DESIGN OF FULL-STRENGTH FULL-DUCTILITY EXTENDED END-PLATE BEAM-TO-COLUMN JOINTS

Antonella B. Francavilla, Massimo Latour, Vincenzo Piluso and Gianvittorio Rizzano
Department of Civil Engineering, University of Salerno

ABSTRACT

The analysis and modelling of the ultimate behaviour of the beam-to-column connections is certainly one of the most studied topics in the field of steel structures. In particular, seismic design of steel frames is commonly carried out to assure the dissipation of the seismic input energy in the so-called “dissipative zones” which have to be properly detailed in order to assure wide and stable hysteresis loops. Once avoided the yielding of columns, beam-to-column joints play a role of paramount importance. In fact, beam-to-column joints can be designed either as Full Strength (FS) or Partial Strength (PS). In the first case, the seismic input energy is dissipated by means of plastic cyclic excursions of the beam ends. In the second case, dissipation requires the plastic engagement of ductile joint components.

This paper addresses the design criteria to be adopted to assure full-strength full-ductility behaviour of Unstiffened Extended End-Plate (U-EEP) beam-to-column joints. The validation of the design procedure is accomplished by three-dimensional finite element analyses with ABAQUS 6.13 software. Finally, in order to clarify the design procedure in detail, a worked numerical example concerning the design of an external joint is presented.

1. INTRODUCTION

Within the analysis of steel structures, the modelling of the ultimate behaviour of beam-to-column joints is one of the most studied topic. As well known, before the introduction of the *semi-rigidity* concept [1,2], steel frame design was accomplished by properly considering a limit assumption regarding the joint behaviour. Depending on the beam-to-column joint typology, it was either assumed that all the ends of the members converging in the joint are subjected to the same rotation and the same displacements or assumed that the joints are able to permit free rotations. The first case leads to *continuous frames,* while the second one to *pinned frames.* The application of the semi-rigidity concept has required the development of a general methodology working out in detail the provision of the rotational stiffness and the flexural

resistance of joints. This resulted in a strong effort, in Europe more than in United States, which has led to the complete definition and codification of the component method [3,4]. This allows the analysis of actual semi-continuous structural systems, starting from the geometrical and mechanical properties of beam-to-column joints.

The component method is essentially based on mechanical models constituted by the assembling of spring elements modelling the joint components. The non-linearity of the joint moment-rotation response is obtained starting from the inelastic constitutive laws adopted for the components. The method is suitable for the modelling of any kind of joint provided that the components are properly identified and their constitutive law is deservedly modelled.

Even though some authors have already investigated some aspects related to the prediction of the plastic deformation capacity [5-8, 39, 40] and of the cyclic behaviour of connections [9-13, 41] past experimental and theoretical researches have often focused their attention mainly on the prediction of the stiffness and resistance of joint components. Therefore, the prediction of the plastic deformation capacity of connections is still an open research field whose primary aim is devoted to the prediction of the plastic rotation capacity of partial strength connections.

Moreover, it cannot be denied that the classification of beam-to-column joints as full-strength or as partial-strength is too simplistic, because it is rigorous only in the pure theoretical case in which both the joint that the connected member exhibit a perfectly plastic behaviour. As soon as the distinction between the joint and the connection is made (Fig.1), allowing the definition of the joint as the combination in series of the connection and the panel zone of the column web, also the concept of ***beam-joint system*** becomes noticeable, being constituted by the combination in series of the beam-to-column joint and the beam end [14].

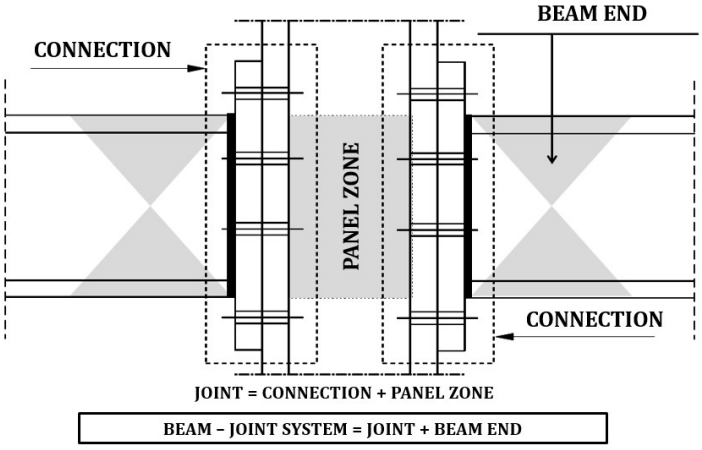


Figure 1 - Beam-joint system

This concept is of primary importance under the point of view of yielding location and, therefore, for seismic design purposes. This statement can be easily explained considering a tri-linear modelling of the moment-rotation curve of both the beam-to-column joint and the beam end (Fig.2). In fact, generally the plastic rotation supply Θ_{pu} of the beam-joint system can be regarded as the sum of two contributions: the plastic rotation of the beam-to-column joint φ_p and the plastic rotation provided by the beam end ϑ_p. Therefore, an accurate evaluation of the moment-rotation curve of the beam-to-column joint is required, because the plastic rotation provided by the beam end is strictly dependent on the flexural resistance that the beam-to-column joint is able to develop [14].

Concerning beam-to-column joint, $M_{j.y}$ is the value of the bending moment leading to first yielding, $M_{j.p}$ is the conventional plastic moment defining the knee of the moment-rotation curve according to Eurocode 3, $M_{j.u}$ is the theoretical ultimate flexural resistance of the beam-to-column joint. Regarding the beam, $s\,M_{pb}$ is the bending moment corresponding to the occurrence of local buckling of the beam compressed flange.

The parameter *s* is the non-dimensional buckling stress depending on the width-to-thickness ratios of the plate elements constituting the beam section and on the longitudinal stress gradient. Starting from the analysis of the experimental data [15,16], by means of a multiple regression analysis, Mazzolani and Piluso [17] defined the following empirical relationship:

$$s = \frac{1}{0.546321 + 1.632533\lambda_f^2 + 0.062124\lambda_w^2 - 0.602125\frac{b_f}{L_e} + 0.001471\frac{E}{E_h} + 0.007766\frac{\varepsilon_h}{\varepsilon_y}} \leq \frac{f_u}{f_y} \qquad (1)$$

where λ_f and λ_w are, respectively, the normalized slenderness parameters of the flange and of the web equal to:

$$\lambda_f = \frac{b_f}{2\,t_f}\sqrt{\frac{f_{ym.bf}}{E}} \quad \text{and} \quad \lambda_w = \frac{d_w}{2\,t_w}\sqrt{\frac{f_{ym.bw}}{E}} \qquad (2)$$

where b_f is the flange width, t_f is the flange thickness, d_w is the compressed part of the beam web, t_w is the web thickness, L_e is the shear length of the beam, E is the Young modulus, $f_{ym.bf}$ is the average value of the yielding resistance of the beam flange, $f_{ym.bw}$ is the average value of the yielding resistance of the beam web, E_h is the hardening modulus, ε_y is the strain corresponding to yielding, ε_h is the strain corresponding to the end of the yield plateau and f_u and f_y are respectively the ultimate and the yielding resistance of the material composing the beam.

Four significant cases can arise [14] (Fig.2):

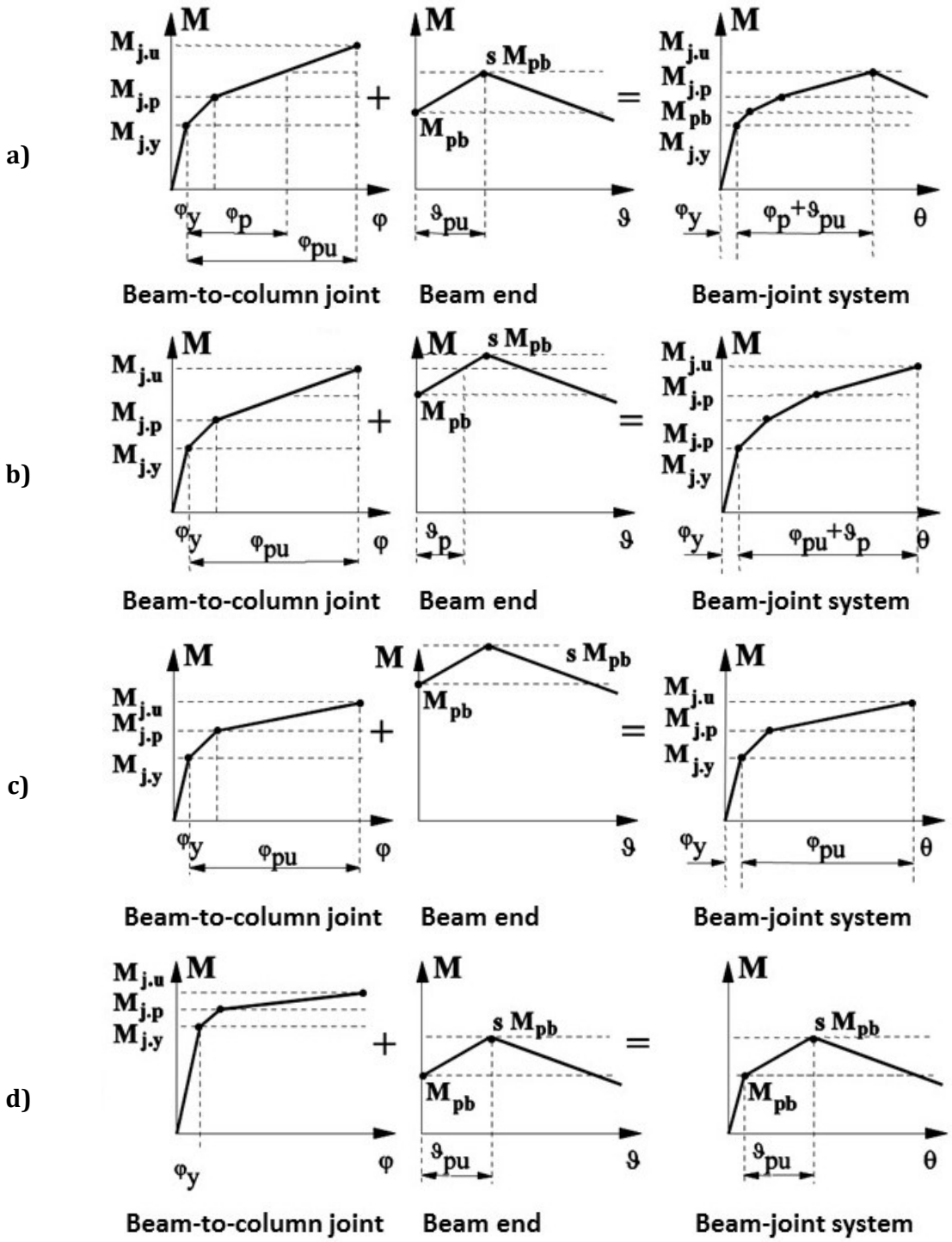


Figure 2 - *Plastic rotation supply of the beam-joint system [14]*

79

80 a) $M_{j.u} \geq s\, M_{pb}$

81 In this case the ultimate resistance of the beam-to-column joint allows the complete
82 exploitation of the beam plastic reserves, so that:

$$\vartheta_p = \vartheta_{pu} \quad \text{and} \quad \varphi_p \leq \varphi_{pu} \tag{3}$$

where ϑ_{pu} is the ultimate plastic rotation of the beam and φ_{pu} is the theoretical value of the ultimate plastic rotation of the beam-to-column joint.

Therefore, the plastic rotation supply of the beam-joint system is given by the sum of the beam plastic rotation supply and a part, for $M_{j.u} > s\,M_{pb}$, or the total value, for $M_{j.u} = s\,M_{pb}$, of the plastic rotation supply of the beam-to-column joint. As the plastic rotation supply of the beam-joint system is greater than the plastic rotation capacity of the connected beam, the beam-to-column joints can be defined as full-strength full-ductility.

b) $M_{pb} \leq M_{j.u} < s\,M_{pb}$

In this case, even though the beam end can be engaged in plastic range, the ultimate resistance of the beam-to-column joint is not sufficient to completely exploit the beam plastic reserves, so that:

$$\vartheta_p < \vartheta_{pu} \quad \text{and} \quad \varphi_p = \varphi_{pu} \tag{4}$$

Therefore, the plastic rotation supply of the beam-joint system is given by the sum of the plastic rotation supply of the joint and of a part of that of the connected beam. The beam-to-column joint can be defined as full-strength (because $M_{j.u} > M_{pb}$), but cannot be defined "a priori" as full-ductility, because the plastic rotation capacity of the beam-joint system is strictly dependent on the contribution (φ_{pu}) due to the beam-to-column joint.

c) $M_{j.u} \leq M_{pb}$

In this case, the ultimate resistance of the beam-to-column joint is not sufficient to engage the beam in plastic range, so that:

$$\vartheta_p = 0 \quad \text{and} \quad \varphi_p = \varphi_{pu} \tag{5}$$

Therefore, the ultimate plastic rotation of the beam-joint system is coincident with the plastic rotation of the beam-to-column joint. The beam-to-column joint can be defined as partial-strength. Nothing can be said, a priori, about on the degree of restoration of rotation capacity, because the plastic rotation capacity of the beam-joint system is strictly dependent on φ_{pu}.

d) $M_{j.y} > sM_{pb}$

In this case, the elastic flexural resistance is sufficient to completely exploit the plastic reserves of the beam, so that:

$$\vartheta_p = \vartheta_{pu} \quad \text{and} \quad \varphi_p = 0 \tag{6}$$

Consequently, the plastic rotation of the beam-joint system is equal to the plastic rotation of the beam end. The beam-to-column joint can be referred as full-strength full-ductility. The difference with respect to case a) is that the beam-to-column joint remains in elastic range ($\varphi_p = 0$).

