Lateral buckling of castellated beams

Radić, Ivan; Markulak, Damir

Source / Izvornik: **Tehnički vjesnik, 2007, 14, 25 - 35**

Journal article, Published version
Rad u časopisu, Objavljena verzija rada (izdavačev PDF)

Permanent link / Trajna poveznica: https://urn.nsk.hr/urn:nbn:hr:133:320643

Rights / Prava: Attribution 4.0 International/Imenovanje 4.0 međunarodna

Download date / Datum preuzimanja: **2024-03-29**

GRAĐEVINSKI I ARHITEKTONSKI FAKULTET OSIJEK
Faculty of Civil Engineering and Architecture Osijek

Repository / Repozitorij:

Repository GrAFOS - Repository of Faculty of Civil Engineering and Architecture Osijek

SVEUČILIŠTE JOSIPA JURJA STROSSMAYERA U OSIJEKU

dabar
DIGITALNI AKADEMSKI ARHIVI I REPOZITORIJI

ISSN 1330-3651
UDC/UDK 624.072.2 : 624.075.4] : 519.61

LATERAL BUCKLING OF CASTELLATED BEAMS

Ivan Radić, Damir Markulak

Prethodno priopćenje

For greater spans and dominant bending stress in major plane, castellated beams are very rational structural solution. These beams have relatively high slenderness and significant web openings. This paper describes the research into the stability of the castellated beams considering resistance to lateral buckling, for which new regulation Eurocode 3 suggests simplified verification of the resistance to lateral buckling. Various spans of the roof castellated beams with IPE and HEA cross-sections are considered. 3D FEM models of these beams are made and the resistance of the lateral buckling is calculated according to the method proposed in Eurocode 3 and the finite element method (FEM). Results of the research indicate that the suggested method of the verification of lateral buckling for castellated beams gives good estimates of their resistance, because comparisons of the given values indicate that differences in resistance to lateral buckling did not exceed 5 % for all analyzed beams.

Keywords: *3D FEM models of castellated beams, castellated beams, lateral buckling, influence of the web openings*

Bočno izvijanje saćastih nosača

Preliminary note

Saćasti nosači su vrlo racionalno konstrukcijsko rješenje kod nosača većih raspona i dominantnog naprezanja savijanjem u ravnini nosača, a karakterizirani su relativno velikom vitkošću i značajnom prošupljenošću hrpta. U radu su opisana istraživanja stabilnosti saćastih nosača iz aspekta otpornosti na bočno izvijanje, za koje se u novim propisima Eurokod 3 predlaže pojednostavljeni dokaz otpornosti na bočno izvijanje. Razmatrani su krovni saćasti nosači različitih raspona s IPE i HEA poprečnim presjecima. Potom su izrađeni 3D modeli istih nosača te je otpornost na bočno izvijanje izračunata predloženom metodom u Eurokodu 3 te metodom konačnih elemenata (MKE). Rezultati istraživanja pokazuju da se predloženom metodom provjere bočnog izvijanja saćastih nosača dobro procjenjuje njihova otpornost, budući da usporedba dobivenih vrijednosti pokazuje da razlike u otpornostima na bočno izvijanje nisu prelazile 5 % za slučaj promatranih nosača.

Ključne riječi: *3D MKE modeli saćastih nosača, saćasti nosači, bočno izvijanje, utjecaj prošupljenosti hrpta*

1 Introduction
Uvod

Castellated beams are made of rolled or welded I sections by cutting the web. After cutting, one of the halves is translated and welded for the other half. That provides the beam which has greater height than the original beam with the openings in the web. Shape of the openings depends on cutting pattern, and the result is a beam with increased bending resistance in major plane. It is possible to create a tapered castellated beam, which gives better efficiency, especially in case of greater spans.

Introduction of castellated beams is closely connected with the development of electrical welding technology in steel construction. The earliest known castellated beams were designed and manufactured in the early 1930's by the Skoda factories in Pilsner (Czech Republic) as roof beams with free span of 12 meters. The initial impetus for the elastic calculation method was given by František Faltus (1942. in the former Czechoslovakia), and the plastic calculation method was introduced in the early 1970's.

Because of the greater height of the castellated beams compared to original beams from which they have been made, these beams have greater bending resistance and the openings in the web simplify the work of the installer and the electrician, since taking pipes across beams presents no problem. Disadvantages of the castellated beams are less fire resistance compared to the plate girders and the need of reinforcement when a beam is subjected to the substantial concentrated loads or when they are used as a continuous beam across several supports.

Web openings may be circular or polygonal, regular or irregular (Figure 2), and Eurocode [1] gives geometrical references for their dimensions and location:

$$\begin{aligned} &h_0 \le 0{,}75 \cdot d_w \text{ - for polygonal web openings} \\ &h_0 \le 0{,}80 \cdot d_w \text{ - for circular web openings} \\ &d_1 = d_2 \ge 0{,}10 \cdot d_w \\ &a_0 \le h_0 \\ &b_0 = w \\ &0{,}25 \cdot a_0 \le w \le 0{,}50 \cdot a_0 \,. \end{aligned} \quad (1)$$

For castellated beams the following modes of failure should be checked:

- shear failure of perforated cross-section
- flexural failure of perforated cross-section
- Vierendeel mechanism around web openings
- yielding or buckling failure due to the moments in the web-posts
- shear failure of web-post or web-post weld.

