DESIGN ASPECTS OF REDUCED BEAM SECTIONS FOR IPE AND HEA EUROPEAN PROFILES

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1. ABSTRACT

Recently, in the EC-8-Part 3 the design of reduced beam section, RBS, for the strengthening of steel members was introduced. The RBS connection effectiveness was investigated widely using U.S. design and construction practices. Only limited data exists from European practice. The paper discusses the design aspects of reduced beam sections with radius cut using IPE and HEA profiles widely used as beam members in Europe. Sizing equations were proposed and also the available ductility of IPE and HEA RBS members was evaluated.

2. INTRODUCTION

The unexpected local brittle damage of beam to column connections of steel moment resisting frames in the Northridge (1994) and Kobe (1995) earthquakes generated concerns regarding the reliability of the current design practice and detailing of connections. Considering steel moment resisting frames, which were affected, one can remark various types of damage at the interface of the beam to column connections, cracks that developed at or near the beam bottom flanges also propagated into the column flange either vertically or horizontally [1]. The undesirable performance of beam to column connections, that showed unexpected low available ductility capacity, gave rise to extensive research activities all around the world [1,2,3]. Numerous solutions to the moment frame connection problem have been proposed [4]. Two key concepts have been developed in order to upgrade the inelastic deformation: strengthening the connection and / or weakening the beam or beams that frame into the connections.
An innovative concept - the Reduced Beam Section, RBS - (or dog bone section) was proposed by Plumier [5], was patented by the steel manufacturer ARBED, which after the

1994 Northridge earthquake waived all patent rights for the benefit of structural community. On the idea of weakening, this connection relies on the selective removal of beam flange material, trough constant, tapered or radius cut , to the beam column connection to reduce the cross sectional area, fig.1. The RBS behave like a fuse, shifting the dissipative zone from the column flange in a predetermined zone, thus protecting against early fracture. Extensive experimental [6,7,8] and analytical [9,10,11] studies demonstrated the effectiveness of this solution. Compared with strengthening concept, the using of RBS gives advantages such as increased inelastic capacity, satisfy in economical way the strong column-weak beam philosophy [10,12], avoids increased material and labor costs due to addition of strengthening plates, doubler plates, special welds.
Concluding the research efforts of the SAC program [1], recommendations for the design and detailing of RBS members were prescribed in FEMA 350 [4] and FEMA 351 [13]. In Europe, also, following the spirit of the above mentioned recommendations, in EC 8, Part 3 (Annex B), Draft 2003, [14] design of such type of connections are presented. It is noteworthy to mention the difference between the U.S. and European design practice, as well as the fact that there is no more experimental studies using the European profiles, except Plumier's studies [5].
In the paper a critical review is made considering the key parameters for the design of RBS connections especially for IPE and HEA European profiles. Furthermore, EC 8 Part 3 [14] proposed some ductility rotation limits without presenting clear information based on ductility capacity. In order to estimate and satisfy the ductility limits, using DuctRot M [15] computer program, which follows the concept of plastic collapse mechanism, an evaluation of the available rotation capacity was made classifying the IPE and HEA reduced beam sections according to the performance limits from [15]. This classification, also, could be useful for the new steel moment resisting frames exploiting the fact that such a section can easily be implemented and developed as a prefabricated element.

3. KEY PARAMETERS FOR THE DESIGN OF RBS CONNECTIONS

The principle of weakening the beam based on two aspects: firstly the sizing of the RBS cut should be made in order to limit the maximum beam moment that can developed at the column face, $M_{cf.sd}$; about 85% to 100% of the beam expected plastic moment, $M_{pl.Rd.be}$. Secondly, the expected plastic moment at the RBS zone, $M_{pl.RBS}$, after yielding and strain hardening, should be smaller than the expected plastic moment at the RBS zone, $M_{sd.RBS}$, *Fig.1*. The above mentioned criteria could be written:

$$M_{cf.sd} \leq (0.85 - 1.0)\, M_{pl.Rd.be} \tag{1}$$

$$M_{pl.RBS} \leq (0.90 - 0.95)\, M_{sd.RBS} \tag{2}$$

The first condition control the stress that can develop at the connection elements (welds, bolts), while the second one ensures the formation of the plastic hinge in the preselected zone. The configuration (shape, size, location) have an effect on the connection performance; various types have been proposed and tested with straight [5], variable [7] and radius cut [6], *Fig 1*. In *table 1* proposals for radius cut from FEMA 350 [4], which prequalified this shape, and EC 8, Part 3 (Annex B) [14] were presented. One can remark that the *a* and *b* values from EC 8, Part 3 (Annex B) are the average values as compared with FEMA 350, while for the *g, s* and *r* the same values were adopted.