Regarding the evaluation of the plastic rotation of the beam end, simple relations are available in literature [18-21]. In addition, the plastic rotation of the beam-to-column joints can be determined starting to the knowledge of the plastic deformation of each component, through an advanced modelling of their force-displacement law (up to the ultimate displacement). In fact, the plastic displacement occurring at the tensile flange level is equal to the sum of the ultimate displacement of the weakest component and of the contributions of the other components. The resulting plastic rotation is given by the ratio of that such plastic displacement and the level arm [14].

It is important to underline that the above classification approach of the beam-to-column connection as full-strength or as partial strength is different from that adopted in Eurocodes. IIn Eurocode 3 Part 1-8 [3], it is dearly stated that a joint can be classified as full-strength by comparing it's design moment resistance $M_{j.Rd}$with the design moment resistance of the connected member $M_{b.Rd}$. Conversely, as the classification approach herein presented in mainly devoted to underline the role of the joint flexural resistance on the plastic rotation supply of the beam-joint system, the comparison is based on the ultimate bending moment. Therefore, it is useful to keep in mind that having $M_{j.Rd} \geq s\, M_{b.Rd}$ does not necessarily mean that $M_{j.Rd} \geq M_{b.Rd}$. However, as previously mentioned, the classification adopted by Eurocode 3 is rigorous only in the pure theoretical case occurring when both the joint and the connected member exhibit a perfectly plastic behaviour.

2. CAPACITY DESIGN OF BEAM-TO-COLUMN JOINTS

The design of beam-to-column joints is traditionally aimed to assure that yielding occurs at the beam ends of the connected beam where the dissipation of the earthquake input energy is expected relying on wide and stable hysteresis loops. To this aim, Eurocode 8 [22] requires that the degree of overstrength required is guaranteed in case of full penetration butt welds or satisfying, in case of other joint typologies, the following relationship:

$$M_{j.Rd} > 1.1 \cdot \gamma_{ov} \cdot M_{b.Rd} \quad (7)$$

where $M_{j.Rd}$ represents the joint design resistance, $M_{b.Rd}$ the plastic moment of the connected beam and γ_{ov} is an overstrength factor accounting for the random variability of the steel yield strength, while the coefficient 1.1 covers the effects of material strain hardening. Eurocode 8 recommends the use of $\gamma_{ov} = 1.25$; conversely, the Italian code [23] suggests a joint overstrength coefficient depending on the steel grade ($\gamma_{ov} = 1.20$ for S235, $\gamma_{ov} = 1.15$ for S275 and $\gamma_{ov} = 1.10$ for S355).

One of the causes of significant and premature joint damage during the seismic events of Northridge and Kobe can be recognised in the use of design criteria not able to assure a sufficient degree of overstrength to allow the full development of the beam plastic rotation capacity. In fact, regarding the overstrength which the beam is able to exhibit, due to strain hardening, it depends on the width-to-thickness ratios of flanges and web. As a consequence, the joint overstrength needed to assure the full-strength requirement is strictly related to the behavioural class of the beam section (i.e. ductile, compact, semi-compact and slender). It means that, decreasing the width-to-thickness ratios of flanges and web, the plastic deformation capacity of the beam increases, but this beneficial effect could be vanished if the beam-to-column joint does not possess the overstrength required by the simultaneous increase of the beam ultimate resistance. In addition, also the influence of random material variability both on the beam flexural resistance and the beam-to-column joint moment resistance has to be properly accounted for.

Only a few studies concerning the influence of random material variability on the behaviour of steel connections are available [24-27]. In particular, it has been proposed [24] to formulate the design requirement for full-strength and full-ductility joints by means of a probabilistic approach calibrated on the basis of the results coming from Monte Carlo simulations [28], including both the random material variability of the plate elements and that of bolt properties [29].

A simplified approach to account for the influence of random material variability is herein proposed by assuming an overstrength factor $\gamma_{ov,rm}$ equal to the ratio between the average value of the yield strength of beam flanges $f_{ym.bf}$ and the nominal yield strength $f_{y.b}$. Conversely, the amount of overstrength due to the development of strain-hardening up to the occurrence of local buckling is taken into account directly considering the width-to-thickness ratios of beam flanges and web. Therefore, the ultimate beam flexural resistance at the plastic hinge location is evaluated as:

$$M_{b.u} = \gamma_{ov.rm} \cdot \gamma_{ov.sh} \cdot \gamma_{M0} \cdot M_{b.p} \quad (8)$$

where:

$$M_{b.p} = \frac{Z_b \cdot f_{y.b}}{\gamma_{M0}} \quad (9)$$

being Z_b the plastic modulus of the beam section, $f_{y.b}$ is the yielding resistance of the beam and γ_{M0} the partial safety factor.

The average yield strength of beam flanges is evaluated accounting for the influence of the flange thickness t_{bf}, so that:

$$\gamma_{ov.rm} = \frac{f_{ym.bf}}{f_{y.b}} = \frac{f_0 - \beta\, t_{bf}}{f_{y.b}} \quad (10)$$

where the parameters f_0 and β depend on the steel grade (Table 1).

The coefficient $\gamma_{ov,sh}$ accounting for the influence of strain hardening is given by $\gamma_{ov.sh} = s$ [17], where s is the parameter given by Eq. (1).

Table 1: Mechanical properties of the material

Steel classes	f_0 [MPa]	β [MPa/mm]	$\frac{E}{E_h}$	$\frac{\varepsilon_h}{\varepsilon_y}$
S 235	313.4	2.254	37.5	12.3
S 275	323.3	0.910	42.8	11.0
S 355	444.2	2.987	48.2	9.8

3. DESIGN PROCEDURE FOR FULL-STRENGTH FULL-DUCTILITY JOINTS

Starting from the average ultimate resistance of the beam provided by Eq. (8), a design procedure aiming to the development of full-strength full-ductility joints is proposed and discussed with reference to extended end-plate joints with four bolts in tension. The end-plate is unstiffened. The design goal is accomplished by properly applying the basic principles of "capacity design" at component level, considering all the joint components defined by Eurocode 3 within the framework of the “component method”.

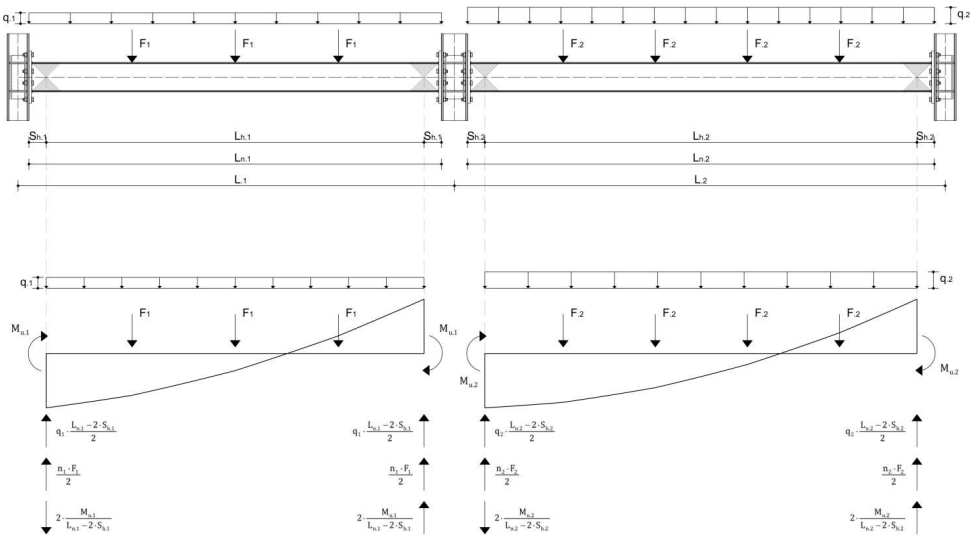


Figure 3 - Reference structural scheme considering seismic actions from left to right

In particular, the proposed procedure starts from the identification of the maximum internal actions which the fully yielded and strain-hardened beam is able to transmit to the joint. The reference structural scheme is depicted in Fig. 3. By denoting with *i* the left end joint and with *j* the right end joint for each beam and considering seismic actions from left to right, the shear action occurring at the plastic hinge locations are given by:

$$V_{bu.i}^{(1)} = \frac{q_1 L_{h1}}{2} + \frac{n_{F1} F_1}{2} - \frac{2 M_{bu.1}}{L_{h1}} \qquad V_{bu.j}^{(1)} = \frac{q_1 L_{h1}}{2} + \frac{n_{F1} F_1}{2} + \frac{2 M_{bu.1}}{L_{h1}}$$
$$V_{bu.i}^{(2)} = \frac{q_2 L_{h2}}{2} + \frac{n_{F2} F_2}{2} - \frac{2 M_{bu.2}}{L_{h2}} \qquad V_{bu.j}^{(2)} = \frac{q_2 L_{h2}}{2} + \frac{n_{F2} F_2}{2} + \frac{2 M_{bu.2}}{L_{h2}} \tag{11}$$

where the vertical loads q are those occurring in the seismic load combination $(G_k + \psi_2 Q_k)$ according to Eurocode 8, L_h is the distance between the two plastic hinges, F are the concentrated forces due to the secondary beams applied symmetrically according to the beam centre and n_F is the number of these forces. The parameter s_h, i.e. the distance between the plastic hinge and the column flange, is taken equal to the beam height.

On the basis of the maximum moment which the beams are able to transmit given by Eq. (8), the bending moment M_{cf} and shear action V_{cf} at the column flange can be evaluated as follows:

a) in the case of external joint i of beam 1:

$$M_{cf.i}^{(1)} = M_{bu.1} - V_{bu.i}^{(1)} \cdot s_{h1} - \frac{q_1 s_{h1}^2}{2} \qquad V_{cf.i}^{(1)} = V_{bu.i}^{(1)} + q_1 s_{h1} \tag{12}$$

b) in the case of internal joint *j* of beam 1:

$$M_{cf.j}^{(1)} = M_{bu.1} + V_{bu.i}^{(1)} \cdot s_{h1} + \frac{q_1 \, s_{h1}^2}{2} \qquad V_{cf.j}^{(1)} = V_{bu.j}^{(1)} + q_1 s_{h1} \tag{13}$$

c) in the case of internal joint i of beam 2:

$$M_{cf.i}^{(2)} = M_{bu.2} - V_{bu.i}^{(2)} \cdot s_{h2} - \frac{q_2 \, s_{h2}^2}{2} \qquad V_{cf.i}^{(2)} = V_{bu.i}^{(2)} + q_2 s_{h2} \tag{14}$$

d) in the case of external joint j of beam 2:

$$M_{cf.j}^{(2)} = M_{bu.2} + V_{bu.i}^{(2)} \cdot s_{h2} + \frac{q_2 \, s_{h2}^2}{2} \qquad V_{cf.j}^{(2)} = V_{bu.j}^{(2)} + q_2 s_{h2} \tag{15}$$

Obviously, the analysis is repeated for the case of seismic actions from left to right and the most severe internal actions are considered.

Regarding the design of column web panel stiffeners, they have to be designed considering the maximum shear action occurring when the beams are in the ultimate conditions and the shear action acting in the columns V_c.

e) In the case of panel zone of external joint i of first bay:

$$V_{wp.Ed} = \frac{M_{cf.i}^{(1)}}{d_{b1} - t_{bf.1}} - \frac{V_{c1} + V_{c2}}{2} \tag{16}$$

f) In the case of panel zone of internal joint:

$$V_{wp.Ed} = \frac{M_{cf.j}^{(1)}}{d_{b1} - t_{bf.1}} + \frac{M_{cf.i}^{(2)}}{d_{b2} - t_{bf.2}} - \frac{V_{c1} + V_{c2}}{2} \tag{17}$$

g) For the external panel zone j of the beam 2:

$$V_{wp.Ed} = \frac{M_{cf.j}^{(2)}}{d_{b2} - t_{bf.2}} - \frac{V_{c1} + V_{c2}}{2} \tag{18}$$

In the case of extended end plate connections with four bolts in tension, according to Eurocode 3, a simplified model can be adopted to design the tension zone by means on an equivalent T-stub whose lever arm is equal to d_b-t_{bf} (Fig. 4). However, according to Eurocode 3, when the this model is adopted as a conservative simplification, also a ductility criterion has to be adopted. Such criterion requires that the total design resistance F_{Rd} shall not exceed 3.8 $F_{t,Rd}$, where $F_{t,Rd}$ is the design tension resistance of a bolt. Notwithstanding this criterion has been herein neglected, because sufficient bolt overstrength is already guaranteed by means of the use of hierarchy criteria. In fact, Eurocode 3 requirement concerning the use of the simplified T-stub model corresponds to a bolt overstrength of about 5% (i.e. 4/3.8=1.05). This bolt overstrength is surely guaranteed when hierarchy criteria foir seismic design are applied because the overstrength coefficient $\gamma_{ov.sh}$ is always > 1.05.

