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Evaluation of Innovative Extended End-Plate Moment Connections Under Cyclic Loading

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Abstract

This paper presents parts of results of an experimental research project on the behavior of end-plate connections subjected to monotonic and cyclic loading conducted at İstanbul Technical University. This research project consists of standard and innovative end-plate connections in that the short I-shaped element (actually cut from the beam section) was placed between the end plate and the column flange. The connections were designed for one of two limit states. In the first limit state, the end plate was designed to be the weakest element of the component for connections with a thin end plate. In addition, these connections were designed as partial strength connections with a moment capacity that resists less than 80% of the plastic moment capacity of the connected beams. In the second limit state, end-plate thickness was varied in order to minimize possible prying forces and the connection, which was designed as a full strength connection having a design resistance at least equal to that of the connected beam. The maximum moments obtained experimentally were compared with predicted moments. Three end-plate moment connection configurations were classified in one of three categories depending on the amount of inelastic rotation at the connections. Results show clearly that the innovative connections satisfy the basic criteria: sufficient strength, sufficient rotation capacity and adequate stiffness for a moment connection.

Key words: Moment connection, Cyclic loading, Rotation capacity, End-plate thickness, Plastic capacity.

Introduction

The study and use of end plates in moment resisting frames for beam-to-beam splices and beam-to-column connections dates back to the early 1960s. It consists of a plate with bolt holes and shop welded to a beam section. These connections are very economic for several reasons. No field welding is required, allowing construction to be done in cold conditions, and construction time to be reduced. They allow a great variety of structural solutions by properly modifying the connection structural detail. In particular, both rotational stiffness and flexural resistance can be properly balanced by choosing an appropriate number of bolts and their location, an appropriate end-plate thickness and its geometrical configuration and, finally, a stiffening detail for the column panel zone. An extended end-plate connection is termed “extended” because the plate extends above or below the flange that will be in tension under load. In the case of two-side extended end-plate connections, the bolt rows are located outside and inside both beam flanges. These type of connections can be used in frames subjected to both vertical loads and horizontal forces due to wind action or seismic motion. Experimental and analytical studies demonstrate that well-designed end-plate moment connections are excellent for seismic loading.

For seismic design purposes, rigid connections are traditionally used in moment resisting frames. Steel frames with semi-rigid connections were not utilized mainly due to their relative flexibility when compared to rigid forms. While this treatment applies for static conditions, responses under dynamic loading may be substantially different. Due to the period elongation of the frames as well as the higher energy dissipation in the connection, semi-rigid frames may attract lower loads and possess higher damping. Consequently, the displacements associated with semi-rigid connected frames may be lower than those experienced in their rigid connected counterparts. Elnashai (1998) showed that steel frames with semi-rigid connections may achieve adequate response to earthquake loads, even in areas of high seismicity.

The prediction of the plastic rotation capacity of beam-to-column joints is important for designing ductile moment-resisting frames. Experimental and analytical research conducted on bolted moment connections has provided a large database for the design of connections under monotonic loads. Research results on cyclic loads, however, are very limited. The knowledge gap surrounding the evaluation of the plastic rotation of connections is still significant. Many methods for predicting the plastic rotation capacity of connections are availible in technical literature (Mazzolani and Piluso, 1996). Accurate analysis of the connections is difficult due to the number of connection components and their inherent nonlinear behavior. The bolts, welds, beam and column sections, connection geometry and the end plate itself can all have a significant effect on connection performance. Any one of these can cause connection failures and some interactions. The limit states for end-plate moment connections under seismic loads can be summarized as the development of the beam plastic capacity and the needed beam inelastic rotation capacity, and failure of the end plate, the bolts and the welds between the beam and the end plate. Criteria to evaluate these limit states have been developed using a tee-stub analogy. Procedures utilizing yield-line theory have been used to analyze this stub model. Yield-line based studies were performed by Zoetemeijer (1974), Mann and Morris (1979), Morrison *et al.* (1986), Abel *et al.* (1994) and Faella *et al.* (1995).