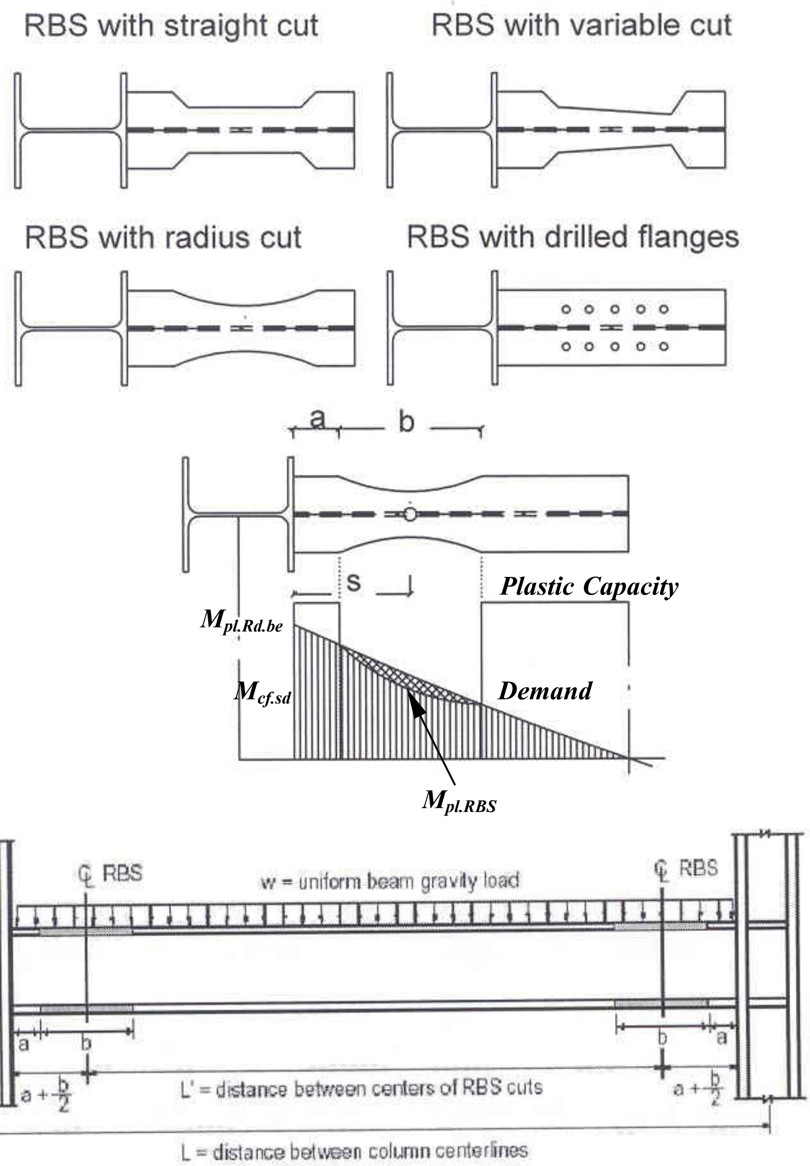

Figure 1: Shapes and conceptual design of reduced beam sections

FEMA 350 [4] / 351 [13]	EC 8, Part 3, Annex B [14]
$a = 0.50 – 0.75\ b_f$	$a = 0.60\ b_f$
$b = 0.65 – 0.85\ d_b$	$b = 0.75\ d_b$
$g = 0.20 – 0.25\ b_f$ (40 – 50 % b_f)	$g = 0.20 – 0.25\ b_f$ (40 – 50 % b_f)
$s = a + b/2$	$s = a + b/2$
$r = 4g^2 + b^2 / 8g$	$r = 4g^2 + b^2 / 8g$

