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Steel Beam-to-Column RBS Connections: FEM Analysis under Cyclic Loading

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Abstract

In a companion paper [1], an optimization scheme for extended-end-plate Reduced Beam Section (RBS) connections of steel-moment-frames was presented, based on the component method of Eurocode 3, on regression analysis and on principles of Mechanics under monotone loading. European beam and column profiles were utilized, in conjunction with geometric restrictions and constraints of North American and European Standards for prequalified radius-cut RBS. The aforementioned method aimed for an excellent seismic performance, the verification and validation of which is the content of the present study. Using FEM modeling and accounting for the assumptions used in the optimum design, after calibration with existing experimental data, the optimum connections were numerically analyzed under cyclic loading, adopting a well-accepted displacement-based protocol. All optimum solutions exhibited an excellent cyclic response, and met very satisfactorily all the performance criteria for seismic design. Results in terms of hysteretic M-φ curves at three characteristic areas of the connections validate the whole analysis, a fact aiming to assist in incorporation the radius-cut RBS concept in European Steel Design Codes and engineering practice.

Keywords

RBS Connections, FE Modeling, Cyclic Loading, Seismic Performance Criteria, Optimum Design, European Standards and Practice

1. Introduction

Since 1995, and after the Northridge and Kobe earthquakes, the Reduced Beam Section connection was introduced as a safety concept for structures in seismic zones. Extensive theoretical and experimental investigations lead to the prequalification of the radius-cut RBS connection in North America [2], but regarding

US and Canadian connection types of steel moment resisting frames. In Europe however, and although the introduction of the RBS was performed by Plumier [3], and also patented in Asia [2], there is poor existing data. Only recently [4] [5] [6] experiments and FEM analyses of RBS connections based on European Practice (beam-to-column welded or extended endplate bolted) were performed, while the only references of the Eurocodes regarding RBS are those given in EC8-Part 3 [7], as also mentioned in the companion paper [1]. Therein, an optimization scheme under monotone loading was adopted, using European I-profiles for the beams and columns of the RBS connection and standard symmetrical extended endplate joints. This lead to eight optimum RBS connections, which in the present work are analyzed via the FEM under cyclic loading, in order to validate the whole design and prove that these connections meet to the maximum possible extent the performance criteria of seismic design [8] [9] [10]. After calibration of the proposed FE model with existing experimental data [6] and adopting a preliminary pushover analysis, the aforementioned optimum connections exhibited an excellent response as per every criterion. Hence, it is anticipated that both parts of the foregoing work will assist in incorporating the RBS concept in Eurocodes 3 and 8 as a new design methodology for both static and seismic applications.

Evidently, there are also other types of RBS connections that have been reported in the relevant literature, as for instance in the work by Rahnavard *et al.* [11], where a comparison of the seismic response of these types with the one of the radius-cut RBS was investigated. Furthermore, fragility functions to estimate the probability of reaching or exceeding different damage states in reduced beam section (RBS) beam-to-column moment connections of steel moment resisting frames have been also reported [12]. In the companion paper [1], extended bibliographic information has been given, and it will not be repeated here for brevity. For a detailed overview of the parameters affecting the response of RBS connections, one may refer to an already cited work by theauthors [2].

2. Finite Element Modeling

2.1. Formulation of the Model

Let as considered a statically optimized connection [1] with European I-profiles for the beams and columns as well as with radius-cut RBS, as shown in **Figure 1**. The geometry and generalized forces of the optimum connection of the companion paper can be found in tabular form in **Appendix**, at the end of the manuscript. Such a connection is thereafter modeled in Abaqus Software [13] utilizing the C3D10M 10-node continuous solid element, suitable for large nonlinear deformations and contact problems. The parts-assembly concept was adopted and a denser mesh for the RBS and the beam-column interface was introduced. Moreover, fixed conditions were imposed at the ends of the column, while adequate constraints were also added to the beam, in order to minimize local buckling phenomena. The final FE model is illustrated in **Figure 2**, for which a mesh convergence was finally achieved.

