PARAMETRIC ANALYSIS AND OPTIMIZATION OF REDUCED BEAM SECTION STEEL FRAME CONNECTIONS

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INTRODUCTION

The RBS connection concept [10] is based on the selective removal of beam material adjacent to the joint either from the web or from the flanges. The reduction in cross-sectional area will reduce the moment capacity at a discrete location of the beam, where yielding will be concentrated, thus protecting the connection from early fracture. After extensive theoretical and experimental work [10] the radius-cut RBS configuration was prequalified in both USA [1,8,9] and Canadian [5] Standards, while its concept was only incorporated in EC8, Part 3 [3] (and not in EC3) for member retrofitting as a mere follow up of the aforementioned Standards without any indication about the type of connection or profiles. The geometry of such a radius-cut RBS is depicted in *Fig. 1*, while the corresponding size limitations are given in expressions (1), where d_b is the depth of the beam.

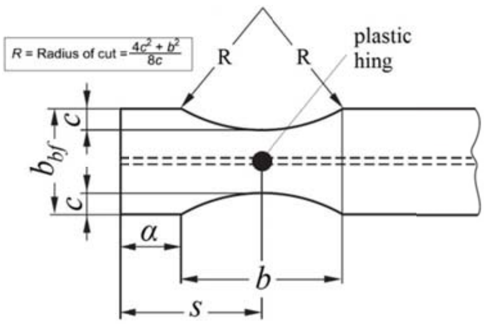

Fig. 1. Radius-cut RBS

$$
\begin{aligned}
&0.5b_{bf} \leq a \leq 0.75b_{fb}\ ,\ 0.65d_b \leq b \leq 0.85d_b\ ,\ 0.10b_{bf} \leq c \leq 0.25b_{bf}\ \text{ AISC/CISC}\\
&a = 0.60b_{bf}\ ,\ \text{b}=0.75d_b\ ,\ \text{c}=0.20b_{bf}\ \text{ EC8}
\end{aligned}
\tag{1}
$$

Extending a recent work by the authors [11], in this work a unified method for optimizing symmetric extended end-plate RBS connections with European I-profiles under monotone loading, which is thereafter verified via FEM simulation under cyclic loading, is offered. The whole approach is documented in terms of required ductility, desirable plastic deformations, formation of 1st plastic hinge within the RBS, avoiding shear joint yielding and local buckling, and may serve as a tool for the acceptance of the RBS as a reliable seismic connection for European Steel Design Practice.

1 OPTIMIZATION UNDER MONOTONE LOADING

1.1 Connection geometry

We consider the symmetric extended end-plate beam to column connection depicted in *Fig. 2*, with continuity plates and radius-cut reduced beam section. The columns are made of HEB or HEA sections, while the beams of HEA or IPE ones, following standard European design practice. The geometric limitations of the RBS are those given in the Introduction, while the following constraints, according to EC3, Part 1.8 [3] are valid for the various connection components, in terms of equalities and inequalities:

$$\frac{b_p}{h_p}=\frac{b_b}{h_b},\quad b_b<b_c,\quad b_p>b_b,\quad b_p=2e+x,\quad h_p=2e_1+4y\ ,\ 3d_0\le x\vee y\le 5d_0,\quad \min\left(e,e_1\right)=1.5d_0 \tag{2}$$

The connection is acted upon by a bending moment M, which is the monotone type of loading accounted for herein.

