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Analytical Study of End-Plate Connection on Cruciform Column Section

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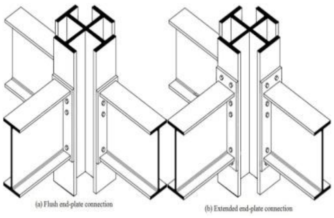
Graphical abstract



Abstract

This paper presents an analytical study on flush end-plate (FEP) and extended end-plate (EEP) connections connected to cruciform column section using component method. The objective of this study is to predict the moment resistance and initial stiffness of FEP and EEP connections on cruciform column section. A series of FEP and EEP connections are tested in laboratory. The connection tests consist of four FEP and four EEP specimens with different configuration. Component method outlined in the publication of Steel Construction Institute and British Constructional Steelwork Association are based on BS5950 and Eurocode 3 (EC3) are used to predict the moment resistance and initial stiffness of the tested specimens. The experimental results are then used to validate the analytical predictions. As compare to the experimental results, all moment resistance of the connections coincide well with analytical predictions. Analytical prediction for initial stiffness using EC3 does not show good agreement with the experimental results. This study shows that the component method can be used to predict the moment resistance of FEP and EEP connections on cruciform column section. Further study need to be carried out for initial stiffness to obtain accurate analytical representation.

Keywords: Flush end-plate; extended end-plate; component method; moment resistance; initial rotational stiffness

1.0 INTRODUCTION

The usage of build-up section is not new in engineering construction. Many researchers have studied on build-up section such as girders and columns. The fabrication and connection on build-up sections are always problematic and increase construction cost and time.

Cruciform column is made up of two universal beam sections where one of the beams is cut into half and welded to the other beam. By joining the two universal beams increase the cross sectional area of the column and hence increase the compression resistance. The fabrication of cruciform column is simpler as compared to other build-up column such as laced column and battened column. Therefore the construction cost and time may also be reduced [1-3].

One of the most important parts in designing steel structures is the connection between members. Connection is the location where two or more elements meet. A steel structure can perform its best to the desired function when the connections between members are design adequately [4]. The fabrication cost of connections are high in steel structures. Optimum connection design is often the main aim for researchers to save cost and avoid wastage [5].

There are three important structural properties need to be identified in connections which are the strength, stiffness and ductility. By having the moment resistance and initial stiffness, the moment rotation curve can be plotted. The moment rotation curve represents the characteristics of connections which are able to indicate the connection properties in terms of strength, stiffness and ductility [6-11]. Experimental tests provide the most accurate information to evaluate the reliability of empirical, analytical, mechanical and finite element models [12].

This study focused on the analytical prediction of steel connection calculated in accordance to the guideline published by Steel Construction Institute (SCI) and British Constructional Steelwork Association (BCSA) using BS5950 and EC3 [13, 14]. The component method is used to study the moment resistance and initial stiffness of EEP and FEP connections. A series of FEP and EEP connections were tested experimentally to validate the analytical predictions. The arrangement of the test specimens are build-up hybrid beam sections connected to cruciform column using either FEP or EEP connections as shown in Figure 1.

2.0 ANALYTICAL STUDY

2.1 Connection Configurations

Eight specimens consist of four FEP connections N1 to N4 and four EEP connections N5 to N8. All 8 specimens are using build-up beam sections with four different sizes connected to cruciform column section using end-plate connections. The configurations of the connections are shown in Table 1.

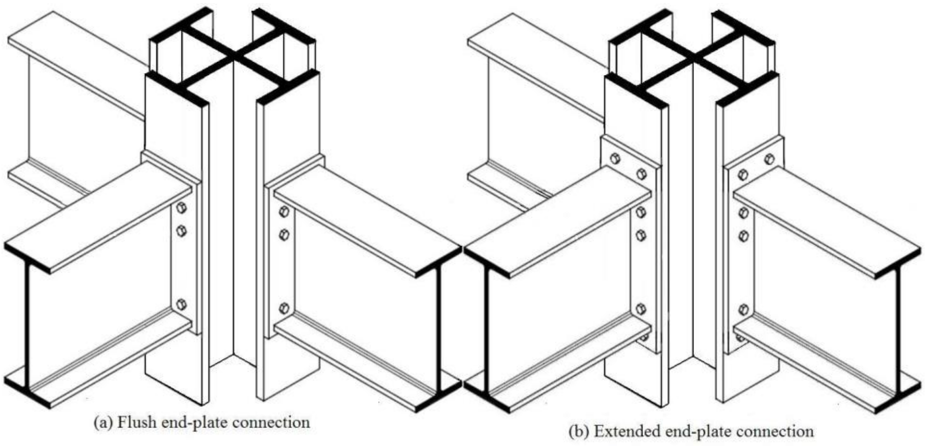


Figure 1 Flush end-plate and extended end-plate connection on cruciform column section [3]

