Available online at www.eccomasproceedia.org
Eccomas Proceedia COMPDYN (2017) 4952-4964

ECCOMAS
Proceedia

COMPDYN 2017
6th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis (eds.)
Rhodes Island, Greece, 15–17 June 2017

FINITE ELEMENT ANALYSES ON SEISMIC RESPONSE OF PARTIAL STRENGTH EXTENDED STIFFENED JOINTS

Roberto Tartaglia[1], Mario D'Aniello[1], Raffaele Landolfo[1], Gian Andrea Rassati[2] and James Swanson[2]

[1] University of Naples 'Federico II',
Department of Structures for Engineering and Architecture
Via Forno Vecchio 36, Naples 80134, Italy
Roberto.tartaglia@unina.it, mdaniel@unina.it, landolfo@unina.it

[2] University of Cincinnati,
Department of Civil and Architectural Engineering and Construction Management
765 Badwin, Cincinnati OH 45221
rassatga@ucmail.uc.edu , swansojr@ucmail.uc.edu

Keywords: Free from damage joints, Steel structures, Finite element analyses, Seismic design.

Abstract. *Extended stiffened end-plate bolted joints are widely used in seismic resistant steel frames. In the United States (US) this type of joint is seismically pre-qualified according to AISC 358. In Europe within the framework of the ongoing EQUALJOINT research project, prequalification criteria for different types of bolted joints are under development. Differently from the US approach, in the EQUALJOINT procedure both full and partial strength joints are seismically qualified. The experimental tests carried out within the EQUALJOINT project confirmed the effectiveness of these intermediate strength levels. Therefore, the aim of this work is investigate the possibility to extend this design approach to US joints. The results of a comprehensive parametric finite element investigation are described and discussed, showing the effectiveness of the proposed design performance criteria.*

Peer-review under responsibility of the organizing committee of COMPDYN 2017.
doi: 10.7712/120117.5775.17542

1 INTRODUCTION

Extended stiffened end-plate bolted (ESEPB) joints are widely used in seismic resistant moment resisting frames. In United States of America (USA) ESEPB joints are seismically prequalified according to the requirements of AISC 358-16 [1]. The prequalification rules are based on the former studies carried out by [2-5].

In Europe, the current version of the Eurocodes (i.e. EN1993:1-8 [6] and EN1998-1 [7]) do not provide either specific requirements or codified prequalification procedures for seismic resistant extended stiffened end-plate joints. Recently, design criteria for seismic resistant ESEPB joints have been developed by D'Aniello et al. [8] within the framework of the EQUALJOINTS research project [9] (hereinafter referred as "EJ" for brevity sake). In particular, D'Aniello et al. [8] revised the ductility criterion of EN1993:1-8 [6] (which was conceived for non-seismic applications) in light of capacity design principles recommended by Eurocode 8 [7] and developed a novel design equation aiming at avoiding type 3 failure mode (i.e. failure of bolts) and enforcing ductile behaviour of ESEPB joints, as well. This purpose is fulfilled by controlling the thickness of the end-plate with respect to the diameter of bolts and accounting for both the material randomness and hardening of the weaker joint components. The introduction of ductility requirements as well as the revision of local hierarchy requirements allow using partial strength ESEPB joints for seismic applications. In common practice, stiffened joints are generally considered as full strength. However, partial strength ESEPB joints can be profitably used either in dual frame structures (e.g. MRFs + CBFs), where it is more convenient to have rigid joints accepting the damage of the connections, or in MRFs mostly designed to satisfy drift and/or stability checks [10]. In such cases, ESEPB joints are more efficient than unstiffened end-plate joints. In Europe, partial strength dissipative joints can be used provided that adequate ductility of the connection is verified, namely 0.035 rad for high ductility and 0.025 rad for moderate ductility. On the contrary, in United States of America the seismically prequalified joints are conceived to have full strength connections, as recommended by AISC 358-16 [1].

The aim of this study is to examine if relaxing the strength requirements of AISC358-16 [1] the US joints can provide adequate ductility to be adopted as dissipative partial strength or if it is necessary to introduce ad-hoc criteria as done in [8]. The paper is organized into two main parts. In the first part, the main differences between American (US) and European (EJ) design procedures are described and compared. Then, to point out their differences and effectiveness an example of two joints designed according to both the approaches is investigated by means of finite element analyses.

2 DESIGN PROCEDURE

2.1 EQUALJOINTS procedure

The seismic design of steel structures, in accordance with the EN1998-1-1 [7] philosophy, is based on the concept of dissipative structures that are designed to develop plastic deformations in specific zones, while the non-dissipative parts should resist elastically the seismic action in order to avoid brittle collapse [10-16]. This behaviour can be obtained imposing that the dissipative zones are the weaker and the non-dissipative are designed to resist the ultimate strength developed by the dissipative elements. The proposed design criteria extends this philosophy to the joints, by establishing a hierarchy among the strengths of macro-components (e.g. the web panel, the connection, the beam as shown in Figure 1, and their sub-components (e.g. end-plate, bolts, welds, etc.), as well.

