Innovative connection for joining I -beam to square hollow section column

Fathy A. Abdelfattah
Faculty of Eng., at Shoubra, Zagazig University, Cairo, Egypt

This paper presents simple and practical moment connection proposed for connecting I beam to Square Hollow Section (SHS) columns. The connection is characterized by its easy fabrication and erection. Normal high strength bolts are used and access from inside the column hollow section is not required. Nonlinear finite element analysis was used to model the behavior of the connection. The finite element model was verified. The efficiency of the connection was investigated and proved. The study extends to investigate the behavior of each part of the connection when different dimensions and details are considered. The interaction of the different parts and their influence on the overall behavior of the connection is studied. Three different yielding mechanisms are proposed. Recommendations and guidelines are suggested for the design of the connection. The results show that the connection can be proportioned to provide a rigid full strength connection.

يقترح البحث وصلة بسيطة و عملية لربط كمره ذات قطاع على شكل I مع عمود ذا قطاع أنبوبى على شكل مربع. تتميز الوصلة بسهولة تصنيعها و تركيبها و تستخدام فيها المسامير عالية المقاومة. أستخدمت طريقة العناصر المحدوده للتمثيل العددى لسلوك الوصله المقترحة. تم مقارنة النتائج المستخرجة من هذة الطريقة مع النتائج المستخرجة من تجارب معملية منشورة سابقا. تم بيان مدى كفاءة الوصله المقترحة. شمل البحث دراسة سلوك الأجزاء المختلفة المكونة للوصلة وتأثير كل منها على سلوك الوصله. تم اقتراح ثلاثة اشكال يمكن أن تحدث فى لوح الأرتكاز بالوصلة تصل فيها قيمة الأجهادات الى حد الخضوع. تم أقتراح توصيات و طريقة لتصميم الوصلة. النتائج أوضحت انة يمكن تصميم الوصلة لتكون ركيزة جسئه و لها مقاومة اعلى من تلك الخاصة بالكمره المتصله بها.

Keywords: Connections, SHS columns, RHS columns

1. Introduction

Structural steel hollow sections are used as columns in multi-storey buildings due to the efficiency of the closed shape [1]. However, problems appear when making moment connections to beams of I - sections. Bolting is not commonly applied due to the difficulty of gaining access to inside of the column section. Fully welding the connection would overcome the problem. However, welding is not an attractive option in site considering quality control and economy. Picard et al. [2] investigated the case where wide flange beam is used. Flanges' forces were transferred to column webs or to the opposite beam's flange through strap angle. Tabuchi et al. [3] studied the case when exterior diaphragms welded to the beam's flanges. Dawe et. al. [4] tested connections at which tension and compression plates with web clip angles or tension plates and seat angles welded to doubler plate to reinforce column's walls. Linderman et al. [5] tested fully welded connections between wide flanges beam and built up box column with and without internal continuity plates. Shanmugam et al. [6] studied connections where beam flanges are smaller than column flange. Several forms of external stiffeners are proposed to stiffen beams' flanges and provide width to it equal to that of column flange. Failure occurs due to yielding of column flange. If, on the other hand, the connection is stiffened, it may fail by buckling or yielding of column webs or beam flanges.

Innovative bolting techniques are presented in the literature. Maquoi et al. [7] proposed connection where threaded studs were welded onto the flange of the hollow column. This is seen not to be practical, as the threads require protection during erection. Kato et al. [8] investigated the case where special circular threaded nuts were welded to the column flange through conical holes. In the 1990 different types of high strength blind bolts are developed [9]. The main concept of it, is the ability of installation and tightening from one side of the connection. Holes diameter for blind bolts should be more restricted than that of ordinary bolts to permit proper upsetting of the sleeve on the blind side. Barnett et al. [10] compared experimentally between the behavior and strength of different kinds of blind bolts. In Japan, a special shaped high strength bolts called “Paddle bolts” are used. Tanaka et al. [11] studied experimentally moment connections where this type is implemented. Koral et al. [9] compared experimentally between the behavior of moment connections when normal high strength bolts of grade A325 and blind bolts are used. High local forces are induced at bolts position. This caused large deformations in the relatively thin column flange. For limit state design methods, the flexibility of column flange will limit the connection moment capacity, as it should satisfy serviceability and ultimate deformation limits [12].