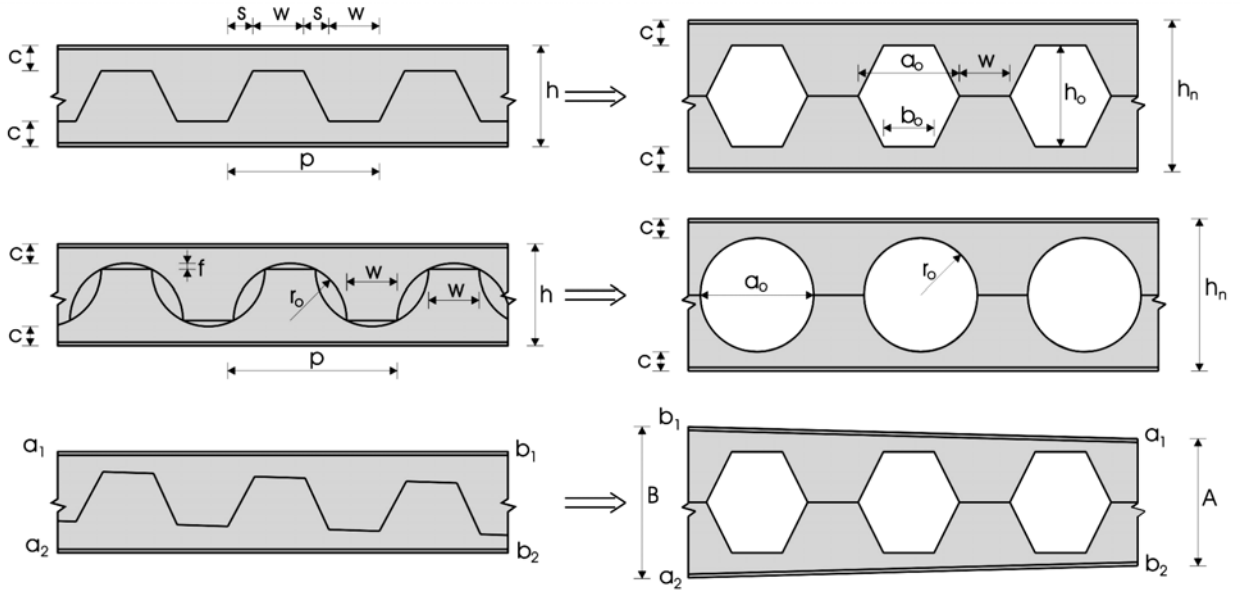


Figure 1. Castellated beams
Slika 1. Saćasti nosači

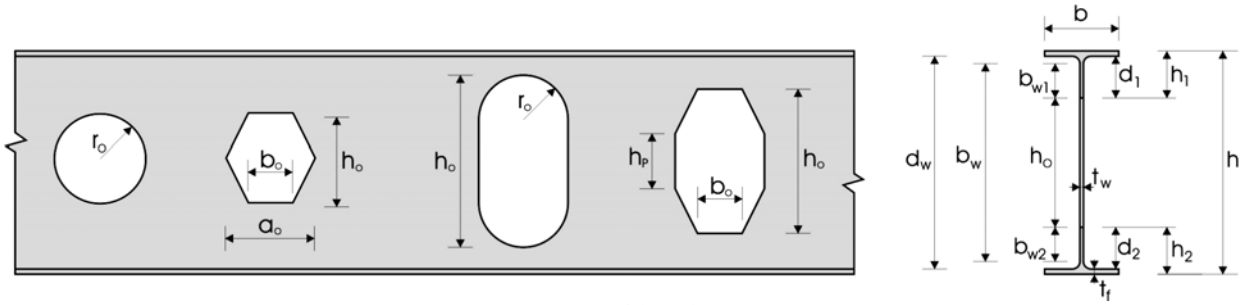


Figure 2. Types of the web openings
Slika 2. Vrste otvora u hrptu

Eurocode 3, [1] gives detailed expressions for analysis of the castellated beams according to the modes of failure mentioned before. It should be emphasized that it is necessary to distinguish primary moment on castellated beams, which is the conventional bending moment on the beam cross-section, and secondary moment (also known as the Vierendeel moment) which results from the action of shear force in the tee sections over the horizontal length of the opening. Therefore the concept of Vierendeel beam proved to be a good model for the calculation of internal forces of the castellated beams.

There are also special requests regarding the classification of cross-sections: for the unperforated cross-section of the beam remote from the web openings Class 1 or Class 2 are required for flanges and a Class 1, 2 or 3 for the web. Unperforated cross-sections with a Class 3 web and Class 1 or Class 2 flanges may be treated as effective Class 2 cross-section with an effective web. The portion of the web in compression should be replaced by an element of $20\varepsilon t_w$ adjacent to the compression flange, with another element of $20\varepsilon t_w$ adjacent to the plastic neutral axis of the effective cross-section.

Because the magnitudes of the required plastic hinge rotations for the development of the Vierendeel mechanism are small, it may be assumed to be sufficient for unstiffened webs of tees above and below web openings to be Class 1, 2 or 3. Webs of stiffened tees should also be Class 1, 2 or 3 after stiffening. Reinforcement of the stiffeners forming part of the cross-section of stiffened tees should be Class 1 or Class 2. The portions of the web forming the steams of unstiffened tees above and below unstiffened web openings may be classified allowing for the restraint of the adjacent portions of the web, as follows:

- for Class 2:

$$l_0 \le 32\varepsilon t_w \text{ or } b_{wc} \le \frac{10\varepsilon t_w}{\sqrt{1-\left(\frac{32\varepsilon t_w}{l_0}\right)^2}} \tag{2}$$

- for Class 3:

$$l_0 \le 36\varepsilon t_w \text{ or } b_{wc} \le \frac{14\varepsilon t_w}{\sqrt{1-\left(\frac{36\varepsilon t_w}{l_0}\right)^2}} \tag{3}$$

where:
b_{wc} - is the tee steam outstand b_{w1} or b_{w2} indicated in figure 2.
l_0 - is the length defined in figure 3.

This paper investigates the possibility of stability loss due to lateral buckling of castellated beams, because they are relatively slender and they have web openings, which have an influence on their resistance.

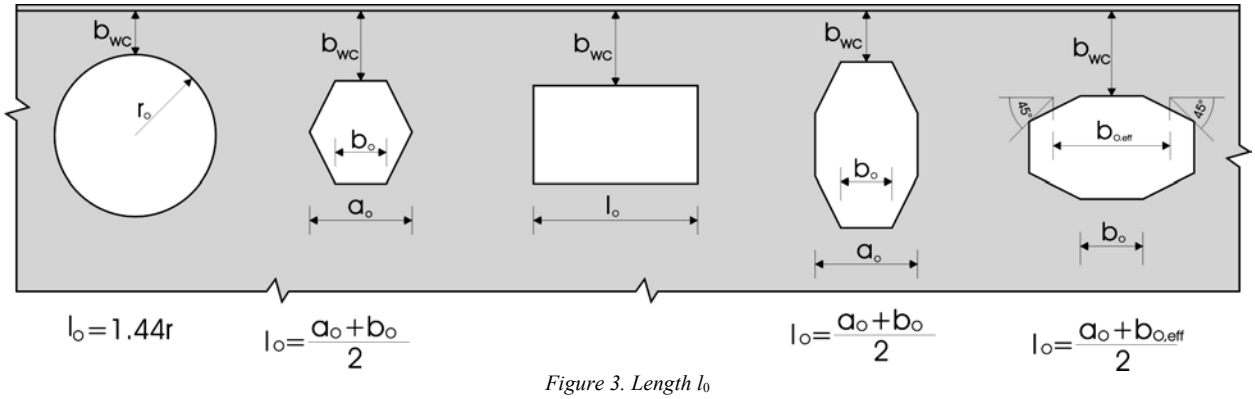


Figure 3. Length l_0
Slika 3. Duljina l_0

2
Formulation of research objective
Formulacija cilja istraživanja

Lateral buckling is the problem of stability loss and bearing capacity failure before plastification of cross-section. Beams primarily constructed for taking forces in their major plane are sensitive to lateral buckling, until they have smaller bending stiffness around minor axis (I and U cross-sections).