Different approaches can be used for predicting connection rotational behavior including empirical models, analytical models, mechanical models, finite element models and experimental testing (Faella et al., 2000). The data for setting up empirical relationships for predicting both joint flexural resistance and the corresponding rotational stiffness have been generated through a wide parametric analysis carried out by means of a mechanical model (Faella *et al.*, 1996) based on the component approach recently codified in Eurocode 3 (1993).

In recent years, extensive investigations of the hysteretic responses of connections under cyclic loading have been conducted experimentally by Bernuzzi *et al.* (1996), Bursi and Galvani, (1997) and/or numerically by Kukreti and Biswas (1997) and Bernuzzi *et al.* (1997).

In general, bolted extended end-plate connections are able to dissipate significant amounts of energy, and are suitable for seismic-resistant structures (Ghobarah *et al.*, 1992, Tsai and Popov, 1989). Notwithstanding, it must be pointed out that the performances of any connection typology are strongly affected by the joint components.

This paper partially presents the results of an experimental research project on the behavior of end-plate connections subjected to monotonic and cyclic loading conducted at İstanbul Technical University (Yorgun *et al.*, 2000). This research project consists of standard and innovative end-plate connections where the short I-shaped element (actually cut from the beam section) was placed between the end plate and the column flange. One purpose of this research was to examined the effect of the gap between the end-plate and the column face (filled with the I-shaped element) on the performance of the connection under the cyclic loading. The comparison between the standard and innovative end-plate connection was made using the test results by Yorgun and Bayramoğlu (2001). Results showed that the innovative end-plate connection performed better than the standard one because of the contribution of the short I-shaped element. This study utilized connections having only 15 mm end-plate thickness in the full-scale tests.

Another part of the research project mentioned above was to investigate the influence of end-plate thickness on the behavior of the innovative connections. End-plate thickness was varied being a parameter and the test under cyclic loading was carried out for the connection having a 20 mm end-plate thickness. In this paper the inelastic rotational capabilities and the moment strength of the connections are evaluated, and also innovative extended moment end-plate connections tested are checked against design methods developed for monotonic loading.

Test Program

The experimental work done at the Structural and Earthquake Engineering Laboratory of the İstanbul Technical University for this study involved the full-scale testing of six end-plate connections subjected to monotonic and cyclic loads. The classical monotonic displacement increase test was performed on three different forms of end-plate connections. Each specimen consists of a single column 3000 mm long with a cantilever beam on one side of the column at its midheight. Loads were applied to the beam tip at a distance of 2365 mm from the column face. Cyclic tests were carried out imposing displacement increments established as a function of monotonic tests following the procedure recommended by ECCS Recommendations No. 45 (1986). Considering the load-displacement curves related to the three monotonic tests performed on the specimens, the initial tangent at the origin was evaluated, E_t, and a tangent with a slope of $E_t/10$ was located. The intersection of the two tangents defined the level of F_y, and e_y is the displacement corresponding to that intersection. After the determination of the the yield displacements in the two directions of loading denoted as e_y^+ and e_y^-, the cycles were applied as follows:

(1) One cycles in the (n $e_y^+/4$, n$e_y^-/4$) interval n = 1,..4; and

(2) Three cycles in the (2ne_y^+, 2ne_y^-) interval n = 1,2, ...

This procedure was continued until the failure of the specimen.

The specimens are described by capital letters EP (which describes the extended end plates) and by numbers (which correspond to the gap between the end-plate and the column face and the end-plate thickness, respectively). The specimens were designated as EP-00-15, EP-15-15 and EP-15-20. In the EP-00-15 specimen, there is no gap; the beams were directly connected to the columns using the standard extended end plate. In EP-15-15 and EP-15-20 specimens, the short I-shaped element (actually cut from the beam section) was placed between the end plate and the column flange to evaluate the performance of the connections. The thickness of the panel zone was twice the thickness of the column web and, thus, the behavior of the other components could be observed. Attention was especially directed toward evaluating the end-plate response.

