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COMPARISON OF PARTIAL STRENGTH CONNECTION BETWEEN EXTENDED AND FLUSH END-PLATE CONNECTIONS WITH TRAPEZOID WEB PROFILED STEEL SECTIONS

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INTRODUCTION

Traditionally building frames and designed as braced or unbraced, usually comprised beams and columns assembled and arranged in a 'rectangular box' manner. The connection between the beam and column is either assumed as pinned, where only nominal moment from the beam is transferred to the column, or rigid or full strength, where full continuity of moment transfer exists. Alternatively, EC 3 (ENV 1993-1-1: 2002) allows building frames to be designed using semi-rigid or partial strength connection, a type of design that utilised a condition between the simple and rigid design; provided that the moment resistance of the connection is known. In designing a multi-storey braced frame for a semi-continuous construction, guides and references are based on the EC 3 specification(ENV 1993-1-1: 2002) where it offers the possibility to account for partial strength joints in designing multi-storey frames, BS 5950-1: 2000 specification(BS 5950-1, 2000) and Steel Construction Institute publication (Couchman, G. H. 1997). When incorporated into the construction of a whole frame, the partial strength connection is associated with semi-continuous construction where partial continuity does exist between the beam and column. Unlike the traditional design approaches (simple and rigid), semi-continuous design requires the need to know the value of moment resistance of the connection. The most important characteristics of partial strength connection are the rotational stiffness (or rigidity), the strength and the ductility (or rotation capacity). These can be achieved by carrying out full scale testing on both the flush and extended end-plate connection.

WHY THE NEEDS TO USE PARTIAL STRENGTH CONNECTIONS?

In the design of braced multi-storey steel frames, the 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 (Allen, P, 1994). 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 reduction of the depth of the beam which result in reduction of the height of the cladding, reduce the steel weight which result in the reduction of overall cost and robust connection are significant, the disadvantages of this approach should also be addressed. The disadvantage in this approach is that it may be marginally more expensive, depending on the cost of labour paid which varies between European and Asian countries to fabricate partial-strength connection rather than simple connections. In Malaysia where the cost of labour is relatively low compared with Europe, the use of the proposed connections will be an added advantage. The benefits of overall cost saving of the partial strength connections have proven to be more than simple connections (Couchman, G. H. 1997). It is reported that the savings in steel weight of using partial strength connection alone (non-composite) in multi-storey braced steel frames using British hot-rolled section was up to 12% (Weynand, K, 1998). The overall cost saving was up to 10% of the construction cost which is quite significant according to the cost of labour in the United Kingdom (Tahir, Md. M, 1997).

FORMATION OF TRAPEZOID WEB PROFILED (TWP) STEEL SECTION?

A trapezoid web profile plate girder is a built-up section made up of two flanges connected together by a thin corrugated web usually in the range of 3 mm to 8 mm. The web is corrugated at an angle of 45 degree and welded to the two flanges using automated machine. The web and the flanges comprised of different steel grade depending on design requirements. TWP section is also classified as a hybrid steel section as two different types of steel grade are used in the development of the section. The steel grade of the flanges is designed for S355 and the steel grade of the web is designed for S275. The flanges are purposely designed for S355 so that the flexural capacity of the beam can be increased and the web is designed for S275 so as to reduce the cost of steel material. The capacity of shear is usually not that critical in the design of the beam (Sulaiman, A, 2006). The use of different steel grades in the fabrication of TWP section leads to further economic contribution to steel frames design besides the use of partial strength connection. The use of thick flanges, thin web and deeper beam for TWP section compared with hot-rolled section of the same steel weight leading to heavier load capacity and greater beam span that can be achieved.

TEST ARRRANGEMENT AND PROCEDURES

In conducting the full-scale test, a test rig was designed by the authors and erected to accommodate a column height of 3 m and a cantilever beam span of 1.5 m. The rig consists of channel sections pre-drilled with 22 mm holes for bolting purposes. The sections were fastened and bolted to form loading frames, which were subsequently secured to the laboratory strong floor as shown in Figure 1. The height of the column was kept at 3 m to represent the height of a sub-frame column of multi-storey steel frame. The column was restrained from rotation at both ends. The beam was restrained as shown in the Figure 1 from lateral movement. The load was applied at a distance of 1.3m from the face of the column using a hydraulic jack. This distance was taken to represent the approximate length of hogging moment occurred on the actual semi-continuous construction steel beam.

After the instrumentation system had been set-up and the specimen had been securely located in the rig, data collection software in the computer was used to check the reading of all connected channels to the instruments on the specimen. Correction factors from calibration and gauge factors from manufacturer were input into the software prior to each test. The specimen was then loaded up to two-third of the predicted value. The reading of load was taken as point load applied for easier monitoring. A 5 kN increment was adopted so that uniform data and gradual failure of the specimen can be monitored. After reaching the two-third value, the specimen was unloaded back and re-initialised. This procedure was taken to enable the specimen is in the state of equilibrium prior to the actual testing.