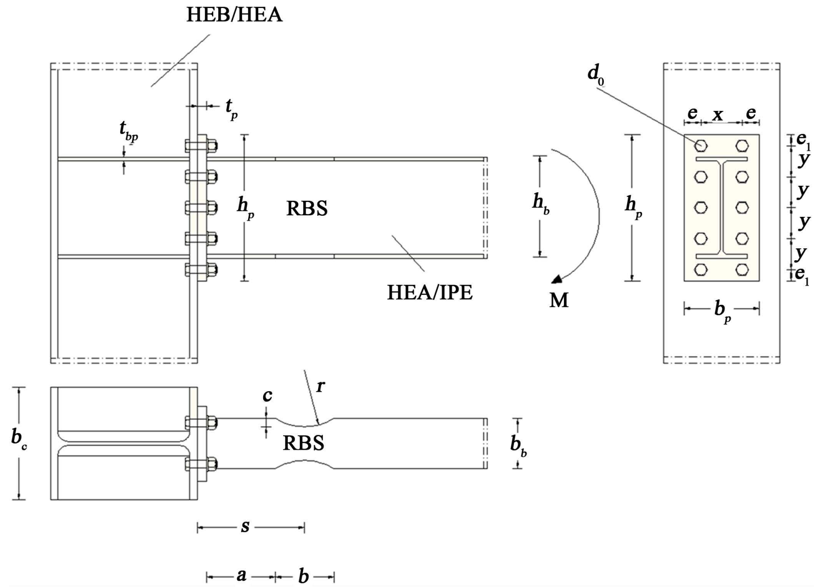


Figure 1. Typical optimized RBS connection, as in [1] (for details see **Appendix**).

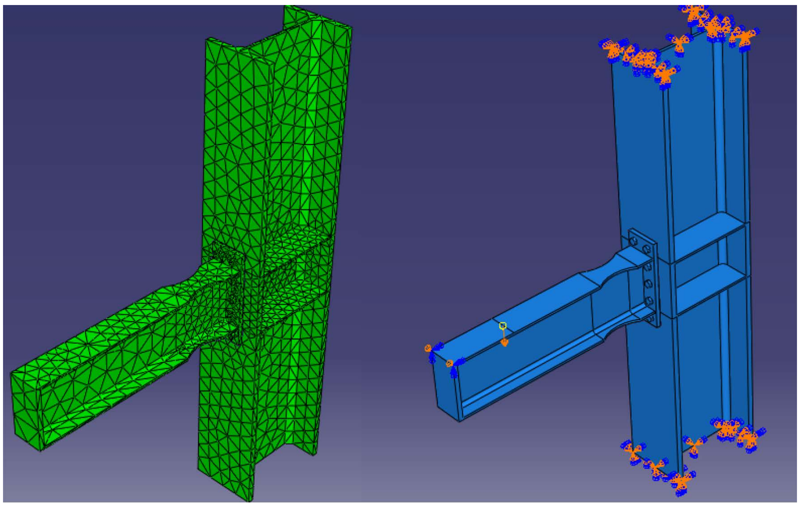


Figure 2. FE model used in the present analysis.

2.2. Material Model

Since the optimum solutions given in the companion paper [1] were made of S235 steel, an adequate material model was introduced in the FEM simulation, according to well accepted theoretical and experimental evidence of the relevant literature [14]. The schematic picture of this trilinear kinematic hardening model is depicted in **Figure 3**, for which it is valid that

$$\varepsilon_y = \frac{\sigma_y}{E} \times 100 = \frac{235}{210000} \times 100 = 0.111904762 \tag{1}$$

$$\varepsilon_u = 31.2\varepsilon_y = 2.372381, \quad \varepsilon_{\max} = 4 \tag{2}$$

where ε_y and ε_u are the yield strain and ultimate strain respectively, $\varepsilon_{\max}$ is the maximum strain and σ_y is the yield stress of S235 steel.