Table 1 Configuration of FEP and EEP connections

Specimen	Connection Type	Welded Beam Size	Column Size	Bolt Row (T-B)	End Plate Size	Bolt Size
N1	FEP	400×140×41.13/12/5	533x210x16 CCUB	1(2-2)	200×12	20
N2	FEP	500×180×63.59/16/5			200×12	20
N3	FEP	450×160×46.86/12/5		2(4-4)	200×12	20
N4	FEP	600×200×85.91/16/6			200×12	20
N5	EEP	400×140×41.13/12/5		2(4-2)	200×12	20
N6	EEP	500×180×63.59/16/5			250×15	24
N7	EEP	450×160×46.86/12/5		3(6-4)	200×12	20
N8	EEP	600×200×85.91/16/8			250×15	24

2.2 Component Method

The component method for determining the moment resistance of the connection was based on the design procedures outlined in the publication of SCI and BCSA using BS5950 [13] and EC3 [14]. The component in connection is break down into its individual components which consist of tension zone, compression zone, horizontal shear zone and vertical shear zone as shown in Figure 2 and Table 2.

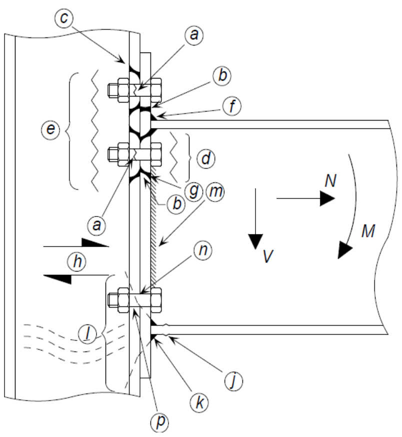


Figure 2 Component method design check on different zones [14]

Table 2 Component method design check on different zones [14]

Zone	Reference	Item
Tension	*a*	Bolt tension
	b	End-plate bending
	c	Column flange bending
	d	Beam web tension
	e	Column web tension
	f	Flange to end-plate weld
	g	Web to end-plate weld
Horizontal Shear	*h*	Column web panel shear
Compression	*j*	Beam flange compression
	k	Beam flange weld
	l	Column web
Vertical Shear	*m*	Web to end-plate weld
	n	Bolt shear
	p	Bolt bearing (plate or flange)

The component method starts with the calculation of the resistance of bolt rows. Bolt resistance for each individual bolt row in the tension zone is calculated. In calculating the bolt resistance, the capacity of column flange bending, column web in tension, end-plate bending and beam web in tension are calculated. Resistance of group bolt rows are then considered using the same procedures as individual bolt row. The least value from the calculation is considered as the control value for the resistance of each bolt row.

The bottom flange of the beam is checked under the compression zone. The compression capacity is determined from the least value of beam flange compression, column web crushing and column web buckling.

In shear zone, the shear is acting in two directions, horizontal and vertical. However, horizontal shear is usually the most critical and is depending on whether it is one sided or two sided connection. For one sided connection, the horizontal shear force is equal to the compression force at the bottom flange of the beam. For vertical shear, the capacity is calculated as the reduced capacity of bolt rows acting in the tension zone while full shear values for bolt rows acting in the shear zone.

Theoretically, the forces in tension beam flange is equivalent to the forces in compression beam flange, while the horizontal shear force is equal to the compression force at bottom of beam flange. Based on this assumption, the least value calculated in tension, compression and shear is taken as the capacity of the connection. Then, mode of failure can be predicted. The moment resistance of the connection is calculated from:

$$M_{j.Rd.EC3} = \sum_{i=1,2}^{n} \left(F_{ri} \times h_i \right) \tag{1}$$

where F_{ri} is the resistance of bolt row in the tension zone,
h_i is distance of the
i^{th} bolt row from the centre of compression
n is the number of bolt rows.

