Steel and Composite Structures, Vol. 6, No. 2 (2006) 139-157
DOI: http://dx.doi.org/10.12989/scs.2006.6.2.139

Cyclic mechanical model of semirigid top and seat and double web angle connections

Raffaele Pucinotti†

Department of Mechanics and Materials, University of Reggio Calabria, Italy

(Received November 30, 2004, Accepted July 27, 2005)

Abstract. In this paper, a cyclic mechanical model is presented to simulate the behaviour of top and seat with web angle beam-to-column connections. The introduced mechanical model is compared with Eurocode 3 Annex J, its extension, and with experimental data. To have a better insight regarding the actual response of the joints, available results of the experiments, carried out on full-scale top and seat angle joints under monotonic and cyclic loading, are first considered. Subsequently, a finite element model of the test setup is developed. The application of the proposed model, its comparisons with the experimental curves and with the Eurocode 3 Annex J and with its modification, clearly show the excellent quality of the model proposed.

Keywords: semirigid joints; steel structures; bolted connections; mechanical model; Eurocode 3; Annex J; finite element model; partial strength.

1. Introduction

On the basis of recent knowledge, it is reasonable to classify all connections as semirigid. In fact, the assumption of fully rigid or ideally pinned connection behaviour, that drastically simplifies the analysis, may be questionable in all cases in which the connection rigidities are intermediate between fully and ideally pinned cases. Semi-rigid connections are complicated substructures consisting of member components, connection components and fasteners. The interaction among the elements is complex, especially under cyclic loading. The number of geometrical and mechanical parameters of the connections is almost countless even for a single type of connection. The most notable characteristic of semi-rigid connections is the presence of large deformations, even when the beams and columns remain at a working stress level. These are characterized by the phenomenon of contact between bolts and connected members, and the phenomenon of bolt slip in the holes, caused by pre-tension in the bolts, when the shear force is higher than the friction force.

Researchers have been aware of the concept of semirigid connections for many years. This concept has been included in US and European codes, but the theoretical knowledge did not actually have an immediate impact on practice. In fact, thus far the joints of steel framed structures under analysis have been assumed to be perfectly pinned when they do not transfer the bending moments (flexible connections), or full fixed when they do transfer the bending moments (rigid connections).

In this paper, a cyclic mechanical model for the inelastic analysis of semirigid and partial-strength top

†Assistant Professor, E-mail: raffaele.pucinotti@unirc.it

and seat angle bolted connections is presented. The model is based on the same "component approach" introduced by Eurocode 3. The proposed model still maintains the Eurocode 3 approach, but introduces a more refined modeling of the cleat-to-column interface. Additionally, it introduces a different expression for the evaluation of the moment capacity of the joint which takes into account the effect of the d/t ("d" is the diameter of the bolt connecting the angle to the column flange, and "t" is the angle thickness) and r/t ("r" is the groove fillet radius).

A series of comparisons are carried out among the experimental curves (Exp.), Mechanical Model (MecMod), Eurocode 3 Annex J (EC3 (no web)) and "modified" Eurocode 3 Annex J (EC3 (web) for top and seat with web angles, and EC3 (web+Hr) for top and seat with web angles plus hardening).

2. Modeling of connection behaviour

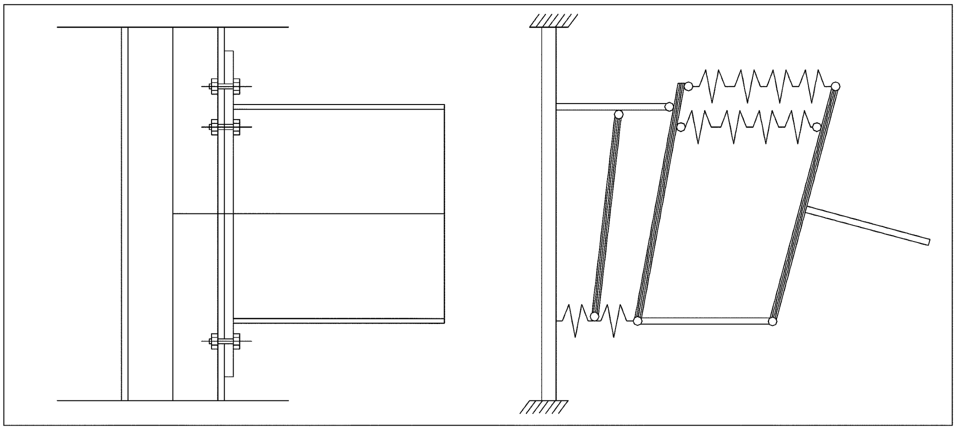
The available methods for the prediction of beam-to-column behaviour can be divided into four categories according to their basis of formulation (Pucinotti 1998): *mathematical models* (describing the curves M-Φ by means of mathematical expressions); *simplified analytical models* (based on concepts of structural analysis: equilibrium, compatibility and material constitutive relations); *mechanical models* (joints are conceived as a set of rigid and deformable components representing the behaviour of single elements); and *finite element models*.

The best means for understanding connection behaviour are experimental tests. The moment-rotation relationship is the most important result from an experimental test. Experimental results have shown that connection moment-rotation relationships are non-linear over the entire range of loading for almost all types of joints. Several mathematical models have been proposed for the analysis of inelastic connection behaviour under monotonic loading and under fully reversed cyclic loading (De Stefano *et al*. 1994a, 1994b, De Luca *et al*. 1995, Pucinotti 2001a).

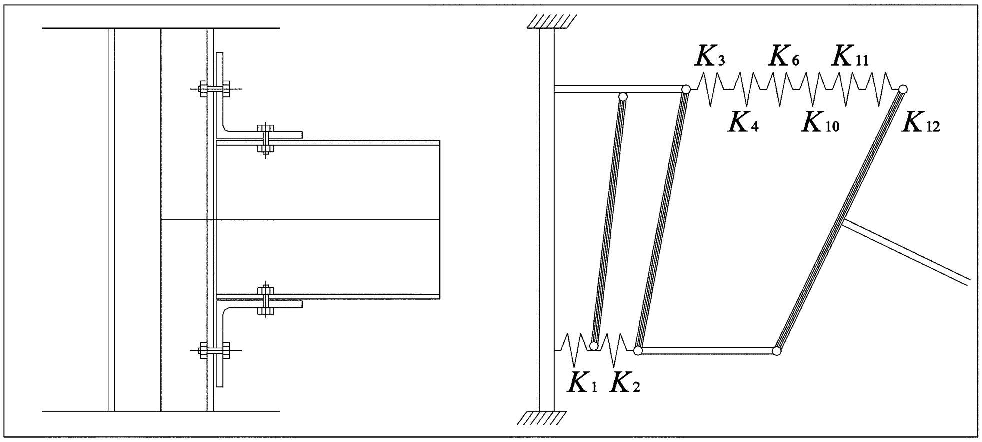
3. Eurocode Annex-J model

The Annex J of the Eurocode 3 (Commission of the European Communities, 1998) addresses the issue of the analysis and design of beam-to-column joints in building frames subjected to predominantly static loading. The EC3 introduces a mechanical model that simulates connection behaviour by a series of different components. Each component is modelled as an elastic spring with a specific stiffness and a specific strength. The appropriate coupling of these springs in a parallel-series fashion gives the global stiffness and strength of the connection (Fig. 1). As far as the global connection strength is concerned, the different failure mechanisms are identified, the minimum value of failure loads being the design resistance of the connection. For each type of joint, the component model requires a preliminary identification of the basic components of the joint. Component stiffness coefficients, K_i, and resistant design forces ($F_{rd,}i$) are then evaluated. Finally, the joint initial rotational stiffness ($S_{j,ini}$) and its design moment capacity ($M_{j,Rd}$) can be computed.