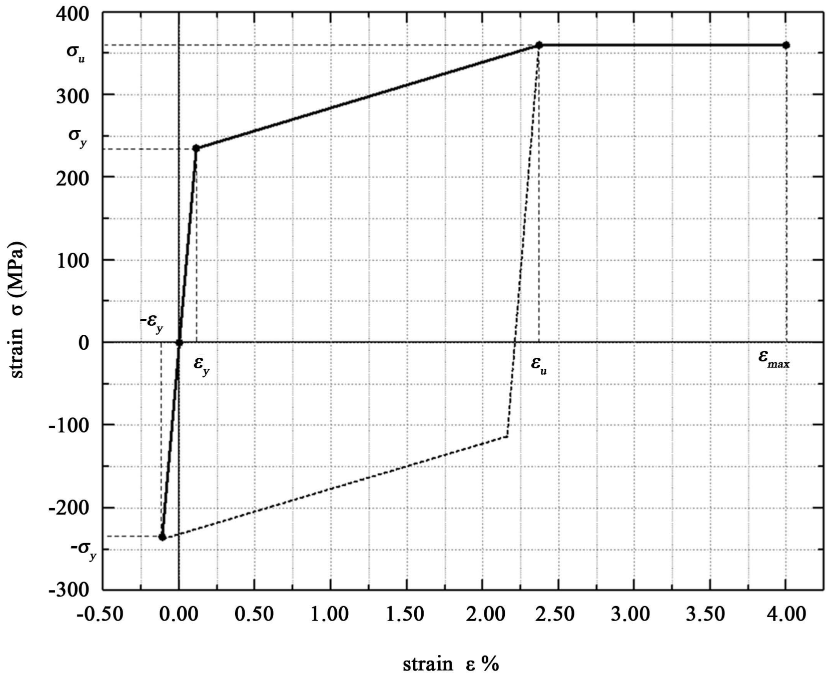


Figure 3. Material model used in the FE simulation (for S235 steel).

2.3. Model Calibration

Before attempting any cyclic analysis, the proposed model was calibrated with existing experimental data [6]. After proper adjustments of this model to the experimental setup (as far as geometry, loading protocol and material properties were concerned), its cyclic response was found in very good agreement with the results of the experiments, as shown throughout **Figure 4**, in terms of hysteretic loops at two characteristic points of the connection.

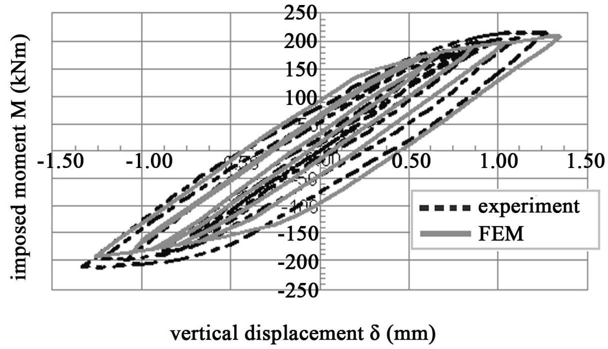
2.4. Loading Protocol and Displacement-Based Cyclic Analysis

The cyclic analysis was based on initially evaluating the failure displacement Δf of each optimum connection, which was achieved via static pushover analysis. The goal was to impose the displacement at a proper point along the beam, such that the shear developing at the middle of the RBS was equal to V_{RBS}, as evaluated during the optimization procedure. The whole scheme is shown in **Figure 5**, where the distance z is calculated and from its value, the failure displacement is appraised.

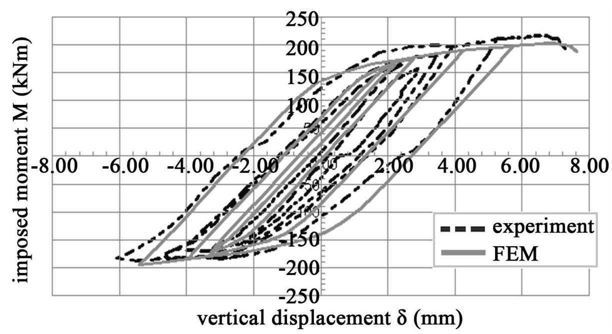
In the sequel, for each optimum solution, a well-established displacement protocol [15] was used, since both the values of z and Δf where found earlier. This protocol, in terms of cycle number vs. dimensionless amplitude imposed, is depicted in **Figure 6**.

2.5. Performance Criteria

For all eight optimum designs, strict seismic performance criteria were considered, in order to validate the whole analysis. These are the following:

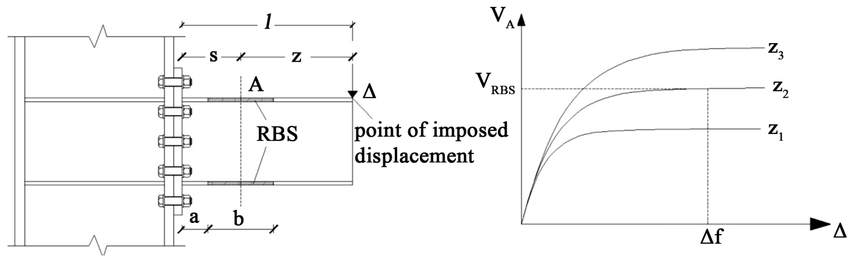


(a)



(b)

Figure 4. Calibration of the proposed FE model, (a) at 3 cm away from the column face; and (b) at the middle of the RBS, in terms of M-δ curves.



