STANDARDIZATION OF PARTIAL STRENGTH CONNECTIONS FOR MULTI-STOREY BRACED STEEL FRAME

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ABSTRACT

Connections are usually designed as pinned or rigid although the actual behaviour is known to fall between these two extreme cases. The use of partial strength or semi-rigid connections has been encouraged by codes and studies on the matter known as semi-continuous construction have proven that substantial savings in steel weight of the overall construction. The objective of this paper is to develop a partial strength connection's table comprised of extended end-plate connection for Perwaja steel sections. The use of this table could enhance the design of semi-continuous construction of multi-storey braced steel frames. The strength of the connection is presented in the form of standardized table which include moment capacity and shear capacity after considering all possible failure modes. The moment capacity, shear capacity, geometrical aspects of the connections, the size of beams, and columns that are suitable with the connections are included in the standardized table. A component method proposed by Steel Construction Institute(SCI) which take into account the requirements in Eurocode 3 and BS 5950:2000 Part 1 were adopted to predict the moment capacity and shear capacity in developing the table. This paper also presents the experimental results of the extended end-plate connection for Perwaja steel sections. Eight experimental tests have been carried out for extended end-plate connections consist of variable parameters such as size and thickness of end-plate, size and number of bolts, size of columns, and beams. However, only two are presented in this paper. The tests were set-up using local hot-rolled steel sections known as Perwaja sections for both beams and columns instead of typical British sections. Geometric parameters such as the thickness of end-plate, the number and size of bolts and the use of deeper beam have contributed significantly to the increase in moment resistance and stiffness of the connections. The study concluded that all the tested extended end-plate connections are eligible to be classified as partial strength connections and the results of maximum moment resistance showed good agreement between experimental and predicted component method in most cases.

Keywords: Partial strength connection, Moment Capacity, Beam-to-column connection, Semi-Continuous Construction, Perwaja Steel Section

1. INTRODUCTION

Conventionally, steel frames are designed either as pinned jointed or rigidly jointed. When designed as pinned jointed, the beams are assumed as simple supported and the columns are assumed to sustain axial and nominal moment (moment from the eccentricities of beam's end reactions) only. The connection is simple but the sizes of the beams obtained from this approach result in heavy and deep beam. On the other hand, rigidly jointed frame results in heavy columns due to the end moments transmitted through the connection. Hence, a more complicated fabrication of the connection could not be avoided.

One approach, which creates a balance between the two extreme approaches mentioned above, has been introduced. This approach, termed as semi-rigid or partial strength is usually associated with a connection having a moment capacity less than the moment capacity of the connected beam [1]. Partial strength connection is the term used for connection in the design of semi-continuous construction for multi-storey steel frames by Eurocode 3 [2]. In semi-continuous frame the degree of continuity between the beams and columns is greater than that in simple construction design but less than that in continuous construction design. The degree of continuity in the use of partial strength connection of beam to column can be predicted to produce an economical beam section that representing the section between pin joint and rigid joints. By adopting this approach, studies conducted on the use of partial strength connection have proven substantial savings in overall steel weight [3; 4]. This is possible as the use of partial strength has contributed to the benefits at both the ultimate and serviceability limit states design. However, the use of partial strength connections for Perwaja Steel Sections (PSS) has not been established yet. To establish the study on the use of PSS with partial strength connection, standardized partial strength connections tables need to be established first. Therefore, this paper intends to establish the standardized tables for partial strength connections for PSS sections based on the proposed method by SCI.

2. PERWAJA STEEL SECTION.

Perwaja Steel Section (PSS) is a structural section referred in this paper as a section produced in accordance with the specifications of JIS G 3192:1994. These sections are used for structural steel that can be welded based on BS EN 10113 (1993) [5]. The section designation used for PSS is not the same as the British Section (BS); the dimension of the section is round up to a more fixed number that is more users friendly. For example, in British Section, the column is designated as UC 203 x 203 x 46, but for PSS, the section is designated as HB 200 x 200 x 49.9. The designation of HB is used to represent both beams and columns sections. The thickness of the PSS section for HB200 x 200 x 49.9 is 8.0mm thick whereas the thickness of BS for UC 203 x 203 x 46 is 7.3 mm thick, lesser than the PSS. The steel design strengths for PSS however are the same as the typical BS, namely S275 and S355. However, in this paper only S275 steel sections were used for the experimental tests.

3. PARTIAL STRENGTH CONNECTIONS

In the design of braced multi-storey steel frames, steel weight of the connections may account for less than 5% of the frame weight; however, the cost of the fabrication is in the range of 30% to 50% of the total cost [6]. The increase in the fabrication of the connections is due to the difficulty in selecting the type of connection, the grades and sizes of fittings, bolt grades and sizes, weld types and sizes, and the geometrical aspects. Although the advantages or benefits of using the partial strength connections such as reduce the depth of the beam which results in reduction to the height of the cladding and reduce the steel weight which result in the reduction in overall cost, their usage is not widely implemented. Therefore, the disadvantages of this approach should also be addressed. This approach may be marginally more expensive, depending on the cost of labour paid which varies between Europe and Asian countries to fabricate partial-strength connection rather than simple connections. In Malaysia where the cost of labour is relatively low as compared with the Europe, the use of the proposed connections could be an added advantage. The benefits of overall cost saving of the partial strength connections have proven to be more than simple connections [4]. It was reported that the savings in steel weight of using partial strength connection alone in multi-storey braced steel frames using British hot-rolled section was up to 12% [7]. The overall cost saving was up to 10% of the construction cost which was quite significant according to the cost of labour in the United Kingdom [8]. The use of partial strength connections could be further enhanced by introducing a standardised connection' table to facilitate designers and steel fabricators in construction industry. The saving in the overall cost with the use of standardized partial strength connections tables can be achieved due to the following reasons [1].