For top and seat angle connections, the following components should be considered (Fig. 1b): stiffness coefficients of the column web panel in shear (K_1), compression (K_2) and tension (K_3), column flange flexural stiffness (K_4), and flange cleat flexural stiffness (K_6), the bolts' tensile stiffness (K_{10}), and, for non-preloaded bolts, their shear stiffness (K_{11}) and their bearing stiffness (K_{12}).



(a)



(b)

Fig. 1 Example of Annex J model: (a) End plate connections, (b) Top and seat angle connections

Therefore, the joint initial stiffness and the design moment capacity are given by:

$$S_{j,ini} = \frac{Ez^2}{\sum_{i=1}^{n} 1/K_i} \tag{1}$$

$$M_{J,Rd} = F_{Rd}z \tag{2}$$

$$F_{Rd} = \min[F_{Rd1}, F_{Rd2}, \ldots, F_{Rdn}] \tag{3}$$

where "E" is the steel Young's modulus, "K_i" is the i-th component stiffness coefficient; "n" is the number of basic joint components, and "z" is the lever arm.

For bolted connections with angle flange cleat, z should be taken as the distance from the mid-thickness of the leg of the angle cleat on the compression flange and the bolt-row in tension.

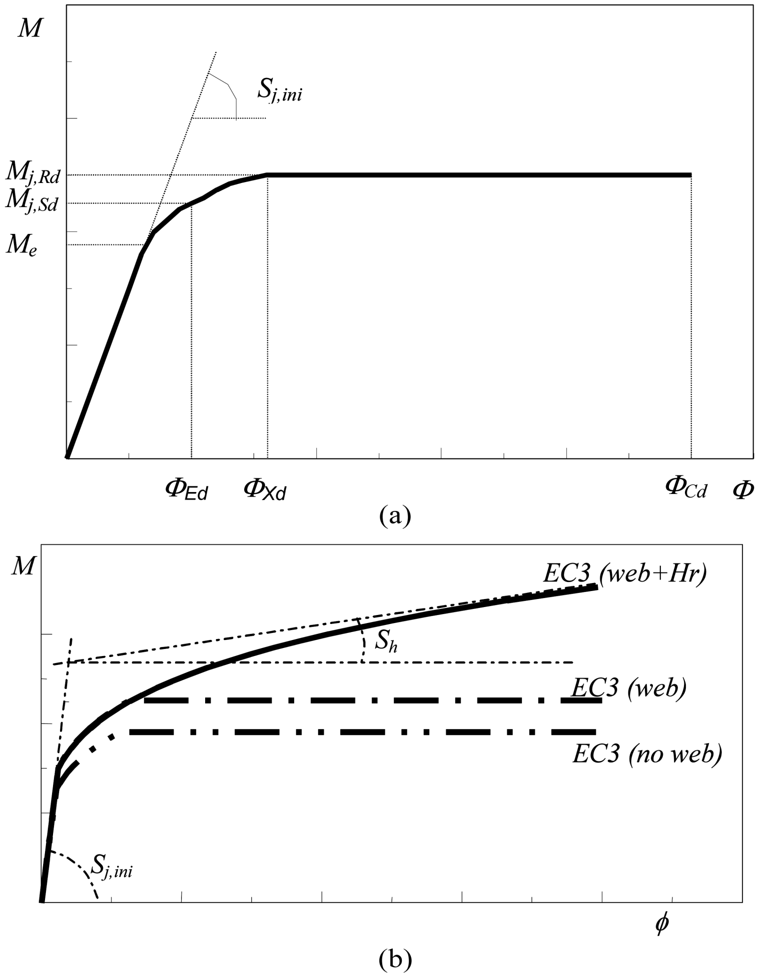


Fig. 2 Curve M-Φ (a) EC3-Annex J model; (b) Extension of EC3-Annex J model

The moment-rotation response is described by a linear elastic relationship Eq. (4) if the moment $M_{j,Sd}$ is lower than the elastic one, M_e (M_e=2/3$M_{j,Rd}$). It is followed by a non-linear part Eq. (5) up to the attainment of $M_{j,Rd}$, which provides the plateau of the M-Φ curve up to the ultimate rotation Φ_{Cd}, as shown in Fig. 2(a).

$$\varphi = \frac{M_{j,Sd}}{S_{j,ini}^{ts,w}} \text{ if } M_{j,Sd} < 2/3 M_{j,Rd} \tag{4}$$

$$\varphi = \frac{(1.5 M_{j,Sd}/M_{j,Rd})^{\psi}}{S_{j,ini}^{ts,w}} M_{j,Sd} \text{ if } 2/3 M_{j,Rd} < M_{j,Sd} < M_{j,Rd} \tag{5}$$

In this equation:

$M_{j,Rd}$ is the design moment resistant of the connection,

$M_{j,Sd}$ is the design applied moment, and

ψ is the shape factor,

ψ=3.1 for top and seat angle connections.

Annex J does not include a mechanical model for top-and-seat with web angle connections; an extension of Annex J for these type of connections (indicated with EC3 (web) in Fig. 2b) was presented by the author in a previous work (Pucinotti 2001a).

In the same work (Pucinotti 2001a), the limitation on the resistance moment was also neglected and the validity of Eq. (5) was extended also to cases of $M>M_{j,Rd}$.

In this case, Eqs. (4, 5) become (Fig. 2b):

$$\varphi = \frac{M_{j,Sd}}{S_{j,ini}^{ts,w}} \text{ if } M_{j,Sd} < 2/3 M_{j,Rd} \tag{6}$$

$$\varphi = \frac{(1.5 M_{j,Sd}/M_{j,Rd})^{\psi}}{S_{j,ini}^{ts,w}} M_{j,Sd} \text{ if } M_{j,Sd} > 2/3 M_{j,Rd} \tag{7}$$

The modified EC3-Annex J curve, (marked as EC3 (web+Hr) in Fig. 2b), shows a hardening behaviour. In this case, for ψ we can conserve the value used for top and seat angle connections; ψ= 3.1.

4. Finite element model

To have a better insight into the actual response of the joints, available results of the experiments, carried out on full-scale top and seat angle joints under monotonic and cyclic loading (Bernuzzi *et al.* 1997), are first considered and then a finite element model of the test setup is developed (Fig. 3). The