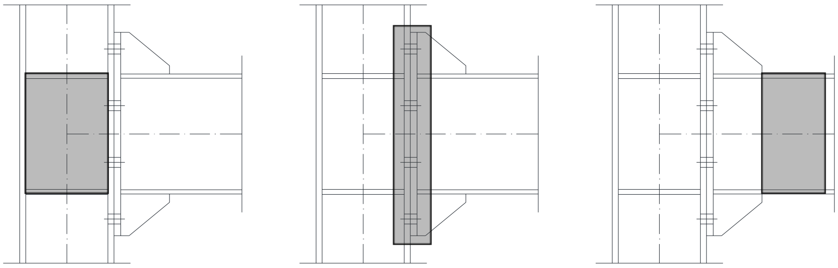


Figure 1 Plastic zones: a) web panel, b) connection and c) beam

The capacity design is applied between each macro-component in order to obtain two different design objectives defined comparing the joint (i.e. web panel and connection) strength to the beam flexural resistance, namely: full and equal strength. In the first case, the joint is designed to be more resistant respect to the beam, in order to concentrate all the plastic deformation in the connected element. On the other hand, equal strength joints are designed to have a balanced strength between the connection and the beam bending capacity; in this way, the plastic deformation should invest both the joint and the beam.

The capacity design requirements to obtain the required joint behaviour can be guaranteed by satisfying the following inequality:

$$M_{wp,Rd} \geq M_{\mathrm{j},Rd} \geq M_{\mathrm{j,E}d} = \alpha \cdot (M_{\mathrm{B},Rd} + V_{B,Ed} \cdot s_h) \tag{1}$$

Where $M_{\mathrm{wp,Rd}}$ is the flexural resistance corresponding to the strength of column web panel; $M_{\mathrm{j,Rd}}$ is the flexural strength of the connection; $M_{\mathrm{j,Ed}}$ is the design bending moment at the column face; $M_{\mathrm{B,Rd}}$ is the plastic flexural strength of the connected beam; s_{h} is the distance between the column face and the tip of the rib stiffener; $V_{\mathrm{B,Ed}}$ is the shear force corresponding to the occurring of the plastic hinge in the connected beam.

The α factor depends on the design performance level. In case of non-dissipative joints it is equal to $\gamma_{\mathrm{sh}} \times \gamma_{\mathrm{ov}}$ (being γ_{sh} is the strain hardening factor corresponding to the ratio between the ultimate over the plastic moment of the beam [1, 17-21], while γ_{ov} accounts for material randomness, depending on the steel grade [7]).

In case of dissipative joints $\alpha \leq 1$, thus accepting that the joint is weaker than the connected beam provided that adequate ductility is guaranteed. In particular, within EQUALJOINTS project [9] different levels of partial strength joints are considered depending on both the strength of the connection and the resistance of column web panel. Differently from the codified definitions of current EC3-1-8 [6], which consider as "partial strength" all connections with resistance smaller or at least equal to the strength of the beam, in EQUALJOINTS the connection with strength equal to the strength of the connected beam (i.e. $\alpha = 1$) is defined as "Equal Strength". If the strength of the connection is lower it is referred as the "Partial Strength", with a fixed threshold of resistance $\alpha = 0.8$ that corresponds to the lower bound limit to qualify the strength of joints in special moment resisting frames according to AISC341-16 [1].

The column web panel can be strong, balanced or weak depending on its strength compared to the strength of the dissipative zone, which is the connection for both equal and partial strength connections. Hence, based on these considerations, Equation (1) can be rearranged for dissipative joints as follows:

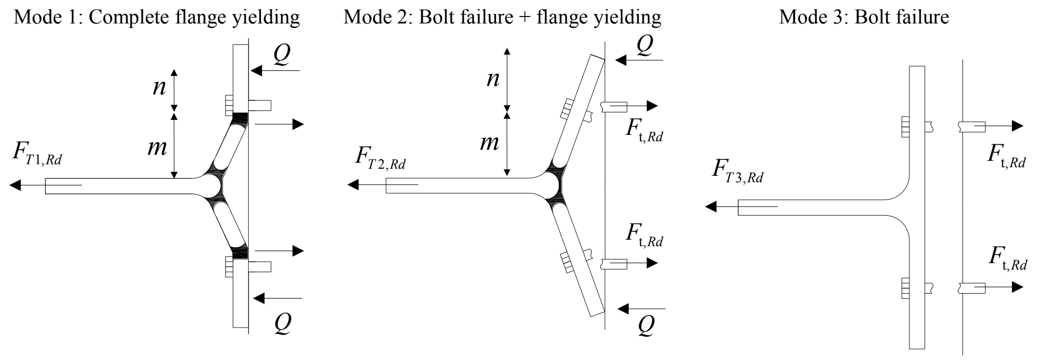
$$M_{wp,Rd} > M_{\mathrm{j},Rd} \geq M_{\mathrm{j,E}d} = \alpha \cdot (M_{\mathrm{B},Rd} + V_{B,Ed} \cdot s_h) \tag{2}$$

$$M_{wp,Rd} = M_{\mathrm{j},Rd} \geq M_{\mathrm{j,E}d} = \alpha \cdot (M_{\mathrm{B},Rd} + V_{B,Ed} \cdot s_h) \tag{3}$$

$$M_{\mathrm{j},Rd} \geq M_{wp,Rd} \geq M_{\mathrm{j,E}d} = \alpha \cdot (M_{\mathrm{B},Rd} + V_{B,Ed} \cdot s_h) \tag{4}$$