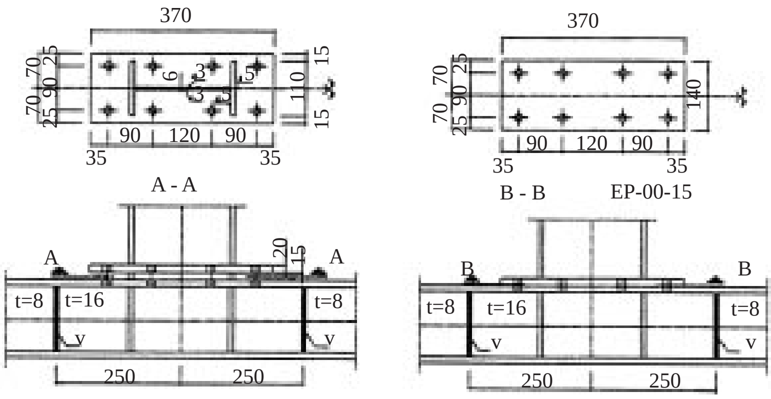

Figure 1. Details of connections.

Table 1. The specimen dimensions.

Location of material	Specimen dimensions [mm]	Yield stress f_y [N/mm^2]	Ultimate stress f_u [N/mm^2]	ε_y (%)	ε_u (%)
Column flange	12 x 160	240	363	0.13	28
Column web	8 x 135	234	351	0.13	29
Panel zone	16 x 135 x 500	263	423	0.12	24
Beam flange	10 x 110	245	355	0.14	25
Beam web	5.5 x 195	300	391	0.13	27
End plate	15 x 140 x 370	273	455	0.13	26
End plate	20 x 140 x 370	295	450	0.14	25

Typical connection details for the test specimens are shown in Figure 1. The specimens are composed of built-up same beam and column sections.

Coupon tests were conducted on samples from the steel sections used. The yield and ultimate strength and the yield and ultimate strain values (f_y ,f_u ,ε_y, and ε_u) and the specimen dimensions are summarized in Table 1. All bolts used were of 16 mm diameter and 10.9 quality.

The test set-up consists of three major components: an actuator to apply the force, a reaction frame supporting the actuator and the laboratory floor supporting the column in the test specimens. The test set-up is shown in Figure 2.

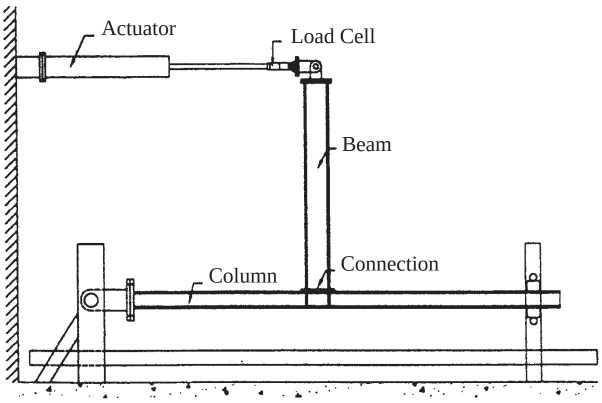

Figure 2. The test set-up.

Lateral supports were provided to prevent out-of-plane instability of the specimen at the end of the beam, where the load was applied. Displacement control was used for the monotonic and cyclic test series. Both monotonic and cyclic loads were applied by a hydraulic actuator whose capacity was +250 kN. The instrumentation consisted of linear variable displacement transducers (LVDTs) to measure displacements at various points and strain gauges to determine yielding on the specimens.

The same cyclic rotation history was used on all testing specimens. After the bolts were tightened, the instrumentation was added and the cycle load was applied. Cyclic loading continued up to the level above the plastic capacity of the connection.

Strength Predictions

The prediction of the behavior of beam-to-column connections can be carried out by means of the component method, which is largely applied in research studies and, recently, in Eurocode 3 (1993). From the theoretical point of view, the component method can be applied to any kind of connection, provided that the basic sources of strength and deformation are properly identified and modeled. These sources are commonly denoted as joint components and can be modeled by means of a force-displacement relationship. In the design of the test specimens, the strength of the panel zone was deliberately changed by altering the panel zone thickness to reduce the risk of shear failure of the panel zone. The connections were designed for one of two limit states. In the first limit state, the end plate was designed to be the weakest element of the component for connections with a thin end plate (t = 15 mm). In addition, these connections were designed as partial strength connections with a moment capacity that resists less than 80% of the plastic moment capacity of the connected beams. In the second limit state, end-plate thickness was varied, being t = 20 mm in order to minimize possible prying forces, and the connection was designed as a full strength connection having a design resistance at least equal to that of the connected beam.