(a) (b)

Figure 5. Geometry (a) and static pushover curves (b) for establishing the point of imposing the cyclic displacement along the beam.

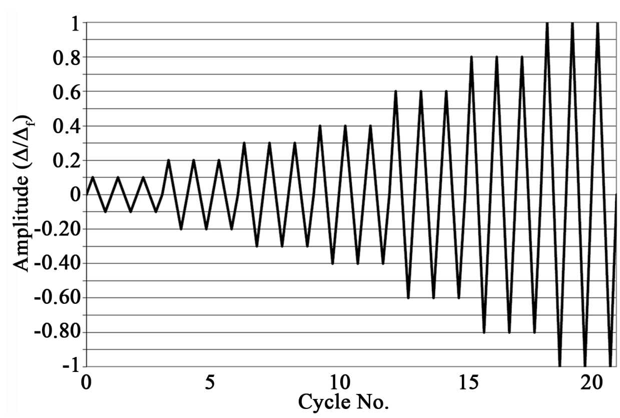


Figure 6. Loading protocol adopted (saw-tooth diagram).

1) Formation of the 1st plastic hinge in the Reduced Beam Section;

2) Development of a plastic rotation within the RBS of at least 0.03 rad and simultaneous minimal such rotations at the interface between beam and end-plate;

3) Large energy dissipation at the RBS, without any local buckling phenomena, that would reduce ductile behavior;

4) Avoidance of yield spreading in the bolts and minimum prying forces;

5) Steady hysteretic response of the panel zone with small energy dissipation; and

6) Elastic response of the column flange connected.

Hence, three specific and characteristic areas of the connections were examined, namely the middle of the RBS, the interface between beam and end-plate, and middle of the panel zone, as illustrated below, in Figure 7.

3. Numerical Results and Discussion

Numerical results, in terms of hysteretic loops—moment/rotation curves were obtained for all the eight statically optimized connections, at the three areas shown in the previous figure. For brevity, the outcome of the pushover and cyclic analysis will be presented 1) for the best optimum solution, *i.e.* connection 2 (see **Appendix**), and 2) as the overall results for the first four optimum connections. Similar response was obtained for the rest of the optimum connections.

More specifically, in Figure 8(a) and Figure 8(b) one may perceive the static pushover curve and the distribution of stresses of connection 2, while in Figures 9(a)-(c) the M-φ curves for this specific connection are shown. Moreover, Figures 10(a)-(c) depict the outcome of the FE cyclic analysis for the 1st four optimum solutions, in terms of M-φ curves, as previously stated.

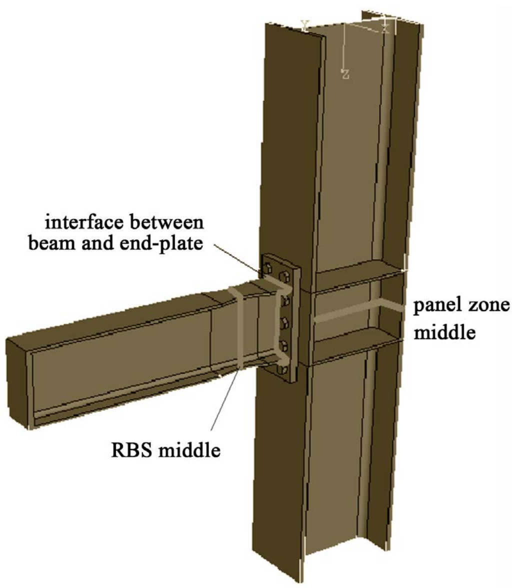


Figure 7. The three characteristic areas of the connections, for which numerical results were obtained.