According to this model, the design of all the joint components has to guarantee the transmission, at the beam flanges' levels, of a compression force C_u and of a tensile force T_u given by:

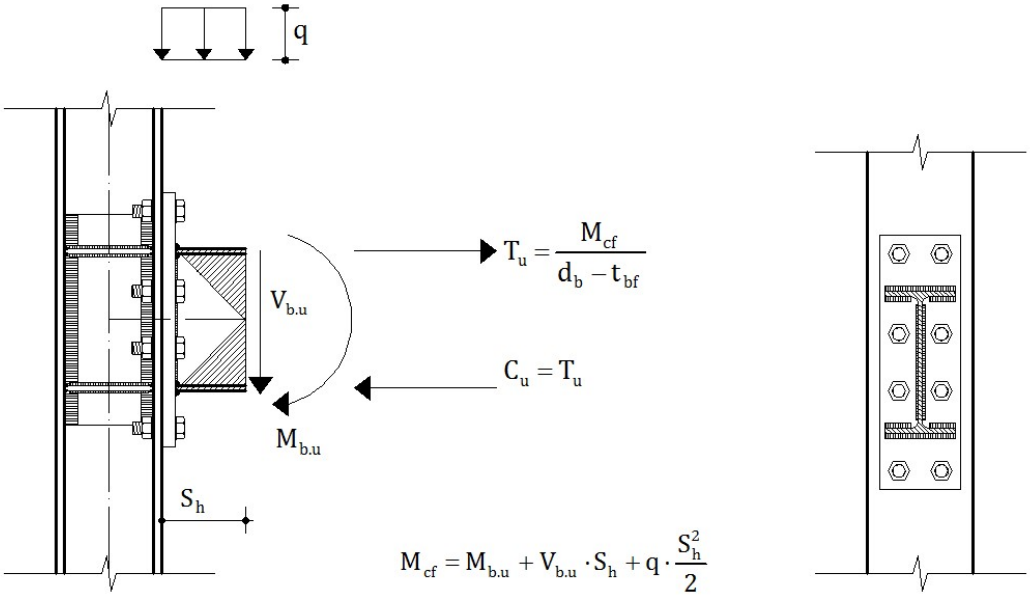


Figure 4: Simplified model for the design of the tension zone

$$T_u = C_u = \frac{M_{cf}}{d_b - t_{bf}} \tag{19}$$

Starting from the knowledge of T_u and C_u values occurring when the beam plastic hinges have attained their ultimate flexural resistance, all the geometrical details of the connecting elements can be designed by means of the resistance formulations provided by Eurocode 3 for the joint components. To this aim, a specific sequence of design operations or resistance checks of the joint components has to be followed:

Step 1: Evaluation, by means of Eq. (8), of the average (because of random material variability) ultimate moment $M_{b,u}$ which the fully yielded and strain-hardened beam is able to transmit.

Step 2: Calculation of bending moment M_{cf} and shear action V_{cf} at the column flange and evaluation of compression force C_u and tensile force T_u to be transmitted at the beam flanges' levels.

Step 3: Design of the bolt diameter accounting for the combined action of shear and tension.

Step 4: Design of throat thickness of welds connecting the end-plate to the beam flange and design of throat thickness of welds connecting the end-plate to the beam web assuming

that they have to transmit a bending moment proportional to the plastic modulus of the beam web alone.

Step 5: Design of the end-plate thickness by modelling the tension zone by means of an equivalent T-stub and assuming that the distance m between the bolt axis and the yield line located close to the beam flange is equal to the minimum allowed by the code, $m=1.2\ d_0$ with d_0 equal to the diameter of the hole; the width of the end-plate can be defined considering code requirements concerning the bolt spacing and the edge distances.

Step 6: Check of the resistance of the column web in shear and design of supplementary web plates if needed. Eurocode 3 introduces a limitation about the thickness of the supplementary plates. In particular, the shear area A_{vc} may be increased no more than $b_s t_{wc}$. If a further supplementary web plate is added on the other side of the web, no further increase of the shear area is allowed. The proposed method does not take into account such limitation.

Step 7: Check of the resistance of the column web in tension and in compression; if needed continuity plates are added and/or supplementary web plates are extended to cover also tension and compression zones.

Step 8: Check of the resistance of the column flange in bending; if not satisfied, backing plates can be adopted to increase the resistance of the equivalent T-stub modelling the column flange in bending. It should be underlined that the adoption of the backing plates is useful when type 1 mechanism occurs, otherwise (type 2 mechanism) the increase of the bolt diameter is needed and the procedure has to be repeated starting from step 3.

4. DESIGN EXAMPLES AND FINITE ELEMENT MODELLING

Reference is made to the layout depicted in Fig. 5.

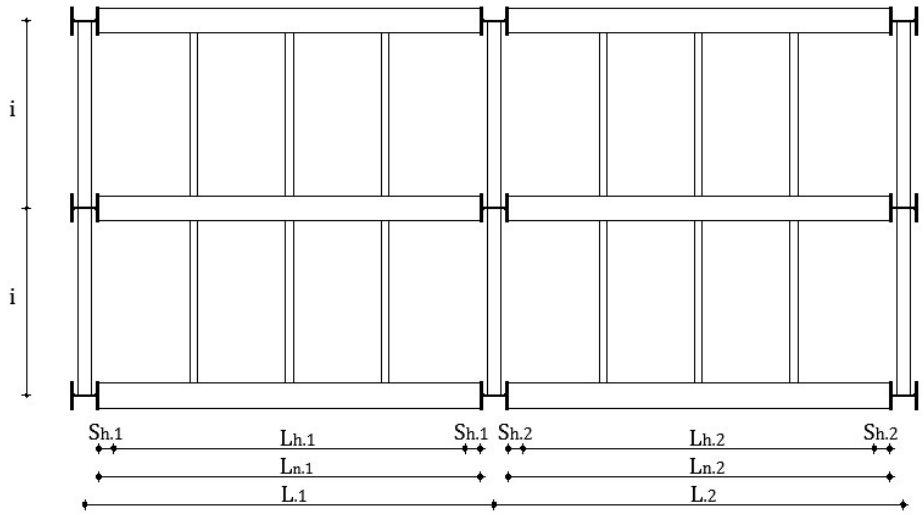


Figure 5: Building plan layout of study cases

Three different solutions have been designed with reference to the external joint of the longitudinal inner frame by varying the geometry of the structure. In Table 2, the input data for the three cases analysed are summarized.

Table 2 – Input data of case studies

Study case	Beam section	Column section	Beam steel grade	Column steel grade	Plates steel grade	Bolt class	q [kN/m]	F [kN]	n_F	L_n [mm]
A	IPE 600	HEM 320	S235	S355	S275	10.9	1.25	65	3	8641
B	IPE 450	HEM 260					1.00	45	3	6232
C	IPE 220	HEM 200					0.75	30	3	3800

For the sake of simplicity the detailed calculations are not reported here and the main results obtained both with the proposed and with the EC8 design procedures are directly summarized in Table 3. Nevertheless, in order to clarify the proposed design method one of the cases, namely the case A, is described in detail in a worked example reported in Annex. From Table 3 it can be observed that the most important difference occurs for the case study C, i.e. in the case with the smallest beam size, where the design bending moment according to the proposed design procedure is about the 36.4% greater than the one required according to Eurocode 8. This difference reduces to the 26.5% for study case B and to 23.6% for study case A. It is worth to observe that this difference, in two cases out of three, exceeds the value of the partial safety

283 factor γ_{M2}=1.25 suggested by EC8 for the design of bolts and welds and, therefore, it is expected
284 that, in these cases, the EC8 design procedure may fail.

Table 3 – Design solutions for analysed study cases

Study case	Design procedure	$\gamma_{ov,rm}$	$\gamma_{ov,sh}$	M_{cf} [kNm]	V_{cf} [kN]	T_u [kN]	d_b [mm]	a_f [mm]	a_w [mm]	t_{ep} [mm]	t_s [mm]	t_{cp} [mm]	a_{cp} [mm]	$\frac{F_{Rd.1}}{T_u}$	$\frac{F_{Rd.2}}{T_u}$
A	Eurocode 8	1.25	1.10	1081	353	1860	33	23	10	45	-	20	8	3.56	1.39
	Proposal	1.15	1.28	1336	405	2299	36	29	10	55	5	20	8	2.79	1.18
B	Eurocode 8	1.25	1.10	524	239	1203	27	18	8	35	-	15	6	3.73	1.41
	Proposal	1.24	1.26	663	278	1524	30	22	9	45	5	15	6	2.94	1.22
C	Eurocode 8	1.25	1.10	88	93	417	16	11	5	20	-	10	4	2.29	1.08
	Proposal	1.25	1.30	120	107	571	20	14	6	25	5	10	4	1.67	1.03

285
286 In order to evaluate the accuracy of the examined design criteria, numerical simulations by
287 means of advanced finite element models have been performed using ABAQUS 6.13 software.
288 Since the behaviour of the analysed connections is strongly affected by in-plane and out-of-
289 plane deformations, by contacts between the connecting elements and the profiles of column
290 and beam and by geometrical and material non linearities, the finite element model has been
291 developed adopting a three-dimensional approach based on the following steps: geometrical
292 characterization of the components, definition of material properties, definition of the
293 interactions between the elements, definition of the boundary conditions and choice of the
294 elements and size of the mesh, calibration and application of a proper initial imperfection
295 model.
296 The simulation has been performed considering the scheme depicted in Fig. 6, restrained
297 with a hinge at the bottom end of the column and a roller at the top end of the column,
298 preventing horizontal displacements of the top column end. The beam end is loaded with a
299 vertical force inducing in the joint a combination of shear and bending consistent with the
300 actions arising in the reference scheme under seismic loads.
301 To this aim, the length of the beam has been assumed equal to M_{cf}/V_{cf}, thus assuring that
302 when the design bending resistance is attained also the corresponding shear action is achieved.
303 The length of the column has been assumed equal to 3500 mm, i.e. the interstorey height of the
304 sample building.

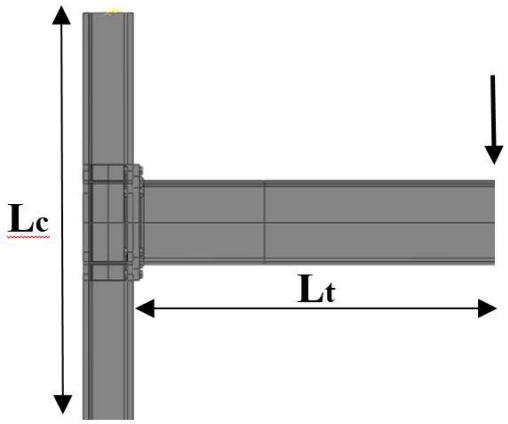


Figure 6: Analysed structural scheme

Regarding the *geometrical definition* of the components, the model is made up of seven repetitive elements: the column, the beam, the end plate, the bolts, the continuity plates and the additional supplementary web plates (Fig. 7).

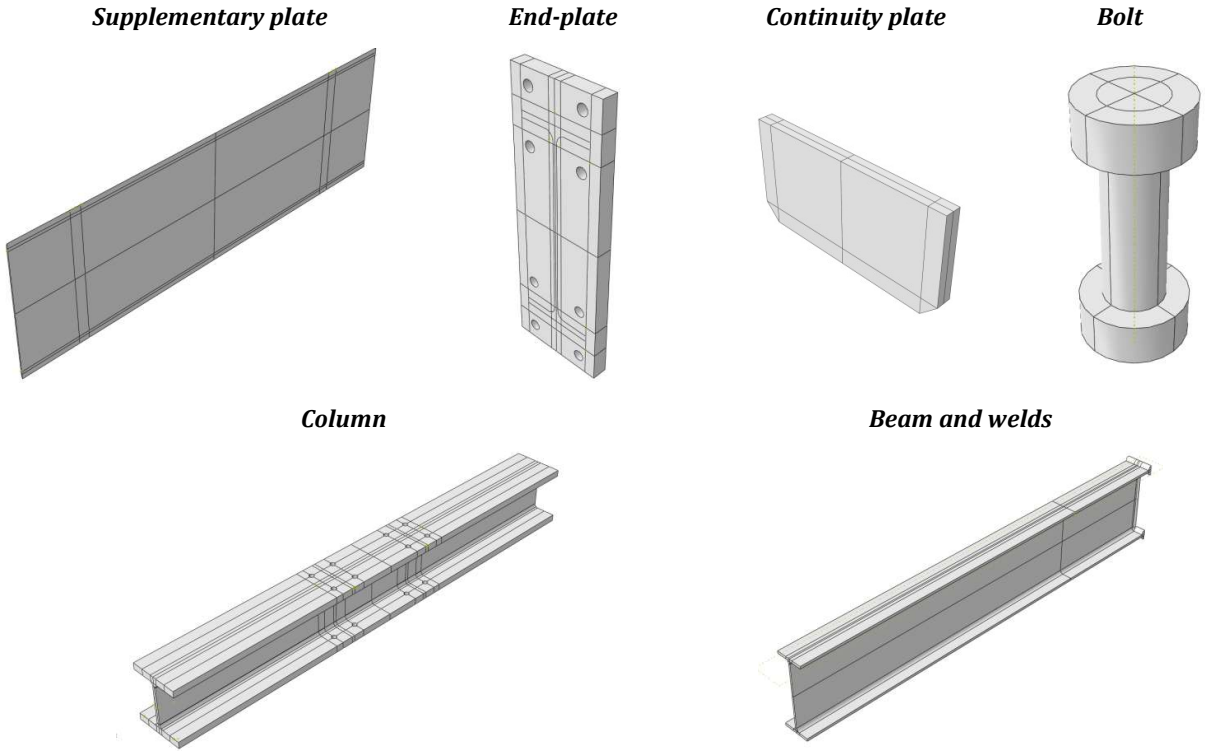
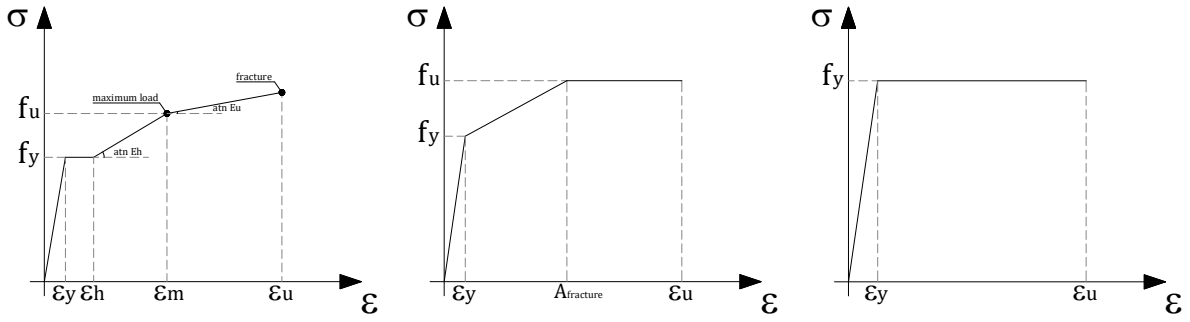