- A reduction in the number of connection types may lead to a better understanding of the cost and type of connection by all steel players such as fabricator, designer, and erector.
- A standardized connection can enhance the development of design procedures and encourage in the development of computer software.
- The use of limited standardized end-plates or fittings can improve the availability of the material leading to reduction in material cost. At the same time, it will improve the order procedures, storage problems and handling time.
- The use of standardized bolts will reduce the time of changing drills or punching holes in the shop which lead to faster erection and less error on site. The drilling and welding process can be carried out at shop as the geometrical aspects of the connection have already been set which lead to fast and quality fabrication.

Based on the collected experimental test data, [9] had developed a computerised databank system at Purdue University Computer Centre. The database was aimed at providing moment-rotation characteristics and corresponding parameters of semi-rigid steel beam-to-column connections used frequently in steel construction [10] then expanded the database by adding another 46 experimental tests data. Recent tests on extended end-plate connections were conducted by Abdalla [11] to investigate the force distribution in high strength bolt. Six full scale beam-to-columns with extended end-plate connections were tested with eight high strength bolts of M20 of Grade 8.8. From the study, it was found that the tension forces in the upper bolts above the beam flange could be reasonably determined using an equal distribution method. However, the use of equal distribution method was not suitable for upper bolts below the beam flange. Coelho has carried out experimental tests on eight statically loaded end-plate moment connections [12]. The specimens were designed to cause failure on the end-plate or bolts. The study concluded that an increase in end-plate thickness has resulted in an increase to the connection's flexural strength and stiffness. However, these tests were carried out for hot–rolled steel section and the connection was a non-composite connection. Shi, Li, Ye, and Xiao have tested on composite joints with flush end plate connection as partial strength connection under cyclic loading. The composite joints with flush end plate connection showed large strength resistance and good ductility.[13] The slippage between concrete slab and steel beam was very small, which showed that full interaction between concrete slab and steel beam were achieved with the use of full shear connectors. The study on the partial strength connection was also carried out by Tahir and Arizu.[14] A series of non-composite connection comprised of flush and extended end-plate connections was tested and compared with composite connection. The beam section was a built up section known as trapezoid web profiled (TWP) steel section whereas the column section comprised of British Section. It was concluded that the moment resistance and the initial stiffness for composite connection were higher than the non-composite connection. Further study on the partial strength connection with TWP steel section was carried out by Tahir, Sulaiman and Saggaff [15]. Two full scales testing with beam set-up as sub-assemblage and beam-to-column connection set-up as flush and extended end-plate connections have been carried out. It was concluded that the use of extended end-plate connection has contributed to significant reduction to the deflection and significant increase to the moment resistance of the beam than flush end-plate connection.

3.1 Standardized partial strength connections

In the design of braced multi-storey steel frames, the increase in the fabrication of the connections due to the difficulty in selecting the type of connection, the grades and sizes of fittings, bolt grades and sizes, weld types and sizes, and the geometrical aspects could be resolved by standardization. Therefore, a standardized partial strength connections tables are introduced to cater for the problems arise due to so many uncertainties in the fabrication of the connections. The advantages of the partial strength approach are that it utilizes the moment resistance of connections to reduce beam depth and weight, while avoiding the use of stiffening in the joints. This practice will reduce the cost of fabrication and ease the erection of steel member in the construction of multi-storey steel frames [1]. The potential benefits of using this approach can be listed as follows[6; 16]:

3.1.1 Lighter beams

In the design of semi-continuous braced steel frame, the required beam plastic modulus is less than those required in simple frame for the same frame. This reduction is possible as the partial strength connection reduced the design moment of the beam due to the partial restraint effect of the connection as illustrated in Fig. 1[1]. The design moment which a beam must resist, decreases as the moment capacity of the connection increases. As a result, a lighter beam can be selected for the design of the beam.

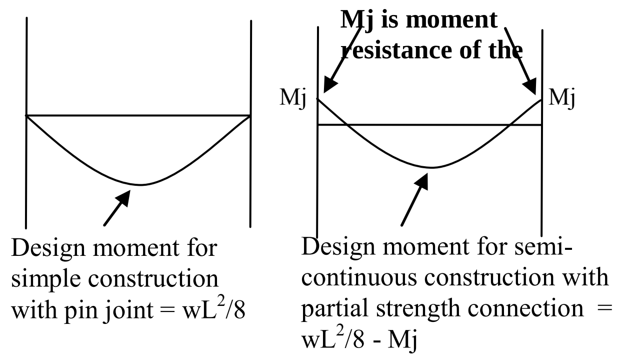


Figure 1: Design moment for beams due to different support conditions

3.1.2 Shallower beams

The partial restraint of the connection will also result in shallower beams. This is due to the increase in stiffness of the connection, which contributes to the decrease in deflection. The use of partial strength connection will reduce the constant coefficient β in the formulae of deflection ($\beta wL^4/384EI$) in simple construction with uniform load, from β equal to 2 for internal beam, and 3 for external beam[4]. The partial strength connection acts as restrained to the deformation of the beam due to applied load. As a result, a reduction in the deflection of the beam can be achieved which lead to the shallower beam. The relationship between connection stiffness and deflection coefficient "Beta" for uniform load on beam is shown in Fig. 2[4].

