Steel and Composite Structures, Vol. 1, No. 4 (2001) 441-458
DOI: http://dx.doi.org/10.12989/scs.2001.1.4.441

Flexural behavior of partially-restrained semirigid steel connections

G. Campione† and N. Scibilia‡

Dipartimento di Ingegneria Strutturale e Geotecnica, Università di Palermo, Viale delle Scienze-I-90128, Italy

Abstract. We analyzed the experimental and theoretical behavior of a particular type of steel joint designed to connect beam to beam and able to transfer both shear forces and bending moments. This joint is characterized by the use of steel plates and bolts enclosed in the width of the beams. The experimental investigation was carried out characterizing the constituent materials and testing in flexure beams constituted by two portions of beams connected in the middle with the joint proposed. Connections having different characteristics in terms of thickness of plates, number and type of bolts were utilized. Flexure tests allow one to determine the load-deflection curves of the beam tested and the moment-rotation diagrams of the connections, highlighting the strength and the strain capacity of the joints. The proposed analytical model allows one to determine the moment-rotation relationship of the connections, pointing out the influence of the principal geometrical and mechanic characteristics of single constituents on the full properties of the joint.

Key words: experimental testing; semirigid connections; monotonic behavior; failure mode; beam-beam joints; load-deflection curves; moment-rotation curves; analytical model.

1. Introduction

In the design of steel structures, for accurate prediction of the global response and of the bearing capacity of structures, knowledge of the behavior of the connection under both monotonic and cyclic actions is required, especially if the structures have to be designed in seismic areas (Mazzolani and Piluso 1996, De Stefano *et al*. 1994).

Taking into account the flexibility of the semirigid connections (Attiogbe and Morris 1991, Chisal 1999) a nonlinear behavior of the structures is expected even when members behave elastically. By considering this effect it is possible to determine the influence of the joint characteristics on the distribution of internal forces and on the stability of compressed members and on the lateral displacements of structures loaded by vertical and lateral forces.

Knowledge of moment-rotation relationships of connections, generally utilized to characterize structural behavior of joints, is very relevant for the study of the serviceability and ultimate state of the structures. For service load conditions the effective compliance of the joints leads to lower stiffness of the structures, associated with a decrease in the natural frequency of the structural system, generally beneficial, but also to greater sensitivity to second-order effects. In the ultimate state, partially or fully restrained connections limit the rotational capacity and reduce the behavior factor of structures.

†Assistant Professor
‡Associate Professor

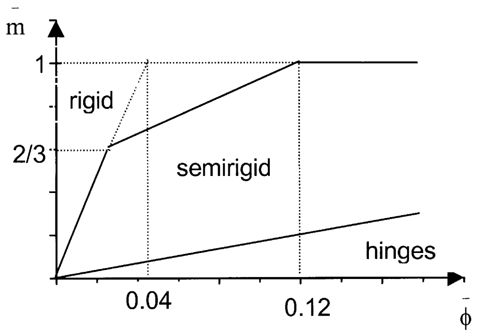

Fig. 1 Dimensionless m-ϕ diagram according to EC3 for unbraced frames

It is interesting to observe that in the simple elastic analysis of steel frames, joints are assumed to be nominally pinned or rigid, permitting a remarkable simplification in the design procedures of joints. Numerous and well-known experimental (Astaneh *et al*. 1991, Adany *et al*. 2001) and analytical (Astaneh *et al*. 1993, Faella *et al*. 1997) investigations have shown that the majority of joint types are semirigid and are characterized by non linear behavior even in service conditions with stiffness between that of the case of a hinge and that of a rigid connection.

Several International codes (AISC 1994, LRDF 1994), allow one to consider the nonlinear behavior of connections in the analysis of steel structures. Three types of connections are also considered defined as: - rigid connections maintaining the angle formed between members; - simple connections capable of transfering only shear or axial forces; - semirigid connections also capable of transfering bending moment, but allowing relative rotations between connected members.

The European Code for steel structures (Eurocodice 3 1994), considers semirigid connections in design, and defines a nominally pinned connection as a connection having initial rotational stiffness S_j and plastic resistance moment lower than 0.25 of the rotational stiffness and of the plastic moment $M_{pl,Rd}$ of the members connected, respectively.

EC3 also classifies connections as rigid or semirigid depending on the resistance and rotational capacities according to the diagram in Fig. 1, distinguished for braced and unbraced frames. In this diagram in the abscissa there is the dimentionless relative rotation of the joints $\overline{\phi} = (E \cdot I_b \cdot \phi)/(L_b \cdot M_{pl,Rd})$ and in the ordinate are the dimentionless moment $\overline{m} = M/M_{pl,Rd}$, E being the modulus of elasticity of steel, L_b and I_b the length and the moment of inertia of the member connected and ϕ the relative rotation of the joint. This code does not prescribe a specific control of ductility for over-strong joints if the plastic moment of the connection is higher than 20% of the plastic moment of the members connected.

Semirigid connections are easier to realize with respect to rigid joints and also they are more commonly utilized in Europe where the trend is to use bolted connections for the assemblage of members.

End plate connections and top and bottom end plates with double web plates or top and bottom ones with double web angle connections are some of the most commonly utilized connections. The use of these different systems of connection also depends on the strength and strain capacities required in relation to the characteristics of the members connected.

It is also possible to fully weld a piece of beam to the column, translating the connection to the beam, obtaining the advantage of reducing the hazard of failure on the fully restrained welded joint. Recent observations from damage occurring in steel structures in destructive earthquakes (Tremblay *et al*.