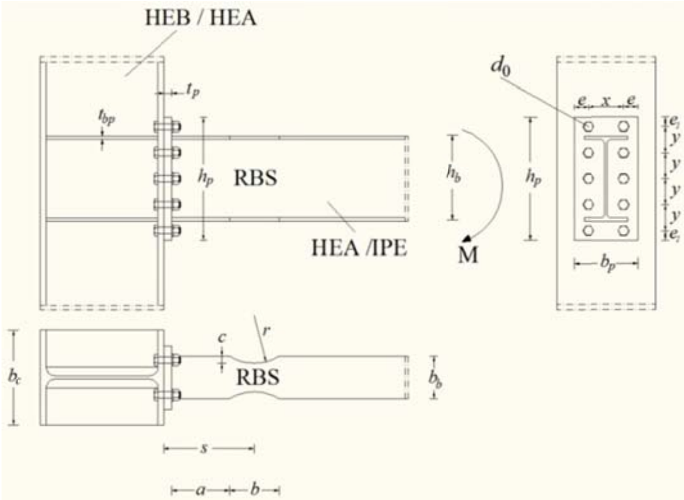

Fig. 2. Geometry of the RBS connection considered

1.2 Calculation of connection design moment resistance $M_{j,Rd}$ via regression analysis

For all four I-profiles chosen, their geometrical, cross-sectional and inertial properties are expressed as functions of only the height of the profile via curve-fitting and regression analysis. The procedure can be found in recent works by the authors [11,12] and will not be repeated here for brevity. In the sequel, the connection moment resistance $M_{j,Rd}$ is evaluated employing the component method [3], leading to a lengthy nonlinear non-continuous function that depends only on the height of the beam and the column. The corresponding *Mathematica* [14] code is given in [12].

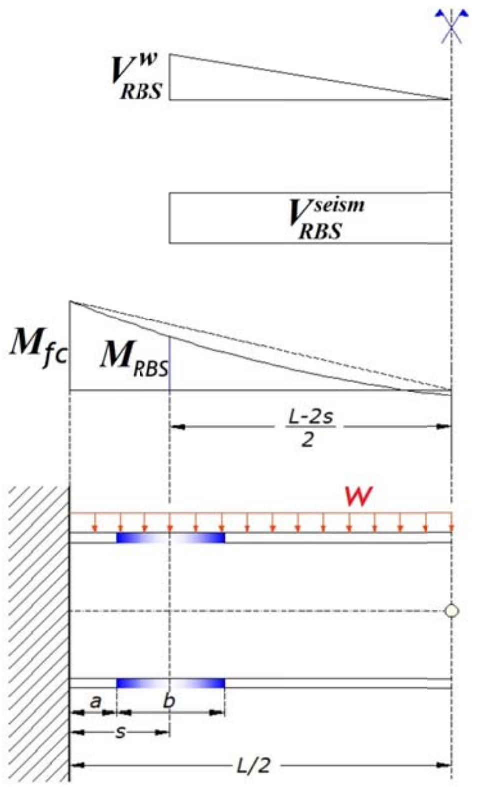

Fig. 3. Geometry and allocation of forces in the beam of a typical steel moment frame with RBS

1.3 Optimization procedure

For a typical Moment Resisting Frame, with a distance between columns (length of beam) equal to L, and with RBS at both sides of the beam, when – under seismic loading – plastic hinges are firstly formed at the weakest RBS section, the allocation of seismic and non-seismic generalized forces are shown in *Fig. 3*. Gravity forces are owed to a uniformly distributed load w along the length of the beam, while the expected maximum moment at the face of the column is designated as M_{fc}.

Hence, this moment is equal to:

$$M_{fc}=M_{RBS}+\left(V_{RBS}\right)s \tag{3}$$

where

$$M_{RBS}=\left(W_{pl,b}-ck\right)f_{RBS}\ \ ,\ k=2t_{fb}\left(h_b-t_{fb}\right) \tag{4}$$

while is evident that

$$V_{RBS}=V^{seismic}_{RBS}+V^{w}_{RBS}\ \ ,\ V^{seismic}_{RBS}=\frac{2M_{RBS}}{L-2s}\ \ ,\ V^{w}_{RBS}=\frac{wL}{2} \tag{5}$$

Combining expressions (3) and (5) we get:

$$M_{fc} = M_{RBS} + \left(\frac{2M_{RBS}}{L-2s} + \frac{wL}{2} \right) s \tag{6}$$

In equation (4), f_{RBS} represents the maximum expected stress that will develop in the RBS. Under cyclic loading conditions and large inelastic deformations, the value of this stress at the weakest RBS section may reach the ultimate strength f_u. Hence, we chose $\max f_{RBS} = f_u$, covering in this manner the overstrength requirement of cl. 6.5.2, 6.5.5 and 6.6.4 of EC3 – Part 1-1 [2]. For an optimum connection as in *Fig. 2*, it is required that:

$$\Delta M = M_{j,Rd} - M_{fc} \geq 0 \text{ and } \Delta M = \text{ minimum} \tag{7}$$