Elastic critical moment for lateral buckling M_{cr} according to elastic theory represents minimum moment which causes lateral buckling and according to [2] for biaxial symmetrical cross-section can be calculated from the following expression:

$$M_{cr} = C_1 \frac{\pi^2 \cdot E \cdot I_z}{(k \cdot L)^2} \cdot \left[\sqrt{\left(\frac{k}{k_w}\right)^2 \cdot \frac{I_w}{I_z} + \frac{(k \cdot L)^2 \cdot G \cdot I_t}{\pi^2 \cdot E \cdot I_z} + (C_2 \cdot z_g)^2} - C_2 \cdot z_g \right] \tag{4}$$

where:
C_1, C_2 - coefficients depending on the loading and end restraint conditions
factor k refers to end rotation on plan.
factor k_w refers to end warping.
I_z - is the second moment of area
I_w - is the warping constant
I_t - is the torsion constant
G - is the shear modulus
z_g - is the distance between the point of load application and the shear center.

Results of the expression (4) are very sensitive for the factors k and k_w whose geometrical interpretation is shown in Figure 4.

RESTRAINT TYPE	IDEALISED END RESTRAINT	k	k_w
I Simply supported		1.0	1.0
II Warping prevented		1.0	0.5
III Lateral bending prevented		0.5	1.0
IV Fixed end		0.5	0.5

Figure 4. Plan view illustrating idealised end restraints
Slika 4. Grafička interpretacija faktora k i k_w

Lateral buckling resistance $M_{b,Rd}$ must be bigger than calculated bending moment $M_{y,Ed}$, is given by multiplying reduction factor χ_{LT} with cross-section bending resistance:

$$M_{b,Rd} = \chi_{LT} \cdot \frac{W_y \cdot f_y}{\gamma_{M1}} \tag{5}$$

Reduction factor for lateral buckling χ_{LT} can be determined according to expression:

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \quad \text{but} \quad \chi_{LT} \le 1{,}0 \tag{6}$$

where:

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y \cdot f_y}{M_{cr}}}, \tag{7}$$

$$\Phi_{LT} = 0{,}5 \cdot \left[1 + \alpha_{LT}\left(\overline{\lambda}_{LT} - 0{,}2\right) + \overline{\lambda}_{LT}^2\right], \quad (8)$$

α_{LT} - imperfection factor corresponding to the appropriate buckling curve which depends on the type and geometrical characteristics of cross-section (W_y depends on the class of cross-section).

Because of the relatively high slenderness, castellated beams, applied on greater spans and significant web openings (even to 75 % in relation to the full web), can be considered as the beams sensitive to lateral buckling. According to [1] the lateral-torsional buckling resistance should be checked using the cross-sectional properties relevant to the mid-length of openings, i.e. netto cross-section in the middle of the opening where the smallest geometrical characteristics should be taken into lateral buckling resistance checking. It means that validation of lateral buckling resistance is the same for castellated beams and standard plate girders, but with reduction of geometrical characteristics castellated beam cross-section.

So the research objective is estimation of accuracy for this simplified approach to lateral buckling resistance of castellated beams, because that is still a stability problem of the beam which doesn't have a uniform cross-sectional properties along the whole length.

3
Methods of problem research
Metode istraživanja problema

From the research objective it arises that it is necessary to determine the influence of reducing lateral buckling resistance of castellated beams by web openings in relation to plate girders with the same geometrical dimensions (same height, width and thickness of flange, and web thickness). Taking this into consideration calculations were performed for the elastic critical moment for lateral buckling according to [2], (M_{cr}^{EC}), and applying finite element method using linear buckling analysis, (M_{cr}^{FEM}).

Generally, linear buckling analysis starts with the equilibrium equation for the element subjected to a constant force:

$$[K]\{u\} = \{P\} \quad (9)$$

where:
$[K]$ - stiffness matrix
$\{u\}$ - displacement vector
$\{P\}$ - applied load vector.

In order to include the differential stiffness effects, the differential stiffness matrix, $[K_D]$, is introduced and this matrix for a typical element is a function of the state of stress in the element.

Hence, by introducing k_{cr} as an arbitrary scalar multiplier for another "intensity" of load, the equilibrium equation becomes:

$$\left([K] + k_{cr}[K_D]\right)\{u^*\} = \{k_{cr}P\}, \quad (10)$$

where $\{u^*\}$ is the modified displacement vector resulting from displacements under the intensity of load, and from differential stiffness effects.

By perturbing the structure slightly at a variety of load intensities, the "intensity" factor k_{cr} to create unstable equilibrium conditions, will be the factor to cause buckling:

$$\left([K] + k_{cr}[K_D]\right)\{u^*\} = 0\,. \quad (11)$$

This requires the solution of an eigenvalue problem:

$$\left[K - k_{cr}K_D\right]\{\varphi\} = 0\,. \quad (12)$$

The solution is nontrivial, (k_{cr} different from 0) only for specific values of k_{cr} that would make the matrix $\left[K - k_{cr}K_D\right]$ singular.

The product of the first load intensity factor or the first eigenvalue k_{cr} with the applied load would give the first buckling load of the model, and the eigenvector φ, the buckled shape.

The described method applied for elastic linear buckling analysis to determine a critical load intensity factor k_{cr} in this paper is performed using software package *Robot Millennium*. So, linear buckling analysis results are critical intensity factors k_{cr} for each shape (mode) of buckling.

The product of calculated load intensity factor and the load which the model is subjected to, gives critical load, which leads to stability loss (or buckling). When a critical load is known it is not difficult to calculate the value of critical bending moment. It depends on static system of the analyzed structural element, type and distribution of load.

This paper analyzes simply supported beams with various spans subjected to uniform load, and critical bending moments can be calculated using the following expression:

$$M_{cr}^{FEM} = k_{cr}\frac{q \cdot L^2}{8}, \quad (13)$$

where:
k_{cr} - critical load intensity factor
q - uniform load
L - span of castellated beam.

For this research two groups of rolled profiles are chosen - IPE and HE-A, considering different geometrical characteristics of cross-section. It is assumed that castellated beams will be used like roof beams so they are designed taking into account the loads of roof cover ("sandwich" panel) and snow.

Distance between the beams made of IPE profiles is 2,20 m, and those made of HE-A profiles is 2,50 m. Analysis is performed on castellated beams whose height is 50 % higher than the original IPE and HE-A profiles. Spans and chosen cross-sections of the analyzed beams are shown in Table 1.

Table 1. Castellated beams
Tablica 1. Saćasti nosači

CROSS SECTION	Original profile height	Height of castellated beam	Span of beam	Distance between beams	Width (a_0) and height (h_0) of opening	Distance of openings (w)
	mm		m		mm	
IPE 240	240	360	8,0	2,20	253,5	126,75
IPE 300	300	450	10,0	2,20	317,0	158,5
IPE 360	360	540	12,0	2,20	380,0	190,0
HE 220A	210	315	8,0	2,50	213,0	106,5
HE 260A	250	375	10,0	2,50	246,0	123,0
HE 300A	290	435	12,0	2,50	296,0	148,0

All beams are subjected to uniform load on whole length, and the load is located on top (compressed) flange. Each of these six models is analyzed for the three different cases of lateral supporting i.e. for the cases:

- k = 1,0 and k_w = 1,0,
- k = 0,5 and k_w = 1,0,
- k = 1,0 and k_w = 0,5.

