DUCTILITY OF HIGH PERFORMANCE STEEL MOMENT CONNECTIONS

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ABSTRACT: Recent developments in high performance construction materials have made them attractive for the design of buildings, as light weight and thin elements are desirable for architectural reasons and for the reduction of environmental impacts of construction. Steel framework is a typical structural system in modern buildings. Current analysis and design of steel-framed buildings are usually carried out under the semi-continuous/partially-restrained philosophy. This type of structural modelling is only feasible if the structural joints are designed for rotation capacity and ductility. Members and joints made up of high performance steel exhibit improved strength but limited deformation capacity compared to mild steel grades. Connections between members, in particular, are the regions where the material is exposed to higher deformations demands. The designer then has to ensure that they undergo large inelastic deformations. To address this topic, an experimental investigation was undertaken on moment connections with end plates made from high strength steel grades S460, S690 and S960 (yield stress of 460 MPa, 690 MPa and 960 MPa, respectively) to provide insight into the nonlinear behaviour of this joint type. The major contributions of this study are (i) the validation of current Eurocode 3 specifications for the design of joints and (ii) the ductility analysis of high performance steel joints to verify whether they have large plastic deformations.

Keywords: Ductility; end plate connections; experimental testing; high-strength steel; resistance; rotation capacity; stiffness

1.   INTRODUCTION

Steel framework is a typical structural system in modern building construction. In this type of construction, beam-column joints have to transfer the beam and floor loads to the columns. Generally, the forces transmitted through the joints can be axial and shear forces, bending and torsion moments. The bending deformations are predominant in most cases, when compared to axial and shear deformations that are hence neglected. The effect of torsion is also negligible in planar frames. Typical beam-column moment-resisting joints in steel-framed structures include bolted end plate connections, bolted connections with angle cleats and welded connections. The behaviour of these joints is intrinsically nonlinear. This nonlinearity arises because a joint is an assemblage of several components that interact differently at distinct levels of applied loads. The interaction between the elemental parts includes elastoplastic deformations, contact, slip and separation phenomena. The analysis of this complex behaviour is usually approximate in nature with drastic simplifications. Tests (both experimental and numerical) are frequently carried out to obtain the actual response, which is then modelled approximately by mathematical expressions that relate the main structural joint properties. In the case of moment-resisting joints, three properties are usually sufficient for characterization of their behaviour (Figure 1): (i) moment resistance, (ii) rotational stiffness and (iii) rotation capacity. Research work on this subject has been intensive over the past years [1-7]. Emphasis was mainly placed on full strength joints and, therefore, only the resistance and stiffness properties were fully characterized. Rules for prediction of strength and stiffness of this joint type were thus included in modern design codes as the European code of practice for the design of structural steel joints in buildings, part 1-8 of Eurocode 3 [8] (EN 1993-1-8), and the specification for structural steel buildings from the American Institute of Steel Construction [9].
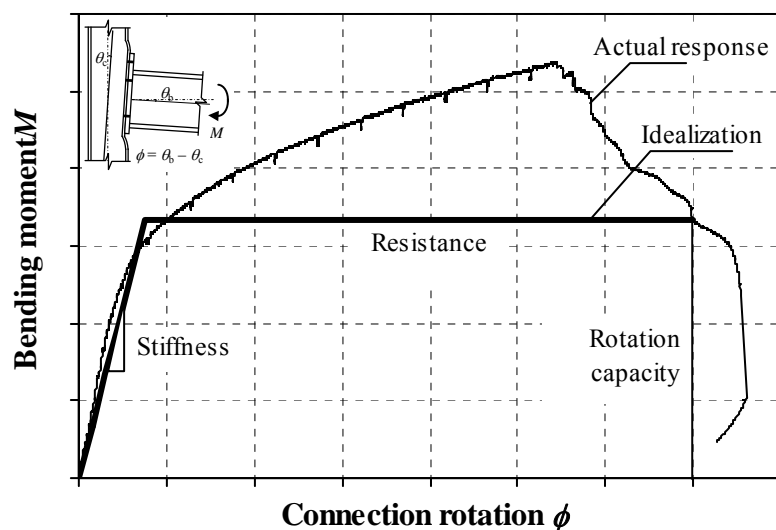
Figure 1. Moment-rotation Response of a Joint (Definitions)

Several authors have recently highlighted the importance of designing structural joints for rotation capacity and ductility [10-13]. The knowledge of the plastic deformation supply of members and joints is particularly important for steel frames that experience abnormal loading conditions, such as fire and seismic events. In these extreme events, very high demands for local and global deformation are imposed on structural elements, connections and details. Connections between members, in particular, are the regions where the material is exposed to higher deformations and, consequently, they influence local ductility requirements and framing performance. EN 1993-1-8 includes simple deemed-to-satisfy criteria to guarantee that the available deformation capacity is higher than the deformation demand of a specific joint. Derivation of such criteria can be found in References 14 and 15. In the context of the component method that considers a joint as an assembly of elemental parts (or components), some researchers have developed simplified approaches to quantify the overall rotation capacity of joints [7,11,12,16,17].

Recent interest in high performance construction materials to produce innovative structures again draws attention to issues related to the deformation capacity of members and, specially, connections. High performance steels (HPS) in construction represent a family of steels with a yield stress above 460 MPa and offer higher performance in tensile stress, toughness, weldability and corrosion compared to mild steel grades [18]. The mechanical characteristics of these steels are different from conventional steels. HPS are generally less ductile and have a smaller (i) yield plateau and (ii) yield stress-to-tensile stress ratio.

Traditionally, the design of structures fabricated from HPS is essentially based on an elastic concept as the service requirements usually govern the structural response. The limit states are then the attainment of the (i) yield stress or (ii) buckling stress at the critical locations. Structural strength and stiffness of members and connections can be derived relatively easily in this case. Designers have however recognized the need to account for inelastic effects directly in structural analysis. Advanced analysis that includes material plasticity, geometric nonlinearities and connection behaviour is then required. Part 1-12 of Eurocode 3 [19] (prEN 1993-1-12) presently forbids elastic-plastic global structural analysis with plastified sections and/or joints acting as plastic hinges. This standard states that the rules for semi-rigid joints are not applicable to HPS and that the resistance of joints should always be based on elastic distribution of forces over the components [19].