After re-initialising the instrumentation system, the specimen was loaded as described above, but the load applied was not restricted to the two-third value. Instead, the specimen was further loaded until as substantial deflection of the beam can be observed. For each loading, a set of reading was taken for deflections, rotations, and applied load. The load application was continuously applied at this point by increment in the deflection of 2 mm instead of the load as before. This procedure was continued until the specimen had reached its failure condition. The failure conditions was considered to have reached when an abrupt or significantly large reduction in the applied load and when a large increase in the deflection of the beam.

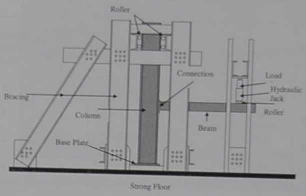


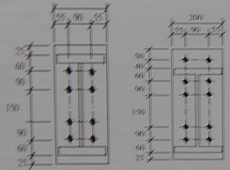
Figure 1. Testing arrangement

DESCRIPTION OF SPECIMENS

Four sets of partial strength connections specimens were arranged for the testing, namely two for flush end-plate(FEP) and two for extended end-plate(EEP) connections. The difference between FEP and EEP is the geometrical configuration of the connection where for the latter the bolt row is extended to the top of beam flange as shown in Figure 2(a and b). The size of beam, size of end plate and the diameter of bolts for both tests were shown in Table 1. For specimen with flush end-plate connection, the size of column was 254x254x107UC and for specimen with extended end-plate connection, the size of column was 305x305x118UC. Each of the specimens was connected to the column flange by different type of partial strength connections. The first arrangement was a flush end plate connection (FEP) comprised of two sets of connections with different size of beam. The beam size is increased from 400 to 450mm deep and the thickness of web is reduced from 6mm for specimen N1 to 4mm thick for specimen N2. The second arrangement was an extended end plate connection (EEP) comprised of two sets of connections which varies the width of the plate from 140 to 160mm and the number of tension bolt from 4 to 6 number. The number of shear bolt is maintained at 4 numbers for extended end-plate connection. The full-scale tests were conducted using a 1.3m metre length beams so as to observe the influences of the stiffness of the connection to the moment resistance of the connection. The arrangement of the cantilever beam was designed to understand the interaction or the effect of using partial strength connection to the moment resistance of connection on the actual beam.

Table 1: Geometrical configuration of the tested connections.

Test No	Model Name	Beam Size TWP	Number of Tension Bolts	Number of Shear Bolts	Endplate Size	Bolt
N1	FEP1P1-2 (Flush end-plate)	400x170x49/12/6	4	2	200 x 12	20
N2	FEP2R20P1 (Flush end-plate)	450x160x50.2/12/4	4	4		
N3	EEP2R20P1 (Extended end-plate)	400x140x39.7/12/4	4	4	200x12	20
N4	EEP3R20P1 (Extended end-plate)	450x160x50.2/12/4	6	4		



(a) Flush End - Plate (b) Extended End - Plate

Figure 2: Geometry configuration of partial strength connections

TEST RESULTS

The test results for the four connections specimens were presented based on the mode of failure and the behaviour of moment versus rotation curve. The test results were compared with the expected mode of failure predicted from the calculated moment resistance value proposed by Steel Construction Institute (SCI) (Brown, D; Allen P., 1996). SCI has proposed the use of "component method" to predict the mode of failure and the moment resistance of the connections by taking into consideration the failure at the tension zone, shear zone(horizontal and vertical shear zone) and compression zone as shown in Figure 3. Details of the discussion on the component method and the test results were presented elsewhere (Saggaff, A, 2006).

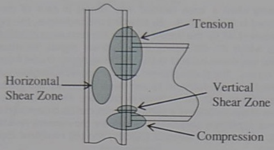


Figure 3: Critical zones to determine the moment resistance of the connection.

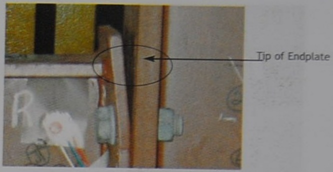


Figure 4(a): Deformation of flush endplate connection specimen N1

Figure 4(b): Deformation of flush endplate connection specimen N2

MODES OF FAILURE

At the initial stage of loading there were definitely no apparent visual deformations that can be observed in all of the tests. This was expected since the application of loads was intended for all components of the joint to be stabilised or to be in equilibrium. In addition, this stage was also meant for checking all of the instrumentation system prior to the actual commencement of the tests. After re-initialising, each specimen was then loaded until there was an indication that a 'failure' has been attained. This indication can be observed from the declination of the load after reaching a period of constant ultimate load. The test was then brought to a stop as any increment of load will deform further the specimen. During the tests, there was no occurrence of any vertical slip at the interface between the end-plate and the column. This was mainly due to the adequate tightness of the bolts carried out during the installation and after the very initial stage of loading.