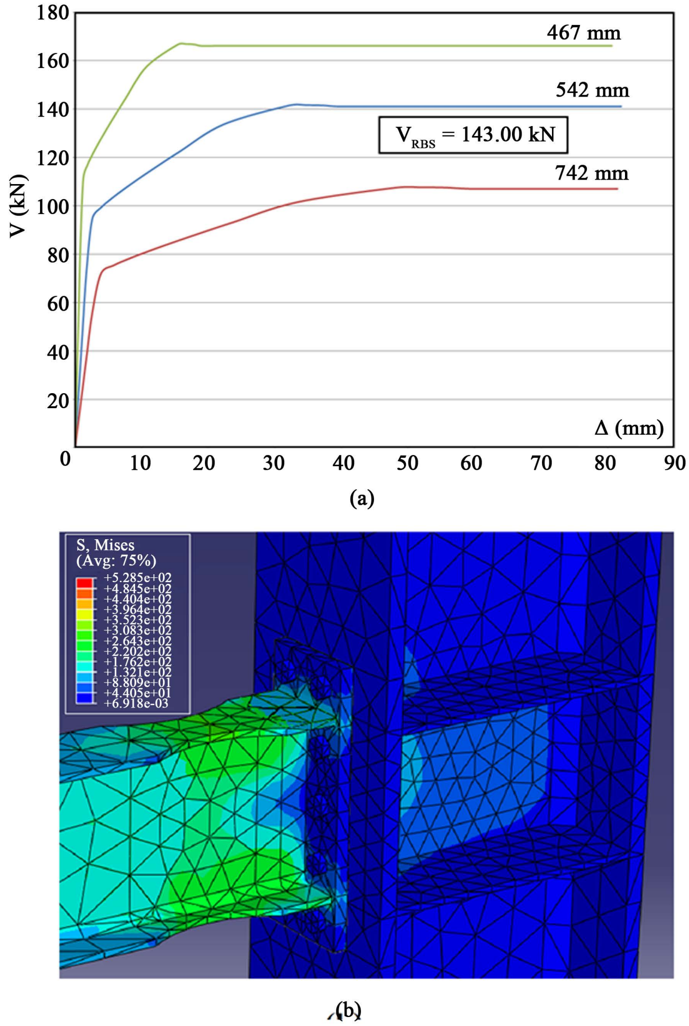


Figure 8. Static pushover curve (a); and distribution of stresses in the FE model of connection 2 (b), as in [1].

From these figures, it is proven that all the performance criteria were met in a very satisfactory manner. No local buckling was encountered (*i.e.* low cycle fatigue), prying forces did not seem to appear and additionally the panel zone was not affected by the increasing amplitude of the displacement until the plastic hinge was formed in the RBS. The bolts and the column flange connected performed as designed and expected, while the desired energy dissipation was concentrated within the RBS. The above results can be interpreted as a success of the whole analysis and optimum design methodology proposed in the foregoing as well as in the companion paper. This fact will hopefully assist in incorporating the RBS concept as a design alternative for moment resisting steel frames in both EC3 and EC8 as well as in everyday European engineering practice.

Future work requires the effect of a much larger axial force on the columns, in order to use alternative modeling of the RBS connections (details given in [1]). The work is ongoing and will produce results that are more interesting in the near future, accompanied by experimental ones.