In order to enforce the most ductile behavior of dissipative joints, the connection should be designed to avoid any brittle failure mode. Starting from the component method [6], the attention focuses on the response of the equivalent T-Stub per bolt row. With this regard, it is well known that the resistance of the equivalent T-stub can be evaluated as the minimum of the corresponding three failure modes, as illustrated in Figure 2, which are described as it follows:

- Mode 1 is characterized by the plasticization of the flange, whereas the bolts are not involved in the failure mechanism.
- Mode 2 is characterized by the combined flange plasticization and failure of the bolts.
- Mode 3 is characterized by the failure of the bolts and it does not involve any plastic engagement of the T-stub flange.



$$F_{T1,Rd} = \frac{4M_{pl1,Rd}}{m} \qquad F_{T2,Rd} = \frac{2M_{pl2,Rd} + n\sum F_{t,Rd}}{m+n} \qquad F_{T3,Rd} = \sum F_{t,Rd}$$

Figure 2 Failure mode of bolted T-Stub

Therefore, the joint ductility depends on the type of failure mode and the corresponding plastic deformation capacity of the activated component. Figure 3 concisely depicts the dependency of the failure mode with the geometric properties and end-plate to bolt strength ratio [6]. In abscissa is reported the ratio β between the flexural strength ($M_{\mathrm{pl,Rd}}$) of the plates, or column flanges, and the axial strength of the bolts ($F_{\mathrm{t,Rd}}$), while the vertical axis reports the ratio η between the T-stub strength (F) over $F_{\mathrm{t,Rd}}$.

In line with Fig. 3, two possible ductility criteria can be adopted to avoid mode 3, namely:

Level-1: $\beta < 1$ this condition imposes either a failure mode I or failure mode II (but very close to mode I), which provides very high ductility.

Level-2: $\beta < 2$ and $\eta < 0.95$, this condition imposes a failure mode II with limited ductility, but avoiding brittle failure.

The level of ductility to be guaranteed obviously depends on the design performance objectives. It is crucial providing the larger ductility for equal strength, less for full strength joints.

As for the rotational capacity of the joint, EN1993:1-8 [6] requires that $M_{\mathrm{j,Rd}}$ to be less than 1.2 $M_{\mathrm{B,pl,rd}}$ and two alternative ways can be pursued: 1) performing experimental tests; 2) controlling the thickness t of both end-plate and column flange. Therefore, in line with the Component method [6], also the hereby proposed design criteria introduces a limitation for the plate thickness respect to the bolt diameter:

$$t \leq \frac{0.42 \cdot d}{\sqrt{\gamma_{ov} \cdot \gamma_{sh}}} \cdot \sqrt{\frac{\gamma_{\mathrm{M0}} \cdot f_{ub}}{\gamma_{\mathrm{M2}} \cdot f_y}} \simeq 0.30 \cdot d \cdot \sqrt{\frac{f_{ub}}{f_y}} \tag{5}$$

Where d is the nominal bolt diameter, f_y is the yield strength of the relevant basic component and f_{ub} is the bolt ultimate strength. Moreover, another verification on the bolts strength was introduced, in order to verify each line resistance with respect to the one of the bolts:

$$F_{t,Rd} \geq \gamma_{ov} \cdot F_{p,Rd} \tag{6}$$

Where $F_{t,Rd}$ is the resistance of the bolts in tension and $F_{p,Rd}$ is the resistance of the entire line. It is important to highlight that all criteria previously described require that the failure of the welds has to be unquestionably avoided, because of their brittle collapse mechanism.

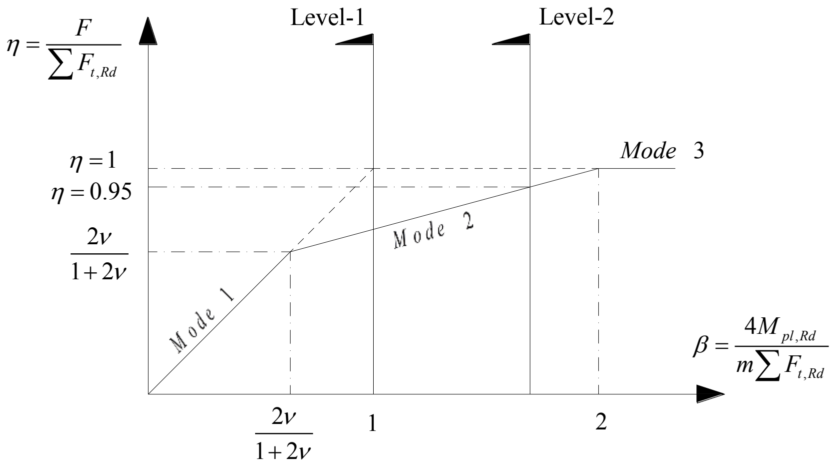


Figure 3 T-Stub resistance and corresponding failure mechanism [6]

2.2 AISC358 procedure and its extension to partial strength joints

AISC 358-16 [1] considers hierarchy of resistances between the beam and the connection, but not clearly between the connection and column web panel. A substantial difference between AISC 358-16 [1] and EQUALJOINTS [8, 9] procedure is the way to compute the resistance of the end-plate. Thus, starting from the equivalent T-stub theory implemented in EN1993:1-8 [6], EJ procedure [8,9] evaluates the resistance of each active bolt row (namely, the two bolt rows above and the one below the beam flange in tension) in function of the length of its corresponding yielding line. On the contrary, American procedure [1] directly provides the length of the yield line based on the adopted joint configuration (i.e. for both 4 bolts and 8 bolts configuration).