Table 1: Geometrical characteristics of the reduced beam section

In order to consider for the European practice the validity and extrapolation of the experimental results obtained from the U.S. practice, one can remark the following issues:

a) Generally, different geometrical characteristics of the IPE and HEA profiles as compared with those tested in U.S are observed (beams from W 12, W 36 sections and columns from W 14 sections). However, the local buckling limits given in FEMA 350 and

351 were respected also for IPE and HEA sections. Furthermore, the local buckling values between those tested and IPE and HEA sections are in the same region.

b) Strong panel zone. The columns used in experiments have very thick webs as compared with those usually used in Europe. It is very important to have a strong panel zone forcing the formation of the plastic hinge in the weakening zone. Therefore, doubler plates should be used in case of IPE and HEA columns, until to obtain suitable experimental data.

c) Different material characteristics, introducing possible different random material variability and strain hardening effects.

d) All tests have been made with columns employing continuity plates. Thus, in the practical design the use of continuity plates is a need.

e) Different beam to column connection detail as compared with those used in European practice (welded beam flange to column flange and web bolted shear tab vs. extended end plate or fully welded).

f) Lack of experimental date using deep IPE columns, used in multistory structures in order to control the drift.

The experiments conducted by Plumier [5] using HEB columns and HEA beams demonstrated high inelastic deformation, however further investigations are needed, taking into account the relevant parameters from the European practice. For the dimensioning of the RBS connections with radius cut the following equations were proposed:

$$M_{pl.Rd.RBS} = \gamma_{ov.RBS} Z_{RBS} f_y \tag{3}$$

where $\gamma_{ov.RBS}$, the overstrength factor considering the random variability of steel mechanical properties and strain hardening effects, Z_{RBS} the plastic resistance modulus of the reduced section [$Z_{RBS} = Z_b - 2g\, t_f\, (d_b - t_f)$] and f_y, the nominal yield limit as given in EC 3. Generally, the overstrength factor in the RBS zone could be written:

$$\gamma_{ov.RBS} = \left(\frac{f_y + f_u}{2f_y}\right)\left(\frac{f_{y.max}}{f_{y.min}}\right) \tag{4}$$

The first factor considers the strain hardening effect while the second one the variability of steel mechanical properties. A value of 1.40 may be used for all cases, except where otherwise noted from experimental tests. In equation B.13.1 proposed in EC 8, Part 3 Annex B [14] the above mentioned effects do not considered. Beam expected plastic moment capacity is given from the following relation:

$$M_{pl.Rd.be} = \gamma_{ov} Z_b f_y \tag{5}$$

where γ_{ov}, the overstrength factor at the beam to column connection, Z_b the beam plastic resistance modulus and f_y, the nominal yield limit as given in EC 3. In relation 5 the overstrength parameter considers only the material random variability, $f_{y.max}$ / $f_{y.min}$, a value of 1.20 may be used except where otherwise noted from experimental tests. In this way checking the condition (1) we can obtain, in a safe mode, the stress reduction at the beam to column connection, through a proper cutout of beam flange, also minimizing the possible effect of increased stain hardening or strain rate in the RBS zone. In figure 2 the effect of overstrength factor on plastic moment capacities was presented. One can remark that the increasing of the overstrength factor in the RBS zone tends to equalize the beam and RBS moment capacities respectively, limiting the effectiveness of the RBS concept. In this way, the possible development of unexpected overstrength, at the beam to column connection elements, works in the part of safety.