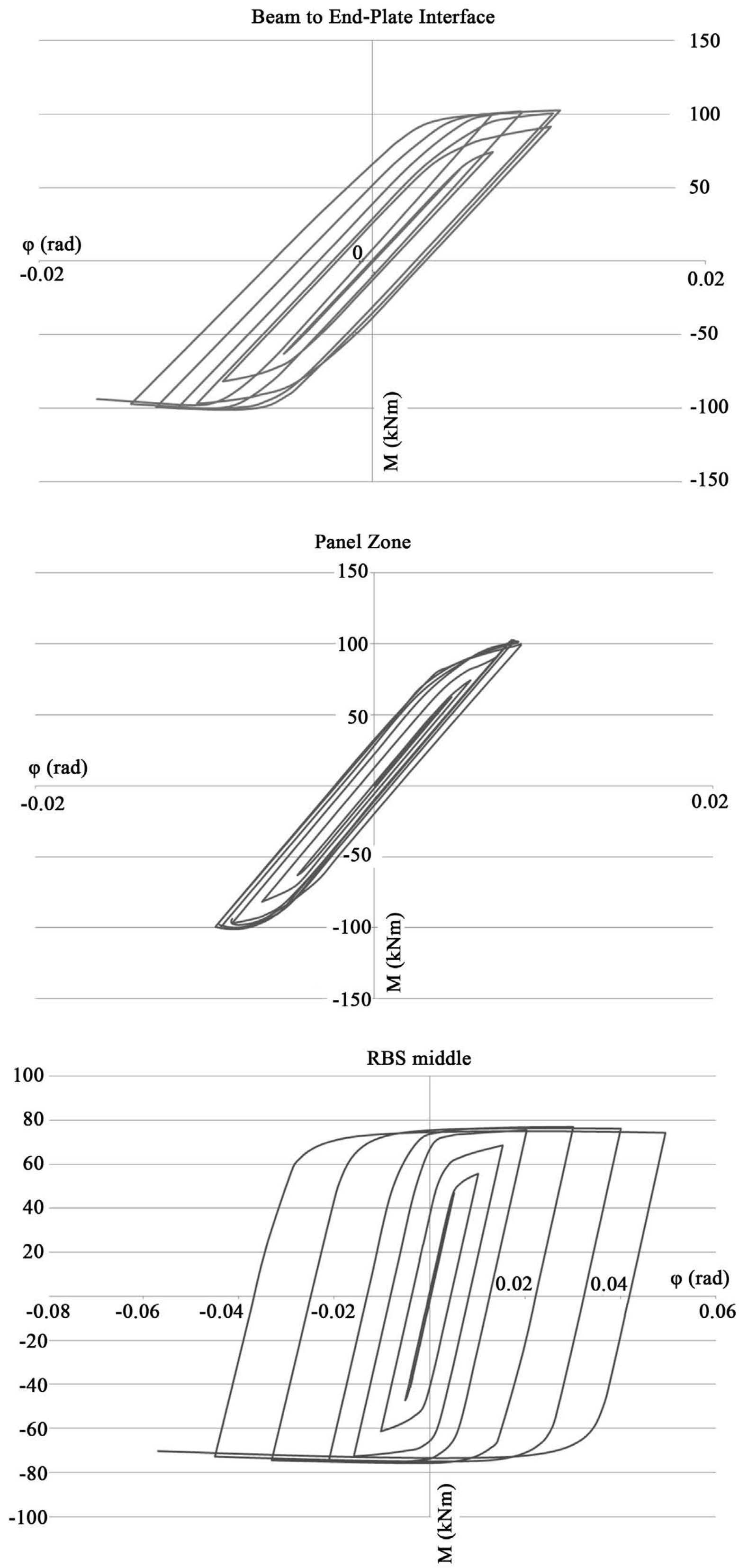


Figure 9. Results, in terms of moment-rotation curves, for connection 2 according to [1], at the three characteristic areas described previously.