Total of 18 3D models are created in *Robot Millennium* using finite elements type *SHELL*. Only half of beam was modeled and on one end were used symmetry boundary conditions, and on the other end were boundary conditions of the case, which we want to model.

It was needed to pay special attention to boundary conditions modeling for different cases of lateral supporting because models are very sensitive to this problem. Even relatively small corrections of boundary conditions can give significant differences in calculation results. Plate girders were modeled as "control models", with the same geometrical dimensions as analyzed castellated beams. For these beams it was possible to apply conventional "manual" lateral buckling resistance verification according to [2]. It is important to accentuate that buckling length of analyzed beams in load plane is equal to beam length, but lateral buckling length out of load plane depends on factor k.

4
Calculation results
Rezultati proračuna

In Figures 5, 6 and 7 are shown boundary conditions of models and buckling shapes for all three cases of lateral supporting which were analyzed. In Table 2 are shown results of performed research - elastic critical moments of lateral buckling: using expression (13) - M_{cr}^{FEM}, and according to [2] - M_{cr}^{C} (expression (4)).

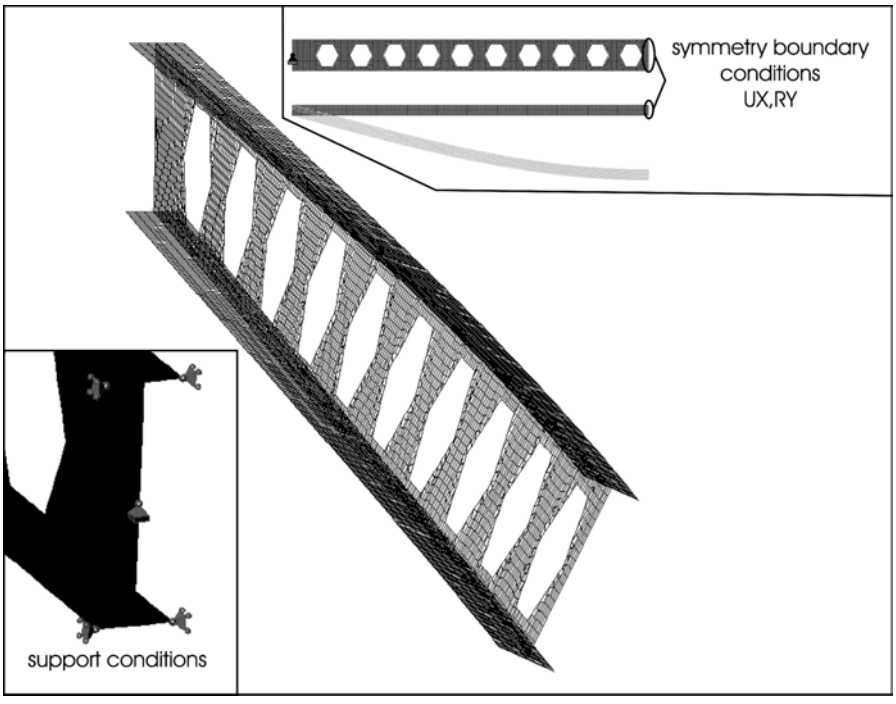


Figure 5. Boundary conditions and buckling shape for case k = 1,0 and k_w = 1,0
Slika 5. Rubni uvjeti i oblik izvijanja za slučaj k = 1,0 i k_w = 1,0

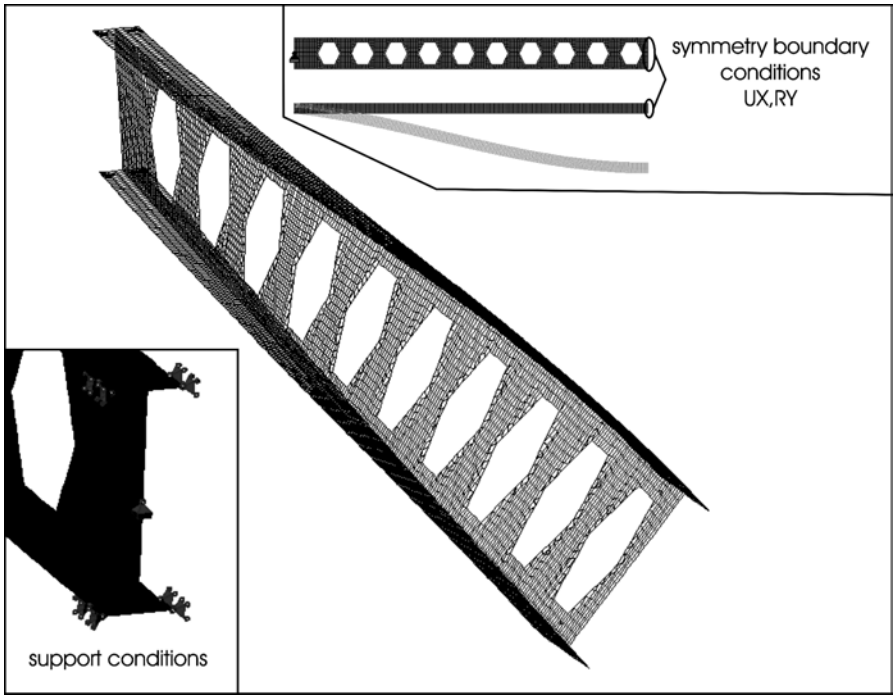


Figure 6. Boundary conditions and buckling shape for case k = 0,5 and k_w = 1,0
Slika 6. Rubni uvjeti i oblik izvijanja za slučaj k = 0,5 i k_w = 1,0

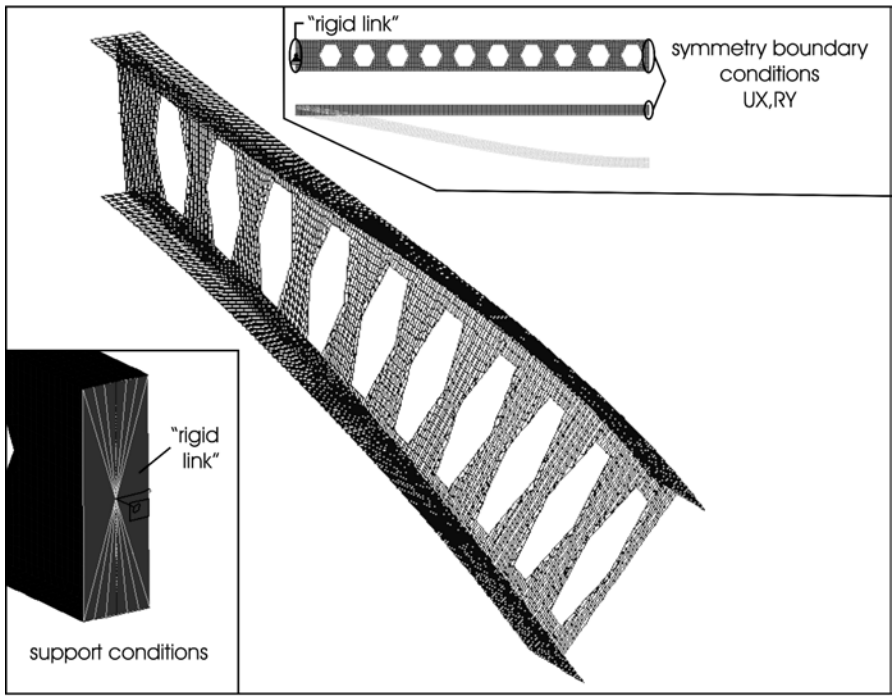


Figure 7. Boundary conditions and buckling shape for case k = 1,0 and k_w = 0,5
Slika 7. Rubni uvjeti i oblik izvijanja za slučaj k = 1,0 i k_w = 0,5