AISC 358-16 [1] does not introduce specific requirements considering that the extended stiffened joints are conceived to be theoretically full strength, without experiencing plastic deformations. Therefore, the limitations on both minimum bolt diameter and thickness of end-plate are given separately and solely related to a strength verification against the design bending moment at the column face M_f, which is similar to $M_{j,Ed}$ of Eq. (1) and defined as follows:

$$M_f = M_{pr} + V_u \cdot s_h = M_{pr} + \left(\frac{2M_{pr}}{L_h} + V_{gravity} \right) \cdot s_h \tag{7}$$

Where M_f is the probable flexural resistance of the beam (i.e. calculated considering the average steel yield stress magnified to account for strain hardening of the plastic hinge); s_h is the distance between the column face and the tip of the rib stiffener; $V_{gravity}$ is the shear force due to gravity load and L_h is the distance between the plastic hinges.

The AISC 358-16 [1] limitations on the minimum diameter for bolts are given as follows:

For bolt row line (4E and 4ES): $$d_{b,required} = \sqrt{\frac{2M_f}{\pi \phi_n F_{nt} \left(h_0 + h_1 \right)}} \tag{8}$$

For bolt row line (8E and 8ES): $$d_{b,required} = \sqrt{\frac{2M_f}{\pi \phi_n F_{nt} \left(h_1 + h_2 + h_3 + h_4 \right)}} \tag{9}$$

where F_{nt} is the nominal tensile strength of a bolt; hi is the distance from the centerline of the beam compression flange to the centerline of the i-th tension bolt row; ho is distance from the centerline of the compression flange to the tension-side outer bolt row. The minimum thickness for the end plate and the column flange should comply the following requirements:

minimum thickness of the end-plate: $$t_{p,required} = \sqrt{\frac{1.11M_f}{\phi_d F_{yp} Y_p}} \tag{10}$$

minimum thickness of the column flange: $$t_{cf} \geq \sqrt{\frac{1.11M_f}{\phi_d F_{yc} Y_c}} \tag{11}$$

where F_{yp} is the specified minimum yield stress of the end-plate material; Y_p is the end-plate yield line mechanism parameter given from the relevant Tables 6.2, 6.3 or 6.4 of AISC358-16 [1]. F_{yc} is the specified minimum yield stress of the column flange material; Y_c is the unstiffened column flange yield line mechanism parameter from the relevant Tables 6.5 or 6.6 of AISC358-16 [1].

In the light of AISC design procedure, either equal or partial strength joints can be designed assuming $M_f^* = \alpha M_f$, as assumed in Section 2.1. In addition, Equation (10) and (11) can be further revised assuming ϕ_d=1.

In order to investigate the influence of the revised criteria and to compare with EQUALJOINTS procedure, two beam-to-column joints were designed alternatively according the two approaches and their nonlinear response was examined by means of finite element (FE) analyses.

The geometrical features of ESEPB joints and the relevant parameters are reported in Table1 and in Fig. 4.

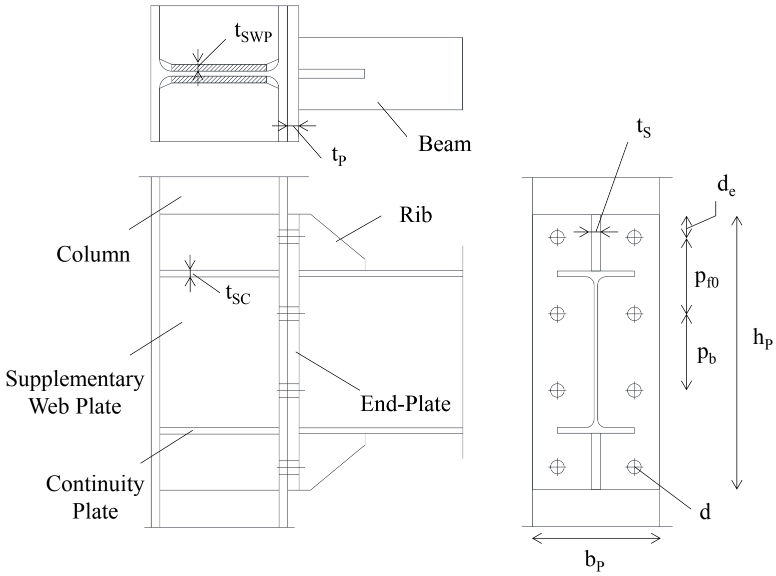


Figure 4 Joint geometry [8]

Table 1 Features of the examined joints

				End-Plate							CP	SWP		Rib	
Design	Column	Beam	Bolt Rows	Bolt	b_P	h_P	t_P	d_e	p_{f0}	p_b	t_{SC}	n	t_{SWP}	Slope	t_S
				[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[-]	[mm]	[-]	[mm]
AISC-Rev	W14×53	W14×38	4	25.4	205	549	19	44	115	231	13	1	16	30°	12.7
Equaljoints	HE 280 B	IPE 360	4	30	280	600	16	50	75	160	15	0	0	40°	20

Table 1 Features of the examined joints

3 FINITE ELEMENT ANALYSES

3.1 Modelling assumptions

Finite element analyses (FEAs) were carried out using ABAQUS 6.14 [23]. The beam-to-column joints were modelled considering a sub-assemblage obtained by extracting the beam and the column at the inflection points of the bending moment diagram induced by lateral loads on the reference MRF. Therefore, according to the sub-structuring procedure the horizontal and vertical displacements are restrained at the column extremities, while a displacement history is applied at the beam tip (see Figure 5).