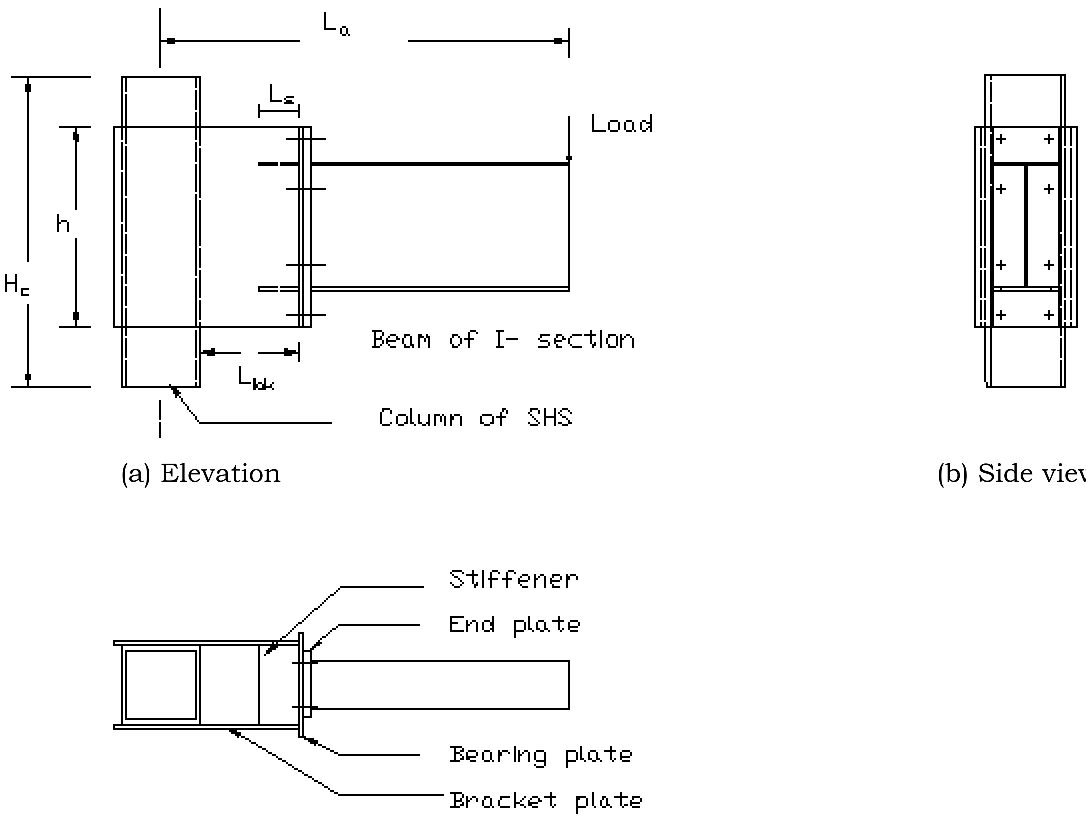
2. The proposed connection

This paper presents simple and practical moment connection for connecting I beam to columns of square or rectangular hollow sections. The connection involves welding two steel plates to column web walls as brackets. Another plate, called herein bearing plate, is fillet welded perpendicularly to the brackets at their ends, fig. 1. An extended end plate is fillet welded to the I beam. Both the bearing plate and the extended end plate are then bolted. The bracket plates provide space between the Square Hollow Section (SHS) column flange and the bearing plate. This space should be enough for adding horizontal stiffeners at the back of the bearing plate coinciding with the beam flanges. Bolts are inserted through this space. Bolts are tightened from the end plate side. The simplicity of the connection is to make welding in workshop and bolting in site having the optional of good quality control of welding and economy and efficiency due to the use of normal high strength bolts.

The connection is considered to consist of two main parts, the bearing plate – end plate joint and the bracket plates. Nonlinear finite element analysis was used to model the behavior of the connection. First, the finite element model was verified. Then, the efficiency of the connection was investigated and proved. The study extends to investigate the behavior of each part of the connection when different dimensions and details are considered. The interaction behavior of the different parts and their influence on the overall behavior of the connection is studied. Three different modes of yielding mechanisms are proposed for the bearing plate. Recommendations and guidelines are suggested for the design of the connection.

3. Finite element analysis

Nonlinear finite element analysis was performed to model the connection behavior. The finite element package ANSYS 5.4 [13] was used. Because of the symmetry about X – Y plane, only half of the connection and the joined beam were modeled, fig. 2. Four nodes isoparametric shell element was used. Each node has 6 degrees of freedom; translations in the X, Y and Z directions and rotations about the X, Y and Z axes of the nodal. The element formulation includes plasticity and large strain capabilities. All the nodes in the X – Y plane were restrained against translation in the Z-axis and against rotation about the X and Y axes. The nodes of the bearing plate and end plate were generated in coinciding patterns. Bolts holes were modeled 2 mm larger than bolt diameter. Eight pairs of coinciding nodes in the end plate and bearing plate defined the circumference of each bolt’s hole. The counterpart nodes in each pair were connected to each other by spring. The translation stiffness of the spring satisfies the load-separation relationship of A325 bolt of 19 mm diameter obtained from the experimental work of Koral et al. [9]. The bolt bending stiffness was not included in the model. At welds positions, nodes were generated in coincide pairs. The welds itself were not modeled but its’ action was modeled by coupling the degrees of freedom of each pair of coinciding nodes at welds positions. Concentrated vertical load was applied at the end of the beam. This subjects the connection to bending moment and shear force. Nonlinear



(a) Elevation (b) Side view

(c) Plane (d) Isometric

Fig. 1. Details of the proposed connection.

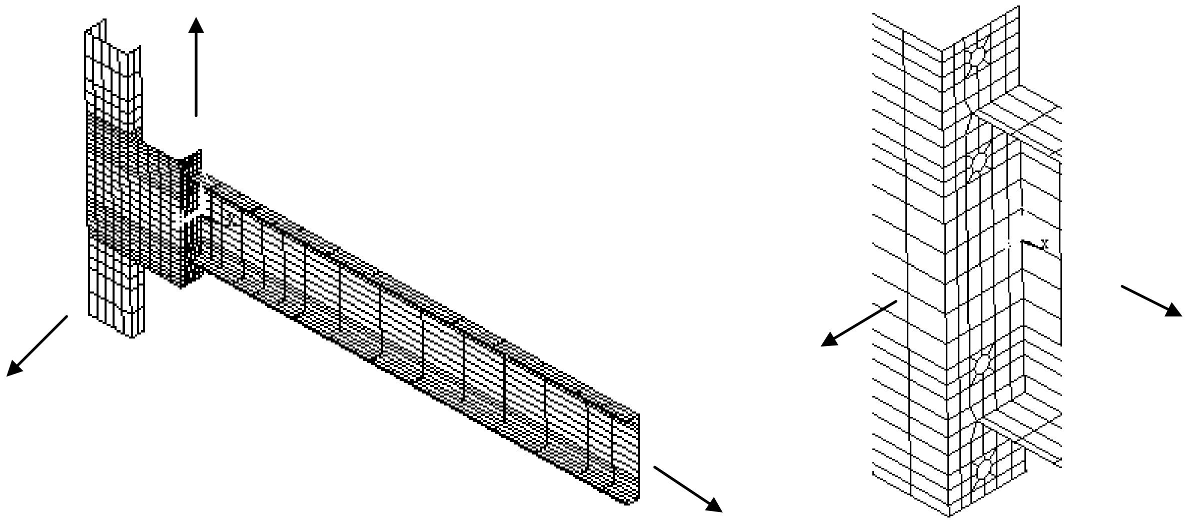


Fig. 2. Finite element mesh.

stress – strain relationship was implemented to model the steel materials. The bilinear kinematic hardening option available in ANSYS [13] was used. It uses the Von Mises yield criteria and describes steel plasticity. Strain hardening behavior is included assuming that the total stress range is equal to twice the yield stress.