For the interpretations of the given results it is necessary to comment the results of lateral buckling critical moments only for castellated beams first, and then compare them with the calculated values of critical moments for the castellated and appropriate plate girders.

Graphical illustrations of the appropriate values of critical moments for castellated beams are shown in Figures 8, 9 and 10. It can be noticed that "manual" procedure according to [2] gives mostly conservative estimation of the critical moment, except for simply supporting case (k =1,0 and k_w =1,0) when values are practically equal (but it is needed to say that these boundary conditions are very rare in practice). For all

other lateral supporting cases FEM 3D models give larger values, whereas for (k = 0,5; k_w = 1,0) case differences between critical moments are smaller and they are in the range of 6 %, and for (k = 0,5; k_w = 1,0) case these differences are greater (in the range of 10 %). This last lateral supporting case (k = 0,5; k_w = 1,0) is hard to model precisely, and in this paper two ways of modeling are used which gave almost the same results. The first way was by using the option "rigid links" which gives a possibility to set the same values of displacement and rotations for all points in specified cross-section (Figure 7). The second way is to replace the relatively short area of supporting edge with the BEAM element, which is a more precise way to control boundary conditions (regarding 3D SHELL model). Generally, greater deviations are noticed on IPE cross-sections with regard to HE-A cross-sections. That can be associated with relative differences between geometrical characteristics of cross-sections (elastic and plastic) of those section types through a web opening ("netto") and out of opening ("brutto"), where that relation on IPE section is twice as high.

Table 2. Elastic critical moments of lateral buckling for plate girders and castellated beams calculated using Robot Millennium and Eurocode 3
Tablica 2. Elastični kritični momenti bočnog izvijanja za punostjene i saćaste nosače dobivenih iz Robot Milleniuma i prema Eurokod 3 propisima

CROSS SECTION		PLATE GIRDERS			CASTELLATED BEAMS		
		k = 1,0 k_w = 1,0	k = 0,5 k_w = 1,0	k = 1,0 k_w = 0,5	k = 1,0 k_w = 1,0	k = 0,5 k_w = 1,0	k = 1,0 k_w = 0,5
		kNm					
IPE 240	M_{cr}^{EC}	27,92	44,56	39,55	25,72	40,87	37,87
	M_{cr}^{FEM}	27,57	46,27	44,17	26,12	44,37	42,85
IPE 300	M_{cr}^{EC}	43,18	68,55	64,27	39,43	62,25	61,51
	M_{cr}^{FEM}	42,55	69,03	71,52	40,25	66,12	69,48
IPE 360	M_{cr}^{EC}	63,89	101,58	93,82	58,78	93,00	90,02
	M_{cr}^{FEM}	63,06	103,05	104,86	59,77	98,85	101,93
HE 220A	M_{cr}^{EC}	118,44	185,06	201,82	114,89	179,17	199,42
	M_{cr}^{FEM}	115,33	189,08	225,11	111,93	184,50	221,94
HE 260A	M_{cr}^{EC}	169,23	264,42	288,72	165,26	257,82	286,02
	M_{cr}^{FEM}	164,29	268,95	322,26	159,08	262,88	318,09
HE 300A	M_{cr}^{EC}	235,96	368,83	401,43	229,73	358,47	397,21
	M_{cr}^{FEM}	229,68	377,95	448,57	223,55	369,42	442,71

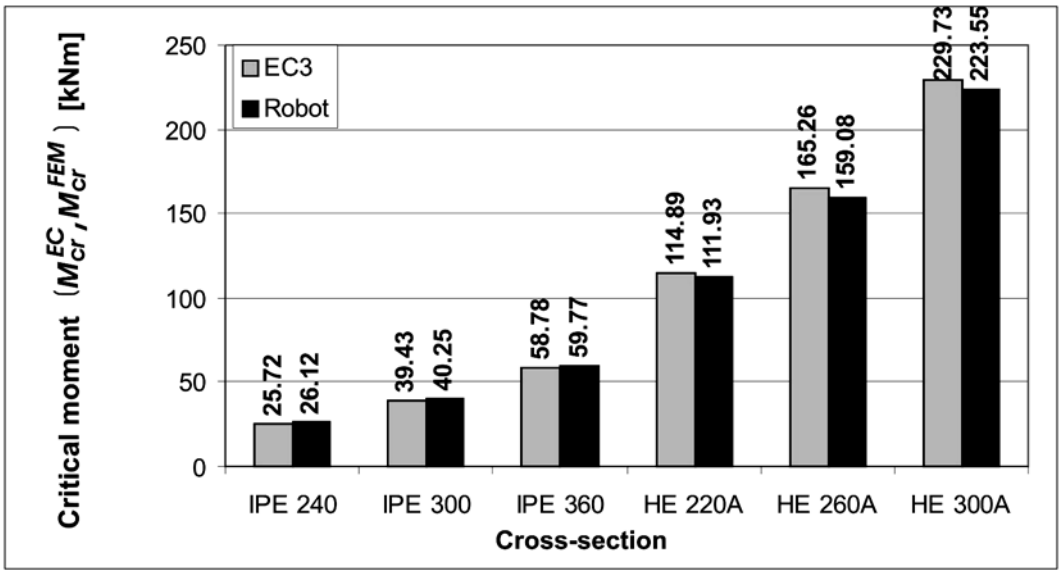


Figure 8. Critical moments of castellated beams for boundary conditions k=1,0 and k_w=1,0
Slika 8. Kritični momenti za saćaste nosače za rubne uvjete k = 1,0 i k_w=1,0