2.2 Initial Stiffness

The initial stiffness of a connection can be calculated based on the provision given in Eurocode 3: Part 1-8 Design of Steel Structures [4]. The initial stiffness of a joint is determined from the flexibilities of its basic components represented by an elastic stiffness coefficient k_i. The formulation for the initial stiffness of a joint is given by:

$$S_j = \frac{Ez^2}{\mu \sum_i \frac{1}{k_i}} \tag{2}$$

where k_i is the stiffness coefficient for basic joint component *i*,
z is the lever arm
μ is the stiffness ratio $S_{j,ini} / S_j$.

The initial stiffness can be calculated from the expression by letting $\mu = 1.0$. The stiffness coefficient k_i is calculated for each component in the critical zones such as the column web in shear, tension and compression zones (k_1, k_2 and k_3), column flange in tension zone (k_4), bolts in tension zone (k_5), and end-plate in tension zone (k_6).

■3.0 RESULTS AND DISCUSSION

3.1 Component Method

The moment resistance for all specimens are calculated using the component method. Table 3 is an example of calculated component method on specimen N4 using EC3. The lesser value in the component of a bolt row will be the bolt row resistance. Failure mode can be predicted from the analytical calculation. From the table, specimen N4 is predicted to fail at column flange.

Table 3 Component evaluation of specimen N4 using Eurocode 3

Specimen (N4)	Component	F_{Rd} (kN)	$M_{j.Rd.EC3}$ (kNm)
Row 1 [a]	End plate in bending Column flange in bending Beam web in tension Column web in tension	282.2 250.3 585.3 691.1	221.6
Row 2 [a]	End plate in bending Column flange in bending Beam web in tension Column web in tension	282.2 250.3 585.3 691.1	
Row 1 and row 2[a] (in group)	Column flange in bending Column web in tension	200.0 760.8	
Column web in transverse compression		687.6	
Beam flange and web in compression		1492	

[a] No reduction on effective resistance of bolt row when sum of effective design tension resistance is less than compressive resistance of a joint.

3.2 Comparison between Experimental and Analytical Results in Terms of Moment Resistance

The moment resistance and initial stiffness for all specimens are calculated based on the component method. The experimental results are plotted in moment rotation curves and the important values such as moment resistance and initial stiffness are extracted from the curves and summarized in Table 4 and Table 5. The comparison between the analytical and experimental results are shown in Table 4.

From the comparison, it shows that the moment resistance obtained from analytical model coincide well with the test results where the moment resistance lies between the ratios of 0.99 to 2.04 for BS5950 and 0.98 to 1.79 for EC3. The ratio for FEP is in the range of 1.22 to 2.04 for BS5950 and 1.19 to 1.79 for EC3. For the extended end-plate lies in the range of 0.99 to 1.61 for BS5950 and 0.98 to 1.41 for EC3. As compared to EC3, moment resistance obtain from BS5950 is more conservative. The highest percentage difference between EC3 and BS5950 is 12.76%. Both design guidelines results are closely match.

The failure modes of all tested specimens are either end-plate yielding or column flange yielding followed by the bolt slippage. According to analytical model from the component method, the failure mode are end-plate yielding for specimens with end-plate thickness of 12mm while specimens with end-plate thickness of 15mm fail in column flange. In this study, the failure modes are govern by the end-plate and column flange. The failure will occur at column flange when the thickness of end-plate is higher. Therefore it is suggested that the thickness of column flange thickness should be higher than the end-plate to avoid failure at column.

From the result, the analytical model by component method can be used to calculate the moment resistance and predict the failure mode of the proposed end-plate connections.

3.3 Initial Stiffness

Along the uncut universal beam section is term as major axis of cruciform column section. The other universal beam section is cut into half and welded to the uncut section is termed as welded axis. On the welded axis the section is not continuous and jointed by fillet weld. In calculating the initial stiffness, the depth of the column, h_c is assumed half of the universal beam depth, h_b for the welded axis.

In Table 5 shows the comparison of initial stiffness with h_c is taken as h_b and half of h_b. The initial stiffness for the welded axis, depth of cruciform column using half of the h_b are closer to the experimental value. This shows that the assumption made is reasonable. Therefore, in predicting the initial stiffness for welded axis, the depth of the column web, h_c is taken as half of the depth of universal beam web, h_b.