The plastic capacity of the beam $M_{b.Rd}$ was computed from the yield stress of the steel, the plastic section modulus and the length of the cantilever beam. The design of the beam-to-column connections was determined by means of the so-called component method that is applied by Eurocode 3, Annex J (1997). The connection plastic moment capacity, $M_{c.Rd}$ is given by

$$M_{c.Rd} = F_{Rd}(h - t_{fb}) \tag{1}$$

where h is the height of the beam and t_{fb} is the thickness of the beam flange, and F_{Rd} is the design resistance of the weakest joint component. This was computed by considering the following components: column web panel in shear, column web in compression, beam flange in compression, bolts in tension, column web in tension, column flange in tension and end plate in tension.

The strength of the tested connections was mainly governed by the strength of the end plate. Therefore, a different approach was also used for predicting the moment strength of the connections depending on end-plate strength. Yield-line theory was first introduced to analyze reinforced concrete slabs and has more recently been adopted for use in the design of end plates. An plate is assumed to reach failure when the yield lines form a kinematically valid collapse mechanism (Hendrick *et al.*, 1985). The elastic deformations of an end plate are assumed to be neglible in comparison to its plastic deformations. The analysis of a yield-line mechanism can be performed by either the equilibrium method or the virtual work method. The virtual work method is simpler than the equilibrium method. In the virtual work method, the end plate is assumed to rotate about the center of the compression flange of the beam section. The external work, done by rotating the plate through a small arbitrary rotation, is set equal to the internal work, done at the plastic hinges formed over the yield lines, which accommodates the total rotation of the plate. The internal energy stored in a particular yield-line mechanism is the sum of the internal energy stored in each yield line forming the mechanism. The internal energy stored in any given yield line is obtained by multiplying the normal moment on the yield line with the normal rotation of the yield line. Thus, the energy stored, W_{in}, in the n^{th} yield line of length L_n is

$$W_{in} = \int_{L_n} m_p \phi_n ds \qquad (2)$$

The internal energy stored in a yield-line mechanism can be written as

$$W_i = \sum_{n=1}^{N} \int_{L_n} m_p \phi_n ds = \sum_{n=1}^{N} m_p \phi_n L_n \qquad (3)$$

where N is the number of yield lines in the mechanism and m_p is the plastic moment capacity of the end plate. To predict the moment strength of connections research done by Abel *et al.* (1994) was used and maximum moments obtained experimentally were compared with predicted moments as shown in Table 2.

Evaluation of the Experimental Results and the Strength Predictions

In the tests, initially, the connections responded elastically up to a computed yield load of the connection. In the EP-00-15 specimens, some inelastic response was developed while gaps between the end plate near the beam tension flange and the column flange cyclically appeared and disappeared. The maximum load reached in this specimen was 32.8 kN at a beam tip deflection of 80.5 mm. In the EP-15-15 and EP-15-20 specimens, the gap between the free edges of the end-plate and the column flange changed along the free edges in the extended portion of the end plate. While it was 15 mm at the beginning of the test, the gap decreased approximately 8 mm in the corner of the end plate at the last tension cycle. The maximum load reached in the test was 36.5 kN at a beam tip deflection of 85.84 mm for EP-15-15 and 38.3 kN at a beam tip deflection of 93.26 mm for EP-15-20.

The beams did not show any signs of yielding during the tests for all specimens. As a result, the inelasticity was almost entirely concentrated in the end plate. The magnitude of the prying force and the location are greatly dependent on the zones of contact between the column flange and the end plate. Because of the contribution of the short I-shaped element, the free edges of the extended portion of the end plate have more flexibility in the innovative end plate connections. As a result, in these connections, the end-plate contribution to the rotation became greater than the standard end plate. In addition, with an increase end-plate thickness, the contribution of the bolts became more significant. In the tests, the ratio between the flexural resistance of end plate bending and the resistance of the bolts is 60% and 80% for the thin and thick end plates, respectively. The cyclic behavior is strictly dependent on this ratio. In fact, in the case of thin end plate, the hysteretic behavior was mainly governed by the plastic deformations of the end plate in bending. Increasing the end-plate thickness, i.e., its flexural resistance with respect to bolt resistance, the plastic deformations of the bolts were increased. Figures 3(a) and (b) show the condition of the connections after the test.