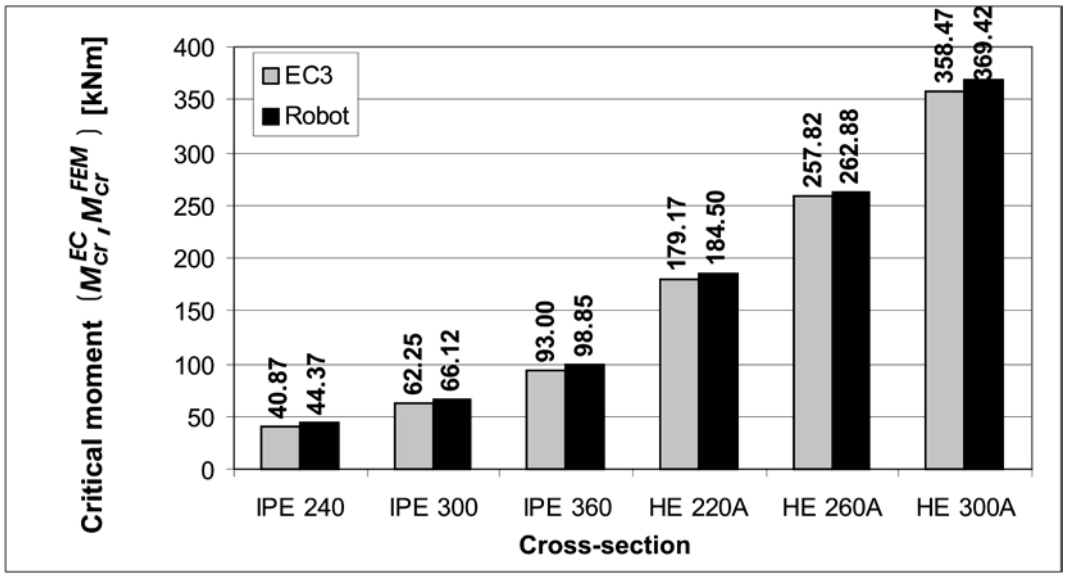


Figure 9. Critical moments of castellated beams for boundary conditions k=0,5 and k_w=1,0
Slika 9. Kritični momenti za saćaste nosače za rubne uvjete k = 0,5 i k_w = 1,0

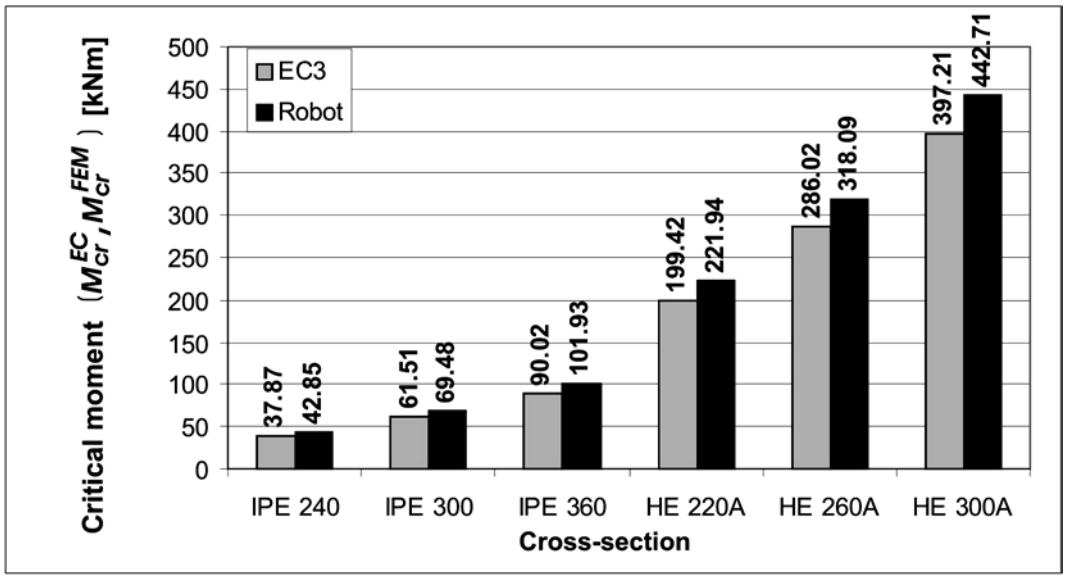


Figure 10. Critical moments of castellated beams for boundary conditions k=1,0 and k_w=0,5
Slika 10. Kritični momenti za saćaste nosače za rubne uvjete k =1,0 i k_w = 0,5

Next step is comparison of the critical moments for the castellated and appropriate plate girders, (Figure 11, 12 and 13). Generally, we can conclude that there is a very good correspondence between the results for castellated and plate girders in relation to the applied method of critical moments calculations - "manual" according to [2] or from FEM models. Also it is shown that differences in moments for castellated and appropriate plate girders are relatively small, and they are for the beams with IPE sections in the range of 5 %, while for the beams with HE-A profiles (stiffener about minor axis) they are even smaller and they are max. 3 %.

The research results show that influence of the web openings is relatively small from the aspect of the analyzed problem. The research results fit well into previously published researches [4].

A good illustration is example of fictive configuration castellated beams shown in Figure 14. This figure shows variation of load intensity critical factor k_{cr} for various castellated beams made of IPE 360 profile for which web openings are progressively increased. Starting model is plate girders for which web opening is equal to zero, and final case is the model of fictive beam with two openings which are stretched along the whole halfspan. Beside the beams with polygonal (hexagonal) web openings the beams with rectangular web openings are also analyzed. Even here illustrated exact example of beam shows the general trend. In the diagram the area which EC defines as maximum allowed web opening the width is shadowed (expression (1)).

In the diagram it can be seen that in this area differences between critical moments are negligible, especially for the beams with polygonal web openings, which have greater lateral buckling resistance than those with rectangular web openings (better "restrains" compressed tee).

