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Application of Eurocode 4 to blind bolted endplate composite joints with CFST columns

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ABSTRACT

In Eurocode 4 (EC4), design rules are given to predict the moment resistance and rotational stiffness of composite joints in braced frames for buildings subjected to static loading. Although these rules cover a wide range of composite joints, they are still limited to the joints in which the column has an open section. The design guidelines for the composite joint with a concrete-filled steel tubular (CFST) column is not available yet. Therefore, this paper aims to extend the application of the EC4 design rules to predict the mechanical properties of the blind bolted endplate composite joints with a CFST column. A design example was also presented to illustrate the applicability of the EC4 to the design of the blind bolted endplate composite joint with a CFST column.

1. INTRODUCTION

CFST structures have been increasingly used in multi-storey buildings due to their excellent performance such as high strength, high ductility and large energy absorption capability. In these buildings, the blind bolted endplate joints have been favourably used to connect composite beams to a CFST column because of their simplicity and economy in fabrication and assembly. A typical blind bolted endplate joint as shown in Fig. 1 consists of the endplate welded to the end of the steel beam. This assembly is then connected to a CFST column using the blind bolts which can be installed from the outer side of the steel tube. Since these joints exhibit the semi-rigid behaviour, they influence the response of a whole structure. Therefore, the effect of the actual semi-rigid behaviour of such joints should be considered in the analysis and design.

The behaviour of a joint is characterised by its moment-rotation curve which is represented by three main properties: (a) the initial rotational stiffness, (b) the moment resistance and (c) the rotation capacity. These properties can be predicted using the component method which was respectively adopted in EC3 Part 1-8 (2005) and EC4

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Part 1-1 (2005) for steel joints and composite joints. The component method allows a wide range of joint configurations to be covered by means of a unified procedure. The background of the component method and its application to steel joints can be found in Weynand et al. (1995), Jaspart (1996; 2002), Steenhuis et al.(1998), among others. At the moment, the design rules for the application of the component method are limited to the joints with open section columns (i.e. I- or H-sections). For the joint with hollow or CFST columns, the new so-called 'column face in bending' component is introduced and knowledge about its behaviour is required.

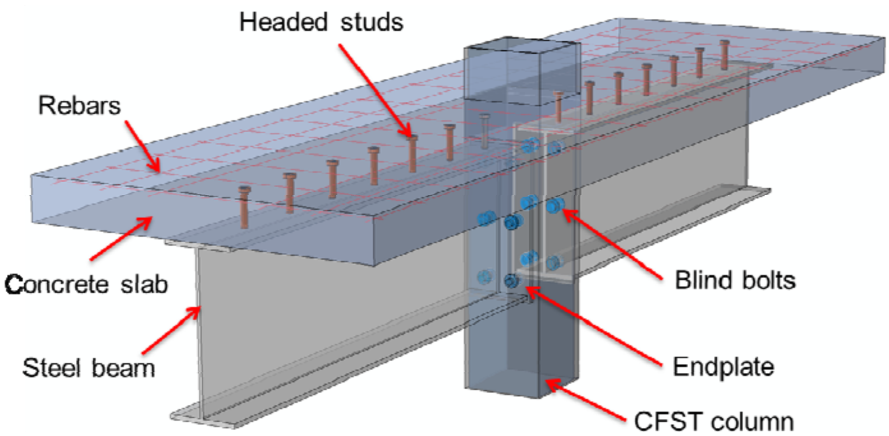


Fig. 1. A typical blind bolted endplate composite joint with a CFST column

This paper aims to extend the application of the component method to the blind bolted endplate composite joints with CFST column based on the rules given in EC4 (2005). The resistance and stiffness of the column face in bending component were obtained using the Neves and Gomes analytical model (Gomes et al. 1996; Neves and Gomes 1996) which was developed based on the study on the beam-to-column joints under minor-axis bending moment. It should be noted that EC4 does not provide any design rules to predict the rotation capacity of a joint. Therefore, this paper adopts the analytical method proposed by Anderson et al. (2000) to predict the rotation capacity.

2. APPLICATION OF EC4 TO THE CONSIDERED JOINT

2.1 Component Method

The basic principle of the component method is based on the mechanics of force transfer in joints. In the component method, a joint is considered as a set of individual basic components represented by springs. Each of these basic components possesses its own level of stiffness and resistance. Once the behaviour of each basic component is evaluated, the mechanical properties of the whole joint can be derived by assembling the contributions of all basic components based on a distribution of the internal forces within the joint. Fig. 2 shows the component model for the composite joint with a single row of bolts in tension. The notation for the spring stiffness coefficient is given in Table 1. It is noted that the stiffness coefficients k_1, k_2 and k_3 defined in EC3 Part 1-8 (2005) are assumed to be infinite due to the presence of infill concrete and the symmetric loading condition.