Figure 7: Components of the finite element model

The *material properties* of the plate elements and of the profiles have been described by means of an elastic-plastic isotropic model by adopting a quadrilinear true stress-true strain law (Fig. 8a). The parameters for different constructional steels are given in Table 2.

The behaviour of the material of the bolts has been modelled using a simplified tri-linear model (Fig. 8b) based on the yield and ultimate nominal strength according to the bolt class. The strain corresponding to the ultimate resistance and the ultimate strain have been evaluated by means of the following relationships: $\varepsilon_m = A_r$ [%] and $\varepsilon_u = \ln(1/1\text{-}Z)$, where A_r is elongation at fracture and Z is the necking ratio given by the ratio between the original cross-sectional area and the minimum cross-sectional area after fracture. The values provided by the manufacturer [30] have been adopted.



a) Plates and profiles b) Bolts c) Welds

Figure 8: Material constitutive laws

The welds have modelled by means of a bilinear elastic-perfectly plastic law (Fig. 8c) with yield strength and ultimate strain defined according to [31].

Regarding the value of the yield strength of all the components, consistently with the design approach which accounts the influence of random material variability considering the average yield strength of the beam flanges, the average value of the yield strength has been adopted for the beam component, while for all others components the nominal characteristic value has been assumed. This assumption is consistent with the proposed design procedure and with ANSI/AISC 358-10 recommendations for prequalified connections for seismic applications [32].

All the *interactions* between the different elements have been defined using the surface-to-surface contact formulation with finite sliding. In particular, the following interactions have been defined (Fig. 9): between the end-plate and the column flange, between the bolt head and the end-plate, between the bolt shank and the plate hole, between the bolt shank and the column flange hole, between the end-plate and the beam end. In the normal direction a "hard contact" has been used, while in the tangential direction a friction coefficient equal to 0.20 has been adopted.

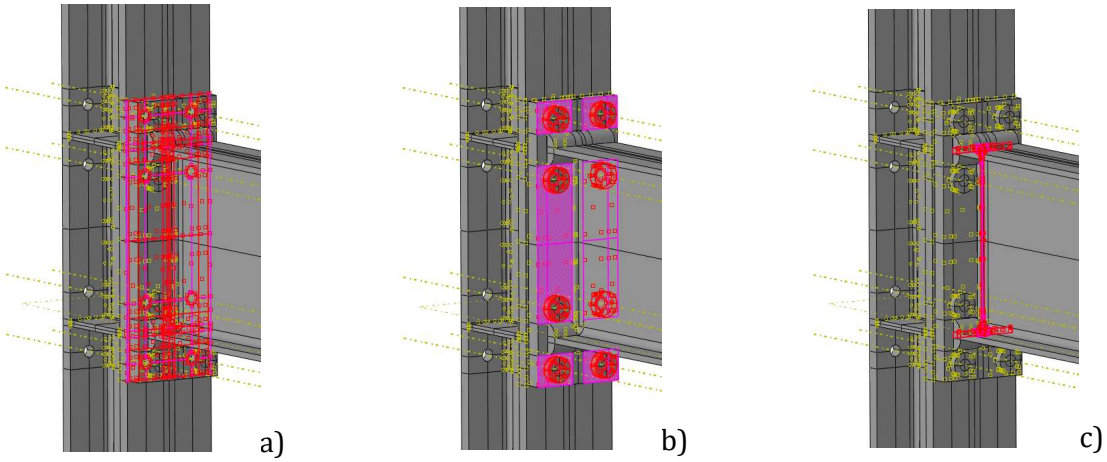


Figure 9: Definition of the contacts: a) end-plate/column flange; b) bolt head/end-plate; c) end-plate/beam end.

337

338 Where there was the need to link the rigid kinematic mechanisms of the section to those of
339 a point externally restrained, the constraints have been modelled by introducing at the end of
340 the section of the members a central node and "coupling internal constraints" (Fig. 10a).
341 In order to simulate the application of an axial force, at the top of the column an external
342 pressure equal to 30% of the yield strength of the material has been applied (Fig. 10b).
343 Regarding the *finite element type*, in order to reduce the computational efforts, eight-node
344 brick elements with reduced integration and first order approximation (C3D8R) have been
345 adopted. The end part of the beam close to the column where local buckling phenomena are
346 expected, for a length equal to 2.5 times the beam height, has been modelled with non linear
347 eight-node brick elements with full integration (C3D8). Such elements, as also reported in [32],
348 are particularly accurate for analysis where buckling effects are significant.

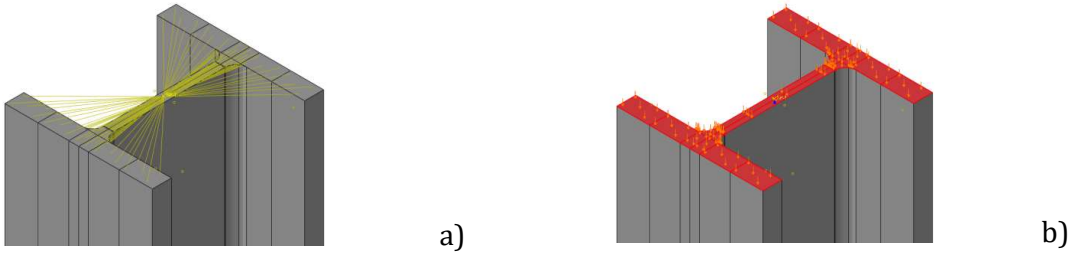


Figure 10: a) Coupling internal constrains; b) application of external pressure

349

350 Preliminarily, a sensitivity analysis has been performed in order to determine the *mesh*
351 *dimension.* The parameters that could influence the results are the number of the elements in
352 the thickness of the plates, the dimension of the mesh of the bolts and the dimension of the
353 elements where the local buckling is expected. In order to obtain accurate results, the following

354 "meshing" procedure has been applied: where local buckling is expected the maximum
355 dimension of the elements has been taken as 20 mm, the plates with elements whose dimension
356 is at least 30 mm and with 2 elements in the thickness, the bolts have been divided using
357 elements with minimum dimension equal to 6 mm with a deviation factor equal to 0.1.
358 In addition, *geometrical imperfections* have been introduced according to the requirements of
359 EN10034 [35], by using a distorted shape of the joint similar to the 1st buckling mode
360 preliminarily evaluated by means of an elastic buckling analysis. The calibration of the
361 distortion of the model has been performed based on the maximum value of the angular
362 distortion of the flanges of steel profiles given by EN 1090-2 [36]. The model finalized to the
363 execution of the "linear buckling analysis" is depicted in Fig. 11.

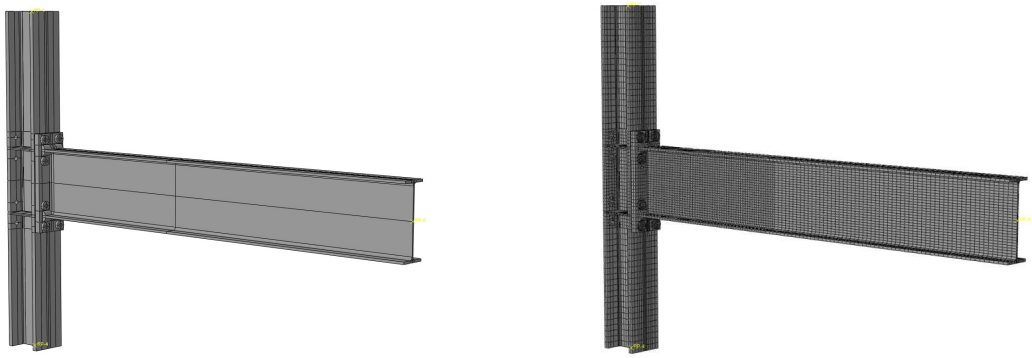


Figure 11: *ABAQUS Model*

364

365 The results of the buckling analyses are reported in Fig. 12 representing the first four
366 buckling modes. In particular, because of the application of the load downwards, the first and
367 the third buckling mode involve the combined buckling of the bottom flange and the
368 compressed part of the web. Similarly, the second and the fourth mode provide the buckling of
369 the upper flange of the profile when the load is applied upward. Therefore, as the analyses
370 developed in this paper are referred to monotonic downward loading conditions, an
371 imperfection pattern proportional to first buckling mode has been introduced.
372 Following this approach, the proportionality coefficient $k_{1°mode}$ for scaling the "buckling
373 eigenmode" has been determined as the ratio between the 80% [33] of the maximum
374 manufacturing tolerance (equal to 2% of the width of the flange [34]) and the sum of the beam
375 flange tip displacements δ_f in the buckled configuration:

$$k_{1°mode} = \frac{0.8 \times 0.02 \times b_f}{2 \times \delta_f} \tag{20}$$

According to the design criteria adopted, it is expected that plastic deformations are mainly located at the beam end. Conversely, the connection components are expected to be subjected to very limited yielding. In fact, it should be noted that, even if the joints have been designed to attain full strength, limited yielding of joint components has to be expected because the formulas used for design, as suggested by Eurocode 3, are based on the definition of design plastic resistances rather than on elastic design resistances.