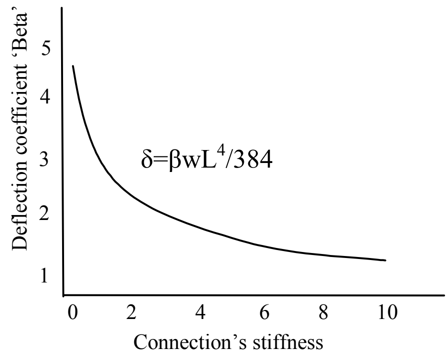


Figure: 2 Deflection coefficient 'Beta' as a function of relative stiffness of connection

3.1.3 Greater stiffness and more robust structure

Connection rotational stiffness means that the ends of a beam are restrained against rotation. Partial strength connection has higher capacity to restrain against rotation, shear, moment and tying force compared to pin connection. The rotation capacity should be in the range of 0.02 to 0.03 radians at failure for the connection to be considered as ductile and stiff enough to be categorized as partial strength[1]. The shear capacity of the connection is designed in such a way that the capacity is higher than the shear capacity of the connected beam, and the moment capacity of the connection can resist up to 50% of the moment capacity of the connected beam (M_{cx}) depending on the size and number of bolts for the proposed standard tables[1]. The tying force of the connection is two to three times greater than the tying force required by BS 5950:2000 Part 1 that is 75kN[17]. Therefore, the connection can be categorized as strong, stiff, and robust.

3.1.4 Lower overall cost

Good connection should be the one which can ease the design process, the preparation of detailing, the fabrication process, and the erection works. It should be also the most cost effective, compared to other types of connection. The saving in the overall cost can be achieved due to the reasons given earlier. Although the advantages or benefits of using the partial strength connections are quite significant, the disadvantages of this approach should also be addressed. The disadvantage in this approach is that it may be marginally more expensive to fabricate partial-strength connection rather than simple connections. However, the benefit of overall cost saving of the partial strength connections have proven to be more than simple connections[3; 4]. It is reported that the savings in steel weight of using partial strength connection in multi-storey braced steel frames using British hot-rolled section was up to 12%[18]. The overall cost saving was up to 10% of the construction cost which is quite significant[18].

3.2 Standardised Extended End-Plate Connections

The use of partial strength connection for hot-rolled British sections has well established by SCI[1]. A series of tests at the University of Abertay, Dundee has been successfully been carried out to verify the predicted moment and shear capacity with the experimental tests capacities[19]. The results confirmed with the predicted values and the standardized tables for the connection have been published by SCI[1]. In the development of standard extended end-plate connections tables for Perwaja sections, one table is presented in this study based on the proposed method. A few tests have been carried out by Steel Technology Centre to support the predicted moment resistance of the connection. Some of the results are presented in this paper. Figure 3, shows a typical extended end plate connection with both beam and column are from Perwaja steel sections. The proposed standard connections have the following attributes which in some cases the attributes are not exactly the same as the one described by SCI in hot-rolled section.

- 12mm thick end plates in conjunction with the use of M20 bolts.
- 15mm thick end plates in conjunction with the use of M24 bolts.
- Strength of end plates was maintained as S275 steel.
- Width of the end plate was kept at 200mm and 250mm with the vertical height of the end-plate was kept at the beam depth plus 50mm.
- Full strength of flange welds with size of weld proposed at 10mm
- Full strength of web welds with size of weld proposed at 8mm
- The vertical and horizontal distance between the bolts was maintained at 90mm.

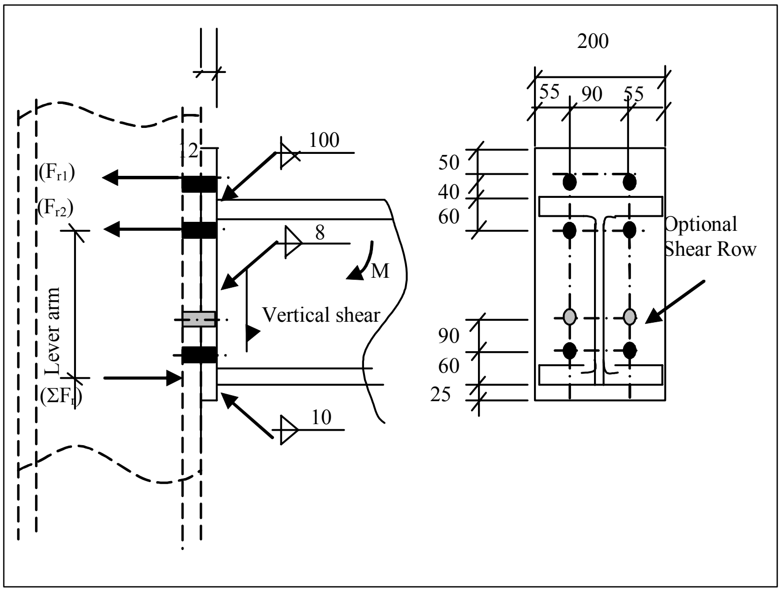


Figure 3: Typical extended end-plate connection.

3.3 Design philosophy of component method.

The design model of component method adopted in this study is actually presented by Steel Construction Institute (SCI) from Eurocode 3 [1]. For checking the details of strength on the bolts, welds, and steel section, modification to suit BS 5950:2000 Part 1[17] have been done. The checking on the capacity of the connections is classified into four zones namely tension zone, horizontal shear zone, compression zone, and vertical shear zone as shown in Figure 4 (Peter A, et al 1996). Details of the checking on each of the zone are listed in Table 1. The basic principles on the distribution of bolt forces need to be addressed first before details of the checking on all possible modes of failures can be discussed.

Table 1. Component design checks

Zone	Reference	Component to be checked
Tension	1.	Bolt tension
	2.	End-plate bending
	3.	Column flange bending
	4.	Beam web tension
	5.	Column web tension
	6.	Flange to end-plate weld
	7.	Web to end-plate weld
Horizontal Shear	8.	Column web panel shear
Compression	9.	Beam flange compression
	10.	Beam flange weld
	11.	Column web crushing
	12.	Column web buckling
Vertical	13.	Web to end-plate weld
	14.	Bolt shear
	15.	Bolt bearing (plate or flange)