Whether or not HPS can be used for plastic design will depend on the outcome of intensive

research. This motivated an experimental and numerical comprehensive investigation on "HPS in Civil Engineering Structures" that is being carried out at the Department of Structural and Building Engineering of the Delft University of Technology. The research work presented in this paper mainly focus on moment connections employing HPS. Experiments on statically loaded bolted end plate connections were undertaken to provide insight into the behaviour of this connection type. Since an end plate connection comprises several critical components, it was decided to focus this study on the single contribution of the assembly end plate-bolts in the tension zone. These components are modelled as equivalent T-stubs [2,4] (Figure 2). Previous work on similar connection details has shown that the deformation characteristics primarily depends on (i) the connection geometry, particularly the ratio between the end plate thickness t_{ep}, and the bolt diameter ϕ_b, (ii) the assurance of a good material ductility for plate and bolts, (iii) the ratio between the resistance of the plate and the bolt and (iv) the welding quality [7,12,14]. Test results were used to (i) validate current specifications for the design of joints in EN 1993-1-8, within the semi-continuous/partially-restrained concept, (ii) to examine the connection response from a strength and deformation point of view and (iii) to verify whether HPS moment connections develop sufficient rotation capacity for global plastic analysis.
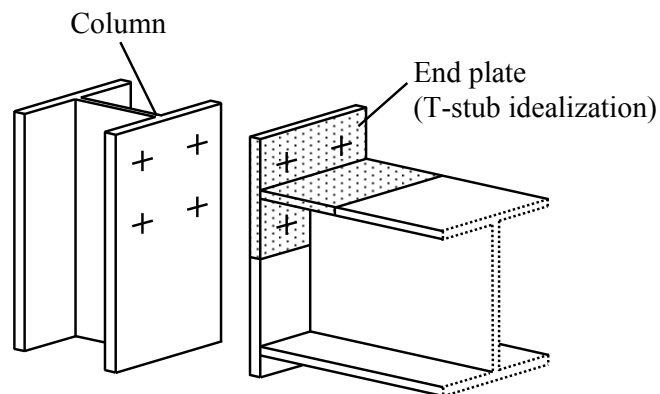

Figure 2. Extended End Plate Connection: T-stub Idealization

This paper specifically deals with the latter topic. A brief description of the experimental programme and results is given. Comparisons with code predictions are also drawn. The paper opens the way for the establishment of requirements for sufficient rotation capacity as well as ductility requirements to avoid brittle failures and concludes by outlining future research that is still required.

2.   EXPERIMENTAL PROGRAMME

2.1   Test Details

The experimental programme consisted of seventeen beam-column bolted end plate connections. The specimens were designed to trigger failure in the assembly end plate-bolts in the tension zone without development of the full plastic moment capacity of the beam (partial-strength joint). Beam and column sections are of class 1 [20].

Extended end plate (EEP) and two different flush end plate configurations (F1EP and F2EP) were tested (Figure 3a). For the second flush end plate configuration (F2EP), a detail of the welding to the beam is also given in Figure 3b. A general review and description of the test programme is given elsewhere [21,22]. The relevant details of the test specimens are given in Figure 3c and Table 1. Bolts were hand-tightened to give a snug fit in all sets.

(a) Configurations

(b) Detail of the Welding of the End Plate to Beam Flange (Outside) for Flush Configuration 2

(c) End Plate Dimensions

Figure 3. End Plate Configurations and Dimensions

Table 1. Details of Test Specimens

Test ID	Column		Beam		End plate					Bolt		
	Section	Grade	Section	Grade	t_{ep} (mm)	Grade	$f_{y.ep}$ (MPa)	ρ_y	ε_f	ϕ_b (mm)	Grade	$f_{u.b}$ (MPa)
F1EP_15_1	HE300M	S355	HE320A	S355	15.30	S460	483	0.84	0.175	24	12.9	1413
F2EP_15_1	HE300M	S355	HE320A	S355	15.30	S460	483	0.84	0.175	24	12.9	1413
EEP_15_1	HE300M	S355	HE320A	S355	15.25	S460	483	0.84	0.175	24	8.8	940
F1EP_15_2	HE300M	S355	HE320A	S355	14.75	S690	774	0.95	0.186	24	12.9	1413
F2EP_15_2	HE300M	S355	HE320A	S355	14.64	S690	774	0.95	0.186	24	12.9	1413
EEP_15_2	HE300M	S355	HE320A	S355	14.62	S690	774	0.95	0.186	24	12.9	1413
F1EP_10_2	HE300M	S355	HE320A	S355	10.15	S690	698	0.93	0.175	24	12.9	1413
F2EP_10_2	HE300M	S355	HE320A	S355	10.25	S690	698	0.93	0.175	24	12.9	1413
EEP_10_2a	HE300M	S355	HE320A	S355	10.10	S690	698	0.93	0.175	24	12.9	1413
EEP_10_2b	HE300M	S355	HE320A	S355	10.10	S690	698	0.93	0.175	24	8.8	940
F1EP_10_3	HE300M	S355	HE320A	S355	10.00	S960	952	0.91	0.154	24	12.9	1413
F2EP_10_3	HE300M	S355	HE320A	S355	10.00	S960	952	0.91	0.154	24	12.9	1413
EEP_10_3	HE300M	S355	HE320A	S355	10.00	S960	952	0.91	0.154	24	12.9	1413
F2EP_10_2(M27)	HE300M	S355	HE320A	S355	10.05	S690	698	0.93	0.175	27	8.8	1013
EEP_10_2(M27)	HE300M	S355	HE320A	S355	10.10	S690	698	0.93	0.175	27	8.8	1013
F2EP_10_3(M27)	HE300M	S355	HE320A	S355	10.00	S960	952	0.91	0.154	27	8.8	1013
EEP_10_3(M27)	HE300M	S355	HE320A	S355	10.03	S960	952	0.91	0.154	27	8.8	1013

The actual mechanical properties for the end plates and bolts are also given in Table 1 (f_y: yield stress; f_u: tensile stress; $\rho_y = f_y/f_u$: yield ratio; ε_f: strain at rupture load). The constitutive law of the several classes of HPS is plotted in Figure 4.
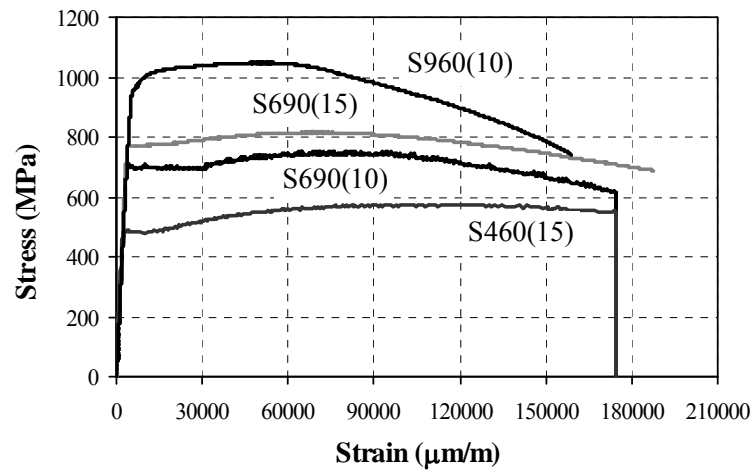
Figure 4. Stress-strain Behaviour of the End Plate Material

