of the dominant frequency band in this group can range from 0.75 to 1.26 sec., while the median period can range from 0.725 to 0.91 sec. A third group of earthquakes (EQ3) is defined by the requirement that the width of the dominant frequency range is within 1.8 to 2.6 sec., without imposing limits on the boundary values for the periods of the dominant frequency range.

The procedure for records selection is carried out in two steps. In the first step, according to the defined limit values of the dominant frequency ranges that meet the set criteria, 45 records from group 1, 34 from group 2 and 55 from group 3 were selected. In the second step of selection, seven records with the smallest cumulative deviation were selected. The acceleration spectra of the selected records from each group, their mean spectrum, the standard deviation, and the mean spectrum ± one standard deviation are presented at Fig. 12.

To meet the requirements of incremental dynamic analysis the selected records were scaled to ten different amplitudes, based on scaling of pick ground acceleration. The records from the first group of earthquakes were scaled for pick ground acceleration in the range from 0.1g to 1.0g, while the records from the second and the third group of earthquakes for pick ground acceleration in the range from 0.05g to 0.5g

5.2 Analysis of the obtained results

From the conducted nonlinear dynamical analyses, a large number of output data, which present the global response of the analysed structures, as well as a large number of results that show the structural response at the local level are obtained.

Maximum top displacements, expressed as a percentage of the total height of the analyed frames, are noted at lower frames. By the increasing of number of stories, the maximal top drifts are reduced, which is particularly pronounced for the case of earthquakes with a dominant frequency range of low periods. Bare frames (BF) have a larger top displacements compared with the infilled frames, which is more emphasized at lower levels of peak ground acceleration.

Infilled frames with strong infill (SI) reach smaller values of maximal top displacement compared to the frames with weak infill (WI), which is more significant for lower levels of peak ground acceleration and for higher frames. The frequency content of the ground acceleration records is the most influential factor on the degree of reached top drifts. The peak top displacement at the masonry infilled frames with an open first floor, calculated as the mean value from the results obtained for peak ground acceleration of 1g, for records characterized by a dominant frequency range of low periods, are within the range of 0.5% to 2% of the total height of the frames, Fig. 13. Similar values of the maximum top displacement, for the action of records with a predominantly frequency range of medium to high periods, as well as for registration with wide frequency content, are obtained for pick ground accelerations which are 3 to 4 times smaller compared to the records with dominant frequency range at low periods. Except to the influence on the level of maximum top displacement, masonry infill has a great influence on the distribution of displacements through the height, as well as on the distribution and amplitudes of the maximum inter-storey drifts.

At lower levels of peak ground acceleration, when the masonry infill is in the elastic domain of behaviour, it significantly stiffens the structure, and therefore reduces the maximum inter-story drifts. At higher levels of seismic action, i.e. when the degradation of the stiffness and strength of the infill occurs, the inter-storey drifts of the infilled frames increase significantly faster compared with the bare frames. Strong infill has a favourable effect on the degree of maximal inter-storey drifts to the certain level of peak ground acceleration. At higher level of PGA, the strongest infill leads to an undesirable failure mechanism, which increases the level of inter-story drifts. Relationship between maximum inter-storey drift and pick ground acceleration for analyzed frames exposed to second group of earthquakes is presented at Fig. 14.

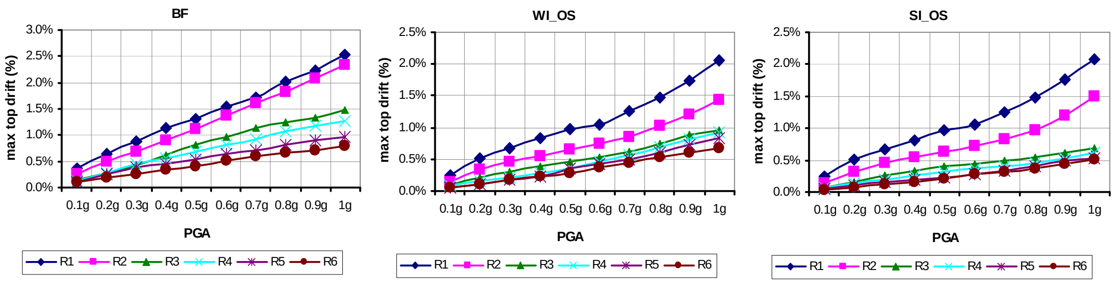

Fig. 13. Maximal top drift in function of pick ground acceleration for analysed frames exposed to first group of earthquakes

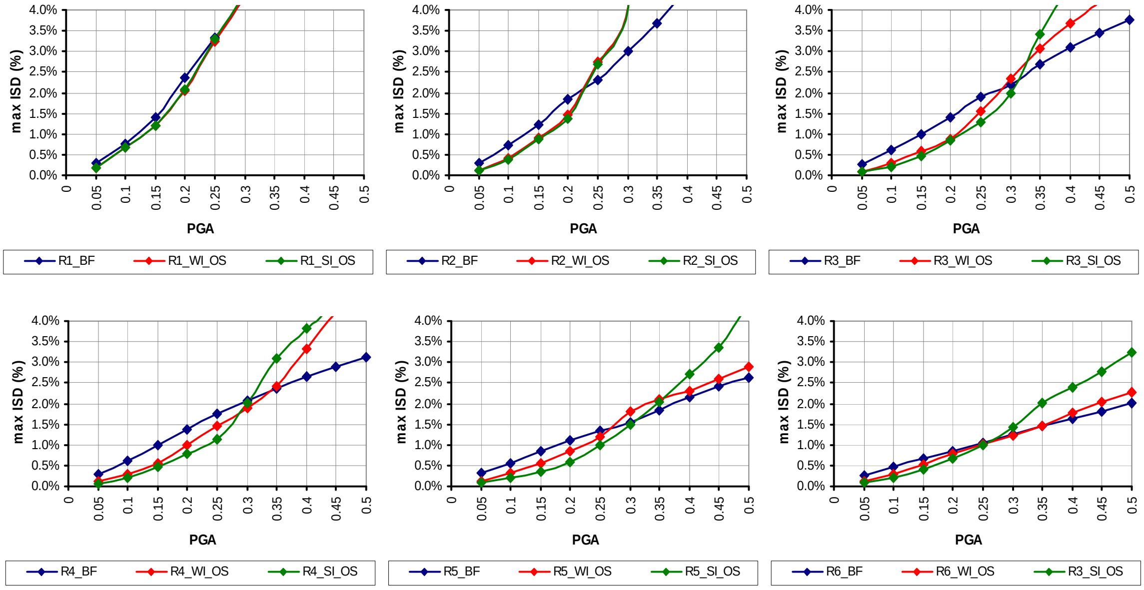

Fig. 14. Maximum inter-storey drift in function of pick ground acceleration for analysed frames exposed to the second group of earthquakes

Soft story mechanism of failure, expressed with the formation of plastic hinges at the ends of the columns on the first storey, where the infill is missing, is observed at the low frames with 2 and 3 storeys, regardless of the quality of the infill, as well as in the five storey frame with a strong infill, regardless of the characteristics of the ground motion, Fig. 15. At the higher frames, the failure mechanism is represented by the formation of plastic hinges in the beams and columns on the first few floors, Fig. 16.

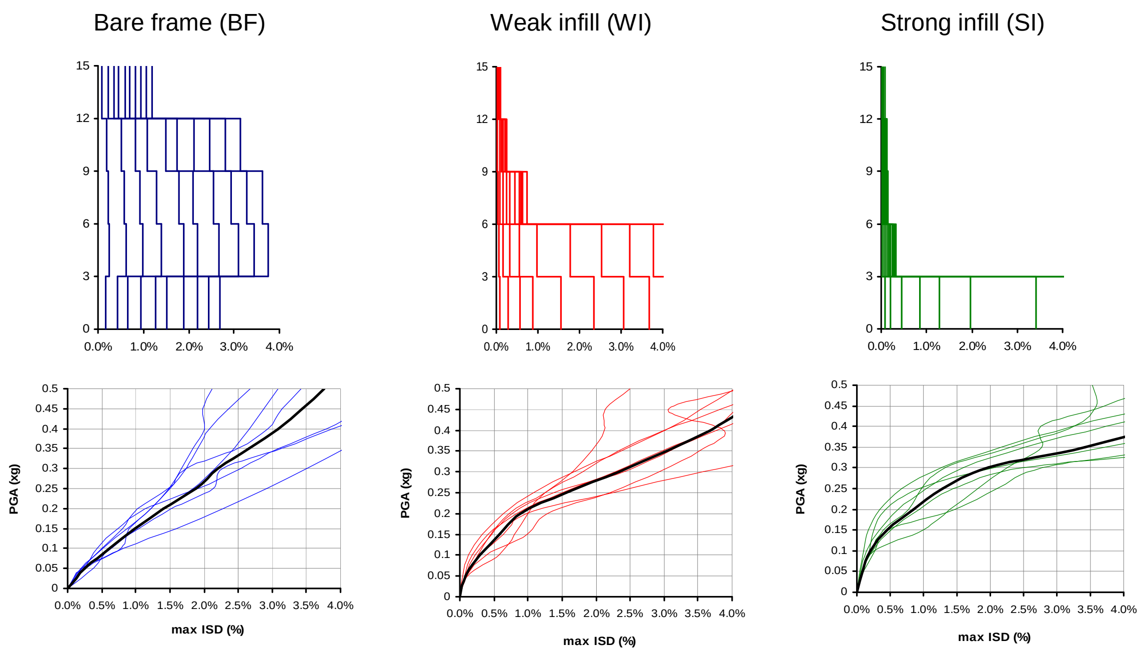

Fig. 15. Distribution of inter-storey drifts and IDA curves for five-storey frame R3 exposed to the second group of earthquakes

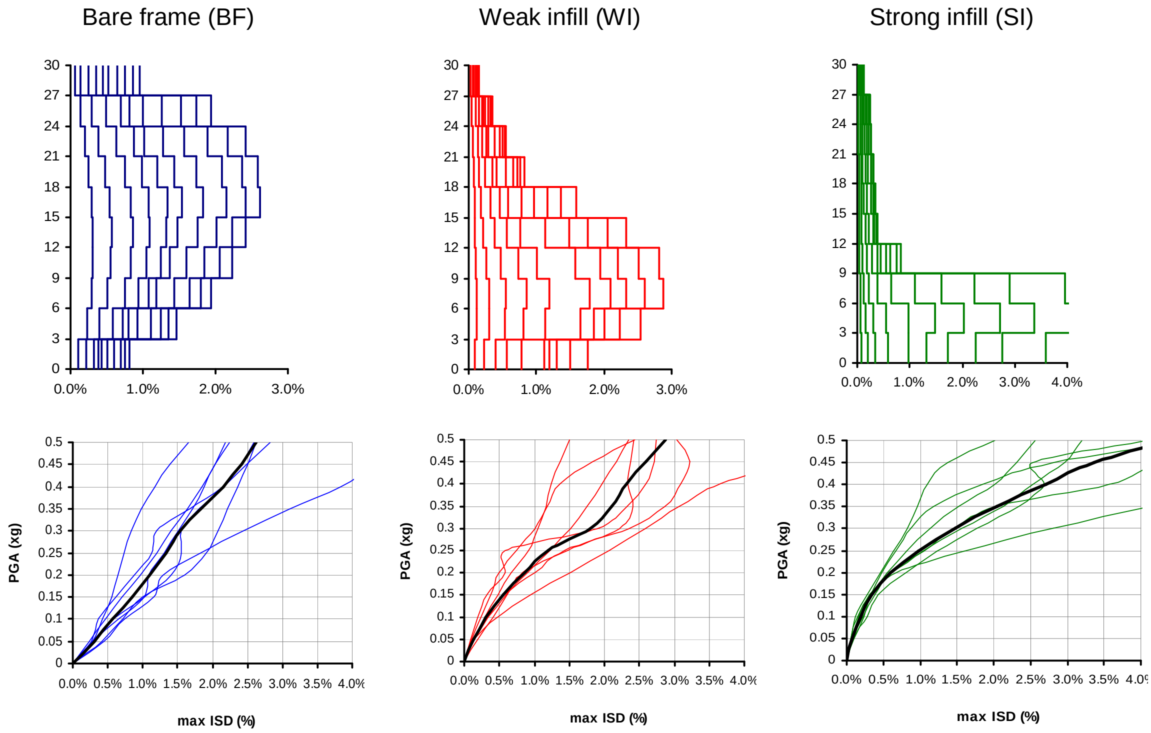

Fig. 16. Distribution of inter-storey drifts and IDA curves for ten storey frame R5 exposed to second group of earthquakes

5.3 Discrete levels of seismic performance

The distribution of inter-storey drifts along the height is a direct indicator of the total damage degree that can occur within the frames for different earthquake scenarios. From the determined incremental curves, which present the relationship between the pick ground acceleration and maximum inter-story drifts, the discrete seismic performance levels can be determined.

Depending on the achieved strains in the infill, which are related with the inter-storey drifts, Table 2, four limit states of seismic performance are defined: operational (δ_A=0.12%), immediate occupancy (δ_B=0.45%), life safety (δ_C = 1.5%) and near collapse with maximum inter-story drift δ_D = 2.5%. It should be noted that the same limit values of the inter-story drifts are used for all floors within the infilled frames, regardless of whether the extreme values appear at the level of the first open floor, or on any of the upper floors. In order to compare the results, the same limit values were used to define the limit states and for the bare frames.

For the action of the first group of earthquakes, the most vulnerable for all levels of seismic performance are low rise frames. The operational limit state for this group of earthquakes is achieved for peak ground acceleration in the range from 0.03g for frame R2 to 0.16g for frames R5 and R6 with weak infill or 0.2g for frame R6 with strong infill. The immediate occupancy limit state in lower frames is achieved at PGA from 0.1g to 0.12g, while in the higher frames, it increases to around 0.45g for frames with weak infill or 0.57g for frames with strong infill. The life safety limit state at lower frames is expected to be achieved for pick ground acceleration of about 0.4g, while in the higher frames this condition is not reached in the considered range of intensity. The near collapse limit state in frames with a strong infill is expected to be reached at peak ground acceleration in the range from 0.7g for frames R1 and R2 to 1g for frame R4. In the case of lower frames with weak infill this limit state is reached for 2% to 4% higher peak ground acceleration compared with the frames with strong infill, while in the higher frames the achievement of this limit state is not registered. The relationship between peak ground acceleration, number of storey and achieved limit states, for the first set of earthquakes is presented in Fig. 17.

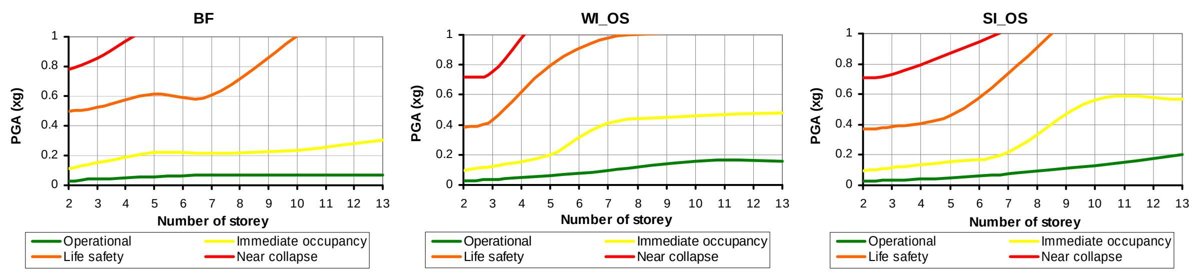

Fig. 17. Limit states of seismic performance for analysed frames exposed to first group of earthquakes

The achievement of the defined limit states in the considered infilled frames, for the action of earthquakes from the second or third group, differs considerably compared to the first group, which is more significant at the higher limit states, Fig. 18 and Fig. 19. The operational limit state is achieved for the PGA from 0.05g to 0.07g, for the action of the second group of records, up to a maximum value of 0.1g for the frame R5 exposed to the third set of records. The state of immediate occupancy, for frames up to 5 storey exposed to the second group of earthquakes, is reached for PGA in the limits from 0.08g to 0.15g, or from 0.1g to 0.2g for the action of earthquakes from the third group. At the higher frames with weak or strong infill, this condition is achieved at average PGA values from 0.13g to 0.16g for the second, or from 0.15g to 0.2g for the third set of records. The life safety limit state is achieved for peak ground acceleration from 0.17g to 0.35g. Lower values of intensity usually are related to the lower frames, where the formation of the soft storey mechanisms at the level of the first storey occurs. Soon after the achievement of life safety limit state, the performance level of near collapse is reached. This limit state is reached for peak ground acceleration in range from 0.22g to 0.42g, depending on the number of storeys and characteristics of the ground acceleration.

According to the achieved levels of seismic performance, it can be concluded that in order to provide the limit state of functionality, masonry infill, through the additionally added stiffness and strength, usually play a positive role, reducing the seismic demand. Due to the possibility of the appearance of a brittle failure in the infill, which can be reflected by the appearance of weak parts in the structure, it can be said that the masonry infilled frames are unreliable for providing higher limit states. In the lower frames with an open first floor, due to the presence of the infill on the upper floors, an undesirable soft storey mechanism usually occurs.

6 COMPARISON OF RESULTS

Nonlinear static and nonlinear dynamic analysis with a satisfactory degree of accuracy can predict the response of a structure when entering into a nonlinear region of behaviour, [8], [11], [12]. Due to take into account material nonlinearity both methods of analysis use similar material models. The same iterative methods are used to achieve the criteria of convergence and identification of the critical zones in the structure and prediction of the possible failure mechanism. Unlike nonlinear dynamic analysis that gives a direct relationship between the achieved nonlinear deformations and the level of seismic action, the nonlinear static analysis gives a continuous representation of the relationship between the total base shear and the top displacements, without providing discrete values of the reached displacement that will correspond to a certain level of seismic hazard. The relationship between the total base shear and the achieved displacement in the nonlinear static analysis depends on the distribution of lateral forces, while in the nonlinear dynamic analysis this relationship depends on the characteristics of the input ground motions, participation of mode shapes as well as the degree of equivalent viscous damping. An overview of the capabilities and general systematization of methods for seismic analysis of structures can be found in [4].

In order to perceive the possibility for prediction of a nonlinear dynamic response by application of a nonlinear static analysis, in the case of masonry infilled frames with open first floor, a direct comparison of the results obtained from these two analyses was made. The comparison of the results is based on two parameters that characterize the capacity curve obtained from the nonlinear static analysis, namely the top displacement and the total base shear. In nonlinear static analysis, for a known vector of distribution of lateral load, these parameters are uniquely defined, that

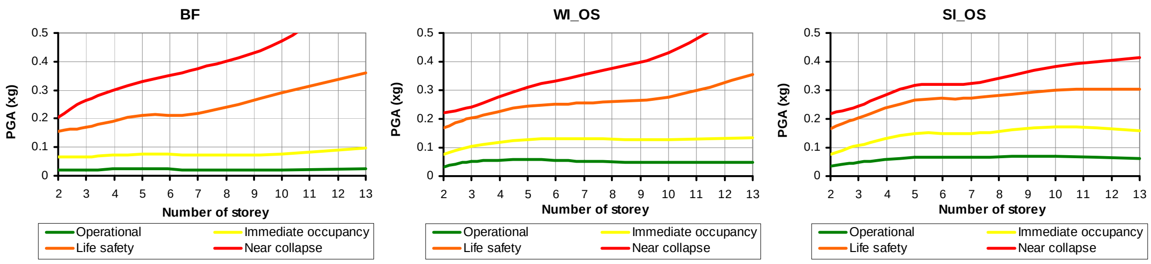

Fig. 18. Limit states of seismic performance for analysed frames exposed to second group of earthquakes

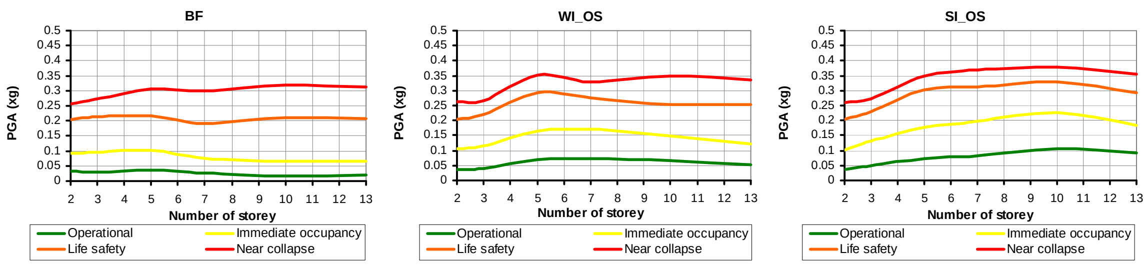

Fig. 19. Limit states of seismic performance for analysed frames exposed to third group of earthquakes

is, each displacement corresponds to a defined level of the base shear. In nonlinear dynamic analysis, due to the degradation of the lateral stiffness and strength during the seismic action, several levels of total base shear can correspond to one level of displacement. Having in mind that nonlinear static analysis aims to present the envelope capacity of the structure, the selection of the results for comparison from the nonlinear dynamic analysis is reduced to the determination of the maximum values of the displacements and the total base shear. However, in that case, a three combinations of extreme values are possible:

- Maximum displacement - corresponding base shear,
- Maximum base shear - corresponding displacement,
- Maximum base shear – maximal displacement.

In the masonry infilled frames, due to the contribution of the infill on bearing capacity of the frame, the maximum base shear is reached at relatively small displacements, so the second and third variants of the combination of extreme values can lead to unrealistic comparison results. Namely, the second variant would limit the response to displacement when the maximum capacity is reached, while the third variant would give a combination of values even after achieving the displacement at maximum force, but would not be able to present the decreasing branch of the capacity curve which occurs after the achievement of the maximum strength and is the result of the degradation of the infill strength. In addition, attention should be paid to the direction of the lateral force and corresponding displacement. In certain cases, it is possible to compare the absolute maximums, not considering the direction, which can also lead to unrealistic results. To illustrate the differences that can arise from the combination of different extreme values, several comparisons have been made. In Fig. 20, part of the results obtained for the case of the frame R5 with strong infill exposed to the input ground motion from the third group of earthquakes are presented. The global hysteresis diagram (total base shear – peak displacement) for PGA equal to 0.45g, the capacity curve obtained from the nonlinear static analysis, for the distribution of the lateral force according to the first mode shape, and the three points corresponding to the three combinations of extreme values for the defined level of peak ground acceleration are presented in Fig. 20a. In addition to the capacity curve, through a series of points, the incremental relationship total base shear - top displacement for the three mentioned combinations of the extreme values obtained from the incremental dynamic analysis for the range of peak ground acceleration from 0.05g to 0.5g with step of 0.05g are presented in Fig. 20b.

From the diagrams presented in Fig. 20 it can be noted that for small values of peak ground acceleration, the three combinations of extreme values give approximately the same results, which mainly overlap with the capacity curve obtained from nonlinear static analysis. The similar results of the three combinations are due to the fact that the maximum values of the displacement are attained at the same or close time moment with the moment when the maximum base shear occurs. This is especially expressed in lower frames where the first mode shape is dominant, so the increment of displacements corresponds to the increment of the total base shear. In higher frames, where higher mode shapes may have a significant impact in the distribution of lateral force by height, the total base shear may begin to decrease, and the top displacement continue to increase. For illustration, Fig. 21 presents one segment (8.1 to 8.42 sec) of the response histories

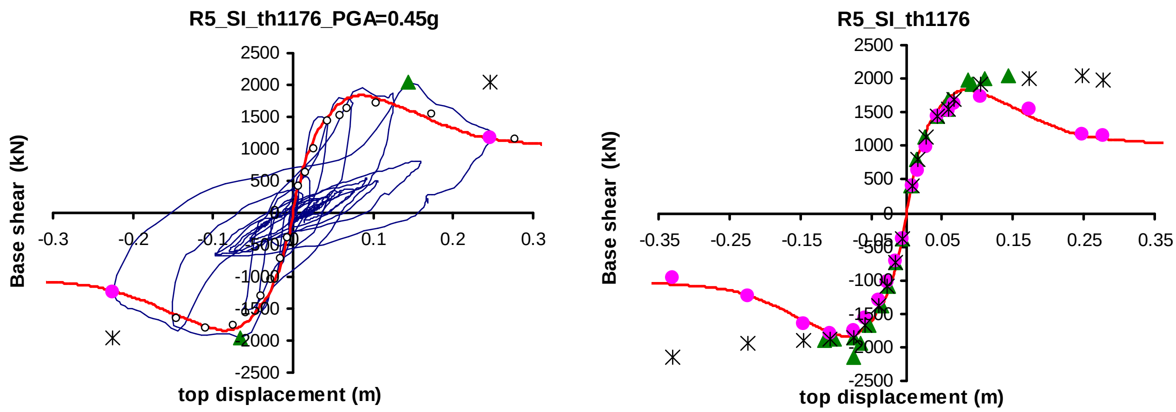

Fig. 20. Comparison of capacity curves obtained from nonlinear static and incremental dynamic analysis for different combinations of extreme values

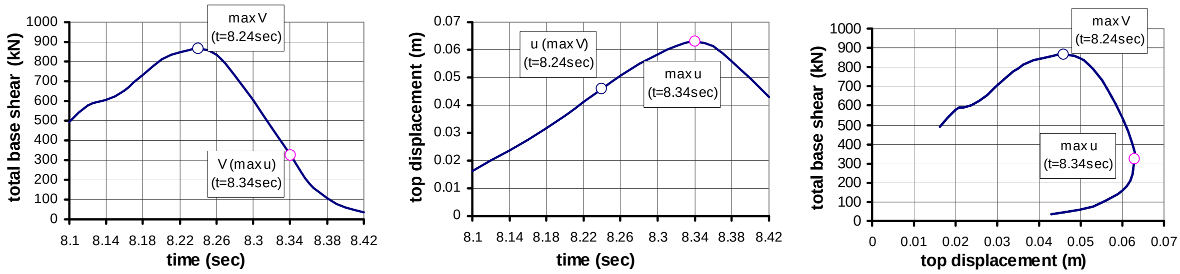

Fig. 21. Different time moments of reaching the extreme values of top displacement and lateral force due to the influence of higher mode shapes

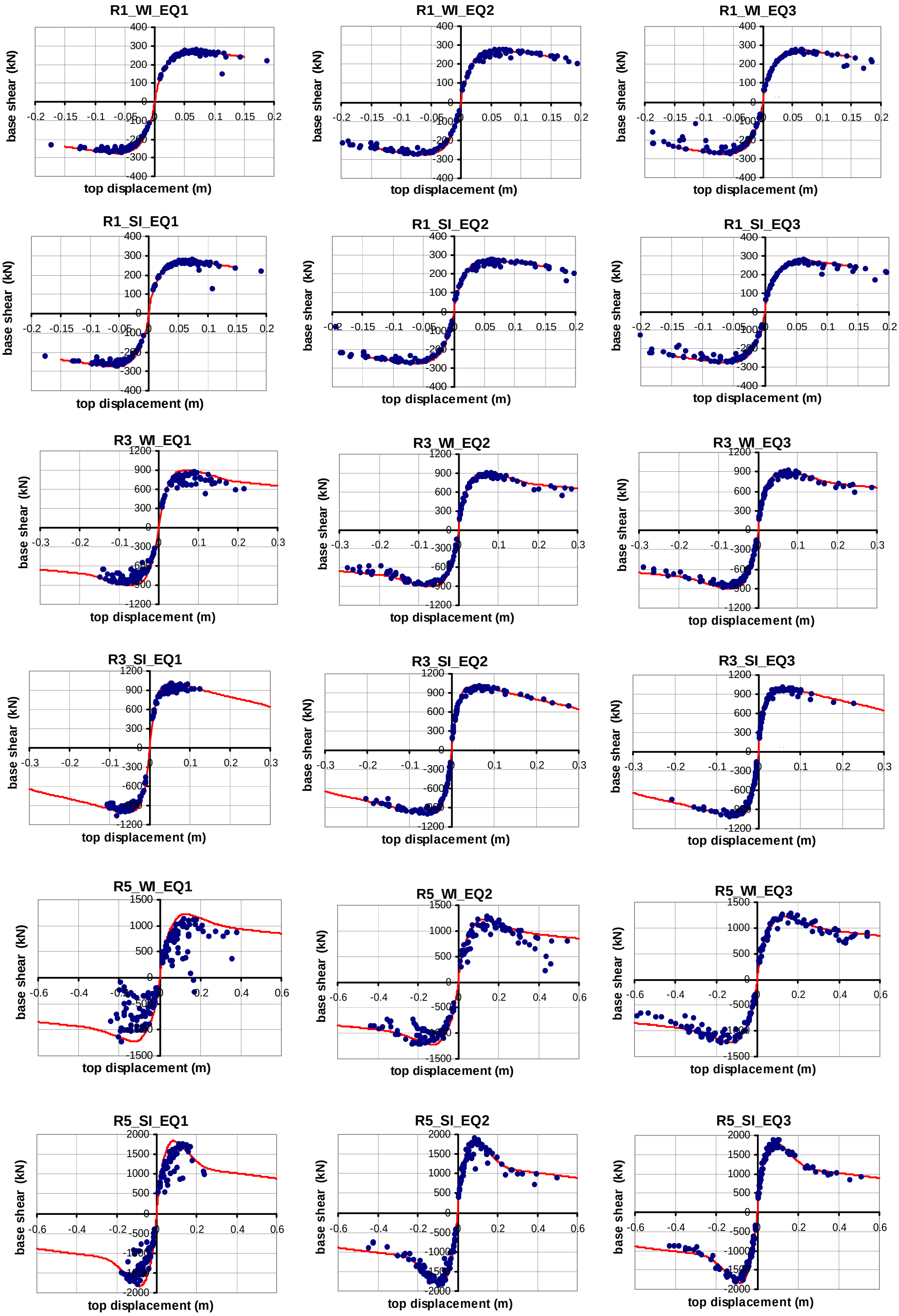

Fig. 22. Total base shear – top displacement relationship, obtained from nonlinear static and incremental dynamic analysis

of the total base shear and the top displacement, as well as the hysteresis diagram total base shear - top

Fig. 22 presents maximum top displacement - appropriate base shear diagrams, for the analysed two (R1), five (R3) and ten (R5) story infilled frames, with the open first floor, obtained from the incremental dynamic analysis for the individual records from the selected three groups of earthquakes, as well as the corresponding capacity curves obtained from the nonlinear static analysis.

From the relationship maximum top displacement – total base shear, obtained from the incremental dynamic analysis of the action of three groups of earthquakes for six analysed frames with a weak and strong infill, as well as from their comparison with the capacity curves obtained from the nonlinear static analysis, for the distribution of the lateral force according to the displacements in the first mode shape, the following observation can be noted:

- In lower frames R1, R2 and R3, there is almost an ideal overlap of the results obtained from the incremental dynamic and nonlinear static analysis, regardless of the quality of the infill and frequency contents or the intensity of the ground motions. Certain deviations can be noticed in the frames R1 and R2 exposed to the third group of earthquakes, as well as within the frames R2 and R3 exposed to the records from the first group of earthquakes. Deviations within the frames R1 and R2 are usually due to the achievement of large top displacements ranging from 1.5 to 2.5% of the total height of the frames at some previous time step, leading to significant degradation of the strength and stiffness. For illustration, Fig. 23 presents the hysteresis diagram total base shear - top displacement for the frame R1 with a weak infill obtained from the record of the third group of earthquakes with a peak ground acceleration equal to 0.5g. The maximum displacement in the negative direction for this record is 11.3 cm and is reached at time t=26.18 sec. once it was previously at the time moment t = 22.6 sec. a maximum displacement of 19.3cm was displacement, for the frame R5 with weak infill exposed to the ground motion from the first group of earthquakes. achieved in a positive direction. Significant degradation of the strength and stiffness has been caused due to the influence of the previous damage history in the opposite direction, so the system is no longer able to follow the envelope curve, i.e. to reach the lateral force at which the maximum displacement was achieved in some previous step.
- Deviations in results for the frame R3 with weak infill exposed to earthquake actions from the first group are due to the occurrence of significant damage in the infill on the upper floors and due to the impact of the higher mode shapes. Therefore, the maximum base shear that occur when the maximum displacements are reached is usually less than the base shear that correspond to the distribution of lateral force according to the first mode shape.
- In higher frames, R4 to R6, the greatest deviations occur in frames with weak infill exposed to the first group of earthquakes. For the action of this group of earthquakes deviations also occur in the frames R5 and R6 with a strong infill, but are smaller compared to the frames with weak infill.
- For the action of the second group of earthquakes, the greatest deviations occur in the frames R5 and R6 with weak infill. This is due to the fact that the period of dominant mode shape in these frames, and especially after the occurrence of damage, is shifted from the dominant frequency range of this group of earthquakes, while the period of the second mode shape enters in the area of maximum spectral amplifications.
- For the action of the third group of earthquakes, in a large number of cases there is a relatively good overlap of the results obtained from the incremental dynamic and nonlinear static analysis, which is mainly due to the frequency bandwidth of the records from this group of earthquakes. Exceptions to this are noted in the highest frame R6 with weak infill.

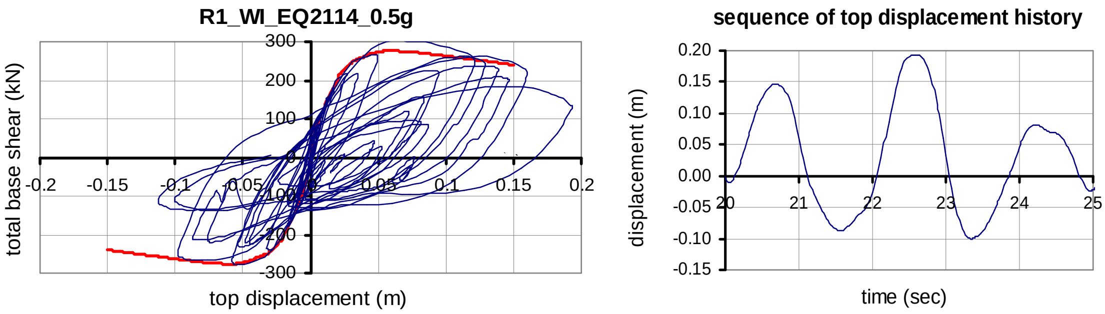

Fig. 23. Strength and stiffness degradation due to previous achieved large displacement in opposite direction

7 CONCLUSIONS

Presence of the masonry infill, its quality and irregular distribution in the height have a significant influence on the fundamental structural characteristics – stiffness, strength and ductility. Masonry infill significantly affects the level of the lateral force that causes the appearance of the first plastic hinge, as well as the bearing capacity of the frames. The frames with a masonry infill and open first floor have greater bearing capacity compared to the bare frames due to the overstrength of the columns, as well as modification in the lateral loads bearing mechanism. With increasing the number of storeys, the ratio between the capacity of the bare frames and the infilled frames with open first floor increases. Unlike the increased capacity, the infilled frames show significantly smaller yield displacements and less ability to develop plastic deformations compared to the bare frames

There is a permanent degradation of the stiffness at the reinforced concrete frames with and without masonry infill with the development of cracks in the reinforced concrete elements and masonry infill, as well as appearance of yielding in the reinforced concrete elements. Degree of degradation in frames with and without infill is approximately equal until top displacement ranging from 0.1% to 0.15% of the total height. With increasing a displacement, the frames with the infill show faster degree of stiffness degradation compared to the bare frames. This tendency is more pronounced at frames with stronger infill compared to those with weaker infill. The number of storeys has insignificant effect on the degree of degradation of the stiffness of the analyzed frames.