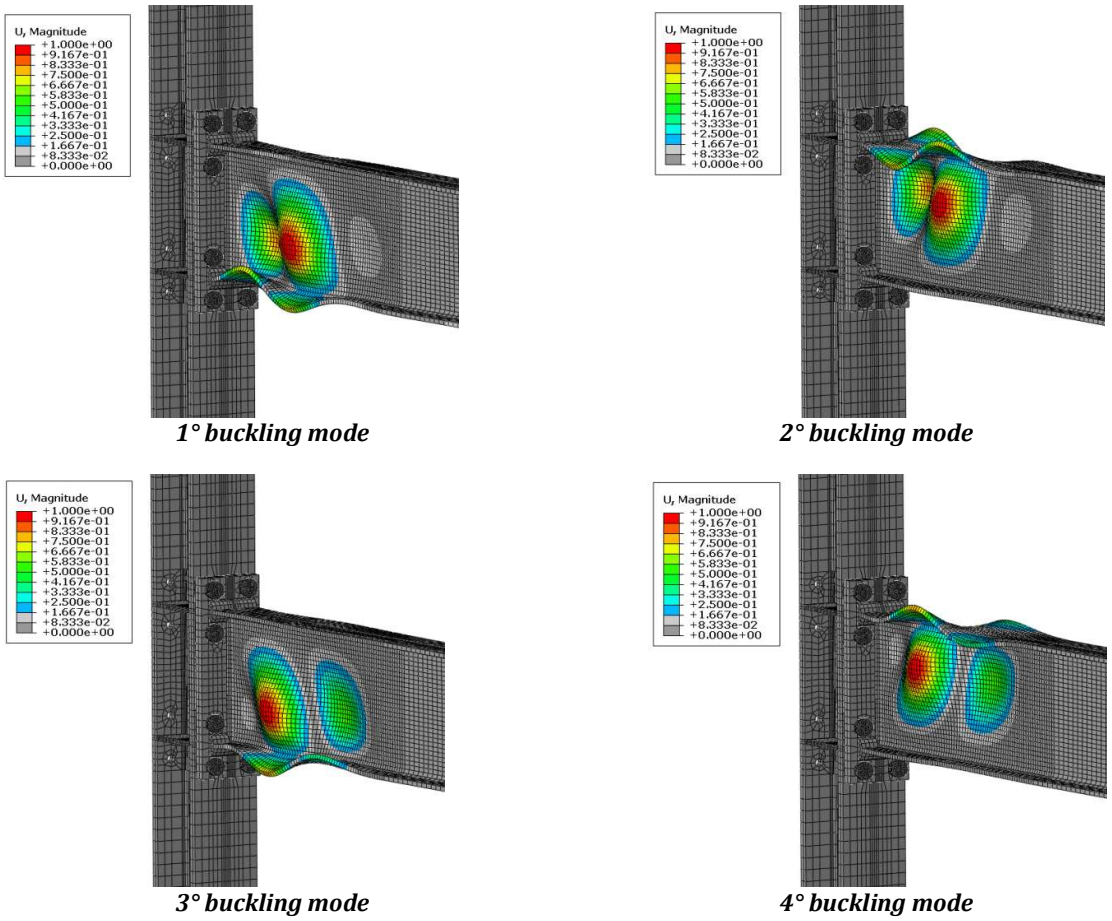
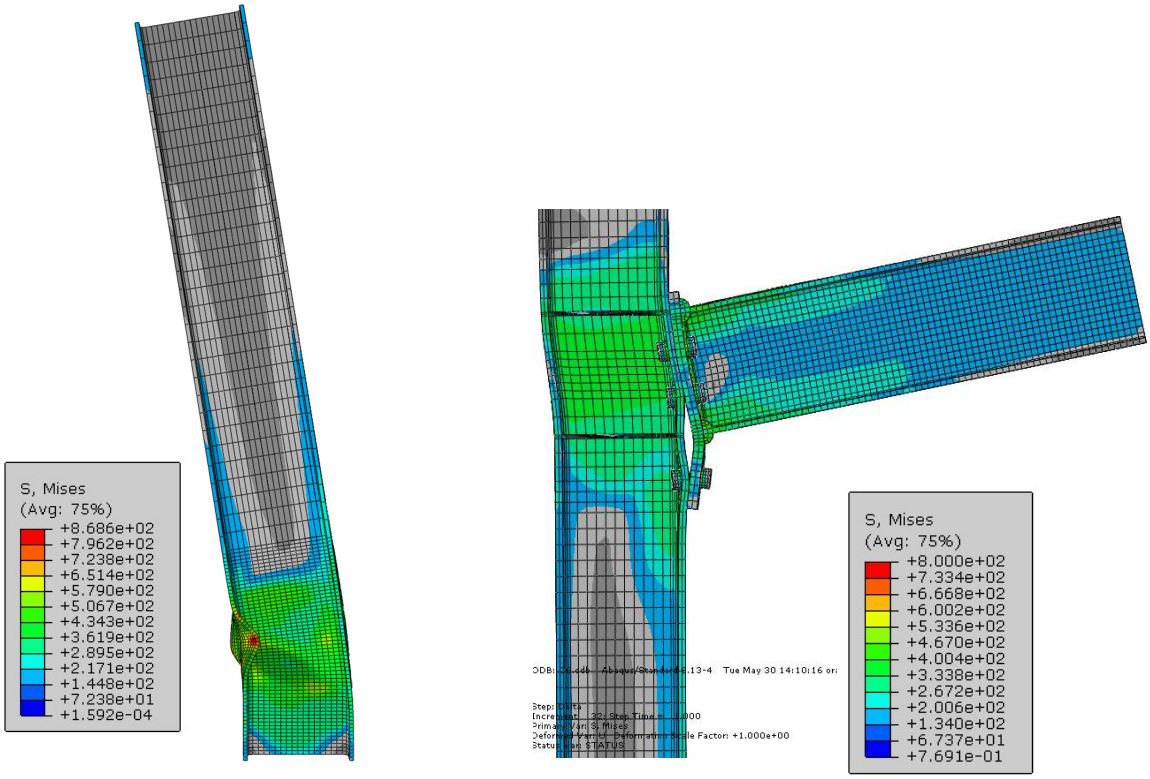


Figure 12: buckling modes of beam-to-column joints

The finite element model has been validated through comparison with some experimental tests collected from technical literature. Specimens from two different experimental campaigns have been considered: a set of tests on full-strength connections described in [37] and a group of tests on partial-strength extended end-plate connections reported in [38]. The first group of data regards cases where the plastic engagement involves the beam end only and it is used to check the model accuracy when the moment-rotation response of the beam-joint system is governed by the local buckling of the beam flange in compression. Conversely, the second group

of data regards partial strength connections in which several joint components undergo plastic deformations. These data are used to check whether or not the FE model is able to reproduce accurately the response of the joint.



a) Simulation of the test with IPE 300 [37]

b) Simulation of test C6 [38]

Figure 13: Examples of the developed validation FE models

In particular, among the first group of specimens [37], two cases have been selected, one regarding a connection fastening an IPE 300 beam, another regarding an HEB 240 profile, both made of S275 steel. These tests are non-uniform bending tests carried out under monotonic loading conditions. Conversely, among the second group of specimens [38] four specimens concerning external beam-to-column joints have been selected, namely those identified in the original manuscript [38] as C5, C6, C7 and C8. They concern extended end-plate joints fastening rather shallow beams (UB 25.4) to columns with a relatively thin flange (UC 46.2). In these tests, the connection and the members are made of an Australian steel grade with nominal yield stress equal to 250 MPa and M20 or M16 bolts made up of a steel grade equivalent to the European 8.8 class. The connections are realized with a typology similar to that considered in this paper, namely with extended end-plate and two internal bolt rows. Case by case, the plates have thickness equal to 16 or 20 mm. The results of the validation study are summarized in

Figs.13-15 and they evidence the good agreement between FE models and tests in all the considered cases. As it is possible to verify easily from the moment-rotation diagrams, the model demonstrates to be able to follow with high accuracy the experimental results during the whole loading process, simulating accurately both the initial strain-hardening and the softening branch arising after the development of local buckling phenomena.

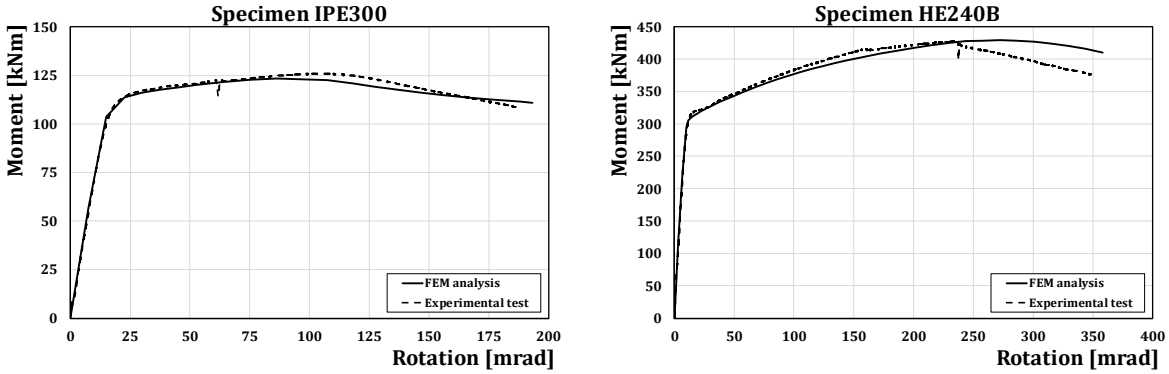


Figure 14: Comparison between experimental results [37] and finite element simulation

In a similar way, the FE model proves to be able to simulate accurately the moment-rotation response of extended end-plate connections (cases C5-C8) when the joint components are engaged in plastic range (Fig.15).

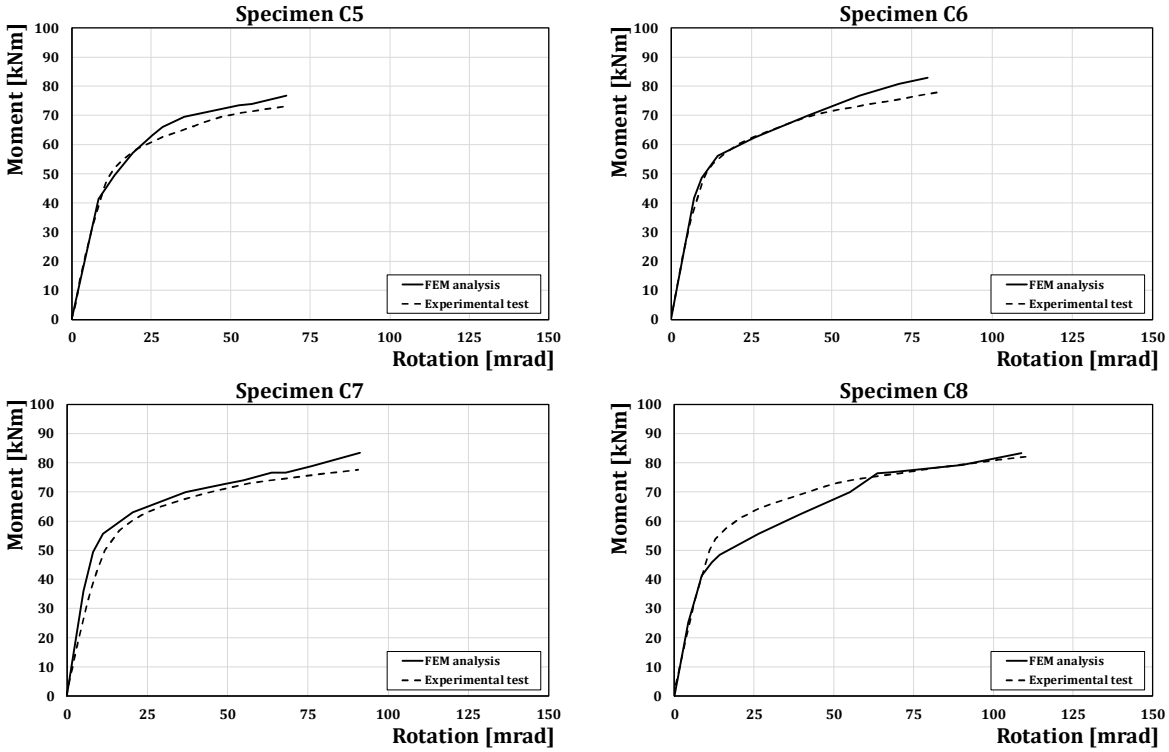


Figure 15: Comparison between experimental results [38] and finite element simulation

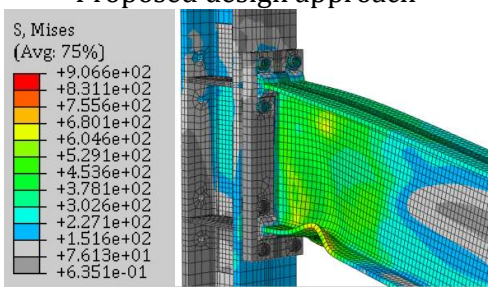
In particular, in the four considered cases the components mainly engaged in plastic range are the end-plate and the column flange in bending, the shear panel and the bolts in tension and shear. Some minor yielding of the beam end also occurs in all the considered cases.

5. RESULTS OF FINITE ELEMENT SIMULATIONS

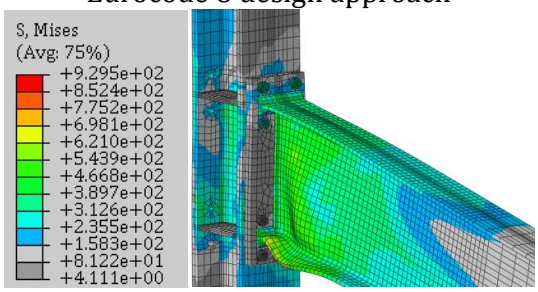
From the overall point of view, as expected, FE analyses showed in almost all the cases the concentration of the plastic deformations at the beam end where the plastic hinge, characterized by the development of plastic local buckling of the compressed beam flange and out-of-plane buckling of the web, occurred (Fig. 16).

STUDY CASE A: IPE 600 BEAM – HEM 320 COLUMN

Proposed design approach

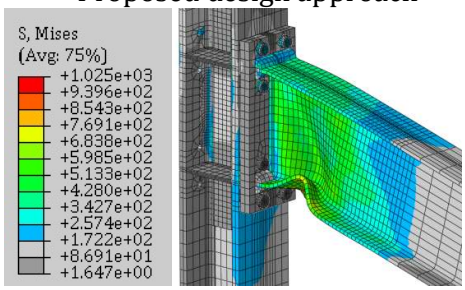


Eurocode 8 design approach

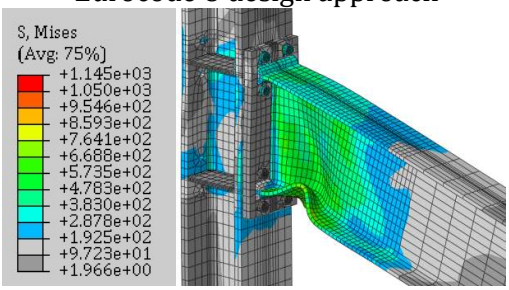


STUDY CASE B: IPE 450 BEAM – HEM 260 COLUMN

Proposed design approach

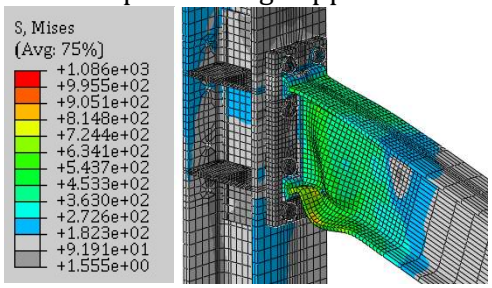


Eurocode 8 design approach



STUDY CASE B: IPE 220 BEAM – HEM 200 COLUMN

Proposed design approach



Eurocode 8 design approach

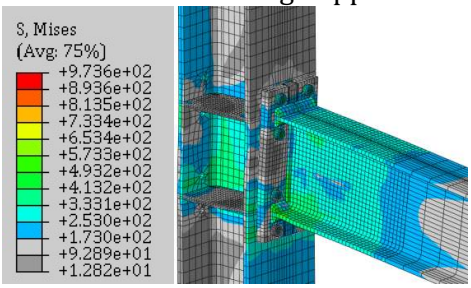


Figure 16: Ultimate behaviour of the analysed joints (Von Mises stresses)

423 The moment-rotation curves of the beam-joint system for all the analysed cases are
424 represented in Figs. 17-19. The curves have been obtained, in all cases, by multiplying the force
425 applied at the end of the cantilever for L_t (Fig.6) and dividing the displacement evaluated in the
426 same point for the same length.