Using the beam-tip load versus the beam-tip deflection, the flexural ductility ratio was defined to be $\delta_{\max}/\delta_{el}$, where δ_{el} is the elastic deflection determined from the experimental results of the beam and $\delta_{\max}$ is the maximum deflection. The ductility ratios in the connections were 1.78, 2.04 and 2.34 as the average ratios for EP-00-15, EP-15-15 and EP-15-20, respectively (Yorgun *et al.*, 2000).

The rotation of the connection was calculated depending on column- and beam-chord rotations. The instrumentation (LVDTs) used in the tests is shown schematically in Figure 4. Using measured displacements, the column rotation is defined as follows:

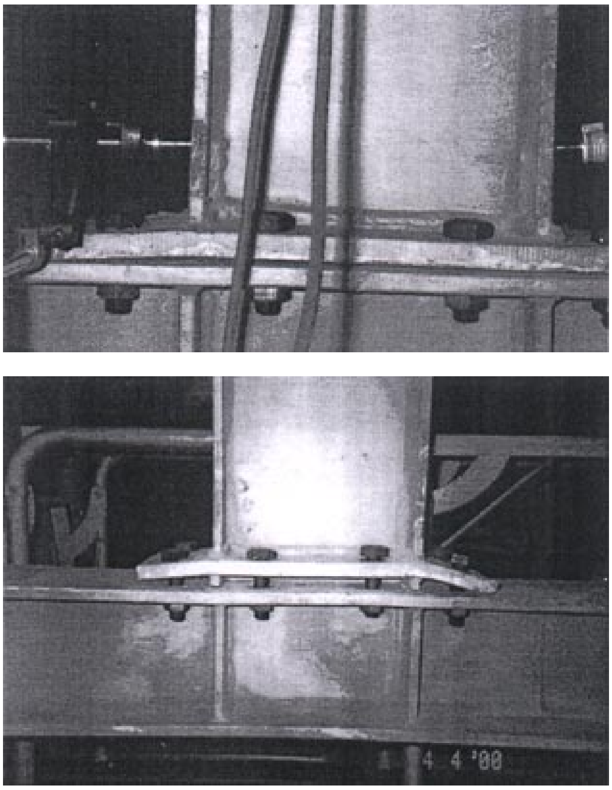

Figure 3. The connections after the test.

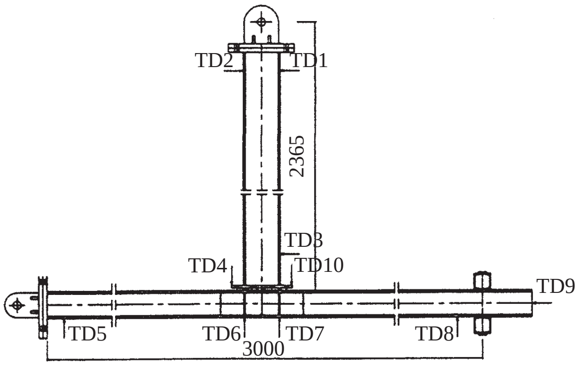

Figure 4. LVDT locations.

$$\phi_{cf} = \frac{\delta_{ct}\text{-}\delta_{cc}}{d_b} \tag{4}$$

where δ_{ct} and δ_{cc} are column flange displacements at beam tension and compression flange levels, respectively, and d_b is the distance between the beam tension and compression flange. The beam chord rotation is defined as

$$\phi_{bc} = \frac{\delta_{be}}{l_b} \tag{5}$$

where δ_{be} and l_b are the vertical displacement at the tip of the beam and the cantilever beam span measured to the face of the column flange, respectively. Using Equations (4) and (5), the rotation of the connection is calculated as follows:

$$\phi = \phi_{bc} - \phi_{cf} \tag{6}$$

Moment-rotation curves for each type of the connections are shown in Figure 5. The applied moment versus the end-plate rotation is shown in Figure 6 for the EP-15-20 connection.