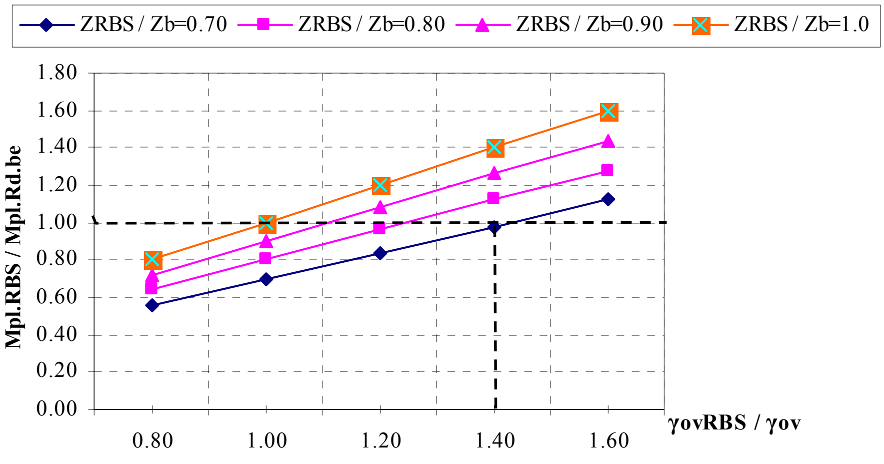

Figure 2: Influence of the overstrength factor on the plastic moment capacities

The moment at the column face could be written, Fig.1:

$$M_{cf.sd} = M_{pl.RBS}\left(1+\frac{2s}{L'}\right)+\frac{ws}{2}\left(L'-s\right) \tag{6}$$

where s, the distance from the column face and the center of RBS cut, L′, the distance between the center of RBS cuts. From the condition (1) and relation (6) we can obtain the proper cutout of beam flange taking into account the overstrength and stress reduction influence respectively:

$$g = \frac{Z_b f_y\left[\gamma_{ov.RBS}\left(1+{2s}/{L'}\right)-C_s\gamma_{ov}\right]+{ws}/{2}\left(L'-s\right)}{2t_f\left(d_b-t_f\right)f_y\gamma_{ov.RBS}} \tag{7}$$

where C_s, a coefficient to account the stress limitation at the beam column connection. A value between 0.85…1.0 may be used [4, 6]. Respecting the condition (2) and considering that $M_{pl.RBS} = C_{RBS}\, M_{sd.RBS}$, we can obtain another sizing formula for the proper reduction of beam flanges:

$$g = \frac{Z_b f_y\left[\gamma_{ov.RBS}-C_{RBS}\gamma_{ov}\right]-C_{RBS}\,{ws}/{2}\left(s-L_c\right)}{2t_f\left(d_b-t_f\right)f_y\gamma_{ov.RBS}} \tag{7′}$$

where the C_{RBS} is a coefficient to account the plastic moment reduction, a value between 0.95…0.90 may be used, L_c, is the distance between column faces. For design purposes the maximum value obtained from (7), (7′) should be taken. The influence of gravitational forces, overstentgth factor and stress limitation at the column face on the flange cutout, g, is presented, fig. 3, 4. From the plotted curves, it is observed that in case of high $\gamma_{ov.RBS}$ / γ_{ov} values in order to limit the stress at the beam column connection a reduction greater than 50% should be made; the upper limit for flange cutout is 50% [4, 13, and 14]. In sizing the RBS zone the values of gravity loads should be considered. Furthermore, significant gravity loads can reduce the ductility capacity and/or shift the location of the plastic hinge [4, 9]. Therefore, in EC 8 Part 3 a clause should be introduced regarding the effect of gravity load as given in FEMA 350 for cases when flexural demand on the beam due to gravity loads exceed 30% of the beam plastic capacity. Another important issue is the effect of weakening on the global stiffness of frame. An increasing of 4 – 7 % of the elastic drift was proposed [4]. Further analytical investigation is needed using different beam, column combinations with IPE, HEA, HEB sections.