4. Contact mechanism

Initially the analysis was performed while the translations in the X direction of every two coincide nodes in the bearing plate and the end plate was coupled. The results were reviewed for tension reaction between coupled nodes. This indicates that the two nodes intend to separate. Uncoupling these nodes satisfies this condition. The analysis was repeated and the results were reviewed again. The deflections in the x direction of every uncoupled coincide nodes should not violate the actual behavior. This may occur when the deflections of the end plate nodes in the negative X direction become more than those of the bearing plate nodes or vice versa. At this case, coincide nodes were coupled again. This is to insure that one plate does not penetrate the other one. Prying forces would induce. These processes of repeating the analysis, reviewing the results and modifying the coupling conditions between coincide nodes were continued until none of the results violate the actual behavior.

5. Details of the specimens

For all the specimens studied, the beam section was chosen to be W 360 X 33 complying with CAN/CSA-S16.1-94 [14]. This is a class 1 section recommended for plastic design. The nominal plastic moment of the beam section is M_p = 162 kN.m. The top and bottom flanges of the beam were stiffened by cover plates of 6 mm thickness for a length of 500 mm at its joint with the end plate. This was made to insure linear elastic behavior of the beam as the study highlights on the connection behavior. The dimensions of the column hollow section, bracket plates, bearing plates and end plates are listed in tables 1, 2, 3 and 4. The steel used for the beam, end plates, bearing plates and bracket plates was in accordance with CAN/CSA G40.21- M300W steel with minimum yield strength of 300 MPa [15]. The column section complies with CAN/CSA G40.21-M89 class H with minimum yield strength of 350 MPa [15]. Eight Bolts of 19 mm diameter and grade A325 were used, fig. 1. The details listed above are typical to those considered in the experimental work of reference [9]. This facilitated the comparison of the experimental and numerical results and the verification of the finite element model.

6. Comparison of experimental and finite element results

The finite element model is used to model the behavior of specimen S1 of ref. [9]. The moment – rotation relationship is obtained and compared to that of the experimental results in ref. [9], fig. 3. The two relationships are nearly typical up to a moment of 100 kN.m. Above this value and up to M_p, the two relations deviate due to the limited experimental results available at this segment. Above M_p the experimental results show stiffer behavior until failure. The differences between the relations refer to the following. The nominal value of F_y specified in the code [15] is used in the finite element model because the actual value is not listed in ref. [9]. This made the finite element results conservative in comparison to the experimental results.

Table 5 presents the yield moment and the moment resistance of the connection. The yield moment M_y is defined herein as the intersection of the initial stiffness of the connection and the strain hardening stiffness. Wheeler et al. [16] used this criterion. It was difficult to define the ultimate moment, as the moment-rotation relationship curve obtained from the finite element results does not show a pronounced peak. This is because the stress – strain curve of the steel material is modeled using a bilinear relation. Failure of Specimen S1 occurred at a rotation of 0.05 [9]. The moment at this rotation $M_{.05R}$ was obtained and considered as the connection moment resistance. The values of the ratios $(M_y/M_{.05R})_{F.E.}$ and $(M_y/M_{.05R})_{exp}$ are 0.91 and 0.87 respectively. Generally, the experimental and finite element results show good agreement.

Table 1
Effect of using different stiffening details

Spec.	Specimen properties*	Stiffeners at**		F. E. results		M_{By}/M_y	M_{Ey}/M_y
		Bearing plate	End plate	M_y/M_p	$M_{.05R}/M_p$		
A0	t_e =19.0 mm t_b = 12.7 mm	---	---	0.72	0.86	1.11	1.88
A1	t_s = 10.0 mm t_{bk} = 12.7 mm	HC + HT	---	0.98	1.22	1.0	1.38
A2	L_a = 2400mm L_{bk} = 250.0 mm	HC	---	0.75	0.95	1.06	1.78
A3	L_s =100.0 mm h = 530.0 mm	HC + HT + VT	---	1.03	1.32	1.43	1.31
A4	H_c = 1200.0 mm F_y = 300 N/mm^2	HC + HT	VT	1.03	1.28	0.95	---
A5	8 bolts of 19 mm diameter and grade A325	HC + HT + VT	VT	1.32	1.63	1.12	---

*Beam section size is W360 X 33 and column section is SHS 203X203X12.7
**HC + HT indicate horizontal stiffeners coinciding with beam flanges in the compression and tension regions of the connection respectively. VT indicates vertical stiffener at the tension region of the connection.

Table 2
Effect of using different values for end plate thickness

Spec.	Specimen properties*		F. E. results		M_{By}/M_y	M_{ey}/M_y
	Con. elements	t_e	M_y/M_p	$M_{.05R}/M_p$		
A6	t_b = 12.7 mm t_s = 10.0 mm	12.0	0.55	0.74	1.76	0.97
A7	t_{bk} = 12.7 mm L_a = 2400mm L_{bk} = 250.0 mm	15.0	0.81	1.05	1.20	1.03
A1	L_s =100.0 mm h = 530.0 mm H_c = 1200.0 mm	19.0	0.98	1.22	1.0	1.38
A8	F_y = 300 N/mm^2 HC + HT ** 8 bolts of 19 mm diameter and grade A325	25.0	1.05	1.31	0.93	1.28

*Beam section size is W360 X 33 and column section is SHS 203X203X12.7
**HC and HT indicate the use of horizontal stiffeners coinciding with the beam flanges at the back of the bearing plate.