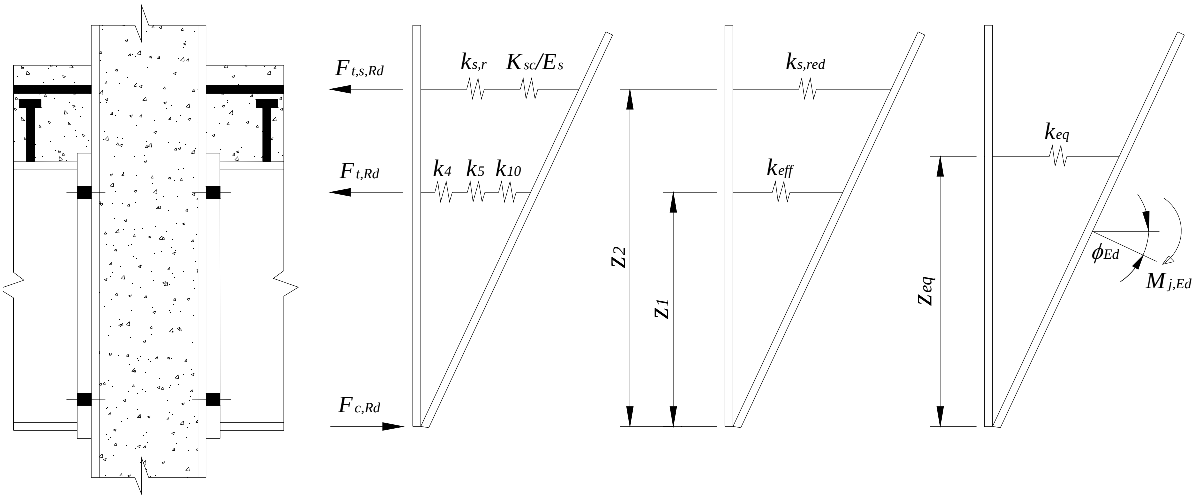


Fig. 2. Component model for the composite joint considered in this study

Table 1 Basic components of the composite joint with a CFST column

Component	Stiffness	References
Column face in bending	k_4	Neves and Gomes (1996)
Endplate in bending	k_5	Table 6.11 of EC 3 Part 1-8 (2005)
Bolt in tension	k_{10}	Table 6.11 of EC 3 Part 1-8 (2005)
Reinforcing bar in tension	$k_{s,r}$	EC4 Part 1-1 (2005)
Slip of shear connection	K_{sc} / E_s	EC4 Part 1-1 (2005)

2.2 Moment Resistance

The moment resistance is the product of the resistances of the weakest spring in a bolt row $F_{t,Rd}$ and the reinforcement $F_{t,s,Rd}$ with the corresponding lever arms z_1 and z_2 (see Fig. 2)

$$M_{j,Ed} = F_{t,Rd} z_1 + F_{t,s,Rd} z_2 \tag{1}$$

The calculation of $F_{t,Rd}$ was illustrated in Section 3 based on EC3 Part 1-8 (2005) and Gomes et al. (1996). The resistance of the reinforcement is expressed by

$$F_{t,s,Rd} = A_s f_{sy} \tag{2}$$

where A_s and f_{sy} are the area and yield stress of the reinforcing bar, respectively.

2.3 Initial Rotational Stiffness

The initial stiffness of the composite joint is obtained as

$$S_{j,\text{int}} = E_a k_{eq} z_{eq}^2 \tag{3}$$

where E_a is Young's modulus of the structural steel. The equivalent stiffness coefficient k_{eq} and corresponding lever arm z_{eq} are expressed as

$$k_{eq} = \frac{k_{eff} z_1 + k_{s,red} z_2}{z_{eq}} \tag{4}$$

$$z_{eq} = \frac{k_{eff} z_1^2 + k_{s,red} z_2^2}{k_{eff} z_1 + k_{s,red} z_2} \tag{5}$$

with k_{eff} and $k_{s,red}$ being respectively the stiffness coefficients of the bolt row and the reinforcement accounting for the slip of the shear connection. They are given as bellow

$$\frac{1}{k_{eff}} = \frac{1}{k_4} + \frac{1}{k_5} + \frac{1}{k_{10}} \tag{6}$$

$$\frac{1}{k_{s,red}} = \frac{1}{k_{s,r}} + \frac{1}{K_{sc} / E_s} \tag{7}$$

The stiffness coefficient of the reinforcement $k_{s,r}$ is given in Table A.1 of EC4 Part 1-1 (2005). For the case of the double-sided connection under balanced loading, $k_{s,r}$ is expressed as follow

$$k_{s,r} = \frac{A_s}{h_c / 2} \tag{8}$$

with h_c being the depth of the column. The elastic stiffness of the shear connection K_{sc} is given in section A.3 of EC4 Part 1-1 (2005) as