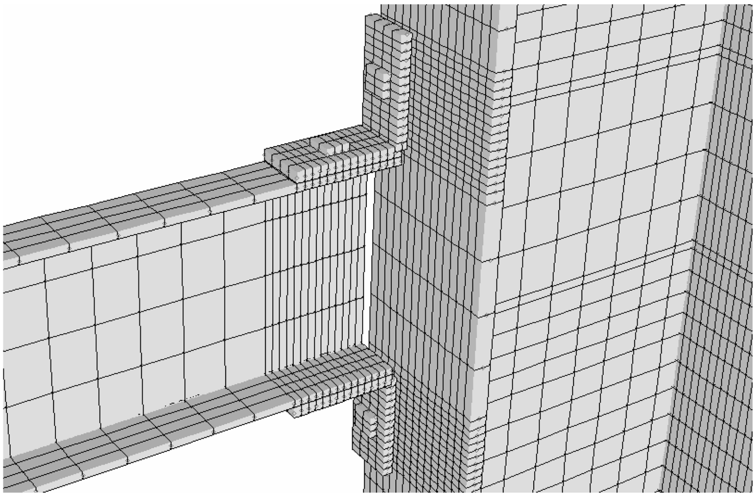


Fig. 3 Finite element model

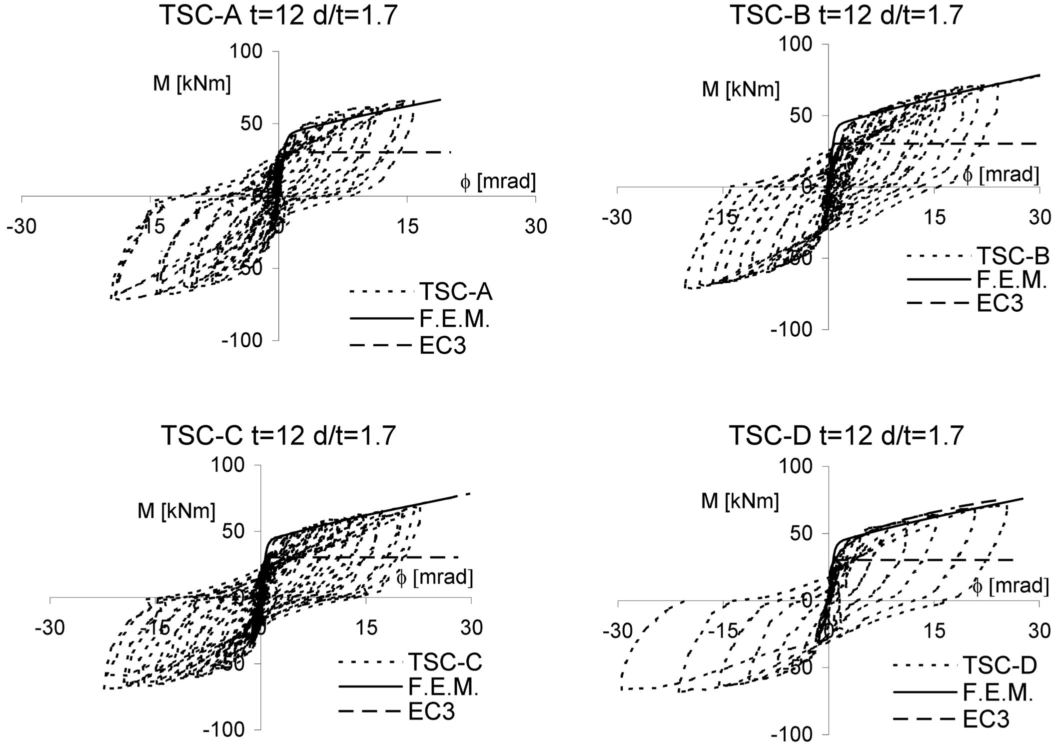


Fig. 4 Comparison among EC3-Annex J model, F.E.M. model and experimental "Bernuzzi" data

most relevant parameters influencing the non-linear response of the joint were considered in the F.E.M. model. The unilateral contact between the column flange and the angular cleat was modelled with a set of discrete gap elements whose initial stiffness, K_t, was estimated by the following expression (Wales and Rossow 1983):

$$K_t = \frac{t_{wc}E}{\ln(1 + H_c)}B_a \tag{8}$$

where t_{wc} is the column flange thickness, E is the Young modulus, H_c is the column height and B_a is the angle base size.

The accuracy of the finite element model was assessed by comparing the analytical predictions with the experimental results. The results of this comparison (see Fig. 4) show that the finite element moment-rotational predictions are in good agreement with the experimental results. Once a reasonable accuracy of the finite element model had been assessed, a parametric analysis was developed to understand the influence of some parameters on the inelastic moment-rotation response. In the parametric analysis, joint moment-rotation curves were derived for various values of the varying parameter "d/t_a" (where "d" is the diameter of the bolt connecting the angle to the column flange and "t_a" is the angle thickness). The results of the finite element model were compared with the inelastic moment-rotation predictions obtained by applying the Eurocode 3-Annex J model. The results of this

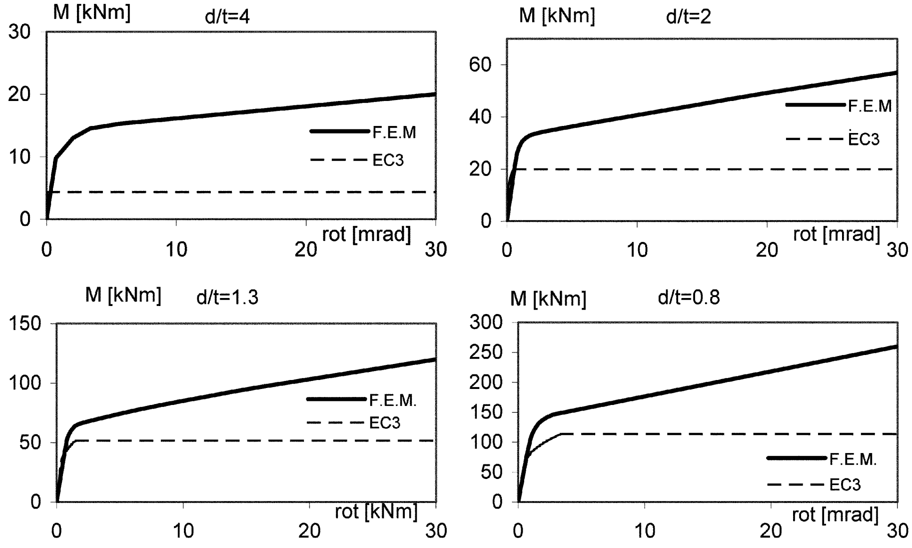


Fig. 5 Parametric analysis and comparison among F.E.M. model and EC3-Annex J model

comparison (Fig. 5) confirm that Eurocode 3 underestimates the joint capacity predicted by the finite element model over the entire range of variation of the investigated parameter d/t_a. The predictions of EC3, which does not take into account the effect of the d/t_a ratio on the joint capacity, are also erratic despite the fact that they are conservative. These results confirm that the EC3 model is suitable for design purposes, but it is not accurate enough to assess the inelastic rotation demand of actual connections.

5. Mechanical model

The present mechanical model (MecMod) is based upon "component" philosophy. This philosophy consists of the assessment of connection responses based on the most relevant components. In the past there were many mechanical models proposed by researchers to simulate both monotonic and cyclic behaviour (Kishi and Chen 1990, De Stefano *et al.* 1994, Pucinotti 2001a, 2001b, Ballio *et al.* 1987, De Stefano and De Luca 1992, De Stefano *et al.* 1994, Bernuzzi *et al.* 1996, Bernuzzi 1997, Bernuzzi *et al.* 1997).

The proposed model simulates the moment-rotation relationship for top and seat angle connections with and without web angles. Based on experimental data and the results of the previous parametric analysis, the model was modified by introducing a different expression for the evaluation of the lever arm, which in turn modifies the joint capacity. The present model is a further extension of a previously presented model (Pucinotti 2001b), where the effect of the unilateral contact between the angle cleat and the column flange was already included. The joint is idealised as two rigid bars connected by two non-linear springs (Fig. 6), representing the axial response of the angles (Pucinotti 2001b). In particular, the rigid bars, AB and CD, represent the column, and the beam, respectively.

A beam (AC in Fig. 7) is incorporated into the model to simulate the flexural response of the outstanding leg of the angle, and a spring (BE in Fig. 7) simulates the bolted effect.