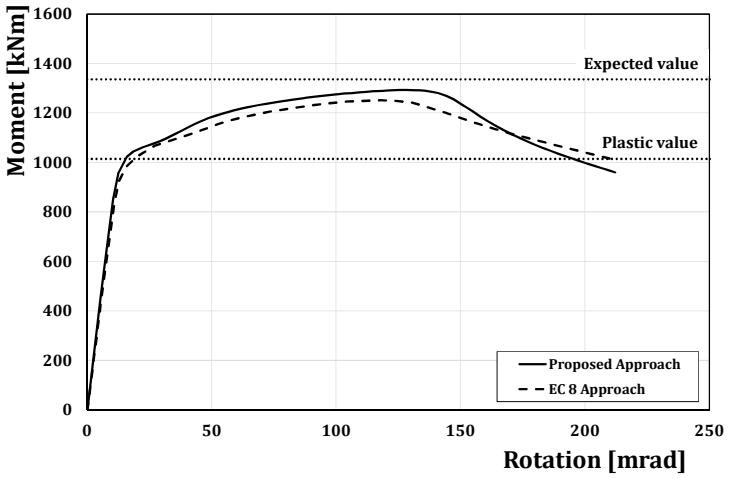


Figure 17: *Moment-rotation curves (case A)*

427
428 The curves reported in in Fig.17 are referred to study case A. As expected, after the initial
429 linear behaviour, at the attainment of the plastic resistance of the section, they provide a non-
430 linear response characterized by the increase of the bending moment, due to the material
431 strain-hardening which continues up to the attainment of the local buckling of the beam
432 compressed flange. The achievement of the maximum bending moment corresponds to the
433 complete development of local buckling. The following softening branch is due to the post-
434 buckling behaviour. In the two cases (EC8 and proposed procedure), as it is possible to check
435 easily from the figure, the values are very similar and, in particular, equal to 1293 kNm
436 (proposed approach) and 1251 kNm (EC8). These values are a little bit lower than the design
437 value, evaluated at the column flange level, equal to 1336 kNm. Such scatter, with respect to the
438 design value, is very low being less than 7%. It is mainly related to the accuracy of the empirical
439 relationship (1) for evaluating $\gamma_{ov.sh} = s$.
440 With reference to the case study B, the shape of the moment-rotation curve is analogous to
441 the previous case and the values of the moment corresponding to the complete development of
442 local buckling of the beam flange are equal to 662 kNm and 666 kNm in case of the joint
443 designed according to the proposed approach and to Eurocode 8, respectively. These values of
444 the bending moment are practically equal to the design value (663 kNm) confirming the
445 accuracy of the formulations used to predict the overstrength factor *s* (Fig. 18).

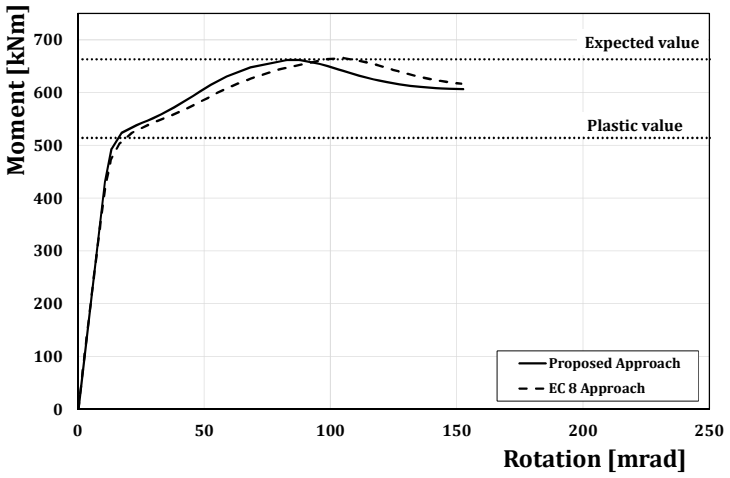


Figure 18: *Moment-rotation curves (case B)*

446

447 Similarly, in the study case C (Fig. 19) the local buckling is achieved in correspondence of a
448 bending moment equal to 115 kNm and 113 kNm, respectively, in case of the joint designed
449 according to the proposed approach and to Eurocode 8. These values are slightly lower (about
450 7% and 8%, respectively) than the design value (equal to 123 kNm). In this case, regarding the
451 ultimate rotation, it is useful to underline that the joint designed according to Eurocode 8,
452 because of an abrupt drop of the flexural resistance, shows a rotation supply (131 mrad) less
453 than the one of the joint designed according to the capacity design criteria proposed.

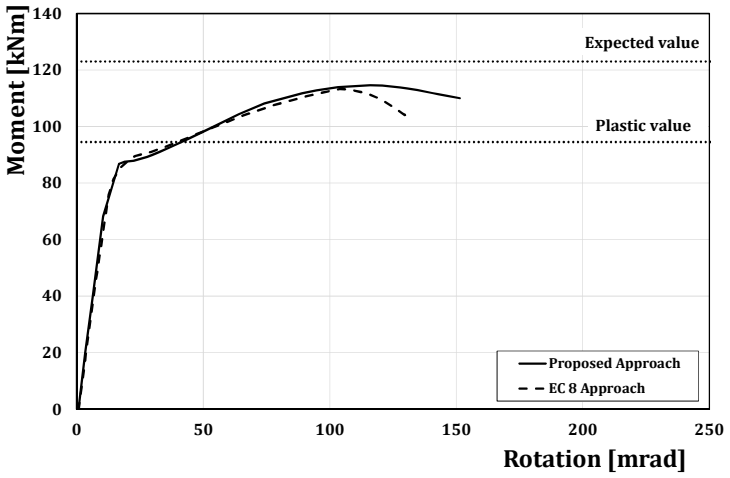


Figure 19: *Moment-rotation curves (case C)*

454

455 This difference is due to the brittle failure of the bolts connecting the end-plate to the column
456 flange.

457 As a conclusion, the comparison between the moment-rotation curves of the joints designed
458 according to the capacity design criteria herein proposed with those designed according to the

459 EC8 provisions, shows that the differences can be very important, especially concerning the
460 failure mode, in those cases where EC8 underestimation of the amount of strain-hardening
461 leads to the bolt failure (case C), undermining significantly the rotation capacity of the beam-
462 joint system. Even though such brittle behaviour is out of the design philosophy of EC8, it has
463 been shown that it can actually occur because of the actual overstrength due to the strain
464 hardening of the material of the connected member, which can be significantly underestimated
465 by the 1.10 factor adopted by EC8 (as reported in the 4th column of Table 3). It is important to
466 underline that such brittle failure of the bolts may happen even if they are usually oversized
467 adopting a partial safety factor equal to 1.25, according to EC3.
468 In any case, due to the fact that the EC8 procedure for connections is not completely
469 rationally addressed, aside from the possible activation of undesired failure modes, in many
470 cases, it also happens that the plastic engagement of the joint components is significant. This
471 means that, in case of severe seismic events, the repair of many parts of the connection has to
472 be accounted for. Conversely, the beam overstrength is more rationally considered by the use
473 of the capacity design criteria proposed, so that all the joint components are sized for actions
474 assuring a negligible plastic engagement.
475 In order to quantify the damage of the joints' components, a damage parameter defined as
476 the ratio between the equivalent plastic deformation (PEEQ) evaluated at the achievement of
477 the rotational capacity of the beam-joint system and the elastic deformation has been
478 considered. To this scope, all the components have been isolated and the corresponding
479 deformation maps have been tracked in order to determine the value of the equivalent plastic
480 deformation summarized, as summarized in the Tab. 4.
481 With reference to the case study A (IPE 600 beam and HEM 320 column), the results
482 provided in Tab. 4 point out that the level of yielding occurring in the connection components
483 is very limited when the beam-to-column joint is designed according to the criteria herein
484 proposed, achieving a maximum value of 5.57 in the welds; conversely, the use of Eurocode 8
485 design criteria leads to a normalised PEEQ equal to 63.44 in the bolts and 38.46 in the welds.
486 Even in the study case B (IPE 450 beam and HEM 260 column), the maximum normalized
487 PEEQ occurs in the welds and is equal to 5.53 for the design procedure proposed while, in case
488 of Eurocode 8, yielding leads to maximum values of normalised PEEQs equal to 23.82 and 21.49
489 in the bolts and in the welds, respectively.
490

Table 4 - Damage to joint components expressed as PEEQ and NPEEQ (NPEEQ=PEEQ/ε_y)

	STUDY CASE A				STUDY CASE B				STUDY CASE C			
	PROPOSED		EUROCODE 8		PROPOSED		EUROCODE 8		PROPOSED		EUROCODE 8	
	PEEQ	NPEEQ	PEEQ	NPEEQ	PEEQ	NPEEQ	PEEQ	NPEEQ	PEEQ	NPEEQ	PEEQ	NPEEQ
Beam	0.3577	319.65	0.3577	319.65	0.8000	714.89	0.9600	857.87	0.9665	863.68	0.1902	169.97
Welds	0.0101	***5.57***	0.0696	**38.46**	0.0100	***5.53***	0.0431	***23.82***	0.0756	***41.78***	0.0540	**29.84**
End-Plate	0.0016	1.25	0.0205	15.68	0.0070	5.35	0.0180	13.75	0.0016	1.22	0.0231	17.64
Column	0.0062	3.65	0.0260	15.35	0.0061	3.61	0.0270	15.97	0.0041	2.43	0.0370	**21.89**
Supplementary plate	0.0007	0.43	-	-	0.0010	0.59	-	-	0.0016	0.95	-	-
Bolts	0.0177	4.12	0.2719	***63.44***	0.0116	2.71	0.0921	**21.49**	0.0012	0.28	0.5053	***117.90***
Continuity plate	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0037	2.83

Finally, in study case C (IPE 220 beam and HEM 200 column), the maximum normalized PEEQs in the joint components occurs in the welds and is equal to 41.78 in the case of the design procedure herein proposed while, in case of Eurocode 8, it occurs in the bolts in tension and is equal to 117.90. Also in this case, damage is highly concentrated at the end of the connected beam even though the bolts achieve their ultimate capacity in case of the joint designed according to Eurocode 8.

It is important to point out that, even though these PEEQ values have been derived with reference to monotonic loading conditions, they give an important information about the strain concentrations occurring in the joint components. These concentrations are of paramount importance as soon as cyclic loading conditions are considered revealing the probable failure mode.

6. CONCLUSIONS

In this paper, rigorous capacity design criteria have been suggested to assure the development of full-strength full-ductility behaviour of beam-to-column joints. The criteria suggested have been applied and validated by means of FE simulations with reference to unstiffened extended end-plate connections. In particular, three different external beam-to-column joints have been designed, according to both the criteria proposed and to Eurocode 8, and their performances have been compared. The validation of the design criteria has been made by means of three-dimensional finite element analyses, carried out by ABAQUS 6.13 software.

The results obtained, on one hand, have confirmed the accuracy of the design approach and, on the other hand, have pointed out some criticisms of EC8 design criteria. In fact, EC8 provisions do not rationally account for the overstrength due to the beam strain-hardening. In particular, in some cases, the underestimation of the overstrength due to strain hardening is

not compensated by the partial safety factor commonly applied in bolt design, thus leading to the brittle failure of the bolts. For the same reason, some joint components are significantly engaged in plastic range when EC8 design criteria are applied so that the resulting behaviour is characterized by a significant sharing of yielding between the connected beam end and such joint components.

The effectiveness of the design criteria herein proposed has been demonstrated comparing the damage level of the joints' components. The results obtained shows that, in case of connections designed according to the criteria proposed, the damage is conspicuously concentrated at the end of the beam which constitutes the main dissipative zone while all the connection's elements practically remain elastic, or only with very limited yielding. Conversely, in case of joints designed according to Eurocode 8, the joint components are significantly engaged in plastic range achieving high strain levels, certainly beyond the yield limit. The developed analyses have demonstrated that following EC8 design procedure, the welds may be engaged in plastic range with deformations up to 38.46 times the yield strain and, in a similar way, the bolts may fail or, in general, undergo severe damages.