$$K_{sc} = \frac{N k_{sc}}{\nu - \dfrac{\nu - 1}{1 + \xi} \dfrac{z_2}{d_s}} \tag{9}$$

with

$$\xi = \frac{E_a I_a}{E_s A_s d_s^2} \quad \text{and} \quad \nu = \sqrt{\frac{(1+\xi) N k_{sc} \ell d_s^2}{E_a I_a}} \tag{10}$$

where N is the number of shear connectors distributed over the length ℓ of the beam in hogging bending which may be assumed to be 15 % of the span, k_{sc} is the stiffness of one shear connector determined by test or taken as 100 kN/mm for a 19 mm diameter headed stud if the test result is not available, $E_a I_a$ denotes the bending stiffness of the steel beam, d_s is the distance between the centroids of the steel beam and the reinforcement. The coefficient k_4 is given by Neves and Gomes (1996) as

$$k_4 = \frac{16t_c^3}{L^2} \frac{\alpha + (1-\beta)\tan\left(35^0 - 10^0 \beta\right)}{(1-\beta)^3 + \frac{10.4(1.50 - 1.63\beta)}{\mu^2}} \tag{11}$$

with $\alpha = c/L$, $\beta = b/L$, $\mu = L/t_c$. The definition of *b*, *c* and *L* is shown in Fig. 3. The resistance of the column face in bending component is given by Gomes et al. (1996) as

$$F_{4,Rd} = M_{pl} k \eta \tag{12}$$

where

$$M_{pl} = 0.25 f_y t_c^2 \tag{13}$$

$$\eta = \frac{4}{1-\beta}\left(\pi\sqrt{1-\beta} + 2\alpha\right) \tag{14}$$

$$k = \begin{cases} 1 & \text{if } (\alpha+\beta) > 0.5 \\ 0.7 + 0.6(\alpha+\beta) & \text{if } (\alpha+\beta) \le 0.5 \end{cases} \tag{15}$$

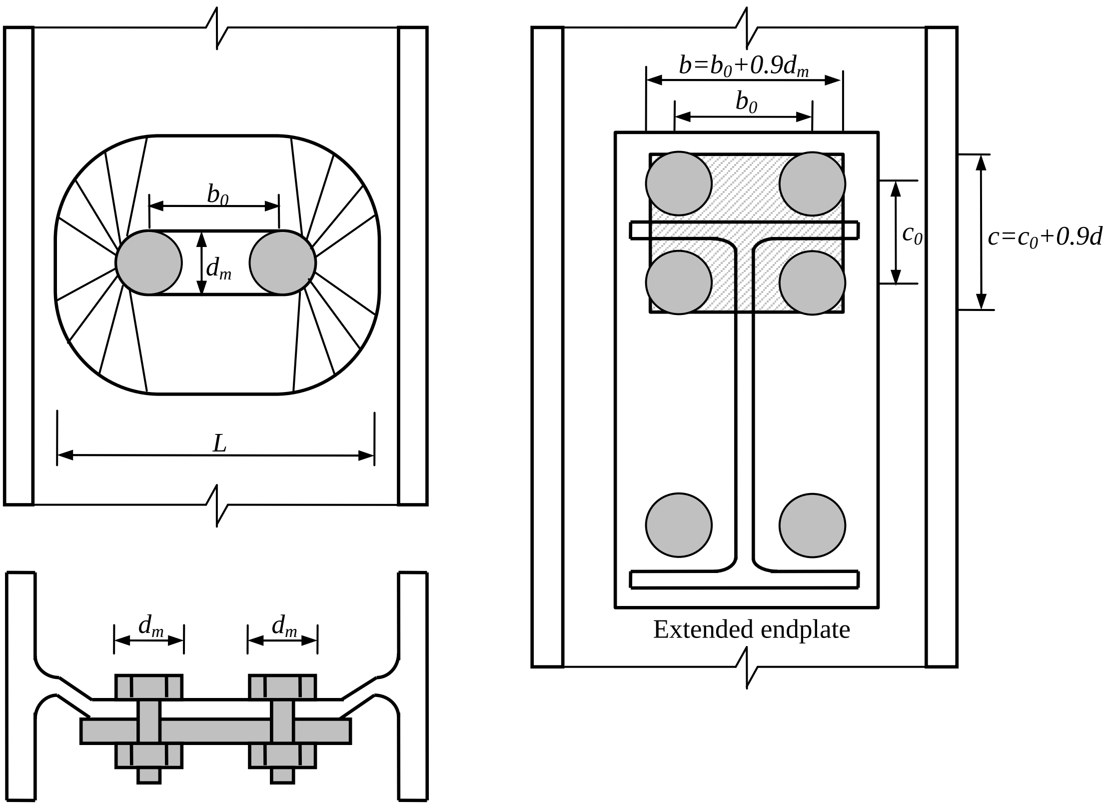


Fig. 3. Neves and Gomes (1996) model for minor-axis joint

2.4 Rotation Capacity

Based on the analytical method proposed by Anderson et al. (2000), the rotation capacity of composite joints resulted from the inelastic elongation of the reinforcement Δ_{us} and the slip of the shear connection s as