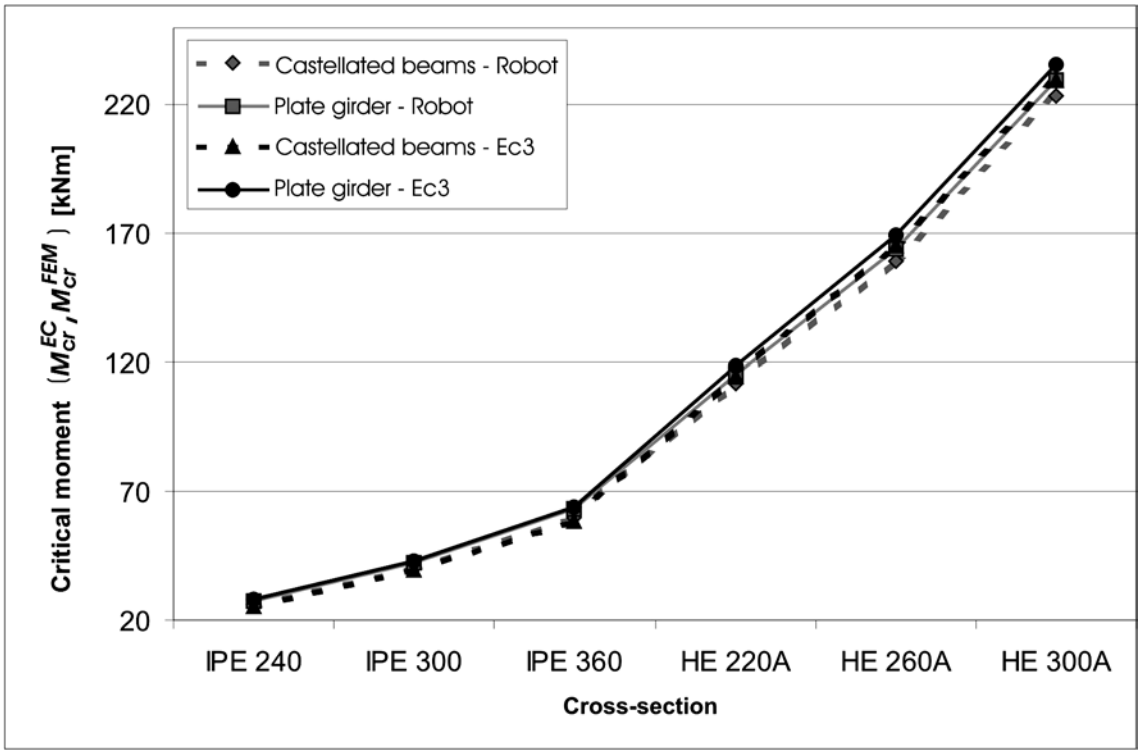


Figure 11. Critical moments of lateral buckling for plate girders and castellated beams calculated using Robot Millennium k =1,0 and k_w =1,0
Slika 11. Kritični momenti za saćaste i pločaste nosače dobiveni na modelima u Robotu Millennium za rubne uvjete k =1,0 i k_w =1,0

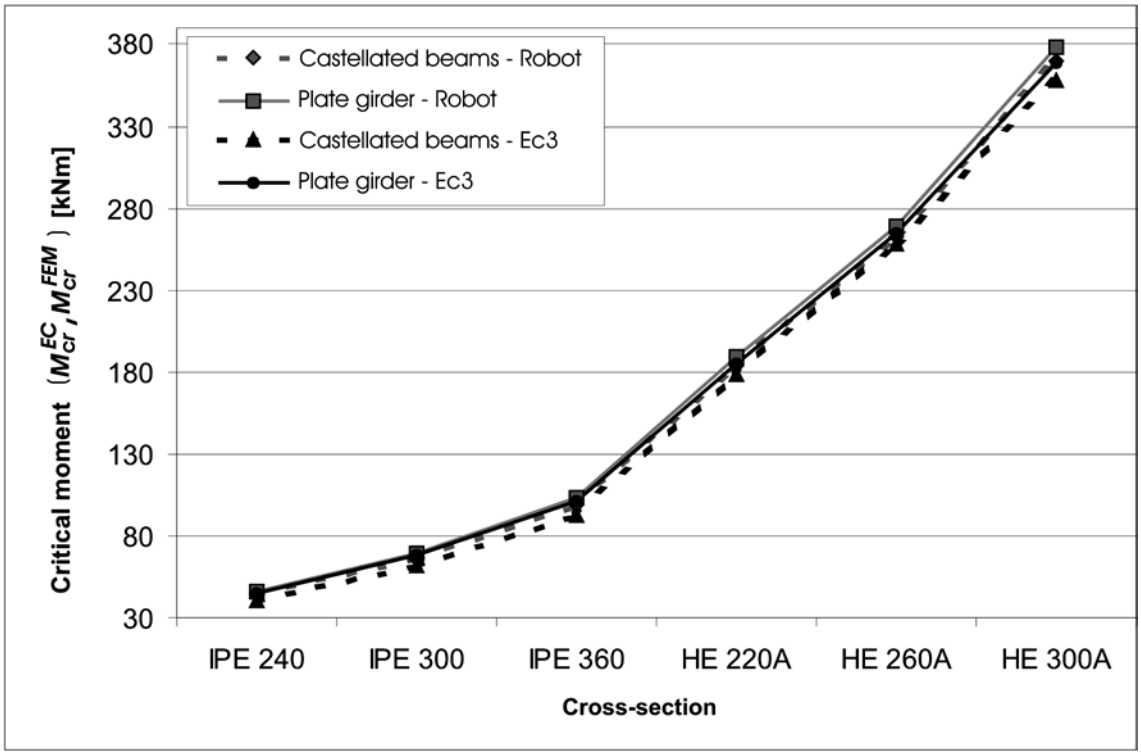


Figure 12. Critical moments of lateral buckling for plate girders and castellated beams calculated using Robot Millennium k = 0,5 and k_w = 1,0
Slika 12. Kritični momenti za saćaste i pločaste nosače dobiveni na modelima u Robotu Millennium za rubne uvjete k = 0,5 i k_w = 1,0

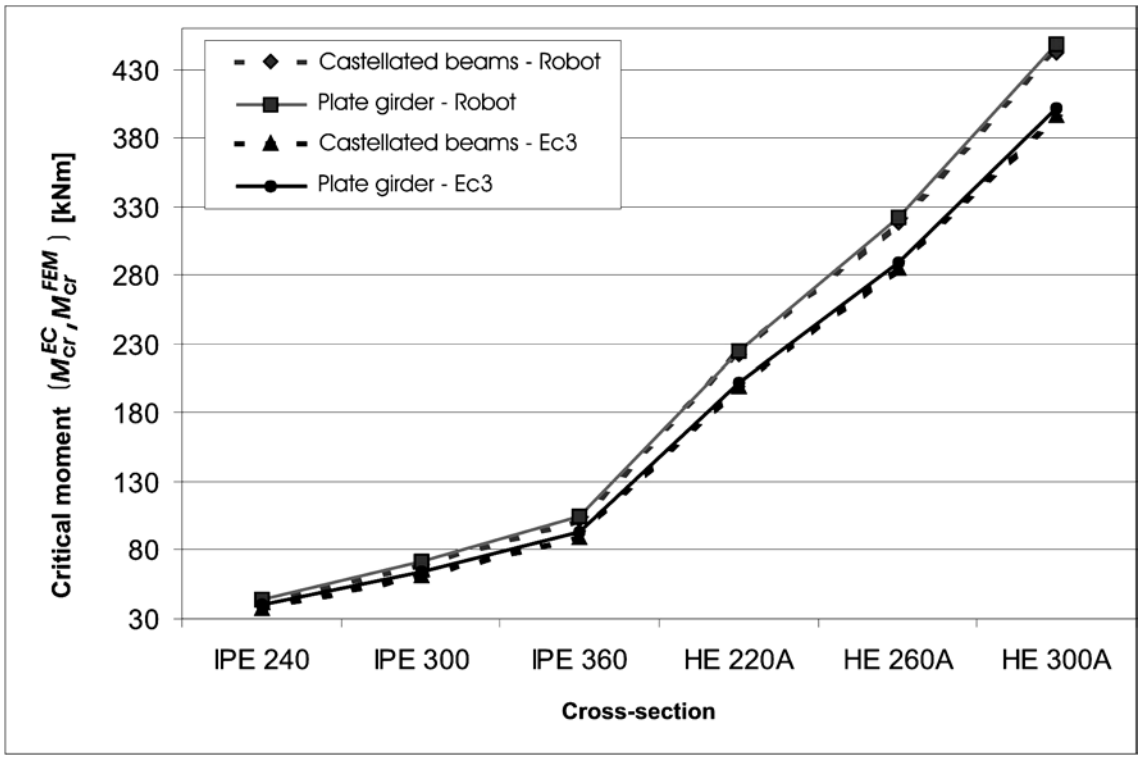


Figure 13. Critical moments of lateral buckling for plate girders and castellated beams calculated using Robot Millennium k =1,0 and k_w = 0,5
Slika 13. Kritični momenti za saćaste i pločaste nosače dobiveni na modelima u Robotu Millennium za rubne uvjete k =1,0 i k_w = 0,5