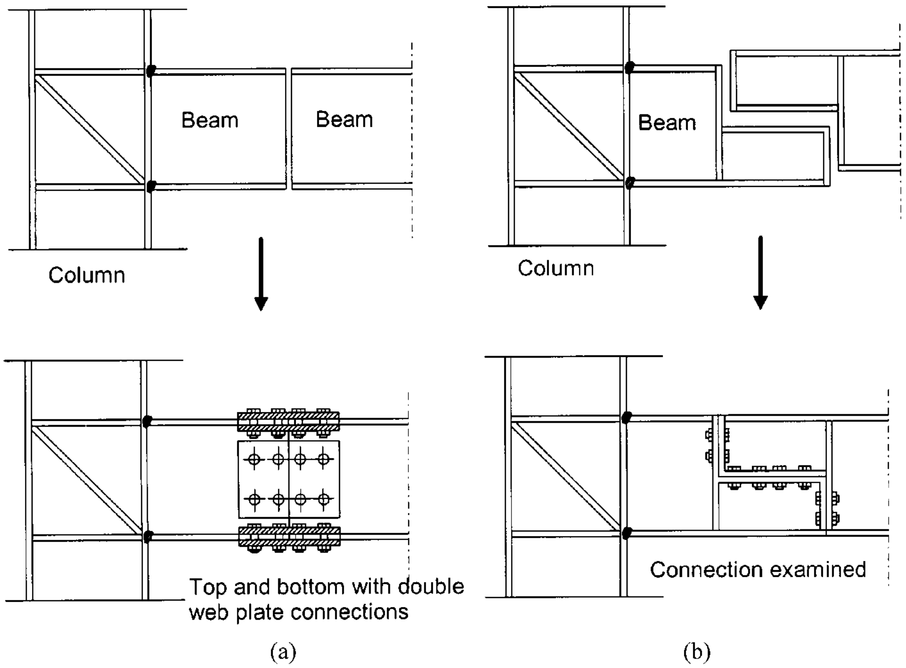

Fig. 2 Beam to beam connections

1995) have shown that very often joints fail in a brittle way on the welds. The beam to beam connection shown in Fig. 2(a) partially solved this problem, because it transfers the connection into the beam, but the joint has dimensions higher than the height of the beams. A good alternative to this connecting system is that shown in Fig. 2(b) recently designed by the authors (Campione *et al*. 2001). It consists in a beam to beam connection characterized by the use of vertical and horizontal flanges welded to the beam and bolted together and having all components enclosed in the height of the beam. With the right number and type of bolts and thickness of flanges it is possible to obtain a fully restrained joint, ensuring at the same time high rotational capacity and high initial stiffness. This kind of joint also offers the advantage of constituting provisional support for the beam before final connection with bolts.

2. Aim of the research

The aim of the experimental research was the study of a particular type of steel joint for beam to beam connection and it is an alternative to the top and bottom connection with double web flanges. This joint is characterized by high strength and rotational capacity and by the presence of components (bolts and flanges) enclosed in the height of the member.

The study lies in the field of semirigid connections and it was organized in the following steps:

- experimental investigation on real scale beams connected with the joints mentioned before, considering different design parameters;
- mechanical interpretation of the flexural behavior of the joints and identification of fundamental parameters (number and type of bolts, thickness of flanges, etc.);

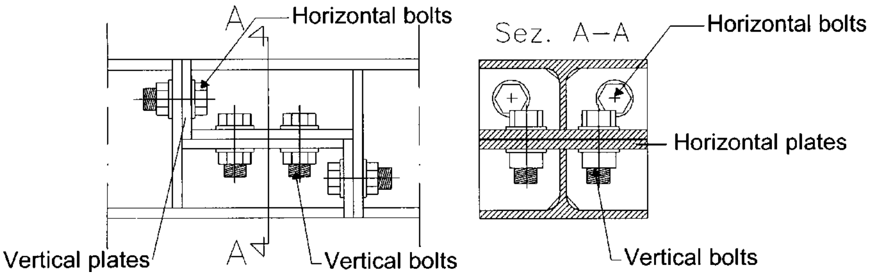

Fig. 3 Joint examined

- analytical formulation of moment-rotation relationship based on initial stiffness, yielding and ultimate moment;
- comparison between analytical end experimental results and validation of results.

3. Experimental investigation

The experimental research was carried out on fourteen real scale beams of type HEA 140 connected in the middle with a joint constituted by vertical and horizontal steel flanges welded to the beams and connected together with horizontal and vertical bolts, as shown in Fig. 3.

The profile HEA140 had the following geometrical characteristics: area $A = 314$ cm^2, moment of inertia $I_b = 1033$ cm^4, height $h = 133$ mm, base $b = 140$ mm, web thickness 5.5 mm and flange thickness 8.5 mm.

3.1. Specimen characteristics

Three type of joints, shown in Fig. 4, different for type and number of bolts and for thickness of flanges, were analyzed. The geometrical characteristics and details of components utilized are given in Table 1. All connections were characterized by the use of vertical flanges having thickness t equal to 8 and 15 mm respectively and horizontal flanges having thickness 8 mm. Two couples of horizontal bolts and vertical bolts (placed on the middle of the height of the beams) were utilized. The number of vertical bolts (type 8.8) was between two and four for each side. Two bolts were utilized in joint type A, three in joint type B and four in joint type C. The geometrical details and design rules (distance between single bolt, between bolt and free end of the flanges etc.) were in accordance with Eurocode 3.

The bolts had nominal diameter 18 mm, length 45 mm and grade 8.8 and 10.9, in accordance with EC3. Direct tensile tests carried out on steel specimens having 8 mm thickness gave yielding stress $f_y = 288$ MPa ultimate stress $f_u = 451$ MPa and maximum strain at failure $\varepsilon_u = 32\%$. According to the Eurocode 3 classification, the steel is type E275.

Direct tensile tests carried out on single bolts only of type 8.8 and 10.9 gave respectively yielding stress $f_{yb} = 660$, and 920 MPa and ultimate strength $f_{ub} = 840$, 1040 MPa, with strain at rupture ε_{ub} equal to 3% approximately. These values are in agreement with those given in the literature (Bernuzzi *et al*. 1991, Wheeler *et al*. 1998). Although the bolts have high strength and according to EC3 they can be utilized as friction type, they were installed tightening the bolts by hand and than, only the bearing