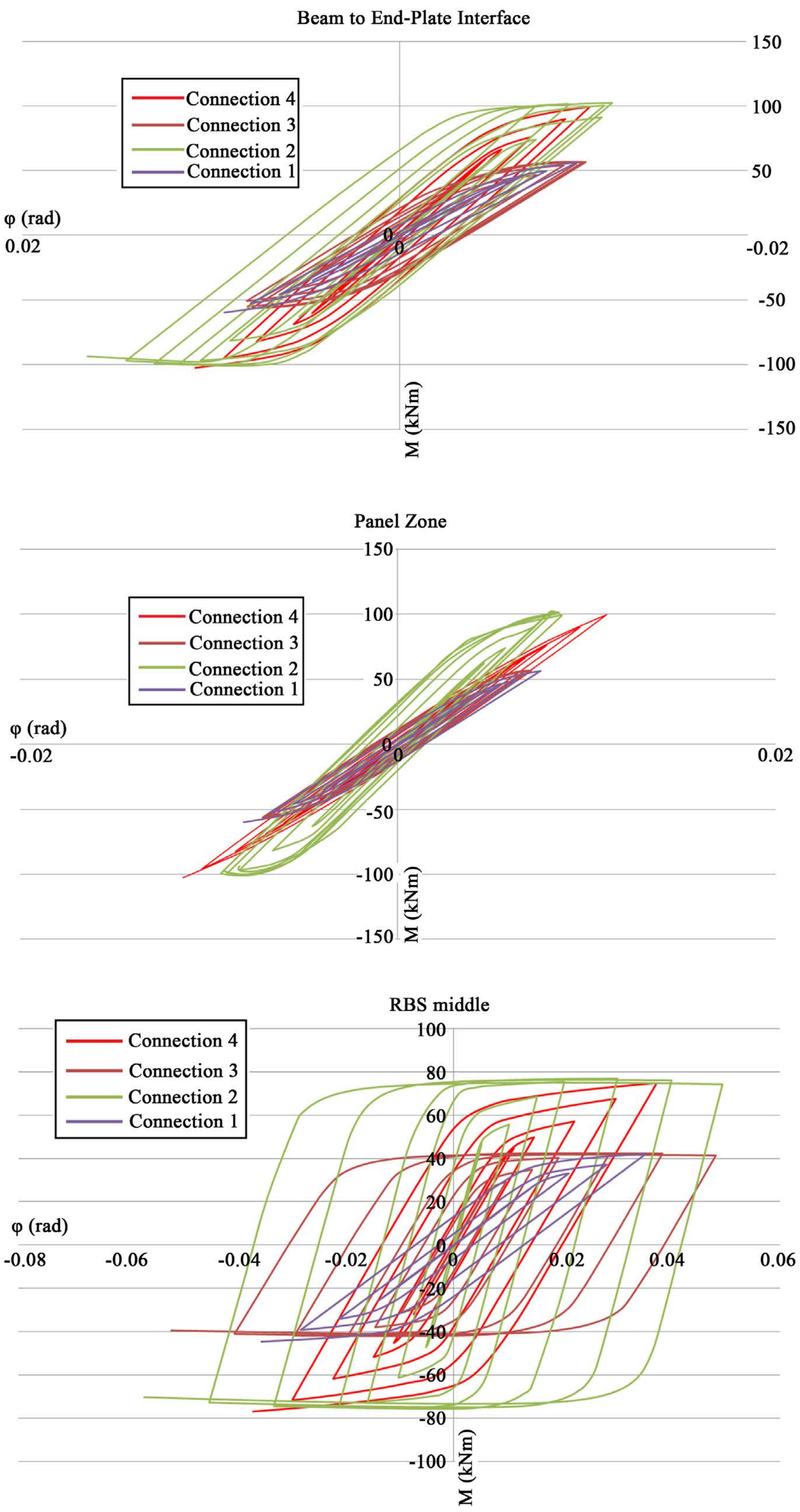


Figure 10. Overall results, in terms of moment-rotation curves, for the 1st four optimized connections, according to [1] (see also **Appendix**).

4. Conclusions

As declared also in the Abstract, the aim of this work was the validation of the results obtained via static optimization procedure [1] by a FE cyclic analysis. This was achieved in a very satisfactory extent, and hence one may draw the following conclusions:

1) The FE modeling of the RBS connections was adequate, since it was properly calibrated with existing relevant experimental result;

2) The displacement-based cyclic FE analysis adopted fully validated the optimization mehtodology under monote loading-outcome of the companion paper;

3) All oprimum designs showed excellent cyclic performance and met all strict performance criteria;

4) The goal of the whole scheme proposed will hopefully assist in incorporating the RBS concept for the seismic design of moment resting steel frames in Europe, both in Standards and practice.

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Conflicts of Interest

The authors declare no conflicts of interest regarding the publication of this paper.

References

[1] Sophianopoulos, D.S. and Deri, A.E. (2017) Steel Beam-to-Column RBS Connections with European Profiles: I. Static Optimization. *Journal of Constructional Steel Research*, **139**, 101-109. https://doi.org/10.1016/j.jcsr.2017.09.028

[2] Sophianopoulos, D.S. and Deri, A.E. (2011) Parameters Affecting Response and Design of Steel Moment Frame Reduced Beam Section Connections: An Overview. *International Journal of Steel Structures*, **10**, 133-144. https://doi.org/10.1007/s13296-011-2003-5

[3] Plumier, A. (1996) Reduced Beam Section: A Safety Concept for Structures in Seismic Zones. *Transaction on Civil Engineering*, *Architecture*, **41**, 46-60.