$$\phi_{Cd} = \frac{\Delta_{us}}{z_2} + \frac{s}{d_b} \tag{16}$$

where d_b is the depth of the steel beam. To account for the tension stiffening effect, a simplified stress-strain relationship for the embedded reinforcement given by CEB-FIP Model Code (1990) was used. The ultimate strength of the embedded reinforcement is expressed as

$$\varepsilon_{smu} = \varepsilon_{sy} - \beta_t \Delta\varepsilon_{sr} + \delta\left(1 - \frac{\sigma_{sr1}}{f_{sy}}\right)\left(\varepsilon_{su} - \varepsilon_{sy}\right) \tag{17}$$

where

$$\sigma_{sr1} = \frac{f_{ctm} k_c}{\rho}\left(1 + \rho\frac{E_s}{E_c}\right) \tag{18}$$

$$\Delta\varepsilon_{sr} = \frac{\sigma_{sr1}}{E_s} - \frac{f_{ctm}}{E_c} \tag{19}$$

$$\rho = \frac{A_s}{A_c} \text{ and } k_c = \frac{1}{1 + d/2z_0} \tag{20}$$

where d is the thickness of the concrete slab and z_0 is the vertical distance between the centroids of the uncracked, unreinforced concrete slab and the uncracked, unreinforced composite section; β_t is taken as 0.4 for short term loading and δ is taken as 0.8 for high ductility bars; f_{ctm} and E_c are respectively the tensile strength and elastic modulus of concrete; A_c and A_r denotes the areas of the concrete slab and the reinforcement, respectively; E_s is Young's modulus of the reinforcement. The inelastic elongation of the reinforcement Δ_{us} is determined based on the formulae proposed by Anderson et al. (2000) as follows

$$\Delta_{us} = 2L_t\varepsilon_{smu} \text{ if } \rho < 0.8\% \tag{21a}$$

$$\Delta_{us} = \left(h_c/2 + L_t\right)\varepsilon_{smu} \text{ if } \rho \geq 0.8\% \text{ and } a_c < L_t \tag{21b}$$

$$\Delta_{us} = \left(h_c/2 + L_t\right)\varepsilon_{smu} + \left(a_c - L_t\right)\varepsilon_{smy} \text{ if } \rho \geq 0.8\% \text{ and } a_c \geq L_t \tag{21c}$$

where a_c is the distance from the face of the column to the first shear connector along the beam and L_t is the 'transmission' length given by Hanswille (1997) as follows

$$L_t = \frac{k_c f_{ctm} \phi_r}{4\tau\rho} \tag{22}$$

where ϕ_r is the diameter of the reinforcement and τ is the average bond stress along the transmission length taken as $1.8 f_{ctm}$. The slip of the shear connection is

$$s = 2s_{el} \frac{F_{scu}}{F_{scel}} \tag{23}$$

where

$$s_{el} = \frac{0.7P_R}{k_{sc}} \tag{24}$$

$$F_{scel} = K_{sc} s_{el} \tag{25}$$

$$F_{scu} = A_s f_{sy} \tag{26}$$

where P_R is plastic resistance of one shear connector which is equal to 106 kN for a 19 mm diameter headed stud based on the test conducted by Loh et al. (2006).

2.5 Moment-Rotation Relationship

To account for the inelastic behaviour of the joint, the initial rotational stiffness $S_{j,\text{int}}$ is reduced when the applied moment $M_{j,Ed}$ exceeds two-thirds of the moment capacity $M_{j,Rd}$. The rotational stiffness is reduced as follow

$$S_j = \frac{S_{j,\text{int}}}{\left(1.5M_{j,Ed} / M_{j,Rd}\right)^{\psi}} \tag{27}$$

where ψ is the coefficient defining the shape of the moment-rotation curve and taken as 2.7 for the bolted endplate composite connection.

3. DESIGN EXAMPLE

A design example is presented to illustrate the application of the EC4 design rules to predict the mechanical properties of blind bolted endplate composite joints with CFST columns. The joints tested by Loh et al. (2006) were examined. The dimensions and properties of all specimens are given in Table 2. All specimens have the same geometric dimensions but vary in the number of shear connectors and reinforcements. The below calculation is illustrated for the specimen CJ1.