The selection and characteristics of ground motion records are one of the most influential factors that directly affect the quality of the obtained results from nonlinear dynamic analysis. In the case of low rise buildings (n=2 and 3 storeys), the quality of infill lack remarkable effect on the structural behaviour. Compared with the behaviour of bare frame, the presence of infill is usually unfavourable for all levels of PGA, leading to the formation of soft storey mechanism. In the case of 5 and 7 storey buildings, the presence of infill reduces the seismic demand up to the PGA of 0.3g. Usually, strong infill corresponds to small inter-storey drift demand, at low level of seismic hazard, compared with the weak infill. Soft storey mechanism has not been observed at high rise buildings. At this type of buildings distribution of damage depend on the mechanical characteristics of infill as well as the frequency content of input ground motion.

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SUMMARY

NONLINEAR STATIC VS. INCREMENTAL DYNAMIC ANALYSIS OF INFILLED FRAMES WITH OPEN FIRST FLOOR

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Ljupco LAZAROV

Masonry infill, as a part of building structures, is characterised with significant in-plane strength and stiffness and it can greatly alter the response of structures exposed to seismic loads. Irregular distribution of infill in plane and along building height can lead to series of unfavourable effects (torsion effects, dangerous collapse mechanisms, soft or weak storey, variations in the vibration period, etc.).

In order to investigate the influence of irregular distribution of masonry infill to the seismic performance of code designed reinforced concrete frames, an extensive nonlinear static and dynamic analysis was performed. Six reinforced concrete frames with different number of storeys, designed as bare frames were analysed. In the phase of assessment, all structures were upgraded with the masonry infill panels in all storeys except the first one. Masonry infill was defined with two different strength and stiffness characteristics. The obtained results show significant influence of masonry infill on the main structural characteristics: strength, stiffness and ductility, as well as on the seismic performance of analysed frames.

Key words: nonlinear static, incremental dynamic; masonry infill; open ground floor

REZIME

NELINEARNA STATIČKA NASUPROT INKREMENTALNE DINAMIČKE ANALIZE OKVIRA SA ISPUNOM SA OTVORENOM PRVOM ETAŽOM

Koce TODOROV
Ljupco LAZAROV

Zidanu ispunu, kao deo građevinske konstrukcije, karakteriše značajna čvrstoća i krutost u ravni, i ona može da u velikoj meri izmeni odgovor konstrukcije izložene seizmičkom opterećenju. Nepravilna raspodela ispune u ravni i po visini građevine može da dovede do niza neželjenih efekata (efekat torzije, opasni mehanizmi loma, efekat "mekog sprata", varijacije perioda vibriranja, itd.).

Da bi se istražio uticaj nepravilne raspodele zidane ispune na seizmičke performanse projektovanih armirano-betonskih okvira prema odgovarajućim propisima, obavljene su obimne istraživanja koristeći nelinearnu statičku i dinamičku analizu. Analizirano je šest armiranobetonskih okvira različite spratnosti, projektovanih bez ispune. U fazi procenjivanja, sve konstrukcije su nadograđene zidanom ispunom u svim spratovima osim prizemlja. Zidana ispuna je definisana pomoću dve različite čvrstoće i krutosti. Dobijeni rezultati pokazuju značajan uticaj zidane ispune na glavne konstruktivne karakteristike: čvrstoću, krutost i duktilnost, kao i na seizmičke performanse analiziranih okvira.

Ključne reči: nelinearna statička, inkrementalna dinamička; zidana ispuna, prizemlje bez ispune

EKSPERIMENTALNO ISPITIVANJE OJAČANIH LEPLJENO LAMELIRANIH DRVENIH NOSAČA SA REDUKOVANOM VISINOM PRESEKA

EXPERIMENTAL TESTING OF REINFORCED END-NOTCHED GLULAM BEAMS

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ORIGINALNI NAČNI RAD
ORIGINAL SCIENTIFIC PAPER
UDK:624.011.1.072.2
doi:10.5937/GRMK1804023T

1 UVOD

Nosači s redukovanom visinom preseka od monolitnog i lepljeno lameliranog drveta vrlo su zastupljeni u građevinskoj praksi. Mesto nagle promene visine poprečnog preseka nosača predstavlja slabu tačku u konstrukciji, te se preporučuje njeno izbegavanje. Međutim, postoje brojni razlozi za redukciju visine elementa. Najčešći razlog za to jeste ograničenje visine iznad oslonaca, ali postoje i drugi, poput poboljšanja bočne stabilnosti nosača, ostvarivanja veze elemenata itd. [1]. U svim ovim slučajevima, neophodan je adekvatan proračun nosača s redukovanom visinom preseka.

Kapacitet nosivosti nosača znatno je umanjen, kao rezultat koncentracije napona na mestu redukcije visine preseka. Redukcija visine preseka na zategnutoj strani elementa izaziva pojavu napona zatezanja upravno na vlakna koji, zajedno sa smičućim naponima, može da izazove pojavu pukotina, tipično od mesta nagle promene visine (slika 1). Pukotine su nepoželjna pojava sa estetske tačke gledišta, ali su takođe veoma opasne po konstrukciju, jer propagacija pukotine s porastom nivoa opterećenja može dovesti do loma. Ojačanje nosača s redukovanom visinom preseka je ekonomično rešenje kako bi se povećala nosivost.

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1 INTRODUCTION

Notched solid timber and glued laminated timber (glulam) beams are very common in structural engineering practice. Notches represent a weak spot in structure, and it is advisable to avoid them. However, there are various reasons for beam notching. The most common one being limitation in construction height at the supports, and others such as: stabilization of structural elements against lateral buckling, intersection of members and joint details etc. [1]. Adequate design of notched beams is necessary in these cases.

The load carrying capacity of timber beams is considerably reduced as a result of stress concentration around the notch. Notches made on the tension side induce tensile stresses perpendicular to grain which, accompanied by shear stresses, can cause longitudinal splitting typically starting at the notch corner (Figure 1). Cracks are unattractive appearance from aesthetic point of view, but are also very dangerous from structural perspective because crack propagation as the load level increases can lead to a failure of a beam. Reinforcement of such members is a cost-effective alternative for enhancing the load carrying capacity of structures in service.

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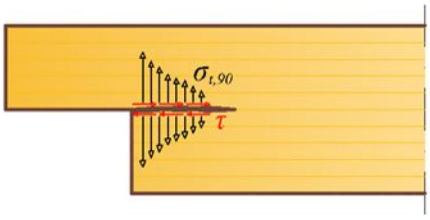
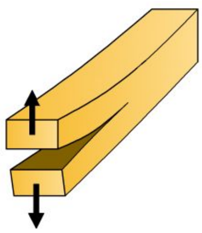


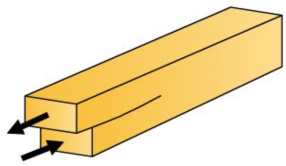
Slika 1. Koncentracija napona na mestu redukcije visine preseka
Figure 1. Stress concentration at the notched end of a beam

Naponsko stanje u okolini pukotine može se opisati različitim oblicima loma. Oblik loma 1 je otvaranje pukotine usled zatezanja upravano na ravan pukotine – slika 2 (a). Oblik loma 2 predstavlja smicanje u ravni gde površine pukotine klizaju jedna po drugoj – slika 2 (b). Kombinacija prethodna dva oblika loma predstavlja kombinovani lom, kao što je prikazano na slici 2 (c). Iako se pojavljuju oba napona (smicanje i zatezanje upravno na vlakna), otvaranje pukotina očigledan je mehanizam loma na mestu nagle promene visine preseka i izazvan je zatezanjem upravno na vlakna drveta. Zbog toga, oblik loma 1 jeste najčešći način loma kod nosača s redukovanom visinom preseka [2]. Međutim, obično postoji i komponenta smičućeg napona, koju treba imati u vidu.

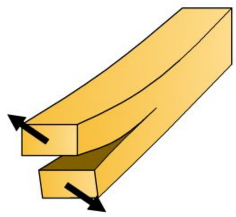
The stress state at a crack can be described by different fracture modes. Mode 1 is a tensile opening mode characterized by separating of crack surfaces in the direction that is perpendicular to them - Figure 2(a). Mode 2 represents in plane shear mode where crack surfaces slide one over the other - Figure 2(b). The combination of the previous two modes is a mixed mode fracture, as shown in Figure 2(c). Although both stresses (shear and tension perpendicular to grain) appear, crack opening is an apparent failure mechanism of a notch and it is caused by tension perpendicular to grain. Therefore, Mode 1 fracture is the most common failure mode of end-notched timber beams [2]. However, shear component usually exists and it should be also taken into consideration.



a) Oblik loma 1
a) *Mode 1 fracture*



b) Oblik loma 2
b) *Mode 2 fracture*



c) Kombinovani oblik loma
c) *Mixed mode fracture*

Slika 2. Oblici loma [1]
Figure 2. Fracture modes [1]

Mesta nagle promene visine nosača treba ojačati kako bi se izbegao krti lom i povećala nosivost. Postoje različite tehnike ojačanja, koje se uglavnom zasnivaju na sprečavanju pojave očekivanih pukotina. Parametri kao što su lakoća ugradnje, vidljivost ojačanja, jednostavnost proračuna i ekonomska isplativost važni su za odabir metode ojačanja. Različiti tipovi elemenata (šipke [3], zavrtnji [4,5], ploče i tkanine [6]) i materijala (materijali na bazi drveta, čelika, napredni kompozitni materijali, kao što su polimeri na bazi karbonskih i staklenih vlakana [7,8]) uspešno se koriste kao ojačanje drvenih nosača s redukovanom visinom preseka.

U proteklim decenijama, mnogi istraživači bavili su se drvenim nosačima s redukovanom visinom preseka i očigledno je da ojačanje i sanacija ovakvih elemenata predstavljaju veoma važnu temu u oblasti drvenih konstrukcija. U svojoj doktorskoj disertaciji, Jockwer [1] je dao detaljnu analizu različitih proračunskih pristupa neojačanih, ali i ojačanih greda s redukovanom visinom preseka. Franke, Franke i Harte [9] bavili su se metodama povećanja nosivosti drvenih greda,

Notched ends of beams should be reinforced so as to avoid brittle failure and increase load carrying capacity of the beams. There are various types of strengthening techniques which are mainly based on preventing the expected cracks. Parameters such as ease of installation, invisibility of reinforcement, simplicity of design approach and cost are all important for determining the strengthening method. Different types of elements (rods [3], screws [4,5], plates and sheets [6]) and materials (wood-based materials, steel, advanced composite materials like carbon or glass fibre based polymers [7,8]) have been successfully used as reinforcement of notched timber beams.

In the past decades, many researchers have dealt with notched timber beams and it is obvious that notched beam strengthening and repair represents very important topic in the field of timber structural design. In the PhD thesis Jockwer [1] gave a thorough analysis of different design approaches of both unreinforced and reinforced notched beams. Franke, Franke and Harte [9]dealt with methods for repair of structural performance uključujući i one s redukovanom visinom preseka. U svom radu, Oudjene i grupa autora [10] predstavljaju numerički pristup za modeliranje neojačanih i ojačanih greda s redukovanom visinom preseka. Dietsch [11] je govorio o neophodnosti novih proračunskih metoda za ojačanje drvenih greda i o njihovoj implementaciji u okviru Evrokoda 5 [12], naglašavajući važnost adekvatnog analitičkog proračuna.

U ovom radu prikazani su eksperimentalni rezultati ispitivanja nosača s redukovanom visinom preseka iznad oslonaca koji su ojačani zavrtnjima. Zavrtnji su efikasno rešenje za ojačanje iz aspekta cene, lakoće i brzine ugradnje [13], zbog čega su izabrani u ovom istraživanju. Pet neojačanih i deset ojačanih nosača s redukovanom visinom preseka iznad oslonaca ispitano je na savijanje do loma, uz razmatranje dve različite šeme ojačanja. Rezultati u pogledu ponašanja opterećenje–deformacije, oblika loma, granične nosivosti i krutosti upoređeni su za ispitane serije. Izvedeni su zaključci o efikasnosti zavrtanja kao ojačanja.

2 EKSPERIMENTALNO ISPITAVANJE

Eksperimentalno istraživanje sprovedeno je u Laboratoriji za ispitivanje konstrukcija, na Građevinskom fakultetu, Univerziteta u Beogradu. Pet neojačanih (serija U) i deset ojačanih (Serija R) nosača s redukovanom visinom preseka iznad oslonaca ispitani su na savijanje do loma. Neojačani nosači korišćeni su kao kontrolna serija. Ojačanje je izvedeno zavrtnjima. Pet ojačanih nosača imalo je zavrtnje postavljene upravno u odnosu na podužnu osu nosača (Serija R-s90), a pet je imalo zavrtnje postavljene pod uglom od 45° u odnosu na podužnu osu nosača (Serija R-s45).

Lepljeno lamelirani nosači su klase čvrstoće GL22h [15], a izrađeni su od drvene građe (smreke) klasifikovane kao klasa čvrstoće C22 prema EN 338 [14]. Pre eksperimentalnog ispitivanja, nosači su čuvani na temperaturi T = 20 ± 2 ºC i relativnoj vlažnosti od RH = 65 ± 5%. Posle ispitivanja, sadržaj vlage meren je na različitim mestima u okviru svakog nosača pomoću digitalnog vlagomera. Sadržaj vlage u testiranim nosačima varirao je od 11,0% do 11,9%.

Ukupna dužina nosača iznosila je 4.000 mm, dok su dimenzije poprečnog preseka bile 100 x 220 mm. Svaki nosač sastavljen je od sedam lamela debljine 32 mm. Na zasečenim krajevima, visina nosača smanjena je na 110 mm (za pola), a dužina zaseka bila je 250 mm. Kao ojačanje u ovom istraživanju, korišćeni su tradicionalni zavrtnji za drvo prečnika 10 mm i dužina 200 mm za Seriju R-s90 i 250 mm za Seriju R-s45 (slika 3). Dužina zavrtnja u navoju iznosila je 125 mm i 160 mm, respektivno. Prema proizvođaču, upotrebljeni zavrtnji su klase čvrstoće 5.6. Dva zavrtnja u jednom redu postavljena su blizu oba kraja nosača s redukovanom visinom preseka kod oslonaca. Uslovi za minimalna međusobna i ivična rastojanja zavrtanja su zadovoljeni, pri čemu je ojačanje postavljeno što bliže mestima nagle promene visine preseka.

of timber beams, including the ones with notches. In paper Oudjene et al. [10] presented a numerical approach for modelling both unreinforced and reinforced notched beams. Dietsch [11]talked about the necessity of new design approaches of strengthened timber beams, including strengthening of notches, and implementation of these in a new section of Eurocode 5 [12], emphasizing the importance of adequate analytical design.

This paper presents experimental results of end-notched glulam beams that were reinforced with screws. Screws are economic and time-efficient solution for reinforcement and they can be easily applied [13], which is the reason why they were chosen in this study. Five unreinforced and ten reinforced end-notched glulam beams were tested in bending to the point of failure, with two different reinforcement schemes considered. The results in terms of load-deformation relationship, failure mode, ultimate load carrying capacity and stiffness were compared between tested beam series. The conclusions on effectiveness of screws as a reinforcement method were made.

2 EXPERIMENTAL TESTING

The experimental research was conducted at the Laboratory of Structures, Faculty of Civil Engineering, University of Belgrade. Five unreinforced (Series U) and ten reinforced (Series R) notched glulam beams were tested in bending to the point of failure. Unreinforced beams were used as a control series. Reinforcing was performed with screws. Five reinforced beams had screws installed perpendicular to beam axis (Series R-s90) and five had screws positioned at an angle of 45° to beam axis(Series R-s45).

The glulam beams were made from spruce timber classified in the strength class C22 according to EN 338 [14], making the beams class GL22h [15]. Before the tests were performed, the beams were conditioned at a temperature of T = 20 ± 2 ºC and a relative humidity of RH = 65 ± 5%. After testing, moisture content was measured in each beam using a digital hygrometer at different locations. The moisture content in tested beams varied between 11.0% and 11.9%.

The overall length of the beams was 4000 mm and the cross section was 100 x 220 mm. Each beam was composed of seven 32 mm thick laminations. At the notched ends, the height of the beams was reduced to 110 mm (by half) and the length of notches was 250 mm. The reinforcement selected in this study was traditional wood screws with a diameter 10 mm and length of 200 mm for Series R-s90 and 250 mm for Series R-s45(Figure 3). Threaded part of screws was 125 mm and 160 mm, respectively. According to the manufacturer the steel grade of screws was 5.6. Two screws in one row were positioned near the both notched ends of the beams. The requirements for minimum screw edge distances and spacing were satisfied while keeping the reinforcement as close as possible to the notch corners.

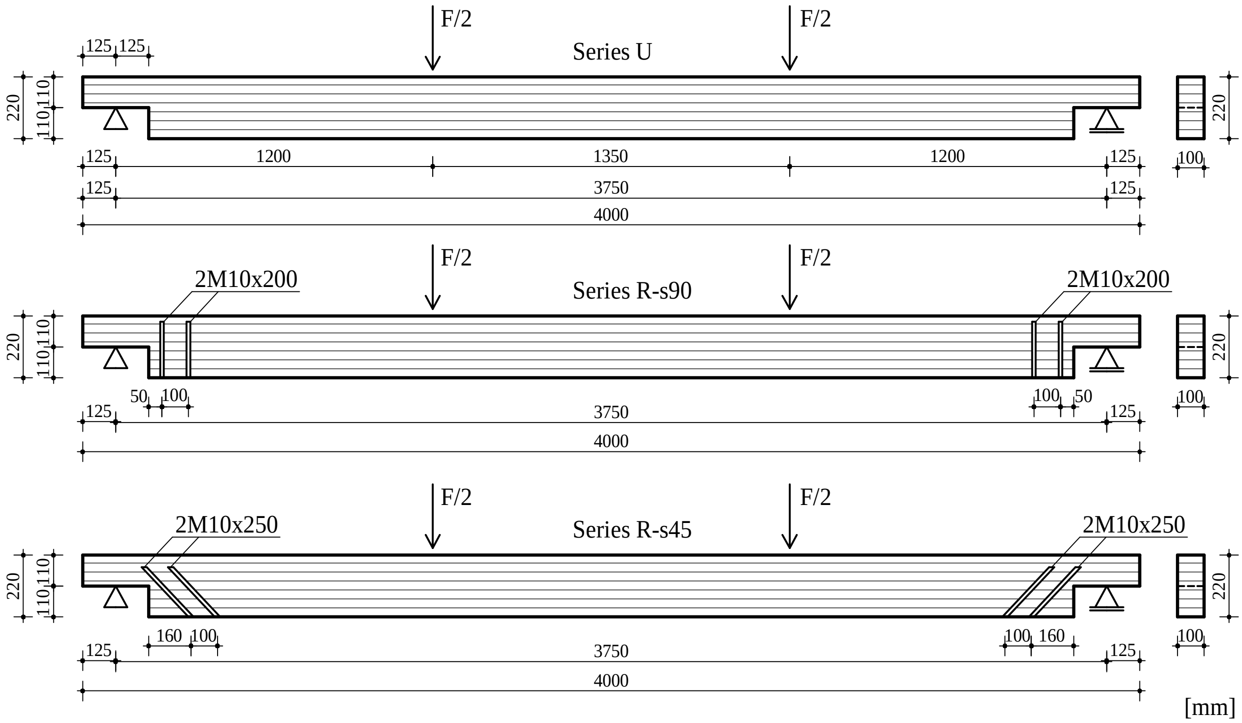
Slika 3. Zavrtnji za ojačanje
Figure 3. Screws used as reinforcement

Svi nosači izloženi su savijanju, u skladu sa EN 408 [16]. Proste grede raspona 3.750 mm ispitane su do loma pod koncentrisanim opterećenjem u dvema tačkama. Rastojanje između tačaka nanošenja opterećenja bilo je 1.350 mm, dok je rastojanje od sile do oslonca iznosilo 1.200 mm. Nosači su bili oslonjeni na valjkasta ležišta, koja su takođe postavljena na mestima unošenja opterećenja. Efekti lokalnih oštećenja na osloncima i na mestima unošenja koncentrisanih sila su minimizirani postavljanjem čeličnih pločica.

Šematski prikaz konfiguracije ispitivanja na savijanja za Seriju U, Seriju R-s90 i Seriju R-s45 prikazan je na slici 4, dok je na slici 5 prikazana postavka eksperimentalnog ispitivanja.

All beams were subjected to bending test in accordance with EN 408 [16]. The beams were tested to failure under monotonic load in four-point bending configuration over a simply supported span of 3750 mm. The distance between two loading points was 1350 mm, while the distance from the loading points to the supports was 1200 mm. The specimens were supported on roller bearings at the ends. Roller bearings were also used at the load application points. The effects of local indentation at load application and support positions were minimized by placing steel plates.

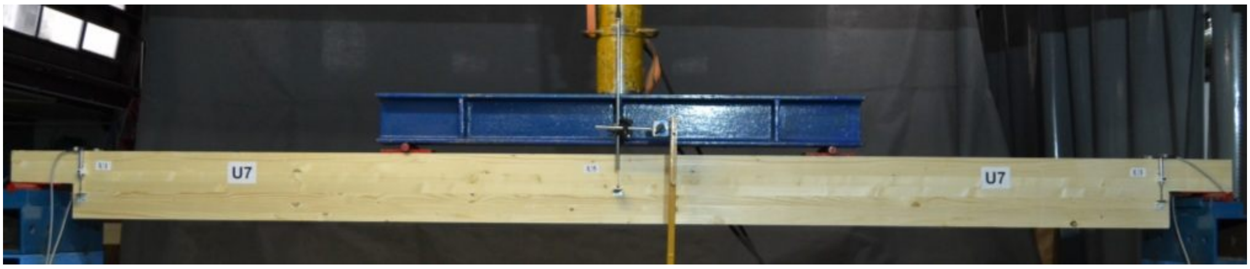
A schematic illustration of the bending test configuration for Series U, Series R-s90 and Series R-s45 is shown in Figure 4, while Figure 5 shows experimental test set-up.



Slika 4. Geometrija i opterećenje greda
Figure 4. Geometry and loading of the beams

Kod nosača predviđenih za ojačanje, posebna pažnja posvećena je ugradnji zavrtanja. Priprema uzoraka Serije R prikazana je na slici 6. Rupe za zavrtnje su vrlo pažljivo izbušene do prečnika 8 mm, s približnom dužinom bušenja 200 mm i 250 mm. Zavrtnji su postavljeni pomoću moment ključa.

When considering the specimens that were going to be reinforced special attention was put on inserting the screws. The preparation of Series R specimens is shown in Figure 6. The holes for screws were pre-drilled very carefully to a diameter of 8 mm, with approximately drilling length of 200 mm and 250 mm. The screws were inserted using a moment wrench.



Slika 5. Postavka eksperimenta
Figure 5. Experimental test set-up

Slika 6. Priprema ojačanih uzoraka
Figure 6. Preparation of reinforced specimens

Opterećenje je aplicirano do loma pomoću hidrauličke prese i mereno doznom. Opterećenje je transformisano s jedne tačke na dve tačke, pomoću čeličnog profila. Monotono statičko opterećenje nanošeno je kontrolisanom brzinom od 4 kN u minuti, kako bi se izazvao lom neojačanih nosača za približno pet minuta. Ojačani nosači testirani su sa istom brzinom opterećenja kako bi se obezbedila uporedivost rezultata ispitivanja. Lom ojačanih nosača postignut je za oko deset minuta.

Elektronski ugibomeri (LVDT) korišćeni su za merenje ugiba u sredini raspona nosača, kao i za merenje otvaranja pukotina na mestima nagle promene visine preseka. Podaci o ugibima sa LVDT-ova i odgovarajući podaci o opterećenju sa dozne zabeleženi su pomoću akvizacionog sistema. Sopstvena težina hidrauličke prese i čeličnog profila uzeti su u obzir. Ovo dodatno opterećenje iznosilo je 1,3 kN.

3 REZULTATI I DISKUSIJA

3.1 Ponašanje opterećenje–ugib i oblici loma

Ponašanje opterećenje–ugib do loma ispitanih nosača prikazano je na slikama 7 i 8.

Efekti redukcije visine preseka na mehaničke karakteristike lepljeno lameliranih drvenih nosača su značajni. Svi ispitani neojačani nosači (Serija U) pokazali su linearno ponašanje do loma. Lom je nastupio na mestima nagle promene visine usled prekoračenja napona na zatezanje upravno na vlakna, kao što je prikazano na slici 9. Otvaranje pukotina (oblik loma 1) na mestu nagle promene visine očigledan je mehanizam loma neojačanih nosača. Međutim, smicanje u ravni pukotine (oblik loma 2) takođe je imalo značajan uticaj. Zbog krte prirode ponašanja drveta pri zatezanju i

The load was applied monotonically using a hydraulic jack until the failure occurred and recorded with a compression load cell. The load was transformed from one point to two points with a steel beam. Monotonic static load was applied in a stroke-controlled rate of 4 kN per minute, so as to cause the failure of the unreinforced beams in approximately 5 minutes. The reinforced beams were tested with the same load rate in order to ensure a fair comparison of test results. The failure of the reinforced beams was achieved in about 10 minutes.

Linear variable differential transducers (LVDTs) were used for the measurement of mid-span deflection of the beams as well as the measurement of crack opening in notch details. The deformation data from LVDTs and corresponding load data from a loading cell were recorded by a computerized data acquisition system. Self-weight of hydraulic jack and steel beam were added to the recorded load. This additional load was 1.3 kN.

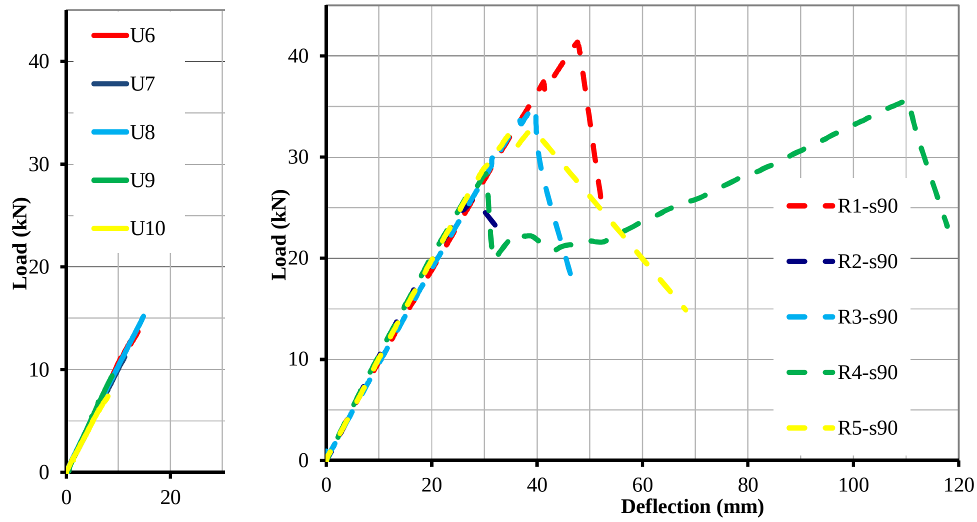
3 RESULTS AND DISCUSSION

3.1 Load-deflection behaviour and failure modes

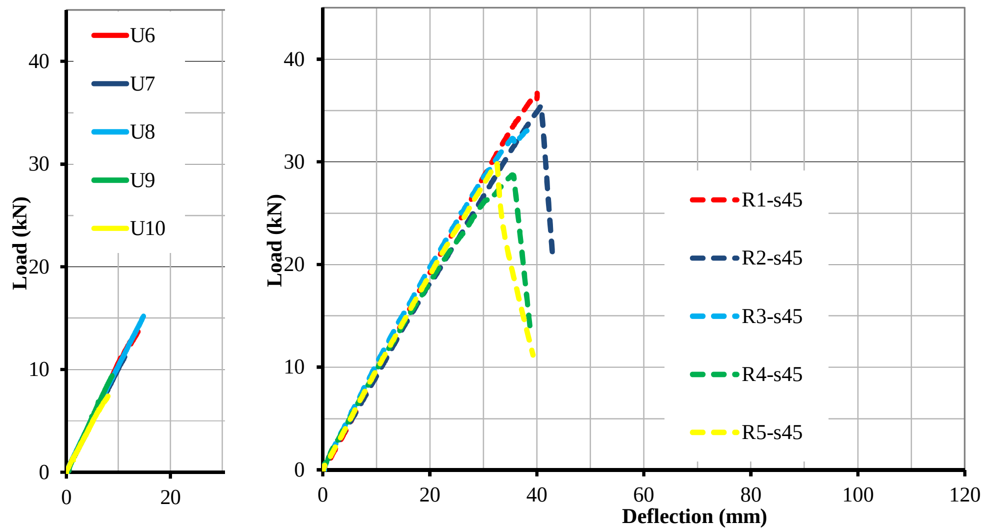
The load-deflection behaviour to the failure of tested beams is shown in Figures 7 and 8.

The effects of the notches on the mechanical properties of glulam beams are significant. All tested beams with unreinforced notches (Series U) exhibited linear load-deflection behaviour until the point of failure. Beams failed at the notch details due to excessive tensile stress perpendicular to grain, as it is shown in Figure 9. Crack opening (Mode 1 fracture) at the notch corner was the obvious failure mechanism of unreinforced notched beams. However, crack shearing (Mode 2 fracture) also had a considerable influence. Due to brittle nature of wood behaviour in tension and in smicanju, lom nosača Serije U bio je iznenadan i bez znakova upozorenja. Pre dostizanja graničnog opterećenja, primećeno je vrlo malo otvaranje pukotina. Nakon razvijanja inicijalne pukotine na mestu redukcije visine preseka, došlo je do nekontrolisanog rasta pukotina. To je uzrokovalo razdvajanje preseka na dva dela (gornji i donji). Putanja pukotine bila je uglavnom pravolinijska, a površine pukotine bile su ravne.

shear, failure of Series U beams was sudden without warning signs. Prior to ultimate load, only very little crack opening was observed. After the development of initial crack at the notch corner, uncontrollable crack growth occurred. This led to a separation of the cross-section in two parts (upper and lower). The crack path was generally clear and straight.



Slika 7. Krive opterećenje-ugib za grede Serije U i Serije R-s90
Figure 7. Load-deflection curves for Series U and Series R-s90 beams



Slika 8. Krive opterećenje-ugib za grede Serije U i Serije R-s45
Figure 8. Load-deflection curves for Series U and Series R-s45 beams

Slika 9. Tipični mehanizam loma greda Serije U
Figure 9. Typical failure mechanism of Series U beams

Ojačani nosači s redukovanom visinom preseka (Serija R) u suštini su imali linearno ponašanje do loma. Devet od deset nosača doživelo je krti lom. Jedan od nosača imao je pad nosivosti, ali je nastavio da nosi opterećenje do loma. Iako je granično opterećenje povećano, ojačanje nije bilo dovoljno da se promeni oblik loma iz kombinovanog usled zatezanja upravno na vlakna i smicanja u lom usled savijanja. Slika 10 prikazuje tipičan lom ojačanih nosača.

Reinforced notched beams (Series R) essentially experienced linear behaviour up to the point of failure. Nine out of ten beams failed in a brittle way. One of the beams had a drop in the load carrying capacity, but continued to carry the load until failure was reached. Although ultimate load was improved, the reinforcement was insufficient to change the failure mode from combined tensile perpendicular to grain and shear to bending failure. Figure 10 shows typical failure of reinforced beams.

Slika 10. Tipični mehanizam loma ojačanih greda – Serija R
Figure 10. Typical failure mechanism of reinforced beams – Series R

Uprkos intervenciji ojačanja, inicijacija pukotina na mestu nagle promene visine preseka se ne može sprečiti. Može se videti da je iniciranje pukotina započelo na relativno niskim vrednostima opterećenja. Ovo se može objasniti veoma malim kapacitetom deformacije drveta pri zatezanju upravno na vlakna. Prekomerno otvaranje pukotina ograničeno je ojačanjem. S daljim opterećivanjem, stabilan rast pukotine propraćen je smicanjem u ravni pukotine. Pri lomu, došlo je do nestabilnog rasta pukotina i znatnog porasta smicanja u ravni pukotine. Može se pretpostaviti da je smicanje dominantan mehanizam loma. U najvećem broju slučajeva, lom je propraćen izvlačenjem zavrtanja.

Vertikalno postavljeni zavrtnji opterećeni su na kombinovano naprezanje paralelno i upravno na ravan smicanja. Na zavrtnjima su postojale jasne plastične deformacije koje ukazuju na to da je formiran plastični zglob u području loma, u slučaju nosača Serije R-s90. Motivacija za postevljanje zavrtanja pod uglom bila je da oni budu aksijalno opterećeni (na zatezanje), u pravcu u kome pokazuju najveću krutost. Zbog toga se očekivalo da nosači Serije R-s45 imaju mnogo veću nosivost, ali zbog nedovoljne dužine sidrenja zavrtanja, oni su doživeli lom ranije nego nosači Serije R-s90. Pošto tradicionalni zavrtnji zahtevaju prethodno izbušene rupe za ugradnju, bolji rezultati u slučaju ojačavanja i sanacije drvenih konstrukcija mogu biti postignuti pomoću samougrađujućih zavrtanja.

Despite the reinforcement intervention, initial cracking of the notch corner cannot be prevented. It can be seen that crack initiation started at relatively low loads. This can be explained by the very small deformation capacity of wood before the tensile strength perpendicular to grain is exceeded. Excessive crack opening was limited by the reinforcement. With further loading the stable crack growth was accompanied by sharing of the crack. At failure, unstable crack growth occurred and crack shearing increased considerably. It can be assumed that the shear failure was dominant failure mechanism. In most cases, failure was accompanied by withdrawal of the screws.

At the notch corner vertical reinforcement screws were subjected to combined loading parallel and perpendicular to the shear plane. There were clear plastic deformations in the reinforcement indicating that plastic hinge was formed in the fracture region in the case of these beams. The idea of inclined screws was to load the reinforcement axially (in tension), the direction in which they demonstrate the highest stiffness. Therefore, Series R-s45 beams were expected to have much higher load carrying capacity, but due to insufficient anchorage length of the screws, they failed even earlier than the beams from Series R-s90. Since conventional screws require pre-drilled holes for installation, better results could be achieved with self-tapping screws for reinforcing and strengthening procedures of timber structures.