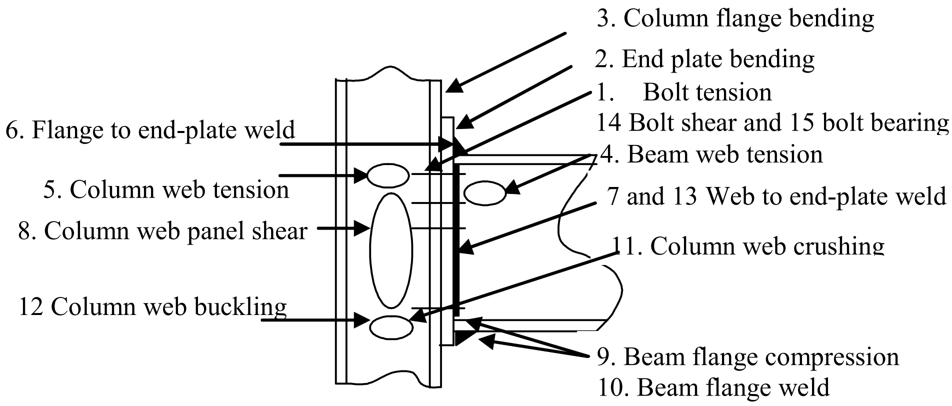


Figure 4. Critical zones that need to be checked for failure

3.3.1 Distribution of Bolt Forces

The moment resistance of a connection transmitted by an end plate connection is through the coupling action between the tension forces in bolts and compression force at the centre of the bottom flange. Each bolt above the neutral axis of the beam produced tension force whereas the bolts below the neutral axis are dedicated to shear resistance only. Eurocode 3(1992) suggests that the bolt forces distribution should be based on the plastic distribution instead of the traditional triangular distribution [2]. Figure 5 shows the forces in the connection and the corresponding distributions. The forces of the bolt are based on the plastic distribution which represents the actual value calculated from the critical zones in Figure. 5. The force from the top bolt row transmits to the end-plate connection as tension force which balanced up by the compression force at the bottom flange of the beam to the column. The end-plate is connected to web and both of the flanges by welding. The formation of tension at the top and compression at the bottom contributes to the development of moment resistance of the connection. Tests on the connections have showed that the centre of compression flange which bears against the column was found to be the centre of rotation of the connection [19]. The force permitted in any bolt row is based on its potential resistance and not just the length of the lever arm.

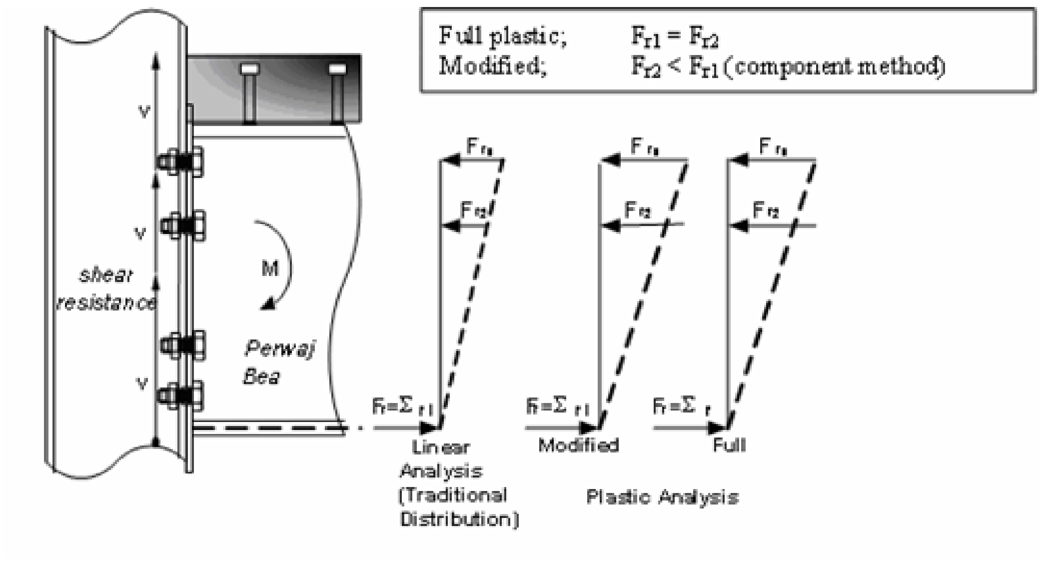


Figure. 5 Elastic and Plastic Analysis of Bolt Forces Distribution in Composite Connection

3.3.2 Tension zone

The resistance at each bolt row in the tension zone may be limited due to bending of column flange, end-plate, column web, beam web, and bolt strength. Column flange or end-plate bending is checked by using Eurocode 3(1992) which converts the complex pattern of yield lines around the bolts into a simple 'equivalent tee-stub'[2]. Details of the procedures are illustrated in SCI publication [1]. This approach results in checking against three type of failure as follows:

Type 1: Complete flange yielding

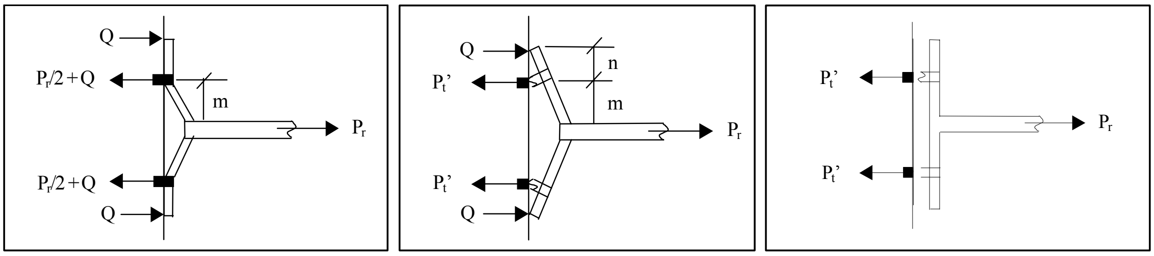
In this failure mode, the strength of the flange (beam or column flange) or end-plate is weaker than to the strength of the bolts. Upon failure, the flange or end-plate will yield but the bolts are still intact as shown in Figure 6(a). As a result, a ductile failure can be achieved. This type of failure is the most preferred failure mode in semi-continuous construction as suggested by SCI as abrupt failure of the connection can be avoided.