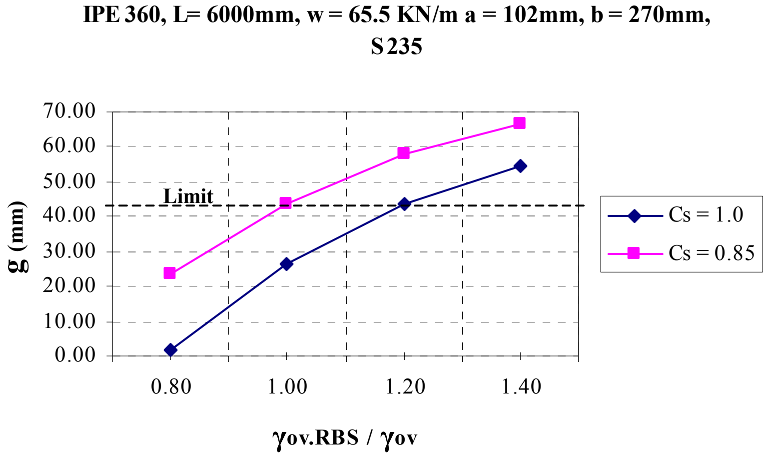

Figure 3: Influence of overstrength and stress limitation on flange reduction

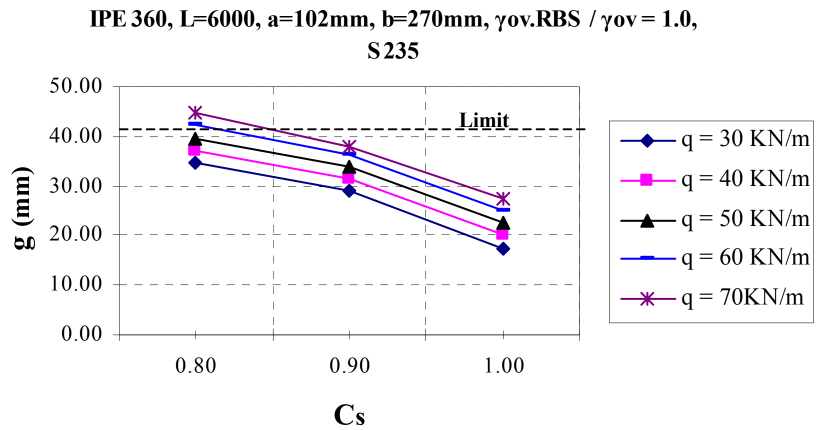

Figure 4: Influence of gravity loads on flange reduction

3. EVALUATION OF THE AVAILABLE DUCTILITY OF RBS CONNECTION

For the evaluation of the available ductility of steel members the DuctRot M computer program was used, which based on the plastic collapse mechanism [15]. As an inelastic capacity index the rotation capacity was considered:

$$\mu_{\theta.av} = \frac{1}{\gamma_M}\left(\frac{\theta_u}{\theta_p} - 1\right) \tag{8}$$

where γ_M, safety factor that accounts model uncertainties, influence of seismic loading, influence of gravity loads, γ_M = 1.30 x 1.30 x 1.20 = 2.0, respectively, coefficients taken from parametrical investigations, θ_u, the ultimate plastic rotation and θ_p, the plastic rotation corresponding to the first plastic hinge. The classification of IPE and HEA beams was made using the member ductility criterion [15], also introducing a classification for different performance levels:

High Ductility (H) – Near Collapse (NC) → $\mu_{\theta.av} > 7.50$

Medium Ductility (M) – Sever Damage (SD) → $4.50 \le \mu_{\theta.av} \le 7.50$

Low Ductility (L) – Repairable Damage (RD) → $1.50 \le \mu_{\theta.av} \le 4.50$

In table 2 the ductility levels for IPE and HEA beams were evaluated in order to inform the designer about the inelastic performance of RBS elements. The unreduced beam elements upgraded using the RBS concept from the lower class of ductility (performance) to the upper one. Furthermore, a reduction of 50 % of the flanges, which is the upper limit, ensures high inelastic performance, even in the great spans.

Profil	L	Unreduced Beam	g (mm) $0.20b_f$	Ductility Level	Perform. Level	g (mm) $0.25b_f$	Ductility Level	Perform Level
IPE 330	6000	M	32	M	SD	40	H	NC
	8000	L		M	SD		M	SD
	10000	L		L	RD		M	SD
IPE 360	6000	M	34	M	SD	42	H	NC
	8000	M		M	SD		M	SD
	10000	L		L	RD		M	SD
IPE 400	6000	M	36	M	SD	45	H	NC
	8000	M		M	SD		H	NC
	10000	M		M	SD		M	SD
IPE 450	6000	M	38	M	SD	47	H	NC
	8000	M		M	SD		H	NC
	10000	L		M	SD		M	SD
IPE 500	6000	H	40	H	NC	50	H	NC
	8000	M		M	SD		H	NC
	10000	M		M	SD		M	SD
IPE 550	6000	H	42	H	NC	52	H	NC
	8000	M		M	SD		H	NC
	10000	M		M	SD		H	NC
IPE 600	6000	H	44	H	NC	55	H	NC
	8000	H		H	NC		H	NC
	10000	M		M	SD		H	NC