ANNEX A: WORKED DESIGN EXAMPLE

Seismic design of beam–to–column joints needs the knowledge of the gravity loads acting on the beams in the seismic load combination, the beam and column sections and the material properties. The design is aimed to the evaluation of the required bolt diameter, throat thickness of fillet welds, end-plate thickness, continuity plate thickness and, if needed, thickness of supplementary web plates. Many relationships are needed to develop all the design steps. Therefore, in order to clarify the proposed procedure, a worked design example is herein shown in detail, with reference to the external joint corresponding to study case A, whose input data are given in Table 2.

Step 1 - ***Evaluation of the average ultimate moment which the fully yielded and strain hardened beam is able to transmit:***

The distance between the plastic hinge and the column flange is:

$$s_h = \frac{d_b}{2} = \frac{600}{2} = 300\ mm$$

The clear length of the beam is $L_n = 9000 - 359 = 8641\ mm$ and the distance between the plastic hinges is $L_h = L_n - 2s_h = 8641 - 600 = 8041 mm$.

The nominal plastic moment of the beam (steel grade S235) is equal to $M_{b.p} = 786$ kNm.

Considering the beam flange thickness, the overstrength coefficient $\gamma_{ov.rm}$ accounting for the random variability of the material is given by (see Table 1):

$$\gamma_{ov.rm} = \frac{f_{ym.bf}}{f_{y.b}} = \frac{f_0 - \beta\ t_{bf}}{f_{y.b}} = \frac{313.4 - 2.254\ \times 19}{235} = \frac{270.57}{235} = 1.15$$

The average value of the yield stress of the web is equal to:

$$f_{ym.bw} = f_0 - \beta\ t_{bw} = 313.4 - 2.254\ \times 12 = 286.35\ MPa$$

The normalized slenderness parameters of flange and web are equal to:

$$\bar{\lambda}_f = \frac{b_{bf}}{2\ t_{bf}} \sqrt{\frac{f_{ym.bf}}{E}} = \frac{220}{2 \cdot 19} \sqrt{\frac{270.57}{210000}} = 0.208$$

$$\bar{\lambda}_w = \frac{d_{bw}}{2\ t_{bw}} \sqrt{\frac{f_{ym.w}}{E}} = \frac{562}{2 \cdot 12} \sqrt{\frac{286.35}{210000}} = 0.865$$

The beam shear length is equal to $L_e = L_h/2 = 8041/2 = 4020.5\ mm$.

The overstrength coefficient accounting for the influence of strain hardening is:

$$\gamma_{ov.sh} = \frac{1}{0.5463 + 1.6325 \cdot 0.208^2 + 0.0621 \cdot 0.865^2 - 0.6021\frac{220}{4020.5} + 0.0015 \cdot 37.5 + 0.0078 \cdot 12.3} = 1.28$$

Therefore, the average value of the ultimate moment $M_{b,u}$ which can be transmitted by the fully yielded and strain-hardened beam is given by:

$$M_{b.u} = 1.15 \cdot 1.28 \cdot 1.05 \cdot 786 \cong 1214\ kNm$$

With reference to the external joint, the value of the shear action at the plastic hinge axis in the ultimate condition is equal to:

$$V_{bu} = \frac{q\ L_h}{2} + \frac{n_F\ F_d}{2} + \frac{2\ M_{bu}}{L_h} = \frac{1.25 \cdot 8.041}{2} + \frac{3 \cdot\ 65.00}{2} + \frac{2\ \cdot 1214}{8.041} \cong 404.6\ kN$$

Step 2 - ***Calculation of bending moment and shear action at the column flange and evaluation of compression force and tensile force to be transmitted at the beam flanges' levels***:

The flexural and shear action, respectively M_{cf} and V_{cf}, at the column flange are given by:

$$M_{cf} = M_{bu} + V_{bu} \cdot s_h + \frac{q\ s_h^2}{2} = 1214 + 404.6 \cdot 0.3 + \frac{1.25 \cdot\ 0.3^2}{2} \cong 1336\ \ kNm$$

$$V_{cf} = 404.6 + 1.25 \cdot 0.30 \cong 405\ kN$$

Consequently, the compression/tensile force to be transmitted at the beam flanges' level is obtained as:

$$T_u = C_u = \frac{M_{cf}}{d_b - t_{bf}} = \frac{1336000}{600 - 19} \cong 2299\ kN$$

Step 3 - ***Design of the bolt diameter:***

For the design of the diameter of the bolts in tension side the following actions have to be considered:

$$F_{t,Ed} = \frac{T_u}{n_b} = \frac{2299}{4} = 574.75\ kN \qquad F_{v,Ed} = \frac{V_{cf}}{2\ n_b} = \frac{405}{2 \cdot 4} = 50.6\ kN$$

where n_b is the number of bolts in tension.

It is important to underline that, in the proposed procedure, the ductility criterion proposed by the Eurocode 3 Part 1-8 (Section 6.2.7.1) [3] is neglected because ductility requirements are already taken into account by means of capacity design principles.

The check under combined shear and tension lead to evaluation of a first value of the minimum resistant area needed to the bolts. According to Eurocode 3, for 10.9 bolt class:

$$A_{res} \geq \frac{\gamma_{M2}}{f_{tb}}\left(\frac{F_{v,Ed}}{\alpha_v} + \frac{F_{t,Ed}}{1.26}\right)$$

$$= A_{res} \geq \frac{1.25}{1000}\left(\frac{50600}{0.5} + \frac{574750}{1.26}\right) \cong 696.69\ mm^2$$

where A_{res} is the tensile stress area of the bolt, f_{tb} is the ultimate resistance of the bolt' material, $F_{t,Ed}$is the tensile resistance of the bolt, $F_{v,Ed}$ is the shear resistance, α_v is a parameter equal to 0.6 for bolt class 4.5, 5.6 and 8.8 and equal to 0.5 for 4.8, 5.8 and 10.9 bolt class.

According to Eurocode 3, in any case, the resistant area of the bolts has to be greater than the value determined considering only the tension action:

$$A_{res} \geq \frac{\gamma_{M2} F_{t,Ed}}{0.9 f_{tb}} = \frac{1.25 \cdot 574750}{0.9 \cdot 1000} \cong 798.26\ mm^2$$

Consequently, bolts M36 have been chosen.

Step 4 – *Design of the welds:*

According to Eurocode 3, the design of the welds has been carried out considering the throat thickness of the fillet weld in its actual position.

With reference to the welds connecting the flange beam to the end-plate, the length of the both internal and external fillets has been assumed as:

$$l_f = b_{bf} - 2 \cdot r_b - t_{bw} = 220 - 2 \cdot 24 - 12 = 160\ mm$$

in which $b_{bf}$is the beam width, r_b is the root radius and t_{bw} is the beam web thickness.

Therefore the required throat thickness of the weld is:

$$a_f \geq \frac{T_u}{\sqrt{2} \cdot l_f}\frac{\beta_w\, \gamma_{M2}}{f_{tk}} = \frac{2299000}{160\sqrt{2}}\frac{0.80 \cdot 1.25}{360} = 28.23\ mm \quad \rightarrow a_f = 29mm$$

where $\beta_w$is a correlation factor given by EC3 [3].

The welds connecting the web beam and the end-plate have to be able to transmit the shear action V_{cf} and the ultimate flexural action $M_{w.u}$ that the web flange transmits:

$$M_{w.u} = \gamma_{ov.rm} \cdot \gamma_{ov.sh} \cdot \gamma_{M0} \cdot M_{w.p} = 1.15 \cdot 1.28 \cdot 1.05 \cdot 212.07 \cong 327.77\ \text{kNm}$$

The length of the fillets is:

$$l_w = d_{bw} - 2\, r_b = 562 - 2 \cdot 24 = 514\ mm$$

where d_{bw} is the height of the beam web. The thickness results to be:

$$a_w \geq \frac{\beta_w\, \gamma_{M2}}{f_{tk}}\frac{1}{l_w}\sqrt{\frac{8\, M_{w.u}^2}{l_w^2} + \frac{3}{4}\, V_{cf}^2} = \frac{0.8 \cdot 1.25}{360}\frac{1}{514}\sqrt{\frac{8\,(327.77 \cdot 10^6)^2}{514^2} + \frac{3}{4}\,(405 \cdot 10^3)^2}$$

$$\cong 9.92\ mm \qquad \rightarrow a_w = 10\ mm$$

Step 5 – ***Design of the end-plate:***

Considering the design criteria already adopted for the bolts, failure mechanism type 3 can be excluded. Therefore, only the resistance formulations for mechanism type 1 and mechanism type 2 have to be considered to check the equivalent T-stub modelling the end-plate in bending. It is assumed that the distance *m* between the bolt axis and the plastic hinge located close to the beam flange is equal to the minimum technologically compatible, $m=1.2\,d_0$ being d_0 the diameter of the bolt hole. In addition, the width of the plate is defined on the basis of the code requirements for bolt spacing and of technological conditions.

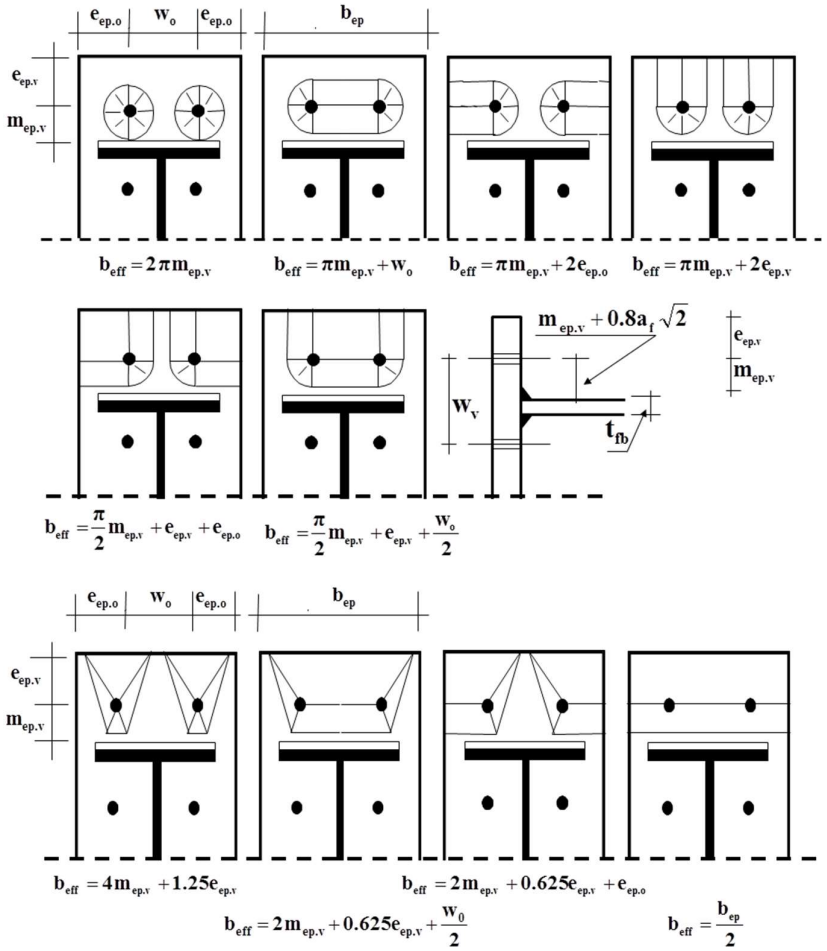


Figure 17 - Determination of the effective length for a single bolt row on the basis of the possible collapse mechanisms

The horizontal distance between the bolts *w* has to satisfy the following limitations:

$$w_{min} \cong t_{cw} + 2\,r_c + 1.8\,d_0 = 21 + 2 \cdot 27 + 1.8 \cdot 37.5 = 142.5 \text{ mm}$$

$$w_{max} = b_{cf} - 2.4\,d_0 = 309 - 2.4 \cdot 37.5 = 219 \text{ mm}$$

where t_{cw} is the thickness of the column web, r_c the root radius of the column, d_0 the hole diameter and b_{cf} the width of the column.

According to the above limitations, the bolt spacing is taken equal to $w_0 = 170\ mm$.

Regarding the width of the end-plate, it should be greater than:

$$b_{ep} = max\{w + 2.4\ d_0; b_{bf}\} = max\{170 + 2.4 * 37.5; 220\} = 260\ \text{mm}$$

and, anyway, smaller than the width of the column that is equal to 309 mm, consequently the width of the end plate is taken equal to 280mm.

As a consequence, the horizontal distance between the bolt axis and the edge of the plate $e_{ep.0}$ is:

$$e_{ep.0} = \frac{b_{ep} - w_0}{2} = \frac{280 - 170}{2} = 55\ \text{mm}$$

For the evaluation of the effective length of the equivalent T-stub, taken $m_x = e_x = 1.2d_0 = 45\ mm$, it results:

$$b_{eff.ep,1} = min\{2\pi m_x; \pi m_x + w;\ \pi m_x + 2e_{ep}\} = min\{282.6;\ 311.3;\ 251.3\} = 251.3\ \text{mm}$$

which accounts for the circular patterns and:

$$b_{eff.ep,2} = min\ \{4m_x + 1.25e_x;\ e_{ep} + 2m_x + 0.625e_x;\ 0.5w + 2m_x + 0.625e_x\} =$$

$$= min\ \{4 \cdot 45 + 1.25 \cdot 45;\ 45 + 2 \cdot 45 + 0.625 \cdot 45;\ 0.5 \cdot 170 + 2 \cdot 45 + 0.625 \cdot 45\} =$$

$$= min\ \{236.25; 173.12; 203.12\} = 203.12\ \text{mm}$$

which accounts for non-circular patterns.