Type 2: Bolt failure with flange yielding

In this failure mode, the strength of the flange or the end-plate and the bolts are about the same. As a result, both the flange or the end-plate and the bolts will yield together upon failure. This mode of failure is shown in Figure 6(b). This type of failure can be used in the design of semi-continuous construction provided that the moment resistance of the connection can be quantified and the connection can also be classified as ductile connection.

Type 3: Bolt failure

In this failure mode, the strength of the bolts is weaker than the strength of the flange. Upon failure, the bolts will yield (or even break) but the flange or the end-plate is still intact. Figure 6(c) shows Mode 3 failure. This type of failure is not suitable for semi-continuous connection and should be avoided as the connections possess an abrupt type of failure.



(a): Type 1 Complete flange failure. (b) Type 2 Bolt failures with flange yield (c) Type 3 Bolt failure.
Figure 6. Three typical types of failures for end-plate and column flange in tension zone.

3.3.3 Compression zone

The checking in the compression zone are the same procedures as mentioned in BS 5950:2000 Part 1 which requires checks on web bearing and web buckling. The compression failure modes can be on the column side or on the beam side. The column side should be checked for web buckling and web bearing due to the compression force applied to the column. The use of stiffener or the effect of having other beam connected to the web of the column is not included so as to reduce the cost of fabrication and simplified the calculation. The compression on the beam side can usually be regarded as being carried entirely by the beam flange, however when large moments combined with axial load, the compression zone will spread to the web of the beam which will effect the centre of compression [1]. Therefore, the stiffening of the web of the beam needs to be done. However, in this study the moment resistance of the connection is not considering the use of stiffener in order to reduce the cost of fabrication.

3.3.4 Shear zone

The column web can fails due to the shearing effect of the tension and compression force applied to the web of the column. The failure to the shearing of the web is most likely to happen before it fails due to bearing or buckling[1]. This is possible because the thickness of the flange is more than the thickness of the web. Again in this shear zone, stiffer is not needed so as to reduce the cost of fabrication.

3.3.5 Welding

Fillet weld is preferred than the butt welds as the welding of beam to the end-plate is positioned at 90 degree which is suitable for fillet weld to be used. The end-plate is connected to the web of the beam by an 8 mm fillet weld, whereas a 10 mm fillet weld is suggested for connecting the end-plate to the flange. The weld is designed is such a way that the failure mode of the connection is not on the welding. This is to ensure the ductility of the connection which is necessary for partial strength connection.

3.4 Validation of the standardized connections tables

The validation of the standardized connections tables is best presented by comparing the predicted values in the table with full scale testing of the connections. Therefore, a series of full scale testing comprised of eight specimens was conducted by the Steel Technology Centre, Universiti Teknologi Malaysia. However, only two of the moment rotation curves are presented in this paper as shown in Figure 7 and 8. The rest of the results are presented elsewhere [20]. Although the tests did not cover the whole ranged of the proposed connections, the comparison of the tests and the predicted values can still be established as the failure mode comprised of mode 1 and mode 2.

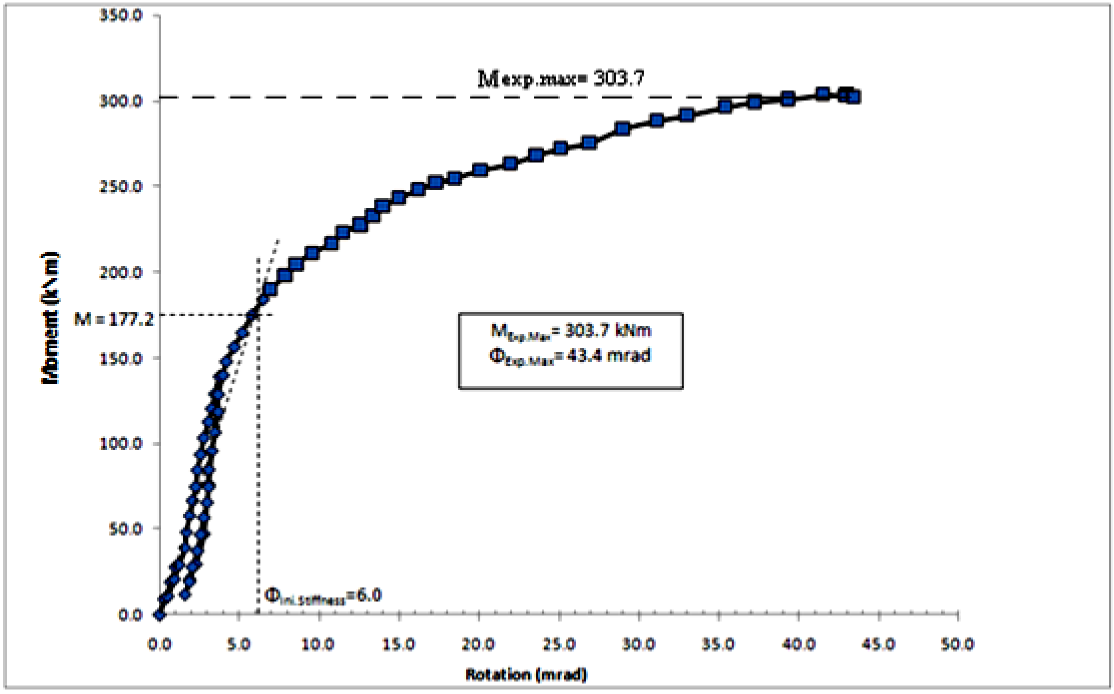


Figure 7 Moment vs rotation for EEP 1.