3.2 Kapacitet nosivosti, deformabilnost i krutost

Rezultati eksperimentalnih ispitivanja u pogledu nosivosti, deformabilnosti i krutosti za tri serije nosača dati su u tabeli 1. Granično opterećenje uzeto je kao maksimalna sila koja je izazvala lom nosača. Srednji ugib uzet je kao vrednost koja odgovara graničnom opterećenju. Krutost na savijanje izračunata je iz linearnog dela krive opterećenje–ugib za svaki nosač, korišćenjem jednačine za ugib u sredini proste grede opterećene u dvema tačkama:

3.2 Load carrying capacity, deformability and stiffness

The results of experimental tests in terms of load carrying capacity, deformability and stiffness for the three series of beams are given in Table 1. The ultimate load was taken as a maximum force, which caused the failure of the beams. The mid-span deflection was taken as the value that corresponded to the ultimate load. The bending stiffness was calculated from liner part of the load-deflection curve of each beam, using the mid-span deflection equation for four-point bending:

$$EI = \frac{1}{48}\frac{\Delta F(3l^2 - 4c^2)c}{\Delta w} \qquad (1)$$

gde je
E – modul elastičnosti;
I – moment inercije;
$\frac{DF}{Dw}$ – nagib krive opterećenje-ugib između 10% i 40% graničnog opterećenja;
l – raspon grede;
c – rastojanje između oslonca i koncentrisane sile.

U tabeli 1 prikazana su poređenja graničnog opterećenja, ugiba u sredini i krutosti na savijanje za ojačane nosače (Serija R-s90 i Serija R-s45) i neojačane nosače (Serija U).

where:
E – modulus of elasticity,
I – moment of inertia,
$\frac{DF}{Dw}$ – slope of load-deflection curve between 10% and 40% of ultimate load,
l – beam span,
c – distance between support and load application point.

Comparisons in relation to ultimate load, mid-span deflection and bending stiffness for the reinforced beams (Series R-s90 and Series R-s45) and unreinforced beams (Series U) are also reported in Table 1.

Tabela 1. Eksperimentalni rezultati
Table 1. Experimental results

Nosač *Beam*	**Granično opterećenje** *Ultimate load* F_{ult} **(kN)**	**Ugib u sredini za granično opt.** *Mid-span deflection for ultimate load* **w (mm)**	**Krutost na savijanje** *Bending stiffness* **EI (kNmm^2 x 10^8)**
Serija U / *Series U*			
U6	15,0	13,8	9,03
U7	12,7	12,1	8,85
U8	16,7	15,1	9,55
U9	10,7	8,8	9,91
U10	8,7	8,0	8,60
Srednja vrednost / *Average*	12,8	11,5	9,19
SD	3,2	3,1	0,53
CV	25,2	26,6	5,8
Serija R-s90 / *Series R-s90*			
R1-s90	42,7	47,7	8,46
R2-s90	29,8	27,8	10,46
R3-s90	36,3	39,7	8,63
R4-s90	37,0	110,4	9,32
R5-s90	34,0	39,2	9,14
Srednja vrednost / *Average*	35,9	52,9	9,20
SD	4,7	32,9	0,79
CV	13,0	62,1	8,5
Poređenje sa Serijom U *Comparison with Series U* (%)	180,5	360	-

Serija R-s45 / *Series R-s45*			
R1-s45 R2-s45 R3-s45 R4-s45 R5-s45	38,0 36,7 34,6 30,0 31,1	40,1 40,8 38,9 59,1 32,6	9,01 8,29 9,19 8,54 8,83
Srednja vrednost / *Average*	34,1	42,3	8,77
SD	3,5	9,9	0,36
CV	10,1	23,5	4,1
Poređenje sa Serijom U *Comparison with Series U* (%)	166,4	267,8	-

Neojačani nosači s redukovanom visinom preseka iznad oslonca (Serija U) imali su prosečno granično opterećenje od 12,8 kN. Nosivost nosača znatno je smanjena zbog nagle promene visine preseka. Veliki koeficijent varijacije (25,2%) u slučaju graničnog opterećenja greda Serije U se može objasniti prirodnom varijabilnošću karkteristika drveta kao materijala. Uvođenje ojačanja na mestima nagle promene visine preseka nosača dovelo je do poboljšanja nosivosti. Ojačani nosači imaju prosečno granično opterećenje od 35,9 kN i 34,1 kN, za zavrtnje postavljene pod uglovima od 90° i 45°. Svi ojačani nosači pokazali su povećanje graničnog opterećenja u poređenju sa opterećenjem nosača bez ojačanja. Ovo povećanje iznosilo je 180,5% i 166,4%. Neojačani nosači s redukovanom visinom preseka potpuno su izgubili nosivost nakon iniciranja pukotine. S druge strane, ojačani nosači nastavili su da nose opterećenje nakon inicijalnog otvaranja pukotine. Međutim, nedovoljna dužina sidrenja nije dozvolila da ojačani nosači dožive lom usled savijanja, pošto je izvalačenje zavrtanja nastupilo pre dostizanja nosivosti koju bi imali nosači bez redukovane visine preseka kod oslonaca.

Ojačani nosači pretrpeli su veće deformacije pre loma u poređenju s neojačanim. Prosečno izmereni ugib u sredini pri graničnom opterećenju bio je 52,9 mm, 42,3 mm i 11,5 mm za nosače Serije R-s90, Serije R-s45 i Serije U, redom. Ojačani nosači pokazali su 3,6–4,6 puta veće ugibe u sredini pri graničnom opterećenju. Dakle, zavrtnji su doprineli poboljšanju deformabilnosti nosača.

Svi nosači imali su slične vrednosti krutosti na savijanje. Ovo je očekivano s obzirom na to da primenjeno ojačanje nije namenjeno poboljšanju krutosti. Serija R-s45 imala je malo nižu vrednost koja se može objasniti varijabilnošću svojstava, što je karakteristično za drvo kao materijal.

The unreinforced notched beams (Series U) had an average ultimate load of 12.8 kN. The load carrying capacity of the beams was considerably reduced due to presence of notches. High coefficient of variation (25.2%) in ultimate load for Series U beams can be explained by natural variability in timber properties. Introduction of reinforcement at the notched ends of the beams resulted in improvement in load carrying capacity. The reinforced beams obtained an average ultimate load of 35.9 kN and 34.1 kN, for screws positioned at the angles of 90° and 45°, respectively. All reinforced beams showed an increase in ultimate load when compared with the loads recorded for the beams without reinforcement. This increase was 180.5% and 166.4%. Unreinforced notched beams completely lost their load carrying capacities after the first crack developed. On the other hand, reinforced beams continued to carry the load after initial cracking. However, insufficient anchorage length did not allow for the reinforced beams to fail in bending, since the withdrawal of the screws occurred before the beams reached the load carrying capacity of beams without notches.

The reinforced beams underwent large deformations before the failure when compared with the unreinforced ones. Average measured mid-span deflection at ultimate load was 52.9 mm, 42.3 mm and 11.5 mm for beams of Series R-s90, Series R-s45 and Series U, respectively. At failure, the reinforced beams exhibited 3.6 – 4.6 times larger mid-span deflections. Hence, screws helped improve the deformability of the beams.

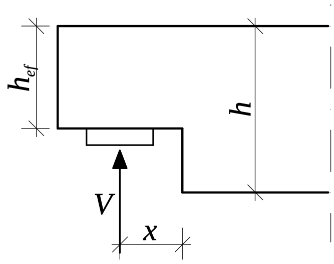
All the beams had similar bending stiffness values. This was expected since the applied reinforcement was not meant to improve the bending stiffness. Series R-s45 had a bit lower value, which can be explained by the variability in timber properties that generally exists when this material is in question.

4 NUMERIČKA ANALIZA

Proračun nosača s redukovanom visinom preseka dat je u Evrokodu 5 [12]. Za nosače s pravougaonim poprečnim presekom i orijentacijom vlakana u pravcu dužine nosača, smičući napon na mestu redukcije visine preseka računa se koristeći efektivnu (redukovanu) visinu nosača h_{ef} (slika 11).

4 NUMERICAL ANALYSIS

The design of notched timber beams is given in Eurocode 5 [12]. For beams with rectangular cross-sections and grain that runs parallel to the length of the member, the shear stresses at the notched support should be calculated using the effective (reduced) height h_{ef} of the beam (Figure 11).



Slika 11. Greda sa redukovanom visinom kod oslonca
Figure 11. End-notched beam

Neophodno je zadovoljiti sledeću nejednakost:

$$t = \frac{1,5V}{b_{ef} h_{ef}} \le k_v f_v \qquad (2)$$

gde je:

V – smičuća sila;

t – smičući napon;

f_n – čvrstoća na smicanje;

b_{ef} – efektivna širina nosača $b_{ef} = k_{cr} b$;

za pravogaoni poprečni presek $k_{cr} = 0,67$

h_{ef} – efektivna visina nosača;

k_n – faktor redukcije kojim se uzima u obzir koncentracija napona na mestu nagle promene visine preseka:

za redukciju visine na suprotnoj strani od oslonca: $k_n = 1$;

za redukciju visine na istoj strani kao oslonac:

$$k_v = \frac{k_n}{\sqrt{h}\left(\sqrt{a(1-a)} + 0,8\frac{x}{h}\sqrt{\frac{1}{a} - a^2}\right)} \le 1;$$

h – visina nosača;

x – rastojanje reakcije oslonca do mesta redukcije visine preseka;

$$a = \frac{h_{ef}}{h}$$

$$k_n = \begin{cases} 5 & \text{za monolitno drvo} \\ 6,5 & \text{za lepljeno lamelirano drvo} \end{cases}$$

Analitički proračun sproveden je za ispitane grede. Kako bi se uporedili eksperimentalni i analitički rezultati, uzimaju se karakteristične vrednosti čvrstoća. Karakteristična čvrstoća na smicanje za lepljeno lameliran drvo klase GL22h je

$$f_{n,k} = 3,5\frac{N}{mm^2}.$$

Faktor k_n usvojen je za lepljeno lamelirano drvo $k_n = 6,5$. Faktor k_n sračunat je za $a = 0,5$, $x = 125\ mm$, $h = 220\ mm$:

$$k_v = \frac{6,5}{\sqrt{220}\left(\sqrt{0,5(1-0,5)} + 0,8\frac{125}{220}\sqrt{\frac{1}{0,5} - 0,5^2}\right)} = 0,398$$

Sračunata vrednost graničnog opterećenja za neoja-

The following expression should be satisfied:

where:

V – shear force

t –shear stress

f_n –shear strength

b_{ef} – effective beam width $b_{ef} = k_{cr} b$

for rectangular cross-section $k_{cr} = 0.67$.

h_{ef} –effective beam height

k_n – reduction factor which takes into account stress concentration at the notch:

for notches at the opposite side to the support: $k_n = 1$

for notches on the same side as the support:

$$k_v = \frac{k_n}{\sqrt{h}\left(\sqrt{a(1-a)} + 0.8\frac{x}{h}\sqrt{\frac{1}{a} - a^2}\right)} \le 1$$

h – beam height

x – the distance from line of action of the support reaction to the corner of the notch

$$a = \frac{h_{ef}}{h}$$

$$k_n = \begin{cases} 5 & \text{for solid timber} \\ 6.5 & \text{for glued laminated timber} \end{cases}$$

Analytical calculations are performed for tested beams. In order to compare the experimental and analytical results, characteristic values for strengths were adopted. Characteristic value of shear strength for glulam class GL22h is

$$f_{n,k} = 3.5\frac{N}{mm^2}.$$

Factor k_n is taken for glued laminated timber $k_n = 6.5$. Factor k_n is calculated for $a = 0.5$, $x = 125\ mm$, $h = 220\ mm$:

$$k_v = \frac{6.5}{\sqrt{220}\left(\sqrt{0.5(1-0.5)} + 0.8\frac{125}{220}\sqrt{\frac{1}{0.5} - 0.5^2}\right)} = 0.398$$

The calculated value of ultimate load for unreinforced čane nosače prema Evrokodu 5 iz Jednačine 2 jeste:

$$F_{ult} = \frac{2}{1,5} k_v \cdot f_{v,k} \cdot b_{ef} \cdot h_{ef} = 13,7\,kN$$

Kada se uporedi s prosečnom eksperimentalnom vrednošću za Seriju U, koja je iznosila 12,8 kN, može se videti da predloženi metod proračuna premašuje granično opterećenje za 7,0%. Može se zaključiti da ovaj proračunski pristup zanemaruje pojavu smicanja u ravni pukotine, koje se javlja na mestu redukcije visine preseka, uzimajući u obzir samo vertikalnu komponentu koja izaziva zatezanje upravno na vlakna drveta.

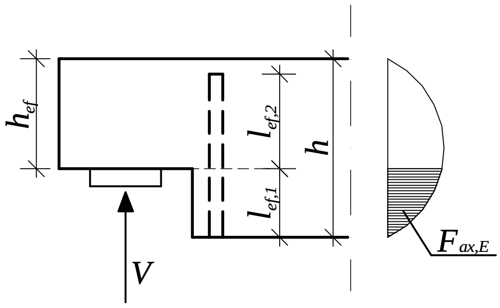
Proračun nosača s redukovanom visinom preseka kod oslonca, dat u Evrokodu 5, ne predviđa proračun mogućih ojačanja. Kao dodatak Evrokodu 5, Nemački nacionalni aneks [17] predlaže metod proračuna ojačanja (slika 12). Ovaj pristup baziran je na ideji da se napon koji se javlja na mestu nagle promene visine nosača raspodeljuje između ojačanja i drveta putem faktora k_a.

notches according to Eurocode 5 from Eq. 2 is:

$$F_{ult} = \frac{2}{1.5} k_v \cdot f_{v,k} \cdot b_{ef} \cdot h_{ef} = 13.7\,kN$$

When compared with the average experimental value for Series U, which was 12.8 kN, it can be seen that the suggested design method overestimated the ultimate load by 7.0%. It can be noted that this design approach ignores the in-plane shear, which appears at the notch, taking into account only the vertical component that causes tensile stress perpendicular to grain of timber.

The design of end-notched shear stress given in the Eurocode 5 does not provide the calculation of possible reinforcement. In addition to Eurocode 5, the German National Annex [17] proposes reinforcement design method at the notch (Figure 12). This design approach is based on the idea that the stress which apperas around the notch is divided between reinforcement and timber using factor k_a.



Slika 12. Vertikalno ojačanje grede sa redukovanom visinom kod oslonca [18]
Figure 12. Vertical reinforcement of end-notched beam [18]

Maksimalna smičuća sila kod oslonca nosača s redukovanom visinom:

Maximum shear force at the support of end-notched beam:

$$\max V = \frac{2}{3} b_{ef} \cdot h_{ef} \cdot f_v] \tag{3}$$

U ovom radu zavrtnji su usvojeni kao ojačanje i deo sile koji oni preuzimaju može se sračunati prema izrazu:

In this paper screws are selected as reinforcement and and part of the load carried by screws can be calculated as follows:

$$F_{ax,E} = k_a V \tag{4}$$

uz uslov:

$F_{ax,E} \le n_{ef} F_{ax,R}$

gde je:

V – smičuća sila;

with condition:

$F_{ax,E} \le n_{ef} F_{ax,R}$

where:

V – shear force

$$k_a = 1,3\left[3(1-a)^2 - 2(1-a)^3\right] \tag{5}$$

$$a = \frac{h_{ef}}{h} \tag{6}$$

$F_{ax,E}$ – aksijalna sila u zavrtnjima;

n – broj zavrtanja;

$F_{ax,R}$ – kapacitet zavrtnja na čupanje.

Za ojačane nosače, karakteristična vrednost kapaciteta zavrtnja na čupanje usvaja se prema [18]:

$F_{ax,E}$ – axial force in screws

n – number of screws

$F_{ax,R}$ – screw withdrawal capacity

For reinforced beams the screw withdrawal capacity was adopted according to [18]:

$$F_{ax,R} = f_{ax,90} \cdot d \cdot l_{ef} \qquad (7)$$

gde je:

$f_{ax,90}$ – čvrstoća vertikalnog zavrtnja na čupanje upravno na vlakana;

d – prečnik zavrtnja;

l_{ef} – dužina sidrenja dela zavrtnja u navoju.

Čvrstoća vertikalnog zavrtnja na čupanje upravno na vlakana je

$$f_{ax,90} = 9{,}8 \frac{N}{mm^2}$$

prema [18], usvojeno na osnovu vrednosti datih u literaturi. Vrednost kapaciteta vertikalnog zavrtnja na čupanje za prečnik zavrtnja $d = 10\ mm$ i dužinu sidrenja $l_{ef} = 90\ mm$ jeste:

$$F_{ax,R} = f_{ax,90} \cdot d \cdot l_{ef} = 8{,}82\,kN$$

Ako se zanemari koeficijent $k_a\ (k_a = 1)$, s pretpostavkom da zavrtnji nose svo opterećenje, granično opterećenje za nosače ojačane vertikalnim zavrtnjima prema [18] jeste:

$$F_{ult} = 2V = 2nF_{ax,R} = 35{,}3\,kN$$

Ova pretpostavka je validna, s obzirom na to da je testiranje vršeno do loma, a ne za eksploataciono opterećenje. Drvo gubi nosivost pri iniciranju pukotine na mestu nagle promene visine preseka, a zavrtnji nastavljaju da nose opeterećenje do loma. Razlika u odnosu na eksperimentalnu vrednost, koja je iznosila 35,9 kN, jeste 1,7%.

Za zavrtnje pod uglom od 45° u odnosu na podužnu osu nosača, čvrstoća zavrtnja na čupanje uzeta je kao

$$F_{ax,45} = 0{,}86 \cdot f_{ax,90}$$

prema [18]. Vrednost kapaciteta kosog zavrtnja na čupanje za prečnik zavrtnja $d = 10\ mm$ i dužinu sidrenja $l_{ef} = 94\ mm$ jeste:

$$F_{ax,R} = 0{,}86 \cdot f_{ax,90} \cdot d \cdot l_{ef} = 7{,}92\,kN$$

Ponovo, ako se zanemari koeficijent k_a, s pretpostavkom da zavrtnji nose svo opterećenje, granično opterećenje za nosače ojačane kosim zavrtnjima jeste:

$$F_{ult} = 2V = 2nF_{ax,R} = 31{,}7\,kN$$

Razlika u odnosu na eksperimentalnu vrednost, koja je iznosila 34,1 kN, jeste 7,0%.

Uočava se dobro slaganje analitičkih i eksperimentalnih rezultata za nosivost ojačanih nosača. Kako je proračun baziran na konceptu da ojačanje prihvata samo vertikalnu komponentu sile koja izaziva zatezanje upravno na vlakna drveta, analitičke vrednosti niže su od eksperimentalnih.

where:

$f_{ax,90}$ – withdrawal strength perpendicular to the grain for vertical screws

d – screw diameter

l_{ef} – anchorage length of the threaded part

The withdrawal strength perpendicular to the grain for vertical screws is taken as

$$f_{ax,90} = 9.8 \frac{N}{mm^2}$$

according to [18], which is adopted based on various literature sources. The value for vertical screw withdrawal capacity for screw diameter $d = 10\ mm$ and anchorage length $l_{ef} = 90\ mm$ is:

$$F_{ax,R} = f_{ax,90} \cdot d \cdot l_{ef} = 8.82\,kN$$

If factor k_a is ignored $(k_a = 1)$, with the assumption that the screws carry the entire load the ultimate load for vertically reinforced notches according to [18] is:

$$F_{ult} = 2V = 2nF_{ax,R} = 35.3\,kN$$

The made assumption is valid, because the tests were carried out to the point of failure and not for the service loads. Timber part of the cross-section in this case loses its load-carrying capacity with crack initiation at the notch, and screws continue to carry the load to failure. The difference from experimentally obtained average value, which was 35.9kN, is 1.7%.

For screws positioned at an angle of 45° to beam axis, the withdrawal strength is taken as

$$F_{ax,45} = 0.86 \cdot f_{ax,90}$$

according to [18]. The value for inclined screw withdrawal capacity for screw diameter $d = 10\ mm$ and anchorage length $l_{ef} = 94\ mm$ is:

$$F_{ax,R} = 0.86 \cdot f_{ax,90} \cdot d \cdot l_{ef} = 7.92\,kN$$

Again, if factor k_a is ignored, with the assumption that the screws carry the entire load, the ultimate load for notches reinforced with inclined screws is then:

$$F_{ult} = 2V = 2nF_{ax,R} = 31.7\,kN$$

The difference from experimentally obtained average value, which was 34.1 kN, is 7.0%.

There is a good agreement between analytical and experimental results for reinforced beams load-carrying capacity. Since the design method is based on the concept that reinforcement carries only the vertical force component that causes tensile stress perpendicular to grain of timber, analytical values underestimate the ultimate load.

5 ZAKLJUČCI

Eksperimentalno ispitivanje sprovedeno u ovom radu uključivalo je ispitivanje na savijanje 15 lepljeno lameliranih drvenih nosača s redukovanom visinom preseka iznad oslonaca do tačke loma (pet neojačanih i deset ojačanih). Zavrtnji su odabrani kao ojačanje. Uticaji ojačanja procenjeni su u smislu ponašanja opterećenje–ugib, oblika loma, nosivosti i krutosti ojačanih nosača, koji su upoređeni s neojačanim nosačima. Takođe, sprovedeno je poređenje sa analitičkim rezultatima. Izvedeni su sledeći zaključci:

– Lom lepljeno lameliranih drvenih nosača s redukovanom visinom preseka iznad oslonaca opterećenih na savijanje nastaje usled koncentracije napona na mestu redukcije visine. Nosivost ovih nosača definisana je prekomernim otvaranjem pukotina. Krt lom tipičan je za neojačane nosače s redukovanom visinom preseka iznad oslonaca.

– Inicijalno otvaranje pukotine ne može se sprečiti ojačanjem. Međutim, pomoću ojačanja pojava loma znatno se odlaže.

– Ojačanje sprečava prekomerno otvaranje pukotina na mestu nagle promene visine preseka. Ipak, konačni lom ojačanih nosača usled prevelikog rasta pukotina uzrokovan je smicanjem.

– Ojačanjem zavrtnjima povećava se nosivost i deformabilnost lepljeno lameliranih drvenih nosača s redukovanom visinom preseka iznad oslonaca.

– Na mestu nagle promene visine preseka, zavrtnji upravni na podužnu osu nosača izloženi su kombinovanom paralelnom i upravnom opterećenju u odnosu na ravan pukotine. Prema tome, ojačanje s visokom čvrstoćom i krutošću u oba pravca neophodno je radi postizanja najboljeg efekta ojačanja.

– Zavrtnji postavljeni pod uglom od 45° u odnosu na podužnu osu nosača dominantno su opterećeni u aksijalnom pravcu. Posebnu pažnju treba posvetiti dužini sidrenja ovih zavrtanja.

– Kako bi se poboljšalo sidrenje, generalno se preporučuje primena samougrađujućih zavrtanja umesto klasičnih koji zahtevaju prethodno bušenje rupa.

– Predložena metoda proračuna u Evrokodu 5 precenila je granično opterećenje neojačanih nosača za 7,0%.

– Kada je reč o ojačanim nosačima, ako se usvoji pretpostavka da svo opterećenje primaju zavrtnji, postoji dobro slaganje rezultata s proračunom datim u Nemačkom nacionalnom aneksu za Evrokod 5.

– Ovo istraživanje daje uvid u mogućnosti ojačanja drvenih nosača s redukovanom visinom preseka. Ono može biti dobra osnova za dalja istraživanja efikasnosti drugih vrsta ojačanja, kao što su šipke od polimera na bazi karbonskih i staklenih vlakana. Takođe, rezultati dobijeni ispitivanjem mogu biti korisni u izradi odgovarajućih analitičkih modela drvenih nosača s redukovanom visinom preseka. Postojeće metode proračuna treba revidirati, jer zanemaruju složeno naponsko stanje koje se javlja na mestu nagle promene visine preseka.

5 CONCLUSIONS

The experimental procedure performed in this research included bending tests of 15 notched glulam beams to the point of failure (five unreinforced beams and ten reinforced ones). The screws were selected as reinforcement. The effects of reinforcing were evaluated in terms of load-deflection behaviour, failure mode, ultimate load carrying capacity and stiffness of reinforced beams, which were compared with the unreinforced notched beams. In addition, comparison with analytical design method was made. The following conclusions were drawn:

– Notched glulam beams when subjected to bending failed due to stress concentration at the notch corner. The load carrying capacity of these beams is defined by excessive crack opening. Brittle failure mechanism is typical for unreinforced notched beams.

– Initial cracking of the notch corner cannot be prevented by the reinforcement. However, by using reinforcement at the notches failure is delayed significantly.

– The reinforcement prevents excessive crack opening at the notch corner. Nevertheless, final failure of the reinforced notches due to excessive crack growth is caused by crack shearing.

– Reinforcing intervention of notched glulam beams with screws increases the load carrying capacity and deformability.

– At the notch corner, reinforcement screws perpendicular to beam axis are subjected to combined parallel and perpendicular to the crack surface loading. Hence, reinforcement with high strength and stiffness in both directions is required to achieve the best reinforcing effect.

– Reinforcement screws inclined at an angle of 45° to beam axis are dominantly loaded in the axial direction. Special attention should be paid to the anchorage length of these screws.

– Generally, in order to improve anchorage, self-tapping screws are recommended to be used instead of the pre-drilled screws.

– The suggested design method in Eurocode 5 overestimated the ultimate load of unreinforced beams 7.0%.

– As for reinforced beams, if the assumption that the entire load is carried by screws is made, there is a good agreement with the proposed design method in the German National Annex to Eurocode 5.

This research gives an insight into reinforcing possibilities of notched timber members. It can be a good basis for further investigation of the effectiveness of other types of reinforcement like carbon or glass fibre based polymer bars. In addition, results obtained from tests can be useful in developing appropriate analytical design models for notched timber beams. Existing design methods should be revised, as they overlook the complex stress state around the notch area

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REZIME

EKSPERIMENTALNO ISPITIVANJE OJAČANIH LEPLJENO LAMELIRANIH DRVENIH NOSAČA SA REDUKOVANOM VISINOM PRESEKA

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Redukcija visine drvenih nosača kod oslonaca znatno umanjuje kapacitet nosivostielementa. U ovom radu je predstavljeno eksperimentalno ispitivanje lepljenih lameliranih drvenih nosača sa redukovanom visinom preseka kod oslonaca opterećenih na savijanje, sa i bez ojačanja. Kao ojačanje upotrebljeni su zavrtnji za drvo, postavljeni pod uglomod 90° i 45° u odnosu na podužnu osu nosača. Neojačni nosači su doživeli krti lom usled otvaranja pukotine i njene propagacije. Ove pukotine su rezultat prekoračenj a čvrstoće drveta na zatezanje upravno na vlakna i čvrstoće drveta na smicanje. Ispitivanje je pokazalo da se upotrebom zavrtanja može povećati kapacitet nosivosti i deformabilnosti posmatranih nosača. Međutim, primena zavrtanja kao ojačanja nije dovela do promene oblika loma iz krtog u duktilni lom usled savijanja. Lom usled smicanja je bio dominantan oblik loma kod ojačanih nosača. Takođe, analitički proračun je izvršen prema Evrokodu 5 kako bi se rezultati uporedili sa eksperimentalnim ispitivanjem.

Ključne reči: zasek, lepljeno lamelirano drvo, zavrtnji, ojačanje, eksperimentalno ispitivanje

SUMMARY

THE POSSIBILITY OF USING BLAST FURNACE SLAG AS CONCRETE AGGREGATE

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Notches made at the ends of timber beams significantly decrease load carrying capacity of a structural member. This paper presents an experimental research on bending behaviour of end-notched glulam beams, with and without reinforcement. Screws for timber were used as reinforcement, positioned at angles of 90° and 45° to the longitudinal beam axis. The unreinforced beams failed due to crack opening and its propagation in a brittle manner. Cracks that appeared in the notch details resulted from combined excessive tensile stresses perpendicular to grain and shear stresses. This study shows that reinforcing the beams at the notched ends can improve their load carrying capacity and deformability. However, applied screws did not help the beams achieve ductile failure in bending. The shear failure was dominant failure mechanism for reinforced beams. In addition, analytical calculations were performed in accordance with Eurocode 5 so as to compare the results with experimental research.

Key words: notch, glulam, screws, reinforcement, experimental investigation

PRIMENA NUMERIČKIH METODA U ANALIZI SPREGNUTIH KONSTRUKCIJA DRVO-BETON

APPLICATION OF NUMERICAL METHODS IN ANALYSIS OF TIMBER CONCRETE COMPOSITE SYSTEM

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ORIGINALNI NAUČNI RAD
ORIGINAL SCIENTIFIC PAPER
UDK:624.016.072.2
doi:10.5937/GRMK1804037M

1 UVOD

Kompozitni konstrukcijski sistemi podrazumevaju racionalno spajanje elemenata od odgovarajućih materijala, tako da se optimalno iskoriste njihova svojstva. Spregnute konstrukcije imaju najširu primenu u inženjerskim konstrukcijama velikog raspona [12], ali mogu se uspešno primeniti i u stambenim i poslovnim objektima. Adekvatnim spajanjem konstrukcijskih elemenata istih ili različitih fizičko-mehaničkih karakteristika u integralni poprečni presek, postiže se osnovni cilj postupka, tj. povećava se nosivost sistema u odnosu na pojedinačne elemente. U zavisnosti od primenjenih materijala, spregnute konstrukcije koje se često primenjuju u građevinarstvu uglavnom su tipa drvo–drvo, beton–beton, čelik–beton i drvo-beton.

Budući da su spregnute konstrukcije – napravljene od različitih materijala i različitim načinima spajanja – dostigle veoma visok stepen primene u građevinarstvu u poslednjih nekoliko decenija, neophodna je njihova preciznija analiza, kao i preciznije projektovanje. Poznato je da upotrebljeni tip sredstava za sprezanje najviše utiče na globalno ponašanje spregnutih konstrukcija. Stoga, od ključnog značaja je to kako da se uvede problem ponašanja veze između spregnutih materijala u analizi i proračunu.

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1 INTRODUCTION

Composite construction systems consider the rational structural composition of the right materials at the right places in order to optimally exploit their properties. Composite structures have the widest application in large-span engineering constructions [12], but they can be applied successfully in residential and commercial buildings. By adequate coupling of the constructive elements of the same or different physical-mechanical characteristics into an integral cross-section, the basic goal of the procedure is achieved, i.e. the capacity of the system is increased in relation to the individual elements. Depending on the applied materials, composite structures that are often in use in the construction industry generally are timber-timber, concrete-concrete, steel-concrete and timber-concrete.

Since the composite structures, made by different materials and methods of joining, have reached very high level of application in the construction industry in last several decades, there is a demand for their more precise analysis and design. It is known that the type of used fasteners mostly influence the overall behaviour of the coupled structures. Therefore, it is of crucial importance how to introduce the problem of the connection behaviour between coupled materials into the analysis and design.

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Spajanje konstitutivnih elemenata može se postići na različite načine, pri čemu jedan od najčešćih postupaka jeste upotreba diskretno postavljenih sredstava za sprezanje (npr. mehanička spojna sredstva, sidra). Spojna sredstva treba da obezbede vezu dva različita materijala, prenoseći smičuće sile između dva elementa, s ciljem obezbeđenja kompozitnog sadejstva konstrukcije. S obzirom na to što je primena štapastih spojnih sredstava najčešća u spregnutim konstrukcijama drvo–beton (SDB), a pošto ponašanje celokupne konstrukcije zavisi od njihovog ponašanja, interes istraživača i projektanata, kao i brojnih studija i istraživačkih radova, odnosi se na ove tipove sredstava za sprezanje. Upotreba mehaničkih spojnih sredstava za sprezanje dva različita materijala, kao što su drvo i beton, ukazuje na to da je ponašanje SDB konstrukcija veoma složeno, budući da spojna sredstva dozvoljavaju izvesno klizanje u spoju koje dovodi do delimične interakcije (elastično sprezanje). Prema tome, analiza i proračun SDB konstrukcija zahteva da se imaju u vidu klizanja u spoju između elemenata.

Razmatrajući jednodimenzionalni problem, prve teorije elastičnog sprezanja kod greda izloženih statičkom dejstvu razvili su Newmark (1943,1951), Granholm (1949), Pleshov (1952) i Goodman (1967). Teoriju elastičnog sprezanja primenili su Girhammar i Gopu (1991) u analizi stubova s klizanjem u spoju, izloženih odgovarajućem slučaju aksijalnog opterećenja, koja je kasnije proširena i generalizovana u njihovom daljem radu. Na osnovu prethodnih istraživanja i analiza, oni su prikazali tačan analitički postupak statičke analize elastično spregnutih nosača s klizanjem u spoju [7], a u narednim radovima [8], [9], [10] predložili su tačne analitičke i pojednostavljene metode za analizu elastično spregnutih sistema s primenom na grede i stubove. U Srbiji, u oblasti sprezanja drvo–beton, teorijske osnove za analizu delimično spregnutih sistema uz eksperimentalne rezultate dao je B. Stevanović (1994) [17], kao i Lj. Kozarić [11] i R. Cvetković [3].

Teorija parcijalnog (elastičnog) sprezanja zasniva se na odgovarajućim pretpostavkama teorije elastičnosti i uzima u obzir klizanje spoja/veze pri njihovom proračunu. Analitički proračun elastičnog sprezanja podrazumeva rešavanje diferencijalnih jednačina, gde se rešenja u zatvorenom obliku mogu formulisati samo za pojedine (jednostavnije) slučajeve konturnih uslova i opterećenja.

U EN1995 [5] usvojen je pojednostavljeni manuelni postupak proračuna („γ-metod”), koji se u praksi široko primenjuje. Ovaj metod prvobitno je primenio Mohler (1956), razmatrajući problem klizanja u spoju između spregnutih elemenata (drvo–drvo), s mehaničkim spojnim sredstvima, ali uz odgovarajuće modifikacije, ovaj postupak se može primeniti i na druge tipove spregnutih konstrukcija, kao što su konstrukcije tipa drvo–beton. „Gama” metod razvijen je za statički sistem proste grede izložene sinusoidnom opterećenju $q=q_0 \cdot \sin(\pi \cdot x/L)$. U ovom slučaju postoji jednostavno rešenje u zatvorenom obliku, koje se može primeniti i na druge vrste opterećenja, a zbog malog odstupanja od tačnog analitičkog rešenja diferencijalne jednačine. Ova metoda zasniva se na efektivnoj krutosti spregnutog sistema i teoriji elastičnog sprezanja, imajući u vidu konzervativni efekat raspodele sila unutar nosača, i skoro u potpunosti pokriva sve parametre koji utiču na

Coupling of the constitutive elements can be achieved in different ways, where one of the most common procedure is the use of number of individual shear connectors (mechanical fasteners, anchors,...). Shear connectors should ensure bond of two different materials, transferring the shear forces between two elements, enabling the composite action of the structure. The interest of researchers and constructors as well as numerous studies and research works refer to these types of fasteners since the application of dowel type connectors is the most common in timber-concrete composite structures (TCC), and additionally the behaviour of the overall construction depends on their behaviour. The use of mechanical fasteners for coupling two different materials such as timber and concrete shows that the behaviour of the TCC system is very complex, since the fasteners allow certain interlayer slip that leads to partially interaction (elastic composite action). Therefore, the analysis and design of TCC structures requires consideration of the interlayer slip between the sub-elements.