2.2 Moment-rotation Curves and Failure Modes

The behaviour of moment connections is typically represented by a moment-rotation curve that describes the relationship between the applied bending moment, M and the corresponding rotation between the members, ϕ. Figure 5 illustrates a typical moment-rotation response of the connections tested. This curve defines three main structural properties: (i) (plastic and ultimate) moment resistance, $M_{j.R}$ and $M_{j.max}$, (ii) (initial and post-yield) rotational stiffness, $S_{j.ini}$ and $S_{j.p-l}$ and (iii) rotation capacity, ϕ_C. The characteristics of these curves for the several test details are set out in Table 2. The failure modes of the specimens are also indicated in the table: (i) mode A: cracking of the end plate in the heat affected zone, (ii) mode B: bolt-thread stripping and (iii) mode C: bolts in tension. Illustration of these modes of failure is given in Figure 6.

The M-ϕ response for representative test specimens is given in Figure 7. Several conclusions can be drawn from the analysis of these graphs and the curves properties in Table 2:
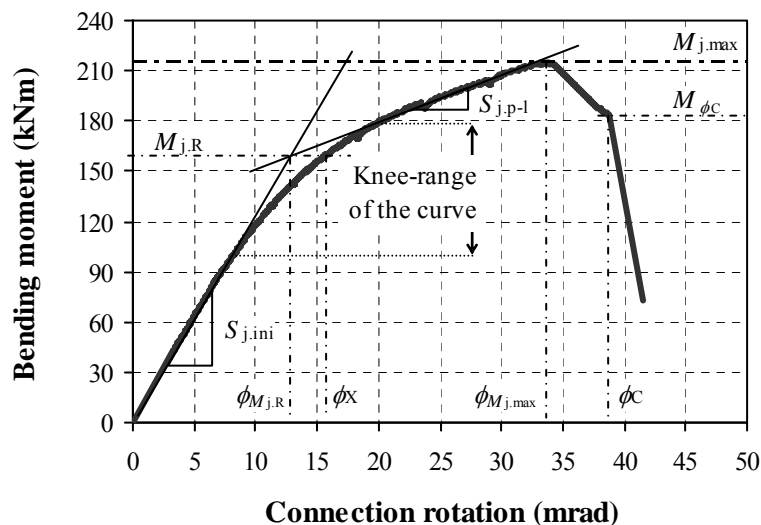
Figure 5. Typical Moment-rotation Response (e.g. F2EP_15_2)

Table 2. Main (Experimental) Characteristics of the Moment-rotation Curves

Test ID	Stiffness (kNm/mrad)		Resistance (kNm)		Rotation capacity (mrad)		Failure mode
	$S_{j.ini}$	$S_{j.p-l}$	$M_{j.R}$	$M_{j.max}$	$\phi_{M_{j.max}}$	ϕ_C	
F1EP_15_1	18.4	0.18	192	198	33	33	Mode A
F2EP_15_1	12.7	2.1	128	172	32	40	Mode A
EEP_15_1	30.0	3.3	270	326	28	28	Mode B
F1EP_15_2	14.1	2.6	175	231	34	37	Mode C
F2EP_15_2	12.3	2.6	160	215	33	39	Mode C
EEP_15_2	35.3	9.7	245	366	20	20	Mode C
F1EP_10_2	7.8	1.9	95	142	39	46	Mode A
F2EP_10_2	7.2	1.4	89	117	35	41	Mode A
EEP_10_2a	17.2	3.2	173	244	36	45	Mode A
EEP_10_2b	19.9	5.4	188	252	37	46	Mode A
F1EP_10_3	9.3	–	171	201	46	46	Mode C
F2EP_10_3	8.0	0.8	155	176	52	–	Mode C
EEP_10_3	20.7	4.0	235	326	38	38	Mode C
F2EP_10_2(M27)	7.2	1.3	98	130	38	38	Mode A
EEP_10_2(M27)	23.2	3.4	195	266	30	52	Mode A
F2EP_10_3(M27)	10.9	1.9	123	173	42	67	Modes A&C
EEP_10_3(M27)	23.0	3.7	253	314	31	44	Mode A

(a) Mode A: Cracking of the End Plate in the Heat Affected Zone (Specimen EEP_10_2(M27))

(b) Mode B: Bolt-thread Stripping (Specimen EEP_15_1, bolt #3)
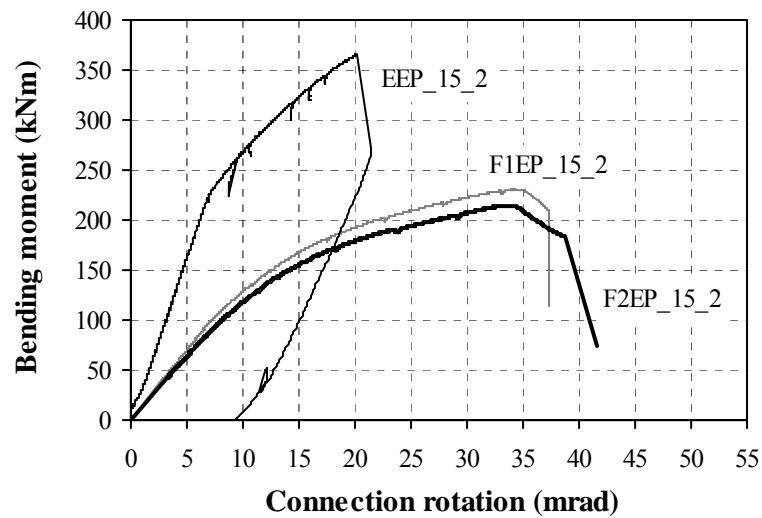

Specimen F1EP_15_2   Specimen EEP_15_2 (general view and detail)
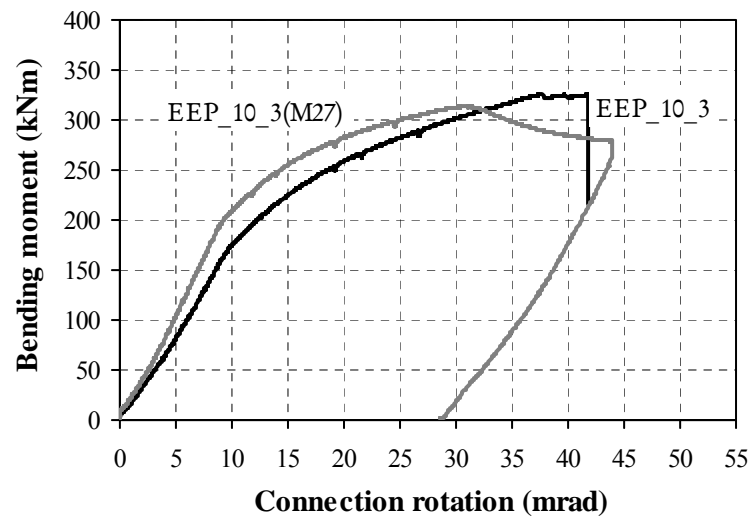
(c) Mode C: Bolts in Tension

Figure 6. Illustration of the Failure Modes

(d) Combined Failure Modes A and C (Specimen EEP_10_3)
Figure 6. Illustration of the Failure Modes (cont.)