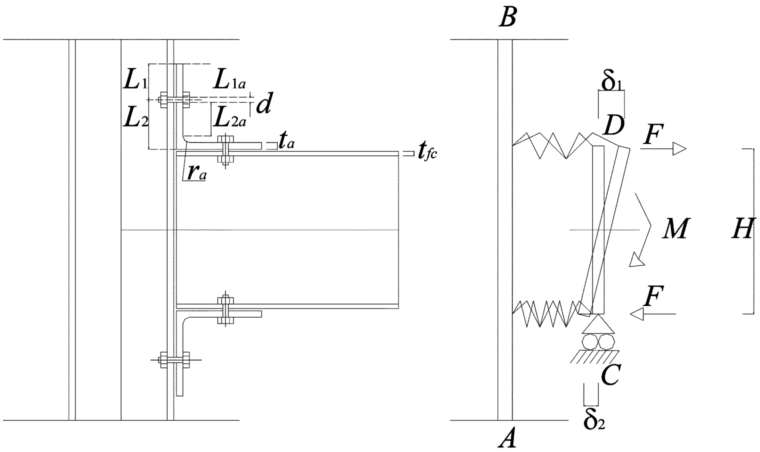


Fig. 6 Top and seat angle connections: Mechanical model

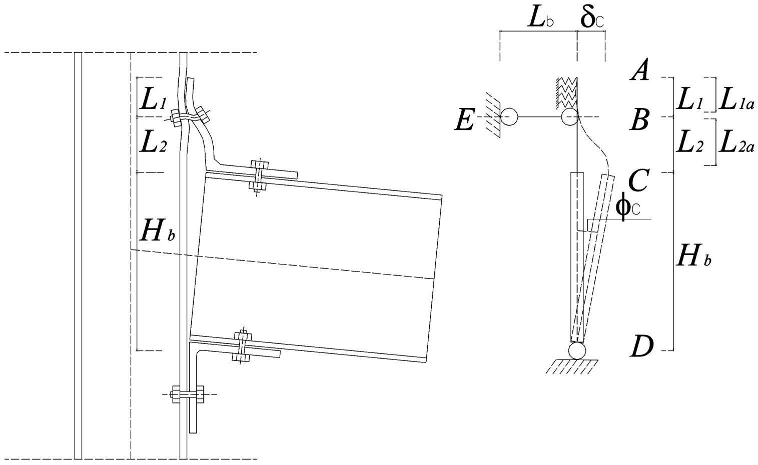


Fig. 7 Top and seat angle connections: Mechanical model

The AB part of the beam is schematised as an elastic beam supported by an assembly of elastic independent springs that represent the stiffness K_t of the column web (Wales and Rossow 1983):

$$K_t = \frac{t_{wc}E}{\ln(1 + H_c)}B_a \tag{9}$$

in which:

t_{wc} = thickness of the web column;
E = Young's modulus;
H_c = height of the beam;
B_a = width of the outstanding leg.

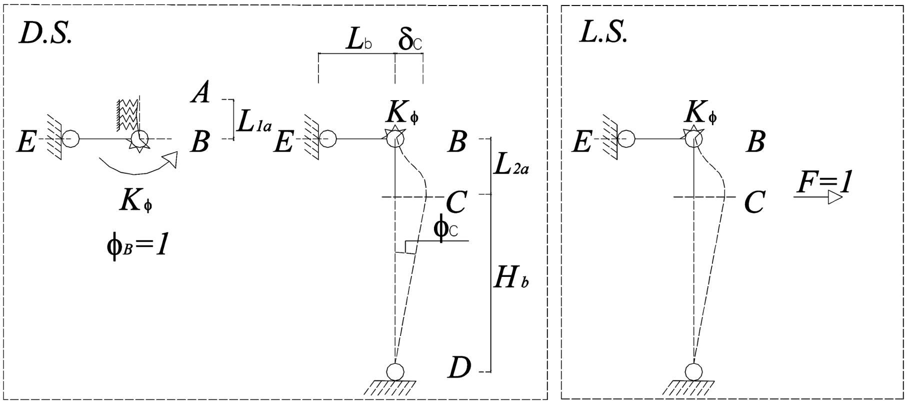


Fig. 8 Application of the principle of virtual forces

The BC part of the beam is schematised as an inelastic beam with linear strain hardening, while the BE part is schematised as an elastic-perfectly-plastic spring.

The end C of the outstanding leg is free to translate vertically, but it must rotate by $\varphi_C = \dfrac{\delta_C}{H_b}$, where δ_C is the vertical translation and H_b is the height of the connected beam.

To obtain δ_C it is possible to apply the principle of virtual forces (Fig. 8), considering a virtual unit load condition applied in C and orthogonal to the beam, which gives the moment distribution $M'(z)$:

$$\delta_C = \int_0^{L_{2a}} M'(z)\chi(z)dz + N'_{BE}\frac{N_{BE}}{K_b} + M'_B\frac{M_B}{K_\phi} \tag{10}$$

where:

χ = curvature of the BC part of the beam;
N'_{BE} = axial load;

K_b = axial stiffness of the bolts = $\dfrac{E\pi d^2/4}{t_a + t_{fc}}$;

t_a = thickness of the angle;
t_{fc} = thickness of the flange column.

Based on the results of the finite element analyses and on the parametric analyses, a modified expression for the evaluation of “L” was proposed (Figs. 6 and 8) in order to take into consideration the effect of the investigated d/t_a ratio and the r_a/t_a ratio (r_a=root fillet radius) on the joint capacity:

$$L_{1a} = L_1 - d/2 \tag{11}$$

$$L_{2a} = L_2 - t_a - \alpha r_a - \beta d \tag{12}$$

where: L_1, L_{1a}, L_2, L_{2a}, t_a, r_a, d are depicted in Figs. 6 and 7;

$$\alpha = -2.019\left(\frac{r_a+t_a}{t_a}\right)^3 + 9.574\left(\frac{r_a+t_a}{t_a}\right)^2 - 10.837\left(\frac{r_a+t_a}{t_a}\right) \tag{13}$$

$$\beta = \left|\begin{array}{ll} \frac{1}{2} & if \quad \frac{d}{t_a} > 1.5 \\ \sqrt{-\left(\frac{d}{t_a}\right)^2 + 3\left(\frac{d}{t_a}\right) - \frac{8}{4}} & if \quad 1 \leq \frac{d}{t_a} \leq 1.5 \\ 0 & if \quad \frac{d}{t_a} < 1 \end{array}\right| \tag{14}$$

The rotational stiffness K_ϕ of the spring B is obtained by means of the solution of the fourth order differential equation applied to the AB part of the outstanding leg (Fig. 8):

$$K_\phi = 2EI\alpha^2[E_1(A_1C - A_2S) - E2(A_3C - A_4S)] \tag{15}$$

where:

$$I = B_a t_a^3/12, \qquad \alpha = \left(\frac{K_t}{4EI}\right)^{(1/4)}, \qquad E_1 = \exp(\alpha L_{1a}) \tag{16}$$

$$E_2 = \exp(-\alpha L_{1a}), \qquad C = \cos(\alpha L_{1a}), \qquad S = \sin(\alpha L_{1a}) \tag{17}$$

In particular, A_1, A_2, A_3 and A_4, have been carried out by means of the boundary conditions (bending moment, $M_{(A)}$=0; shear, $V_{(A)}$=0; orrizontal displacement of B, $v_{(B)} = -V_{(B)}/K_b$; rotation of B, $\varphi_{(B)} = 1$):

$$A_1 = A_3 = \frac{1}{(c - a/bd)}, \qquad A_2 = 2A_1 + A_4, \qquad A_4 = -A_1 a/b \tag{18}$$

where:

$$a = E_1 S + 2E_1 C + E_2 S, \qquad b = E_1 C + E_2 C \tag{19}$$

$$c = 3\alpha E_1(C - S) + 2\alpha E_2(C - S), \qquad d = \alpha E_1(C - S) - 2\alpha E_2(C + S) \tag{20}$$

In Fig. 9, a monotonic non-linear F-δ_C relationship is reported.