Considering one-dimensional problem, the first theories for partial composite action for beams subjected to static loads were developed by Newmark (1943,1951), Granholm (1949), Pleshov (1952) and Goodman (1967). The application of partial composite action theory was performed by Girhammar and Gopu (1991) in analysis of columns with interlayer slip subjected to one particular axial loading case which was extended and generalized in their further work. Based on previous research and analysis, they presented an exact static analysis of partial composite structures with interlayer slip [7] and afterwards in papers [8] , [9] , [10] they proposed an exact and simplified methods for analysis of the partial composite structures applied to the beams and columns. In Serbia, in the field of timber-concrete composites, the theoretical basis for analysis of partially composite system was given by B.Stevanović (1994) [17] and later on by Lj.Kozaric [11] and R.Cvetkovic [3], which was followed by experimental data.

The theory of partial (elastic) composite action is based on the corresponding assumptions of the theory of elasticity and takes into account the interlayer slip in the connection at their calculation. The exact calculation of the partial composite action implies solving differential equations where closed form solutions can be formulated only for some particular (simple) cases of boundary and loads conditions.

In EN1995 [5] the simplified manual design procedure ("γ-method") widespread in practice is adopted. This method was originally applied by Mohler (1956), considering the problem of interlayer slip between composite members (timber-timber) coupled with mechanical fasteners, but, with appropriate modifications, this procedure can be applied to the other types of composite constructions such as timber-concrete system. "Gamma" method was developed in the case of simply supported beam subjected to sinusoidal load $q=q_0 \cdot \sin(\pi \cdot x/L)$. In this case, there is a simple closed-form solution, that could be applied to the other types of loads as well, due to a slight deviation from the exact analytical solution of the differential equation. This method is based on the effective stiffness of the composite system and on the theory of elastic ponašanje SDB konstrukcija.

Takođe, za proračun spregnutih sistema, moguće je primeniti aproksimativne metode zasnovane na diferencijalnoj [14] ili varijacionoj formulaciji [13].

Diferencijalna formulacija zasniva se na izvođenju diferencijalnih jednačina koje opisuju problem u određenom domenu, gde rešenje zavisi od graničnih uslova. U rešavanju problema, potrebno je naći nepoznatu funkciju koja će zadovoljiti diferencijalnu jednačinu, kao i granične uslove. Rešavanjem izvedenih diferencijalnih jednačina, dobija se analitičko rešenje problema, gde se rešenja u zatvorenom obliku mogu dobiti samo za ograničen broj jednostavnih proračunskih modela. Ako je proračunski model kompleksan, tada se najčešće primenjuju aproksimativne metode, pogodne za dobijanje prihvatljivog rešenja. Metode reziduuma u takvim slučajevima jesu pogodan način za formulisanje numeričkog rešenja.

U varijacionoj formulaciji problema, potrebno je naći nepoznatu funkciju ili više funkcija koje zadovoljavaju uslov stacionarnosti funkcionala, gde u ovom slučaju nepoznata funkcija mora da zadovolji odgovarajuće dodatne uslove koji nisu implicitno sadržani u funkcionalu. Da bi se primenila varijaciona formulacija, neophodno je da za razmatrani problem postoji funkcional.

Na osnovu diferencijalne i varijacione formulacije problema, razvijene su brojne metode i postupci za određivanje približnih rešenja, pri čemu je od metoda reziduuma najzastupljenija Galerkinova metoda, dok je od varijacionih to Ritz-ova metoda.

Metod konačnih elemenata (MKE) jeste jedan od najčešće korišćenih numeričkih metoda u strukturalnoj analizi, pri čemu se formulacija konačnog elementa zasniva na rešenju diferencijalnih jednačina metodama reziduuma ili korišćenjem varijacione formulacije. MKE zasnovana na Galerkinovoj metodi (ili drugim metodama reziduuma) može se primeniti na mnogo širi skup diferencijalnih jednačina, jer nije potrebno imati odgovarajuću varijacionu formu, kao što je slučaj kada se koristi MKE bazirana na Raileigh-Ritz-ovoj metodi [1]. Na osnovu prethodnog izlaganja, može se zaključiti da je primena pojednostavljenih i/ili aproksimativnih numeričkih metoda za analizu i proračun SDB konstrukcija dobrodošla i preporučena. Upravo iz tog razloga, približne metode zasnovane na diferencijalnoj ili varijacionalnoj formulaciji [16] imaju široku primenu, jer mogu biti implementirane u programe za strukturalnu analizu, kako bi se obezbedio poseban alat inženjerima za proračun elastično spregnutih konstrukcija.

U radu je prikazana Galerkinova metoda u analizi SDB konstrukcija [14]. Analiziran je izbor probnih funkcija koje opisuju problem elastičnog sprezanja, kao i njihov uticaj na konačne rezultate. Za poređenje dobijenih rezultata, sprovedene su i analize prema analitičkim rešenju [17] i „gama” postupku [5]. Na osnovu predloženih numeričkih modela, model koji najbolje opisuje problem elastičnog sprezanja izabran je za dodatnu komparativnu analizu sa eksperimentalnim podacima [18]. Pored toga, predstavljena je i upotreba Ritz-ove metode u analizi SDB konstrukcija. Dobijeni rezultati prema Ritz-ovoj metodi, s različitim probnim funkcijama, analizirani su i upoređeni sa analitičkim rešenjem i „gama” postupkom. Sve analize su sprovedene upotrebom programa MATLAB [15].

coupling, taking into account the conservative effect of the distribution of forces within the girders, and so far most fully covers all the parameters that affect the behaviour of TCC.

Also, for the calculation of composite systems, it is possible to apply approximate methods based on the differential [14] or the variation formulation [13].

The differential formulation is based on the derivation of differential equations that describe the problem in a particular domain, where the solution depends on the boundary conditions. In solving the problem, it is necessary to find unknown function that satisfies differential equation as well as the boundary conditions. By solving the derived differential equations, an analytical solution of the problem arises, where the closed-form solution can be obtained only for a limited number of simpler design models. If the design model is complex, then the approximate methods are most commonly used and suitable for obtaining an acceptable solution. Residue methods are in such cases a convenient way to formulate a numerical solution.

In the variational formulation of the problem, it is necessary to find unknown function or several functions that satisfy the requirement of functional stationarity, where the unknown function must also satisfy the corresponding additional conditions that are not implicitly contained in the functional. In order to apply the variational formulation, it is necessary that functional exists for considered problem.

Numerous methods and procedures for determination of approximate solutions have been developed based on the differential and variational formulation of the problem. The Galerkin method is the most frequently applied one from the residue methods, while the Ritz method is most often used for variational formulation.

Finite element method (FEM) is one of the most used numerical methods in structural analysis where the final element formulations is based on the solution of differential equations by residual methods or using the variation formulation.FEM based on the Galerkin method (or other weighted residual methods) can be applied to a much broader set of differential equations because it is not necessary to have a proper variational form as it is the case when using Rayleigh-Ritz based FEM [1]. Based on the previous exposition, it can be concluded that the application of simplified and/or approximate numerical methods for the analysis and design of TCC structures is welcome and recommended. Therefore, the approximate methods based on differential or variational formulation [16] are widely used, because they can be implemented in structural analysis software in order to provide a specific tool for engineers for designing partial composite structures.

This paper presents the Galerkin method in the analysis of the TCC system [14]. The selection of trial functions that describe the problem of elastic composite action as well as their influence on the final results was analyzed. For comparison of the obtained results, analysis were performed according to analytical solution [17] and the "gamma" method [5]. On the basis of the proposed numerical models, a model that best describes the problem of elastic coupling was chosen for further comparative analysis with the experimental data [18]. In addition, the use of the Ritz method was also presented in the analysis of the TCC system. The obtained results according to the Ritz method with different trial functions were analyzed and compared to the analytical and the "gamma" method solutions. All analysis were performed using MATLAB software [15].

2 FORMULACIJA JEDNAČINA SDB SISTEMA

Za proračun spregnutih nosača od drveta i betona, gde se koriste mehanička spojna sredstva, primenjuje se teorija elastičnog sprezanja [17], [11].

Osnovne pretpostavke teorije elastičnosti koje se uvode jesu sledeće:

- drvo i beton su izotropni, elastični materijali – važi Hukov zakon;
- važi Bernulijeva hipoteza, odnosno ravni preseci i posle deformacije ostaju ravni i upravni na deformisanu osu preseka;
- spojna sredstva postavljena su na određenom razmaku i mogu se smatrati ekvivalentnom kontinualnom vezom s konstantnom elastičnosti spoja duž celog nosača;
- poprečni preseci betona i drveta konstantni su duž raspona;
- drvo i beton imaju jednake ugibe u svakoj tački spoja;
- aksijalna sila deluje u težištu betonskog preseka.

Pri savijanju SDB nosača, nastaje pomeranje (klizanje *v*) u spojnoj ravni dva materijala. Klizanje elemenata sprečeno je spojnim sredstvima, što uzrokuje pojavu sile klizanja (smičuće sile u kontaktnoj ravni) T_s koja izaziva silu pritiska N_1 i momenat savijanja M_1 – u gornjem, a silu zatezanja N_2 i momenat savijanja M_2 – u donjem elementu nosača, slika 1 (gde su sa *A* i *I* obeležene geometrijske karakteristike poprečnih preseka gornjeg i donjeg elementa, a sa *E* moduli elastičnosti primenjenih materijala). Intenziteti sila zavise od krutosti i deformabilnosti spojnog sredstva, odnosno njegovog modula pomerljivosti *K* [2].

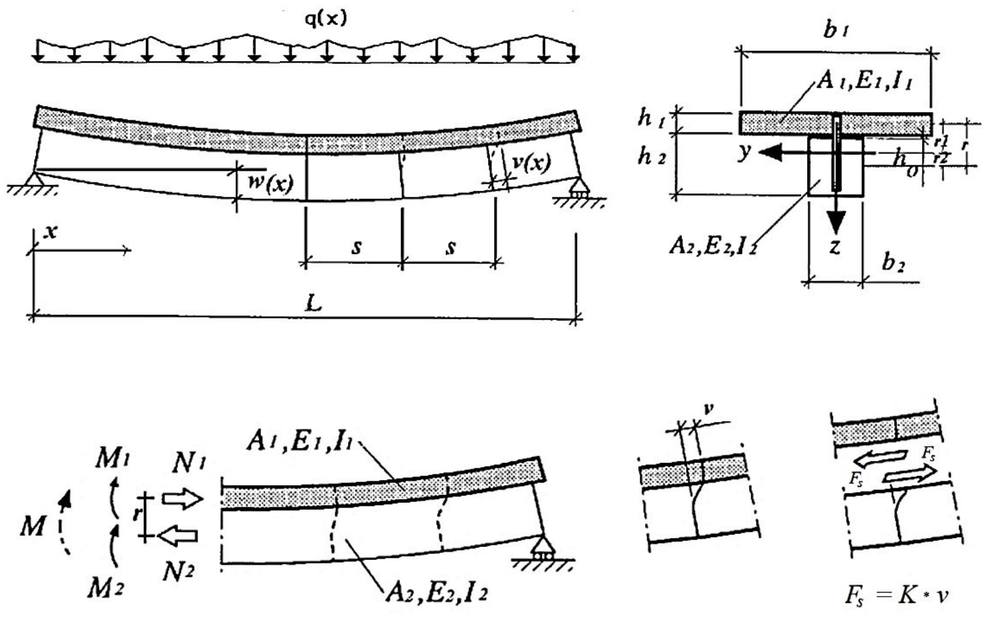
2 GOVERNING EQUATION OF TCC SYSTEM

The theory of elastic coupling [17], [11] is used for the calculation of TCC structures, where mechanical fasteners are used.

The basic assumptions of the theory of elasticity that are introduced are as follows:

- timber and concrete are isotropic, elastic materials and Hook's law applies,
- Bernoulli's hypothesis is valid, i.e. plane sections initially perpendicular to the midsurface will remain plane and perpendicular on deformed axis,
- coupling means are set at certain distances and can be considered as equivalent continuous connection with the constant elasticity along the beam,
- cross sections of concrete and timber are constant along the span,
- concrete and timber have equal deflections at each point of the connection,
- axial force acts at the centre of gravity (centroid) of the concrete section.

In TCC structure, one element slips (*v*)over the other along TC interface in the case of bending. Sliding of elements is prevented by the coupling means with appearance of interlayer slip (shear in contact interface) force T_s with compression force N_1 and the bending moment M_1 in the upper and the tensile force N_2 and the bending moment M_2 in the lower element of the structure, Figure 1 (where notation *A* and *I* represent geometrical properties of cross-sections of upper and lower element, while *E* represent the modulus of elasticity of applied materials). The intensities of forces depend on the stiffness and deformability of the coupling means and its slip modulus *K* [2].



Slika 1. Klizanje u kontaktnoj ravni SDB nosača [2]
Figure 1. Interlayer slip of TCC beam [2]

Kada se razmatra spregnuta greda drvo–beton, statičkog sistema proste grede, opterećena ravnomerno raspodeljenim opterećenjem *q(x)*, bez spoljašnje aksijalne sile, problem elastičnog sprezanja može se predstaviti diferencijalnom jednačinom drugog reda u aksijalne sile u betonu:

The problem of partial composite action could be represented with differential equation of the second order in the function of the axial force in concrete while observing the composite timber-concrete simply supported beam system with uniformly distributed load *q(x)* without an external axial force:

$$N_1''(x) - \alpha^2 N_1(x) = \beta M(x) \quad (1)$$

gde je:

where are:

$$\alpha^2 = k \cdot \left(\frac{1}{A_1 E_1} + \frac{1}{A_2 E_2} + \frac{r^2}{(EI)_0} \right) \quad (2)$$

$$\beta = \frac{k \cdot r}{(EI)_0} \quad (3)$$

M(x) – moment kruto spregnutog preseka ($k \to \infty$);

k – krutost spoja („raspodeljen” modul pomerljivosti) [N/m^2], *k=K/s;*

K – modul pomerljivosti spojnog sredstva [N/m], određen ispitivanjima;

s – rastojanje spojnih sredstava za sprezanje;

r – rastojanje između težišta betonskog i drvenog dela preseka.

Takođe, problem SDB nosača može se izraziti putem diferencijalne jednačine četvrtog reda u funkciji vertikalnog pomeranja:

M(x)- the moment of the fully composite section ($k \to \infty$),

k - the slip modulus per-unit length (“smeared” slip modulus)[N/m^2], *k=K/s,*

K-the slip modulus [N/m], determined by testing,

s-the spacing between connections,

r - the distance between centroid of flange and web elements.

In addition, the problem of the TCC beam could be expressed through the differential equation of the fourth order in function of vertical displacement:

$$w''''(x) - \alpha^2 \cdot w''(x) = \frac{\alpha^2 \cdot M(x)}{(EI)_\infty} - \frac{M''(x)}{(EI)_0} \quad (4)$$

gde je:

where is:

$$(EI)_\infty = \frac{\alpha^2 \cdot (EI)_0}{\alpha^2 - \beta \cdot r} = E_1 I_1 + E_2 I_2 + \frac{r^2 \cdot E_1 A_1 \cdot E_2 A_2}{E_1 A_1 + E_2 A_2} \quad (5)$$

$(EI)_0$ i $(EI)_\infty$ predstavljaju savojnu krutost za nespregnutu ($k \to 0$) i kruto spregnutu ($k \to \infty$) gredu, respektivno.

Rešavanje diferencijalnih jednačina (1 ili 4) predstavlja složen zadatak, i to naročito za različite slučajeve opterećenja i/ili granične uslove. U literaturi, za različite slučajeve opterećenja i uslove oslanjanja, mogu se pronaći analitička rešenja. U radu [17], analitička rešenja za aksijalnu silu *N*, silu klizanja u spoju T_s i vertikalno pomeranje *w*, za statički sistem proste grede i kontinualno opterećenje, data su jednačinama (6–8).

$(EI)_0$ and $(EI)_\infty$ are the bending stiffness of non-composite ($k \to 0$) and fully composite ($k \to \infty$) beam, respectively.

Solving the differential equations (1 or 4) is a complex task, especially for different load cases and/or boundary conditions. In the literature, analytical solutions for different load cases and support conditions could be found. According to [17], analytical solutions for axial force *N*, interlayer slip force T_s and vertical displacement *w*, for a simply supported beam system and continuous load, are given by the equations (6-8).

Aksijalna sila

Axial force

$$N(x) = \frac{\beta}{\alpha^2} M(x) \left[1 - 2 \frac{\cosh \alpha \frac{l}{2} - \cosh \alpha (\frac{l}{2} - x)}{x(l - x) \alpha^2 \cosh \alpha \frac{l}{2}} \right] \quad (6)$$

Sila klizanja

Slip force

$$T_s(x) = \frac{\beta}{\alpha^2} T(x) \left[1 - \frac{\sinh \alpha (\frac{l}{2} - x)}{(\frac{l}{2} - x) \alpha \cosh \alpha \frac{l}{2}} \right] \quad (7)$$

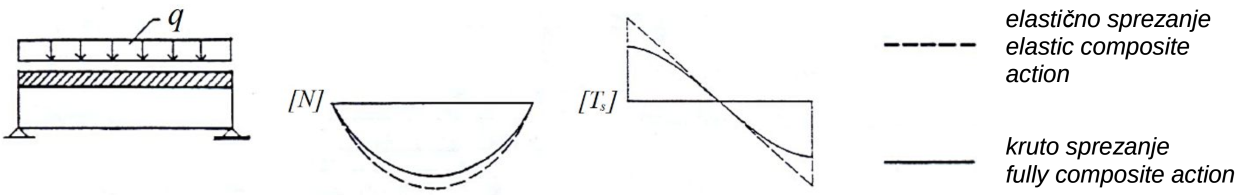
Vertikalno pomeranje

Vertical displacement

$$w(x) = w_\infty(x) + \frac{q \cdot \beta \cdot r}{\alpha^6 \cdot (EI)_0} \cdot f(x)$$
$$w_\infty(x) = \frac{q \cdot L^4}{24 \cdot (EI)_\infty} \cdot \left(\frac{x}{L}\right) \cdot \left(1 - 2 \cdot \left(\frac{x}{L}\right)^2 + \left(\frac{x}{L}\right)^3\right) \tag{8}$$
$$f(x) = \left(\frac{\cosh\left(\alpha \cdot \left(x - \frac{L}{2}\right)\right)}{\cosh\left(\alpha \cdot \frac{L}{2}\right)} + \frac{x \cdot L \cdot \alpha^2}{2} - \frac{x^2 \cdot \alpha^2}{2} - 1\right)$$

Prikaz kvalitativne promene aksijalne sile N to jest sile klizanja u spoju T_s SDB nosača, dat je na slici 2.

A demonstration of the qualitative change of the axial force N i.e. slip force T_s in TCC system is shown in Figure 2.



Slika 2. Prikaz kvalitativne promene normalne tj. sile klizanja u SDB sistemu [17]
Figure 2. Demonstration of qualitative change of the axial i.e. slip forces in TCC [17]

3 APROKSIMATIVNE METODE

Problemi teorije elastičnosti opisani su pomoću diferencijalne formulacije (diferencijalne jednačine i odgovarajućih graničnih uslova) ili u varijacionoj formulaciji u obliku funkcionala. Iako rešenja ovih problema u matematičkom smislu egzistiraju kao jednoznačna, nalaženje analitičkih rešenja predstavlja zahtevan i često nerešiv zadatak. Stoga, približne metode često se koriste prilikom određivanja rešenja za ove probleme. Posebno značajne jesu one metode gde se kao polazna osnova koristi pretpostavka o rešenju u obliku aproksimativnih ili probnih funkcija, pri čemu je jedna od najčešće primenjivanih metoda reziduuma Galerkinova metoda, dok je od varijacionih metoda to najčešće Ritz-ova metoda.

Nepoznata funkcija $u(x)$ diferencijalne jednačine problema aproksimira se približnim rešenjem $\bar{u}(x)$, izraz (9), koje se može predstaviti kao suppozicija proizvoda poznatih baznih funkcija Φ_m i nepoznatih koeficijenta c_m.

3 APROXIMATE METHODS

The problems of the theory of elasticity are described by means of the differential formulation (differential equations and corresponding boundary conditions) or in the variational formulation in the form of the functional. Although the solutions to these problems in the mathematical sense exist as unambiguous, finding analytical solutions is a delicate and often unsolvable task. Therefore, the approximate methods are often used to find solutions to these problems. Of particular interest are those methods in which the assumption of a solution in the form of approximate or trial function is used as the baseline, wherein one of the most commonly applied weight residual methods is Galerkin method, and commonly applied variational method is Ritz method.

The unknown function $u(x)$ of problem's differential equation has to be approximated by the approximate solution $\bar{u}(x)$, expression (9), that could be represented as a superposition of products of known basis functions Φ_m and unknown coefficients c_m.

$$\bar{u}(x) = \sum_{m=1}^{n} c_m \cdot \Phi_m(x_m) \tag{9}$$

gde je:

Φ_m – skup izabranih linearno nezavisnih funkcija $\Phi_m(x_m)$;

c_m – nepoznati parametri, konstante ili funkcije, koje treba odrediti.

Najčešći oblici probnih funkcija jesu polinomi ili trigonometrijske funkcije. Funkcije Φ_m unapred se usvajaju, imajući u vidu granične uslove po

where are:

Φ_m - set of chosen linearly independent functions $\Phi_m(x_m)$,

c_m- unknown parameters, constants or functions to be determined.

The most common trial functions are polynomials or trigonometric functions. Functions Φ_m are adopted in advance by taking into account of essential boundary pomeranjima. Kada je reč o drugim uslovima, izbor funkcija Φ_m uglavnom je proizvoljan, ali kvalitet rešenja umnogome zavisi baš od izbora funkcija Φ_m. Poželjno je da funkcije Φ_m zadovoljavaju i granične uslove po silama, te da njihov oblik kvalitativno odgovara tačnom analitičkom rešenju. Dakle, kvalitativno poznavanje prirode rešenja veoma je korisno da bi se izbegao pogrešan izbor funkcija koje po svom obliku predstavljaju grubo odstupanje od analitičkog rešenja.

Galerkinova metoda ima širu primenu od Ritz-ove metode, jer se može primeniti pri rešavanju onih problema za koje funkcional ne postoji. U mehanici deformabilnih tela, ove dve metode su ekvivalentne, jer daju rezultate iste tačnosti. Izborom istih probnih funkcija u Ritz-ovoj i Galerkin-ovoj metodi, dobijaju se isti koeficijenti c_m (ista rešenja).

conditions. As regards other conditions, the choice of functions Φ_m is generally arbitrary, but the quality of the solution largely depends on the choice of functions Φ_m. It is desirable that the functions Φ_m also satisfies natural boundary conditions, and thattheir shape qualitatively corresponds to the exact analytical solution. Therefore, qualitative knowledge of the nature of the solution is very useful in order to avoid the wrong choice of functions that in their form represent a rough deviation from the analytical solution.

Galerkin method has wider application than Ritz's because it can solve even those problems in which the functional does not exist. In the mechanics of deformable bodies, these two methods are equivalent, as they give results of the same accuracy. By choosing the same trial functions in Ritz and Galerkin method, the same coefficients of c_m (i.e. same solutions) will be obtained.

3.1 Metoda reziduuma

Neka je posmatrani fizički problem, u domenu Ω, koji može da bude 1D do 3D, definisan diferencijalnom jednačinom:

3.1 Weighted residual method

A physical problem is observed in the domain Ω, which can be 1D to 3D, defined with a differential equation:

$$L(u) - \boldsymbol{f}_{\Omega} = 0 \tag{10}$$

gde je:

L – odgovarajući linearni diferencijalni operator;

$u(x)$ – nepoznata funkcija problema, koja zavisi od koordinate x unutar prostora Ω, pri čemu funkcija $u(x)$ zadovoljava date granične uslove na granicama domena Ω;

f_Ω – vektor slobodnih članova u jednačini u domenu Ω.

Nepoznata funkcija problema $u(x)$ aproksimira se s približnom funkcijom $\bar{u}(x)$, izraz (9), koja zadovoljava granične uslove po pomeranjima (esencijalne granične uslove), ali ne mora da zadovoljava i uslove po silama (prirodne granične uslove). Kako je $\bar{u}(x)$ približno rešenje jednačine (10), dobija se ostatak ili reziduum:

where are:

L - corresponding linear differential operator,

$u(x)$ - unknown function of the problem, that depends on the coordinate x within the domainΩ, where the function$u(x)$ satisfy the given boundary conditions at the boundaries of the domain Ω,

f_Ω – given force term in domainΩ.

The unknown function of the problem $u(x)$ is approximated with the approximate function $\bar{u}(x)$, equation (9), which satisfies the boundary conditions upon the displacements (essential conditions), but does not have to satisfy the conditions by forces (natural conditions). As $\bar{u}(x)$ is approximate solution of the equation (10), the residue or residuum is obtained:

$$L(\overline{\boldsymbol{u}}) - \boldsymbol{f}_{\Omega} = \boldsymbol{R}(\overline{\boldsymbol{u}}) \neq 0 \tag{11}$$

Kako je jednačina (10) sistem jednačina, odnosno matrična jednačina, ostatak jeste $R(\bar{u})$ vektor. Naravno, kada bi $\bar{u}(x)$ bilo tačno analitičko rešenje, onda bi vektor ostatka $R(\bar{u})$ bio jednak nultom vektoru. Ideja metode jeste da se vektor ostatka svede na nulti vektor „u prosečnom smislu”. Stoga, uvode se linearno nezavisne težinske funkcije $W(\bar{u})$, uz uslov da integral skalarnog proizvoda vektora težinskih funkcija i vektora ostatka unutar domena Ω bude jednak nuli:

Since equation (10) is a system of equations i.e. a matrix equation, than the residue $R(\bar{u})$ is a vector. Of course, if $\bar{u}(x)$ would be the exact solution, then the residue vector $R(\bar{u})$ would be equal to the zero vector. The idea behind the method is to reduce the residue vector to the zero vector "in the average sense". Because of that, linearly independent weight functions $W(\bar{u})$ are introduced with the condition that the integral of the scalar product of the weight function vector and the residual vector within the domain Ω is equal to zero:

$$\int_{\Omega} \boldsymbol{W}^T(\overline{\boldsymbol{u}}) \cdot \boldsymbol{R}(\overline{\boldsymbol{u}}) \cdot d\Omega = \int_{\Omega} \boldsymbol{W}^T(\overline{\boldsymbol{u}}) \cdot (L(\overline{\boldsymbol{u}}) - \boldsymbol{f}_{\Omega}) \cdot d\Omega = 0 \tag{12}$$

Skalarni proizvod dva vektora jednak je nuli ukoliko su ti vektori međusobno ortogonalni. Prema tome, integralna jednačina (12) predstavlja uslov ortogonalnosti vektora ostatka na izabrani vektor težinskih funkcija.

Metode reziduuma sastoje se u nalaženju funkcija $\bar{u}(x)$ za koje će integralna jednačina (12) biti zadovo-

The scalar product of the two vectors is equal to zero if these vectors are mutually orthogonal. Accordingly, the integral equation (12) is a condition of the orthogonality of the residual vector to the selected vector of weight functions.

Residue methods consist of finding functions $\bar{u}(x)$ for which the integral equation (12) will be satisfied. If the ljena. Ako je jednačina (12) zadovoljena za bilo koji vektor težinskih funkcija, onda će se vektor ostatka približavati nultom vektoru.

Na taj način, približno rešenje $\bar{u}(x)$ aproksimira tačno rešenje $u(x)$. Sva rešenja $\bar{u}(x)$ koja zadovoljavaju (10) moraju da zadovoljavaju i (12), bez obzira na izbor težinskih funkcija. Dimenzija vektora težinskih funkcija odgovara broju nepoznatih koeficijenata c_m razmatranog problema.

Kao jedna od osnovnih varijanti metode reziduuma, koja usvaja težinske funkcije kao bazne funkcije Φ_m kojima je aproksimirano traženo rešenje, jeste Galerkinova metoda [16].

Na osnovu diferencijalnih jednačina elastičnog sprezanja (1 ili 4) i uslova (12), moguće je definisati sledeće relacije za određivanje problema SDB nosača u funkciji aksijalne sile u betonu ili u funkciji pomeranja za slučaj SDB grede opterećene kontinualnim opterećenjem q, prema sledećim izrazima:

equation (12) is satisfied for any weight functions vector, then the residue vector will approach the zero vector.

In this way, the approximate solution $\bar{u}(x)$ approximates the exact solution $u(x)$. All solutions $\bar{u}(x)$ that satisfy (10) must satisfy (12) regardless of weight functions' choice. The dimension of the weight functions vector corresponds to the number of unknown coefficients c_m of the considered problem.

As one of the basic variants of the residual method, which adopts weight functions as basis functions Φ_m for which the required solution is approximated, is Galerkin's method [16].

Based on the differential equations of the elastic coupling (1 or 4) and the condition (12), it is possible to define the following relations for determining the problem of the TCC girder trough the axial force in the concrete or trough displacements for the case of a TCC beam loaded with continuous load q, according to fallowing expressions:

$$\int_0^L \Phi_m(x) \cdot [N_1''(x) - \alpha^2 N_1(x) - \beta M(x)] \cdot dx = 0 \quad , \quad m = 1,2,\dots,n \tag{13}$$

$$\int_0^L \Phi_m(x) \cdot \left[\bar{w}''''(x) - \alpha^2 \bar{w}''(x) - \frac{\alpha^2 M(x)}{(EI)_\infty} + \frac{M''(x)}{(EI)_0}\right] \cdot dx = 0 \quad , \quad m = 1,2,\dots,n \tag{14}$$

Integralna formulacija koja u sebi implicitno sadrži diferencijalnu jednačinu problema, naziva se slaba formulacija (13 ili 14) koja izražava uslove i relacije koje moraju biti zadovoljene u prosečnom ili integralnom smislu.

Kako je diferencijalna jednačina problema parnog reda ($2r = 4$), parcijalnom integracijom izraza (12) red izvoda r u probnim funkcijama moguće je smanjiti sa $r = 4$ na $r = 2$. Parcijalnom integracijom izraza (13 ili 14) postiže se da odabrane probne funkcije moraju zadovoljavati samo granične uslove po pomeranjima, koji moraju biti zadovoljeni izborom samih probnih funkcija, dok su uslovi po silama već uključeni u formulaciju problema parcijalnom integracijom.

Rešavanjem integrala, dobija se sistem od n jednačina po nepoznatim koeficijentima c_m i približno rešenje za traženu funkciju $u(x)$ može se dobiti određivanjem koeficijenata c_m.

An integral formulation that implicitly contains a differential equation of the problem is called a weak formulation (13 or 14) that expresses the conditions and relations that must be satisfied in the average, or in an integral sense.

Since the differential equation of the problem is of even order ($2r = 4$), it is possible to reduce the required order of derivation in the trial functions by partial integration of the expression (12) from $r = 4$ to $r = 2$. By partial integration of expressions (13 or 14) is achieving that selected trial functions must satisfy only the essential conditions, that have to be satisfied by the selection of trial functions themselves, while the force conditions are already included into the formulation of the partial integration problem.

By solving the integrals, a system of n equations by unknown coefficients $c_m$is obtained andan approximate solution for the required function $u(x)$ could be derived by determination of coefficients c_m.

3.2 Varijaciona metoda

Kako se Ritz-ova metoda zasniva na varijacionoj formulaciji, potrebno je zadovoljiti uslov stacionarnosti funkcionala koji opisuje razmatrani problem. Za rešavanje problema u mehanici deformabilnih tela, funkcional je jednak ukupnoj potencijalnoj energiji, a stacionarna vrednost odgovara njenoj minimalnoj vrednosti. U *Teoriji konstrukcija*, ovaj metod je najpoznatiji varijacioni postupak. Razlog jeste to što postoji funkcional u obliku potencijalne energije [13].

Kada da se posmatra jednodimenzionalni linijski problem s domenom definisanosti $x Î [x_1,x_2]$, funkcional (potencijalna energija) izražava se putem integrala $I(u)$ u celom domenu:

3.2 Variational method

As the Ritz method is based on a variational formulation, it is necessary to satisfy the requirement of extremum of a functional that describes the problem under consideration. To solve problems in the mechanics of deformable bodies, the functional is equal to the total potential energy, and the stationary value corresponds to its minimum value. In the theory of structures this method is the most famous variation procedure. The reason for that is that there is a functional in the form of potential energy [13]. In case of one-dimensional beam problem with the defined domain $x Î [x_1,x_2]$, functional (potential energy) is expressed as an integral $I(u)$over the entire domain:

$$I(u)=\int_{x1}^{x2}\Pi\left(x,u(x),\frac{du(x)}{dx},\frac{d^2u(x)}{dx},\cdots\right)dx \tag{15}$$

gde je:

Π (...) – funkcional funkcija *u(x)*, *du(x)/dx*, $d^2u(x)/dx$,...

Uslov stacionarnosti funkcionala prikazuje se uslovom da je prva varijacija funkcionala jednaka nuli:

where is:

Π (...) - represents the functional of functions*u(x)*, *du(x)/dx*, $d^2u(x)/dx$,...

Extremum of a functional is represented by requirements that the first variation of the functional be zero:

$$\delta\Pi=0 \tag{16}$$

ili zapisano u razvijenom obliku:

or shown in the developed form:

$$\delta\Pi=\frac{\partial\Pi}{\partial c_1}\delta c_1+\frac{\partial\Pi}{\partial c_2}\delta c_2+.\dots\dots\dots\dots+\frac{\partial\Pi}{\partial c_n}\delta c_n=0 \tag{17}$$

Kako su koeficijenti $c_1,c_2,...,c_n$ međusobno nezavisni parametri, onda se $\delta\Pi$=0 svodi na sledeći uslov:

Since $c_1,c_2,...,c_n$ are mutually independent parameters, then $\delta\Pi$=0 is represented by following condition:

$$\frac{\partial\Pi}{\partial c_m}=0 \qquad (m=1,2,\dots,n) \tag{18}$$

što predstavlja sistem algebarskih jednačina po nepoznatim koeficijentima c_m.

Na osnovu diferencijalne formulacije problema elastičnog sprezanja, moguće je definisati funkcional na osnovu opštih varijacionih principa [8]. Pošto se najčešće koriste mehanička spojna sredstva za SDB nosače, usled spoljašnjeg opterećenja javljaju se izvesna pomeranja (tj. klizanja u spoju) na kontaktu između drveta i betona. Pored rada unutrašnjih sila (M_1, N_1, M_2 i N_2 where $N=N_1=-N_2$), potrebno je uzeti u obzir i deformacioni rad usled klizanja u spoju. Funkcional, ili ukupna potencijalna energija spregnutog sistema, u slučaju proste grede na koju deluje raspodeljeno opterećenje *q(x)*, može se prikazati u sledećem obliku [19]:

which represents a system of algebraic equations with unknown coefficients c_m.