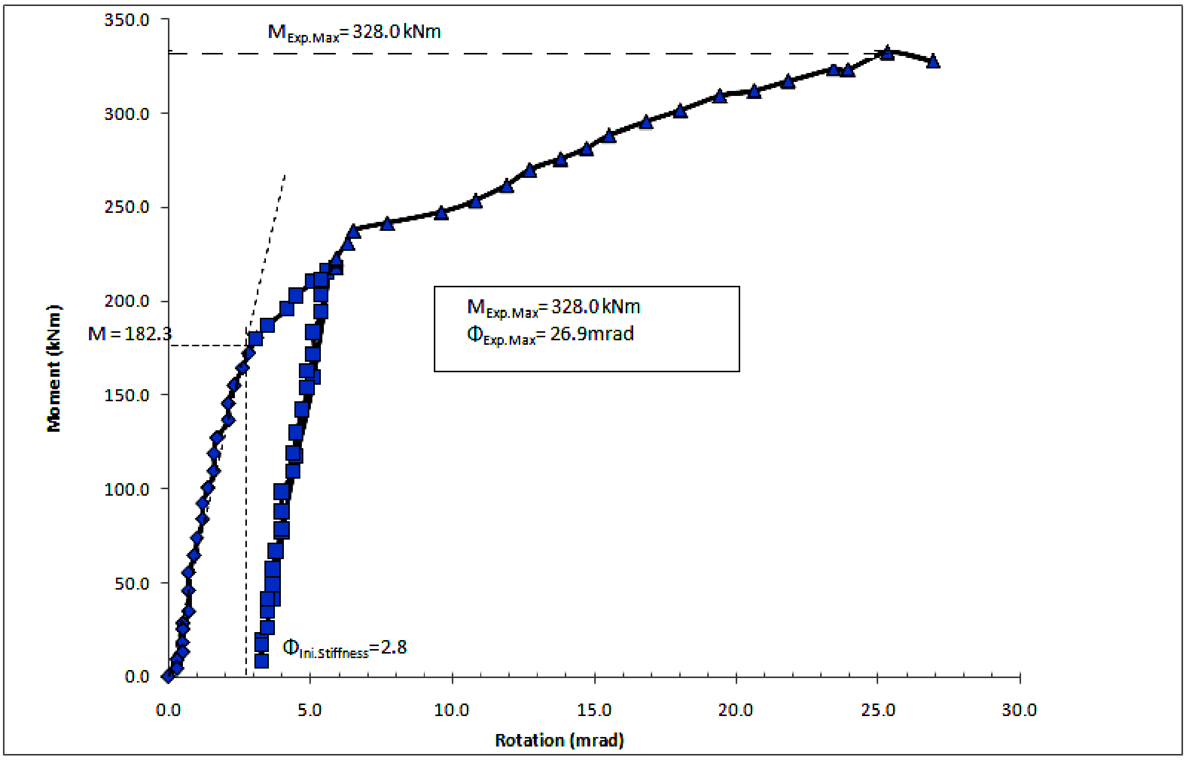


Figure 8 Moment vs rotation for EEP 3.

Fig 7 and 8 show some of the results of the experiment by plotting the moment-rotation curves of the connection. The test results show good agreement with the predicted values. The failure modes of end-plate of the connections are shown in Fig 9 and 10 as expected from the calculation. Details of the method of testing and the discussion of the result have been published elsewhere. [20].

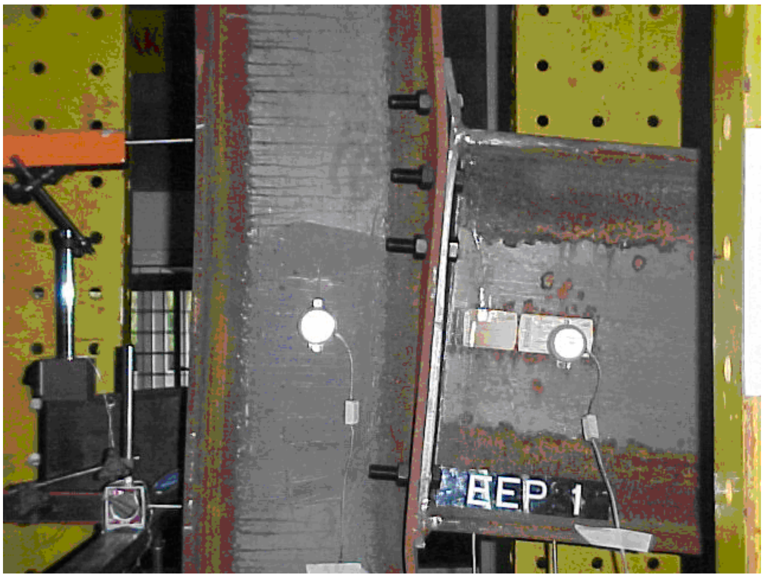


Figure 9. Deformation of end-plate and column flange in tension zone.

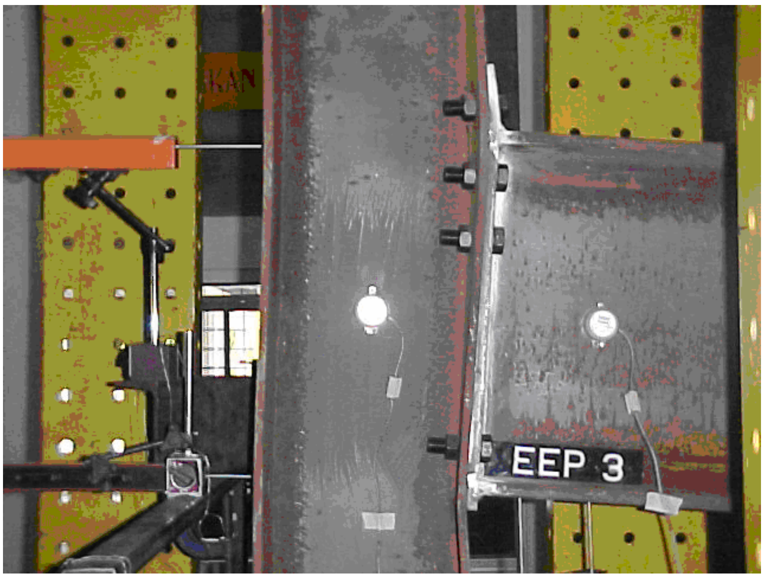


Figure 10. Deformation of end-plate and column flange in tension zone.