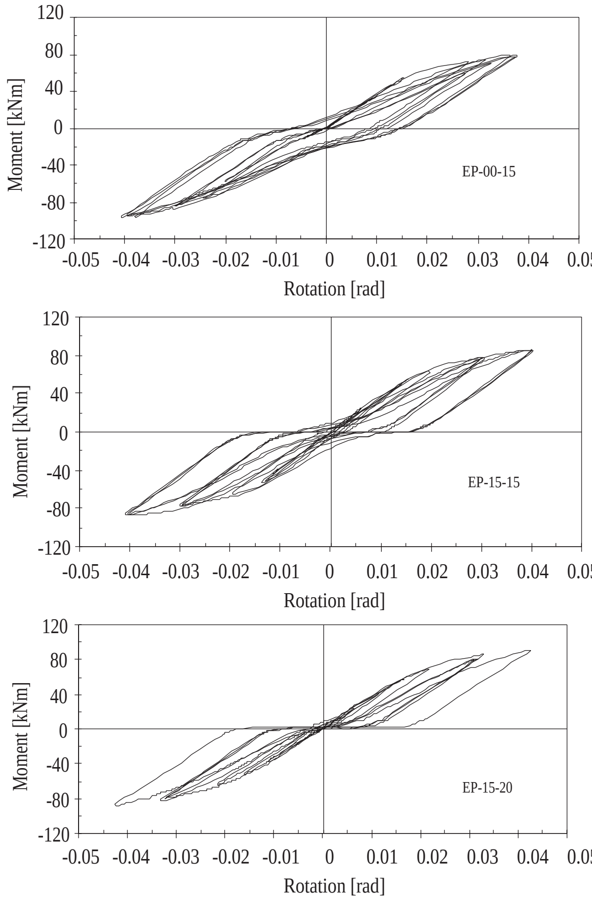

Figure 5. Moment-rotation curves.

The elastic response of the connection throughout loading is represented by a relationship between moment and total rotation at low loads. This relationship is found by taking a linear interpolation of moment versus total rotation data points from test data obtained prior to any yielding in the connection. The elastic response is then extrapolated for all subsequent loads realized throughout the test. Finally, the inelastic rotation is found by subtracting the elastic response from the total rotation.

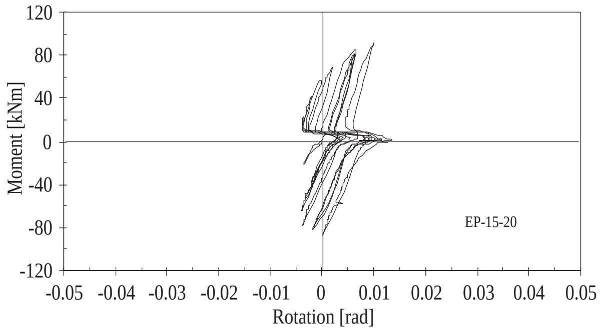

Figure 6. The applied moment versus end-plate rotation.

The rotation of the connections and maximum strength results from these tests are presented in Table 2; where $M_{\max}$ is maximum attained moment at the last cycle, $M_{c.Rd}$ is plastic moment capacity of the connection according to EC3, $M^*_{c.Rd}$ is the predicted plastic moment capacity of the connection according to Abel *et al.* (1994), $M_{b.Rd}$ is plastic moment capacity of the beam, ϕ_p is the total maximum rotation of the connection at the last cycle and ϕ_{pi} is the inelastic rotation of the connection.

The maximum moment strength of the connections is much greater than the strength predictions from the design method of Eurocode 3 and yield-line theory (Abel *et al.*, 1994). Finally, the design methods of Eurocode 3 used to predict connection strength are shown to be conservative when compared with the experimental strength for the innovative end-plate connections.

Additionally, there are three types depending on the amount of inelastic rotation of the connections of seismic resisting frames: ordinary moment frames, intermediate moment frames, and special moment frames. The most restrictive classification, special moment frames, requires 0.03 radians of inelastic rotation from the connections. The intermediate moment frame classification applies to connections that can occur between 0.02 radians and 0.03 radians rotation. Finally, the ordinary moment frame classification applies to connections that can incur between 0.01 and 0.02 radians rotation (Seismic, 1997). According to the inelastic rotation exhibited by the connections, all connections tested can be qualified in the intermediate moment frame classification.