Based on the differential formulation of the partially composite problem, it is possible to define a functional according to variation principles [8]. As the mechanical fasteners are commonly used for coupling in TCC, a certain displacements (i.e. an interlayer slip) occur on the TC interface due to the external load. Besides the strain energy due to internal forces (M_1, N_1, M_2 i N_2 where $N=N_1=-N_2$), it is also necessary to take into account the strain energy due to interlayer slip. Functional, or total potential energy of the composite system, in the case of simply supported beam with uniform distributed load *q(x)*, can be shown in the following form [19]:

$$I=W_i-W_e \tag{19}$$

$$W_i=\frac{1}{2}\int_0^l\frac{{M_1}^2(x)}{E_1I_1}dx+\frac{1}{2}\int_0^l\frac{{M_2}^2(x)}{E_2I_2}dx+\frac{1}{2}\int_0^l\frac{N^2(x)}{EA^*}dx+\frac{1}{2}\int_0^l\frac{(N'(x))^2}{k}dx \tag{20}$$

$$W_e=\int_0^l w(x)\cdot q(x)\cdot dx \tag{21}$$

gde je:

where is:

$$EA^*=\frac{E_1A_1\cdot E_2A_2}{E_1A_1+E_2A_2} \tag{22}$$

W_i – potencijalna energija deformacije;

W_e – potencijal sila.

Kako moment savijanja *M(x)* možemo izraziti preko pomeranja *w(x)*, koristeći uslov jednakih rotacija spregnutih elemenata (drveta i betona), uvodeći odnos *g(x)*, izraz (20) prikazujemo u sledećem obliku:

W_i- strain energy due to internal forces,

W_e -potential energy due to external forces.

As the bending moment *M(x)* can be expressed by deflection *w(x)*, using the condition of equal rotations of the composite members (timber and concrete), introducing the relation *g(x)*, the expression (20) is represented in the following form:

$$W_i = \frac{(EI)_0}{2}\int_0^l [w''(x)]^2 dx + \frac{(E_2I_2)^2}{EA^*}\int_0^l \left[\frac{w''(x)}{g(x)}\right]^2 dx + \frac{1}{2\cdot k}\int_0^l \left[\left(-\frac{E_2I_2\cdot w''(x)}{g(x)}\right)'\right]^2 dx \tag{23}$$

gde je:

where:

$$g(x) = \frac{M_2(x)}{N(x)} = \frac{E_2I_2}{r}\cdot\left(\frac{1}{EA^*} - \frac{N''(x)}{N(x)}\right) \tag{24}$$

$$N(x) = \frac{M_2(x)}{g(x)} = -\frac{E_2I_2\cdot w''(x)}{g(x)} \tag{25}$$

Uvedeni odnos *g(x)* izveden je iz uslova kompatibilnosti pomeranja na spoju dva elementa, koji može da se zapiše u sledećem obliku:

The introduced relation *g(x)* was derived from the compatibility of displacements at the interface of the two subelements, that could be written in the following form:

$$\frac{N(x)}{EA^*} - \frac{M_2(x)\cdot r}{E_2I_2} = \frac{N''(x)}{k} \tag{26}$$

Poznajući rad unutrašnjih sila W_i, određen je funkcional za elastično spregnuti SDB nosač. Kako se u izrazu *g(x)* javlja normalna sila *N(x)*, za rešavanje problema, pored pretpostavljanja probne funkcije za pomeranje *w(x),* potrebno je pretpostaviti i probnu funkciju za normalnu silu *N(x)*. Primenom varijacionih principa na funkcional, Ritz-ovom metodom, možemo rešiti problem elastičnog sprezanja, odnosno odrediti pomeranje nosača i unutrašnje sile u spregnutom nosaču.

Knowing the strain energy due to internal forces W_i, functional for a partial TCC system is determined. As in the expression *g(x)* the normal force *N(x)* appears, for solving the problem, beside assumed trial function of displacement *w(x),* it is also necessary to assume the trial function for *N(x)*. By applying variation principles to a functional, with Ritz method, the problem of partially composite system can be solved, which means to determine displacement and internal forces in the composite members.

4 APROKSIMACIJA REŠENJA – PROBNE FUNKCIJE

Pogodne, a samim tim i najčešće, probne funkcije su polinomi ili trigonometrijske funkcije. Probne funkcije treba da zadovolje sledeće uslove:

- da su neprekidne i do potrebnog reda diferencijabilne;
- pored esencijalnih graničnih uslova, treba da zadovolje prirodne granične uslove;
- treba da oblikom kvalitativno odgovaraju tačnom rešenju;
- da budu potpune, npr. u slučaju polinoma određenog stepena, takođe treba da budu uključeni i svi članovi nižeg stepena.

Na osnovu dobro poznatih rešenja iz literature, jednačine (6) i (8), kao i kvalitativnog poznavanja oblika rešenja (slika 2), u ovom radu za probne funkcije izabrane su tri funkcije (hiperbolična, sinusna i funkcija oblika polinoma).

Usvojene probne funkcije opisuju zakon promene aksijalne sile *N*, sile klizanja u spoju T_s *(N')* i pomeranje *w* duž spregnutog nosača i kvalitativno odgovaraju analitičkim rešenjima (slika 2). Pomoću izraza (27, 28 i 29), date su odabrane probne funkcije za *N* i/ili *w*, dok je na slici 3 prikazan oblik funkcije i njen prvi izvod duž nosača.

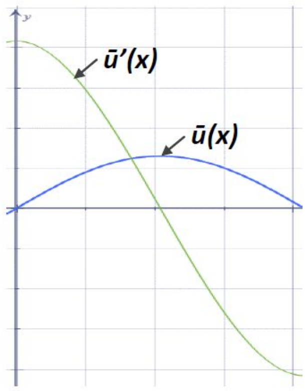
4 APPROXIMATION OF THE SOLUTION – TRIAL FUNCTIONS

Suitable, and therefore, most often, trial functions are polynomials or trigonometric functions. Trial functions should satisfy the following conditions:

- to be continuous and differentiable till the necessary order,
- in addition to essential, they also have to satisfy natural boundary conditions,
- to correspond qualitatively by the form to the analytical solution,
- to be complete, e.g. in the case of polynomials of a certain degree, all members of the lower degrees should also be included.

Based on the well-known solutions from the literature, equations (6) and (8), as well as on the qualitative flow of the solution (Fig. 2), three functions (hyperbolic, sinusoidal and polynomial functions) were selected for trial functions in this paper.

The adopted trial functions describe the law of the change of the axial force *N*, slip forces T_s *(N')* and displacement *w* along the composite girder and qualitatively correspond to the solutions (Fig. 2). By means of expressions (27, 28 and 29), selected trials for *N*and/or*w* are given, while on Fig. 3 the shape of the function and its first derivative along the beam are shown.



Slika 3. Probna funkcija i njen prvi izvod
Figure 3. Trial function and its first derivative

1. Hiperbolična funkcija / *Hyperbolic function*:

$$\bar{u}(x) = c_1 \cdot \Phi_1(x) = c_1 \cdot \left(\frac{\cosh\left(x - \frac{L}{2}\right)}{\cosh\left(\frac{L}{2}\right)} + \frac{x \cdot L}{2} - \frac{x^2}{2} - 1 \right) \tag{27}$$

2. Sinusna funkcija / *Sinusoidal function*:

$$\bar{u}(x) = c_1 \cdot \Phi_1(x) = c_1 \cdot \sin\left(\frac{\pi \cdot x}{L}\right) \tag{28}$$

3. Polinom / *Polynomial function*:

$$\bar{u}(x) = c_1 \cdot \Phi_1(x) = c_1 \cdot \left(\frac{x}{L}\right) \cdot \left(1 - 2 \cdot \left(\frac{x}{L}\right)^2 + \left(\frac{x}{L}\right)^3\right) \tag{29}$$

U Galerkinovoj metodi, jednu od tri predložene funkcije treba usvojiti za probnu funkciju (za *N* ili *w*) prema odabranoj integralnoj formulaciji (jednačine 13 ili 14). U Ritz-ovoj metodi, potrebno je usvojiti dve probne funkcije (za *N* i *w*).

5 NUMERIČKE ANALIZE I POTVRDA REZULTATA

5.1 Opis analiziranog modela

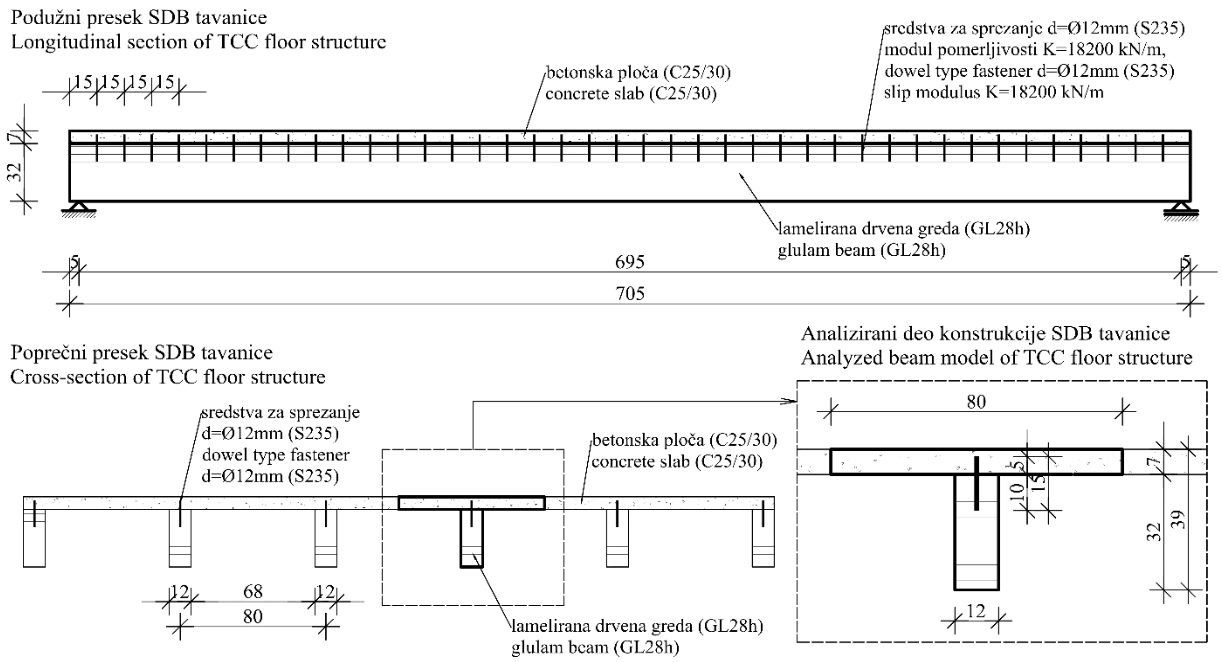
Razmatrana je SDB konstrukcija tavanice za numeričku analizu metodama Galerkina i Ritz-a. Raspored elemenata i spojnih sredstava, kao i njihove dimenzije i svojstva primenjenih materijala prema evropskim standardima, prikazani su na slici 4.

In Galerkin method, one of three suggested trial functions has to be adopted (for *N* or *w*) according to chosen integral formulation (Eq 13 or 14). In Ritz method it is necessary to adopt two trial functions(for *N* and *w*).

5 NUMERICAL ANALYSIS AND VERIFICATION OF THE RESULTATS

5.1 Description of analyzed structural model

The TCC floor structure is considered for the numerical analysis by Galerkin and Ritz method. The disposition of the elements and fasteners, as well as their dimensions and properties of the applied materials according to European standards, are shown in Fig. 4.



Slika 4. Analizirani model međuspratne SDB konstrukcije [13]
Figure 4. Analyzed model of the TCC floor structure [13]

Konstrukcija tavanice sastoji se od lameliranih lepljenih drvenih (LLD) greda koje su – zajedno s betonskom pločom – spregnute vertikalno postavljenim štapastim spojnim sredstvima. U ovom radu, modul pomerljivosti K određen je Gelfijevim modelom [6]. Tavanica je opterećena sopstvenom težinom elemenata konstrukcije g, dodatnim stalnim opterećenjem d_g, kao i korisnim opeterećenjem p. Smatra se da će LLD grede biti poduprte u fazi izlivanja i očvršćavanja betonske ploče, te će spregnuti presek primati korisno i ukupno stalno opterećenje. Moguće je analizirati izdvojen deo spregnute tavanice (LLD greda s betonskom pločom efektivne širine), jer se smatra da u analiziranoj SDB tavanici betonska ploča nosi u jednom pravcu, a da su LLD grede statičkog sistema proste grede opterećene ravnomerno raspodeljenim opterećenjem. Sprovedena je numerička analiza SDB nosača prema Galerkinovoj i Ritz-ovoj metodi primenom programa MATLAB 2014 [15] u kome su napisani potprogrami/kodovi za njihov proračun. Takođe, pojednostavljenim „γ postupkom" izvršena je analiza kako bi se odredile referentne vrednosti predložene Evrokodom.

5.2 Numerička analiza primenom Galerkinove metode

Za potrebe numeričke analize, definisane su dve grupe modela na osnovu izbora probnih funkcija i integralnih formulacija (jednačine 13 ili 14) za aksijalnu silu $N(x)$ ili ugib $w(x)$: Modeli grupe A (N-HIP, N-SIN, N-POL) i Modeli grupe B (w-HIP, w-SIN, w-POL).

Sračunata su vertikalna pomeranja (w), momenti savijanja (M_1, M_2), aksijalne sile (N_1, N_2) i naponi (σ_1 i σ_2) u betonu / drvetu (gornje i donje vlakno) za presek u sredini raspona grednog nosača, kao i smičuće sile (F_s) u spojnim sredstvima i sile klizanja (T_s) na kontaktu betona i drveta nad osloncem. Rezultati numeričke analize modela iz grupe A i B upoređeni su s rezultatima analitičkog rešenja, a njihova procentualna odstupanja prikazana su na slikama 5, 6. Rezultati pojednostavljenog „γ-postupka" takođe su poređeni sa analitičkim rešenjem.

Sa slika 5 i 6, može se uočiti da rezultati numeričkih modela grupe A imaju manja odstupanja u odnosu na analitičko rešenje nego modeli grupe B.

Analiza rezultata dobijenih primenom različitih probnih funkcija za aproksimativno rešenje normalne sile N, pokazuje da se minimalno odstupanje javlja kada se usvoji hiperbolična funkcija, a maksimalno ako se usvoji sinusna funkcija. Modeli N-POL i N-SIN imaju znatna odstupanja kod napona ($\sigma_{1,b}$ and $\sigma_{2,t}$) i to čak do 21%, a manja odstupanja za sile klizanja i za smičuće sile (T_s and F_s) do 6,5%. Sve vrednosti su manje od onih dobijenih analitičkim rešenjem. Poredeći modele grupe A sa rezultatima „γ-metode", nijedan od ovih modela nema veća odstupanja u apsolutnom smislu, ali može se primetiti da modeli N-POL i N-SIN daju manje vrednosti.

Analiza rezultata dobijenih primenom različitih probnih funkcija za aproksimativno rešenje pomeranja w, pokazuje da se minimalno odstupanje javlja ukoliko se usvoji funkcija oblika polinoma, a maksimalno ako se usvoji hiperbolična funkcija. Modeli w-HIP i w-SIN imaju značajna odstupanja kod napona ($\sigma_{1,b}$ and $\sigma_{2,t}$) i to čak do 41%, a za sile klizanja i za smičuće sile (T_s and F_s) čak i do 28%. Model w-HIP pokazuje znatno manje

The floor structure consists of a glulam beams that are coupled with concrete slab by vertically arranged dowel type fasteners. In this paper, the slip modulus K is determined by the Gelfi model[6]. The floor structure is loaded by the self-weight of the structural elements g, by additional permanent load d_g, as well as by the imposed load p. It is considered that the timber glulam beams will be supported in the stage of pouring and hardening of the concrete slab, and the composite section will receive imposed and total permanent load. It is possible to analyze the part of the composite floor structure separately (glulam beam with the effective width of the concrete slab), because in analyzed TCC floor system all concrete slabs are one-way and glulam beams are simply supported with uniformly distributed load. Numerical analysis according to Galerkin and Ritz method of TCC structure was performed and several subprograms/codes are written in MATLAB 2014 [15]. The simplified "γ-procedure" was also performed in order to obtain the referent values suggested by Eurocode.

5.2 Numerical analysis by Galerkin method

For the purpose of numerical analysis, two groups of models are defined on the basis of selection of trial functions and integral formulation (eqs. 13 or 14) for axial force $N(x)$ ordeflection $w(x)$:Models of Group A (N-HIP, N-SIN, N-POL) and Models of Group B (w-HIP, w-SIN, w-POL).

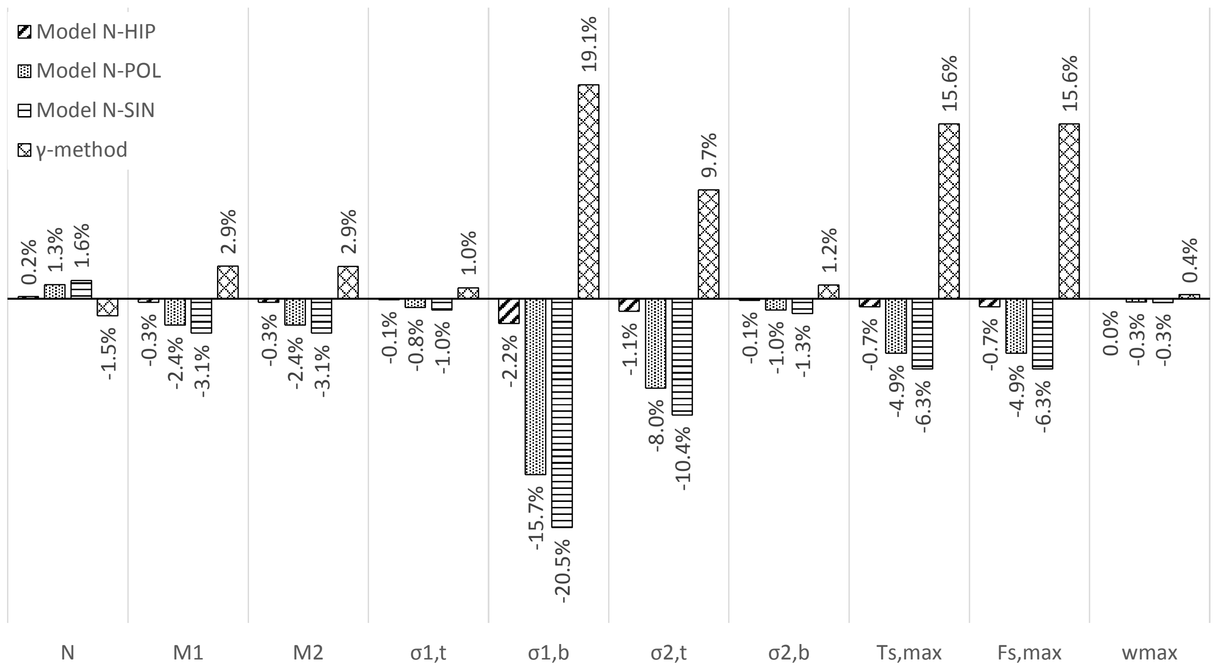
Vertical displacements (w), moments (M_1, M_2), axial forces (N_1, N_2) and stresses (σ_1 i σ_2) for the cross-section of concrete / timber element (top and bottom) in the middle of the beam span were calculated, as well as shear forces (F_s) in connectors and slip force (T_s) values at the concrete-timber contact in support zones. Results of performed numerical analysis for Models of group A and B were compared with analytical solution and their percentage deviations are shown at Figures 5,6. Results of simplified "γ-procedure" were also compared with analytical solution.

From Figs. 5 and 6, it can be noticed that numerical results of group A models have smaller differences in relation to the analytical solution than the models of group B.

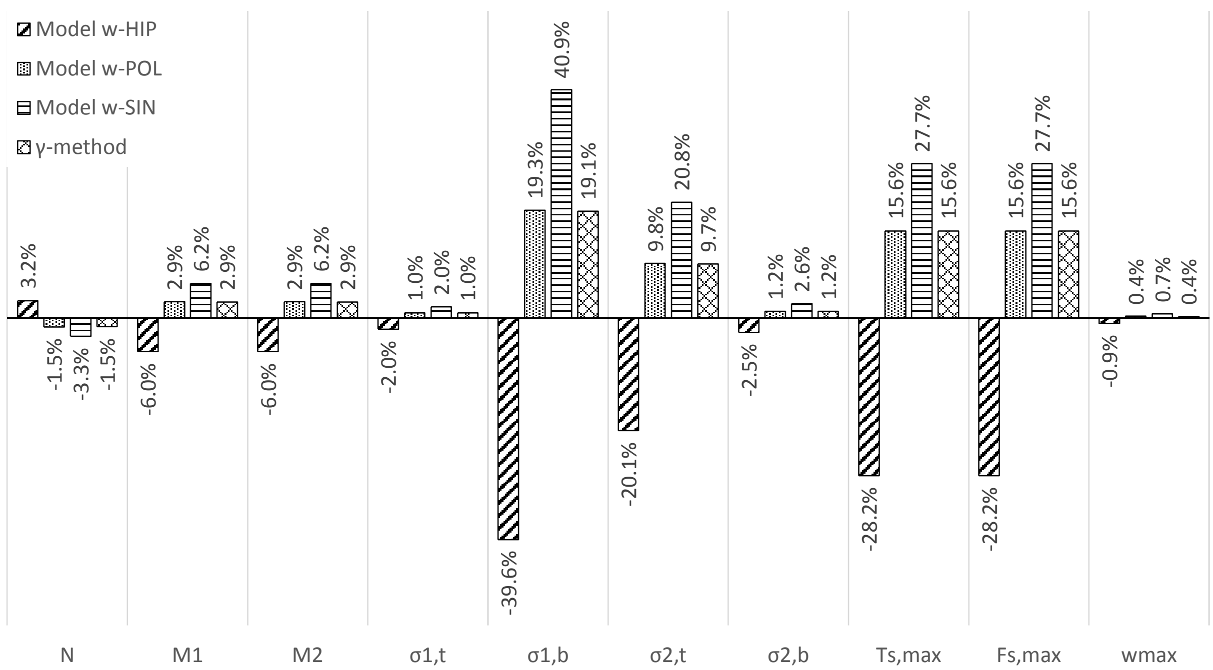
Analysis of results obtained by different trial functions for the approximate solution of the normal force N shows that a minimal deviation occurs when a hyperbolic function is adopted, and the maximum one if it is the sinusoidal function. The models N-POL and N-SIN have significant deviations in stresses ($\sigma_{1,b}$ and $\sigma_{2,t}$) even up to 21%, and minor deviations in the slip and shear forces (T_s and F_s) up to 6.5%. All values are smaller than those obtained by analytical solutions. Comparing the Group A models with results of the "γ" method, none of the models has greater deviations in absolute sense, but it can be noticed that the N-POL and N-SIN models give smaller values.

Analysis of results obtained by different trial functions for the approximate displacement w solution shows that a minimal deviation occurs if the polynomial function is adopted, and the maximum one if it is a hyperbolic function. The w-HIP and w-SIN models have significant deviations in stresses ($\sigma_{1,b}$ and $\sigma_{2,t}$) even up to 41%, and for slip and shear forces (T_s and F_s) even up to 28%. Model w-HIP shows significantly smaller values vrednosti u poređenju sa analitičkim rešenjem. Poredeći modele grupe B s rezultatima „γ-metode”, može se uočiti je da najbolje podudaranje sa „γ-metodom” ima model w-POL.

comparing to analytical. By comparing the models of group B with results of the "γ-method", it can be seen that the best match with the "γ-method" has the w-POL model.



Slika 5. Procentualna odstupanja modela grupe A u odnosu na analitičko rešenje
Figure 5. Percentage deviations of models of group A in relation to the analytical solution



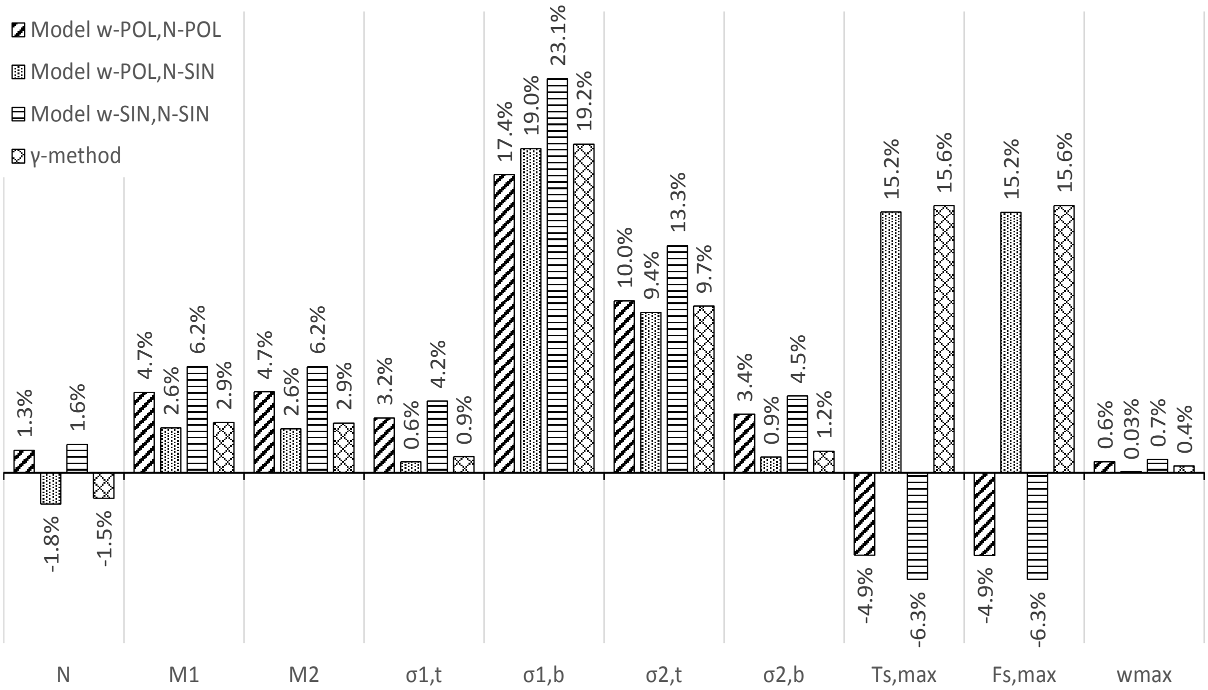
Slika 6. Procentualna odstupanja modela grupe B u odnosu na analitičko rešenje
Figure 6. Percentage deviations of models of group B in relation to the analytical solution

5.3 Numerička analiza primenom Ritz-ove metode

Za potrebe numeričke analize, definisana je grupa modela na osnovu istovremenog izbora dve probne funkcije za aksijalnu silu *N(x)* i ugib *w(x)*: Modeli grupe C (w-POL,N-POL; w-POL,N-SIN, w-SIN,N-SIN). Svi uticaji analizirani Galerkinovom metodom sračunati su i prema Ritz-u, a njihova procentualna odstupanja – u odnosu na analitičko rešenje – prikazana su na slici 7. Rezultati pojednostavljenog „*γ*-postupka" takođe su prikazani i upoređeni sa analitičkim rešenjem.

5.3 Numerical analysis by Ritz method

For the purpose of numerical analysis, the group of models is defined on the basis of simultaneous selection of two trial functions for axial force *N(x)* anddeflection *w(x)*: Models of Group C (w-POL,N-POL; w-POL,N-SIN; w-SIN,N-SIN). All effects analyzed by Galerkin method are calculated according Ritz as well and their percentage deviations compared to analytical solution are presented on Fig 7. Results of simplified "*γ*-procedure" were also compared with analytical solution.



Slika 7. Procentualna odstupanja modela grupe C u odnosu na analitičko rešenje
Figure 7. Percentage deviations of models of group C in relation to the analytical solution

Na slici 7, može se primetiti da minimalno odstupanje od analitičkog rešenja pokazuje varijantni model (w-POL,N-SIN), dok se maksimalna odstupanja javljaju kod varijantnog modela (w-SIN,N-SIN). Rezultati određeni varijantnim modelom (w-POL,N-SIN) jesu na strani sigurnosti, jer daju neznatno veće vrednosti (do 3%) za unutrašnje sile i pomeranja, dok odstupanja za normalne napone i sile klizanja / smičuće sile, dostižu vrednosti do 19% i 15% respektivno, u odnosu na analitičko rešenje. Razlog za takvo povećanje leži u činjenici da su normalni naponi i sile klizanja / smičuće sile izvedene veličine osnovnih nepoznatih *w(x)* i *N(x)*, pa su kumulativne greške veće. Očigledno je da izbor probnih funkcija za *w(x)* i *N(x)* ima značajan uticaj na konačni rezultat, kao i na izvedene statičke veličine. Takođe, može se primetiti da model (w-POL,N-SIN) ima najbolje poklapanje s pojednostavljenim „*γ*-metodom", predloženim u EN 1995. Iako varijantni model (w-POL,N-POL) daje manja odstupanja od varijantnog modela (w-SIN,N-SIN), poredeći ih sa analitičkim rešenjem, može se uočiti da dobijene vrednosti precenjuju ili potcenjuju analitičke, te ovi modeli nisu na strani sigurnosti.

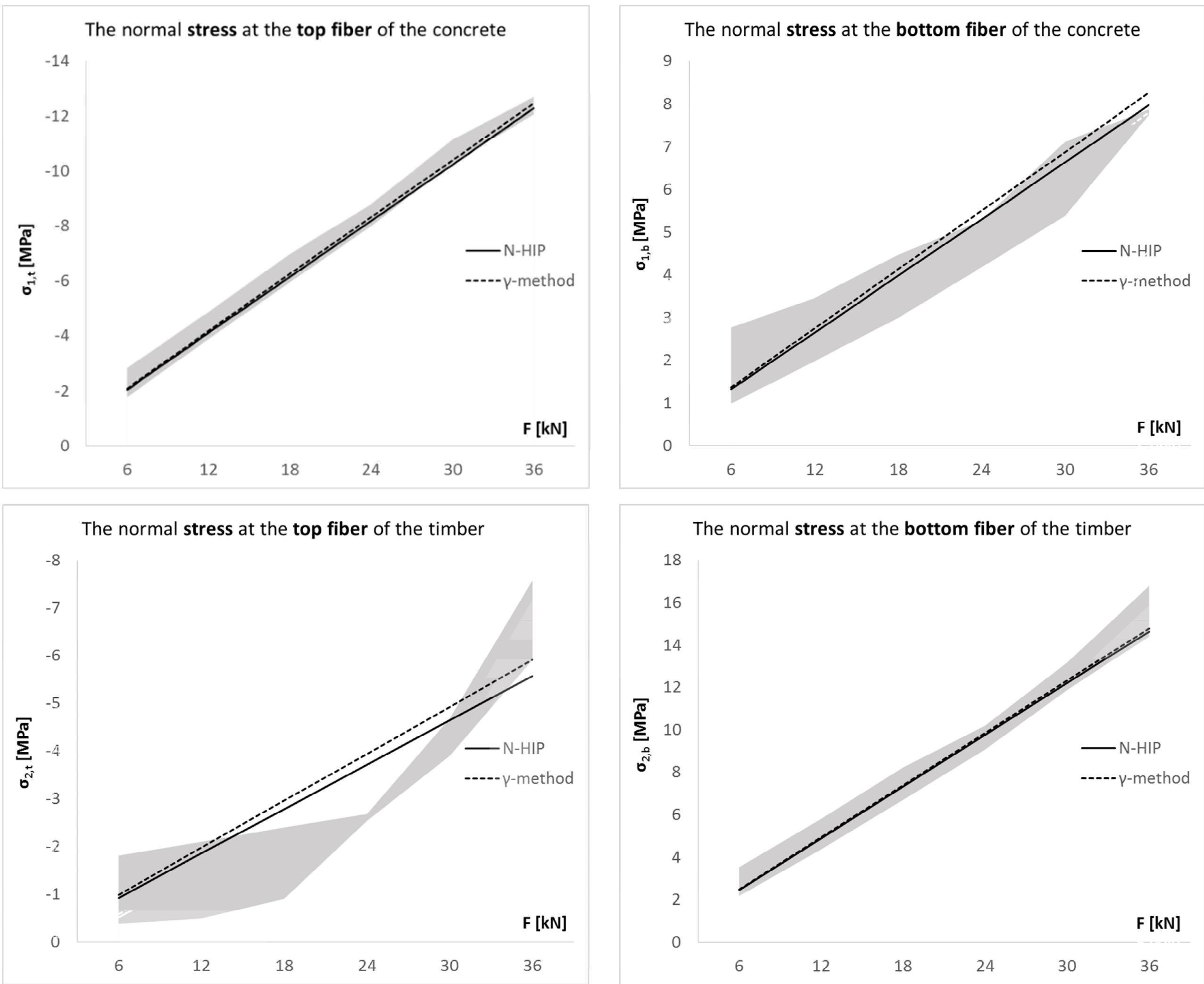
From Fig. 7, it can be noticed that the minimum deviation from the analytical solution shows the variant model (w-POL,N-SIN), while the max deviation occurs in variant model (w-SIN,N-SIN). The results obtained by variant model (w-POL,N-SIN) are on the safe side because they give a slightly higher values (up to 3%) for internal forces and displacements, while deviations for normal stresses and slip/ shear forces, arise up to 19% and 15% respectively, comparing to analytical solution. The reason for such increase lays in the fact that normal stresses and slip/ shear forces are derived values from baseline unknowns *w(x)* and *N(x),* so the cumulative errors are higher. It is obvious that the selection of trial functions for *w(x)* and *N(x)* has the significant impact on final result, as well as on derived statical values. It can be also noted that model (w-POL,N-SIN) has the best match with the approximate "γ-method" proposed in EN 1995. Although variant model (w-POL,N-POL) shows smaller deviations from variant model (w-SIN,N-SIN), comparing these two models with analytical solutions it can be seen that obtained values overestimate or underestimate analytical ones, but both models are not on the safe side.

5.4 Potvrda Galerkinove metode

Radi provere Galerkinove metode u analizi SDB konstrukcija, upoređene su dobijene numeričke vrednosti sa eksperimentalnim podacima. Kako se pokazalo da N-HIP model na najbolji način opisuje problem SDB, ovaj model izabran je za komparativnu analizu sa eksperimentalnim rezultatima SDB greda (EP1 and EP2) [18], gde su upotrebljena mehanička spojna sredstva. Dijagrami na slici 8 predstavljaju napone u poprečnom preseku konstitutivnih elemenata SDB greda za presek u sredini raspona, u odnosu na intenzitet sile tokom faza nanošenja opterećenja (F = 6, 12, 18, 24, 30 i 36 kN). Osenčene površine predstavljaju anvelopu eksperimentalno dobijenih rezultata za grede EP1 i EP2, dok pune i isprekidane linije predstavljaju rezultate numeričke analize pomoću N-HIP modela i „*γ*-metode”, respektivno. Razmatrajući napone ($\sigma_{1,b}$ and $\sigma_{2,t}$) na kontaktu dva materijala, znatno odstupanje od eksperimentalnih rezultata može se primetiti za napon $\sigma_{2,t}$, pri opterećenjima F = 18 i 24 kN. Primetno je odlično poklapanje numeričkih vrednosti sa eksperimentalnim rezultatima kod gornjeg i donjeg vlakna spregnutog poprečnog preseka, kao i donjeg vlakna u betonskoj ploči.