Profil	L	Unreduced Beam	g (mm) $0.20b_f$	Ductility Level	Perform. Level	g (mm) $0.25b_f$	Ductility Level	Perform Level
HEA 240	6000	L	48	M	SD	60	M	SD
	8000	L		L	RD		M	SD
	10000	L		L	RD		L	RD
HEA 260	6000	L	52	M	SD	65	M	NC
	8000	L		M	SD		M	SD
	10000	L		L	RD		M	SD

Table 2: Available ductility of RBS beams

Profil	L	Unreduced Beam	g (mm) $0.20b_f$	Ductility Level	Perform. Level	g (mm) $0.20b_f$	Ductility Level	Perform Level
HEA 280	6000	L	56	M	SD	70	M	SD
	8000	L		M	SD		M	SD
	10000	L		L	RD		M	SD
HEA 300	6000	L	60	M	SD	75	M	SD
	8000	L		M	SD		M	SD
	10000	L		M	SD		M	SD
HEA 320	6000	M	60	M	SD	75	H	NC
	8000	L		M	SD		H	NC
	10000	L		M	SD		M	SD
HEA 340	6000	M	60	M	SD	75	H	NC
	8000	M		M	SD		H	NC
	10000	L		M	SD		M	SD
HEA 360	6000	M	60	H	NC	75	H	NC
	8000	M		M	SD		H	NC
	10000	L		M	SD		M	SD
HEA 400	6000	M	60	H	NC	75	H	NC
	8000	M		H	NC		H	NC
	10000	M		H	NC		H	NC
HEA 450	6000	M	60	H	NC	75	H	NC
	8000	M		H	NC		H	NC
	10000	M		H	NC		H	NC
HEA 500	6000	H	60	H	NC	75	H	NC
	8000	M		H	NC		H	NC
	10000	M		H	NC		H	NC
HEA 550	6000	H	60	H	NC	75	H	NC
	8000	M		H	NC		H	NC
	10000	M		H	NC		H	NC

Table 2: Available ductility of RBS beams

4. CONCLUSIONS

The paper is focused on the design aspects of RBS connection accounting the European practice (steel sections, design codes). Basic design relations, as compared with those given in EC 8, Part 3, Annex B, were proposed in order to size the RBS zone in a effective mode, accounting basic parameters as strain hardening, stress limitation, material random variability. Furthermore, the current informative design specifications do not express explicitly the inelastic deformation. An evaluation of the available ductility for IPE and HEA members was proposed, facilitating the choice of the designer for a proper flange cutout associated with the expected inelastic performance. However, concerning the strain hardening effect and deformation capacity no definitive conclusions can be drawn due to limited of experimental data using the European practice. In this way further investigation are needed through experimental research, considering subassemblies with IPE column and

IPE beams, HEA columns and IPE beams, HEB columns and IPE beams, combinations widely used in practice, with different connection configurations (bolted connections, fully welded connection with or without continuity plate, with or without doubler plates).