(a) 15 mm End Plates, S690

(b) 10 mm Extended End Plates, S960
Figure 7. Moment-rotation Curves for Specimens Employing S690 and S960

1. The rotational stiffness of the joints increases with the end plate thickness; because this property mainly depends on the Young modulus, there are no relevant variations with the plate steel grade.
2. The moment resistance enhances with the end plate thickness and the plate steel grade.
3. The rotation capacity decreases with the thickness of the plate; in general, it also decreases with the plate steel grade, though this variation is also linked to the governing failure mode.

4.   The behaviour of the two flush end plate configurations is identical over the entire elastoplastic range (Figure 7a).

5.   The behaviour of extended end plate joints is much stiffer than "parent" flush end plate configurations; moment resistance is also larger but the deformation capacity is smaller (Figure 7a).

6.   The joint performance, in terms of resistance and stiffness, with bolts M24 grade 12.9 and bolts M27 grade 8.8 is equivalent; however, from a ductility point of view, bolt grade 8.8 should be taken (Figure 7b). Bolts 12.9 exhibited very limited ductility and hardly any deformation in bending. In fact, in some cases (e.g. specimens with extended end plates) failure of the bolt occurred due to excessive rotation near the bolt head. Therefore, a strong recommendation against the use of this class of bolts is made.

The usual bolt diameter/end plate thickness combination for mild steel grades (e.g. bolts M24 in 15 mm plates) was checked for cases where HPS was present. Alternatively, the test programme included M27 8.8 bolts in 10 mm end plates, to check whether the overall ductility improved. In the latter case, very strong bolts were used in relatively thin end plates. The results however were rather disappointing since the rotation capacity did not increase as expected. The overall behaviour was dominated by cracking of the plate in the heat affected zone.

3.   VERIFICATION OF EUROCODE 3 PREDICTIONS ON TEST RESULTS

The design of joints made up of HPS up to S700 is covered in prEN 1993-1-12, as mentioned above. This pre-norm does not allow the design of HPS joints within the semi-continuous/partially-restrained concept [19]. Experiments have however shown that yielding of the end plate also occurs with HPS S690 or S960. Still, the adoption of this type of structural modelling for global analysis is only adequate if the joint develops sufficient rotation capacity so that a ductile failure mechanism of the whole structure can be formed prior to fracture of the joint. Literature shows that end plate connections can achieve rotation capacity provided that the end plate is a "weak link" relative to the bolts [12,14,23]. The conclusions however were validated for mild steel grades. Current tests employ end plates and bolts with similar mechanical properties, in terms of yield stress and yield ratio. It has to be investigated whether this influences the above premises.

In this section, a comparison between test results and current EN 1993-1-8 design provisions is undertaken. The code gives quantitative rules for the prediction of the joint flexural plastic resistance and initial stiffness. These structural properties are evaluated below by using the actual geometrical and mechanical properties [21]. The recommendations on rotation capacity are also verified to investigate if there is enough rotation capacity according to EN 1993-1-8. The provisions are compared with the test results below. The partial safety factors were taken as unitary.

The initial stiffness is evaluated in the framework of the component method. The method is illustrated in Figure 8 for the case of bolted extended end plate connections (with two bolt rows in tension). For the computation of the joint rotational stiffness, the active joint components for this configuration, according to EN 1993-1-8 are: column web in shear (cws), column web in compression (cwc), column web in tension (cwt), column flange in bending (cfb), end plate in bending (epb), and bolts in tension (bt). The welds connecting the end plate and the beam are not taken into account for computation of the rotational stiffness, as well as components beam web and flange in compression (bfc) and beam web in tension (bwt). The basic joint components are modelled by means of extensional springs characterized by an elastic stiffness k_e. These components are grouped according to their type of loading (tension, compression or shear) and are

then assembled into a mechanical model in order to evaluate the M-ϕ response of the whole joint. Mechanical (component) models use a set of rigid and flexible parts (springs) to simulate the interaction between the various sources of joint deformation. The springs are combined in series or in parallel depending on the way they interplay with each other. Springs in series are subjected to the same force whilst parallel springs undergo the same deformation.

The following expression can be readily derived from the code [8]:

$$S_{j.ini} = \frac{z^2}{1/k_{ec} + 1/k_{et}}$$

(1)

whereby z: lever arm; k_{ec} and k_{et}: stiffness of the assembly of components in the compression/shear zone and in the tension zone, respectively (Figure 8).
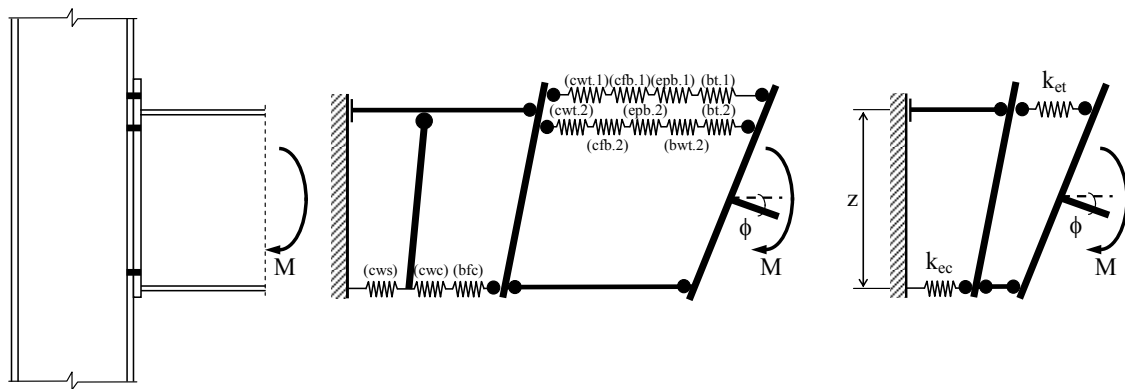
Figure 8. Component Method: Identification of Active Components and Mechanical Model Adopted in EN 1993-1-8

End plate moment connections transmit moment by coupling tension force(s) in the bolts with compression at the opposite flange. The joint design flexural resistance, $M_{j.R}$, in the absence of an axial force, is then calculated from simple equilibrium considerations:

$$M_{j.Rd} = \sum_{i=1}^{n} F_{ti.Rd} h_i$$

(2)

where $F_{ti.R}$: resistance of bolt row i in the tension zone [8] and h_i: distance of the i-th bolt row from the centre of compression (subscript "d" indicates "design value").