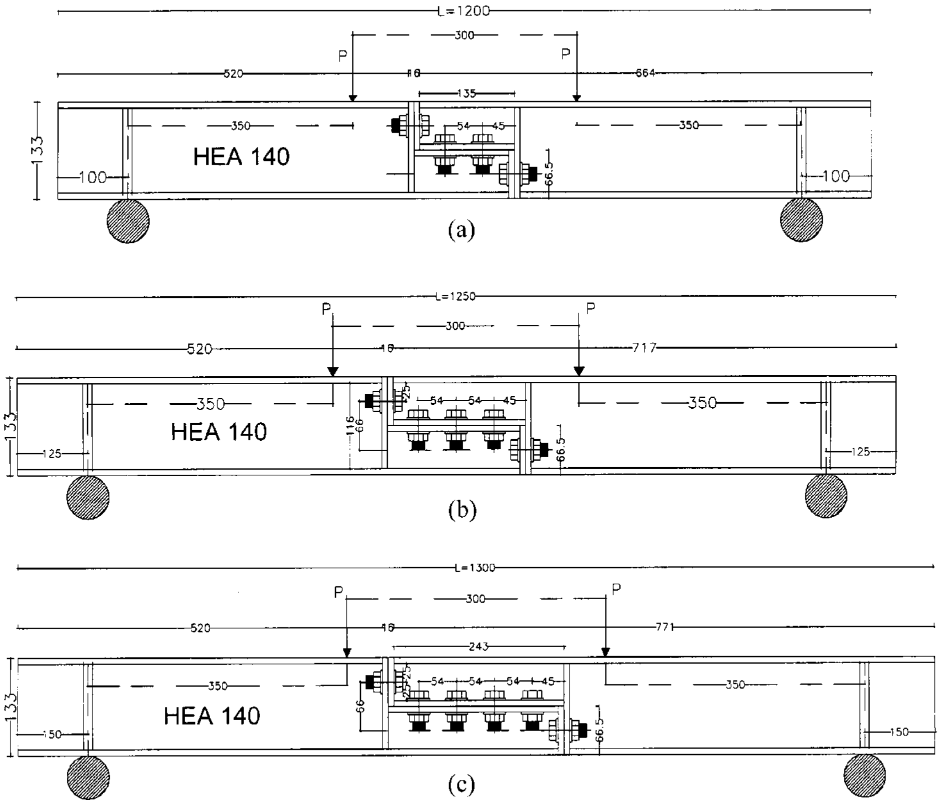

Fig. 4 Types of beams tested

Table 1 Geometrical characteristics of specimen

Type	Thickness of vertical flanges (mm)	$N°$ horizontal bolts	$N°$ vertical bolts	Grade of horizontal bolts
A1	8	4	/	/
A2	8	4	4	10.9
A12	15	4	4	8.8
A13	15	4	4	10.9
B1	8	4	6	8.8
C1	8	4	8	8.8

action is considered.

All welds were executed with materials having the same characteristics as the flange materials and no defects were observed after welding. In any case no ruptures were observed in welding during the tests.

3.2. Test set-up

Tests were carried out on simply supported beams loaded in four-point bending tests, two loads being applied which induced a constant moment without shear in the central part of the beam. Fig. 5 shows

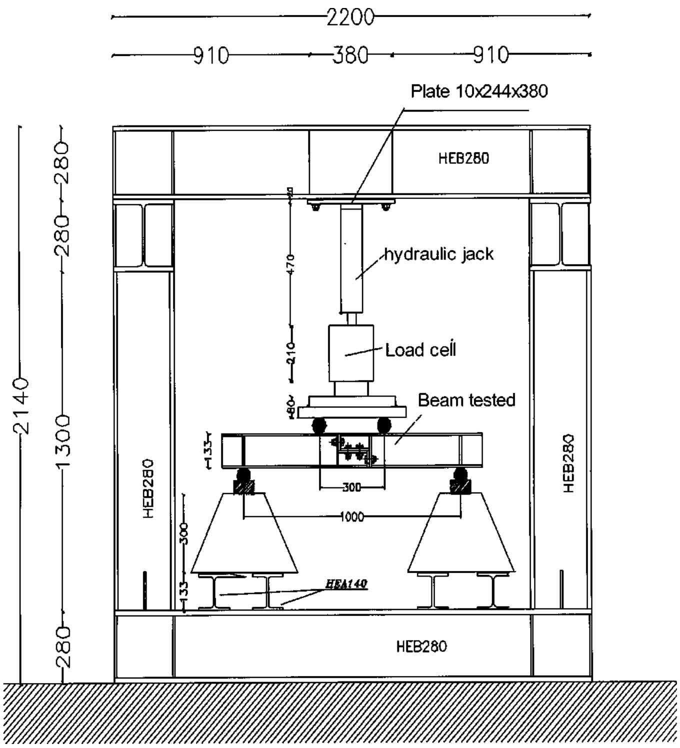

Fig. 5 Test set-up

the test set-up utilized to perform tests.

A stiff steel frame was utilized to load beams in flexure. Loads were applied locally at two points distant 300 mm by means of steel cylinders having 50 mm diameter and interposed between the beam tested and a rigid beam chosen to transfer the load from the load cell to the beam. The load was applied through a hydraulic jack having 400 kN bearing capacity and was recorded by means of a load cell with digital recording.

Beams were strengthened locally in the sections near the cylindrical supports by means of steel web stiffeners to prevent local buckling.

Deformations were recorded by using electronic transducers placed on the beams as shown in Fig. 6. Five transducers were placed with a vertical axis to record deflections of the beams, and displacements in loaded and supported sections. Two further transducers were placed with a horizontal axis in the middle part of the beam and near to the connection to record deformation at the top and bottom of the beam.

Unfortunately no strain gauges were placed on the steel plates and for this reason no accurate measurement of yielding stresses was possible. Fig. 7 shows a detail of the joints with transducers placed for recording of displacements. From the same figure there emerges the position of the horizontal and vertical bolts.

Although the primary objective of the paper is to investigate the flexural behavior of the proposed connection, and with the chosen test set-up in the central part of the beam there is not shear, it is very

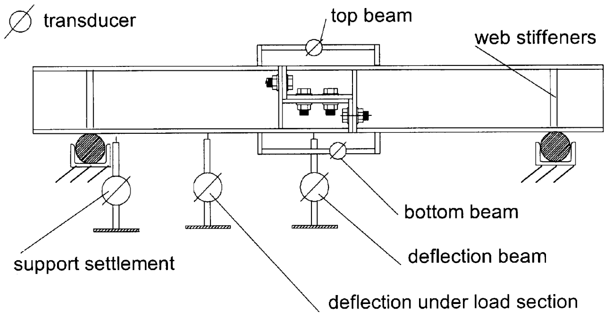

Fig. 6 Test set-up for deformation recording

important to estimate the shear resistance of the connection if it has to be utilized in locations where shear is high.