3.5 Discussion of standardised table.

The standard table as shown in Table 2 illustrates the geometrical configuration of the suggested connections and the capacities of the connections. The suggested size of column and beam used for the proposed connection is listed in the designated table. The moment capacity of the connection is listed base on the size of the beam. The smallest suggested size of beam is taken as 300x120. The TWP section is not economical to produce beam smaller than 300x150. The largest suggested size of beam is taken as 600x300. The size for partial strength connection is limited to 600mm so as to maintain the ductility of the connection that is crucial to form a plastic hinge in semi-continuous design. The shear capacity of the connection is based on the shear capacity of the tension bolt row and lower bolt rows. However, the lower bolt row will carry most of the shear force. The increase in moment capacity depends on the size of bolt, the number of bolt, the size of end-plate, and the thickness of end-plate. The notation used for the designated connection such as (EEP2BRM20,200W12THK)) meaning that the connection is extended end-plate with two bolt row of M20 grade 8.8, and end-plate size of 200mm wide and 12mm thick. The vertical shear capacity of connection in Table 2 is increased from 258kN without optional shear bolt row to 442kN with shear bolt row. The increment of the vertical shear capacity is not exactly double as the determination of the shear capacity depends on the number of row of the tension bolt too.

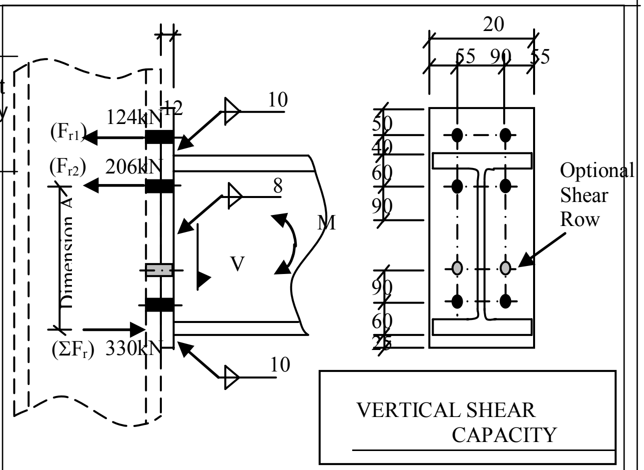
Table 2. Standard table for extended end-plate for 2 row M20 8.8 bolts, 200x12 end-plate (EEP,2BRM20,200W12THK)

2 ROW M20 8.8 B0LTS
200 x 12 DESIGN GRADE S275 EXTENDED END PLATE

Beam Side

BEAM – FLANGE and WEB S275

Beam Serial Size D x B	Dimension 'A' (mm)	Moment Capacity (kNm)
600 x 300	513	182
600 x 200	528	187
500 x 300	414	149
450 x 300	429	154
450 x 200	371	135
400 x 200	380	138
350 x 250	322	119
300 x 200	330	121
300 x 150	273	102



Column Side

DESIGN GRADE S275				COLUMN	DESIGN GRADE S355			
Panel Shear Capacity (kN)	Tension Zone F_{R1} (kN)	Tension Zone F_{R2} (kN)	Compn. Zone	Serial Size	Compn. Zone	Tension Zone F_{R1} (kN)	Tension Zone F_{R2} (kN)	Panel Shear Capacity (kN)
1000	√	√	√	350 x350 x 154.2	√	√	√	1302
849	√	√	√	134.9	√	√	√	1105
725	√	√	√	129.3	√	√	√	944
605	√	√	√	113.0	√	√	√	787
1037	√	√	√	300 x 300x 104.8	√	√	√	1350
816	√	√	√	93.0	√	√	√	1062
732				83.5				
703	√	√	√	250 x 250 x 81.6	√	√	√	915
595	√	√	√	71.8	√	√	√	774
503	√	√	√	63.8	√	√	√	649
882	√	√	√	200 x 200 x 56.2	√	√	√	1149
685	√	√	√	49.9	√	√	√	892
459	√	175	√	175 x 175 x 40.4	√	√	√	598

Tension Zone:
√ Column satifactory for bolt row tension values shown for the beam side.
xxx Calculate reduced moment capacity using the reduced bolt row values.
Compression Zone:
√ Column capacity exceeds ΣF_r.
S (xxx) Column requires stiffening to resist ΣF_r (value is the column web capacity).

4. DISCUSSION OF TESTS RESULTS.

The behaviour of the connections depends on the geometrical configurations of the connection. However, significant effect to the behaviour of the connections can only be achieved by a proper combination of the connections' parameters. For example the end-plate thickness of 12mm should be used together with M20 bolts and 15mm end-plate should be used together with M24 bolts. This is to avoid premature deformation of end-plate or the bolts if the combined connections parameters are not properly selected. The size of the beam, the number, size and distance of the bolt and the thickness of the end-plate may significantly affect the moment resistance and the rotation stiffness of the connection. To understand further the effect of these geometrical configurations the moment resistance, rotational stiffness, and the ductility of the connection should be compared and discussed by referring to the M-Φ curves. These effects can be well understood by comparing the behaviour of the tested specimens based on the moment resistance and the initial stiffness of the connections. The use of 12mm thick end-plate with M20 bolts of Grade 8.8 and 15mm thick end-plate with M24 bolts of Grade 8.8 thick were suggested by SCI. This practice was done to ensure the ductility of the connection and deformation capacity of the end-plate and the tension capacity of the bolt can be balanced up or in equilibrium. The effect of increasing the size of bolts from M20 bolts in conjunction with 12mm thick end-plate to M24 bolts in conjunction with 15mm thick end-plate can be seen by comparing specimen EEP 1 with specimen EEP 3 as shown in Table 3. This increment may due to the contribution on the use of M24 bolts with two bolt rows that dominate the increase in moment resistance of the connection.