Table 3
Effect of using different values for bearing plate thickness

Spec.	Specimen properties*		F. E. results		M_{By}/M_y	M_{ey}/M_y
	Con. Elements	t_b	M_y/M_p	$M_{.05R}/M_p$		
A1	t_e = 19.0 mm t_s = 10.0 mm t_{bk} = 12.7 mm L_a = 2400mm L_{bk} = 250.0 mm L_s =100.0 mm h = 530.0 mm H_c = 1200.0 mm F_y = 300 N/mm^2 HC + HT ** 8 bolts of 19 mm diameter and grade A325	12.7	0.98	1.22	1.0	1.38
A9		15.0	1.18	1.45	1.15	1.14
A10		19.0	1.32	1.60	1.66	1.02
A11		25.0	1.33	1.65	2.85	1.02

*Beam section size is W360 X 33 and column section is SHS 203X203X12.7 **HC + HT indicate the use of horizontal stiffeners coinciding with the beam flanges at the back of the bearing plate.

Table 4
Effect of using different values for bracket plates' thickness

Spec.	Specimen properties				F. E. results	
	Con. elements	Beam	Column	t_{bk}	M_y/M_p	$M_{.05R}/M_p$
A12	t_e = 19.0 mm t_b = 12.7 mm t_s = 10.0 mm L_a = 2400mm L_{bk} = 250.0 mm L_s =100.0 mm h = 530.0 mm H_c = 1200.0 mm F_y = 300 N/mm^2 HC + HT* 8 bolts of 19 mm diameter and grade A325	W 360 X 33 F_y =300 N/mm^2	SHS 203x203x12.7 F_y =350 N/mm2	8.0	0.89	1.13
A13				12.7	0.95	1.22
A14				15.0	0.99	1.23
A15				19.0	1.0	1.23

*HC + HT indicate the use of horizontal stiffeners coinciding with the beam flanges at the back of the bearing plate.

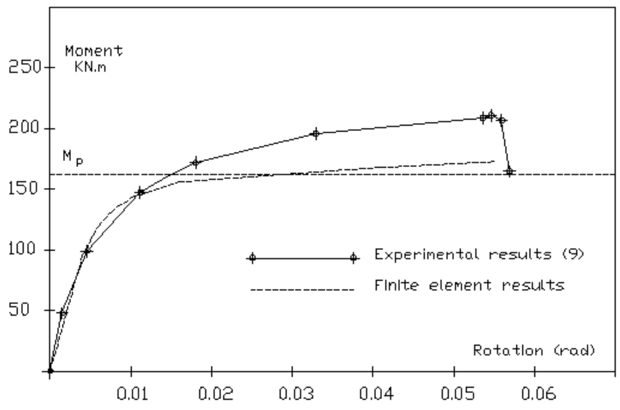


Fig. 3. Comparison of the finite element and experimental results of specimen S1 in [9].

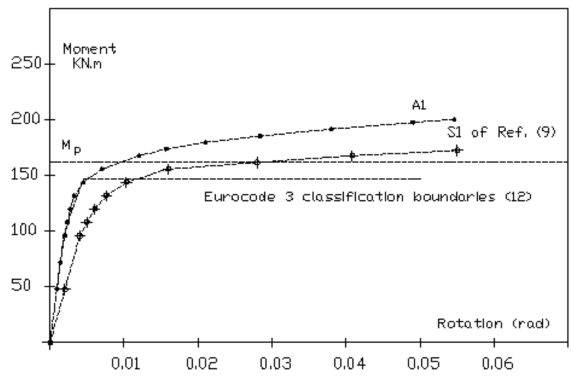


Fig. 4. Comparison between the finite element results of specimens S1 in [9] and A1 of table 1.

Table 5
Comparison between experimental and finite element results

Results of	M_y	$M_{.05\,R}$
F. E.	156 kN. m.	171 kN. m.
Experimental	178 kN. m.	205 kN. m.
$M_{F.E.}$ / $M_{exp.}$	87.6 %	83.4 %

7. efficiency of the proposed connection

Fig. 4 compares between the moment-rotation relationships obtained from the finite element analysis for specimen S1 of ref. [9] and specimen A1 of table 1. In specimen S1, the beam was welded to extended end plate, which is then bolted to the flange of the SHS column. Two stiffeners were used inside the column section. Specimen A1 is typical to the proposed connection in this study, fig. 1. The bearing plate was stiffened at its back by two horizontal stiffeners. Specimens S1 and A1 have the same restraining conditions and the same dimensions for the different details. The thickness of the bearing plate and bracket plates in A1 was taken equal to the thickness of the column web walls. Specimen A1 shows stiffer behavior and higher moment resistance than that of S1. The connections behaviors were classified according to the Euro code 3 [12]. The boundaries of moment-rotation relationships specified in the code [12] were calculated and superimposed on fig. 4. S1 is classified, as a semi-rigid-full strength connection while A1 is a rigid-full strength connection. The rotation capacity if A1 need not be checked as the connection moment resistance exceeds 1.2 times the design plastic moment resistance of the beam [12]. This is not satisfied in the results of S1.

8. End plate – bearing plate joint

This section presents the behavior of the bearing plate and end plate. The bearing plate provides flexible support to the end plate. The effects of using different stiffener details and different values for the end plate and bearing plate thickness are studied. The column was rigidly clamped at its back. The results are presented in terms of the moment-rotation relationships. The joint rotation R_j is calculated, according to the Euro code [12], at the intersection of the beam and the end plate. The rotations of the bracket plates R_b and the column R_c were not included.