It is interesting to observe that in the presence of a shear action *V*, because of the particular arrangements of the connection, only half part of each beam web connected can support the shear force; consequently, depending on the geometrical characteristics of the beam (thickness and height of the beam), on the load condition and on the span of the beam, shear failure can be achieved. This kind of failure can be avoided adopting adequate web thickness of the beam or if the beam has an adequate length. For example, if a beam of length *L* is connected with the proposed joint at distance h from the end portion and is loaded in the mid-point through a vertical load *P*, it is possible to determine the critical length L_c. This length is obtained by posing that the load producing shear failure in the web of the beam is equal to the load producing flexural failure of the flanges. In the particular case above mentioned is $L_c = 8\sqrt{3}Z/(h\ t_w)$, Z being the plastic modulus of the section and t_w the web thickness. For effective length of the beam lower than L_c shear failure is expected, but adopting an effective length higher or an adequate web thickness shear failure can be avoided.

Fig. 7 Particular of joint with transducers

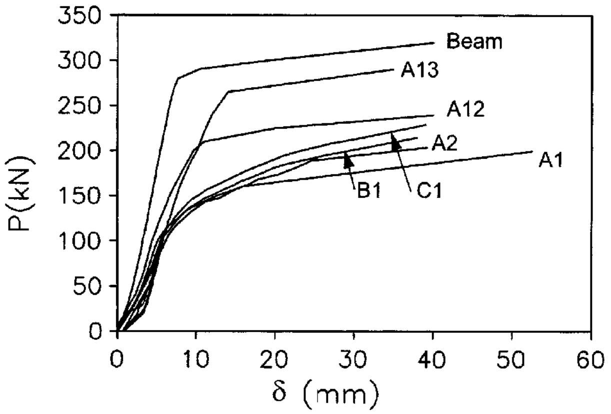

Fig. 8 Load-deflecion curves of tested beams

4. Experimental results

Fig. 8 shows the load-deflection (P-δ) curves for the different specimens tested. The loads, as already mentioned in the previous section, were recorded by means of a load cell, while displacements were recorded by means of transducers, purged of extraneous deformations. The same figure shows the load-deflection curves of the beam without connections.

The trend of the curves shows that all beams initially behave elastically, and then inelastic deformations occur until the yielding occurs. After this load, the stiffness of the beam decreases drastically and strain-hardening behavior is observed up to the rupture of some components (bolts of flanges) and also prying action on the bolts was observed. After this stage a dramatic loss of bearing capacity is observed. No further information about the softening response of the joints could be obtained because load control tests were performed.

From the shape of the curves it emerges that connections of types A1, A2, B1 and C1 have almost the same flexural strength and initial tangent stiffness. Ultimate moment of the connection is almost equal to 60% of the ultimate moment of the beam and the moment-rotation relationship is deeply nonlinear before the maximum strength is reached.

It was also observed that connection behavior is slightly influenced by the number of vertical bolts placed on horizontal flanges and the strength depends essentially on the characteristics of the bottom vertical flanges (for positive moment) and of the corresponding bolts. In any case the connection behaves in a ductile manner.

In joints of type A1 failure is governed by complete plasticization of vertical flanges until the maximum strain capacity of materials is reached, as shown in Fig. 9(a). Instead in the case of connections of type B1 failure is governed by horizontal bolt rupture, and also yielding of vertical flanges was observed, as shown in Fig. 9(b).

Better behavior was observed in connections of type A12 and A13, in which, due to the higher thickness of the vertical flanges (15 mm instead of 8 m), and due to the higher number of horizontal bolts, the ultimate moment values are close to those of the beam, and punching shear failure was observed also in combination with yielding in flexure of flanges.

In joints of type A13 a higher initial stiffness was observed because of the type and number of horizontal bolts utilized, reducing slippage of flanges in contact.

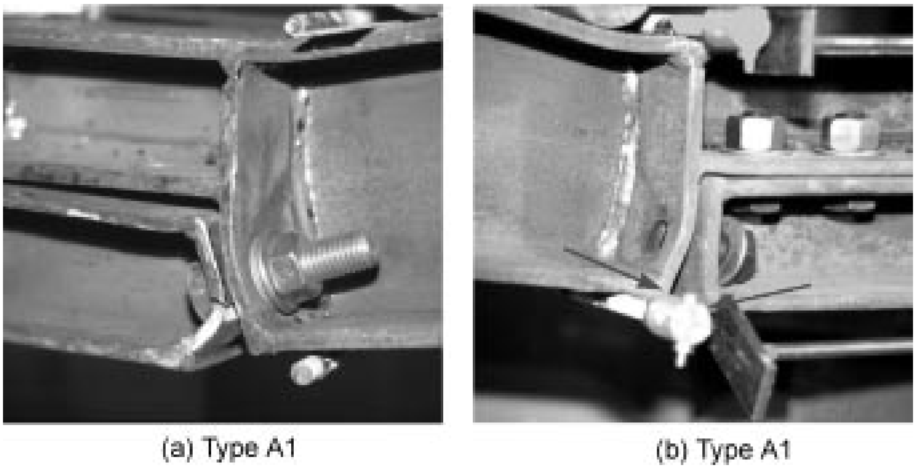

Fig. 9 Failure mode of joints

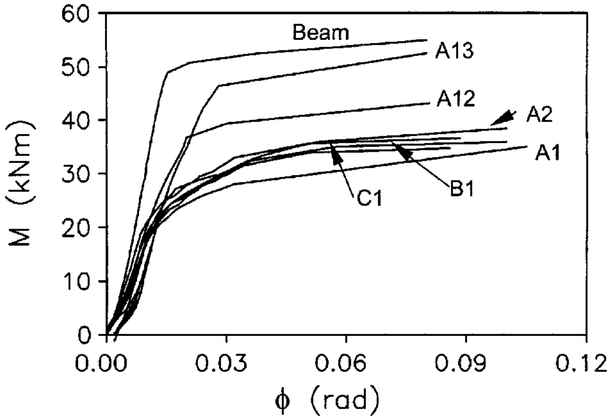

Fig. 10 Moment-rotation diagrams for joints tested

Fig. 10 shows the moment-rotation (M-ϕ) diagrams of the connections examined, together with those of the beam, showing the very high resistance of connections and the very high rotational capacity ensuring rotational ductility of the beam. In each diagram the moment was obtained as $M=P\times a$, a being the shear span. The rotation ϕ was calculated as the ratio between the difference between the two readings of horizontal transducers placed at the top and the bottom of the connection and the vertical distance between these transducers.