The ratios of experimental results to analytical initial stiffness lie in the range of 0.49 to 1.39. The ratios are in the range of 0.64 to 1.07 for FEP connections and 0.49 to 1.39 for EEP connections. Analytical model based EC3 is not suitable to predict the initial stiffness due to the limitation of the connected column are not the standard size column. Further analysis are required to study the actual behaviour of the proposed connection.

Table 4 Comparison between analytical and experimental results for moment resistance

Specimen	$M_{j,max}$ (kNm)	$M_{j,Rd.BS5950}$ (kNm)	Ratio[a]	$M_{j,Rd.EC3}$ (kNm)	Ratio[b]	$M_{j,Rd.EC3}$ / $M_{j,Rd.BS5950}$ (%)
N1	149.5	73.3	2.04	83.6	1.79	12.3
N2	193.7	105.0	1.85	108.1	1.79	2.92
N3	205.4	135.2	1.52	154.9	1.33	12.76
N4	263.9	216.9	1.22	221.6	1.19	2.12
N5	222.3	138.5	1.61	158.1	1.41	12.41
N6	279.5	282.3	0.99	285.5	0.98	1.11
N7	304.9	209.0	1.46	233.2	1.31	10.38
N8	492.7	358.3	1.38	386.4	1.28	7.29

[a] Ratio of experimental moment resistance, $M_{j,max}$ to analytical moment resistance based on BS5950 $M_{j,Rd.BS5950}$.
[b] Ratio of experimental moment resistance, $M_{j,max}$ to analytical moment resistance based on EC3 $M_{j,Rd.EC3}$.

Table 5 Comparison between analytical and experimental results for initial stiffness

Specimen	$S_{j,ini}$	$S_{j,ini.EC3,h}$	$S_{j,ini.EC3,h/2}$	Ratio[a]	Ratio[b]	Ratio[c]	Axis
N1	17.2	18.7	28.8	0.92	0.60	0.92	Major
N2	55.2	32.9	51.6	1.68	1.07	1.07	Welded
N3	25.8	26.3	41.6	0.98	0.62	0.98	Major
N4	58.0	56.9	90.9	1.02	0.64	0.64	Welded
N5	23.5	32.8	50.5	0.71	0.46	0.71	Major
N6	125.6	56.4	90.3	2.23	1.39	1.39	Welded
N7	25.4	48.7	77.0	0.52	0.33	0.52	Major
N8	74.0	92.8	149.6	0.80	0.49	0.49	Welded

[a] Ratio of experimental inital stiffness, $S_{j,ini}$ to analytical initial stiffness calculated from $h_c = h_b$.
[b] Ratio of experimental inital stiffness, $S_{j,ini}$ to analytical initial stiffness calculated from $h_c = h_b / 2$.
[c] Predicted analytical initial stiffness.

3.3 Performance of FEP and EEP

Comparison between the performance of FEP and EEP connections using EC3 are shown in Table 6. The comparison is made to validate the performance between FEP specimen N1 to N4 and EEP specimen N5 to N8. The beam size, column size and bolt configuration for N1 with N5, N2 with N6, N3 with N7 and N4 with N8 are identical except for EEP specimens have an extra bolt row above the beam flange.

The ratios of moment resistance between EEP and FEP are from 1.51 to 2.64 while the ratios of initial rotational stiffness are 1.65 to 1.85. The extra bolt rows in extended end-plate greatly increase the performance of the connections in terms of strength and stiffness.

Table 6 Comparison between flush end-plate and extended end-plate connections

Specimen	Connection Type	Moment Resistance EC3 (kNm)	Ratio[a]	Initial Stiffness (kNm/mRad)	Ratio[b]
N1 (FEP) N5 (EEP)	FEP	83.6	1.89	18.7	1.75
	FEP	158.1		32.8	
N2 (FEP) N6 (EEP)	FEP	108.1	2.64	51.6	1.75
	FEP	285.5		90.3	
N3 (FEP) N7 (EEP)	EEP	154.9	1.51	26.3	1.85
	EEP	233.2		48.7	
N4 (FEP) N8 (EEP)	EEP	221.6	1.74	90.9	1.65
	EEP	386.4		149.6	

[a] Ratio of moment resistance of EEP to FEP.
[b] Ratio of initial rotational stiffness for EEP to FEP.