The ductility of seismic-resistant steel frames is deeply influenced by the behavior of their connections. Two different approaches can be taken in the design of beam-column connections of seismic-resistant steel frames. The first is based on the location of the dissipative zones at the beam ends. The second approach relies on the energy dissipation through the cyclic plastic bending on the connections. It is clear that in the first case connections should possess sufficient strength so that plastic hinges can be formed at the beam ends providing ductility on the frame. In the second case, the key parameters of the connection behavior are their ductility and energy dissipation capacity under cyclic loading. According to current codes for moment resisting frames, beam-to-column connections should be designed to develop at least the bending strength of the connected members, or should have sufficient ductility. Traditionally, the ductility of a steel moment connection is measured by cyclic moment rotation tests. The purpose of this experimental study is to provide additional information on the effect of the local geometric details on the behavior of steel moment connections. The geometric details considered include the thickness of the end plate and the gap between the end plate and column face. This study is limited by the effect of these parameters on the inelastic behavior of tested connections. The modern design procedure of steel structures in seismic zones is based upon the concept of energy dissipation. The extended end-plate connections tested failed to achieved the inelastic rotation needed for the connection to be used in a moment frame designed for seismic loading.

Table 2. Comparison of moment resistance and rotation capacity.

Connection	M_{max} (kNm)	$M_{max}/M_{c.Rd}$	$M_{max}/M^*_{c.Rd}$	$M_{max}/M_{b.Rd}$	Maximum Rotation	
					ϕ_p	ϕ_{pi}
EP-00-15	77.57	1.26	1.83	1.05	0.0376	0.0226
EP-15-15	84.43	1.37	1.98	1.14	0.0401	0.0249
EP-15-20	90.34	1.47	1.20	1.22	0.0426	0.0292

Classification by rigidity leads to the three main categories of nominally pinned, rigid and semi-rigid connections. The first two categories are traditional. Rigid connections transmit all end reactions, and their deformation is sufficiently small that their influence on the moment distribution in the structure or on its overall deformation may be ignored. Semi-rigid connections are designed to provide a predictable degree of interaction between members, based on the design moment-rotation characteristics of the joints. The design of the structure has to be based on actual load versus the deformation characteristics of the joints. Nominally pinned connections are widely used when the lateral stiffness of the structure is guaranteed by appropriate bracing systems. For moment-resisting frames rigid connections often lead to relatively expensive constructional details. The intermediate third category has been introduced to fill the gap between pinned and rigid connections and is accepted in the updated codes (i.e. Eurocode 3). In order to include semi-rigid joints effect in frame design, it is necessary to present the $(M - \phi)$ curve of the connection by means of experimental data or theoretical prediction methods (Mazzolani and Piluso, 1996).

Beam-to-column connections can be classified on the basis of rotational stiffness and flexible resistance. According to Eurocode 3, the boundary curves of the classification diagram are expressed through the non-dimensional parameters

$$K = \frac{K_i L}{E I_b} \tag{7}$$

$$\overline{m} = \frac{M_u}{M_{b.Rd}} \tag{8}$$

$$\overline{\phi} = \phi \frac{E I_b}{M_{b.Rd} L} \tag{9}$$

where K_i is the initial rotational stiffness of the connection, and $M_{b.Rd}$, I_b and L are, respectively, the plastic moment, the moment of inertia and the length of the connected beam. With reference to the rotational stiffness, beam-to-column connections can be classified as

- Nominally pinned, for $K \leq 0.5$
- Semi-rigid, for $0.5 < K < K^*$
- Rigid, for $K \geq K^*$

The value of K^* depends on the type of frame, braced or unbraced, assuming a value of 8 in the first case and of 25 in the second case.

Even though the semi-rigidity concept has been introduced many years ago, steel structures are usually designed by assuming that beam-to-column joints are either pinned or rigid. This design assumption allows a great simplification in structural analysis, but it neglects the true behavior of joints. The rotational behavior and degree of semi-rigidity of extended end-plate connections is influenced by the spacing between the bolts and the beam section, the end-plate thickness, the column flange thickness and the stiffened joints. Since extended end-plate joints are traditionally considered ‘rigid’, frames accepted into past on the basis of a rigid frame analysis would be rejected now if joint action is incorporated in the design analysis.