Moreover, lateral torsional restrains are introduced in the FE model to simulate the beneficial effect of the slab on the beam.

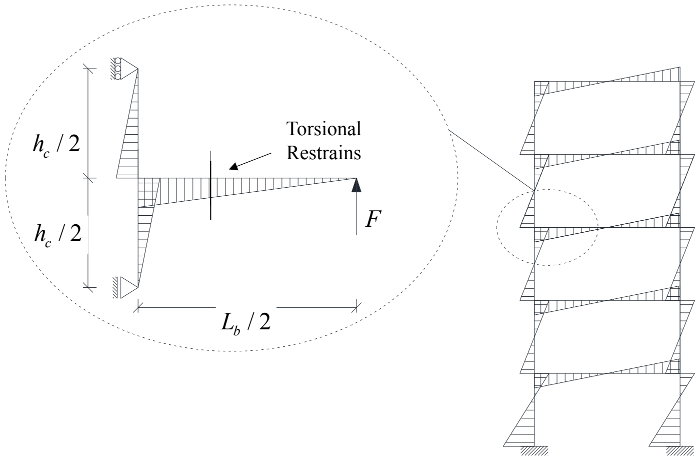


Figure 5 Joint geometry sub-structuring [8]

The material non linearity was considered applying a true stress-true strain curve coming from coupon tests performed during experimental tests carried out within the EQUALJOINTS research project [9]. Geometrical non linearity was neglected to focus the attention only on the joint behaviour, disregarding the beam imperfections.

C3D8I element (i.e. 8-node linear brick, incompatible mode) was adopted for meshing of all parts constituting the joint, i.e. column, beam, plates, bolts and welds.

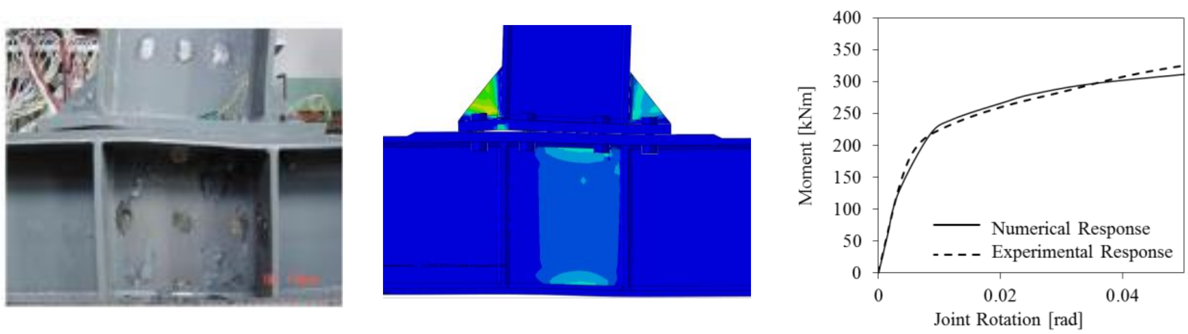
Three type of contact were introduced in the FE model, as follows:

- ”Hard” contact: to model the normal contacts;
- Coulomb friction with friction coefficient equal to 0.3 for tangential behaviour;
- “Surface-to-surface” interaction: to model the contacts between adjacent surfaces.

S355 steel material, with average yield stress as recommended by EN1998-1 [7], was used for the beam, column, and plates constituting the European joints; while ASTM A992 steel with average as recommended by AISC 341-10 [23].

The Von Mises yield criterion was adopted to model steel yielding; plastic hardening was simulated using both nonlinear kinematic and isotropic hardening law on the basis of the data provided by Dutta et al. [24]. Both fillet and full penetration welds were modelled as elastic perfectly plastic material [25, 26].

The material of European high strength bolts is modelled as described by [27], while for American bolts the assumptions given by [28] were considered.