Table 2 gives the following mechanical characteristics for the different types of connections examined and for the beam: - ultimate load P_u and corresponding moment M_u and relative displacements and rotations δ_u, ϕ_u; - yielding load P_y and corresponding moment M_y and relative displacements and rotations δ_y, ϕ_y; - initial stiffness of beam R_{ki} and of connections R_{km}. It is interesting to observe that while the yielding moment or yielding load are easily identified for a beam without connections, instead in the case of beams with connections this was not immediately, and conventional values must be adopted (Adany *et al*. 2001).

Table 2 Experimental test results

Type	Beam					Connection				
	P_y (kN)	δ_y (mm)	R_{ki} (kN/m)	P_u (kN)	δ_u (mm)	M_y (kNm)	ϕ_y (rad)	M_u (kNm)	ϕ_u (rad)	R_{km} (kNm/rad)
Beam	280	7.66	40.00	310	30.00	49.00	0.015	55.00	0.080	3619
A1	100	5.12	23.48	200	52.00	22.00	0.012	35.00	0.105	2141
A2	105	4.79	24.55	205	50.00	23.11	0.012	35.87	0.100	2230
A12	140	7.52	25.79	240	40.00	36.75	0.020	43.90	0.082	2450
A13	150	6.21	29.82	300	38.00	46.37	0.028	52.68	0.078	2456
B1	106	5.08	21.19	220	34.00	23.11	0.010	38.50	0.100	2295
C1	135	6.88	25.68	230	33.00	27.12	0.017	36.40	0.090	2396

5. Analytical model and comparison with experimental results

In the present section a simplified analytical model able to predict the complete moment-rotation relationship of the connections examined is proposed. The connection, according to the load scheme chosen, is loaded in flexure. Horizontal and vertical bolts are loaded in tension and in shear, and steel flanges are subjected to flexure and shear actions in the presence of an axial load. Local effects like prying action occurred during the tests and was observed that it reduces the bearing capacity of the bolts; to taking into account this effect in the analytical model the yielding and the ultimate stresses of the bolts were reduced (40%).

Under these actions, connections deform, taking on a configuration like that shown in Fig. 11.

A simplified physical model adopted, validated by experimental observations, is shown in Fig. 12. It is constituted by a rigid frame able to rotate across a fixed point (point O), concentrating elastic and plastic deformations in equivalent springs, placed at point A, C and E and simulating the behavior in series of bolts in tension and of flanges in flexure.

5.1. Initial stiffness of connection and moment-rotation relationship

With reference to the scheme in Fig. 12, having one degree of freedom represented by the rigid rotation ϕ, the initial stiffness of the connection is evaluated. The scheme consists in two rigid frames

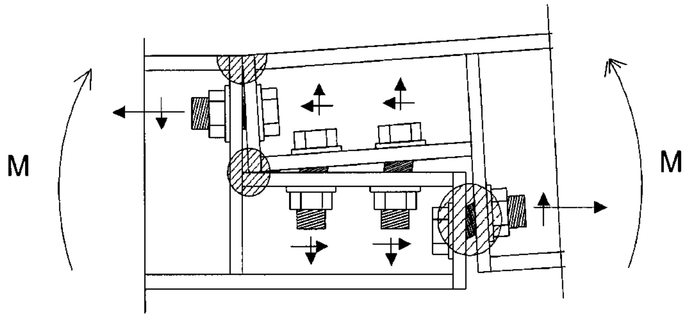

Fig. 11 Effect of bending moment on the joint

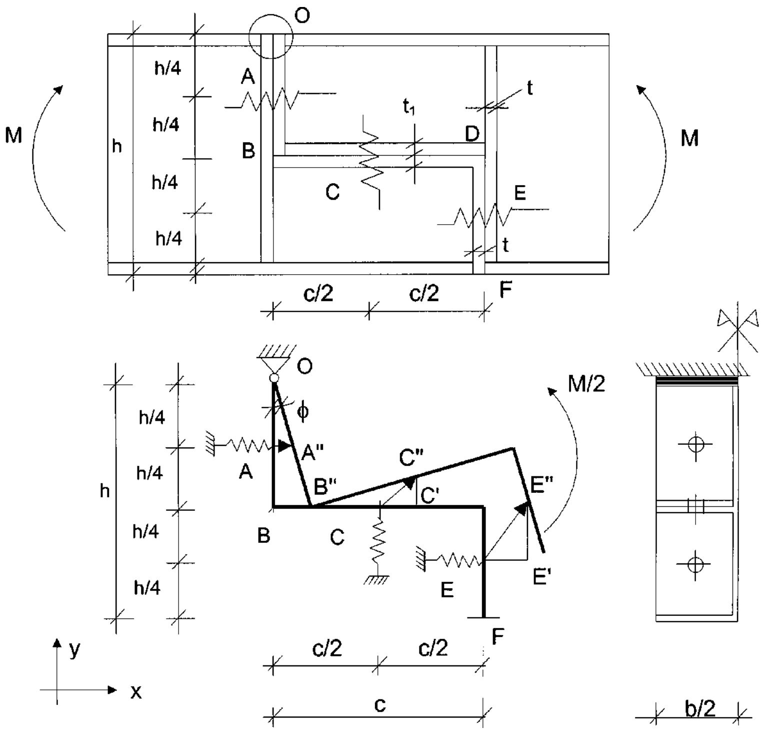

Fig. 12 Mechanical model for analysis of the joint

simply pinned, each consisting of three pieces (two vertical and one horizontal) having length $h/2$ and c respectively. These frames are connected at points A, C and E through elasto-plastic springs simulating the behavior of bolts in tension and in shear and the flexural behavior of flanges, placed together in series. Axial and shear deformations on plates were neglected and small deformations were considered.

Preliminarily details of the kinematic model are given, while in the following sections the spring characteristics are given.