Table 2. Summary of specimens tested by Loh et al. (2006)

Specimens	CJ1	CJ2	CJ3	CJ4
Reinforcing bars (ratio, %)	4ϕ16 (1.29)	4ϕ16 (1.29)	4ϕ16 (1.29)	2ϕ16 (0.65)
Number of stud	5	3	2	3
First stud from column face (mm)	100	140	300	140
Degree of shear connection (%)	110	66	44	133
Steel column	200×200×9 mm, grade 350 h_c=200 mm, t_c = 9 mm			
Steel beam	250UB25.7 (248×124×8×5 mm), grade 350 d_b = 248 mm, b_b = 124 mm, t_w = 5 mm, t_f = 8 mm A_b = 3144 mm^2, EI = 6.76 MNm^2			
Concrete slab	d = 120 mm, b_{eff} = 515 mm, A_c = 61,800 mm^2 f_c = 17.5 MPa, f_{ctm} = 1.75MPa, E_c = 19,787 MPa Rebar: f_{sy} = 500 MPa, f_{su} = 600 MPa, e_{su} = 8%			
Joint	Bolt: M20 Grade 8.8 (see Fig. 4 for the arrangement) A_b = 245 mm^2, f_{by} = 640 MPa, f_{bu} = 800 MPa Endplate: 300×200×10 mm, grade 350			
Headed stud	ϕ19×100 mm, shear span length ℓ = 1250 mm k_{sc} = 100 kN/mm, P_k = 100 kN			

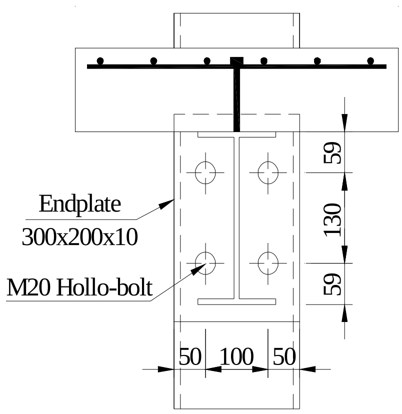


Fig. 4. Layout of bolt

3.1 Moment Capacity

Tensile resistance of each bolt row (consisting of two Hollo-bolts M20 Grade 8.8):

$$F_{b,Rd} = 2\left(k_2 f_{ub} A_b / \gamma_{M2}\right) = 2\left(0.9 \times 0.800 \times 245 / 1.25\right) = 282.24 \text{ kN}$$

Tensile resistance of the reinforcement:

$F_{t,s,Rd} = A_s f_{sy} = 804 \times 0.500 = 401.92$ kN

Resistance of the column face in bending component (Gomes et al. 1996):

$c = 29$ mm (hole diameter for the blind bolt M20)
$b = 29 + 100 = 129$ mm
$L = h_c - 2t_c = 200 - 2 \times 9 = 182$ mm
$\alpha = c / L = 0.16$, $\beta = b / L = 0.71$
$M_{pl} = 0.25 \times 0.350 \times 9^2 = 7.09$ kN

$$\eta = \frac{4}{1 - 0.71}\left(\pi\sqrt{1 - 0.71} + 2 \times 0.16\right) = 27.66$$

$\alpha + \beta > 0.5 \rightarrow k = 1$
$F_{4,Rd} = 7.09 \times 1 \times 27.66 = 196.07$ kN

Resistance of the endplate in bending component (EN 1993-1-8 2005):

$$m = \frac{100 - 4}{2} - 0.8 \times 8 = 41.1 \text{ mm}, \quad m_2 = 59 - 8 - 0.8 \times 8 = 44.6 \text{ mm}, \quad e = 50 \text{ mm}$$

$$\lambda_1 = \frac{m}{m + e} = \frac{41.1}{41.1 + 50} = 0.45$$

$$\lambda_2 = \frac{m_2}{m + e} = \frac{44.6}{41.1 + 50} = 0.49$$

$\rightarrow \alpha = 6.05$ (from Fig. 6.11 of EC3 Part 1-8 (2005))
$l_{eff,1} = 2\pi m = 258.24$ mm
$l_{eff,2} = \alpha m = 248.66$ mm

Mode 1

$l_{eff} = \min\left(l_{eff,1}, l_{eff,2}\right) = 248.66$ mm
$M_{pl,1} = 0.25 \times l_{eff} t_p^2 f_{yp} = 0.25 \times 248.66 \times 10^2 0.350 = 2175.70$ kNmm

$$\mathrm{F}_{5,Rd,1} = 4\frac{M_{pl,1}}{m} = 211.75 \text{ kN}$$

Mode 2

$l_{eff} = l_{eff,2} = 248.66$ mm, $n = \min\left(e, 1.25m\right) = 50$ mm
$M_{pl,2} = 0.25 l_{eff} t_p^2 f_{yp} = 2175.70 \text{kNmm}$

$$\mathrm{F}_{5,Rd,2} = \frac{2M_{pl,2} + nF_{b,Rd}}{m + n} = \frac{2 \times 2175.7 + 50 \times 282.24}{41.1 + 50} = 202.67 \text{ kN}$$