Table 3 compares the code predictions with the experiments (subscripts "EC3" and "exp", respectively). The code overestimates structural stiffness properties. This is in line with experimental evidence from connections made up of mild steels [12]. The predictions for resistance compare well with test results.

The code recommendations on rotation capacity are also verified (Table 3). EN 1993-1-8 simply gives some guidelines on the subject: a bolted end plate joint may be assumed to have sufficient rotation capacity for global plastic analysis, provided that both of the following criteria are satisfied: (i) the moment resistance of the joint is governed by the resistance of either the column flange or the end plate in bending and (ii) the maximum (subscript "max") thickness t of either the column flange or end plate – not necessarily the same component as (i) – fulfils:

$$t_{max} = 0.36 \phi_b \sqrt{f_{u.b}/f_y}$$

(3)

where ϕ_b: bolt diameter; $f_{u.b}$: tensile strength of the bolt and f_y: yield stress of the relevant basic component. These guidelines are yet insufficient to ensure adequate ductility in partial-strength joints.

Table 3. EN 1993-1-8 Predictions of the Structural Properties of
the Joints Tested and Ratio to the Experiments

Test ID	Stiffness (kNm/mrad)			Resistance (kNm)			Rotation capacity	
	$S_{j.ini.EC3}$	$S_{j.ini.exp}$	Ratio	$M_{j.R.EC3}$	$M_{j.R.exp}$	Ratio	$t_{ep.max}$ (mm)	Eq. (3) verifies?
F1EP_15_1	23.3	18.4	1.27	149	192	0.78	14.78	No. (t_{ep} = 15.30 mm)
F2EP_15_1	23.9	12.7	1.88	152	128	1.19	14.78	No. (t_{ep} = 15.30 mm)
EEP_15_1	58.7	30.0	1.96	244	270	0.90	12.05	No. (t_{ep} = 15.25 mm)
F1EP_15_2	22.6	14.1	1.60	167	175	0.95	11.67	No. (t_{ep} = 14.75 mm)
F2EP_15_2	21.5	12.3	1.75	164	160	1.03	11.67	No. (t_{ep} = 14.64 mm)
EEP_15_2	58.0	35.3	1.64	369	245	1.51	11.67	No. (t_{ep} = 14.62 mm)
F1EP_10_2	12.4	7.8	1.59	104	95	1.09	12.29	Yes. (t_{ep} = 10.15 mm)
F2EP_10_2	12.4	7.2	1.72	104	89	117	12.29	Yes. (t_{ep} = 10.25 mm)
EEP_10_2a	31.9	17.2	1.85	184	173	1.06	12.29	Yes. (t_{ep} = 10.10 mm)
EEP_10_2b	34.3	19.9	1.72	184	188	0.98	10.03	No. (t_{ep} = 10.10 mm)
F1EP_10_3	11.8	9.3	1.27	138	171	0.81	10.53	Yes. (t_{ep} = 10.00 mm)
F2EP_10_3	12.3	8.0	1.54	140	155	0.90	10.53	Yes. (t_{ep} = 10.00 mm)
EEP_10_3	31.9	20.7	1.54	247	235	1.05	10.53	Yes. (t_{ep} = 10.00 mm)
F2EP_10_2(M27)	12.1	7.2	1.68	101	98	1.03	11.71	Yes. (t_{ep} = 10.05 mm)
EEP_10_2(M27)	32.8	23.2	1.41	184	195	0.94	11.71	Yes. (t_{ep} = 10.10 mm)
F2EP_10_3(M27)	12.1	10.9	1.11	137	123	1.11	10.03	Yes. (t_{ep} = 10.00 mm)
EEP_10_3(M27)	32.8	23.0	1.43	247	253	0.98	10.03	Yes. (t_{ep} = 10.03 mm)

4.   CONSIDERATIONS ON THE JOINT DUCTILITY

Analysis and design of steel frameworks are usually carried out under the semi-continuous/partially-restrained philosophy. The consistency of this type of structural modelling requires the establishment of accurate criteria regarding (i) the rotation capacity and (ii) the ductility of the joints. The rotation capacity is the angle through which the joint can rotate for a given resistance level without failing [8]. The ductility properties of a joint reflect the length of the yield plateau of the moment-rotation response. Both criteria should be based on the mechanical and geometrical characteristics of the joint components.

To meet the criterion for rotation capacity, the available joint rotation must be higher than the required joint rotation. For mild steel grades, it is generally accepted that a minimum of 35-40 mrad ensures "sufficient rotation capacity" of a bolted joint in a partial-strength scenario. Wilkinson et al. suggest that a moment connection in steel moment resisting frames in a seismic area must develop a minimum plastic rotation, ϕ_p, of 30 mrad [24]. The validation of this criterion for HPS however requires further investigation. Table 4 computes the experimental values of joint plastic rotation, which corresponds to the difference between the ultimate joint rotation, ϕ_C, and the first yielding rotation, ϕ_y [7]:

$$\phi_y = \frac{2/3\, M_{j.R}}{S_{j.ini}} \qquad (4)$$

Previous experimental results from tests carried out by the authors [12,26,27] are also included in this table. Tests FS1 and FS4 correspond to a bolted end plate connection between a beam IPE300,

steel grade S355, and a rigid column. The test configuration is similar to the extended end plate specimens reported herein. The end plates had a 10 mm thickness and employed steel S355 in test FS1 and S690 in test FS4.

In general, thinner end plates meet the above criterion.

The requirements for ductility ensure that brittle failures are avoided, i.e. the inelastic deformations are sufficiently large. The joint ductility strongly depends on the material performance (for plates and bolts). Nowadays, the quality of HPS meets similar standards to mild steel grades. Uniaxial tension tests on HPS coupons have shown that these steels can achieve elongations at fracture up to 20%, which is considered excellent [18,25]. However, the assurance of a good material ductility does not necessarily imply that the whole structure will behave ductile. The behaviour of steel is strongly dependent on the load history and the presence of residual strains due to prior occurrence of large deformations.

The ductility can be quantified by means of the joint ductility index ϑ_j that relates the rotation capacity of the joint, ϕ_C to the rotation value corresponding to the joint plastic resistance, $\phi_{Mj.R}$ [12,26]:

$$\vartheta_j = \frac{\phi_C}{\phi_{M_{j.R}}} \tag{5}$$

Table 4 evaluates the joint ductility index ϑ_j for the several specimens. Experimentally, the rotation capacity was defined at the rotation level for which failure of one or more components occurred.