For each value of the angle ϕ it is possible to establish the relationship between this parameter ϕ and the displacement of the point on the rigid frame considered as follows:

$$\delta_{Ax} = h/4 \times \phi; \quad \delta_{Cx} = h/2 \times \phi; \quad \delta_{Cy} = c/2 \times \phi; \quad \delta_{Ex} = 3/4h \times \phi; \quad \delta_{EY} = c \times \phi \tag{1}$$

The reactive forces in the springs are:

$$F_{ix} = k_{ix} \times \delta_{ix} \tag{2}$$

$$F_{iy} = k_{iy} \times \delta_{iy} \tag{3}$$

$k_{i,x}$ $k_{i,y}$ (i=A, C, E) being the stiffness of equivalent springs.

To determine the moment-rotation diagrams of connections it is possible to utilize the following rotational equilibrium equation and for each ϕ value to determine the corresponding moment M:

$$M = F_{Ax} \times h/4 + F_{Cx} \times h/2 + F_{Cy} \times c/2 + F_{Ex} \times 3/4\ h + F_{Ey} \times c \tag{4}$$

By substituting Eq. (1) in Eq. (2) and Eq. (3) and then in Eq. (4), in which ϕ is set equal to one, it is possible to obtain the initial stiffness of the connection.

5.2. Behavior of single components

5.2.1. Bolt characterization

To characterize mechanically the behavior of horizontal bolts an equivalent elasto-plastic spring with strain hardening was considered. The spring behaves elastically with initial stiffness $k_{b,a}$ until the forces F_{yb} is reached. Then there is a linear behavior with reduced stiffness until to the ultimate forces F_{ub} and ultimate elongation corresponding to the maximum strain in the bolts are reached. The initial stiffness $k_{b,a}$ is defined as $E_b \times A_b / l_b$, E_b being the initial modulus of the bolts, A_b the gross section of the bolts and l_b the effective length of the bolts (including the thickness of the flanges and half the thickness of the nuts).

The force F_y is 0.6 $f_{yb}\, A_{b,n}$ and the ultimate force F_u is 0.6 $f_{ub}\, A_{b,n}$., $A_{b,n}$ being the net area of the cross section of the bolts and 0.6 a reduction coefficient to take into account the prying effects. The ultimate elongation of each bolt is $\delta_{u,b} = \varepsilon_{ub}\, l_b$.

Analogously to characterize the behavior of bolts in shear an elastic-plastic spring is adopted having initial stiffness for vertical and horizontal bolts respectively:

$$k_{b,s,y} = \frac{G \cdot A_b}{\chi_b \cdot 2 \cdot t_l}; \quad k_{b,s,x} = \frac{G \cdot A_b}{\chi_b \cdot 2 \cdot t} \tag{5}$$

G being the shear modulus of elasticity, χ_b the shear factor equal to 1.175 for a circular cross-section.

Regarding the strength problem of vertical bolts it is observed that they are subjected to combined actions of axial and shear forces, but axial forces are very reduced because of the position of the bolts in the joints. For this reason it is reasonable not to consider interaction phenomena between shear and axial forces and the maximum shear force is adopted equal to $A_b \times \tau_b$, $\tau_b = 0.6\, f_{yb}$ being in the case of yielding and 0.6 f_{ub} in the case of ultimate state.

5.2.2. Flange characterization

The plate elements considered in the analytical model are the eight vertical flanges (four for each side of the connection, two of them on the upper side and two on the lower side) across the points A and E, and the four horizontal ones across the point C. All panels were loaded in a very small area corresponding to the area of bolts in contact with the flange. The boundary conditions of the rectangular panels were those of plates fixed on three sides and free on the latter. To characterize the behavior of the plates an equivalent grid, consisting in two mutually orthogonal beams, was considered. The beams had the same boundary conditions as the plates and length $h/2$ and c respectively for vertical and horizontal flanges. The width of the equivalent beams was assumed to be, in accordance with EC3, conservative values of 15 t in vertical flanges and of 15 t_1 in horizontal flanges, t or t_1 being the thickness of the beams, as shown in Fig. 13 and Fig. 14.

Applying a unit force in the common joint of the grid of the beams it is possible to obtain the initial stiffness of the flanges:

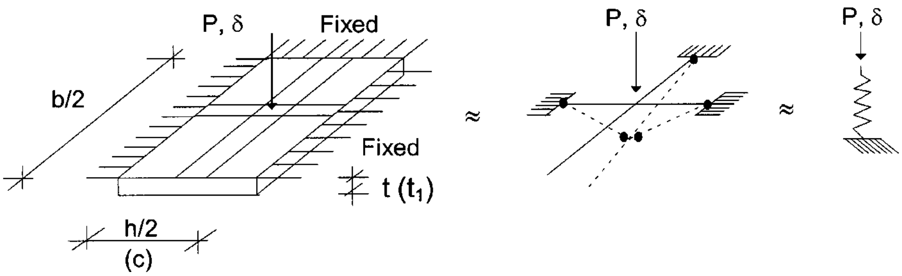

Fig. 13 Equivalent models for steel two-dimensional steel members

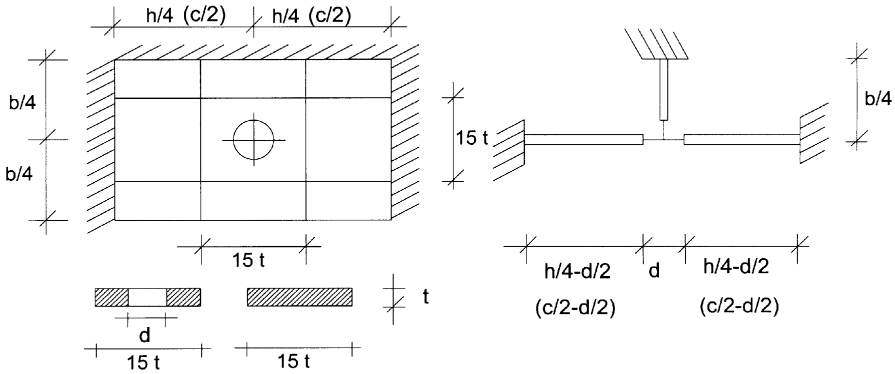

Fig. 14 Geometrical details of equivalent beams

$$k_{ix} = 2 \cdot \frac{E \cdot t^3}{1-\upsilon^2} \cdot (15 \cdot t) \cdot \left[\frac{1}{h^3} \cdot \frac{1 + 64 \cdot \left(\frac{b}{h}\right)^3}{\left(\frac{b}{h}\right)^3} \right] \text{ vertical plates} \tag{6}$$

$$k_{iy} = 2 \cdot \frac{E \cdot t_1^3}{1-\upsilon^2} \cdot (15 \cdot t_1) \cdot \frac{1}{b^3}\left[1 + 8 \cdot \left(\frac{b}{c}\right)^3\right] \text{ horizontal plates} \tag{7}$$