■4.0 CONCLUSIONS

The main conclusions that can be drawn from this study are:

1. The component method adopted in this study shows reasonable agreement with the experimental results in terms of moment resistance for FEP and EEP. The ratio between analytical values and experimental values are in the range of 0.99 to 2.04 for BS5950 and 0.98 to 1.79 for EC3.
2. Both BS5950 and EC3 design guidelines results are closely match and can be used to predict the moment resistance of the proposed cruciform column end-plate connections. The highest percentage difference between EC3 and BS5950 is 12.76%.
3. The initial stiffness calculated using Eurocode 3: Part 1.8 Design of Steel Structures shows the ratio between analytical values and experimental values are in the range of 0.49 to 1.39.
4. EC 3: Part 1.8 can be used to predict initial stiffness of cruciform column end-plate connections with the assumption of the depth of cruciform column for welded axis is taken as half of the depth of universal beam.
5. The performance of EEP connection is higher than the FEP connection in terms of strength and stiffness with the same beam depth and connection configurations. The ratios of moment resistance between EEP and FEP are from 1.51 to 2.64 while the ratios of initial rotational stiffness are 1.65 to 1.85.

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References

[1] Tahir, M. M. and P. N. Shek. 2005. Performance of Cruciform Column Using Universal Beam Sections under Axial Compression Load. *Jurnal Teknologi*. 43(B): 51–66.

[2] Tahir, M. M., P. N. Shek, A. Sulaiman and C. S. Tan. 2009. Experimental Investigation of Short Cruciform Columns Using Universal Beam Sections. *Construction and Building Materials*. 23: 1354–1364.

[3] Shek, P. N., M. M. Tahir, C. S. Tan and A. B. H. Kueh. 2011. Experimental Investigation of End-Plate Connection with Cruciform Column Section. *Advanced Materials Research*. 250–253: 3730–3733.

[4] British Standard Institution. 2005. Eurocode 3: *Design of Steel Structures, Part 1-8: Design of Joints.* London: BSI.

[5] Tahir, M. M., P. N. Shek and C. S. Tan. 2009. Push-Off Tests on Pin-Connected Shear Studs with Composite Steel-Concrete Beams. *Construction and Building Materials*. 23(9): 3024–3033.

[6] Yee, Y. L. and R. E. Melchers. 1986. Moment Rotation Curves for Bolted Connections. *Journal of Structural Engineering*. 112(3): 615–635.

[7] Azizinamini and J. B. Radziminski. 1989. Static and Cyclic Performance of Semi-Rigid Steel Beam-to-Column Connections. *Journal of Structural Engineering*. 115(12): 2979–2999.

[8] Bjorhovde, R., A. Colson and J. Brozzetti. 1990. Classification System for Beam-to-Column Connections. *Journal of Structural Engineering*. 116(11): 3059–3076.

[9] Abolmaali, A., J. H. Matthys, M. Farooqi and Y. Choi. 2005. Development of Moment-Rotation Model Equations for Flush End-Plate Connections. *Journal of Structural Steel Research*. 61: 1595–1612.

[10] Shi, Y., G. Shi and Y. Q. Wang. 2007. Experimental and Theoretical Analysis of the Moment Rotation Behaviour of Stiffened Extended End-Plate Connections. *Journal of Constructional Steel Research*. 63: 1279–1293.

[11] Lee Y. H., C. S. Tan, Y. L. Lee, M. M. Tahir, S. Mohammad and P. N. Shek. 2013. Numerical Modelling of Stiffness and Strength Behaviour of Top-Seat Flange-Cleat Connection for Cold-Formed Double Channel Section. *Applied Mechanics and Materials*. 284–287: 1426–1430.

[12] Faella, C., V. Piluso and G. Rizzano. 2000. *Structural Steel Semirigid Connections: Theory, Design and Software.* CRC Press LLC.

[13] Steel Construction Institute and British Constructional Steelwork Association. 1996. *Joints in Steel Construction–Volume 1: Moment Connections.* Berkshire: SCI.

[14] Steel Construction Institute and British Constructional Steelwork Association. 2013. *Joints in Steel Construction–Moment-Resisting Joints to Eurocode3*. Berkshire: SCI.