Table 4 also includes the rotation values at maximum load and the corresponding ductility levels, $\vartheta_{j.max\ load}$:

$$\vartheta_{j.max\ load} = \frac{\phi_{M_{j.max}}}{\phi_{M_{j.R}}} \tag{6}$$
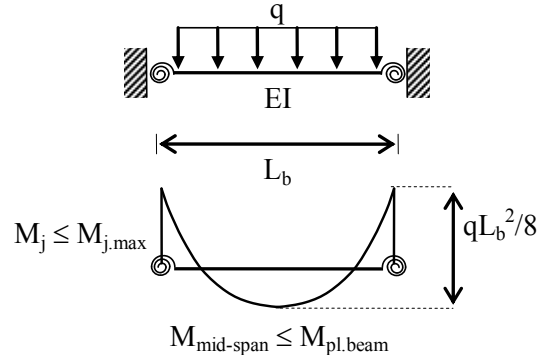
Some conclusions can be drawn from the analysis of the values in Table 4:

1. Within the same test series, the ductility indexes are similar for both flush end plate configurations; this means that the welding detail did not play an important role in the joint behaviour.
2. The extended end plate configuration yields higher ductility indexes but lower rotation capacity, except for specimens with 15 mm end plates.
3. Bolts 8.8 ensure a more ductile behaviour when compared to bolts 12.9 (see results for specimens EEP_10_2a and EEP_10_2b.
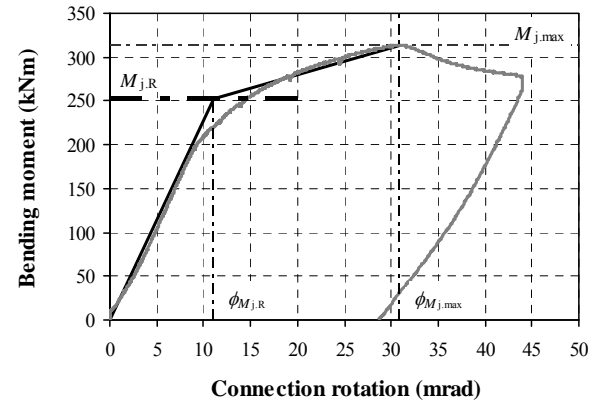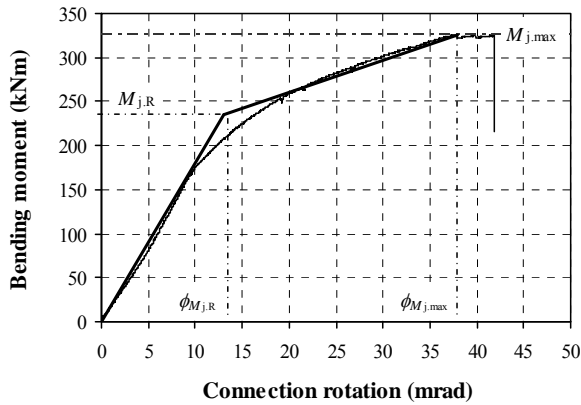
In order to investigate the influence of HPS grades in the deformation demand of a joint when inserted into a braced frame, a simple beam model is analysed (Figure 9a). Beam length, L_b, is taken as $20h_b$ and $50h_b$ (h_b: beam section height). Rotational springs at the beam ends simulate the connection to the column. The beam is subjected to a uniformly distributed load, q. With partial-strength joints, the beam flange remains elastic and the first plastic hinges are formed in the joints. The third plastic hinge is formed in the mid-span, provided that the joint delivers enough rotation.

Table 4. Experimental Evaluation of the Joint Ductility Indices $\vartheta_{j.max\ load}$ and ϑ_j and the Plastic Rotation Supply $\phi_{p.u}$

Test ID	$\phi_{y.exp}$ (mrad)	$\phi_{Mj.R.exp}$ (mrad)	$\phi_{Mj.max}$ (mrad)	$\phi_{C.exp}$ (mrad)	$\phi_{p.u.exp}$ (mrad)	$\vartheta_{j.max\ load}$	ϑ_j
F1EP_15_1	7	12	33	33	26	2.75	2.75
F2EP_15_1	7	10	32	40	33	3.20	4.00
EEP_15_1	6	9	28	28	22	3.11	3.11
F1EP_15_2	8	12	34	37	29	2.83	3.08
F2EP_15_2	9	13	33	39	30	2.54	3.00
EEP_15_2	5	7	20	20	15	2.86	2.86
F1EP_10_2	8	12	39	46	38	3.25	3.83
F2EP_10_2	8	12	35	41	33	2.92	3.42
EEP_10_2a	7	10	36	45	38	3.60	4.50
EEP_10_2b	6	9	37	46	40	4.11	5.11
F1EP_10_3	12	*	46	46	34	—	—
F2EP_10_3	13	*	52	—	—	—	—
EEP_10_3	8	11	38	38	30	3.45	3.45
F2EP_10_2(M27)	9	14	38	38	29	2.71	2.71
EEP_10_2(M27)	6	9	30	52	46	3.33	5.78
F2EP_10_3(M27)	8	14	42	67	59	3.00	4.79
EEP_10_3(M27)	7	12	31	44	37	2.58	3.67
FS1	5	7	77	111	106	11.9	17.1
FS4	7	10	44	64	57	4.6	6.8

(a) Beam: Parameters

(i) e.g. EEP_10_3   (ii) e.g. EEP_10_3(M27)

(b) Moment-rotation Characteristics of the Joints: Actual Behaviour and Bilinear Approximation

Figure 9. Beam Model

The actual profiles from the tested specimens are adopted in the analysis. For good comparison of results, a modification to the actual beam mechanical behaviour is introduced. All the above connections use a beam of grade S355. An actual frame however would employ beams and end plates of similar steel grade, i.e. end plates S690 and beams S690, for instance. In this analysis, the steel grade of the beam and the end plate is identical. This solution was not adopted in the tests because rolled profiles of steel grades S690 and S960 are hardly economically available for research purposes. For this beam-end plate steel grades combination, the degree of partial-strength reduces considerably.

The actual behaviour of the various joints is approximated by a simplified bilinear relationship (Figures 5 and 9b). Joints F2EP_15_1, F2EP_15_2, EEP_10_2a, EEP_10_3, EEP_10_2(M27), EEP_10_3(M27), FS1 and FS4 are used for further comparisons.