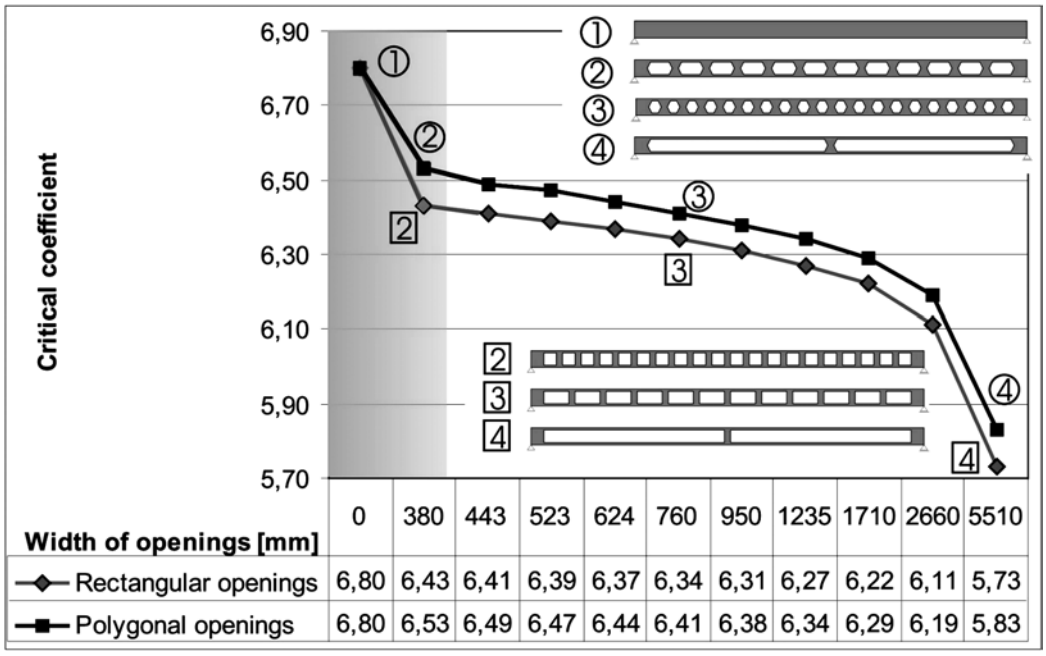


Figure 14. Alteration of critical factors k_{cr} for different configuration "castellted" beams
Slika 14. Promjena kritičnih koeficijenta k_{cr} za različite konfiguracije "saćastih" nosača

5
Conclusion
Zaključak

In this paper the research into the castellated beams stability from the aspect of the lateral buckling resistance is described, for which new Eurocode 3 suggests simplified lateral buckling resistance verification. The suggested method for lateral buckling verification can be performed as well as the method for standard plate girders but with reduction of geometrical characteristics of cross-section so that geometrical characteristics of cross-section through openings should be used in verification.

The research results show that web openings of the castellated beams have little influence on determining of M_{cr}. Values of lateral buckling critical moments obtained on 3D FEM models, depending on lateral boundary conditions, give practically the same or a little higher values (up to 10 %) in relation to "manual" calculation method.

It is very indicative that the parallel FEM analysis of the castellated and appropriate plate girders shows that differences in obtained lateral buckling critical moments are practically negligible (up to max. 5 %), so it can be concluded that the simplification of critical moments which can be calculated is the same as that of the critical moments for standard plate girders but taking into consideration geometrical characteristics of the cross-section through openings. This method gives good estimation of critical moment. The IPE or slender beams, where the relation "netto" and "brutto" of cross-section geometrical characteristics is somewhat higher, give some greater deviation, while deviation on HE-A profiles is practically negligible.

Using complex 3D FEM models of castellated beams it is possible to determine lateral buckling resistance of castellated beams more precisely. It can be accomplished by variation of relevant parameters: standard supporting conditions, lateral supporting conditions, detailed model of beam geometry with possible transverse stiffeners and intermediate supporting along span, introducing possible geometry and material imperfections, various load conditions etc. So in the next researches FEM models of continuous beams can be made with various lateral support conditions and load type.

6
References
Literetura

[1] CEN - European Committee for Standardization (1992): Eurocode3 - Design of steel structures - Part 1-1: General rules and rules for buildings, ENV 1993-1-1:1992/A2:1998, Annex N

[2] CEN - European Committee for Standardization (2003): Eurocode3 - Design of steel structures - Part 1-1: General rules and rules for buildings, prEN 1993-1-1:2003 (E)

[3] Robot Millennium v. 17.0, User's Manual, UPDATE November 2003

[4] Demirdjian, S. Stability of Castellated beam webs, Master Thesis, 1999., Department of Civil Engineering and Applied Mechanics, McGill University Montreal, Canada.

[5] Fuller, I. J. Development of an Object Oriented Handbook and a Numerical Methods Application for Steel Structures, Masther Thesis, Department of Civil and Environmental Engineering, Carleton University, Ottawa, Ontario, Canada, September 1999.

[6] Androić, B.; Dujmović, D.; Džeba, I. Metalne konstrukcije 1, Institut građevinarstva Hrvatske, Zagreb, 1994.

[7] Tkalčević, V.; Džeba, I.; Androić, B. Proračun saćastih nosača prema Eurokodu 3, Građevinar 58 (2006) 9, str. 709-716.

[8] Deljak, M. Bočno izvijanje prošupljenih nosača, Diplomski rad, Sveučilište J. J. Strossmayer u Osijeku, Građevinski fakultet, 2006.

[9] Markulak, D. Čelične konstrukcije – interna skripta – dio 1, Sveučilište J. J. Strossmayer u Osijeku, Građevinski fakultet, Osijek, 2004.

Author's Address (Adresa autora):

Ivan Radić, dipl. ing. građ.
Sveučilište J. J. Strossmayera u Osijeku
Građevinski fakultet
Drinska 16a, 31000 Osijek, Croatia
e-mail: radic@gfos.hr

prof.dr.sc. Damir Markulak, dipl. ing. građ.
Sveučilište J. J. Strossmayera u Osijeku
Građevinski fakultet
Drinska 16a, 31000 Osijek, Croatia
e-mail: markulak@gfos.hr