5.4 Verification of Galerkin method

In order to verify the application of Galerkin's method in TCC system, the comparison of numerical obtained values and experimental data was done. As it was shown that the N-HIP modeldescribes the problem of TCC partial composite action on the best way, this model was applied for comparative analysis with experimental test results of TCC (EP1 and EP2) beams [18], where mechanical fasteners were used. The diagrams in Figure 8 show the stresses in cross-section of constitutive elements of TCC beams in the middle of the span, in relation to force intensity throughout experimental loading phases (F = 6, 12, 18, 24, 30 and 36 kN). The shaded surfaces represent the envelopes of the results obtained from experiments for beams EP1 and EP2, while the full and dashed lines represent the results of the numerical analysis by N-HIP model and by "*γ*-method" respectively. Considering the stresses ($\sigma_{1,b}$ and $\sigma_{2,t}$) on the contact of two materials, deviations from the experimental results can be noticed, significantly for stress$\sigma_{2,t}$ at loads F = 18 and 24 kN. A good match with experimental results on the top and bottom sides of the cross-section as well as in bottom of the concrete slab is obvious.



Slika 8. Uporedna analiza rezultata dobijenih N-HIP modelom i eksperimentalnih podataka
Figure 8. Comparative analysis of numerical model N-HIP and experimental data

6 ZAKLJUČAK

Na osnovu predstavljenih analiza primenom Galerkinove i Ritz-ove metode, može se zaključiti da izbor probnih funkcija u formulaciji problema ima najveći uticaj na konačne vrednosti dobijenih i prikazanih rezultata. U Galerkinovoj metodi, značajan uticaj ima i sam izbor osnovnih nepoznatih (*w* i *N*). Takođe, kvalitativno poznavanje prirode rešenja može značajno doprineti smanjenju odstupanja rezultata (greške) od analitičkog rešenja. U prikazanoj analizi primenom Galerkinove metode, predloženi N-HIP model smatra se najpogodnijim za rešavanje problema elastičnog sprezanja. Kada se primenjuje varijaciona formulacija, funkcional može biti definisan putem jedne promenljive ili više njih (sila/pomeranje), dok će nepoznate koje su izabrane za osnovne biti određene s većom tačnošću od ostalih izvedenih veličina.

Kako su metode reziduuma ili varijacione metode prilično uobičajeni oblik formulacije u MKE, predstavljene metode Galerkina i Ritz-a mogu se uspešno primeniti prilikom definisanja spregnutog KE za elastično spregnute konstrukcije drvo–beton, a time se dobija efikasan inženjerski alat u praksi. Uvođenjem Evrokoda za spregnute sisteme drvo–beton [4], očekuje se preciznije definisanje osnovnog ulaznog parametra za različita sredstva spajanja – modula pomerljivosti spojnog sredstva *K*, a time i realniji odgovor spregnutih nosača u numeričkim analizama.

6 CONCLUSION

Based on the presented analysis using Galerkin's method and Ritz method, it can be concluded that the selection of trial functions in problem formulation has the major influence on the final effect values of obtained and presented results. An important influence is also the choice of baseline unknowns (*w* and *N*) for Galerkin's method. Also, the qualitative knowledge of solution nature can significantly contribute tothe reduction of errors in obtained results related to analytical solution. In presented analysis by Galerkin's method the proposed N-HIP model qualifies as the most appropriate in order to solve the problem of partial coupling. When using a variational formulation, functional could be defined trough one or more unknowns (forces/displacements), while the unknowns that are chosen as basic will be determined with more accuracy than the derived ones.

As the weighted residual method or the variation formulation in the FEM is quite usual form, presented the Galerkin and Ritz methods could be successfully applied when defining a composite FE for partially timber-concrete composite systems, thereby enabling an efficient engineering tool in practice. With introduction of Eurocode for timber-concrete composite structures [4], it is expected that more precise definition of basic input parameter - slip modulus for different types of fasteners *K*, will contribute to the more realistic response of composite beams in numerical analysis.

ZAHVALNOST

Ovaj rad proistekao je iz istraživačkog projekta TR 36043 „Razvoj i primena sveobuhvatnog pristupa u projektovanju novih i proceni sigurnosti postojećih konstrukcija u cilju smanjenja seizmičkog rizika u Srbiji", koji je finansiralo Ministarstvo nauke Republike Srbije.

ACKNOWLEDGMENTS

The work has been done within the scientific research project TR 36043 "Development and application of a comprehensive approach to the design of new and safety assessment of existing structures for seismic risk reduction in Serbia", which is funded by the Ministry of Science of Serbia.

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REZIME

PRIMENA NUMERIČKIH METODA U ANALIZI SPREGNUTIH KONSTRUKCIJA DRVO-BETON

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Analiza i proračun spregnutih drvo–beton (SDB) konstrukcija, gde je veza konstitutivnih elemenata ostvarena mehaničkim spojnim sredstvima, predstavlja kompleksan zadatak zbog uzimanja u obzir pomerljivosti sredstava za sprezanje tj. klizanja na kontaktu dva materijala. Primena pojednostavljenih postupaka i metoda u analizi SDB nosača predstavlja pogodan i poželjan način proračuna, koji inženjerima u praksi omogućava efikasan alat. Široko rasprostranjen, pojednostavljen proračun tzv. γ-metod dat je u EN 1995. Metode zasnovane na diferencijalnoj ili varijacionoj formulaciji često su u upotrebi kada su u pitanju programi za strukturalnu analizu konstrukcija. U radu je prikazana Galerkin-ova i Ritz-ova metoda za analizu i proračun SDB nosača za slučaj proste grede izložene raspodeljenom opterećenju. Analiziran je izbor probnih funkcija koje opisuju problem elastičnog sprezanja, kao i njihov uticaj na konačne rezultate. Za potrebe numeričke analize, na osnovu predloženih numeričkih modela, napisani su kodovi u MATLAB-u. Model primenjen u analizi Galerkinovom metodom, koji najbolje opisuje problem elastičnog sprezanja, izabran je za komparativnu analizu sa eksperimentalnim podacima.

Ključne reči: Ritz-ova metoda, Galerkinova metoda, γ-metod, slaba formulacija, numerička analiza, MKE, sprezanje drvo–beton, elastično sprezanje, klizanje u spoju.

SUMMARY

APPLICATION OF NUMERICAL METHODS IN ANALYSIS OF TIMBER CONCRETE COMPOSITE SYSTEM

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The analysis and design of composite timber-concrete (TCC) structures, where the connection of the constituent elements is achieved by dowel type fasteners, is a complex task due to taking into account the slip of the coupling means, i.e. interlayer slip on the contact surface of two materials. The application of simplified procedures and methods in the analysis of the TCC system is a convenient and desirable way of design that enables efficient tool for engineering practice. Widespread simplified calculation procedure, so called "γ-method", is adopted in EN 1995. Methods based on differential or variational formulation are commonly applied when software for structural analysis are used. Galerkin's and Ritz's methods for analysis and design of TCC systems in the case of simply supported beam loaded with uniformly distributed load are shown in this paper. The selection of trial functions that describe the problem of elastic composite action as well as their influence on the final results were analyzed. For the purposes of numerical analysis, based on the proposed numerical models, several codes are written in MATLAB. The model applied in analysis by Galerkin's method, that best describes the problem of elastic coupling, was chosen for further comparative analysis with experimental data.

Key words: Ritz's method, Galerkin's method, γ-method, Weak form, Numerical analysis, FEM, Timber-concrete composite, Partial interaction, Interlayer slip.

ELASTIČNA KRITIČNA SILA PLOČA I LIMENIH NOSAČA POD DEJSTVOM LOKALIZOVANOG OPTEREĆENJA

ELASTIC CRITICAL LOAD OF PLATES AND PLATE GIRDERS SUBJECTED TO PATCH LOAD

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ORIGINALNI NAUČNI RAD
ORIGINAL SCIENTIFIC PAPER
UDK:624.073.44
doi:10.5937/GRMK1804055J

1 UVOD

Poslednjih decenija problemi stabilnosti i postkritičnog ponašanja limenih nosača značajno privlače pažnju. Intenzivno je istraživano ponašanje limenih nosača (zavarenih I poprečnih preseka) pri dejstvu lokalizovanog opterećenja, odnosno pod uticajem delimično raspodeljenog opterećenja po nožici, i to na mestima gde neposredno ispod zadatog opterećenja ne postoje vertikalna (poprečna) ukrućenja. Važnost ovog problema porasla je sa opštim trendom izbegavanja vertikalnih ukrućenja izuzimajući preseke iznad oslonaca, i u slučaju postojanja pokretnog opterećenja. Sem kranskih nosača opterećenih točkovima krana, važan primer ovakvog opterećenja odnosi se na prevlačenje konstrukcije preko privremenih ili stalnih oslonaca, tokom montaže kontinualnih čeličnih mostova.

Analiza ponašanja nosača pri dejstvu lokalizovanog opterećenja obuhvata određivanje raspodele lokalnih normalnih napona u rebru nosača, određivanje elastične kritične sile izbočavanja i određivanje granične nosivosti nosača. Granična nosivost igra bitnu ulogu pri proračunu. Naučnici iz mnogih zemalja bavili su se teorijskim i eksperimentalnim istraživanjima ovog problema tokom

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1 INTRODUCTION

The stability problems and ultimate load behaviour of steel plate girders have attracted a lot of attention during the last decades. The behaviour of the plate girder (welded I-girder) subjected to patch load or partially distributed load on the flange in the plane of a web, without a vertical (transverse) stiffener bellow the load was also intensively investigated. This problem has got the importance with a general trend to avoid vertical stiffeners except at supports and also in the case of moving loads. Except for crane girders loaded by crane wheels, a remarkable realistic load case in which this situation arises is the launching phase of multi-span steel plate girder bridges during construction over temporary or permanent supports.

In the analysis of the behaviour of the patch loaded girder the attention is directed towards the distribution of the local direct stresses under the load in the web, the elastic critical load of the web panel and the ultimate load of the girder. For the development of design procedures, the ultimate load is of a great importance. The research workers in many countries have investigated theoretically and experimentally this problem over

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poslednjih pedeset godina. Međutim, i značajan broj eksperimenata i složenog teoretskog rada nije pružio potpun uvid u rešenje problema, tako da su i dalje širom sveta u toku značajna istraživanja različitih segmenata ove oblasti.

U početnim fazama istraživanja, ulagani su napori da se granična nosivost dovede u vezu sa elastičnom kritičnom silom izbočavanja [8], [5], [15]. Međutim, vrlo brzo je utvrđeno da ovo u opštem slučaju nije moguće. Takođe, ni analitički postupci nisu našli odgovarajuću primenu, s obzirom na kompleksnost problema i veliki broj međusobno zavisnih promenljivih. Kao posledica navedenog, većina predloženih rešenja je empirijske prirode, zasnovana na eksperimentalnim istraživanjima. S razvojem kompjuterskih programa koji se zasnivaju na metodi konačnih elemenata (MKE) i povećanim mogućnostima samih računara, omogućen je razvoj „numeričkih eksperimenata" koji su danas u širokoj upotrebi.

S razvojem Evrokodova za proračun građevinskih konstrukcija, primenjen je novi pristup pri proračunu lokalizovanog opterećenja, u skladu s rešenjima datim za druge probleme stabilnosti. Glavni elementi pri proračunu graničnog lokalizovanog opterećenja jesu elastična kritična sila, opterećenje pri kom dolazi do plastifikacije i koeficijent redukcije. Elastična kritična sila (izbočavanja) dobila je ponovo značaj s obzirom na to što se koristi pri određivanju relativne vitkosti $\overline{l}_F$ prema sledećem izrazu [3]:

the last fifty years. Even an increasing number of experimental results and laborious theoretical work has not offered a complete insight into the problem and intensive researches in various segments of this problem are still ongoing worldwide.

In the initial phases of investigations, attempts were made to relate the ultimate load to the elastic critical (buckling) load [8], [5], [15]. Very soon it was found that generally, it was not possible. Also, analytical procedures were not successfully applied, as the problem is very complex, depending on many interrelated parameters. As a corollary, a majority of the suggested solutions were of empirical nature, based on experimental researches. The development of computer programs based on the finite element (FE) method and increased capabilities of computers enabled the development of "numerical experiments" that are nowadays widely applied.

With the development of Eurocodes for the design of civil engineering structures, a new approach was applied to harmonize solutions of the patch load problem with other stability problems. The main elements in the determination of the ultimate patch load are elastic critical load, yield load, and reduction factor. The determination of the elastic critical (buckling) load got again the importance, as it is needed for the determination of the nondimensional slenderness $\overline{l}_F$ according to the following equation [3]:

$$\overline{l}_F = \sqrt{\frac{l_y t_w f_{yw}}{F_{cr}}} \tag{1}$$

gde su:

l_y – efektivna opterećena dužina,
t_w – debljina rebra,
f_{yw} – granica razvlačenja rebra,
F_{cr} – kritična sila izbočavanja, definisana kao:

where are:

l_y – the effective loaded length,
t_w – the web thickness,
f_{yw} – the yield strength of the web,
F_{cr} – the critical buckling force, defined as:

$$F_{cr} = 0.9 k_F E \frac{t_w^3}{h_w} \tag{2}$$

gde su:

k_F – koeficijent izbočavanja,
E – modul elastičnosti,
h_w – visina rebra.

where are:

k_F – the buckling coefficient,
E – the modulus of elasticity,
h_w – the web depth.

Rešenja primenjena u Evrokodu 3 za proračun čeličnih konstrukcija [3] koriste uprošćene izraze za određivanje koeficijenata izbočavanja. Na primer, za limeni nosač na kom je poprečna sila aplicirana na gornjoj nožici, na mestu između dva vertikalna ukrućenja, koja su na međusobnom rastojanju *a*, koeficijent izbočavanja treba odrediti prema jednačini 3:

The solution applied in Eurocode 3 for design of steel structures [3] uses simplified expressions for the buckling coefficients. For example, for a plate girder under patch load applied on the top flange, between two vertical stiffeners on a distance *a*, the buckling coefficient should be obtained using Eq. 3:

$$k_F = 6 + 2\left(\frac{h_w}{a}\right)^2 \tag{3}$$

Kao što se može primetiti, izraz iz jednačine 3 ne uzima u obzir dužinu na kojoj je aplicirano poprečno opterećenje. Naveden izraz za određivanje koeficijenta izbočavanja usvojen je uprošćenjem izraza koji je predložio Lagerkvist [8]:

As it could be observed, the given expression in Eq. 3 does not account the distribution length of an applied patch load. This expression for determination of buckling coefficient is adopted by simplifying the expression proposed by Lagerqvist [8]:

$$k_F = \left(1+\frac{s_s}{2h_w}\right)\left(5.3+1.9\left(\frac{h_w}{a}\right)^2+0.4\sqrt[4]{b}\right),\ b=\frac{b_f \cdot t_f{}^3}{h_w \cdot t_w{}^3} \tag{4}$$

gde su:

s_s – dužina zadatog opterećenja,

b_f – širina nožice,

t_f – debljina nožice.

Za razliku od jednačine 3, izraz 4 uključuje odnos krutosti nožice i rebra, kao i odnos između dužine zadatog opterećenja i visine rebra.

Tokom osamdesetih godina prošlog veka započeta su istraživanja lokalizovanog opterećenja na Građevinskom fakultetu Univerziteta u Beogradu u okviru internacionalne saradnje sa Univerzitetom u Kardifu (sa T. M. Robertsom) i sa Češkom akademijom nauka u Pragu (sa M. Škaloudom). U kasnijem periodu, saradnja je nastavljena s Građevinskim fakultetom Univerziteta Crne Gore u Podgorici (sa D. Lučićem i njegovim saradnicima), gde su takođe sprovedena intenzivna istraživanja [2]. Postignuti rezultati eksperimentalnih ispitivanja [9] upoređeni s prediktivnim vrednostima koje daje Evrokod 3 pokazali su [10] da standard predviđa izrazito konzervativne rezultate. Ovaj rad je pokušaj da se daju izvesna poboljšanja navedenih procedura.

Elastična kritična sila izolovanog rebra nosača upoređena je sa elastičnim kritičnim opterećenjem za I-nosač. Zatim su eksperimentalno određena granična opterećenja upoređena s rezultatima koje predviđa Evrokod 3, kao i s modifikovanim procedurama koje obuhvataju predloženu korekciju za određivanje elastične kritične sile uzimajući u obzir dužinu rasprostiranja zadatog lokalizovanog opterećenja.

Danas su tehnike numeričke analize u širokoj upotrebi u istraživanjima, kao i u analizi i proračunu čeličnih konstrukcija. Numeričke analize zasnovane na prinicipu primene MKE najčešće su primenjivan alat u ovoj oblasti i uspešno su korišćene u mnogim radovima koji se tiču određivanja kritične sile izbočavanja limenih nosača pod dejstvima različitih opterećenja. U ovoj numeričkoj analizi korišćen je komercijalni program *Abaqus* [1]. Radi sticanja boljeg uvida i poređenja rezultata, geometrija i opterećenje nosača modelirani su u skladu sa onim primenjenim u eksperimentu [9].

2 NUMERIČKA SIMULACIJA

Kao što je prethodno navedeno, cilj ovog rada je poređenje elastične kritične sile izolovanog rebra nosača variranih uslova oslanjanja, s kritičnom silom izbočavanja I-nosača. Analizirane su dve ploče šematski prikazane na slici 1, označene kao *Model 1* sa odnosom dimenzija a/h_w = 1 i *Model 2* sa odnosom dimenzija a/h_w = 2 (a = 1000 mm). Debljina rebra, tj. ploče je t_w = 4 mm. Dužina zadatog raspodeljenog opterećenja s_s je varirana. Stepen slobode 2 sprečen je duž vertikalnih ivica, dok je stepen slobode 3 sprečen duž sve četiri ivice. Analizirana su tri različita slučaja:

(a) slobodno oslonjenja ploča bez dodatnih sprečenih pomeranja,

(b) ploča slobodno oslonjena duž vertikalnih i

where are:

s_s – the patch load length,

b_f – the flange width,

t_f – the flange thickness.

Unlike Eq. 3, expression 4 includes a ratio between flange and web stiffness and a ratio between load length and web depth.

The investigation of patch load has started at the Faculty of Civil Engineering University of Belgrade initially in the eighties within an international cooperation with the University of Cardiff (with T. M. Roberts) and with the Czech Academy of Sciences in Prague (with M. Skaloud). In the last period, the collaboration was continued with the Faculty of Civil Engineering University of Montenegro in Podgorica (with D. Lučić and his collaborators), where intensive researches were also carried out [2]. The results obtained in the experimental research [9] compared with the procedures applied in Eurocode 3 had shown [10] significantly conservative results given by Eurocode 3. An attempt is made in this paper to give some improvements to that procedure.

Elastic critical load of an isolated web plate for different boundary conditions is compared with an elastic critical load of an I-girder. Moreover, the experimentally determined ultimate load is compared with the procedure given in Eurocode 3, as well as with the modified Eurocode procedure including the suggested correction for determination of the elastic critical load accounting the distribution length of an applied patch load.

Nowadays, numerical analysis techniques are widely used in research involving structural steel and in analyses and designs of steel structures and elements. The FE method based numerical analysis is the most popular computational tool in this field and it has been successfully applied in many papers regarding the critical load of plate girders under different loading conditions. The commercial multi-purpose FE analysis software Abaqus was used for the numerical analysis [1]. In order to get a better insight into the problem, geometry and loading of the girders were considered according to those applied in the accompanying experimental research [9].

2 NUMERICAL SIMULATION

Introductory it was stated that the purpose of this paper is the comparison of the elastic critical loads for an isolated web plate, varying boundary conditions, with critical loads of an I-girder. The two plates schematically presented in Fig. 1, labelled as *Model 1* with an aspect ratio a/h_w = 1 and *Model 2* with an aspect ratio a/h_w = 2 (a = 1000 mm), were investigated. The web plate thickness is t_w = 4 mm. The length of an applied uniform load s_s was varied. The degree of freedom 2 is only constrained in the vertical edges while the degree of freedom 3 is constrained in all edges. Three different cases were analysed:

(a) simply supported plate with no additional constraints, ukliještena duž horizontalnih ivica (sprečen je stepen slobode 4),

(c) ploča ukliještena duž svih ivica (duž vertikalnih ivica sprečen je stepen slobode 5, a duž horizontalnih, stepen slobode 4).

Radi jednostavnosti, navedeni slučajevi su u nastavku rada označeni kao SS, CS i CC, respektivno.

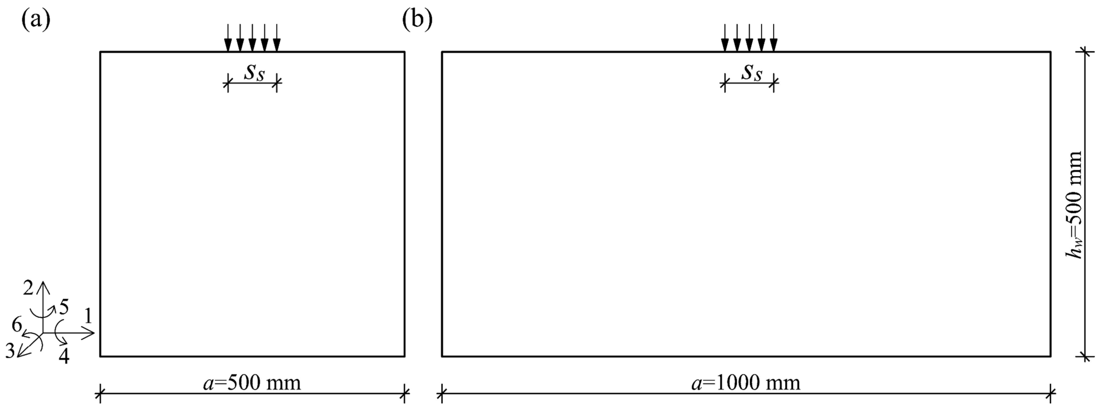
Analogno prethodno opisanim Modelima 1 i 2, formirana su dva modela I-nosača prikazana na slici 2. Širina i debljina nožice su 120 mm i 8 mm, respektivno. Granični uslovi zadati su u skladu s postavkom eksperimenta opisanom u [9]. Razmatrani materijal je homogen i elastičan sa zadatim modulom elastičnosti E = 210 GPa i Poasonovim koeficijentom v = 0.3.

(b) plate simply supported on the vertical and clamped on the horizontal edges (degree of freedom 4 is constrained),

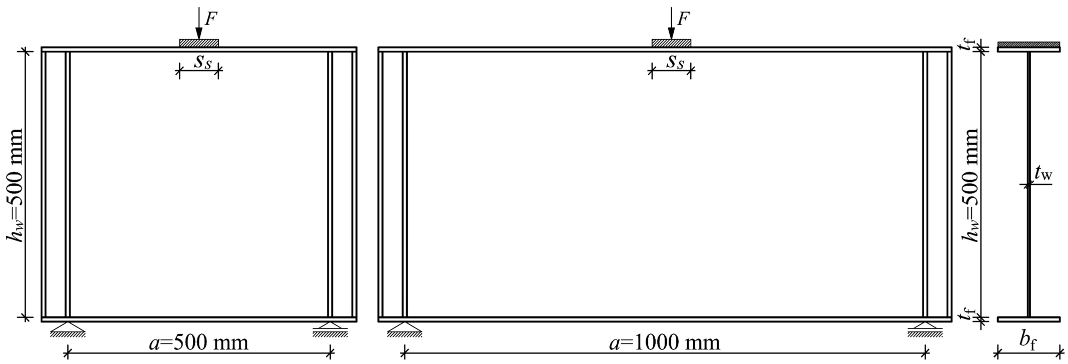
(c) plate clamped along all edges (degree of freedom 5 is constrained in the vertical edges and the degree of freedom 4 in the horizontal edges).

For the sake of brevity, through the rest of the paper, these cases will be marked as SS, CS, and CC, respectively.

In the same vein, the two models of an I–girder according to the previously described Model 1 and Model 2 are displayed in Fig. 2. The flange width and thickness were set to 120 mm and 8 mm, respectively. The boundary conditions are according to the experimental procedure described in [9]. The considered material for all cases is homogenous with an elastic modulus of E = 210 GPa and Poisson's ratio of v = 0.3.



Slika 1. Ploča pod lokalizovanim opterećenjem: (a) Model 1; (b) Model 2
Figure 1. Plate under patch load: (a) Model 1; (b) Model 2



Slika 2. Model 1 i Model 2 I-nosača
Figure 2. Model 1 and Model 2 for I–girders

U analizi je korišćen opštenamenski četvorostrani površinski konačni element S4R iz baze elemenata date u programu *Abaqus*, sa četiri čvora, s redukovanom integracijom i sa po 6 stepeni slobode u svakom čvoru. Nakon analize konvergencije mreže konačnih elemenata, izvršene na izolovanoj ploči Modela 1, usvojena je jedinstvena mreža veličine 5 mm za sve numeričke modele. Promena koeficijenta izbočavanja pri promeni veličine konačnih elemenata slobodno oslonjene ploče Modela 1, grafički je prikazana na slici 3(a).

For the FE analysis, a general-purpose four-node quadrilateral shell element with reduced integration and six degrees of freedom per node S4R from the Abaqus element library was used. According to the mesh convergence study performed on the isolated web plate of Model l, the adopted finite element size is 5 mm for all numerical runs. The variation of the buckling coefficient due to change of the finite element size in the case of Model 1 for a simply supported plate is graphically presented in Fig. 3(a).

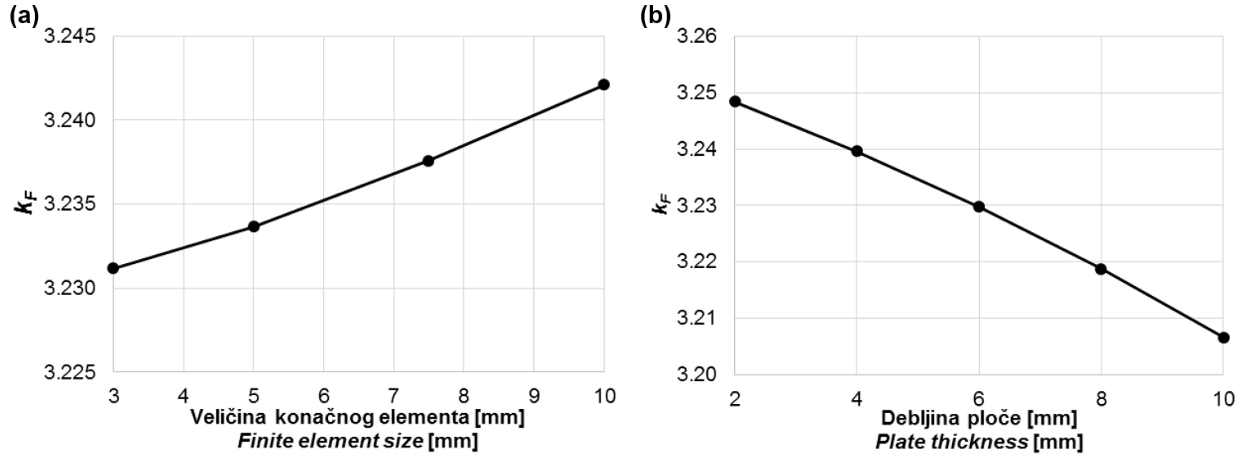
The results of the calculated elastic critical load are

Rezultati sračunate elastične kritične sile prikazani preko bezdimenzionog koeficijenta izbočavanja k_F za Model 1 i Model 2 tabelarno su prikazani u tabelama 1 i 2, respektivno. Konačno, može se zaključiti da dobijene vrednosti daju dobro podudaranje s vrednostima datim u literaturi ([4], [6], [7], [11], [12], [13], [14]).

Koeficijent izbočavanja zavisi od graničnih uslova, vrste opterećenja i odnosa a/h_w. S druge strane, ne zavisi od debljine ploče i karakteristika materijala. Međutim, za različite vrednosti debljine slobodno oslonjene ploče Modela 1, mogu se uočiti blage varijacije rezultata kao što je prikazano na slici 3(b).

presented using the non-dimensional buckling coefficient k_F and they are tabulated in Table 1 and Table 2 for Model 1 and Model 2, respectively. Conclusively, it may be stated that they show a good agreement with the corresponding values found in the literature ([4], [6], [7], [11], [12], [13], [14]).

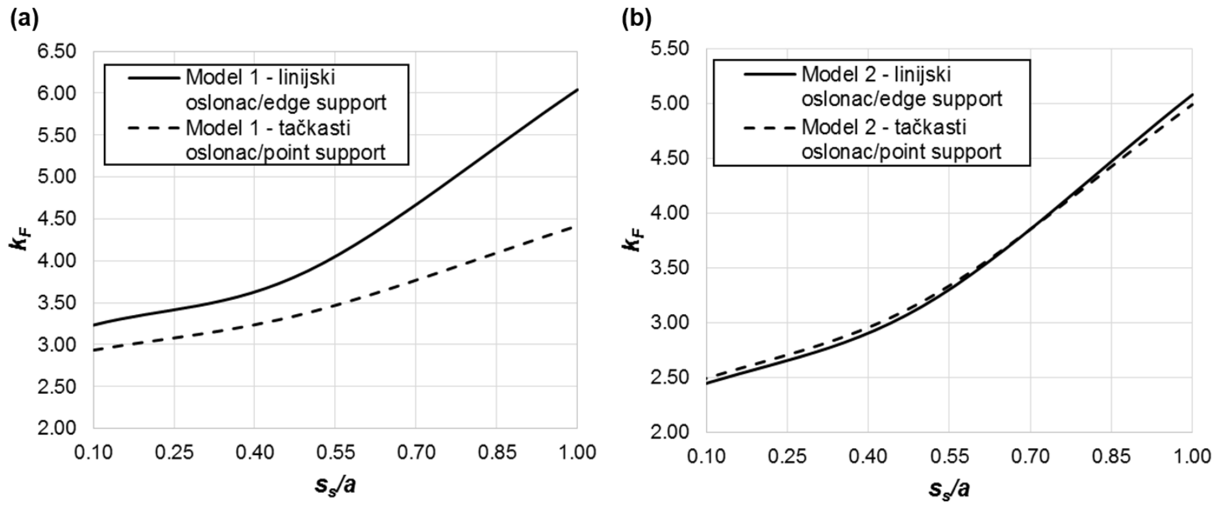
The buckling coefficient depends on boundary conditions, loading type and an aspect ratio a/h_w. On the other hand, it is not influenced by plate thickness or material properties. However, for a different plate thickness applied on a simply supported plate of Model 1, a slight variation in the results could be noticed, as charted in Fig. 3(b).



Slika 3. Variranje koeficijenta izbočavanja pri promeni: (a) veličine KE; (b) debljine ploče
Figure 3. Variation of the buckling coefficient due to change of: (a) FE size; (b) plate thickness

Druga osetljivost koeficijenta izbočavanja prikazana je na slici 4. Slobodno oslonjena ploča je analizirana kada je stepen slobode 2 sprečen duž svih vertikalnih ivica, tj. linijski oslonac je postavljen kao što je prethodno opisano, i zatim kada su umesto linijskih, zadati tačkasti oslonci u donjim uglovima ploče. U slučaju Modela 2,

Another sensitivity of the buckling coefficient is pictured in Fig. 4. A simply supported plate model is analysed when the degree of freedom 2 was constrained along the vertical edges, i.e. the edge support is set as previously described, and point supports are applied in the lower corners of the plate. In the case of Model 2,



Slika 4. Koeficijent izbočavanja za ploču sa linijskim osloncima duž vertikalnih ivica i za ploču sa tačkastim osloncima u donjim uglovima ploče
Figure 4. Buckling coefficient for the plate supported on the vertical edges and for the plate with point supports in the corners

ova promena graničnih uslova ne utiče na vrednost koeficijenta k_F. Suprotno tome, u slučaju Modela 1 i raspodeljenog opterećenja veće dužine, razlike u rezultatima dostižu i do 25%. Budući da je zadato opterećenje lokalnog karaktera, za odnose dimenzija $a/h_w \geq 2$, ovi granični uslovi ne igraju značajnu ulogu i njihov uticaj je zanemarljiv.

3 REZULTATI I DISKUSIJA

Cilj ovog dela rada je da predstavi rezultate numeričke analize u vidu koeficijenta izbočavanja za izolovano rebro nosača i za sam I-nosač. Rezultati svih numeričkih modela su opisani i o njima je diskutovano. Takođe, predstavljeni su novi izrazi za određivanje koeficijenta izbočavanja u funkciji dužine raspodeljenog lokalizovanog opterećenja. Svrha predstavljenih izraza je da dâ preciznije vrednosti granične nosivosti koristeći proceduru datu u Evrokodu 3.

U tabelama 1 i 2 dat je sumarni prikaz rezultata numeričke analize i koeficijenata izbočavanja određenih prema Evrokodu 3 (jednačina 3) i prema Lagerkvistovom izrazu (jednačina 4), dok je grafička prezentacija data kroz sliku 5. Može se primetiti da ploča SS za oba modela i za sve dužine zadatog lokalizovanog opterećenja daje ekstremno niske vrednosti kritičnog opterećenja. Sa uvođenjem uklještenja duž horizontalnih ivica (CS ploča), vrednosti koeficijenta k_F su veće, ali su i dalje ispod vrednosti k_F za I-nosače, posebno u slučaju Modela 1. Granični uslovi koji su najbliži rezultatima ponašanja I-nosača su uklještene ivice, i horizontalne i vertikalne (CC ploča). Kasnije u nastavku rada diskutovano je o vezi između koeficijenata k_F za CC ploču i za I-nosač.

Bitno je istaći i drugu interesantnu stavku u vezi s graničnim uslovima uklještenja za veće dužine s_s. Sa slike 5 može se uočiti da kruto vertikalno ukrućenje (slika 2) utiče na koeficijent izbočavanja više nego uklješten granični uslov, dok za male vrednosti s_s ovaj uticaj nije prisutan s obzirom na to što je opterećenje lokalnog karaktera. Zatim, zadato uklještenje duž vertikalnih ivica (razlika između CC i CS ploče) ukazuje na činjenicu da sprečene rotacije duž ivica ne utiču na vrednosti koeficijenta izbočavanja za dimenzije nosača $a/h_w \geq 2$.