In order to avoid a situation when ΔM becomes equal to zero, i.e. simultaneous failure of connection and RBS, a safety-overstrength factor is used in equation (6), which finally becomes:

$$M_{fc} = 1.1M_{RBS} + \left(\frac{2M_{RBS}}{L-2s} + \frac{wL}{2} \right) s \tag{8}$$

The second right-hand side term of the above is thereafter directly correlated to M_{RBS} by writing

$$M_{fc} = M_{RBS} + \mu M_{RBS} \tag{9}$$

and the value of parameter μ is evaluated, so that it meets the following two characteristic requirements: (a) expression (9) to be valid for a wide range of beam lengths L and distributed loads w and (b) at the position where the 1st plastic hinge is formed, there will be no effect of shear force on the bending resistance, i.e.

$$V_{Ed} / V_{pl,Rd} \leq 0.50 \tag{10}$$

Since the acting shear is equal to V_{RBS} and according to expression (9) we get

$$V_{Ed} = \frac{\mu M_{RBS}}{s} \xrightarrow{(4)} V_{Ed} = \mu \frac{\left(W_{pl,b} ck \right) f_{RBS}}{s} \tag{11}$$

Taking into account that $V_{pl,Rd} = \dfrac{A_V f_u}{\sqrt{3}}$ and setting $\gamma = \mu\sqrt{3}$, inequality (10) yields:

$$c \geq \frac{W_{pl,b}}{k} - \frac{0.5sA_v}{\gamma k} \tag{12}$$

Given that for every beam section the quantities s, $W_{pl,b}$, A_v and k are known, the only parameter to be calculated is γ. Furthermore, since $c \leq 0.25b_b$ one may evaluate the maximum values of γ. The final expressions for M_{fc} are found equal to:

$$M_{fc} = 1.1M_{RBS} + 0.37M_{RBS} \text{ for IPE beams }, \quad M_{fc} = 1.1M_{RBS} + 0.335M_{RBS} \text{ for HEA beams} \tag{13a,b}$$

For the satisfaction of the 1st of the aforementioned requirements, M_{fc} is expressed as:

$$M_{fc} = 1.1M_{RBS} + \left(\frac{\sigma M_{RBS}}{L-2s} \right) s \tag{14}$$

Without loss of generality - applicability, and within the proposed methodology, we vary the value of parameter σ from zero (ignoring gravity forces) to four (4), representing a situation where gravity forces and seismic forces are equal to each other. Thereafter, using *Mathematica* [14] we search for the relation between L and s that satisfy relations (13). The corresponding outcome is:

$$\begin{aligned}&\text{IPE beams } (\mu = 0.37) &&: L_{\min} = 7.40541s \ (\sigma = 2),\ L_{\max} = 12.8108s \ (\sigma = 4)\\ &\text{HEA beams } (\mu = 0.335) &&: L_{\min} = 7.9701s \ (\sigma = 2),\ L_{\max} = 13.9403s \ (\sigma = 4)\end{aligned} \tag{15}$$

These results satisfy to a very good extent the minimum allowable beam depth over length ratio for SMF and IMF of ANSI/AISC 358-10 [9].

According to all the above, we seek an optimum connection configuration that satisfies the following relations, which in fact constitute a fully parameterized minimization problem under the constraints given in relations (2), where f_p is the *objective function* and p all parameters involved in the calculation of $M_{j,Rd}$ and M_{RBS}, given above.

$$f(p) = M_{j,Rd} - \psi M_{RBS} \geq 0,\ f(p) = \min,\ \ \psi = 1.1 + \mu = 1.47 \text{ (IPE)} \ - \ \psi = 1.1 + \mu = 1.435 \text{ (HEA)} \tag{16}$$

1.4 Optimized connection configurations

Applying the procedure outlined previously, eight optimal designs of RBS end-plate connections are produced [12] – two for every beam-column profile combination, for S235 steel material. The "best" practical configuration (in terms of minimum *objective function*) is presented in *Fig. 4*, with details given in *Table 1*.