The results of this analysis are set out in Tables 5-6 and plotted in the graphs from Figures 10 (L_b = $50h_b$ in all graphs) and 11, in terms of connection rotation vs. ratio $\varsigma = qL_b^2/M_{pl.beam}$. Figure 11, in particular, shows that the beam span length mainly influences the results in the post-yield behaviour domain of the joints. Load q is incremented from zero until (i) a plastic hinge is formed in the mid-span ($M_{mid-sp} = M_{pl.beam}$) or (ii) the maximum rotation of the joints is reached. In all cases, the second condition governs (Tables 5 and 6, sixth column). For each test, the plastic moment of the beam section is computed as follows:

$$M_{pl.beam} = W_{pl.b} f_{y.ep.nom} \tag{7}$$

whereby $W_{pl.b}$: plastic (actual) beam cross-section modulus and $f_{y.ep.nom}$: nominal yield stress of the corresponding end plate (Table 7, second and third columns).

Table 5: Results from the Beam Analysis for $L_b = 20h_b$

Test ID	ϕ (mrad)	ς	Failure conditions $\left(\phi = \phi_{M_{j.max}}\right)$		
	($\varsigma = 2.5$)	($\phi = 30$ mrad)	q_u (kN/m)	$M_j/M_{pl.beam}$	$M_{mid-sp}/M_{pl.beam}$
F2EP_15_1	8.9	4.9	98.0	0.23	0.41
F2EP_15_2	14.7	3.7	113.0	0.19	0.30
EEP_10_2a	11.9	3.9	126.0	0.22	0.33
EEP_10_3	15.9	3.4	154.0	0.21	0.28
EEP_10_2(M27)	8.8	4.3	124.0	0.24	0.31
EEP_10_3(M27)	14.1	3.5	140.0	0.20	0.24
FS1	2.9	7.5	65.0	0.72	0.59
FS4	5.4	5.3	69.0	0.44	0.28

Table 6: Results from the Beam Analysis for $L_b = 50h_b$

Test ID	ϕ (mrad)	ς	Failure conditions $\left(\phi = \phi_{M_{j.max}}\right)$		
	($\varsigma = 2.5$)	($\phi = 30$ mrad)	q_u (kN/m)	$M_j/M_{pl.beam}$	$M_{mid-sp}/M_{pl.beam}$
F2EP_15_1	12.3	3.6	11.3	0.23	0.23
F2EP_15_2	23.7	2.8	13.5	0.19	0.18
EEP_10_2a	19.6	3.1	15.3	0.22	0.20
EEP_10_3	24.3	2.7	19.5	0.21	0.17
EEP_10_2(M27)	13.2	3.4	15.8	0.24	0.19
EEP_10_3(M27)	21.9	2.8	18.1	0.20	0.15
FS1	2.9	7.1	9.2	0.72	0.45
FS4	5.4	5.1	10.5	0.44	0.25
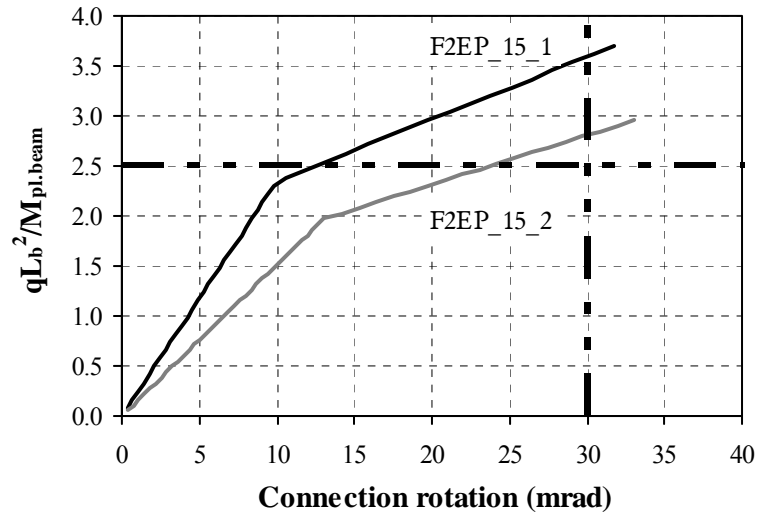
(a) Specimens F2EP
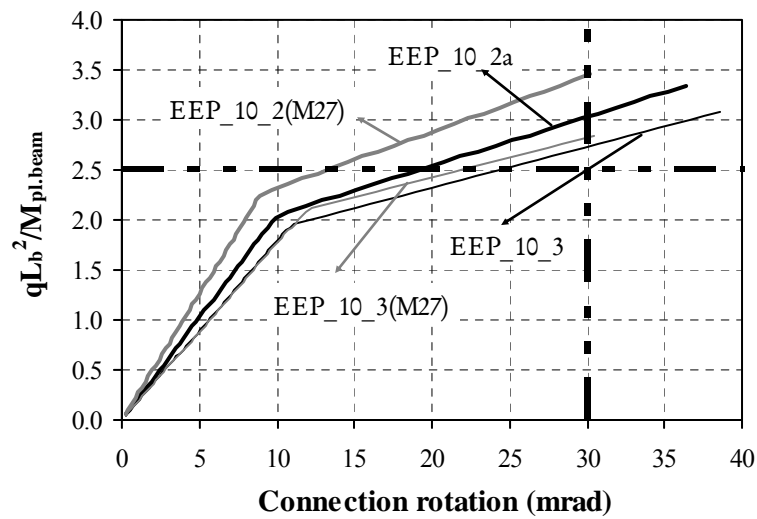
(b) Specimens EEP
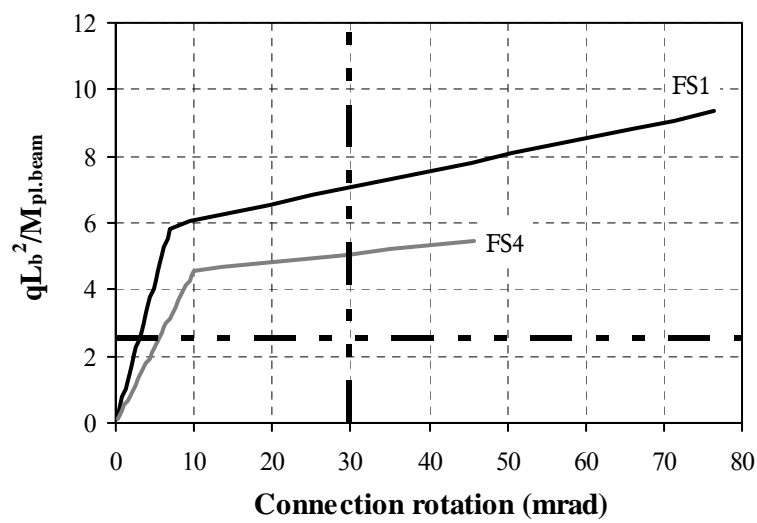
(c) Specimens FS1 and FS4