8.1. Stiffeners details

Specimens A0, A1, A2, A3, A4 and A5 have the same dimensions but different stiffeners details, table 1. No stiffeners were used in A0. Stiffeners were used only at the back of the bearing plate in specimens A1, A2 and A3. One horizontal stiffener is provided in A2 coinciding with the beam flange at the compression region of the joint. Two horizontal stiffeners are provided coinciding with the beam flanges in A1 and A3. Vertical stiffener was added in A3 coinciding with the

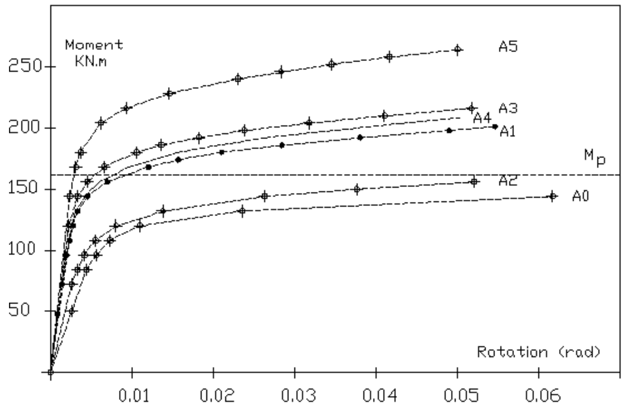


Fig. 5. Moment-rotation relationships of specimens A0 to A5 of table 1 at which different stiffening details are used.

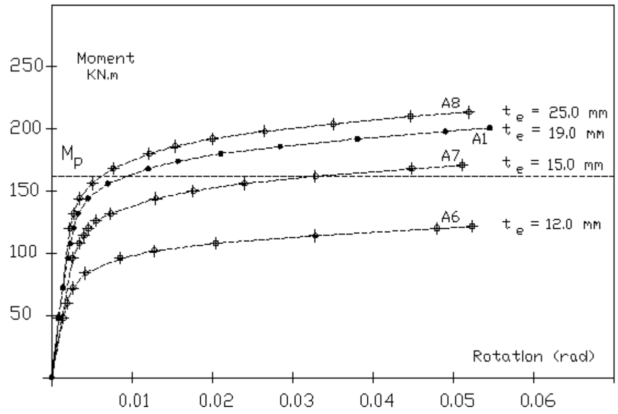


Fig. 6. Moment-rotation relationships when using different end plate thickness.

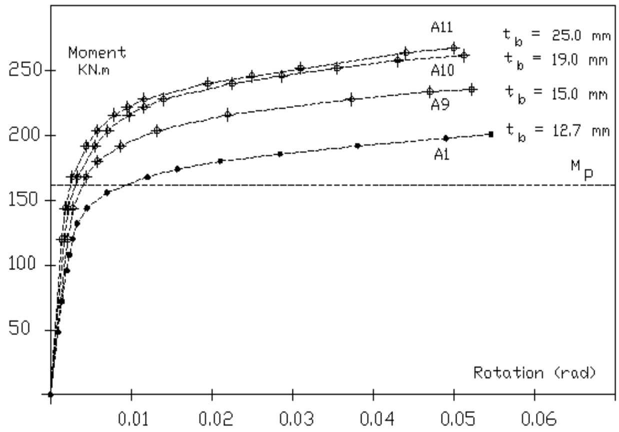


Fig. 7. Moment-rotation relationships when using different bearing plate thickness.

beam web at the tension region of the joint. In A4, the bearing plate was stiffened as in A1. Further, vertical stiffener was used in line with the beam web at the extended part of the end plate. In A5, the bearing plate and end plate were stiffened as in A3 and A4 respectively. Joints A0 and A2 show moment resistance less than M_p, fig. 5. Joints A1, A3 and A4 show the same stiffness but different moment resistance. The values of M_y are nearly equal to M_p. Joint A5 shows stiffer behavior and higher moment resistance with about 20% than those of A3. The stiffener of the end plate in A5 provides additional fixed support to it. This did not allow the end plate to deform with the same magnitude as in A3 where it functions as a beam spanning between the bolts. The results in general show that using stiffeners at the back of the bearing plate would enhance the joint stiffness and moment resistance. The degree of this enhancement depends on the stiffeners details. This advantage is difficult to be provided when the end plate is bolted directly to the flange of SHS column except in built up sections.

8.2. End plate and bearing plate thickness

Fig. 6 shows the moment-rotation relationships of joints A1, A6, A7 and A8. These joints have the same details but different values for the end plate thickness t_e, table 2. Joint A6 moment resistance is 74% of M_p. Increasing t_e with 3 mm in A7 improved $M_{.05R}$ with 40%. Increasing t_e in A1 with 6 mm as in A8 improved $M_{.05R}$ with 6.5%. The results show that increasing t_e is not linearly proportional to the increase in the joint moment resistance. The same behavior is seen when increasing the bearing plate thickness t_b, fig. 7. Specimens A1, A9, A10 and A11 are typical but have different values for t_b, table 4. Increasing t_b from 12.7 mm in A1 to 15mm in A9 caused an improvement in the moment resistance equals 22.5% of M_p. However, the results of A10 and A11 are nearly equal. The joint behavior depends on the relative stiffness of both the end plate and bearing plate. Increasing t_b and t_e would improve the joint stiffness, M_y and $M_{.05R}$. However, increasing the strength of one plate to exceed that of the other one would not improve the joint moment

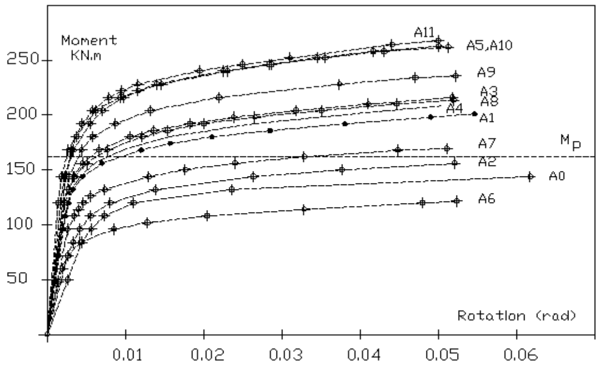


Fig. 8. Comparison of the results of all the specimens in tables 1, 2 and 3.

resistance. This is because failure of the plate that has less strength would cause failure for the joint.