The first yielding load and the corresponding displacement can be obtained by analyzing the equivalent grid and by imposing the condition that in the most stressed section of the beam the yielding moment is reached. For vertical flanges we have:

$$P_{vy} = \frac{M_y' \cdot (h/2)^2}{b^3} + \frac{8 \cdot M_y''}{h/2} \tag{8}$$

$$\delta_{vy} = \frac{M_y' \cdot (h/2)^2 \cdot (1-\upsilon^2)}{2 \cdot E \cdot t^3} \tag{9}$$

Analogously for horizontal flanges:

$$P_{hy} = \frac{M_y' \cdot c^2}{b^3} + \frac{8 \cdot M_y''}{c} \tag{10}$$

$$\delta_{hy} = \frac{M_y' \cdot c^2 \cdot (1-\upsilon^2)}{2 \cdot E \cdot t_1^3} \tag{11}$$

M_y' being the first yielding moment of the cross section of the beam on the hole and M_y'' that of the gross section, having the following expressions in the case of vertical flanges:

$$M_y' = \frac{t^2}{6} \cdot (15 \cdot t - d) \cdot f_y; \quad M_y'' = \frac{5}{2} \cdot t^3 \cdot f_y \tag{12}$$

For horizontal flanges some expressions of Eq. (12) are utilized but using t_1 instead of t.

Adopting a step by step analysis and supposing materials having elasto-plastic behavior the following ultimate load and corresponding displacements are obtained:

- for vertical flanges

$$P_{vu} = 8 \cdot \frac{M_u' + M_u''}{h} + \frac{2 \cdot M_u''}{b} \tag{13}$$

$$\delta_{vu} = \left(\frac{P_{vu}}{4} \cdot h - M_u'\right) \cdot \frac{h^2}{8} \cdot \frac{1-\upsilon^2}{E \cdot t^3} \tag{14}$$

- for horizontal flanges

$$P_{hu} = 4 \cdot \frac{M_u' + M_u''}{c} + \frac{M_u''}{c} \tag{15}$$

$$\delta_{hu} = \left(\frac{P_{Cu}}{4} \cdot c - M_u'\right) \cdot \frac{c^2}{2} \cdot \frac{1-\upsilon^2}{E \cdot t_1^3} \tag{16}$$

M_u' and M_u'' being:

$$M_u' = \frac{t^2}{4} \cdot (15 \cdot t - d) \cdot f_y \quad M_u'' = \frac{15}{4} \cdot t^3 \cdot f_y \tag{17}$$

If punching shear failure occurs before yielding of flanges in flexure, the maximum yielding force in spring is assumed, according to EC3, equal to:

$$P_{yp} = 0.6 \times \pi \times d_1 \times t \times f_y \; ; \qquad d_1 = 1.5 \times d \tag{18}$$

For horizontal flanges same expressions of Eq. (18) are utilized, but using t_1 instead of t.

The initial stiffness, being n_{bo} the number of horizontal bolts and n_{bv} the number of vertical bolts of the connection is obtained by using Eq. (10), setting $\phi = 1$, proves to be:

$$k_i = \frac{5}{16} \cdot h^2 \cdot \frac{1}{\dfrac{1}{n_{bo}\dfrac{A_{bo} \cdot E_b}{l_b}} + \dfrac{1}{4 \cdot \dfrac{E \cdot t^3}{1-\upsilon^2} \cdot (15 \cdot t) \cdot \left[\dfrac{1}{h^3} \cdot \dfrac{1 + 64 \cdot \left(\dfrac{b}{h}\right)^3}{\left(\dfrac{b}{h}\right)^3}\right]}}$$

$$+\frac{c^2}{8}\cdot\frac{1}{\dfrac{1}{n_{bv}\dfrac{A_{bv}\cdot E_b}{l_b}}+\dfrac{1}{4\cdot\dfrac{E\cdot t_1^3}{1-\upsilon^2}\cdot(15\cdot t_1)\cdot\dfrac{1}{b^3}\left[1+8\cdot\left(\dfrac{b}{h}\right)^3\right]}}$$

$$+n_{bo}\frac{A_{bo}\cdot G_b}{\chi_b}\cdot\frac{c^2}{2\cdot t}+n_{bv}\frac{A_{bv}\cdot G_b}{\chi_b}\cdot\frac{h^2}{8\cdot t_1} \qquad (19)$$

5.3. Yielding and ultimate moment of the connection

To evaluate the yielding moment of connections it is possible to utilize Eq. (4) iteratively increasing the ϕ values until the first component yields (bolts and flanges acting in the same position are considered in series).

To evaluate the ultimate moment of connections the same procedure as used to calculate the yielding moment can be utilized, but considering the effective stress values in equivalent springs.

As already mentioned, all equivalent springs considered in the present model represent the effect of bolts and flanges placed in series and they are characterized by a tri-linear model in which each branch is characterized by different slopes (stiffness). The initial slope k_1 was the ratio between P_y and δ_y and the slope of the second branch was characterized by a reduced stiffness having value k_2 defined by the ratio between the difference $(P_u–P_y)$ divided by $(\delta_u–\delta_y)$, and finally the last branch that was assumed practically horizontal (very small slope $\approx 1/1000\ k_2$ to perform stable analyses).

Table 3 gives for all cases examined a comparison between experimental values and analytical values obtained with the proposed model, showing good agreement.

Finally, Fig. 15 shows a comparison between complete experimental and analytical results, for some cases representative, showing the good agreement and the capacity of the model to identify and calibrate the different contributions of components in terms of both strength and strain capacity.

6. Conclusions

A theoretical and experimental investigation regarding the behavior of a particular type of partially restrained semirigid connection is presented. The joint analyzed is able to connect beam to beam and is characterized by the use of flanges and bolts enclosed in the width of the beams and also offers the advantage of constituting a simple support for the beams before bolt connection.

The experimental investigation was carried out varying the type and number of bolts and the thickness of flanges and shows that this kind of joint allows one to obtain good performance in terms of both strength and ductility.