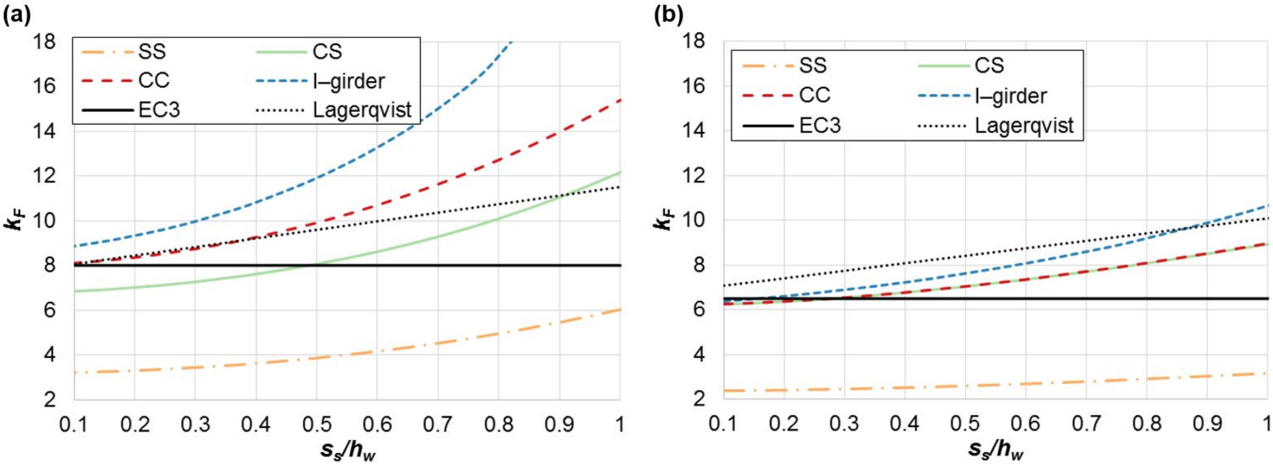
this change of boundary conditions does not affect the value of k_F. On the contrary, in the case of Model 1 and a longer uniform load, the difference in the results comes up to 25 %. Expectedly, those boundary conditions do not play a decisive role for higher aspect ratios since the applied load is localized and for an aspect ratio $a/h_w \geq 2$ their influence is negligible.

3 RESULTS AND DISCUSSION

The aim of this chapter is to highlight the obtained numerical results of the buckling coefficient considering an isolated web plate and I-girder. The results for all described numerical models are listed and discussed thoroughly. Furthermore, new expressions for the determination of the buckling coefficients, as a function of patch load length, are presented. The purpose of the proposed expressions is to improve the ultimate load calculated using the procedure given in Eurocode 3.

Table 1 and Table 2 show a comparison summary of the numerical results and the buckling coefficients obtained by Eurocode 3 (Eq. 3) and according to Lagerqvist's expression (Eq. 4), while a graphical representation is shown in Fig. 5. One can instantaneously see that the SS plate for both models and for all lengths of an applied patch load gives extremely low values of the critical loads. Introducing the clamped constraint on the horizontal edges (CS plate) the k_F values are improved but still below the k_F of the I-girder, especially for Model 1. Boundary conditions that lead to the results closest to the I-girder behaviour are clamped edges, both vertical and horizontal (CC plate). Later on in this chapter, relations between k_F values for the CC plate and for the I-girder are discussed.

It is noteworthy to observe another interesting point regarding the clamped boundary conditions for large values of s_s. It is clear from Fig. 5 that the rigid transversal stiffeners (see Fig. 2) influence the buckling coefficient more than clamped constraint whereas for small values of s_s their influence is not present since the load is localized. Additionally, the clamped constraint on the vertical edges (the difference between the CC and CS plate) reveals the fact that constrained rotations along these edges do not have an influence on the



Slika 5. Poređenje koeficijenta izbočavanja za: (a) Model 1; (b) Model 2
Figure 5. Comparison of the buckling coefficient for: (a) Model 1; (b) Model 2

Tabela 1. Koeficijent izbočavanja Modela 1 za različite granične uslove
Table 1. Buckling coefficients for different boundary conditions for Model 1

s_s [mm]	*s/a*	s/h_w	k_F					
			SS	CS	CC	I-nosač *I–girder*	EC3	Lagerkvist *Lagerqvist*
0	0.00	0.00	3.20	6.78	8.00	8.61	8.00	7.67
25	0.05	0.05	3.21	6.80	8.02	8.69	8.00	7.86
50	0.10	0.10	3.23	6.85	8.10	8.85	8.00	8.05
75	0.15	0.15	3.27	6.92	8.21	9.06	8.00	8.25
100	0.20	0.20	3.32	7.02	8.35	9.32	8.00	8.44
125	0.25	0.25	3.38	7.13	8.53	9.61	8.00	8.63
150	0.30	0.30	3.46	7.27	8.74	9.96	8.00	8.82
175	0.35	0.35	3.54	7.44	8.98	10.36	8.00	9.01
200	0.40	0.40	3.64	7.62	9.26	10.81	8.00	9.21
250	0.50	0.50	3.88	8.07	9.91	11.90	8.00	9.59
300	0.60	0.60	4.18	8.62	10.70	13.27	8.00	9.97
350	0.70	0.70	4.54	9.29	11.64	15.01	8.00	10.36
400	0.80	0.80	4.97	10.10	12.73	17.37	8.00	10.74
450	0.90	0.90	5.47	11.05	13.98	21.60	8.00	11.12
500	1.00	1.00	6.04	12.17	15.41	31.95	8.00	11.51

Tabela 2. Koeficijent izbočavanja Modela 2 za različite granične uslove
Table 2. Buckling coefficients for different boundary conditions for Model 2

s_s [mm]	*s/a*	s/h_w	k_F					
			SS	CS	CC	I-nosač *I–girder*	EC3	Lagerkvist *Lagerqvist*
50	0.05	0.10	2.37	6.24	6.25	6.38	6.50	7.06
100	0.10	0.20	2.40	6.36	6.37	6.61	6.50	7.39
150	0.15	0.30	2.45	6.54	6.54	6.90	6.50	7.73
200	0.20	0.40	2.51	6.77	6.77	7.23	6.50	8.07
250	0.25	0.50	2.59	7.04	7.04	7.63	6.50	8.40
300	0.30	0.60	2.68	7.35	7.35	8.08	6.50	8.74
350	0.35	0.70	2.78	7.70	7.71	8.60	6.50	9.07
400	0.40	0.80	2.90	8.08	8.09	9.20	6.50	9.41
500	0.50	1.00	3.15	8.93	8.96	10.65	6.50	10.08
600	0.60	1.20	3.44	9.86	9.94	12.56	6.50	10.75
700	0.70	1.40	3.78	10.85	11.03	15.17	6.50	11.43
800	0.80	1.60	4.16	11.92	12.24	19.00	6.50	12.10
900	0.90	1.80	4.60	13.11	13.56	25.20	6.50	12.77
1000	1.00	2.00	5.08	14.47	15.00	47.26	6.50	13.44

Suprotno tome, one utiču na vrednost koeficijenta izbočavanja za slučaj ploča dimenzija $a/h_w = 1$ (Model 1) za sve vrednosti s_s. Potrebno je sprovesti dodatnu parametarsku analizu kako bi se doneli zaključci o ovom graničnom uslovu i njegovom uticaju na koeficijent izbočavanja.

Sa slike 5, može se još uočiti da izraz koji daje Evrokod 3 vodi do konstantnog koeficijenta izbočavanja pri promeni dužine raspodeljenog opterećenja. Za veće dužine s_s, razlike u rezultatima prema Evrokodu 3 i onim dobijenim za I-nosač jesu značajne. S druge strane, Lagerkvistov izraz ima linearan trend rasta. Međutim, predložena jednačina ne daje dobro podudaranje s numerički dobijenim rezultatima za I-nosač, posebno u slučaju Modela 2, gde koeficijenti izbočavanja prema Lagerkvistu čak nisu na strani sigurnosti.

S obzirom na opisane nedostatke jednačina 3 i 4 pri

buckling coefficient for an aspect ratio $a/h_w \geq 2$, as stated before. Conversely, they influence the buckling coefficients with a uniform factor for an aspect ratio $a/h_w = 1$ (Model 1) for all values of s_s. A further parametric study should be made in order to make an airtight conclusion about this boundary condition and its influence on the buckling coefficient.

It could be also observed in Fig. 5 that the Eurocode expression gives constant values of buckling coefficient although the load length is varied. For large values of s_s, differences in Eurocode 3 and I-girder results are significant. On the other side, the Lagerqvist's expression has a linear trend line. However, the proposed equation does not give a good fit to numerically obtained results for I-girder, especially in the case of Model 2, when the buckling coefficients according to Lagerqvist are not even on the safe side.

određivanju koeficijenta izbočavanja za analizirani I-nosač, predlaže se korišćenje modifikovanih izraza za određivanje k_F. Definisane su polinomske funkcije koje dobro aproksimiraju dobijene diskretne vrednosti koeficijenta k_F za I-nosač u funkciji dužine raspodeljenog poprečnog opterećenja. Uzimajući u obzir jednostavnost proračunskih procedura u Evrokodu 3, poželjno je koristiti polinome prvog ili drugog reda. Na primer, sledeće jednačine daju dobru aproksimaciju koeficijenta k_F za I-nosač:

Due to the described drawbacks of Eq. 3 and Eq. 4 for prediction of the buckling coefficient for the analysed I-girder models, we propose obtaining k_F through modified expressions. Polynomial functions of load length are defined to fit the obtained discrete values of k_F for the I-girder. Keeping in mind the simplicity of the design procedure in Eurocode 3, a first or second order polynomial function is desirable. The following forms give a good approximation for the k_F values of the I-girder:

za Model 1: / for Model 1:

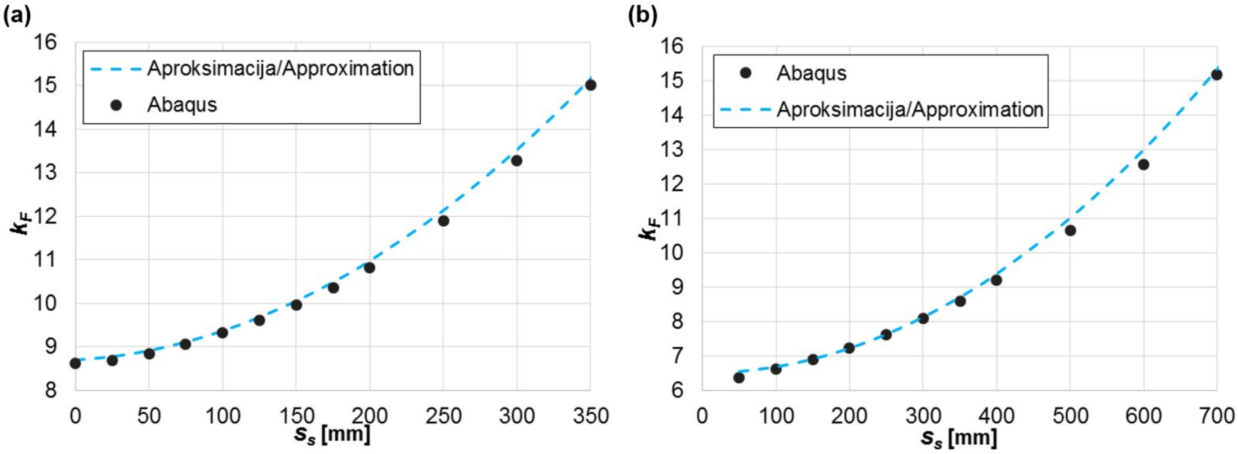
$$k_F = 4.7 \cdot 10^{-5} s_s^2 + 2 \cdot 10^{-3} s_s + 8.7 \tag{5}$$

za Model 2: / for Model 2:

$$k_F = 1.8 \cdot 10^{-5} s_s^2 + 6.5 \tag{6}$$

Na slici 6 grafički je predstavljeno poređenje koeficijenata izbočavanja za I-nosač, dobijenih numeričkom analizom i aproksimiranih koeficijenata prema predloženim izrazima. Treba imati na umu da su ove vrednosti koeficijenata određene samo za slučajeve analizirane geometrije. Potrebno je sprovesti detaljnu parametarsku analizu kako bi se odredili jedinstveni opšti izrazi u funkciji geometrijskih parametara, tj. odnosa širine i visine rebra, debljina rebra i nožice, širine nožice, itd.

A graphical comparison of the numerically obtained buckling coefficient of the I-girder and coefficients approximated by the proposed expressions is shown in Fig. 6. One should bear in mind that these values for the constants are determined only for the analysed geometries. A detailed parametric study is necessary in order to obtain a unique general expression as a function of geometric parameters, i.e. web aspect ratio, thickness of the web and flange, flange width, etc.



Slika 6. Vrednosti k_F za I-nosač dobijene iz programa Abaqus i preko predloženog izraza za: (a) Model 1; (b) Model 2
Figure 6. k_F values for I-girder obtained in Abaqus and through the proposed expressions: (a) Model 1; (b) Model 2

Analogno navedenim funkcijama za određivanje koeficijenta izbočavanja za I-nosač, moguće je dati jednačine koje opisuju koeficijent izbočavanja ukljieštene ploče (CC ploča):

In the same manner as it is done for I-girders, it is possible to give equations for describing the buckling coefficient for clamped plate (CC plate):

za Model 1: / for Model 1:

$$k_F = 3 \cdot 10^{-5} s_s^2 + 8 \tag{7}$$

za Model 2: / for Model 2:

$$k_F = 6 \cdot 10^{-6} s_s^2 + 0.003 \cdot s_s + 6 \tag{8}$$

Kao što je prethodno spomenuto, s obzirom na sličnosti u dobijenim numeričkim rezultatima, namera je bila da se pronađe veza između koeficijenta izbočavanja

As previously mentioned, it was intended to find a relation between buckling coefficients for the I-girder and CC plate, due to the observed similarity in the za I-nosač i za uklještenu ploču. Kod nosača, lokalizovano poprečno opterećenje je aplicirano na nožici, koja omogućava propagaciju opterećenja u rebro. Ugao rasprostiranja napona kroz nožicu definisan je standardom za projektovanje kao 45º [3]. Kako su koeficijenti k_F za I-nosač veći od odgovarajućih vrednosti za CC ploču, smatra se da je ova razlika posledica rasprostiranja opterećenja. Ako se pretpostavi da je nagib pod kojim se napon rasprostire 1:7 umesto definisanih 1:1, odgovarajuća dužina rasprostranjenog napona s_s' može se sračunati kao $s_s+2\cdot 7t_f$. Izbočavanje CC ploče sa zadatim lokalizovanim opterećenjem dužine s_s', daje vrlo dobro slaganje sa ponašanjem I-nosača pri izbočavanju za aplicirano opterećenje dužine s_s. Navedeno je prikazano kroz tabelu 3, gde je za računanje koeficijenta k_F za CC ploču sa zadatim opterećenjem dužine s_s', korišćena prethodno data jednačina 7. Navedene vrednosti mogu se uporediti s vrednostima koeficijenta k_F za I-nosač, koje su dobijene putem numeričkog modela. Slično, u slučaju Modela 2, pretpostavljeno je rasprostiranje napona pod nagibom 1:3.5. Poređenje koeficijenata izbočavanja dato je u tabeli 4. Važno je spomenuti da opisane relacije važe za manje dužine lokalizovanog opterećenja, do odnosa $s/h_w \leq 0.70$.

numerically obtained results for these two models. On girders, the patch load is introduced on flange plate, that enables spreading of load into the web. The angle of stress distribution through the flange is defined in the standard for design as 45º [3]. As k_F values for the I-girder are larger than k_F values for the CC plate, it is believed that this difference comes as a result of a load distribution. If the stress distribution with a slope 1:7 is assumed instead of the defined slope 1:1, the corresponding distributed stress length s_s' could be calculated as $s_s+2\cdot 7t_f$. The buckling behaviour of the CC plate with an applied load length s_s', gives a very good match with the behaviour of the I-girder with an applied load length s_s. This is presented in Table 3, where for calculation of k_F for the CC plate with an applied s_s' load length, previously given Eq. 7 is used. The presented values can be compared to the k_F values for I-girder, which are obtained in numerical simulation. Similarly, in the case of Model 2, the stress distribution slope 1:3.5 is assumed. Comparison between buckling coefficients is given in Table 4. It must be mentioned that the described relations are valid for smaller patch load lengths, up to the ratio $s/h_w \leq 0.70$.

Tabela 3. Veza između koeficijenta izbočavanja CC ploče i I-nosača za Model 1
Table 3. Relation between buckling coefficients for CC plate and I-girder for Model 1

s_s [mm]	s/a	s/h_w	k_F	$k_F(s_S)$	$s_S'=s_S+2\cdot 7t_f$ [mm]	$k_F(s_S')$	k_F
			CC	Izraz 7 *Eq. 7*		Izraz 7 *Eq. 7*	I-nosač *I-girder*
0	0.00	0.00	8.00	8.00	112	8.39	8.61
25	0.05	0.05	8.02	8.02	137	8.56	8.69
50	0.10	0.10	8.10	8.08	162	8.79	8.85
75	0.15	0.15	8.21	8.17	187	9.05	9.06
100	0.20	0.20	8.35	8.30	212	9.35	9.32
125	0.25	0.25	8.53	8.47	237	9.69	9.61
150	0.30	0.30	8.74	8.68	262	10.06	9.96
175	0.35	0.35	8.98	8.92	287	10.47	10.36
200	0.40	0.40	9.26	9.20	312	10.92	10.81
250	0.50	0.50	9.91	9.88	362	11.93	11.90
300	0.60	0.60	10.70	10.70	412	13.09	13.27
350	0.70	0.70	11.64	11.68	462	14.40	15.01

Tabela 4. Veza između koeficijenta izbočavanja CC ploče i I-nosača za Model 2
Table 4. Relation between buckling coefficients for CC plate and I-girder for Model 2

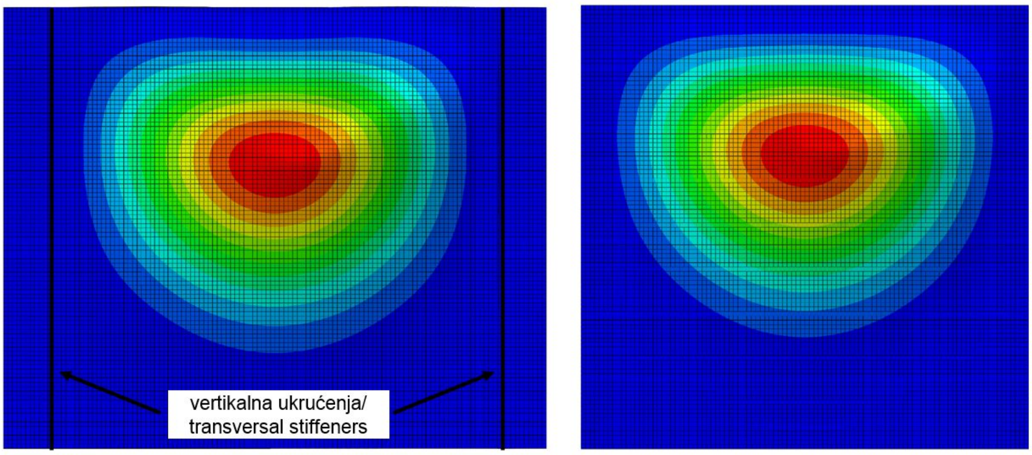
s_s [mm]	s/a	s/h_w	k_F	$k_F(s_S)$	$s_S'=s_S+2\cdot 3.5\cdot t_f$ [mm]	$k_F(s_S')$	k_F
			CC	Izraz 8 *Eq. 8*		Izraz 8 *Eq.8*	I-nosač *I-girder*
50	0.05	0.10	6.25	6.17	106	6.39	6.38
100	0.10	0.20	6.37	6.36	156	6.61	6.61
150	0.15	0.30	6.54	6.59	206	6.87	6.90
200	0.20	0.40	6.77	6.84	256	7.16	7.23
250	0.25	0.50	7.04	7.13	306	7.48	7.63
300	0.30	0.60	7.35	7.44	356	7.83	8.08
350	0.35	0.70	7.71	7.79	406	8.21	8.60

Interesantno je diskutovati o tome da li se pretpostavljeno rasprostiranje napona za analizirane slučajeve Modela 1 i Modela 2 može dovesti u vezu sa odnosom dimenzija a/h_w. Drugim rečima, postavlja se hipoteza da na rasprostiranje napona ne utiču samo debljina ili krutost nožice, već takođe i dužina pojasa između vertikalnih ukrućenja. Kako bi se doneli konačni zaključci, neophodno je sprovesti još numeričkih simulacija za različite slučajeve geometrije nosača.

Kao dalje poređenje ponašanja I-nosača i CC ploče, na slici 7 upoređeni su prvi oblici izbočavanja Modela 1 za dva slučaja modeliranja. Može se uočiti da izbočeni oblici Modela 1 izolovanog rebra, izloženog lokalizovanom opterećenju dužine s_s = 212 mm i Modela 1 I-nosača, izloženog lokalizovanom opterećenju dužine s_s=100 mm, odgovaraju jedan drugom u grafičkom smislu.

It is interesting to speculate on whether the assumed stress distributions for the analysed cases of Model 1 and Model 2, are in correlation with the aspect ratios a/h_w. In other words, the hypothesis is that stress distribution is not only influenced by flange thickness or stiffness, but also by the length of the zone between vertical stiffeners. For making final conclusions in this direction, more numerical simulations of different girder geometries should be obtained.

As a further comparison between the I-girder and CC plate, the first buckling mode for Model 1 for these two cases is juxtaposed in Fig. 7. It can be seen that the buckled shape for Model 1 for an isolated plate subjected to patch load length s_s = 212 mm and the buckled shape of an I-girder subjected to patch load length s_s = 100 mm graphically correspond to each other.



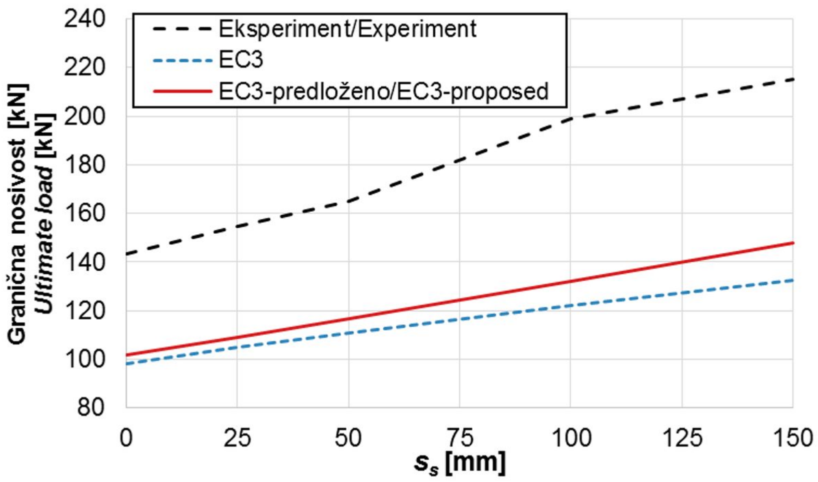
Slika 7. Prvi mod izbočavanja Modela 1 modeliranog kao: (a) I-nosač (s_s = 100 mm); (b) izolovano rebro nosača (s_s = 212 mm)
Figure 7. First buckled shape for Models 1: (a) I-girder (s_s = 100 mm); (b) isolated web plate (s_s = 212 mm)

Vrednosti eksperimentalno određenih graničnih opterećenja i graničnih opterećenja određenih prema Evrokodu 3, upoređene su grafički na slici 8 i numerički u tabeli 5. Granična nosivost prema Evrokodu 3 je sračunata na dva načina. Prvo su vrednosti dobijene prema važećem izrazu za određivanja koeficijenta izbočavanja prikazanom u jednačini 3 (sa koeficijentom k_F koji ne zavisi od opterećene dužine s_s). U drugom slučaju, procedura definisana Evrokodom 3 modifikovana je prema jednačini 5, odnosno 6, za određivanja koeficijenta k_F predloženim u ovom radu.

Kako se može primetiti, granična nosivost sračunata prema Evrokodovim procedurama značajno je niža od eksperimentalno dobijenih graničnih opterećenja, pri čemu je razlika izraženija za duža raspodeljena opterećenja. Drugim rečima, Evrokod 3 daje konzervativne vrednosti graničnih nosivosti. Međutim, preporučeni modifikovani izrazi s jednostavnom izmenom koja obuhvata samo koeficijent izbočavanja, utiču na poboljšanje vrednosti graničnog opterećenja, posebno u slučaju većih dužina poprečnog opterećenja.

Values of the experimentally obtained ultimate loads and corresponding ultimate loads according to the procedure applied in Eurocode 3 are pictorially compared in Fig. 8 and numerically in Table 5. The ultimate loads according to Eurocode 3 are calculated following two approaches. Firstly, the values are obtained according to the current procedure for determination of the buckling coefficient presented in Eq. 3 (with k_F not dependent on the loading length s_s). Secondly, the Eurocode defined procedure is modified by using Eq. 5 and Eq. 6 for k_F determination proposed in this paper.

Noticeably, the ultimate loads calculated by the current Eurocode procedures are considerably lower than the experimentally obtained ultimate loads and the difference is more pronounced for higher patch load lengths. In other words, the Eurocode design prediction gives rather conservative values for the ultimate load. However, the proposed modification of the Eurocode procedure with a simple correction regarding only the buckling coefficient improves predictive values, especially for longer loading lengths.



Slika 8. Poređenje graničnog opterećenja za različite pristupe
Figure 8. Comparison of the ultimate load by different approaches

Tabela 5. Granično opterećenje dobijeno eksperimentalnim putem, prema Evrokodu 3 i prema modifikovanom postupku
Table 5. Ultimate loads obtained experimentally, according to Eurocode 3 and modified Eurocode procedure

s_s [mm]	*P* [kN]		
	Eksperiment *Experiment*	EC3	EC3-predloženo *EC3-proposed*
0	143.3	98.3	102.0
20	154.6	104.8	109.2
50	165.0	110.9	116.7
100	199.0	122.3	132.0
150	215.0	132.6	148.0

4 ZAKLJUČCI

U ovom radu analizirano je izbočavanje ploča i I-nosača pod dejstvom lokalizovanog opterećenja. Upoređene su elastična kritična sila izbočavanja izolovanog rebra nosača za zadate različite uslove oslanjaja i elastična kritična sila I-nosača.

Primećena je veza između izbočavanja uklještene ploče i I-nosača. Uzimajući u obzir rasprostiranje napona kroz nožicu, moguće je dovesti u vezu izbočavanje uklještene ploče sa izbočavanjem I-nosača. Takođe, pretpostavlja se da na rasprostiranje napona ne utiču samo debljina ili krutost nožice, već i odnos dimenzija a/h_w i rastojanje između vertikalnih ukrućenja. Zaključci u ovom pravcu mogu dovesti i do poboljšanja u definisanju efektivnih opterećenih dužina.

Kako bi se poboljšale vrednosti koje Evrokod daje za proračun čeličnih konstrukcija, predloženi su novi izrazi za određivanje koeficijenta izbočavanja. Predloženi izrazi obuhvataju određivanje koeficijenta izbočavanja i u funkciji dužine apliciranog poprečnog opterećenja, dok su dalje procedure za određivanje graničnog opterećenja nepromenjene u odnosu na Evrokod 3. Predloženi izrazi poboljšavaju prediktivne vrednosti granične nosivosti, posebno u slučajevima većih dužina jednakopodeljenog lokalizovanog opterećenja. Međutim, kako bi se objasnio uticaj dužine opterećenja na koeficijent izbočavanja i kako bi bila određena direktna veza između izbočavanja izolovanog rebra nosača i izbočavanja I-nosača,

4 CONCLUSIONS

Buckling of plates and I-girders under patch load is analysed in this paper. Elastic critical load of an isolated web plate for different boundary conditions is compared with an elastic critical load of an I-girder.

The connection between buckling of a clamped plate and an I-girder is observed. Accounting stress distribution through flange plate, it is possible to relate buckling of a clamped plate to an I-girder. Also, it is assumed that not only flange thickness or stiffness influences the stress distribution, but also aspect ratios a/h_w and the length between vertical stiffeners. Conclusions in this direction may lead to the improvement of the definition of effective loaded length.

In order to improve the results from Eurocode for steel structures design, a new expressions for the determination of the buckling coefficient are proposed in this paper. The expressions include the calculation of the buckling coefficient also as a function of patch load length, while the procedure for obtaining the ultimate load is the same as in Eurocode 3. The proposed expressions improve predictive ultimate load values, especially for longer patch load lengths. However, in order to elucidate the influence of the length of patch load on buckling coefficient and to make a straightforward connection with an isolated web plate considering different geometries of an I-girder, further analyses are required.

uzimajući u obzir uticaj različitih geometrija nosača, neophodno je sprovesti dalje analize.

Opisani izrazi daju bazu za parametarske analize, kao i veliki broj numeričkih rezultata koji se dalje mogu koristiti kako bi se preciznije odredile granične nosivosti limenih nosača korišćenjem koeficijenta izbočavanja. Budući da su određena granična opterećenja prema predloženim izrazima i dalje ispod eksperimentalno dobijenih vrednosti, predstavljeni izrazi mogu se koristiti kako osnova za poboljšanje ostalih parametara u okviru proračunskih procedura u Evrokodu 3, npr. efektivne opterećene dužine.

In a nutshell, the presented expressions enable a fruitful background for parametric analyses and production of a large number of numerical tests that could be used in order to better determine the ultimate loads of plate girders using the buckling coefficients. Furthermore, since the ultimate loads are still too far below the experimentally obtained ultimate loads, the present expressions can be also exploited in order to improve different elements currently present in the design procedure in Eurocode 3, i.e. the effective loaded length.

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REZIME

ELASTIČNA KRITIČNA SILA PLOČA I LIMENIH NOSAČA POD DEJSTVOM LOKALIZOVANOG OPTEREĆENJA

Isidora JAKOVLJEVIĆ
Saša KOVAČEVIĆ
Nenad MARKOVIĆ

Određivanje elastične kritične sile (sile izbočavanja) predstavlja važan element u određivanju granične nosivosti limenih nosača prema Evrokodu 3 za proračun čeličnih konstrukcija. U radu se daje analiza kritične sile izbočavanja izolovane ploče koja odgovara rebru I-nosača s variranim uslovima oslanjanja i kritične sile koja odgovara samom I-nosaču, usled dejstva lokalizovanog opterećenja. Dati su zaključci o njihovoj vezi. Takođe, upoređena su eksperimentalno određena granična opterećenja I-nosača s vrednostima prema Evrokodu 3, za modele korišćene u ovoj analizi. Predložena je izmenjena procedura za određivanje granične nosivosti, koja sledi algoritam dat u Evrokodu 3, a prema kojoj se modifikuje izraz za određivanje koeficijenta izbočavanja. Kako bi se poboljšalo određivanje granične nosivosti, predloženo je računanje koeficijenata izbočavanja i u funkciji dužine lokalizovanog opterećenja.

Ključne reči: koeficijent izbočavanja, elastična kritična sila, lokalizovano opterećenje, izbočavanje ploče, izbočavanje I-nosača

SUMMARY

ELASTIC CRITICAL LOAD OF PLATES AND PLATE GIRDERS SUBJECTED TO PATCH LOAD

Isidora JAKOVLJEVIC
Sasa KOVACEVIC
Nenad MARKOVIC

The determination of the elastic critical load (buckling load) is an important element of the assessment of the ultimate load of plate girders according to Eurocode 3 for design of steel structures. An analysis of the critical load of a plate corresponding to the web of an I-girder for different boundary conditions and the critical load of the I-girder itself subjected to patch load is given in the paper. Conclusions regarding their correspondence are given. Also, experimentally determined ultimate loads of I-girders are compared with predictive values according to Eurocode 3 for the models used in this analysis. A modified procedure for the ultimate load determination is proposed by following the Eurocode 3 algorithm and changing only buckling coefficient. In order to improve the ultimate load prediction, it is suggested to calculate the buckling coefficient also as a function of patch load length.

Key words: buckling coefficient, elastic critical load, patch loading, plate buckling, plate girder buckling

IN MEMORIAM

Akademik prof. dr **Dušan Milović**, dipl.inž.građ.
Academician Prof. Dr. **Dusan Milovic**, B.Sc.Eng.Civ,
(1925-2018)

Povodom 150 godina rada Saveza inženjera i tehničara Srbije, naš časopis je prvi broj u 2018. godini posvetio akademiku Dušanu Miloviću. Izbor nije bio slučajan, jer je želja bila da se odabere ličnost koja je u naučnom radu dala izuzetan doprinos struci, zapažen ne samo u nas već i u svetu. U uvodniku je glavni urednik istakao sve značajne rezultate akademika Milovića, kao i izvode iz navoda izuzetnih autoriteta u oblasti građevinarstva o njegovim radovima. Objavljena je i njegova biografija, pa je ovaj in memoriam tekst kraći nego što bi bio da nije onog što je objavljeno u br. 1 za ovu godinu u našem časopisu. Ističemo da se najveći broj autora odazvao pozivu glavnog i odgovornog urednika da napiše radove za taj broj, među njima i veliko ime u oblasti Geotehničkog inženjerstva u svetu, prof. Hanz Brandl, s Tehničkog univerziteta iz Beča. Od značaja je da je u tom broju publikovan Milovićev rad: Nosivost šipova – teorijske i terenske metode (Bearing capacity of piles-Theory and field tests) koji predstavlja sintezu njegovih obimnih istraživanja u ovoj oblasti. Akademik Milović se veoma radovao kada je dobio njemu posvećen časopis. Nažalost, nedugo zatim je preminuo.

Vest da nas je 15. avgusta 2018. godine zauvek napustio akademik dr Dušan Milović, redovni profesor u penziji Fakulteta tehničkih nauka u Novom Sadu, primili

On the occasion of 150 years of work of the Union of Engineers and Technicians of Serbia, the first issue of our journal in 2018 has been dedicated to academician Dušan Milović. The choice was barely random, as we wanted to choose a person who made an extraordinary contribution to science and profession, recognized both in our country and worldwide. In the editorial, the editor-in-chief highlighted all significant results achieved by academician Milović and cited several experts from the field of civil engineering concerning his works. This In memoriam text is shorter than it would be if his biography was not published in the issue No.1 for 2018, which was dedicated to academician Milović. We would like to emphasize that most of the authors responded to the invitation of the editor-in-chief to write paper for the above issue, including Prof. Heinz Brandl from the *Vienna University* of *Technology*, a renowned expert in the field of Geotechnical Engineering. It is important to note that it also contains Milović's work titled *Bearing Capacity of Piles-Theory and Field Tests*, a paper which represents the synthesis of his extensive research in this field. Academician Milović was very pleased when he received the journal dedicated to him. Unfortunately, shortly afterwards he passed away.

The news that on August 15, 2018 academician Dr Dušan Milović, a full-time professor in retirement at the Faculty of Technical Sciences in Novi Sad has left us for good we received with great sorrow. He was born on smo sa žalošću. Rođen je 28. marta 1925. godine u Novoj Varoši. Osnovnu školu završio je u Nišu, a gimnaziju u Beogradu 1943. godine. Diplomirao je na Građevinskom fakultetu u Beogradu, konstruktivni smer, 1954. godine. Doktorsku tezu pod naslovom „Inženjerske osobine lesa u Jugoslaviji" odbranio je 1959. godine na Rudarsko-geološkom fakultetu u Beogradu. Valja naglasiti da je to u Srbiji prvi doktorat tehničkih nauka iz oblasti Mehanike tla i Fundiranja.