For comparison, all the moment-rotation relationships of joints A0 to A11 are presented in fig. 8. Improving the behavior of A1 is carried out by increasing t_e and t_b as in A8 and A10 respectively. Stiffening the end plate and the bearing plate as in A3 and A5 is another method. The behavior of A8 and A10 are found typical to that of A3 and A5 respectively. Specimens A6 and A2 show the same stiffness. A7 behaved in a stiffer manner. The rest of the joints show nearly the same stiffness. Joints A0, A2, A6 and A7 are classified according to the Euro code 3 [12] as semi – rigid joints while the other joints are rigid. All the joints are classified as full strength joints except A0 and A6, which are classified as a partial strength joint.

8.3. Yield line mechanism

Three different modes of yielding mechanisms are proposed for the bearing plate, fig. 14. They depend on the stiffening details at the back of the bearing plate in the tension region of the connection. Mode 1 consists of sagging and hogging yield lines forming when no stiffeners were used as the case of A2. Mode 2 is considered when horizontal stiffeners are used as in A1. Mode 3 applies when both horizontal and vertical stiffeners are used as the case of A3. The following equation was derived for estimating the bearing plate thickness at yielding.

$$t_b = [T / \{F_y * W\}]^{1/2} . \qquad (1)$$

$$W = (n * C_f / P) + (3.5 P / X) + K . \qquad (2)$$

Where T is the tension component of the applied moment in the beam flange, C_f is the width of the column flange, P is the bolts pitch and X is the distance between bolt center and bearing plate side. The values of n and K are as follows:

For mode 1: n = 0.50 and K = 2X /P
For mode 2: n = 1.0 and K = 2X /P
For mode 3: n = 1.75 and K = 7 P/ B where B is the horizontal distance between the bolts' holes centers in the same row.

The proposed yielding mechanisms are applied to the specimens considered herein. The yielding moment values of the bearing plates M_{By} were calculated. Further, the yielding moment values of the end plates M_{Ey} were also calculated using the proposed yielding mechanism by Packer et al. [17] for extended end plate connections. The results are presented in terms of the ratios M_{By}/M_y and M_{Ey}/M_y where M_y is the yielding moment of the joint obtained from the finite element analysis, tables 1, 2 and 3. The results show that yielding of the joints was mainly due to the formation of yielding mechanism in the bearing plate in joints A0, A1, A2, A4 and A8 and in the end plate in joints A5, A6, A7, A10 and A11. In joints A3 and A9, the yielding strength of the end plate and bearing plate are comparable. Joints yielding would be due to partial initiation of yielding mechanisms in both the bearing plate and the end plate.

9. Bracket plates thickness

Fig. 9 shows moment - rotation relationships of specimens A12, A13, A14 and A15. They are typical but have different values for t_{bk}, table 4. The rotation in this case is equal to the summation of that of the end plate-bearing plate joint and the bracket plates. Specimen A12 shows the least moment resistance. Increasing t_{bk} to 12.7 mm as in A13 enhanced both the connection stiffness and resistance. The improvements in M_y and $M_{.05R}$ were 6.6 % and 8.6 % respectively. The further increase in t_{bk} did not show significant

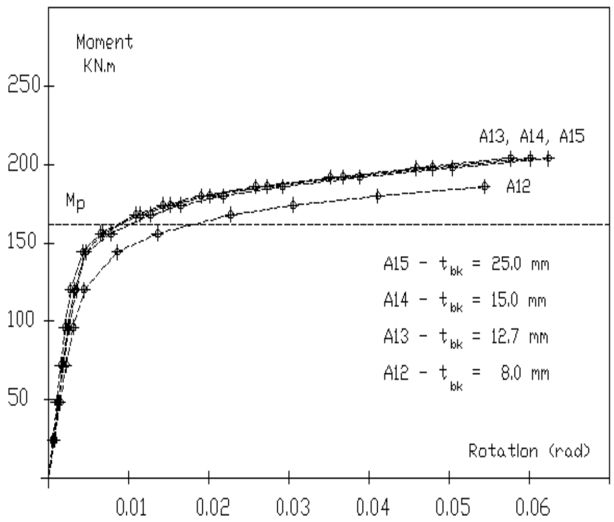


Fig. 9. Moment-rotation relationships when using different bracket plates of different thickness values.

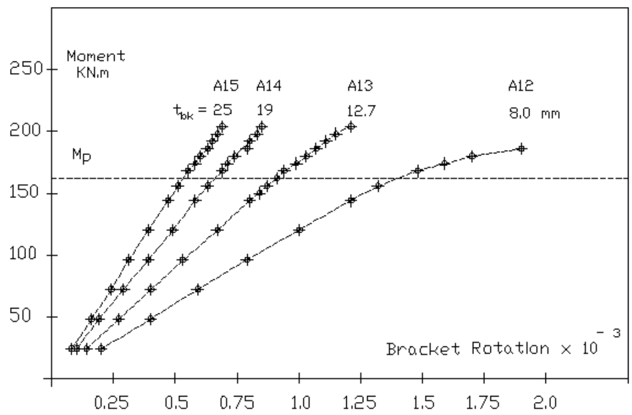


Fig. 10. Rotation due to bracket plates in specimens A12, A13, A14 and A15.

improvement. Fig. 10 shows the rotation of the bracket plates only. The results show linear moment – rotation relationships up to $M_{.05R}$ for all the specimens except A12. The use of different values of t_{bk} caused different rotation values for the bracket plates, R_b. However, this did not cause significant effect on the connections rotations because the values of R_b are relatively very small in comparison to R_j. fig. 11 shows the percentage of bracket plates' rotations to the total rotation of the specimens. Initially, R_b / R_T % were ranging from 15 % to 30 %, but at $M_{.05R}$ the values were less than 5 %.