Mode 3

$\mathrm{F}_{5,Rd,3} = F_{b,Rd} = 282.24$ kN
$\mathrm{F}_{5,Rd} = \min\left(F_{5,Rd,1}, F_{5,Rd,1}, F_{5,Rd,1}\right) = 202.67$ kN
$\mathrm{F}_{t,Rd} = \min\left(F_{4,Rd}, F_{5,Rd}\right) = 196.07$ kN (column face in bending governed)

Moment resistance of the joint:

$$M_{j,Ed} = F_{t,Rd} z_1 + F_{t,s,Rd} z_2 = 196.07 \times 0.185 + 401.92 \times 0.334 = 170.51 \text{ kNm}$$

3.2 Initial Stiffness

Stiffness coefficient of the reinforcement:

$$k_{s,r} = \frac{A_s}{h_c / 2} = \frac{804}{200 / 2} = 8.04 \text{ mm}$$

Stiffness of the shear connection:

$$\xi = \frac{E_a I_a}{E_s A_s d_s^2} = \frac{6.76 \times 10^{12}}{200,000 \times 804 \times 214^2} = 0.92$$

$$\nu = \sqrt{\frac{(1+\xi) N k_{sc} \ell d_s^2}{E_a I_a}} = \sqrt{\frac{(1+0.92) 5 \times 10^5 \times 1250 \times 214^2}{6.76 \times 10^{12}}} = 2.85$$

$$K_{sc} = \frac{N k_{sc}}{\nu - \frac{\nu - 1}{1+\xi} \frac{h_s}{d_s}} = \frac{5 \times 100}{2.85 - \frac{2.85 - 1}{1+0.92} \frac{334}{214}} = 371.92 \text{ kN/mm}$$

Stiffness coefficient of the rebar accounting for the slip in shear connection:

$$k_{s,red} = \frac{k_{s,r}}{1 + E_s k_{s,r} / K_{sc}} = \frac{8.04}{1 + 200,000 \times 8.04 / 371,920} = 1.51 \text{ mm}$$

Stiffness coefficient of the column face in bending component:

$$\mu = L / t_c = 182 / 9 = 20.22$$

$$k_4 = \frac{16 t_c^3}{L^2} \frac{\alpha + (1-\beta) \tan\left(35^0 - 10^0 \beta\right)}{(1-\beta)^3 + \frac{10.4(1.50 - 1.63\beta)}{\mu^2}} = 3.30 \text{ mm}$$

Stiffness coefficient of the endplate in bending component:

$$l_{eff} = \min\left(l_{eff,1}, l_{eff,2}\right) = 248.66 \text{ mm}$$

$$k_5 = 0.9 l_{eff} \left(t_p / m\right)^3 = 0.9 \times 248.66 (10 / 41.1)^3 \, 3.22 \text{ mm}$$

Stiffness coefficient of the bolt in tension:

$$k_{10} = \frac{1.6 A_b}{L_b} = \frac{1.6 \times 245}{44} = 8.91 \text{ mm}$$

Effective stiffness coefficient of a bolt row:

$$k_{eff} = \frac{1}{\frac{1}{k_4}+\frac{1}{k_5}+\frac{1}{k_{10}}} = \frac{1}{\frac{1}{3.30}+\frac{1}{3.22}+\frac{1}{8.91}} = 1.38 \text{ mm}$$

Equivalent stiffness coefficient of the joint:

$$z_{eq} = \frac{k_{eff} z_1^2 + k_{s,red} z_2^2}{k_{eff} z_1 + k_{s,red} z_2} = \frac{1.38\times185^2 + 1.51\times334^2}{1.38\times185 + 1.51\times334} = 283.97 \text{ mm}$$

$$k_{eq} = \frac{k_{eff} z_1 + k_{s,red} z_2}{z_{eq}} = \frac{1.38\times185+1.51\times334}{283.97} = 2.67 \text{ mm}$$

Initial stiffness of the joint:

$$S_{j,\text{int}} = E_a k_{eq} z_{eq}^2 = 43.13 \text{ kNm/mrad}$$

3.3 Rotation Capacity
Elongation of the reinforcement:

$$\rho = \frac{A_s}{A_c} = \frac{804}{120\times515} = 0.013$$

$$k_c = \frac{1}{1+\frac{d}{2z_0}} = \frac{1}{1+\frac{120}{2\times181.52}} = 0.75$$

$$\sigma_{sr1} = \frac{f_{ctm}k_c}{\rho}\left(1+\rho\frac{E_s}{E_c}\right) = \frac{1.75\times0.75}{0.013}\left(1+0.013\frac{200,000}{19,787}\right) = 114.41 \text{ MPa}$$