Table 3: Effect of increasing the size of bolts and the thickness of the end-plate.

Specimen	Connection's parameters Thickness Width	Max. moment resistance from experimental tests, M_{max} (kNm)	Percentage difference %	Initial Stiffness, $S_{j,ini} = M_R/\Phi$ **(kNm/mrad)**	Percentage difference %
EEP 1 (3.No. M20 bolts) vs EEP 3 (3 No. M24 bolts)	 12 200 HB500x200x102 HB300x300x83.5 15 200	 303.7 328.0	8.0%	 29.53 65.11	120.5%

5. CONCLUSIONS

This study concluded that it is possible to determine the moment capacity of extended end plate connection connected to a column flange by adopting the method proposed by SCI, even for different geometric parameters such as Perwaja section. The capacities of the connection depend on the geometrical aspects of the connection such as the size of bolt, number of bolt, size of end-plate, thickness of end-plate, size of beam and size of column. For the size of column, the reduction of moment capacity is due to the effect of compression of the beam flange to the column flange without the need of stiffener. The suggested weld size for flange and web is strong enough to prevent any failure at the weld. The increment of moment capacity of the connection can be concluded as follows:-

- The increase in the number of bolt row from one row to two rows has contributed to an increase in the moment capacity.
- The increase the size of bolt from M20 with 12mm thick end-plate to M24 with 15mm thick end-plate has contributed to an increase in the moment capacity up to 120%.
- The shear capacity of the connection depends on number of bolt used in the connection. However, the lower bolt row contributed to most of the shear capacity of the connection
- The extended end-plate connection can be used in the design of semi-continuous construction in multi-storey braced steel frames as the connection is classified as partial strength connection.

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REFERENCES

[1] Steel Construction Institute and British Constructional Steelwork Association, Joints in Steel Construction. Volume 1: Moment Connections, London: SCI & BCSA, 1995.

[2] Eurocode 3(2005), Design of Steel Structures: ENV 1993-1-1: Part 1.1: General Rules and Rules for Buildings, CEN, Brussels, 1992.

[3] Md Tahir, M, Structural and Economic Aspects of The Use of Semi-Rigid Joints in Steel Frame, UK: University of Warwick, PhD Thesis, 1995.
[4] G. H. Couchman, Design of Semi-continuous Braced Frames, UK: Steel Construction Institute, 1997.
[5] British Standards Institute BS EN 10113 (1993). *Hot rolled products in weldable fine grain structural steels.* British Standards Institution, London.
[6] Allen, P et al (1992) Steel Construction Institute and British Constructional Steelwork Association, *"Joints in Simple Construction. Volume 1: Design Method",* London.
[7] Weynand, K., Jaspart, J. P., Steenhuis, M. (1998). "Economy Studies of Steel Frames with Semi-Rigid Joints". *Journal of Constructional Steel Research.* Vol 1. No. 1-3. Paper No. 63.
[8] M. Md. Tahir, Design of Semi-Continuous Construction for Multi-Storey Braced Steel Frames Using TWP sections, Malaysia: Steel Technology Centre, UTM, 2003.
[9] Chen, W. F. and Kishi, N. (1989). Semi-rigid Steel Beam-to-Column Connections: Database and Modelling. *Journal of Structural Engineering.* Vol. 115, No. 1, pp 105-119.
[10] Abdalla, K. M. and Chen, W. F. (1995). Expanded Database of Semi-Rigid Steel Connections. *Computer And Structures.* Vol. 56, No. 4, pp 553-564.
[11] Abdalla, K. M, Abu-Farsakh, G.A.R, Barakat, S.A (2007) "Experimental investigation of force-distribution in high-strength bolts in extended end-plate connections", *Steel and Composite Structures*, SCS, Vol. 7, No 2 pp 87-103.
[12] Coelho, Ana M. Girao, Bijlaard Frans S.K. and Simoes da Silva, Luis (2004), Experiemental assessment of the ductility of extended end-plate connections", *J. Eng. Struct*, 26, pp 1185-1206.
[13] Shi, W.L., Li, G.Q., Ye, Z. M. and Xiao, R.Y. (2007), "Cyclic Loading Tests on Composite Joints with Flush End Plate Connections", *International Journal of Steel Structures*, Vol 7, pp 119-128.
[14] Tahir, Md. M and Arizu (2008), "Experimental Tests on Composite and Non-Composite Connections Using Trapezoid Web Profiled Steel Sections", *International Journal of Steel Structures,* Vol 8, No. 1, pp 43-58.
[15] Tahir, Sulaiman, and Saggaff (2008), "Structural Behaviour of Trapezoidal Web Profiled Steel Beam Section Using Partial Strength Connection", *Electronic Journal of Structural Engineering*, Vol 8, pp 55-66.
[16] Steel Construction Institute and British Constructional Steelwork Association, Joints in Simple Construction. Volume 2: Practical Applications, First Edition, UK: SCI & BCSA, 1992.
[17] British Standards Institute BS 5950-1. (2000). *Structural Use of Steelwork in Building Part 1: Code of Practice for Design – Rolled and Welded Sections*. British Standards Institution, London.
[18] Tahir, Md M (1997). *"Structural and Economic Aspects of the Use of Semi-Rigid Joints in Steel Frames".* Ph.D Thesis. University of Warwick, United Kingdom.
[19] Bose, B. Tests to verify the performance of standard ductile connections, Dundee Institute of Technology, 1993.
[20] Azman (2001). *"Behaviour of partial strength connections with Prewaja Steel Section".* M.Phil Thesis. Universiti Teknologi Malaysia, Malaysia.