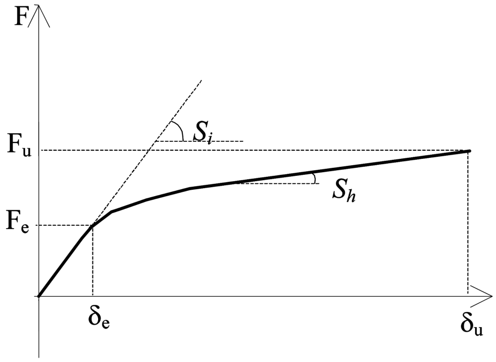


Fig. 9 Elasto-plastic relationship with linear strain hardening

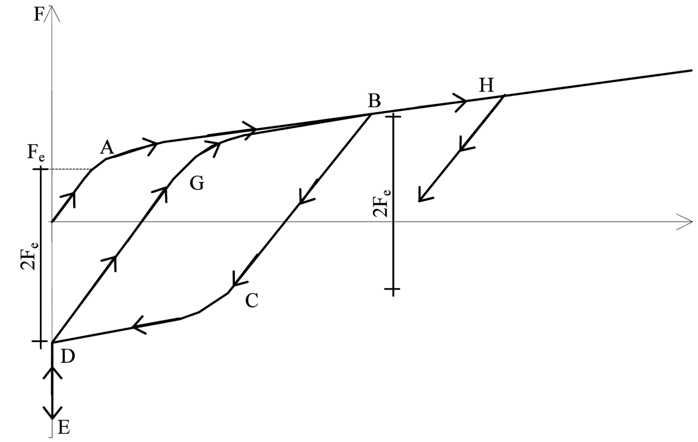


Fig. 10 Extension at the cyclic case

Based on a previous experimental study (Bernuzzi 1995, 1997), the response of the outstanding leg in the cyclic case can be defined by the following phases (Fig. 10):

- unloading phase
  - BC: linear elastic relationship of breadth $2F_e$ and stiffness S_i;
  - CD: post-elastic behaviour with stiffness S_h;
  - DE: contact between the outstanding leg and the column flange;
- reloading phase
  - ED: reloading with contact between the outstanding leg and the column flange;
  - DG: elastic linear relationship of breadth $2F_e$ and stiffness S_i;
  - GH: post-elastic behaviour with stiffness S_h;

The relationships between the applied load and the corresponding moment and displacement (Fig. 10) are:

$$M = F \cdot H; \qquad \varphi = \frac{\delta_1 - \delta_2}{H} \tag{21}$$

For top and seat with web angle connections, in the mechanical model, a number of "components" equal to bolt-rows of the web cleat will be added (Fig. 11). The stiffness K_{tai} of the column web in correspondence of the i-th bolt row is given by the formula (Wales and Rossow 1983):

$$K_{twi} = \frac{t_{wc}E}{\ln(1 + H_c)} B_{awi}$$

in which:

t_{wc} = thickness of the web column;
E = Young's modulus;
H_c = height of the beam;
B_{awi} = width of the portion of the outstanding leg of web angles (Fig. 11).

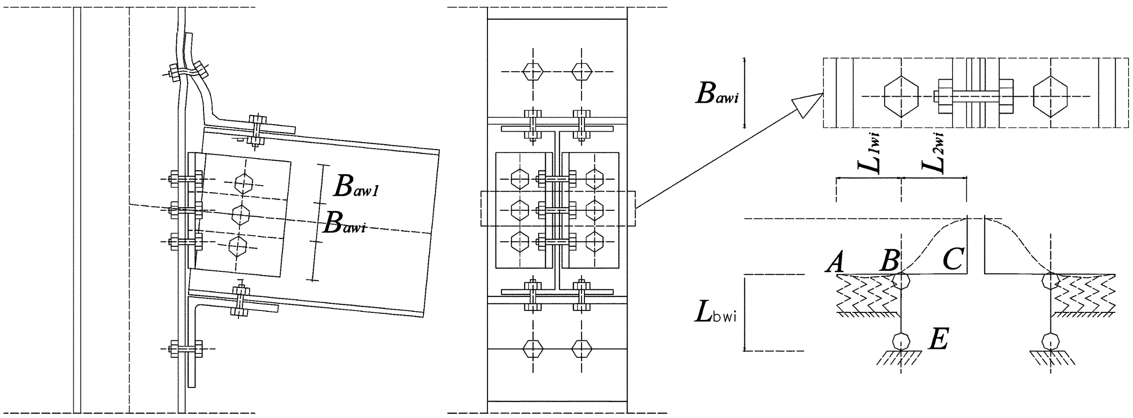


Fig. 11 Mechanical model with the addition of the web angles

In this case, the BC part (Fig. 11) of the beam is also schematised as an inelastic beam with linear strain hardening, while the BE part is schematised as an elastic-perfectly-plastic spring.

The end C of the portion of the outstanding leg of the web angle is free to translate horizontally, but its rotation is $\varphi_{Cwi} = 0$.

δ_{Cwi} is obtained by the application of the principle of virtual forces:

$$\delta_{Cwi} = \int_0^{2L_{2wi}} M'(z)\chi(z)dz + N'_{BE}\frac{N_{BE}}{K_{bwi}} + M'_B\frac{M_B}{K_{\phi wi}} \tag{23}$$

where:

χ = curvature of the BC part of the beam;
N'_{BE} = axial load;
K_{bwi} = axial stiffness of the bolts = $\dfrac{E\pi d_w^2/4}{t_{aw}+t_{fc}}$;
t_{aw} = thickness of the web angle;
t_{fc} = thickness of the flange column.
$K_{\phi wi}$ = rotational stiffness (obtained similarly at the Eq. 15)

and:

$$L_{1awi} = L_{1wi} - d_w/2 \tag{24}$$

$$L_{2awi} = L_{2wi} - t_{aw} - \alpha_w r_{aw} - \beta_w d_w \tag{25}$$

where: L_{1wi} and L_{2wi} are depicted in Fig. 11, while r_{aw} is the root fillet radius of the web angle, and d_w is the diameter of the web bolts.

$$\alpha_w = -2.019\left(\frac{r_{aw}+t_{aw}}{t_{aw}}\right)^3 + 9.574\left(\frac{r_{aw}+t_{aw}}{t_{aw}}\right)^2 - 10.837\left(\frac{r_{aw}+t_{aw}}{t_{aw}}\right) \tag{26}$$

$$\beta_w = \begin{cases} \dfrac{1}{2} & if \quad \dfrac{d_w}{t_{aw}} > 1.5 \\ \sqrt{-\left(\dfrac{d_w}{t_{aw}}\right)^2 + 3\left(\dfrac{d_w}{t_{aw}}\right) - \dfrac{8}{4}} & if \quad 1 \le \dfrac{d_w}{t_{aw}} \le 1.5 \\ 0 & if \quad \dfrac{d_w}{t_{aw}} < 1 \end{cases} \tag{27}$$

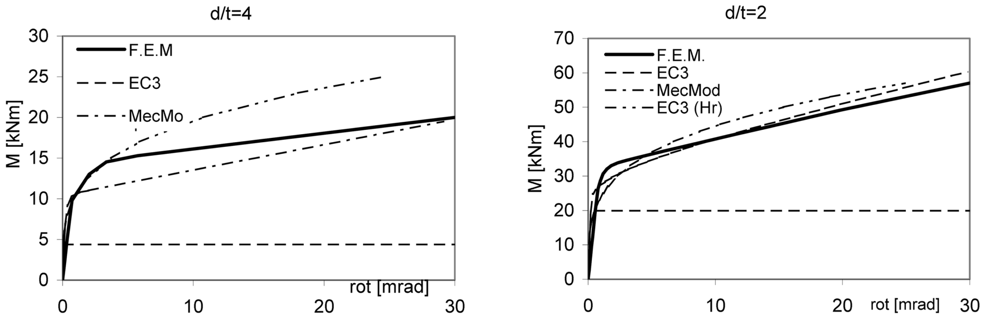


Fig. 12 Comparison among F.E.M. model, EC3-Annex *j* model, Modified EC3-Annex *j* model and mechanical model

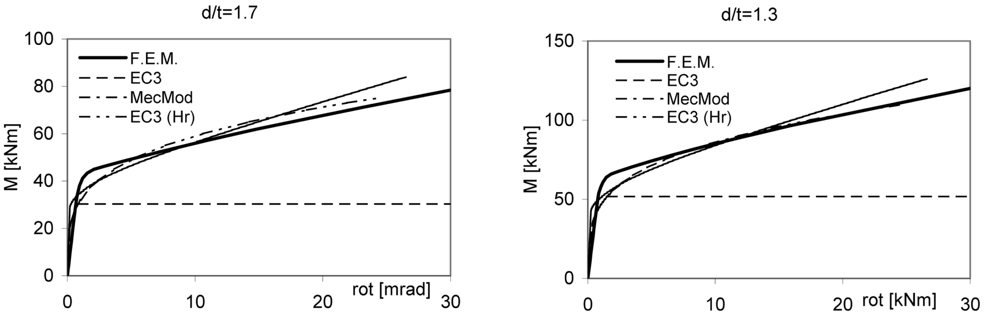


Fig. 13 Comparison among F.E.M. model, EC3-Annex *j* model, Modified EC3-Annex *j* model and mechanical model