The analytical model proposed allows one to predict the moment-rotation relationship based on the initial stiffness of the connection, the yielding and ultimate moments and the corresponding deformation values. It also allows one to identify and calibrate design parameters of connections and fits very well with experimental results.

Finally, although the primary objective of the paper was to investigate the flexural behavior of the proposed connection, it is very important to consider also the shear resistance of the connection if it has

Table 3 Comparison between analytical and experimental results

Type	Experimental			Analytical		
	M_y (kNm)	ϕ_y (rad)	M_u (kNm)	M_y (kNm)	ϕ_y (rad)	M_u (kNm)
Beam	49.00	0.0150	55.00	44.46	0.0200	54.67
A1	22.00	0.0120	35.00	18.70	0.0160	35.04
A2	23.11	0.0128	35.87	19.38	0.0155	36.30
A12	36.75	0.0120	43.90	34.63	0.0085	46.88
A13	46.37	0.0128	52.68	45.30	0.0090	52.61
B1	23.11	0.0130	38.50	19.18	0.0150	40.38
C1	27.12	0.0170	36.40	19.39	0.0155	40.33

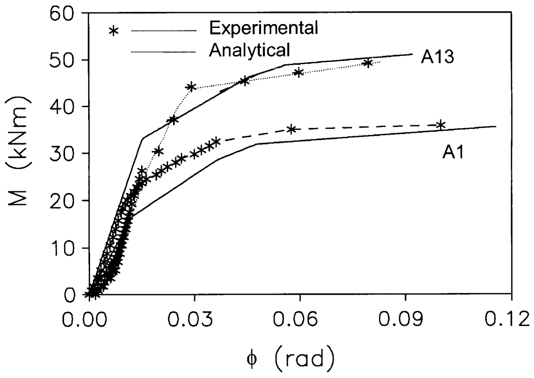

Fig. 15 Comparison between analytical and experimental results

to be utilized in locations where shear is very high, avoiding shear failure for example adopting adequate web thickness of the beam.

References

AISC (1994), “Load and resistance factor design specification for structural steel building”, American Institute of Steel Construction, Inc. Chicago, September.

Astaneh-Asl, A., Mc Mullin, K., M. and Call, S. M. (1993), “Behavior and design of steel single plate shear connections”, *J. Struct. Eng*., ASCE, **119**(8), 2421-2432.

Astaneh-Asl, A. and Nader, M., N. (1991), “Experimental studies and design of steel tee shear connections”, *J. Struct. Eng*., ASCE, **116**(10), 2883-2901.

Adany, S., Calado, L. and Dunai, L. (2001), “Experimental study on the cyclic behaviour of bolted end-plate joints”, *Steel and Composite Structures*, **1**(1), 33-50.

Attiogbe, G. and Morris, G.A (1991), “Moment-rotation functions for steel connections”, *J. Struct. Eng*., ASCE, **117**, june, 1703-1718.

Bernuzzi, C., Zandonini, R. and Zanon, P. (1991), “Rotational behaviour of end plate connections”, *Costruzioni Metalliche*, **2**, 74-103.

Campione, G., Scibilia N. and Gaeta, G., (2001), "Comportamento sperimentale di un giunto trave-trave a parziale ripristino di resistenza", (only available in Italian) in printing on *Proc. of National Conference* "Architettura e acciaio", Venezia, Italy.

Chisal, M., L. (1999), "Modelling M-ϕ curves for standard beam-to column connections", *Eng. Struct.*, **21**, 1066-1075.

De Stefano M., De Luca A. and Astaneh-Asl, A. (1994), "Modelling of cyclic moment-rotation response of double-angle connections", *J. Struct. Eng.*, ASCE, **120**(1), 212-229.

Eurocode 3 (1994), *Steel Structures. European Committee for Standardization* (CEN), ENV1-1.

Faella, C., Piluso, V. and Rizzano, G. (1997) "A new method to design extended end plate connections and semirigid braced frames", *J. Constr. Steel Res.*, **41**(1), 61-91

Manual of Steel Construction: Load and Resistance Factor Design (LRFD) (1994), 2nd Ed., Am. Inst. of Steel Constr. Chicago, I11.

Mazzolani F.M. and Piluso, V., (1996), *Theory and Design of Seismic Resistant Steel Frames*, E&FN Spon an imprint on Chapman & Hll, 2-6 Boundary Row, London SE1 8HN, UK.

Tremblay, R., Timler, P., Bruneau, M. and Filiatrault A., (1995), "Performance of steel structures during the 1994 Northridge earthquake", *Can. J. Civ. Eng*., **22**, 338-360.

Wheeler, A. T., Clarke M. J., Hancock G. J. and Murray, T. M., (1998), "Design model for bolted moment end plate connections jointing rectangular hollow sections", *J. Struct. Eng*., ASCE, **124**(2), 164-173.

Notation

ϕ	: relative rotation of connection
E	: modulus of elasticity of steel
L_b	: length of the beam
I_b	: moment of inertia of beam
$M_{pl,Rd}$	: plastic design moment of the beam
J	: moment of inertia of beam tested
A	: area of the transverse cross-section
h	: height of beam
f_y	: yielding stress of steel
f_u	: ultimate strength of steel
ε_u	: ultimate strain of steel
f_{yb}	: yielding stress of bolt
f_{ub}	: ultimate stress of bolt
ε_{ub}	: ultimate strain of bolt
P_y	: yielding load of beam
P_u	: ultimate load of beam
M_y	: yielding moment of connection
M_u	: ultimate moment of connection
δ_y	: displacement at yielding of beam
δ_u	: displacement at ultimate load of beam
ϕ_y	: rotation at yielding of connection
R_{ki}	: initial stiffness of connection
c	: longitudinal dimension of connection
k_a	: initial stiffness of bolt
E_b	: modulus of elasticity of bolt
A_b	: gross area of bolt
L_b	: effective length of bolt
$A_{b,n}$	: net area of bolt
G	: shear modulus of bolt
χ_b	: shear factor of transverse cross-section of bolt

n_{bv} : number of vertical bolts
n_{bo} : number of horizontal bolts
ν : Poisson coefficient of steel
t : thickness of vertical flanges
t_1 : thickness of horizontal flanges
SC