Figure 10. Behaviour "Available Rotation Capacity" vs. "Ratio $qL_b^2/M_{pl.beam}$" ($L_b = 50h_b$)

The ratio $\varsigma = qL_b^2/M_{pl.beam}$ gives an idea of the degree of utilization of the beam. The graphs in Figure 10 highlight the demand for higher rotations in higher steel grades for the same value of ς. Tables 5 and 6 that compare the joint rotation for such a ratio of 2.5 (second column) clearly show this tendency. If now the results are compared for a specific rotation of 30 mrad, for example (third column on Tables 5 and 6) the values of the above ratio decrease with the end plate yield stress. The failure conditions, for $\phi = \phi_{Mj.max}$, are also characterized, in terms of maximum load, q_u, the bending moment acting (i) at the joints M_j and (ii) at mid-span $M_{mid\text{-}sp}$, computed with reference to the plastic moment of the beam section. The ratio $M_j/M_{pl.beam}$ corresponds to the degree of joint partial-strength. As expected, the maximum load increases when higher stress steel grades are employed. But, the degree of joint partial-strength decreases and the beam efficiency reduces. Finally, Table 7 compares the increase in steel grade for the various specimens and the corresponding increase in load capacity. This correspondence is not comparable. Taking specimen EEP_10_2(M27) (S690) and EEP_10_3(M27) (S960) as example, the improvement in steel grade is 40% whilst the maximum load only increases 10%.
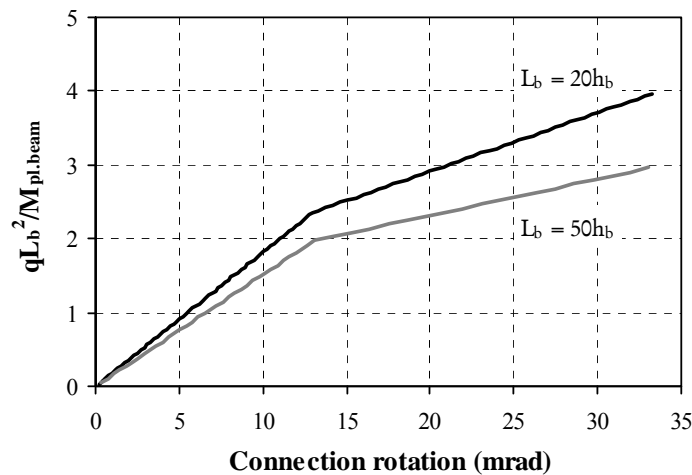

Figure 11. F2EP_15_2: "Behaviour Available Rotation Capacity" vs. "Ratio $qL_b^2/M_{pl.beam}$" for Different Beam Spans

Table 7. Computation of the Increase in Stele Grade and Corresponding Increase in Maximum Load

Test ID	$f_{y.ep.nom}$ (MPa)	$M_{pl.beam}$ (kNm)	Increase in steel grade	q_u (kN/m)	Increase in max. load
F2EP_15_1	460	749	1.5	11.3	1.2
F2EP_15_2	690	1123		13.5	
EEP_10_2a	690	1123	1.4	15.3	1.3
EEP_10_3	960	1563		19.5	
EEP_10_2(M27)	690	1123	1.4	15.8	1.1
EEP_10_3(M27)	960	1563		18.1	
FS1	355	223	1.9	9.2	1.1
FS4	690	433		10.5	

5. CONCLUSIONS

The primary goals of this research work were (i) the experimental characterization, (ii) the validation of current Eurocode 3 specifications for mild steels and (iii) the deformation analysis of HPS moment connections.

Comparison of test results with EN 1993-1-8 procedures for the design of joints within the semi-continuous/partially-restrained concept shows that (i) the T-stub idealization of the tension zone of moment connections adopted in EN 1993-1-8 gives accurate results in terms of prediction of the design resistance even when HPS grades are employed, (ii) the stiffness properties are overestimated and (iii) the guidelines for verification of sufficient rotation capacity can be quite conservative in some cases (Tables 3 and 4).

It was found out that the rotation capacity of the connection specimens using HPS satisfy high deformation demands. Plastic rotation capacities of 30 mrad (and above) were achieved with thinner end plates and 15 mm flush end plates. However, a simple beam analysis with actual joint behaviour has shown that (i) the efficiency of HPS moment connections has no correspondence to the improvement in steel quality and (ii) the deformation demands of these connections are higher than for mild steel grades. Both aspects must further be looked into in order to establish accurate ductility and deformation requirements that allow the designer to take full advantage in the use of this new family of constructional steels. The next logical step forward to this investigation is the numerical implementation of these joint configurations to propose simple expressions for evaluation of the rotation capacity based on geometrical and mechanical parameters.

ACKNOWLEDGMENTS

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NOMENCLATURE

Lower cases

a_w	Throat thickness of a fillet weld
f_u	Ultimate or tensile stress
f_y	Yield stress
h	Depth
h_i	Distance of bolt row i to the centre of compression
k_e	Stiffness of an active joint component
k_{ec}	Stiffness of the assembly of components in the compression/shear zone
k_{et}	Equivalent stiffness of the assembly of components in the tension zone
n	Number of bolt rows
q	Load
t	Thickness
z	Lever arm

Upper cases

E	Young modulus
$F_{ti.R}$	Potential resistance of bolt row i in the tension zone at plastic conditions
L	Length
M	Bending moment
$M_{j.R}$	Joint flexural plastic resistance
$M_{j.max}$	Maximum bending moment
$M_{\phi C}$	Bending moment at fracture of the joint
$S_{j.ini}$	Initial rotational stiffness of a joint
$S_{j.p-l}$	Post-yield rotational stiffness of a joint
W_{pl}	Plastic cross-section modulus

Greek letters

ε_f	Strain at rupture load
ϕ	Connection rotational deformation
ϕ_b	Bolt diameter
ϕ_C	Rotation capacity of a connection
$\phi_{Mj.max}$	Rotation of the connection at maximum load
$\phi_{Mj.R}$	Connection rotation "analytical" value at which the moment resistance first reaches $M_{j.R}$
ϕ_p	Plastic rotation of a connection
ϕ_y	First yielding connection rotation
ϑ_j	Joint ductility index
$\vartheta_{j.max\ load}$	Joint ductility index evaluated for the rotation at maximum load
ρ_y	Yield ratio
ς	Ratio $qL_b^2/M_{pl.beam}$

Subscripts

b	Beam; bolt
d	Design value
ep	End plate
exp	Experimental
EC3	Eurocode 3
j	Joint
max	Maximum
mid-sp	Mid-span
nom	Nominal value
pl	Pure plastic conditions
u	Ultimate conditions
y	Yield