Table 1. Details of the "best" optimum connection

No	Connection Design Dimensions and Properties					
	Beam / Column	x (mm)	y (mm)	c (mm)	$M_{j,Rd} - 1.47M_{RBS}$	$V_{RBS}/V_{pl,RBS}$ (%)
2	IPE270/HEB450	95	90	33	2.3 kNm	42

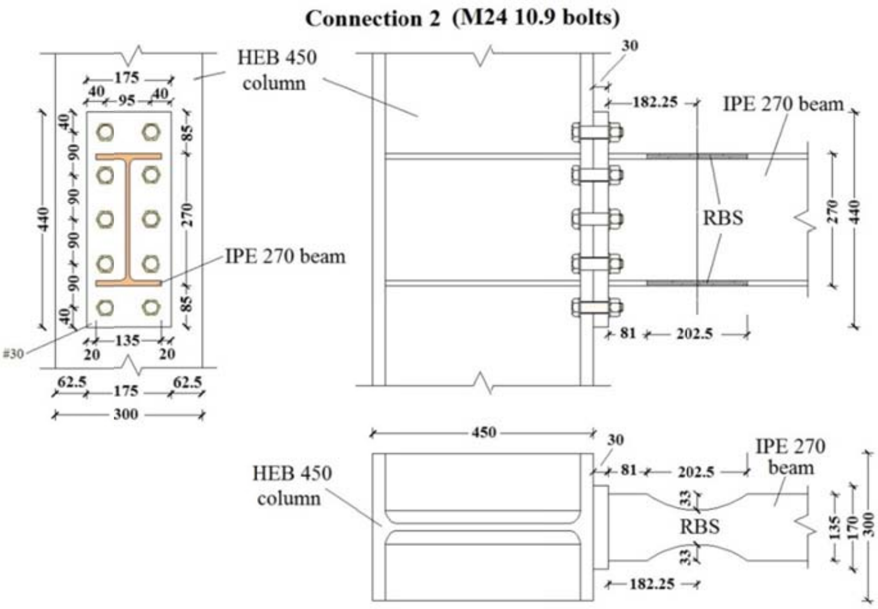

Fig. 4. "Best" practical statically optimized RBS connection

2 FE VERIFICATION UNDER CYCLIC LOADING

2.1 Model description

The reliability of the previously described optimization method is verified via FEM under cyclic loading. In particular, Abaqus Software [15] is used for the simulation, by means of continuum deformable solid element (C3D10M) – suitable for large deformations and contact problems. Dense meshing is prescribed in control regions and suitable restraints are set to avoid lateral buckling of the beam part. Successful calibration of the model under cyclic loading is initially achieved, by comparing the results with the experimental outcome of a recent relevant work [13]. Thereafter, a displacement based loading protocol [6] is defined, imposed at distance z away from the centre of the RBS (which in fact determines the length of the beam in the model), in such a manner that the seismic shear turns out equal to V_{RBS}. This value of z in return, a product of nonlinear static pushover analysis, defines the ultimate displacement Δf, necessary for the amplitude of the cyclic displacement. The trilinear model of kinematic hardening is used for S235 steel [2,7]. The material and connection FE model are shown in *Fig. 5*.