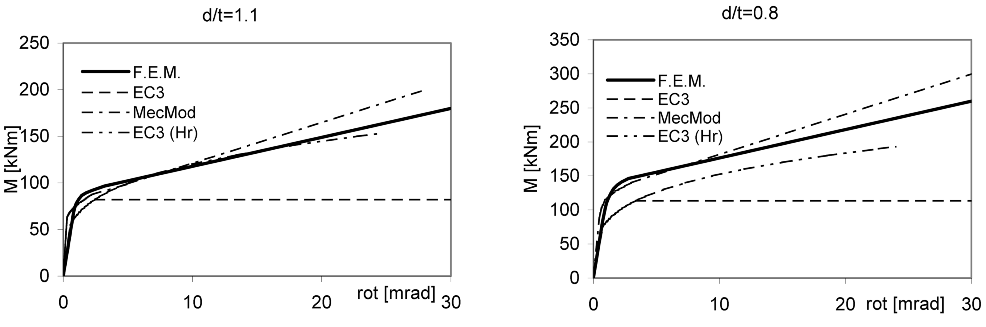


Fig. 14 Comparison among F.E.M. model, EC3-Annex *j* model, modified EC3-Annex *j* model and mechanical model

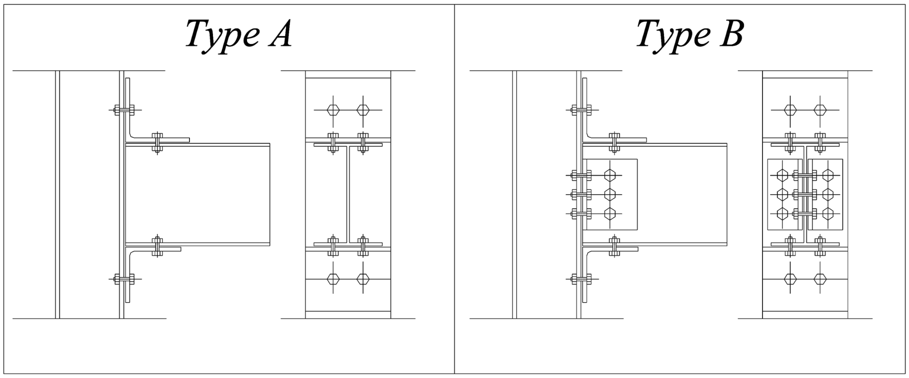


Fig. 15 Type of investigated connections

Table 1 Bernuzzi: geometrical and mechanical characteristics of the joints

TEST	Type of joint	Beam	Column	Flange angle Web angle	Type of bolt	f_{ya} f_{ua} [N/mm^2]	f_{yfc} f_{ufc} [N/mm^2]	f_{yfb} f_{ufb} [N/mm^2]
TSC-A	Type A	HE600B	IPE300	L120×120×12 /	M20 8.8	313.20 459.20	/ /	/ /
TSC-B	Type A	HE600B	IPE300	L120×120×12 /	M20 8.8	313.20 459.20	/ /	/ /
TSC-C	Type A	HE600B	IPE300	L120×120×12 /	M20 8.8	313.20 459.20	/ /	/ /
TSC-B	Type A	HE600B	IPE300	L120×120×12 /	M20 8.8	313.20 459.20	/ /	/ /

f_{ya} = Yield stress of flange Cleats f_{yfc} = Yield stress of Column flange f_{yfb} = Yield stress of Beam flange
f_{ua} = Ultimate stress of flange Cleats f_{ufc} = Ultimate stress of Column flange f_{ufb} = Ultimate stress of Beam flange

Table 2 "Sericon" data bank: geometrical and mechanical characteristics of the joints

TEST	Type of joint	Beam	Column	Flange angle Web angle	Type of bolt	f_{ya} f_{ua} [N/mm^2]	f_{yfc} f_{ufc} [N/mm^2]	f_{yfb} f_{ufb} [N/mm^2]
101003	Type A	IPE 200	HE160B	L150×90×15 /	M16 -	/ /	280.0 422.3	351.0 456.0
101006	Type A	IPE 200	HE160B	L150×90×15 /	M16 10.9	/ /	280.0 422.3	351.0 456.0
101012	Type AB	IPE 300	HE160B	L150×90×15 /	M16 10.9	/ /	280.0 422.3	303.0 447.0

f_{ya} = Yield stress of flange Cleats f_{yfc} = Yield stress of Column flange f_{yfb} = Yield stress of Beam flange
f_{ua} = Ultimate stress of flange Cleats f_{ufc} = Ultimate stress of Column flange f_{ufb} = Ultimate stress of Beam flange

6. Applications

The prediction of the proposed model is compared to the finite element analysis, with Eurocode 3, and with the modified Eurocode 3. The results of this comparison (Figs. 12, 13 and 14) show that the

Table 3 "Chen" data bank: geometrical and mechanical characteristics of the joints

TEST	Type of joint	Beam	Column	Flange angle Web angle	Type of bolt	f_{ya} f_{ua} [N/mm^2]	f_{yfc} f_{ufc} [N/mm^2]	f_{yfb} f_{ufb} [N/mm^2]
8S1 (A36)	Type B	W8×21	W12×58	6×3.5×5/16×6 4×3.5×1/4×5.5	M19.1 A325	285.40 480.40	285.40 480.40	285.40 480.40
8S2 (A36)	Type B	W8×21	W12×58	6×3.5×3/8×6.0 4×3.5×1/4×5.5	M19.1 A325	285.40 480.40	285.40 480.40	285.40 480.40
8S3 (A36)	Type B	W8×21	W12×58	6×3.5×3/8×6.0 4×3.5×1/4×5.5	M19.1 A325	285.40 480.40	285.40 480.40	285.40 480.40
8S4 (A36)	Type B	W8×21	W12×58	6×6.0×3/8×6.0 4×3.5×1/4×5.5	M19.1 A325	285.40 480.40	285.40 480.40	285.40 480.40
8S5 (A36)	Type B	W8×21	W12×58	6×4.0×3/8×8.0 4×3.5×1/4×5.5	M19.1 A325	285.40 480.40	285.40 480.40	285.40 480.40
8S6 (A36)	Type B	W8×21	W12×58	6×4.0×5/16×6 4×3.5×1/4×5.5	M19.1 A325	285.40 480.40	285.40 480.40	285.40 480.40
8S7 (A36)	Type B	W8×21	W12×58	6×4.0×3/8×6.0 4×3.5×1/4×5.5	M19.1 A325	285.40 480.40	285.40 480.40	285.40 480.40
8S8 (A36)	Type B	W8×21	W12×58	6×3.5×5/16×6 4×3.5×1/4×5.5	M22.2 A325	277.7 477.10	277.7 477.10	277.7 477.10
8S9 (A36)	Type B	W8×21	W12×58	6×3.5×3/8×6.0 4×3.5×1/4×5.5	M22.2 A325	277.7 477.10	277.7 477.10	277.7 477.10
8S10 (A36)	Type B	W8×21	W12×58	6×3.5×1/2×6.0 4×3.5×1/4×5.5	M22.2 A325	277.7 477.10	277.7 477.10	277.7 477.10

f_{ya} = Yield stress of flange Cleats f_{yfc} = Yield stress of Column flange f_{yfb} = Yield stress of Beam flange
f_{ua} = Ultimate stress of flange Cleats f_{ufc} = Ultimate stress of Column flange f_{ufb} = Ultimate stress of Beam flange