Definitely, the effective length of the equivalent T-stub is:

$$b_{eff.ep} = min\ \{b_{eff.1};\ b_{eff.2};\ 0.5\ b_{ep}\} = min\{251.3; 203.12; 0.5 \cdot 280\} = 140\ mm$$

The thickness of the end-plate required to avoid the collapse of the equivalent T-stub according to type-1 mechanism is:

$$F_{1,Rd} = 2\frac{b_{eff} t_{ep}^2}{m}\frac{f_{y.ep}}{\gamma_{M0}} = T_u \rightarrow \quad t_{ep.1} = \sqrt{\frac{m_x\ T_u\ \gamma_{M0}}{2 \cdot\ b_{eff.ep} \cdot f_{y,ep}}} = \sqrt{\frac{45 \cdot 2299000 \cdot\ 1.05}{2 \cdot\ 140 \cdot 275}}$$

$$\cong\ 37.56\ mm$$

Similarly, to avoid the collapse of the equivalent T-stub according to type-2 mechanism the required end-plate thickness is:

$$F_{2.Rd} = 2\,\frac{\frac{f_{y.ep}}{\gamma_{M0}}\frac{b_{eff}t_{ep}^2}{2} + 2\,F_{t.Rd}\,n}{m+n} = T_u \qquad \rightarrow \qquad t_{ep.2} = \sqrt{\frac{2\,\gamma_{M0}}{b_{eff.ep}\,f_{y.ep}}\left[\frac{T_u(m_x+e_x)}{2} - 2\,F_{t.Rd}e_x\right]} =$$

$$= \sqrt{\frac{2\cdot 1.05}{140\cdot 275}\left[\frac{2299000\,(45+45)}{2} - 2\cdot 588240\cdot 45\right]} \cong 52.50\ mm$$

Therefore, the thickness of the end-plate has been assumed equal to *55mm*.

Step 6 - *Check of the resistance of the column web in shear and design of supplementary web plates if needed:*

The shear resistant area of the column section is given by:

$$A_{vc} = A - 2\,b_{cf}\,t_{cf} + (t_{cw} + 2r_c)t_{cf} = 31200 - 2\cdot 309\cdot 40 + (21 + 2\cdot 27)\cdot 40$$
$$= 9480\ mm^2$$

The resistance of the column web panel, without continuity and/or supplementary plates, is:

$$V_{wp,Rd} = \frac{0.9\cdot A_{vc}\cdot f_{y,cw}}{\sqrt{3}\cdot\gamma_{M0}} = \frac{0.9\cdot 9480\cdot 355}{\sqrt{3}\cdot 1.05} \cong 1665\ \text{kN}$$

Since continuity plates in the both compression and tension zones have been considered, the plastic shear resistance of the column web panel is incremented by the contribution due to the resistant mechanism activated by the continuity plates.

The plastic moment of the column flange is given by:

$$M_{pl.cf.Rd} = \frac{b_{cf}t_{cf}^2 f_{y,c}}{4}\frac{1}{\gamma_{M0}} = \frac{309\cdot 40^2\cdot 355}{4}\frac{1}{1.05} \cong 41.79\ \text{kNm}$$

Therefore the contribution due to the additional resistant mechanism activated by the continuity plates results:

$$V_{wp.add.Rd} = \frac{4\cdot M_{pl.cf.Rd}}{d_s} = \frac{4\cdot 41.79}{0.581} = 287.7\ \text{kN}$$

where d_s is the distance between the centrelines of the stiffeners.

The total resistance of the column web panel is:

$$V_{wp.Rd.tot} = V_{wp.Rd} + V_{wp,add,Rd} = 1665 + 287.7 \cong 1953\ \text{kN}$$

Whereas the shear resistance of the column web panel is lower than the action transmitted by the beam in its ultimate conditions, supplementary web plates are needed whose width is taken equal to:

$$b_{s,max} = d_{cw} - 2r_c = 279 - 2\cdot 27 = 225\ \text{mm}$$

According to Eurocode 3, the resistance of the material constituting the supplementary plates has to be the same of the column; the thickness of the stiffeners results to be:

$$t_s \geq \frac{\sqrt{3}\ \cdot \gamma_{M0}\left(T_u - V_{wp,add,Rd}\right)}{0.9 \cdot b_s \cdot f_{y,wc}} - \frac{A_{vc}}{b_s} = \frac{\sqrt{3}\ \cdot 1.05 \cdot (2299000 - 287700)}{0.9 \cdot 225 \cdot 355} - \frac{9480}{225}$$

$$\cong 9.76\ mm$$

Consequently, it is possible to use a couple of supplementary plates whose thickness is 5 mm or a single supplementary web plate whose thickness is equal to 10 mm.

Step 7 - ***Check of the resistance of the column web in tension and in compression.***

Since continuity plates have been considered in the evaluation of the shear resistance of the column web panel, their design is required. The transverse stiffeners can be designed according to two possible approaches. The first approach requires that the action transmitted from the beam flanges in their ultimate conditions, equal to T_u, is absorbed relying exclusively on the tensile/compression resistance of continuity plates, neglecting the resistance of the column web. The second approach allows the reduction of the thickness of the continuity plates, taking advantage of the contribution due to the resistance of the column web.

In accordance to the latter, the resistance of the column web in compression and the resistance of the continuity plates have to be determined; the former is given by:

$$F_{cwc.Rd} = b_{eff.cwc}\ (\mathrm{t_{cw}} + \mathrm{t_{s.tot}}) \cdot \frac{f_{y.cw}}{\gamma_{M0}} = \frac{546.02\ (21 + 10)\ 355}{1.05} \cong 5723\ \mathrm{kN}$$

where $\mathrm{t_{cw}}$ is the thickness of the column web and $b_{eff.cwc}$ is the effective length of the column web given by:

$$b_{eff.cwc} = t_{fb} + 2\sqrt{2}\,a_f + 5\left(t_{fc} + r_c\right) + 2\,t_{ep} = 19 + 2\sqrt{2}\ \cdot 29 + 5(40 + 27) + 2 \cdot 40 \cong 546.02\ \mathrm{mm}$$

and $\mathrm{t_{s.tot}} = 10\ mm$ is the thickness of the supplementary web plates.

Obviously, if $F_{cwc.Rd} \geq T_u$ it is possible to evaluate the possibility of omitting the continuity plates. In such a case, it is necessary to check again the resistance of column web in shear according to Step 6.

Subsequently, the welds have been designed:

$$a_{cp} \geq \frac{\beta_w t_{cp} f_{y.cp}}{\sqrt{2} f_{tk}} = \frac{0.85 \cdot 20 \cdot 275}{\sqrt{2} \cdot 430} \cong 7.68\ mm \qquad \rightarrow \quad a_{cp} = 8mm$$

Step 8 - *Check of the resistance of the column flange in bending:*

In bolted connections, an equivalent T-stub in tension may be used to model the design resistance of the column flange in bending. As highlighted for the end-plate in bending, failure mode according to mechanism type-3 can be excluded because of the design criterion adopted for the bolts. Therefore, the design resistances for mechanism type-1 and type-2 have to be evaluated. In particular, the following equation has to be considered:

$$F_{1,Rd} \geq T_u \qquad\qquad F_{2,Rd} \geq T_u$$

where:

$$F_{1,Rd} = 2\frac{b_{eff} t_{cf}^2}{m}\frac{f_{y.cf}}{\gamma_{M0}} \qquad \text{and} \qquad F_{2.Rd} = 2\frac{\frac{f_{y.cf}}{\gamma_{M0}}\frac{b_{eff} t_{cf}^2}{2} + 2\, F_{t.Rd}\, n}{m+n}$$

in which b_{eff} is the effective length of the equivalent T-stub corresponding to a single bolt row, t_{cf} is the thickness of the column flange, m is the distance between the bolt line and the plastic hinge arising at the T-stub stem, n is the distance between the bolt line and the end of the plate where the contact forces are concentrated and $f_{y.cf}$ is the yield resistance of the column flange.

With reference to Fig. 18a it is possible to define:

$$m_c = \frac{w - t_{wc} - 1.6\, r_c}{2} = \frac{170 - 21 - 1.6 \cdot 27}{2} = 52.9\ mm$$

while the horizontal distance between the bolt axis and the edge of the column flange is:

$$e = \frac{b_c - w}{2} = \frac{309 - 170}{2} = 69.5\ mm$$

The vertical distance between the first and second bolt rows is:

$$w_v = 2\left(m + 0.8\, a_f\, \sqrt{2} + t_{fb}/2\right) = 2\left(45 + 0.8 \cdot 29\, \sqrt{2} + 19/2\right) = 174.62\ mm$$

In presence of continuity plates whose fillet welds have a throat thickness equal to 8 mm, it results:

$$m_2 = \frac{w_v - t_{cp} - 1.6\, a_{cp}\sqrt{2}}{2} = \frac{174.62 - 20 - 1.6 \cdot 8\sqrt{2}}{2} = 68.26\ mm$$

According to Eurocode 3 the effective length, in presence of transverse stiffeners, is given by:

$$b_{eff} = min\{2\pi m_c\ ;\ \alpha \cdot m_c\} = min\ \{2\pi \cdot 52.9\ ;\ 5.93 \cdot\ 52.9\} \cong 313.7\ mm$$

where the parameter α has been determined considering the geometrical parameters λ_1 and λ_2:

$$\lambda_1 = \frac{m_c}{m_c + e} = \frac{52.9}{52.9 + 69.5} = 0.43 \qquad \lambda_2 = \frac{m_2}{m_c + e}\frac{68.26}{52.9 + 69.5} = 0.57$$

679 by means the abacus in Fig. 18b:

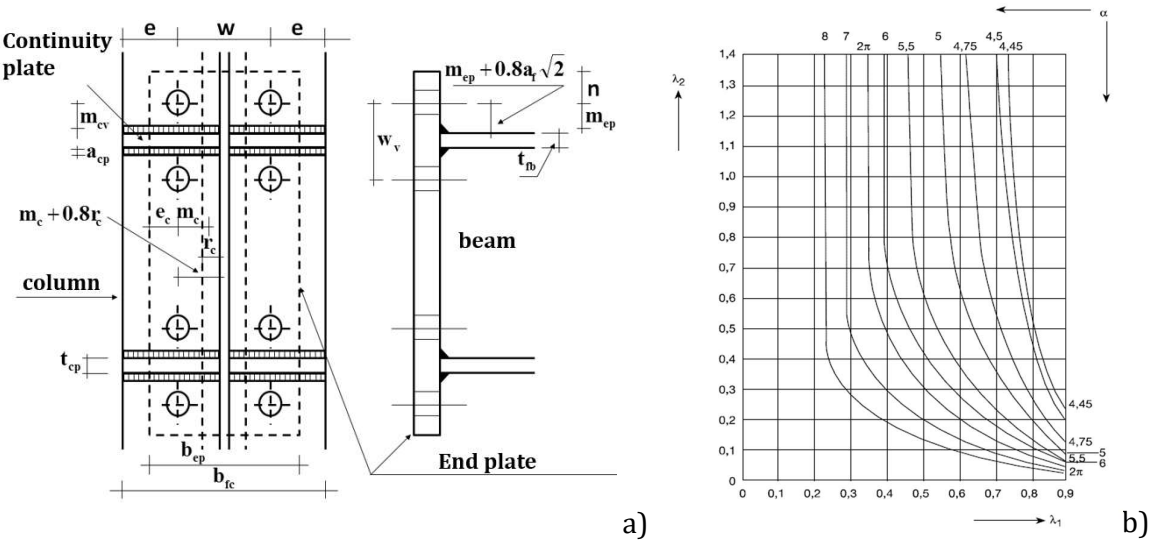


Figure 18: *Column flange: a)geometrical properties; b) abacus*

680

681 Thereafter, the design resistances for mechanisms type-1 and type-2 are given by:

$$F_{1,Rd} = 2\frac{f_{y.cf}\, b_{eff.cfb}\, t_{cf}^2}{m_c}\frac{1}{\gamma_{M0}} = 2\frac{355 \cdot 313.7 \cdot 40^2}{52.9}\frac{1}{1.05} \cong 6416\ \text{kN} \geq T_u$$

$$F_{2.Rd} = 2\frac{f_{y.cf}\frac{b_{eff.cfb}t_{cf}^2}{2} + 2\, F_{t.Rd}\, n}{m_c + n}\frac{1}{\gamma_{M0}} = 2\frac{355\frac{313.7 \cdot 40^2}{2} + 2 \cdot 588240 \cdot 55}{52.9 + 55}\frac{1}{1.05}$$
$$\cong 2715\ \text{kN} \geq T_u$$

682 where $n = \min\{e;\ e_{ep}; 1.25m_c\} = \min\{69.5;\ 55; 1.25 \cdot 52.9\} = 55\ mm$.

683 Since the both design resistances are greater than the action T_u, derived by means of capacity
684 design principles, the check of the column flange in bending is satisfied.

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