Experimental result PEEQ distribution Moment rotation curves

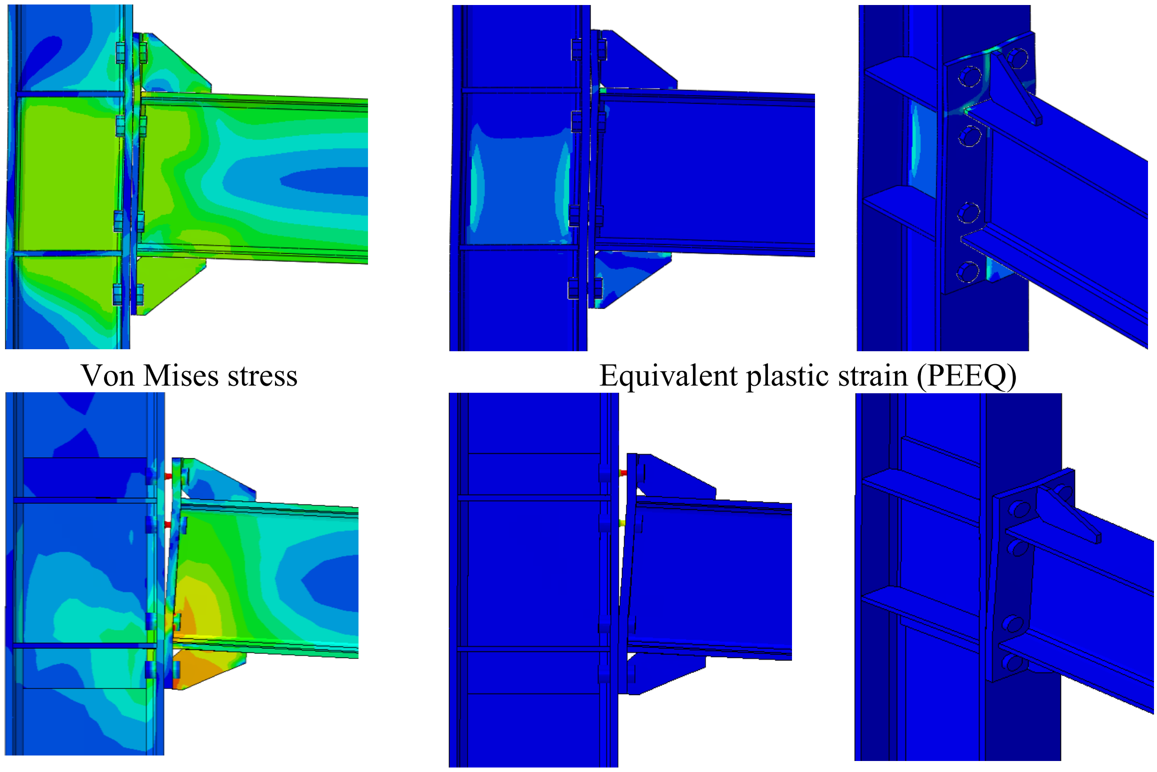
Figure 6 Experimental tests performed by Shi et al. [29] compared with finite element simulation

The FEM assumptions were validated against one of the experimental tests carried out by Shi et al. [29]. Figure 6 depicts the comparison between numerical and experimental results in terms of joint moment rotation curves and distribution of the equivalent plastic deformation (PEEQ). As it can be noted the FEM modelling procedure is able to catch both the initial elastic stiffness and the resistance; moreover also the distribution of the plastic deformation between the experimental tests and the FEM model are comparable [30].

3.2 Numerical Results

The differences between both the beam and the column steel profile of the two investigated beam-to-column assemblies are negligible. Even the values of the yield stress of the steel profiles are very similar.

The results obtained from FEAs show that both stiffness and resistance of the two assemblies are very similar. However, significant differences can be noted in terms of failure mode.



a) Von Mises stress — Equivalent plastic strain (PEEQ)

b) Von Mises stress — Equivalent plastic strain (PEEQ)

Figure 6 Failure modes of the partial strength joints: a) Equaljoints procedure; b) modified AISC procedure

Figure 6 depicts the comparison between the simulated failure modes, while the relevant response curves are reported in Figure 7. As it can be noted, the partial strength joint designed according to [8,9] exhibits a ductile failure mode without strength degradation beyond the threshold of 0.04 radians of chord rotation. On the contrary, the joint designed with the modified AISC358 procedure shows a low ductile response with significant degradation occurring at 0.025 radians due to the brittle failure of bolts in tension. This result is also confirmed by the curves reported in Figure 8, where the ratio between the dissipated energy by each components with respect to the whole dissipated energy by the joints is plotted. Figure 8a shows that in the joint designed according to European procedure [8,9] the energy is dissipated mainly by the column web panel in shear, but also into the other components i.e. end-plate, beam and ribs participate to the failure mode. Contrariwise, consistently with the PEEQ distribution showed in Figure 6, in the American joint, the most of energy is dissipated by the bolts (Figure 8b). Figure 8c depicts the comparison between the energy dissipated by the bolts into both joints, where it can be noted the noticeably higher plastic engagement of the bolts into the American connection.

This comparison highlights that the design rules of current AISC358-16 [1] cannot be extended to partial strength joints, but further design rules are necessary to guarantee satisfactory ductility. On the contrary, the ductility limitations introduced in [8,9] ensure satisfactory ductility.

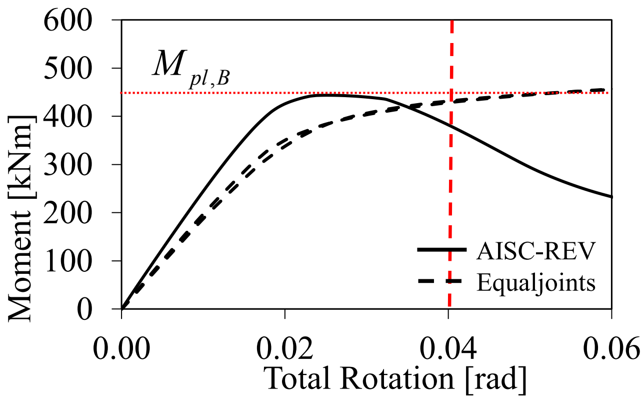


Figure 7 Response curves of the partial strength joints: Equaljoints vs. modified AISC procedure