Table 4 "Chen" data bank: geometrical and mechanical characteristics of the joints

TEST	Type of joint	Beam	Column	Flange angle Web angle	Type of bolt	f_{ya} f_{ua} [N/mm^2]	f_{yfc} f_{ufc} [N/mm^2]	f_{yfb} f_{ufb} [N/mm^2]
14S1 (A36)	Type B	W14×38	W12×96	6×4.0×3/8×8.0 4×3.5×1/4×8.5	M19.1 A325	285.40 480.40	285.40 480.40	285.40 480.40
14S2 (A36)	Type B	W14×38	W12×96	6×4.0×1/2×8.0 4×3.5×1/4×8.5	M19.1 A325	372.10 561.70	372.10 561.70	372.10 561.70
14S3 (A36)	Type B	W14×38	W12×96	6×4.0×3/8×8.0 4×3.5×1/4×5.5	M19.1 A325	285.40 480.40	285.40 480.40	285.40 480.40
14S4 (A36)	Type B	W14×38	W12×96	6×4.0×3/8×8.0 4×3.5×3/8×8.5	M19.1 A325	285.40 480.40	285.40 480.40	285.40 480.40
14S5 (A36)	Type B	W14×38	W12×96	6×4.0×3/8×8.0 4×3.5×1/4×8.5	M22.2 A325	277.7 477.10	277.7 477.10	277.7 477.10
14S6 (A36)	Type B	W14×38	W12×96	6×4.0×1/2×8.0 4×3.5×1/4×8.5	M22.2 A325	277.7 477.10	277.7 477.10	277.7 477.10
14S8 (A36)	Type B	W14×38	W12×96	6×4.0×5/8×8.0 4×3.5×1/4×8.5	M22.2 A325	277.7 477.10	277.7 477.10	277.7 477.10
14S9 (A36)	Type B	W14×38	W12×96	6×4.0×1/2×8.0 4×3.5×1/4×8.5	M22.2 A325	277.7 477.10	277.7 477.10	277.7 477.10

f_{ya} = Yield stress of flange Cleats f_{yfc} = Yield stress of Column flange f_{yfb} = Yield stress of Beam flange
f_{ua} = Ultimate stress of flange Cleats f_{ufc} = Ultimate stress of Column flange f_{ufb} = Ultimate stress of Beam flange

inelastic rotational predictions of the proposed model are much closer to the finite element model. The predictions of the proposed model are more consistent (covering the whole range of variation of the investigated parameter "d/t_a") than the results obtained by Eurocode 3, which does not take into account the effect of the d/t and r/t ratios in the evaluation of the lever arm. Finally, satisfactory results are obtained by the applications of the modified Eurocode 3. In fact, its application shows a good assessment only for values of the investigated parameter "d/t_a", variables from 1.1 to 2, while it overestimates the resistances for $d/t_a>2$ and underestimates in the case of $d/t_a<1.1$.

The proposed model is used to simulate the monotonic and cyclic behaviour of beam-column connections.

Herein the experimental curves contained in two data banks, and the Bernuzzi experimental tests (Bernuzzi *et al.* 1996) are considered.

The first is the one set up by Chen (Kishi and Chen 1986); the second was carried out within the European Cost Project. It is called "Sericon" (Weynand 1992).

In Fig. 15, the schemes of the different types of top and seat connections considered (type A and B) are reported. Joint type A does not include web angles while joint type B includes a double web angle connection. The geometric characteristics and the mechanical property of the studied connections are shown in Tables 1, 2, 3 and 4.

Figs. 16 to 18 show some comparisons among the experimental curves (Exp.), Mechanical model (MecMod), Eurocode 3 Annex J (EC3 (no web)) and the "modified" Eurocode 3 Annex J (EC3 (web) for top and seat & web angles, and EC3 (web+Hr) for top and seat & web angles plus hardening).

Eurocode 3 predictions underestimate the resistance and do not seem satisfactory, in terms of stiffness, because they sometimes both overestimate and underestimate it.

The "modified" Eurocode 3 application (top and seat with web cleats, top and seat with web cleats plus hardening) has shown a better accuracy, but it underestimates the resistance and it sometimes both overestimates and underestimates the stiffness.