Od 1954. do 1966. godine radio je kao naučni saradnik u Institutu za ispitivanje materijala Srbije. Od 1966. do 1971. godine radio je na Univerzitetu u Šerbruku, Kvebek, Kanada kao pozvani nastavnik, a kasnije kao vanredni profesor i šef katedre za Mehaniku tla i Fundiranje. On je formirao Laboratoriju za mehaniku tla, koja je dobila njegovo ime. Već nakon tri godine Milovićevog rada u ovoj instituciji, katedru je priznao Nacionalni savet Kanade za naučna istraživanja kao jedan od centara izvrsnosti u Kanadi. Godine 1969. Univerzitet u Šerbruku je zbog velike reputacije koju je stekao, Dušana Milovića izabrao u najviše zvanje redovnog profesora.

Iz porodičnih razloga se 1971. godine vraća u Srbiju i do 1980. godine radio je u svojstvu savetnika u Institutu za građevinarstvo Vojvodine u Subotici, a posle formiranja Građevinskog fakulteta u Subotici (1974) izabran je za redovnog profesora i prvog dekana. Od 1980. sve do penzionisanja 1992. godine, bio je redovni profesor na Fakultetu tehničkih nauka (FTN) u Novom Sadu i šef katedre za Mehaniku tla i Fundiranje. Tri godine je bio direktor Instituta za industrijsku gradnju FTN.

Ceneći zapažene rezultate i domete u naučnom i stručnom radu, 1981. godine Vojvođanska akademija nauka i umetnosti izabrala je prof. D. Milovića za dopisnog, a 1987. godine za njenog redovnog člana. Srpska akademija nauka i umetnosti uključila ga je 1991. godine u svoj sastav u statusu njenog redovnog člana. Bio je član Društva za Mehaniku tla i Fundiranje Srbije (i njegov predsednik), član Jugoslovenskog Društva za Mehaniku tla i Fundiranje (i član predsedništva), Delegat Jugoslovenskog Društva za Mehaniku tla i Fundiranje u Svetskom Društvu za Mehaniku tla i Fundiranje, član Predsedništva SANU i Savetnik u Komitetu za uzimanje uzoraka pri Svetskom društvu za Mehaniku tla i Fundiranje. Tokom dugogodišnjeg rada na Univerzitetu držao je predavanja studentima na Građevinskim fakultetima u Novom Sadu, Subotici i Šerbruku. Bio je mentor više magistarskih radova i doktorskih disertacija u Kanadi i kod nas.

Akademik Milović je, u periodu od 1954. do 1995. godine, pored nastavnog i obrazovnog rada, usmerio svu svoju aktivnost na rešavanje teorijskih problema u oblasti Mehanike tla kao i na eksperimentalnom proučavanju temeljnog tla i temeljnih konstrukcija različitih objekata. To je ostvario u 16 naučno-istraživačkih projekata u kojima je bio rukovodilac i glavni istraživač. Projekte je finansirao Fond za naučni rad Srbije (3), Conseil National de Recherches, Ottawa u Kanadi (3), SIZ za naučni rad Vojvodine (7), Jugoslovensko-američki Joint Venture projekat (1) i Fond za naučni rad Srpske akademije nauka i umetnosti (2). Originalna teorijska rešenja i rezultate eksperimentalnih ispitivanja objavio je u 227 naučnih radova, od kojih je u 98 radova prvi, a u 139 jedini autor.

March 28, 1925 in Nova Varos. He finished the primary school in Niš, the grammar school in Belgrade, in 1943. He graduated from the Faculty of Civil Engineering in Belgrade in 1954, engineering department. In 1959 he defended his doctoral thesis under the title "Engineering Characteristics of Loess in Yugoslavia" at the Faculty of Mining and Geology in Belgrade. It should be emphasized that this is the first PhD in technical sciences from the field of soil mechanics and funding in Serbia.

From 1954 to 1966, he worked as scientific associate at the Institute for Testing of Materials of Serbia. Between 1966 and 1971 he worked at the University of Sherbrooke, Quebec, Canada as invited lecturer, and later as associate professor and the head of the Department of Soil Mechanics and Funding. He established the Laboratory for Soil Mechanics, which carries his name. After three years of Milović's work in this institution, the chair was recognized by the National Scientific Research Council of Canada as one of the centers of excellence in Canada. In 1969, as a result of his great reputation, Dušan Milović was elected as full professor in the University of Sherbrooke.

For family reasons, in 1971 he returned to Serbia and until 1980 worked as consultant at the Institute for Civil Engineering of Vojvodina in Subotica. After the founding of the Faculty of Civil Engineering in Subotica (1974) he was elected as full professor and the first Dean. From 1980 until his retirement in 1992, he was full professor at the Faculty of Technical Sciences (FTN) in Novi Sad and the head of the Department of Soil Mechanics and Funding. For three years he was the director of the Institute for Industrial Engineering at the Faculty of Technical Sciences.

Having appreciated his remarkable results and achievements in scientific and professional work, in 1981 the Vojvodina Academy of Sciences and Arts awarded Prof. Milović with associate membership, and in 1987 with regular membership. In 1991, the Serbian Academy of Sciences and Arts included him in his membership in the status of a regular member. He was member of the Association for Soil Mechanics and Funding of Serbia (and its president), member of the Yugoslav Association for Soil Mechanics and Funding (and member of the Presidency), delegate of the Yugoslav Association for Soil Mechanics and Funding in the World Association for Soil Mechanics and Funding, member of the Presidency of the SANU and Advisor to the Committee for Sampling with the World Association for Soil Mechanics and Funding. During many years of work at the University he lectured at Faculties of Civil Engineering in Novi Sad, Subotica and Sherbrooke. He mentored several master theses and doctoral dissertations in Canada and in Serbia as well.

In the period from 1954 to 1995, in addition to teaching and educational work, academician Milović focused his entire activity on solving theoretical problems in the field of soil mechanics, as well as on experimental studies of foundation soil and foundation structures of various objects. He accomplished this in 16 scientific-research projects in which he was the leader and the principal researcher. The projects were funded by the Fund for Scientific Work of Serbia (3), the Conseil National de Recherches, Ottawa in Canada (3), SIZ for scientific work of Vojvodina (7), the Yugoslav-American Joint Venture Project (1) and the Scientific Research Fund of the Serbian Academy of Sciences and Art (2). He published original theoretical solutions and results of experimental tests in 227 scientific papers, of which in

Njegovi radovi su citirani preko 210 puta u časopisima sa SCI liste i još dugo će biti predmet interesovanja mnogih istraživača u svetu.

U oblasti direktnog fundiranja proširio je primenu teorije elastičnosti i prikazao rešenja za određivanje veličine sleganja i ugaonih distorzija za sve oblike i sve relativne krutosti temelja, za razne slučajeve opterećenja i kompleksne modele tla, uključujući anizotropna svojstva tla, ograničenu debljinu deformabilne sredine nedeformabilnim substratumom, kao i višeslojne sisteme. Rešenja dobijena metodom konačnih elemenata i dvostrukim Fourier-ovim redovima prikazana su na internacionalnim kongresima geomehanike i u časopisima svetskog renomea (osam radova u Londonu, Parizu, Berlinu, Moskvi i Tokiju u periodu od 1970. do 1973. godine) i smatraju se pionirskim i veoma značajnim.

U oblasti dubokog fundiranja, Dušan Milović je došao do rešenja problema određivanja veličine graničnog i dozvoljenog opterećenja šipova pomoću podataka dobijenih iz terenskih opita statičke penetracije. Metodom konačnih razlika prikazao je rešenje za određivanje horizontalnog pomeranja šipa, rotacije i sila u presecima za šip bilo koje krutosti, sa slobodnom i uklještenom glavom, usled dejstva kako vertikalnog tako i horizontalnog opterećenja. Za proveru tačnosti teorijskih rešenja koristio je rezultate terenskih opita probnim opterećenjem u razmeri 1:1.

Počev od istraživanja analiziranih u doktorskoj disertaciji, i kasnije u okviru Projekata Yu – SAD saradnje, značajne su aktivnosti Dušana Milovića usmerene na teorijske studije i terenska ispitivanja lesnog tla. Rezultati ovih istraživanja naišli su na zapaženi interes u svetu. Razlog za to je to što se les javlja ne samo u našoj zemlji, već je veoma rasprostranjen u Rusiji, Kini, Americi i drugim zemljama. U pomenutim zemljama registrovana su veoma teška oštećenja objekata i pri relativno niskim intenzitetima opterećenja. Prof. Milović je na temelju opsežnih terenskih i laboratorijskih ispitivanja odredio dominantne parametre koji utiču na ponašanje lesnog tla i modifikovao teoriju proračuna ukupnih i diferencijalnih sleganja. Predloženi postupak je posebno vredan, jer se može uspešno primeniti na bilo koju lokaciju s lesnim tlom u svetu. Njegovi radovi o lesu su među vodećim u svetu.

Pored aktivnog višedecenijskog rada na nastavnom i naučnom planu, aktivno je učestvovao na rešavanju najsloženijih praktičnih problema fundiranja mnogobrojnih građevinskih objekata. Za više od 220 objekata dao je rešenje za siguran i ekonomičan način fundiranja, među kojima su visoke zgrade, silosi za žito, mostovi, energane, šećerane, sportski centri, luke, brodogradilišta i drugi značajni građevinski objekti. Ove probleme je rešavao i u zemlji i u inostranstvu (Iraku, Čehoslovačkoj, Poljskoj i Kanadi).

Rezultate istraživanja prikazao je na svetskim kongresima za Mehaniku tla i Fundiranje i to: London 1957, Pariz 1961, Montreal 1965, Meksiko Siti 1969, Moskva 1973, Tokio 1977, Stokholm 1981, San Francisko 1985, Rio de Žaneiro 1989, Hamburg 1997, Osaka 2005. i Čikago 2013. Pored toga, prikazao je radove na tri svetska kongresa za Inženjersku geologiju u Buenos Ajresu 1986, Lisabonu 1994. i Vankuveru 1998. godine.

98 he was the principal, and in 139 the only author. His papers were cited more than 210 times in journals from the SCI list and will long be the subject of interest of many researchers worldwide.

In the field of direct funding, he expanded the application of the theory of elasticity and presented solutions for determining the amount of settling and angular distortions for all shapes and all relative foundation stiffness for various load cases and complex soil models, including anisotropic soil properties, limited thickness of the deformable environment by non-deformable substrate, as well as multilayer systems. Solutions obtained using the finite element method and the double Fourier series were presented at international conferences on geomechanics and in world renowned journals (eight papers in London, Paris, Berlin, Moscow, and Tokyo over the period from 1970 to 1973), and are considered pioneering and very significant.

In the field of deep founding, using data obtained from field experiments of static penetration, Dušan Milović came to the solution of the problem of determining the amount of boundary and permissible load of piles. Using the finite difference method he presented a solution for determining the horizontal displacement of piles, rotation and force in cross sections for a pile of any rigidity, with free and clamped head, due to the effects of both vertical and horizontal loads. He used the results of field experiments with a test load in scale 1:1 to check the accuracy of theoretical solutions.

Starting from the research analyzed in the doctoral dissertation, and later within the framework of the YU-US cooperation projects, significant activities of Dušan Milović were directed towards theoretical studies and field tests of loess soil. The results of these tests were received with significant interest in the world. The reason for this is that, in addition to our country, loess is also widespread in Russia, China, America and other countries. In these countries, very serious damage to buildings was registered even at relatively low load intensities. On the basis of extensive field and laboratory tests, Prof. Milović determined dominant parameters that affect the behaviour of loess soil and modified the theory of calculation of overall and differential settle. The proposed procedure is especially valuable because it can be successfully applied to any location with loess soil in the world. His papers on loess are among the world's leading works on the subject.

In addition to active and decades-long work in teaching and scientific field, he also actively participated in solving the most complex practical problems of funding numerous civil engineering structures. He provided solutions for safe and economical way of funding for more than 220 buildings, among which are high buildings, grain silos, bridges, energy facilities, sugar plants, sports centers, ports, shipyards and other important engineering facilities. These problems he solved in our country and abroad (Iraq, Czechoslovakia, Poland and Canada) as well.

He presented the results of his research at world congresses for soil mechanics and funding in London (1957), Paris (1961), Montreal (1965), Mexico City (1969), Moscow (1973), Tokyo (1977), Stockholm (1981), San Francisco (1985), Rio de Janeiro (1989), Hamburg (1997), Osaka (2005) and Chicago (2013). In addition, he presented his works at three world congresses for engineering geology: Buenos Aires (1986), Lisbon (1994) and Vancouver (1998).

S radovima je učestvovao i na sledećim evropskim, regionalnim i dunavskim kongresima za Mehaniku tla i Fundiranje: Budimpešta 1963, 1971, 1984. i 1990, Visbaden 1963, Čikago 1965, Haifa 1967, Beograd 1970, Bangkok 1971, Pariz 1971, 1980. i 1984, Stokholm 1974, Beč 1976, Bratislava 1977, Brno 1979, Cirih 1982, Amsterdam 1982, Nagoja 1985, Peking 1986. i 1988, London 1989, Firenca 1990, Vankuver 1991, Dalas 1992, Gent 1993 i Kopenhagen 1995. godine. Akademik Milović je na više internacionalnih skupova, podneo uvodna predavanja: Čikago 1965, Meksiko 1969, u Montrealu 1969, u Brnu 1979, i Londonu 1989. Bio je generalni izvestilac na Podunavskoj konferenciji za Mehaniku tla u Budimpešti 1990, kao predsednik tehničke sekcije za koplapsibilna tla u Dalasu 1992, i na Evropskoj konferenciji u Kopenhagenu 1995.

Kao retko koji naučnik iz Srbije, akademik Milović je publikovao brojne radove u svetski priznatim časopisima: Geotechnique, Soils and foundations, Journal of the American Society for Testing and Materials ASTM, Sols Soils, L'Ingenieur Constructeur, Le Genie Civil, Travaux, Bauingenieur.

Objavio je i sledeće knjige i monografije: Analiza napona i deformacija u mehanici tla 1974, Geomehanika 1976, Mehanika tla 1977, Mehanika tla 1982, Mehanika tla 1987, Problemi fundiranja na lesnom tlu 1987, Stresses and displacements for shallow foundations 1992, Greške u Fundiranju 2005, Problemi interakcije tlo - temelj - konstrukcija 2009.

U našem časopisu je publikovao veći broj radova, i uživao je poštovanje ne samo čitalaca već i svih članova ranijih redakcija i sadašnjih članova Redakcije.

Akademik Milović je dobitnik Oktobarske nagrade grada Beograda 1962, odlikovan je Medaljom za zasluge za narod, odlikovan je Ordenom rada sa srebrnim vencem, počasni je i zaslužni član Saveza građevinskih inženjera i tehničara Srbije.

Akademik Dušan Milović je svetski poznat i priznat istraživač koji je značajno doprineo razvoju Mehanike tla kako u nas tako i u svetu. Bio je cenjen i uvažavan profesor i istraživač koji je sa ogromnim elanom prenosio svoja znanja na studente i saradnike. Njegovi radovi i knjige pisani su veoma jasnim jezikom, inženjerski precizno, s dragocenim poređenjima teorijskih i eksperimentalnih istraživanja. Iako nije više među nama, ostala su njegova dela koja će još dugo biti putokaz i inspiracija mnogim istraživačima. Smrću profesora Milovića akademska zajednica ostala je bez izuzetne ličnosti, posvećenog istraživača koji je do poslednjih dana života neumorno radio i stvarao. Ipak, njegovim odlaskom najviše je izgubila porodica, sinovi Uroš i Strahinja, snahe i unučad Mišel, Jovana, Dušan i Aleksandar, te im i ovom prilikom izjavljujemo duboko saučešće.

Redakcija
Novembra 2018.

With his papers he also participated in the following European, regional and Danube conferences for soil mechanics and funding: Budapest (1963, 1971, 1984 and 1990), Wiesbaden (1963), Chicago (1965), Haifa (1967), Belgrade (1970), Bangkok (1971), Paris (1971, 1980 and 1984), Stockholm (1974), Vienna (1976), Bratislava (1977), Brno (1979), Zurich (1982), Amsterdam (1982), Nagoya (1985), Beijing (1986 and 1988), London (1989), Florence (1990), Vancouver (1991), Dallas (1992), Ghent (1993) and Copenhagen (1995). Academician Milović delivered introductory lectures at several international conferences: Chicago (1965), Mexico (1969), Montreal (1969), Brno (1979), and London (1989). He was the general reporter at the Danube Conference on Soil Mechanics in Budapest in 1990, as the president of the technical section for collapsible soils in Dallas (1992), and at the European Conference in Copenhagen (1995).

As a rare example among scientist from Serbia, academician Milović published many papers in world-renowned journals: Geotechnique, Soils and Foundations, Journal of the American Society for Testing and Materials ASTM, Sols Soils, L'Ingenieur Constructeur, Le Genie Civil, Travaux, Bauingenieur.

He published the following books and monographs: Stress and Strain Analysis in Soil Mechanics (1974), Geomechanics (1976), Soil Mechanics (1977), Soil Mechanics (1982), Soil Mechanics (1987), Problems of Funding in Loess Soil (1987), Stresses and displacements for shallow foundations (1992), Errors in Funding (2005), Problems of the soil-foundation-structure interaction (2009).

He published a number of papers in our journal, and in addition to readers, he was respected both by members of previous editorial boards and the present members of the editorial board.

Academician Milović was the winner of the October Award of the City of Belgrade in 1962; he was decorated with the Medal for Services for the People, the Order of Work with a Silver Wreath, and is honorary and meritorious member of the Association of Civil Engineers and Technicians of Serbia.

Academician Dušan Milović is world-renowned and well-known researcher who significantly contributed to the development of soil mechanics both in our country and worldwide. He was appreciated and respected professor and researcher who conveyed his knowledge to students and associates with great enthusiasm. His papers and books were written in clear language with engineering precision, and valuable comparisons of theoretical and experimental research. Although he is no longer with us, his work remains and will continue to be a direction for further research and inspiration for many researchers. With the loss of Prof. Milović, the academic community has remained without an extraordinary person, a dedicated researcher who worked and created tirelessly until the last days of his life. However, his departure was the greatest loss for his family: his sons Uroš and Strahinja, his daughter-in-law and grandchildren Michelle, Jovana, Dušan and Aleksandar, and once more, on this occasion we declare our deepest sympathies.

Editorial board
November 2018

ДГКС
Друштво грађевинских конструктера Србије
ASES
Association of Structural Engineers of Serbia

ДРУШТВО ГРАЂЕВИНСКИХ КОНСТРУКТЕРА СРБИЈЕ
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ДРУШТВО ГРАЂЕВИНСКИХ КОНСТРУКТЕРА СРБИЈЕ
Бр. 45/2018
16. 11. 2018 год
БЕОГРАД

ИСПРАВКА У ВЕЗИ ПУБЛИКАЦИЈА СА 15. КОНГРЕСА ДРУШТВА ГРАЂЕВИНСКИХ КОНСТРУКТЕРА СРБИЈЕ

Накнадним увидом у материјале публиковане поводом 15. Конгреса ДГКС (Зборник радова - стр. 3 и Књига резимеа - стр. 3) уочено је да је у списку референци наведених у тексту: 'Признање за животно дело у грађевинском конструктерству Дејану Бајићу, дипл.инж ' ненамерно начињена грешка. Наиме, проф. др Радомир Фолић случајно је изостављен из списка аутора конструкције Српског народног позоришта у Новом Саду. Проф. др Радомир Фолић је био аутор и заједно са проф. др Мирком Аћићем одговорни пројектант ове конструкције.

У име уредника публикација и Председништва ДГКС извињавам се проф. др Радомиру Фолићу на начињеном пропусту.

Председник ДГКС

Проф. др Златко Марковић, дипл.инж.грађ.

CORRECTION WITH REGARD TO PUBLICATIONS FROM THE 15th CONGRESS OF ASSOCIATION OF STRUCTURAL ENGINEERS OF SERBIA

Based on a subsequent insight gained into materials published on the occasion of the 15th Congress of ASES (Proceedings - page 3 and Book of Summaries - page 3), an unintentional mistake has been discovered in references cited in the text: "Acknowledgment for Life Achievement in Construction Design to dipl. Ing. Dejan Bajic". Namely, prof. Dr Radomir Folic was accidentally omitted as the author of structures of the Serbian National Theater in Novi Sad. Together with Professor Mirko Aćić Ph.D., Professor Radomir Folić Ph.D. was the chief designer of this structure.

On behalf of the editors of the Publications and the Presidency of ASES I apologize to Professor Radomir Folić Ph.D. on the omission.

President of ASES
Professor Dr Zlatko Marković, B.C.E.

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 disciplina (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.

Uputstva za pripremu rukopisa

Rukopis otkucati jednostrano na listovima A-4 sa marginama od 31 mm (gore i dole) a 20 mm (levo i desno), u Wordu fontom Arial sa 12 pt. Potrebno je uz jednu kopiju svih delova rada i priloga, dostaviti i elektronsku verziju na navedene E-mail adrese, ili na CD-u. Autor je obavezan da čuva jednu kopiju rukopisa kod sebe.

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.

* U**putstvo 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 telefon, 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.

Formule i jednačine treba pisati pažljivo vodeći računa o indeksima i eksponentima. Autori uz izraze u tekstu definšu simbole redom kako se pojavljuju, ali se može dati i posebna lista simbola u prilogu.

Prilozi (tabele, grafikoni, sheme i fotografije) rade se u crno-beloj tehnici, u formatu koji obezbeđuje da pri smanjenju na razmere za štampu, po širini jedan do dva stupca (8 cm ili 16,5 cm), a po visini najviše 24,5 cm, ostanu jasni i čitljivi, tj. da veličine slova i brojeva budu najmanje 1,5 mm. Originalni crteži treba da budu kvalitetni i u potpunosti pripremljeni za presnimavanje. Mogu biti i dobre, oštre i kontrastne fotokopije. Koristiti fotogrfije, u crno-beloj tehnici, na kvalitetnoj hartiji sa oštrim konturama, koje omogućuju jasnu reprodukciju.

U popisu literature na kraju rada daju se samo oni radovi koji se pominju u tekstu. Citirane radove treba prikazati po abecednom redu prezimena prvog autora. Literaturu u tekstu označiti arapskim brojevima u uglastim zagradama, kako se navodi i u Popisu citirane literature, napr [1]. Svaki citat u tekstu mora se naći u Popisu citirane literature i obrnuto svaki podatak iz Popisa se mora citirati u tekstu.

U Popisu literature se navode prezime i inicijali imena autora, zatim potpuni naslov citiranog članka, iza toga sledi ime časopisa, godina izdavanja i početna i završna stranica (od - do). Za knjige iza naslova upisuje se ime urednika (ako ih ima), broj izdanja, prva i poslednja stranica poglavlja ili dela knjige, ime izdavača i mesto objavljivanja, ako je navedeno više gradova navodi se samo prvi po redu. Kada autor citirane podatke ne uzima iz izvornog rada, već ih je pronašao u drugom delu, uz citat se dodaje «citirano prema...».

Autori su odgovorni za izneseni sadržaj i moraju sami obezbediti eventualno potrebne saglasnosti za objavljivanje nekih podataka i priloga koji se koriste u radu.

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Rukopisi i prilozi objavljenih radova se ne vraćaju. Sva eventualna objašnjenja i uputstva mogu se dobiti od Redakcionog odbora.

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Izdavanje časopisa "Građevinski materijali i konstrukcije" finansijski su pomogli:

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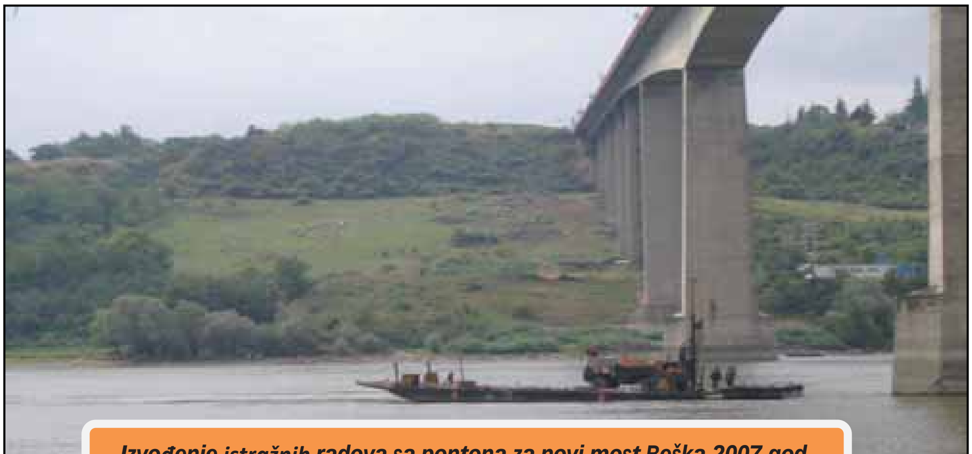


Izvođenje istražnih radova sa pontona za novi most Beška,2007.god.

Geotehnička istraživanja i ispitivanja – in situ

Od terenskih istražnih radova izdvajamo izvođenje istražnih bušotina (IB), standardnih penetracionih opita (SPT), statičkih penetracionih opita (CPT i CPTU), opita dilatometarskom sondom (DMT i SDMT), ispitivanja vodopropustljivosti tla različitim terenskim metodama (VDP), ugradnja pijezometara i dr.
Terenske metode ispitivanja šipova zauzimaju značajno mesto u našoj delatnosti, a na tržištu se izdvajamo kao lideri u toj oblasti u protekloj deceniji.

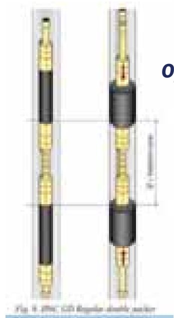

DLT-dinamičko ispitivanje šipova

Ispitivanje šipova

SLT metoda (Static load test) ispitivanje nosivosti šipova statičkim opterećenjem;
DLT metoda (Dynamic load test) ispitivanje nosivosti šipova dinamičkim opterećenjem;
PDA metoda (Pile driving analysis) omogućava praćenje i optimizaciju procesa pobijanja prefabrikovanih betonskih i čeličnih šipova u tlo;
PIT (SIT) metoda (Pile(Sonic) integrity testing) koristi se za ispitivanje integriteta izvedenih šipova (dužine, prekida, suženja ili proširenja).


CPT/CPTU opiti


Aktivno klizište


oprema za ispitivanje vodopropusnosti stena pod pritiskom do 10 bar-a metodom LIŽONA

Laboratorija za puteve i geotehniku

Laboratorija za puteve i geotehniku akreditovana je kod Akreditacionog tela Srbije – ATS prema SRPS ISO/IEC 17025:2006. U njoj se vrše ispitivanja tla (identifikaciono–klasifikaciona ispitivanja, fizičko–mehanička modelska ispitivanja), kamenog agregata i brašna, bitumena i bitumenskih emulzija, asfaltnih mešavina. U okviru laboratorijskih ispitivanja na terenu vrši se kontrola kvaliteta ugrađenog materijala i izvedenih radova (prethodna, tekuća, kontrolna ispitivanja i izvođenja opita in situ).

Projektovanje puteva i sanacija klizišta

U okviru projektovanja značajno mesto u radu zauzimaju geotehnička istraživanja terena i projekti sanacije klizišta - nestabilnih kosina useka i nasipa puteva i prirodno nestabilnih padina . Značajna su i projekovanja svih vrsta fundiranja specijalnih geotehničkih konstrukcija. Ističe se i iskustvo u oblasti putarstva, na projektovanju novih, rehabilitacija i rekonstrukcija postojećih puteva svih rangova sa pratećim objektima i dimenzionisanjem kolovoznih konstrukcija.

Nadzor

Naši inženjeri imaju veliko iskustvo u kontroli i proveri kvaliteta izvođenja svih vrsta radova, kontroli građevinske dokumentacije i praćenju radova u skladu sa njom, kao i rešavanju novonastalih situacija tokom izvođenja radova.

BUILDING TRUST

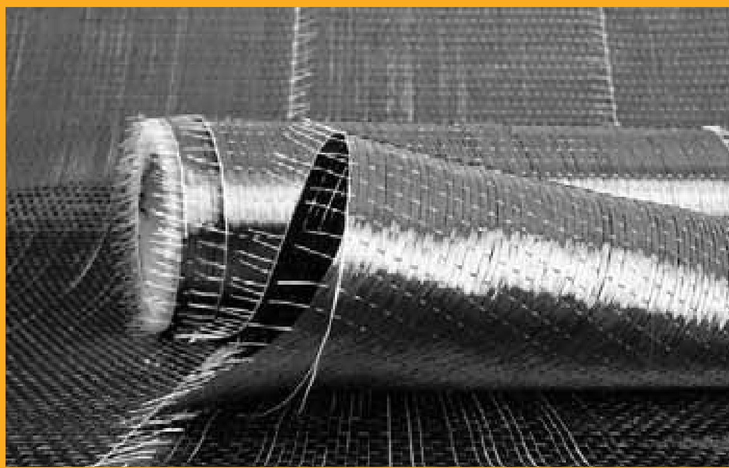
NAPREDNA SIKA REŠENJA U OBLASTI STRUKTURALNIH OJAČANJA

Kompanija Sika pruža trajnu dodatnu vrednost vlasnicima građevinskih objekata, njihovim konsultantima i izvođačima, kao i tehničku podršku tokom svih faza projekta, od ispitivanja uslova i razvoja inicijalnog koncepta ojačanja pa sve do uspešnog završetka i primopredaje projekta

SIKA – VAŠ PARTNER NA GRADILIŠTU

- Globalni lider na tržištu građevine i građevinske hemije
- Najbolja tehnička ekspertiza i praksa za sanaciju betona i strukturalna ojačanja
- Odlična reputacija kod vodećih izvođača i ugovarača posla

SIKA VREDNOSTI I INOVACIJE U GRAĐEVINI



- 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
- Sika mreža obučenih i iskusnih građevinskih stručnjaka

JEDINSTVENA SIKA REŠENJA U ZAHTEVNIM USLOVIMA

- Rešenja za gotovo sve uslove apliciranja
- Kontrolisano vreme rada, vreme sazrevanja i očvršćavanja za različite vremenske uslove
- Posebna rešenja završnih ojačanja za korišćenje kod betona slabije jačine i drugih podloga

POTVRĐENI SIKA SISTEMI I TEHNIKE APLICIRANJA

- Preko 40 godina iskustva u strukturalnim ojačanjima, sistemima i tehnikama
- Proizvodi i sistemi sa brojnim testovima i procenama kako internim tako i eksternim
- Najviši međunarodni standardi proizvodnje i kontrole kvaliteta

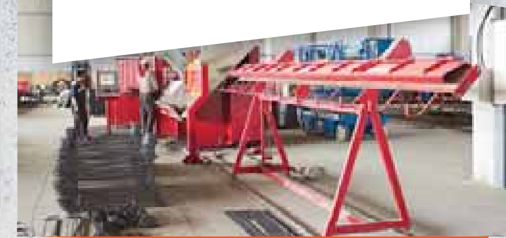
PUT INŽENJERING

Put inženjering d.o.o punih 25 godina radi kao specijalizovano preduzeće za izgradnju infrastrukture u niskogradnji i visokogradnji, kao i proizvodnjom kamenog agregata i betona. Preduzeće se bavi i transportom, uslugama građevinske mehanizacije i specijalne opreme.

Za spravljanje betona koristimo drobljeni krečnjački agregat sa našeg kamenoloma, deklarisanih frakcija, kontrolisane vlažnosti. Kompletan proces proizvodnje i kontrole kvaliteta vršimo prema važećim standardima.

Kao generalni izvođač radova, vršimo koordinaciju svih učesnika na projektu, planiranje, praćenje i nabavku materijala, kontrolu kvaliteta izvedenih radova, poštujući zadate vremenske rokove i finansijski okvir investitora.

Obradu armature vršimo brzo, stručno i kvalitetno, sa kompjuterskom preciznošću i dimenzijama po projektu.



Osnovi princip našeg poslovanja zasniva se na individualnom pristupu svakom klijentu i pronalaženje najoptimalnijeg rešenja za njegove transportne i logističke potrebe.

Koristeći inovativne tehnike i kvalitetan građevinski materijal iz sopstvenih resursa, spremni smo da odgovorimo na mnoge zahteve naših klijenata iz oblasti niskogradnje.

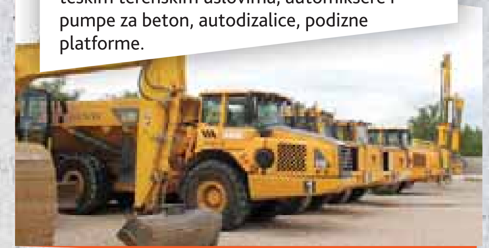
Naša kompanija u oblasti visokogradnje primenjuje sistem prefabrikovnih betonskih elemenata koji u odnosu na klasičnu gradnju ima brojne prednosti.

Usluge građevinskom mehanizacijom vršimo tehnički ispravnim mašinama, sa potrebnim sertifikatima kako za rukovaoce građevinskim mašinama tako i za same mašine.

Osnovna prednost prefabrikovane konstrukcije jeste brzina kojom konstrukcija može biti projektovana, proizvedena, transportovana i namontirana.

Prednapregnute šuplje ploče su konstruktivni elementi visokog kvaliteta, proizvedeni u fabrički kontrolisanim uslovima.

Raspolažemo opremom i mašinama za sve zemljane radove, kipere i dampere za rad u teškim terenskim uslovima, automiksere i pumpe za beton, autodizalice, podizne platforme.



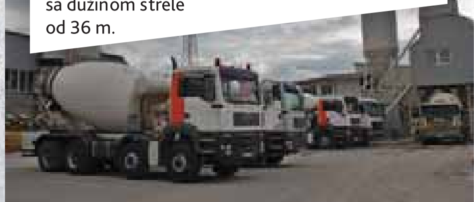
Izvodimo hidrograđevinske radove u izgradnji kanalizacionih mreža za odvođenje atmosferskih, otpadnih i upotrebljenih voda, izvođenjem hidrograđevinskih radova u okviru regulacije rečnih tokova, kao i izvođenjem hidrotehničkih objekata.

Izrađujemo betonske "New Jersey profile" koji se u svetu koriste za preusmeravanje saobraćaja i zaštitu pešaka u toku izgradnje puta, kao i Betonblock sistem betonskih blokova.

Sakupljanje i privremeno skladištenje otpada vršimo našim specijalizovanim vozilima i deponujemo na našu lokaciju sa odgovarajućom dozvolom. Kapacitet mašine je 250 t/h građevinskog neopasnog otpada.

Površinski kop udaljen je 35 km od Niša. Savremene drobilice, postrojenje za separaciju i sejalica efikasno usitnjavaju i razdvajaju kamene agregate po veličinama. Tehnički kapacitet trenutne primarne drobilice je 300 t/h.

Uslugu transporta vršimo automikserima, kapaciteta bubnja od 7 m³ do 10 m³ betonske mase. Za ugradnju betona posedujemo auto-pumpu za beton, radnog učinka 150 m³/h, sa dužinom strele od 36 m.



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