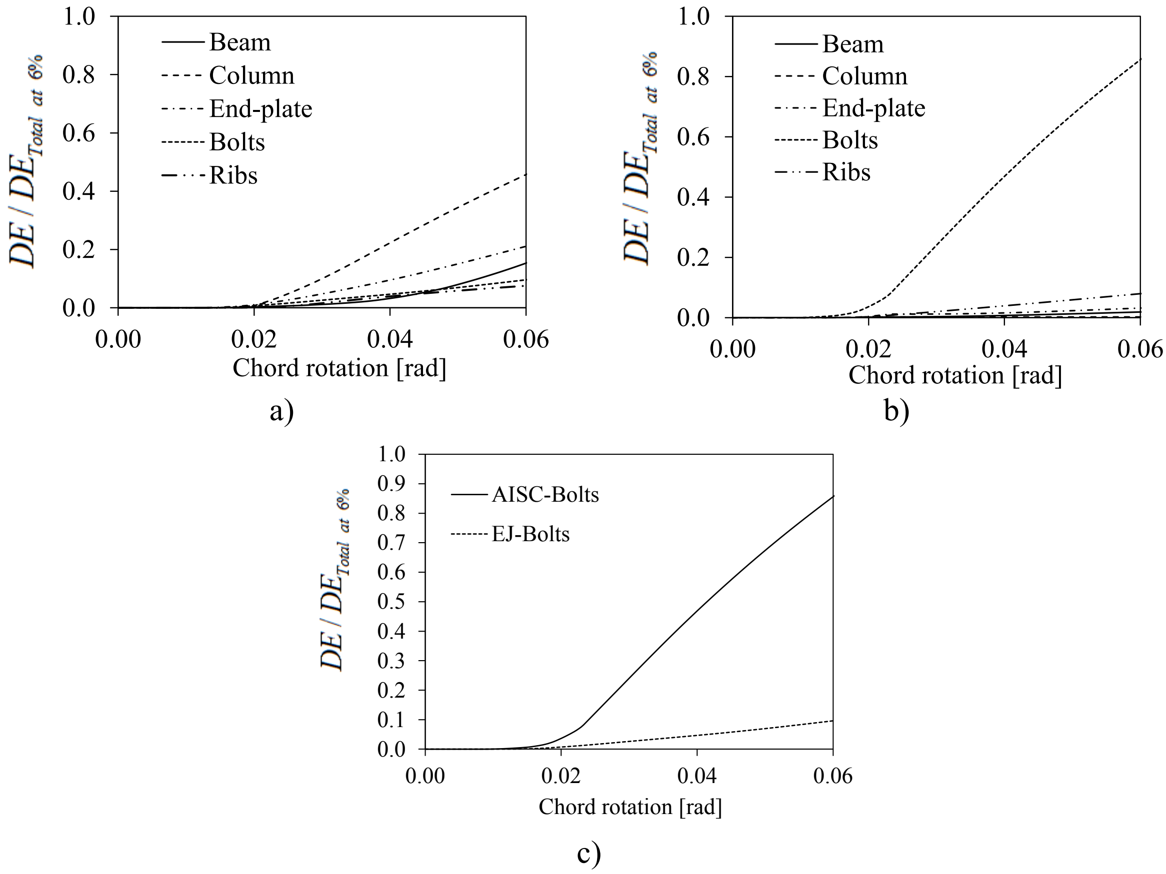


Figure 8 Dissipated energy for all the joints component respect to the global one at 6% of rotation: a) Equaljoints vs. b) modified AISC

4 CONCLUSIONS

This paper describes the design criteria developed within EQUALJOINTS project [9] for partial strength extended stiffened end-plate joints. In addition, the design rules of AISC358-16 [1] are described and possible extension to partial strength joints is examined. The main differences of these design procedures are discussed and compared by means of finite element analyses. Based on the results, the following remarks can be drawn:

- The EQUALJOINTS design criteria [8, 9] recommend ductility requirements to avoid the failure of bolts and enforce the plastic deformations into the end-plate of the connection.
- The finite element analyses confirm that this design procedure guarantee satisfactory ductility.
- AISC358-16 [1] does not provide similar requirements, because this code imposes that the strength of the connection should be higher than the resistance of the connected beam. Therefore, scaling the design resistance of the connection does not guarantee any ductility of the joints.
- The finite element simulations highlight that AISC358-16[1] needs further rules and requirements to design ductile partial strength extended stiffened end-plate joints.

5 ACKNOWLEDGMENT

The research leading to these results has received funding from the European Union's Research Fund for Coal and Steel (RFCS) research programme under grant agreement n° RFSR-CT-2013-00021.

REFERENCES

[1] AISC (American Institute of Steel Construction). "Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications." ANSI/AISC 358-16, Chicago, IL, 2016.

[2] T. M. Murray. AISC Design Guide 4, "Extended End-Plate Moment Connections", AISC (American Institute of Steel Construction), Chicago, IL, 1990.

[3] E.A. Sumner, T. M. Murray. Performance of Extended Moment End-Plate Connections Subject to Seismic Loading. U.S.–Japan Workshop on Seismic Fracture Issues in Steel Structures, San Francisco, California, February 28 - March 1, 2000.

[4] E.A. Sumner, T. M. Murray. Behavior of Extended End-Plate Moment Connections Subject to Cyclic Loading. *Journal of Structural Engineering* ASCE, **128(4)**, 501-508, 2002.

[5] T. M. Murray, E.A. Sumner. AISC Design Guide 4, "Extended End-Plate Moment Connections—Seismic and Wind Applications, 2nd Edition". AISC (American Institute of Steel Construction), Chicago, IL, 2004.