$$\Delta\varepsilon_{sr} = \frac{\sigma_{sr1}}{E_s} - \frac{f_{ctm}}{E_c} = 0.000484$$

$$\varepsilon_{sy} = \frac{f_{ys}}{E_s} = \frac{500}{200,000} = 0.0025, \quad \varepsilon_{su} = 8\%, \quad \beta_t = 0.4, \quad \delta = 0.8$$

$$\varepsilon_{smu} = \varepsilon_{sy} - \beta_t\Delta\varepsilon_{sr} + \delta\left(1-\frac{\sigma_{sr1}}{f_{ys}}\right)\left(\varepsilon_{su}-\varepsilon_{sy}\right) = 0.05$$

$$L_t = \frac{k_c f_{ctm}\phi_r}{4\tau\rho} = \frac{0.75\times1.75\times16}{4\left(1.8\times1.75\right)0.013} = 128.40 \text{ mm}$$

$a_c = 100 < L_t$ and $\rho = 0.13 \geq 0.8\,\%$

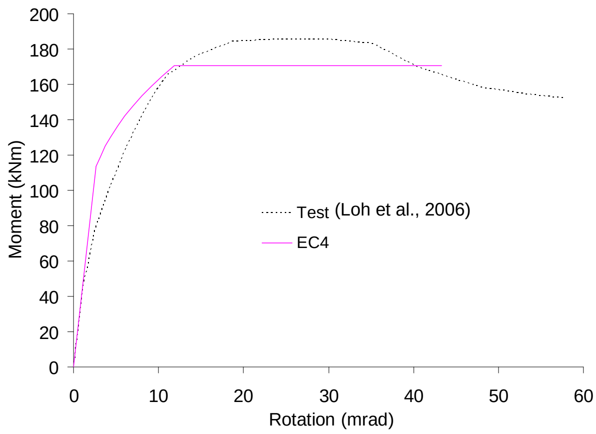
$$\rightarrow \Delta_{us} = \left(0.5h_c + L_t\right)\varepsilon_{smu} = \left(0.5\times200+128.40\right)0.05 = 11.45 \text{ mm}$$

Slip of the shear connection:

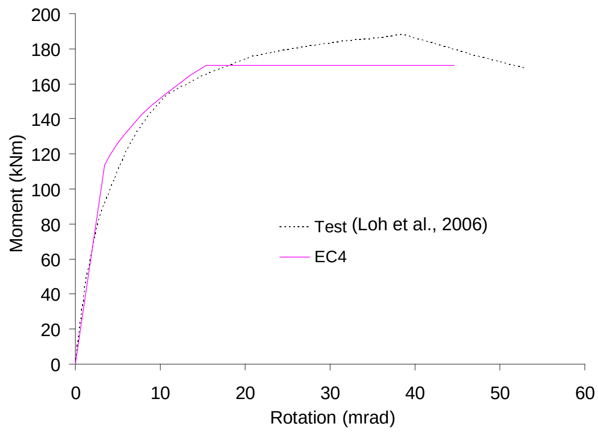
$$s_{el} = \frac{0.7P_R}{k_{sc}} = \frac{0.7 \times 100}{100} = 0.7\,\text{mm}$$

Table 3 Comparison between EC4 predictions and experimental results (Loh et al. 2006)

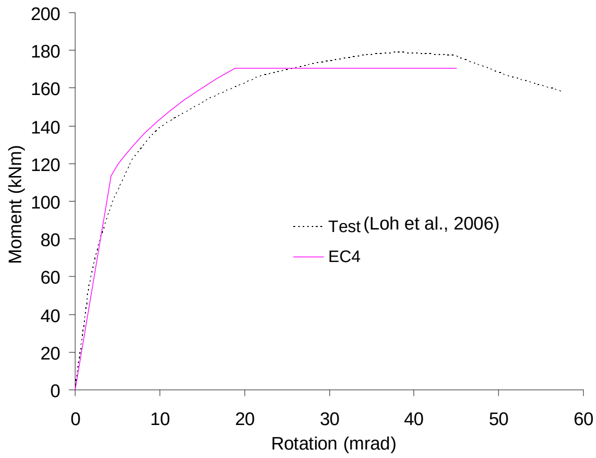
Specimen	Initial stiffness (kNm/mrad)			Moment resistance (kNm)		
	EC4	Test	EC4/Test	EC4	Test	EC4/Test
CJ1	43.13	40.00	1.08	170.51	185.80	0.92
CJ2	33.15	38.30	0.87	170.51	187.90	0.91
CJ3	26.95	33.30	0.81	170.51	178.90	0.95
CJ4	25.14	32.50	0.77	103.39	143.30	0.72