5. BIBLIOGRAPHY

[1] SAC 96-03, Interim guidelines. FEMA 276/A, SAC Join Venture, California, USA.
[2] AIJ, 2001, "Recommendations for the Design of Structural Steel Connections". Architectural Institute of Japan, Tokyo, Japan.
[3] Mazzolani F.M. "Moment resistant connections of steel frames in seismic areas: design and reliability", Editor, *E& F.N SPON, London*, 2000.
[4] FEMA 350. "Recommended Seismic Design Criteria for New Steel Moment Frame Buildings", Washington D.C., 2000, U.S.A.
[5] Plumier A. "Reduced beam sections; a safety concept for structures in seismic zones", *Buletinul stiintific al universitatii 'Politehnica'din Timisoara, Romania*, Tom. 41, fasc. 2, pp 46-59.
[6] Moore K, O. Malley J, Engelhardt M. "Design of reduced beam section moment frame connection", Structural steel educational council, Technical information and product service, California, U.S.A.
[7] Chen S.J., Chu J.M., Chou Z.L. " Dynamic Behavior of steel frames with beam flanges shaved around connection", *J. Construct. Steel Research*, Vol. 42, No.1, 1997, pp 49-70.
[8] Popov E., Blondet M., Stepanov L. " Application of dog bones for improvement of seismic behavior of steel connections", Report No. UCB/EERC 96/05, 1996, U.S.A
[9] Anastasiadis A., Gioncu V. " Influence of joint details on the local ductility of steel moment resisting frames, *3rd National Greek Conference on Steel Structures*, Thessaloniki, Greece, 1998, pp 311-319.
[10] Anastasiadis A., Gioncu V., Mazzolani F.M. "New upgrading procedures to improve the ductility of steel MR-frames", *XVII C.T.A Congress*, Napoli, Italy, 1999, pp. 193-204.
[11] Faggiano B., Landolfo R. " Seismic analysis of steel MR frames with dog bone connections, *12th European Conference on Earthquake Engineering*, London, 2002, paper reference. 309.
[12] Anastasiadis A., Mateescu G., Gioncu V. " Improved ductile design of steel MR frames based on constructional details", *9th International Conf. On Metal Structures*, Timisoara, Romania, 2000, pp. 367-376.
[13] FEMA 351. " Recommended Seismic Evaluation and Upgrade Criteria for Exisitng Welded Steel Moment Frame Buildings, Washington D.C., 2000, U.S.A.
[14] EC 8, Part 3. "Design of Structures for Earthquake Resistance. Strengthening and Repair of Buildings", prEN 1998-3:200X, Final Draft, January 2003.
[15] Gioncu V., Mazzolani F.M. "Ductility of seismic resistant steel structures, *SPON Press, London*, 2002.

ΣΧΕΔΙΑΣΜΟΣ ΔΟΚΩΝ ΜΕΙΩΜΕΝΗΣ ΔΙΑΤΟΜΗΣ ΜΕ ΧΡΗΣΗ ΕΥΡΩΠΑΙΚΟΥ ΤΥΠΟΥ ΠΡΟΦΙΛ IPE & HEA

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1. ΠΕΡΙΛΗΨΗ

Σημαντικές απρόσμενες βλάβες σε πλαισιακού τύπου φορείς, στην περιοχή σύνδεσης δοκού – στύλου, παρουσιάστηκαν κατά τους σεισμούς του Northridge (1994) και του Kobe (1995) προβληματίζοντας ιδιαίτερα την επιστημονική κοινότητα ταυτόχρονα σηματοδοτώντας σημαντική ερευνητική δραστηριότητα για την επίλυση των αστοχιών. Γενικά, εκφράστηκαν δύο σχεδιαστικές φιλοσοφίες: της ενίσχυσης σύνδεσης δοκού στύλου, και της μείωσης της διατομής της δοκού κοντά στην περιοχή της σύνδεσης δοκού – στύλου. Η δεύτερη λύση παρουσιάζει σημαντικά οικονομικά αλλά και δομοστατικά πλεονεκτήματα αναφορικά με την εξασφάλιση ισχυρού στύλου – ασθενούς δοκού. Η ερευνητική δραστηριότητα απέδωσε κωδικοποιημένες συστάσεις για τον σχεδιασμό των ανωτέρω λύσεων, τόσο στις Η.Π.Α. όσο και στην Ευρώπη. Ωστόσο, η εισαγωγή κάποιων διατάξεων θα πρέπει να λαμβάνει υπόψη τις κατασκευαστικές πρακτικές που χρησιμοποιούνται στην Ευρώπη.
Στην παρούσα εργασία παρουσιάζονται απόψεις-προτάσεις για τον σχεδιασμό δοκών μειωμένης διατομής, υποδεικνύονται βασικές παράμετροι σχεδιασμού και τέλος δίνονται σχέσεις διαστασιολόγησεις. Επιπρόσθετα, εκτιμάται η ανελαστική ικανότητα δομικών μελών μειωμένης διατομής IPE και HEA, με βάση την πλαστική στροφική ικανότητα, παρέχοντας πληροφορίες για την τάξη μεγέθους της πλαστιμότητας.