10. Yielding of bracket plates

The bracket plates are subjected to bending moments about both the Y-Y and Z-Z axes in addition to shear force. The 2 M_{Z-Z} is equal to the moment the connection is subjected to. The bearing plate is assumed to function as a beam spanning between the bracket plates and the welds provide fixed restraining conditions to it. The value of M_{Y-Y} at each support, in the tension region of the connection, is calculated as follows:

$$M_{Y-Y} = 2P_B X (C_f - X) / C_f. \tag{3}$$

Where P_B is bolt load. The finite element results show that both the horizontal stiffener and parts of the bracket plates' depth are most affected by M_{Y-Y}. This effective depth is assumed to equal (e_x + 3 P + t_s). Another moment induced about the Y-Y axis in the compression region of the connection. It is less in value and in the opposite direction to that in the tension region. This is because direct bearing transfers the load in the compression region. In A12, the ratio of the elastic section modulus of one bracket plate about the Z-Z axis to that of the effective parts of the bracket plate and the stiffener about the Y-Y axis equals 3.2 while the value of M_{Z-Z} / M_{Y-Y} = 9.6. This means that 75% of the induced bending stresses are due to M_{Z-Z} while 25% of it is due to M_{Y-Y}. The bracket plates would yield in this case when the connection is subjected to a moment of 168.5 KN.m. This is 98% of the value obtained from the finite element results, fig. 10. When considering A13, the percentages of the bending stresses due to M_{Z-Z} and M_{Y-Y} are 71% and 29% respectively. The bracket plates would yield when the applied moment on the connection equals 1.53 M_p. It should be noted that shear stresses interaction was not considered, as the applied shear force value is less than 0.6 of the plastic shear capacity of the bracket plates [18].

11. Connection overall behavior

Fig. 12 presents the total rotation of A1. This includes the rotations due to end plate – bearing plate joint, bracket plates and the column; i.e. R_T = R_j + R_b + R_c. The column

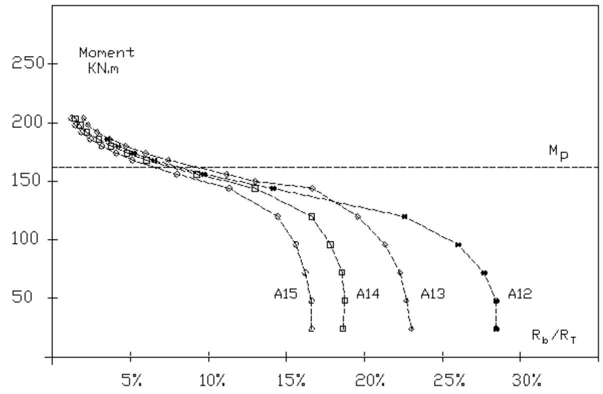


Fig. 11. Percentage of bracket plates rotations to the total rotation in specimens A12, A13, A14 and A15.

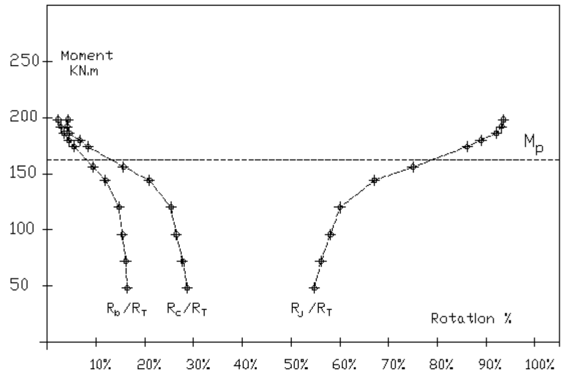


Fig. 13 Percentage of column, bracket plates and end plate-bearing plate joint rotations to connection total rotation of specimen A1.

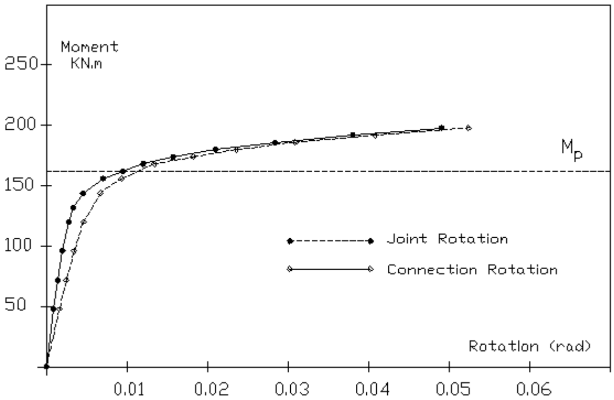


Fig. 12. Moment-rotation relationships of the end plate-bearing plate joint and the connection specimen A1.

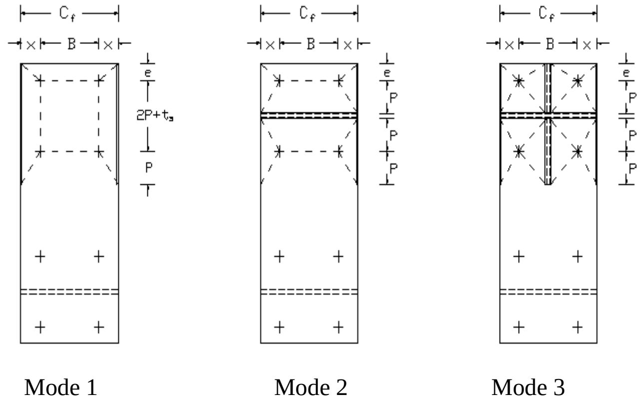


Fig. 14. Yielding mechanisms when using different stiffening details at the back of the bearing plate.

length was taken equal to 3.0 m, a standard floor height. It was restrained only at its upper and bottom edges. The nodes at these positions were hinged to model its continuity and to allow its rotation. For comparison, the rotation of the joint R_j of the same specimen is also presented in fig. 12. The two relations show relatively small differences in the stiffness and rotation up to M_p. Fig. 13 presents the values of R_j/R_T%, R_b/R_T% and R_c/R_T % of A1 at different values of the applied moment. Initially, the summation of the bracket plates and column rotations is comparable to that of the joint, R_b/R_T % + R_c/R_T % = 45 % and R_j/R_T % = 55 %. At connection failure, R_j / R_T is more than 90%. The results show that the bracket plates and column rotations do not have significant effect on the connection behavior. The end plate–bearing plate joint is the dominant part.