[4] Pachoumis, D.T., Galoussis, E.G., Kalfas C.N. and Christitsas, A.D. (2009) Reduced Beam Section Moment Connections Subjected to Cyclic Loading: Experimental analysis and FEM Simulation. *Engineering Structures*, **31**, 216-223. https://doi.org/10.1016/j.engstruct.2008.08.007

[5] Pachoumis, D.T., Galoussis, E.G., Kalfas, C.N. and Efthimiou, I.Z. (2010) Cyclic Performance of Steel Moment-Resisting Connections with Reduced Beam Sections—Experimental Analysis and Finite Element Model Simulation. *Engineering Structures*, **32**, 2683-2692. https://doi.org/10.1016/j.engstruct.2010.04.038

[6] Sofias, C.E., Kalfas, C.N. and Pachoumis, D.T. (2014) Experimental and FEM Analysis of Reduced Beam Section Moment Endplate Connections under Cyclic Load-

ing. *Engineering Structures*, **59**, 320-329.
https://doi.org/10.1016/j.engstruct.2013.11.010

[7] EN 1993-8 (2005) Eurocode 8: Design of Structures for Earthquake Resistance—Part 3: Assessment and Retrofitting of Buildings. Brussels.

[8] FEMA 355D (2000) State of the Art Report on Connection Performance. Washington DC.

[9] EN 1993-1-8 (2005) Eurocode 3: Design of Steel Structures—Part 1-8: Design of Joints. Brussels.

[10] FEMA 350 (2000) Recommended Seismic Design Criteria for New Steel Moment Frames. Washington DC.

[11] Rahvavard, R., Hassanipour, A. and Siapholo, N. (2015) Analytical Study of New Types of Reduced Beam Section Moment Connections Affecting Cyclic Behavior. *Case Studies in Structural Engineering*, **3**, 33-51.
https://doi.org/10.1016/j.csse.2015.03.001

[12] Lignos, D.G., Kolios, D. and Miranda, E. (2010) Fragility Assessment of Reduced Beam Section Moment Connections. *Journal of Structural Engineering* (*ASCE*), **136**, 1140-1150. https://doi.org/10.1061/(ASCE)ST.1943-541X.0000214

[13] Abaqus Unified FEA (2017) Complete Solutions for Realistic Simulations, Simulia, Dassualt Systems.

[14] Simoes da Silva, L., Rebelo, C., Nethercot, D., Marques, L., Simoes, R. and 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*, **65**, 832-849. https://doi.org/10.1016/j.jcsr.2008.07.017

[15] Filiatrault, A., Wanitcorcul, A. and Constantinou, M. (2008) Development and Appraisal of a Numerical Cyclic Loading Protocol for Quantifying Building System Performance. Technical Report MCEER-08-0013, Buffalo.

Appendix

The geometry and the generalized forces developing in the eight statically optimized connections of the companion paper [1] are given in the following Table A1 and Table A2.

Table A1. Geometry of optimized connections.

No	Beam	Column	x (mm)	y (mm)	a (mm)	b (mm)	c (mm)	e/e_1* (mm)	t_p (mm)	Bolts
1	IPE220	HEB400	100	75	66	165	25	35	20	M20/8.8
2	IPE270	HEB450	95	90	81	202.5	33	40	30	M24/10.9
3	IPE220	HEA550	100	70	66	165	27	35	20	M20/8.8
4	IPE270	HEA500	85	95	81	202.5	33	40	30	M24/10.9
5	HEA220	HEB400	110	110	132	157.5	53	60/35	20	M20/8.8
6	HEA220	HEB500	100	90	132	157.5	47	65/40	30	M24/10.9
7	HEA220	HEA450	100	100	132	157.5	52	70/35	20	M20/8.8
8	HEA220	HEA360	100	90	132	157.5	48	70/40	30	M24/10.9

*if only one number appears, it means that $e = e_1$.

Table A2. Additional geometry and generalized forces of the optimum connections.

No	Base plate dimensions (mm)		Values of Generalized Forces within each Connection*			
	b_p	h_p	M_{jrd} (kNm)	M_{RBS} (kNm)	Objective Function as defined in [1]	V_{RBS} (kN)
1	170	370	68.84	44.13	3.98	102
2	175	440	116.23	77.503	2.3	143
3	170	350	65.09	41.42	4.2	95.78
4	165	460	116.33	77.503	2.398	143
5	230	400	143.878	98.86	2.014	157.143
6	240	350	165.162	108	10.182	171.686
7	240	370	154.185	100.384	10.134	159.567
8	240	350	157.483	106.483	4.68	169.262

*In all connections the value of the ratio $V_{RBS}/V_{pl,RBS}$ was found less than 42%.