[6] CEN 2005. CEN (European Committee for Standardization). Design of steel structures, part 1.8: Design of joints. EN 1993:1-8, Brussels, 2005.

[7] CEN 2005. CEN (European Committee for Standardization). Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings. EN 1998-1, Brussels, 2005.

[8] M. D'Aniello, R. Tartaglia, S. Costanzo, R. Landolfo. Seismic design of extended stiffened end-plate joints in the framework of Eurocodes. *Journal of Constructional Steel Research*, **128**, 512-527, 2017.

[9] EQUALJOINTS – European pre-QUALified steel JOINTS: RFSR-CT-2013-00021. Research Fund for Coal and Steel (RFCS) research programme.

[10] A. Tenchini, M. D'Aniello, C. Rebelo, R. Landolfo, L. da Silva, L. Lima. High strength steel in chevron concentrically braced frames designed according to Eurocode 8. *Engineering Structures,* **124**, 167–185, 2016.

[11] F.M. Mazzolani, V. Piluso. Plastic design of seismic resistant steel frames. *Earthquake Engineering and Structural Dynamics*, **26 (2)**, 167-191, 2017.

[12] R. Montuori, E. Nastri, V. Piluso. Theory of Plastic Mechanism Control for MRF-EBF dual systems: Closed form solution. *Engineering Structures*, **118**, 287-306, 2016.

[13] R. Montuori, E. Nastri, V. Piluso. Theory of plastic mechanism control for eccentrically braced frames with inverted y-scheme. *Journal of Constructional Steel Research*, **92**, 122-135, 2014.

[14] D. Cassiano, M. D'Aniello, C. Rebelo, R. Landolfo, L. da Silva. Influence of seismic design rules on the robustness of steel moment resisting frames. *Steel and Composite Structures, An International Journal*, **21(3)**, 479-500, 2016.

[15] A. Tenchini, M. D'Aniello, C. Rebelo, R. Landolfo, L. da Silva, L. Lima. High strength steel in chevron concentrically braced frames designed according to Eurocode 8. *Engineering Structures* **124**, 167–185, 2016.

[16] M. Latour, V. Piluso and G. Rizzano, Free from damage beam-to-column joints: Testing and design of DST connections with friction pads, *Engineering Structures* **85**, 219–233, 2015.

[17] M. D'Aniello, E.M. Güneyisi, R. Landolfo, K. Mermerdaş. Predictive models of the flexural overstrength factor for steel thin-walled circular hollow section beams. *Thin-Walled Structures*, **94**, 67-78, 2015.

[18] M. D'Aniello, E.M. Güneyisi, R. Landolfo, K. Mermerdaş. Analytical prediction of available rotation capacity of cold-formed rectangular and square hollow section beams. *Thin-Walled Structures*, **77**,141-152, 2014.

[19] E.M. Güneyisi, M. D'Aniello, R. Landolfo, K. Mermerdaş. Prediction of the flexural overstrength factor for steel beams using artificial neural network. *Steel and Composite Structures, An International Journal*, **17(3)**, 215-236, 2014.

[20] M. D'Aniello, R. Landolfo, V. Piluso, G. Rizzano. Ultimate Behaviour of Steel Beams under Non-Uniform Bending. *Journal of Constructional Steel Research*, **78**, 144–158, 2012.

[21] M. Güneyisi, M. D'Aniello, R. Landolfo, K. Mermerdaş. A novel formulation of the flexural overstrength factor for steel beams. *Journal of Constructional Steel Research*, **90**, 60-71, 2013.

[22] Dassault (2014), Abaqus 6.14 - Abaqus Analysis User's Manual, Dassault Systèmes Simulia Corp.

[23] ANSI/AISC 341-16 (2016). "Seismic Provisions for Structural Steel Buildings". American Institute of Steel Construction.

[24] A. Dutta, S. Dhar, S. K.Acharyya. Material characterization of SS 316 in low cycle fatigue loading. *Journal of Material Science*, **45**,1782–1789, 2010.

[25] M. Latour, G. Rizzano, A. Santiago, L. da Silva. Experimental analysis and mechanical modeling of T-stubs with four bolts per row. *Journal of Constructional Steel Research*, **101**,158–174, 2014.

[26] M. Latour, V. Piluso, G. Rizzano. Rotational behaviour of column base plate connections: Experimental analysis and modelling. *Engineering Structures*, **68**, 14-23, 2014.

[27] M. D'Aniello, D. Cassiano and R. Landolfo, Monotonic and cyclic inelastic tensile response of European preloadable gr10.9 bolt assemblies, *Journal of Constructional Steel Research*, **124**, 77–90, 2016.

[28] J.A. Swanson, D.S. Kokan, R.T. Leon. Advanced finite element modelling of bolted T-stub connection components. *Journal of Constructional Steel Research*, **58(5–8)**,1015–31, 2002.

[29] G. Shi, Y. Shi, Y. Wang. Behaviour of end-plate moment connections under earthquake loading. *Engineering Structures*, **29**, 703–716, 2007.

[30] R. Tartaglia, M. D'Aniello. Nonlinear performance of extended stiffened end plate bolted beam-to-column joints subjected to column removal. *The Open Civil Engineering Journal*, **11(3-15)**, 2017.