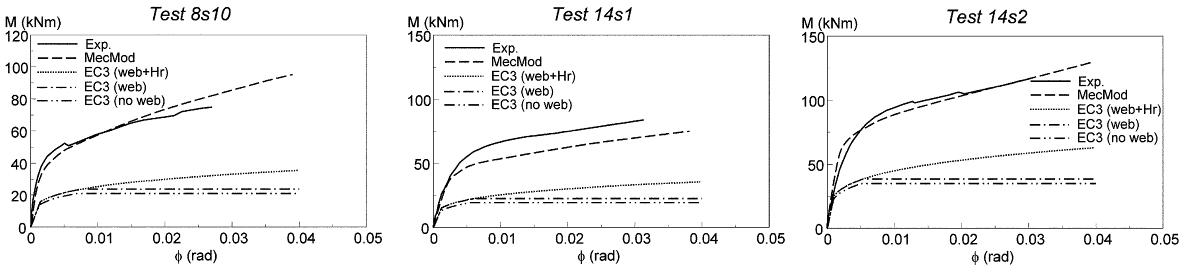


Fig. 16 Comparison among Annex J, MecMod and experimental "Chen" data

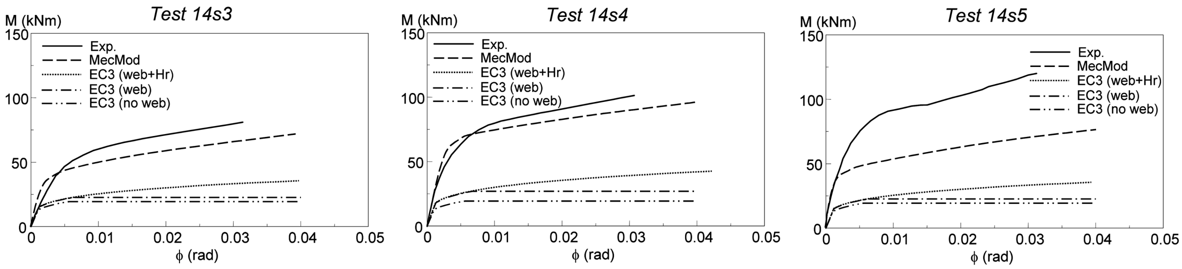


Fig. 17 Comparison among Annex J, MecMod and experimental "Chen" data

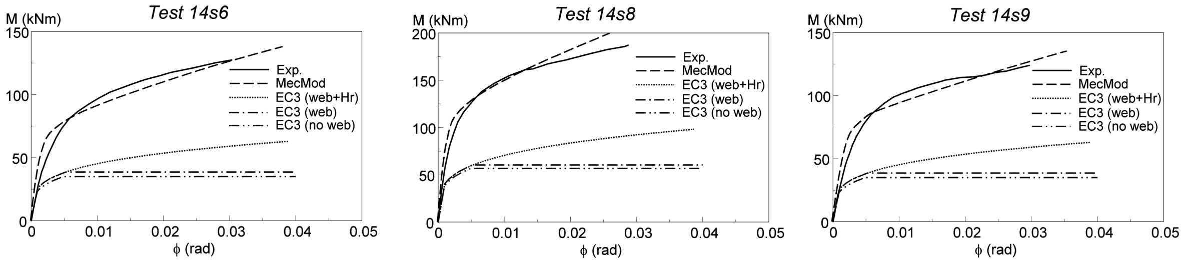


Fig. 18 Comparison among Annex J, MecMod and experimental “Chen” data

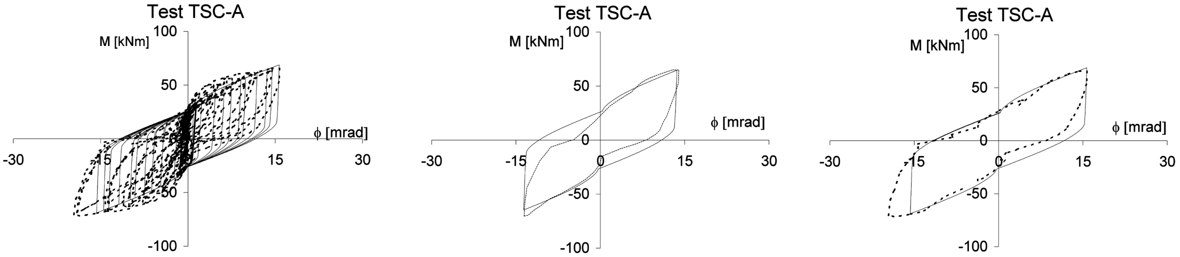


Fig. 19 Comparison among Annex J, MecMod. and experimental “Bernuzzi” curve

The use of the Eurocode 3, considering the web cleat contribution, produces an increment of about 30% of the strength. The application of Eurocode 3, considering the web cleat plus hardening contribution, has shown more confidence with the actual behaviour of the connections.

The application of the mechanical model (MecMod) has shown a better valuation of actual behaviour of the connections, especially regarding the prediction of the design moment resistance. The MecMod seems to predict the actual behaviour of different connections better.

In Fig. 19, momento-rotation curves of the MecMod are confronted with Bernuzzi experimental curves. The MecMod shows a good capability of simulating the actual cyclic behaviour of this type of connection. It is important to emphasize that the model, in this first stage, does not take into account the phenomena of stiffness and resistance degradation. In the same figures, where two different cycles obtained by the mechanical model are compared with two experimental cycles, it is possible to see that the model simulates, with sufficient approximation, the shape of the hysteresis cycle and that its valuation of actual behaviour of the connections is better, expecially regarding the prediction of the design moment resistance.

7. Conclusions

A cyclic mechanical model for the inelastic analysis of semirigid and partial-strength top and seat angle bolted connections is presented. The model is based on the same “component approach” introduced by the Eurocode 3. The proposed model still maintains the Eurocode 3 approach, but introduces a more refined modeling of the cleat-to-column interface. Moreover, it introduces a different expression for the evaluation of the moment capacity of the joint which takes into account the effect of the *d/t* (“*d* ” is the diameter of the bolt connecting the angle to the column flange, and “*t* ” is the angle thickness) and the *r/t* (“*r*” is the groove fillet radius). The proposed mechanical model is suitable to be included in the existing code for the analysis of framed MRSF's including this kind of joint. The comparison between the mechanical model and the experimental results has shown the ability of the proposed model to interpret with good approximation the actual behaviour of the connection both in terms of moment resistant and stiffness. Furthermore, the model is able to appraise with adequate approximation both initial and hardening stiffness even if it tends to overestimate the initial stiffness. The presented model can be improved in the future by adding the contribution of other "components", and by taking into account the main phenomena that interest these types of connections (cyclic degradation of the material, and sliding of bolts, among others).

The results of the conducted analyses show that the model is more accurate than the results of the Eurocode 3-Annex J model and of its modification. The model gives estimates of the required inelastic rotation demand that can be used to assess the actual joint inelastic capability.

The model is not suitable when the weakest joint component does not coincide with the angles.

References

Ballio, G., Calado, L., De Martino, A., Faella, C. and Mazzolani, F. M. (1987), "Cyclic behaviour of steel beam-to-column joints experimental research", *Costruzioni Metalliche*, **2**, 69-88.

Bernuzzi, C. (1995), "Cyclic response of components of steel connections". *Proc. of XV Congresso C.T.A.*, **2**.

Bernuzzi, C., Zandonini, R. and Zanon, P. (1996), "Experimental analysis and modelling of semi-rigid steel joints under cyclic reversal loading", *J. Construct. Steel Res.,* **38**(2), 95-123.

Bernuzzi, C. (1997), "Prediction of the behaviour of top-and-seat cleated steel beam-to-column connections under cyclic reversal loading", *J. Earthq. Eng.,* **2**(1), 25-58.

Bernuzzi, C., Calado, L. and Castiglioni, C. A. (1997), "Ductility and load carrying capacity prediction of steel beam-to-column connections under cyclic reversal loading", *J. Earthq. Eng*., **1**(2), 401-432.

Calado, L. and Ferriera, J. (1994), "Cyclic behaviour of steel beam-to-column connections – An experimental research", *Proc. of Behaviour of Steel Structures in Seismic Areas (STESSA'94)* Eds. F.M. Mazzolani and V. Gioncu, E & FN Spon, 381-389.

Calado, L. and Pucinotti, R. (1996), "Prove sperimentali su collegamenti trave-colonna in acciaio con saldatura a completa penetrazione", *Relatorio IC - IST, AI N°.4/96*, Departamento de Engenharia Civil, Lisboa.

Commission of the European Communities (1993). Eurocode 3: Design of Steel Structures.

Commission of the European Communities (1998). Eurocode 3, Annex j : Joints in Building Frames, ENV 1993 – 1 – 1: 1992/A2.

De Luca, A., De Martino, A., Pucinotti, R. and Puma, G. (1995), "(Semirigid) top and seat angle connections: Review of experimental data and comparison with eurocode 3", *Proc. of XV Congresso C.T.A.,* **2**, 315-336.

De Stefano, M. and Astaneh, A. (1991), "Axial force-displacement behaviour of steel double angles", *J. Construct. Steel Res.*, **20**, 161-181.

De Stefano, M. and De Luca, A. (1992), "Mechanical models for semirigid coconnections", *Proceedings, First World Conf. on Constructional Steel Design*, 276-279.

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

De Stefano, M., Bernuzzi, C., D'Amore, E., De Luca, A. and Zandonini, R. (1994b), "Semirigid top and seat cleated connection: A comparison between Eurocode 3 approach and other formulation," *Proc. International Workshop and Seminar on Behaviour of Steel Structures in Seismic Areas,* eds. F.M. Mazzolani and V. Goincu, Chapman & Hall, London, 568-579.

Faella, C., Piluso, V. and Rizzano, G. (2000), *Structural Steel Semirigid Connections – Theory, Design and Software.* CRC Press, Boca Raton, Florida.

Kishi, N. and Chen, W. F. (1986) "Database of steel beam-to-column connections", Structural Engineering Report, N° CE-STR-86-26, School of Civil Engrg., Purdue University.

Kishi, N. and Chen, W. F. (1990), "Moment-rotations of semirigid connections with angles", *J. Struct. Eng.*,

ASCE, **116**(7), 1813-1834.
Mele, E., Calado, L. and Pucinotti, R. (1997), "Indagini sperimentali sul comportamento ciclico di alcuni collegamenti in acciaio". *Proc. 8th Nat. Conf. Earthquake Engineering*. ANIDIS, Taormina, Italy.
Pucinotti, R. (1998), "I collegamenti nelle strutture in acciaio: analisi teoriche e sperimentali", Thesis for Ph.D. in Structures Engineering of the University of Catania and Reggio Calabria Italy.
Pucinotti, R. (2001a), "Top and seat & web angle connections: Prediction via mechanical model", *J. Construct. Steel Res.,* **57**(6), 663-696.
Pucinotti, R. (2001b), "Cyclic mechanical model for top and seat angle connections", *Proc. of XVIII Congresso C.T.A.*, **2**, 93-102.
Wales, M. W. and Rossow, E. C. (1983), "Coupled moment-axial force behaviour in bolted joints", *J. Struct. Eng.*, ASCE, **109**(5).
Weynand, K. (1992). "Sericon - Databank on joints building frames", *Proc. of the 1st COST C1 Workshop,* Strasburg.

DN