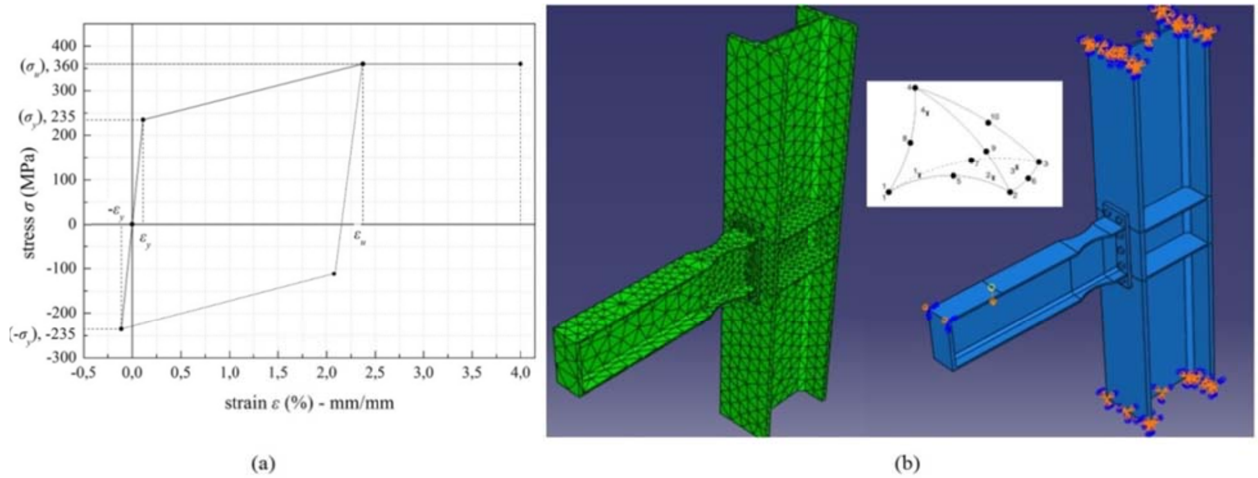

Fig. 5. Material and connection FE model

2.2 Results and associated performance criteria

For all eight statically optimized connections results were obtained via the aforementioned FE cyclic displacement analysis, in terms of moment – rotation curves at the centre of the RBS, the interface between beam and plate and at the panel zone. In order that the performance is considered adequate, the following criteria should be met [1,3,4,8,9]: (a) formation of the 1st plastic hinge within the RBS, (b) development of plastic rotations in the RBS at least 0.03 rad, while these remain minimal at the beam-column interface, (c) large amount of energy dissipation in the RBS without local buckling, (d) not spreading of yielding in the bolts and simultaneous minimum action of prying forces, (e) stable hysteretic behaviour of the panel zone with small amounts of energy dissipation and (f) elastic response of the column flange attached to the end plate.

It was found that all eight connections met the above criteria quite satisfactorily, and the best overall performance was exhibited by connection 2, facts fully validating the optimization procedure performed earlier under monotone loading. The results associated with this connection are shown in *Fig. 6*.

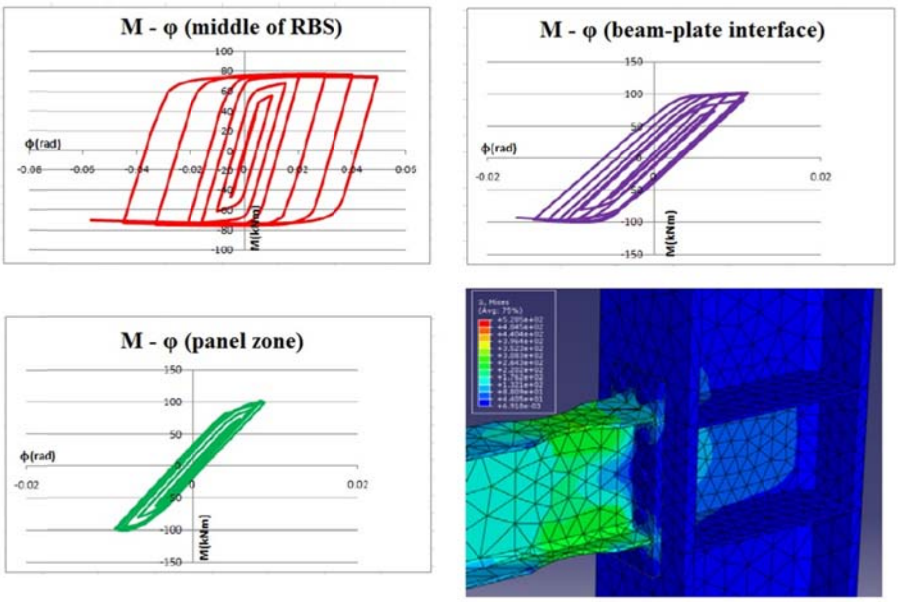

Fig. 6. FE results of connection 2 under cyclic loading

3 CONCLUSIONS

Using a combination of curve-fitting techniques, nonlinear constrained global optimization and based on the well-established EC3 component method and EC8 recommendations, extended-end-plate radius-cut RBS connections are optimized under static loading conditions, utilizing European I-profiles for beams and columns. Cyclic loading imposed in the optimized connections via FEM modelling verifies the numerical approach through desired ductile behaviour.