Due to the semi-rigidity of the joints, the horizontal deflections will then be larger than the frame with rigid joints. In order to fulfil the serviceability limit state requirements, the decrease in joint stiffness can be compensated for by an increase in column sizes or the stiffened joints. In other words, frames can be sized considering the influence of the joint semi-rigidity in all steps of design process. In order to provide detailed information on the seismic response parameters of semi-rigid frames in comparison with rigid (i.e. fully welded alternatives) experimental work should be undertaken on steel frames. These studies consist of different earthquake records for different structures (i.e. different plastic moment of the beam and the length of the connected beam).

Conclusions

Bolted end-plate connections are widely used in constructional steel design and end-plate beam-column connections provide a viable alternative for moment-resisting frames in seismic design. According to many fabricators, end-plate connections do not have sufficient erection tolerances. Therefore, the beam is fabricated shorter and the gap between the end plate and the column face is made in order to fit the beam to the distance between the supporting columns. In the present paper, the effect of the gap between the end plate and the column face (filled with an I-shaped element, i.e. beam section) on the performance of a connection subjected to cyclic loading has been examined. The comparison between standard end-plate connections and innovative end-plate connections was performed using experimental results and design methods.

There was no deformation in the I-shaped element placed between the end plate and the column flange and it remained elastic. Hysteretic loops for specimens exhibited stable characteristics, but the specimens did not develop the same level ductility ratios.

The primary purpose of the testing process was to investigate the influence of end-plate thickness and gap (filled an with I-shaped element, i.e. beam section) and to check this against design criteria developed for monotonic loading. Predicted capacities obtained using the component method codified in Eurocode 3 were compared to data obtained experimentally. The maximum moment strengths of the connections were much greater than the strength predictions from the design method of Eurocode 3. Finally, the design method used to predict connection strength was shown to be conservative when compared with the experimental strength for the innovative end-plate connections.

The beams did not show any signs of yielding during the tests for all specimens. As a result, the inelasticity was almost entirely concentrated in the end plate. The comparisons indicate that the gap (filled with an I-shaped element, i.e. beam section) and end-plate thickness influence the ductility of the connection. In the innovative connections, the end-plate contribution to the rotation became greater than the standard end plate. For a thin end plate, the hysteretic behavior was mainly governed by the plastic deformations of the end plate in bending. After increasing the end-plate thickness, i.e., its flexural resistance with respect to bolt resistance, the contribution of the bolts became more significant and the plastic deformations of the bolts increased. According to the inelastic rotation exhibited by the connections, all connections tested can be qualified in the intermediate moment frame classification.

Nomenclature

d_b	distance between the beam tension and compression flange
e_y	positive or negative yield displacement
E_t	initial tangent at the origin
f_y	yield stress
f_u	ultimate stress
F_y	yield load in the positive or negative force range
F_{Rd}	design resistance of the weakest joint component
l_b	cantilever beam span measured to the face of the column flange
L	length of the connected beam
K_i	initial rotational stiffness of the connection
m_p	plastic moment capacity of the end plate
M_{max}	maximum attained moment at the last cycle
$M_{c.Rd}$	plastic moment capacity of the connection according to Eurocode 3
$M*_{c.Rd}$	predicted plastic moment capacity of the connection according to Abel
$M_{b.Rd}$	plastic moment capacity of the beam
$\boldsymbol{N}$	number of yield lines in the mechanism
t_{fb}	thickness of the beam flange
W_{in}	the energy stored
ε_y	yield-strain value
ε_u	ultimate strain value
δ_{el}	elastic deflection determined from the experimental results of the beam δ_{max} maximum deflection
δ_{ct}	column-flange displacements at beam-tension flange level
δ_{cc}	column-flange displacements at beam-compression flange level
δ_{be}	vertical displacement at the tip of the beam
ϕ	rotation of the connection
ϕ_{bc}	rotation of the beam
ϕ_{cf}	rotation of the column
ϕ_p	inelastic rotation of the connection

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