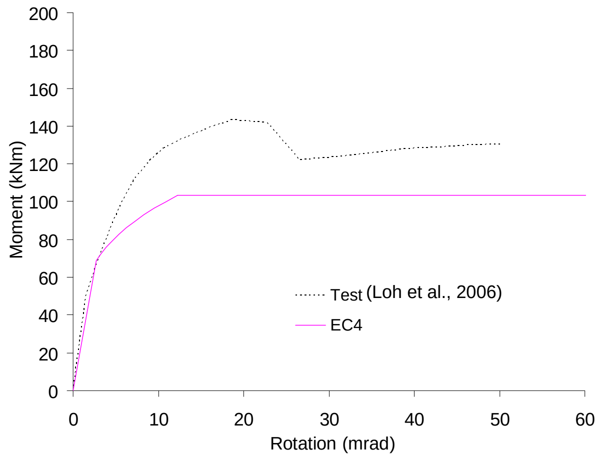
(a) CJ1



(b) CJ2



(c) CJ3



(d) CJ4

Fig. 5. Comparison of moment-rotation curves of four specimens

$$F_{scel} = K_{sc} s_{el} = 371.920 \times 0.7 = 260.34 \text{ kN}$$
$$F_{scu} = A_s f_{sy} = 401.92 \text{ kN}$$
$$s = 2 s_{el} \frac{F_{scu}}{F_{scel}} = 2 \times 0.7 \frac{401.92}{260.34} = 2.16 \text{ mm}$$

Rotation capacity of the joint:

$$\phi_{Cd} = \frac{\Delta_{us}}{z_2} + \frac{s}{d_b} = \frac{11.45}{334} + \frac{2.16}{248} = 0.0343 + 0.0087 = 0.04313 \text{ rad} = 43.13 \text{ mrad}$$

The obtained predictions for initial rotational stiffnesses and moment resistances of four specimens were compared with the experimental results reported by Loh et al. (2006) in Table 3. The comparison of the moment-rotation curves of four specimens was also plotted in Fig. 5. In general, the component method introduced in EC4 predict rather well the initial stiffness and moment resistance of all composite joint considered in this study, except for the case of the specimen CJ4.

4. CONCLUSIONS

The application of the component method adopted by EC4 has been extended in this paper to the blind bolted endplate composite joints with CFST columns. A new basic component named 'column face in bending' was introduced for the studied joints. The resistance and stiffness of this basic component were obtained from the Gomes and Neves model which was developed for the beam-to-column minor-axis joints. The present model has been validated by comparing the obtained predictions with experimental results tested by Loh et al. (2006) at the University of New South Wales. It can be seen that the present model predicts rather well both the initial rotational stiffness and moment resistance of the considered joint.

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REFERENCES

Anderson, D.,J. M. Aribert,H. Bode and H. J. Kronenburger (2000), "Design rotation capacity of composite joints" *Structural Engineer*, **78**(6), 25-29.
CEB-FIB (1990), *CEB-FIP Model Code, Design Guide 1990*, Thomas Telford.
EN 1993-1-8 (2005), *Eurocode 3: Design of steel structures-Part 1.8: Design of joints*.

EN 1994-1-1 (2005), *Eurocode 4: Design of composite steel and concrete structures-Part 1-1: General rules and rules for buildings*.

Gomes, F.,J. P. Jaspart and R. Maquoi (1996), "Moment capacity of beam-to-column minor-axis joints" *IABSE Colloquium on Semi-Rigid Structural Connections*, 319-326.

Hanswille, G. (1997), "Cracking of concrete mechanical models of the design rules in Eurocode 4" *Composite Construction in Steel and Concrete, ASCE*, 421-433.

Jaspart, J. P. (1996), "Design of steel connections according ENV 1993-1" *IABSE Congress on Structural Engineering in Consideration of Economy, Environment, Energy, Workshop on Structural Connections and Assembly Techniques*, 961-966.

Jaspart, J. P. (2002), "Design of structural joints in building frames" *Progress in Structural Engineering and Materials*, **4**(1), 18-34.

Loh, H. Y.,B. Uy and M. A. Bradford (2006), "The effects of partial shear connection in composite flush end plate joints Part I-experimental study" *J. Constr. Steel Res.*, **62**(4), 378-390.

Neves, L. C. and F. Gomes (1996), "Semi-rigid behaviour of beam-to-column minor-axis joints" *IABSE Colloquium on Semi-Rigid Structural Connections*, 207-218.

Steenhuis, M.,J. P. Jaspart,F. Gomes and T. Leino (1998), "Application of the component method to steel joints" *Proceedings of the Control of the Semi-Rigid Behaviour of Civil Engineering Structural Connections Conference, COST C1, Liege, Belgium*, 125-143.

Weynand, K.,J. P. Jaspart and M. Steenhuis (1995), "The stiffness model of revised Annex J of Eurocode 3" *Proceedings of the 3rd International Workshop on Connections*, 441-452.