The results of this investigation, based on simple rules of Mechanics and Plasticity, aspire to be the basis for an optimization procedure of parameters of RBS connections. The adaptation of this construction solution and its acceptance from the Eurocodes, are estimated to oppose the scepticism and suspicious state that enclose the weakening section strategies, considering that they consist of economical, easily produced, cost competitive and reliable connections, that could contribute to the construction progress.

REFERENCES

[1] FEMA-350, 2000. *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*, Sac Joint Venture, Federal Emergency Management Agency, Washington D.C., USA.

[2] EN 1993-1-1, 2005. *Eurocode 3: Design of Steel Structures – Part 1-1: General Rules and rules for buildings*, CEN, Brussels, Belgium.

[3] EN 1993-1-08, 2005. *Eurocode 3: Design of Steel Structures – Part 1-8: Design of joints*, CEN, Brussels, Belgium.

[4] EN 1998-3, 2005. *Eurocode 8: Design of structures for earthquake resistance – Part3: Assessment and retrofitting of buildings*, CEN, Brussels, Belgium.

[5] Canadian Institute of Steel Construction, 2008. *Moment Connections for Seismic Applications*, Lakeside Group Inc., Ontario, Canada.

[6] Filiatrault, A., Wanitkorkul, A., Constantinou, M., 2008. *Development and Appraisal of a Numerical Cyclic Loading Protocol for Quantifying Building System Performance*, Technical Report MCEER-08-0013, University of Buffalo, USA.

[7] Simões da Silva, L., Rebelo, C., Nethercot, D., Marques, L., Simões, R., Vila Real, P.M.M., 2009. “Statistical Evaluation of the lateral-torsional buckling resistance of steel I-beams, Part 2: Variability of steel properties”. *Journal of Constructional Steel Research*, ISSN 0143-974X, Elsevier Science Limited, Vol. 65, pp. 832-849.

[8] ANSI/AISC 341-10, 2010. *Seismic Provisions of Structural Steel Buildings*, American Institute of Steel Construction, Chicago, Illinois, USA.

[9] ANSI/AISC 358-10, 358s1-11, 2011. *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications, incl. Supplement No 1*, American Institute of Steel Construction, Chicago, Illinois, USA.

[10] Sophianopoulos, D.S., Deri, A.E., 2011. “Parameters Affecting Response and Design of Steel Moment Frame Reduced Beam Section Connections: An Overview”. *International Journal of Steel Structures*, ISSN 1598-2351, Springer Science-Business Media, Vol. 11, pp. 133-144.

[11] Sophianopoulos, D.S., Deri, A.E., 2011. “Steel End-Plate Beam-to-Column RBS Connections: Optimum Design under Monotone Loading Utilizing European I-profiles”. *Proc. 6th European Conference on Steel and Composite Structures*, 31 August – 2 September, 2011, Budapest, Hungary, ISBN 978-92-9147-013-4, Vol. A, pp. 501-506.

[12] Deri, A.E., 2013. *Parametric Analysis and Optimization of Reduced Beam Section Steel Frame Connections*, Ph.D. Thesis, Department of Civil Engineering, University of Thessaly, Volos, Greece (in Greek), http://phdtheses.ekt.gr/eadd/handle/10442/28151.

[13] Sofias, C.E., Kalfas, C.N., Pachoumis, D.T., 2014. “Experimental and FEM analysis of reduced beam section moment endplate connections under cyclic loading”. *Engineering Structures*, ISSN 0141-0296, Elsevier Science Limited, Vol. 59, pp. 320-329.

[14] Mathematica 8, *Technical computing software*, Wolfram Research, Champaign, Illinois, USA.

[15] Abaqus Ver. 6.8, *Users Manuals* (10 Volumes), 2008, Dassault Systems, Simulia Corporation.