12. Loading transfer mechanism

Initially, tightening the bolts to their proof loads induce contact forces between the end plate and bearing plate. The tension force in the beam flange transfers to the bearing plate by the release of the contact force. At this stage, the stiffness of the joint depends on the elastic flexural stiffness of the end plate and bearing plate. This depends on the values of t_e,, t_b , the distance between the bracket plates and the bolts pattern. Adding horizontal and vertical stiffeners at the back of the bearing plate increase its stiffness to a degree that it functions as a rigid support. At certain level of the applied moment, the bolts are subjected to tension force that exceeding their proof loads. Part of the beam flange tension force transfers by the release of the contact force. The other part is transferred through the bolts. The bolts start to elongate. Separation between the end plate and bearing plate would occur. Direct bearing of the end plate against the bearing plate transfers the compression force in the beam flange. The bolts in the compression region of the joint in addition to the induced friction between the end plate and the bearing plate would transfer any applied shear loads. All the forces are then transferred from the bearing plate to the bracket plates through the welds connecting them. Finally, the forces are transferred to the SHS column web walls through the weld joining the bracket plates.

13. Design recommendations

It is recommended that stiffeners should be used at the back of the bearing plate to provide rigid support to the end plate. The bracket plates, bearing plate and the welds would be in elastic state at yielding of the connection. In this case, the analogy to the behavior of end plate connections joining I section members can be applied. The proportion of the end plate and the bolts would define the failure mode of the connection. The use of relatively thick end plate would promote bolts fracture. This tends to cause a sudden decrease in strength. The use of thin end plate would promote its yielding. Marginal increase in strength is gained after end plate yielding due to the strain hardening effect. The value of the induced prying forces should be considered.

14. Design procedure

The following guidelines are suggested for the design of the proposed connection:

1. The bracket plates should be designed for the applied bending moment and shear force. The thickness at which yielding would start is initially estimated as follows;

$$t_{bk} = 6\ G\ M_{Z\text{-}Z} / (h^2 F_y)\ . \qquad (4)$$

Where G is a factor that compensate for the induced moment about the Y-Y axis and taken equal to 1.35 and h is the height of bracket plate. The elastic section modules of the effective parts of bracket plate about the Y-Y and Z-Z axes are calculated as described before. The bending stresses should satisfy the interaction equation of bending stresses specified in codes of practice. To avoid local buckling, the value of t_{bk} should also satisfy the conditions of elements under flexure compression specified in the codes [12, 14], [18, 19].

2. Different welds configurations may be used for the welds between the bracket plates and the column web walls. The welds group is subjected to eccentric load. Tables provided in codes of practice such as those in AISC (18) and CAN/CSA (14) can be used.

3. The thickness of the bearing plate t_b can be defined using equations 1 and 2 according to its stiffening details.

4. The welds between the bracket plates and the bearing plate can be designed as mentioned in step 2.

5. Different models are presented in the literature for the proportion of the end plate and bolts in end plate connections joining I section members such as those in references [12] and [16]. These models can be used when the design recommendations listed above are satisfied.

6. When using codes of allowable stress method of design, the value of F_y in equations 1, 2 and 4 should be changed with the value of the allowable bending stress specified in the code for the type of steel used.

The equations considered in the design procedure are applied on the specimens considered in this study. Verification of these equations is given by the comparison of the results to those obtained from the finite element analysis, tables 1, 2 and 3.

15. Conclusions

This paper presents simple and practical moment connection proposed for connecting I beam to SHS and RHS columns, fig 1. The connection is characterized by its easy fabrication and erection. Normal high strength bolts are used and access from inside the column hollow section is not required. Nonlinear finite element analysis was used to model the behavior of the connection. The finite element model was verified, fig 3. The efficiency of the connection was investigated and proved, fig. 4. The connection can be proportioned to provide a rigid full strength connection. The connection consists of two main parts, the end plate-bearing plate joint and the bracket plates. The bracket plates' rotations do not have significant effect on the total rotation of the connection, fig. 13. The end plate – bearing plate joint is the dominant part. Three different modes of yielding mechanisms are proposed and used for the joints considered in the study, fig 14. The results obtained agree with those obtained from the finite element results. End plate – bearing plate joint behavior depends on the relative stiffness of the end plate and bearing plate. Using stiffeners and/or increasing the bearing plate and end plate thickness would improve the joint stiffness, yielding moment and moment of resistance. However, increasing the strength of one plate to exceed that of the other one would not improve the joint moment of resistance. This is because failure of the plate that has less strength would cause failure for the joint. Recommendations and guidelines are suggested for the design of the connection.

Notations

B	is the horizontal distance between the bolts' holes centers in the same row,
C_f	is the column flange width,
e_x	is edge distance of bolts,
F_y	is the nominal yield stress specified in the Codes,
G, K, n	are factors,
H	is bracket plate height,
Hc	is column height,
L_a	is the distance of the load to the connection centerline,
L_{bk}	is bracket plate cantilever length,
L_s	is stiffener length,
M_{By}	is the bearing plate yielding Moment,
M_{Ey}	is the end plate yielding moment,
M_p	is the nominal plastic moment of the beam section,
$M_{.05R}$	is the joint or the connection moment of resistance,
M_y	is the joint or the connection yielding moment,
$M_{Y\text{-}Y}$	is the induced moment on each bracket plate about the Y-Y axis,
$M_{Z\text{-}Z}$	is equal half the moment the connection is subjected to,
P	is the bolts pitch,
P_B	is bolt load,
R_b, R_c	is the rotations due to the bracket, plates and the column respectively,
R_j	is the rotations due to the end plate -bearing plate joint,
R_T	is the total rotation of the Connection,
T	is the tension component of the applied moment in the beam flange,
t_b	is the bearing plate thickness,
t_{bk}	is the bracket plate thickness,
t_e	is the end plate thickness,
t_s	is stiffener thickness, and
X	is the distance between bolt center and bearing plate side.

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Received April 10, 2005
Accepted July 13, 2005