The first visible deformation observed was around the vicinity of the connection; and this deformation was limited to the tension region (the critical zone) of the joint due to the tension forces exerted through the top bolt rows. For the flush end-plate connection, the form of the deformation was the translation as the deformation of the tip of the end-plate away from the face of the column. This was in accordance to the first sign of yielding of the end-plate where the tip of the end-plate deformed as shown in Figure 4(a and b). The same mode of failure occurred for both N1 and N2 specimens. The deformation of the connection appeared to be symmetrical on both sides of the connection when looking from the plan view of the joint. Further loading of the specimens has resulted into more deformation of the tip of the end-plate.

For the extended end-plate connection tests, higher capacity was expected due to the addition of one row of bolts at the extended top portion of each end-plate. Hence, at the initial stage of loading, there was apparently no visible deformation in all specimens even up to the one third of the predicted load. Gradually, however, at about two third of the predicted load, the end-plates (at the tension region of the connections) had begun to show some deformation. Unlike the flush end-plate, since there existed one row of bolts at the extended top portion of the endplate, the deformation of the connection translated the end-plate away from the face of the column in a 'Y-shape' form. Again, this deformation corresponded to the mode of failure as shown in Figure 5 which occurred for both N3 and N4 specimens.

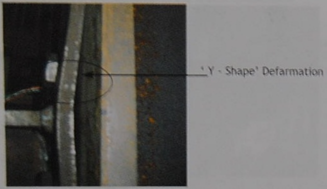


Figure 5: Deformation of extended endplate specimen into Y-shape.

As the applied load progressed, the end-plate started to yield and deformed. The deformation of the end-plate was then followed by the elongation of the bolts. There was hardly any deformation on the columns throughout the experimental programme of the isolated tests. This was as expected since the columns for all specimens (UC 254 x 254 x 107 for the flush end-plate connections and UC 305 x 305 x 118 for the extended endplate connections) were designed to adequately sustain the panel shear and the compression action along the bottom flange of the beam.

MOMENT - ROTATION CURVES

The prediction of moment resistance and the lost in stiffness is very much dependent on the stiffness of the connected members, types of joints, and orientation of the column axis (Tahir, Md. M, 2006). Beam-to-column connections generally have non-linear moment-rotation curves. Initially, the connections have a stiff initial response which is then followed by a second phase of much reduced stiffness. This second phase is due to in-elastic deformation of the connections' components or those of members of the frame in the immediate vicinity of the joint. These deformations need to be accounted for because they contribute substantially to the frame displacements and may affect significantly the internal force distribution. The structural analysis needs to consider this non-linearity of joint response to predict accurately both stiffness and resistance for a semi-continuous frame in case the joint behaviour exhibits a form of material non-linearity. The examples of shape of the experimental results for the M-* curve are shown in Figure 6 and Figure 7 for FEP connections and Figure 8 and Figure 9 for EEP connections. The moment resistance (MR) listed in Table 1 was determined by estimating when a "knee method" was formed in each of the M-* curves plotted in Figure 6 to 9. By adopting this technique, the experimental values of MR for the overall joint for the tests were established. For curves which do not clearly show a linear region, an assumed straight line was drawn parallel to the unloading region traced from the exact M-* curves. The graphs showed that the connections behaved linearly in the first stage followed by non-linear behaviour and gradually losing the stiffness with the increase in rotation. From this plot, the behavioural characteristics of a particular joint can be determined base on the three significant parameters, which are the moment resistance (strength), the rotational stiffness (rigidity) and the rotational capacity (ductility).

Table 1 also presents the theoretical values calculated from the component method proposed by SCI as mention earlier. Details of the component method are presented in SCI publication (SCI, 2002). The overall results showed that the experimental values of moment resistance were greater than the theoretical values with the ratio ranged in between 0.77 to 1.17 as shown in Table 1.

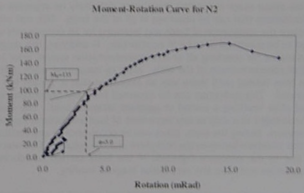


Figure 6: Moment versus rotation for specimen N1 (FB1P1-2)

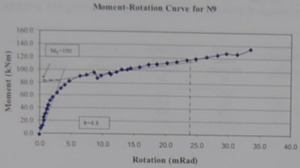


Figure 7: Moment versus rotation for specimen N2 (F2R20P1)

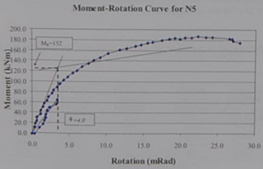


Figure 8: Moment versus rotation for specimen N3 (E2R20P1)

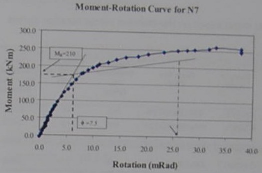


Figure 9: Moment versus rotation for specimen N4 (E3R20P1)

Table 1: Comparison between the experimental and theoretical values of moment resistances

Specimen	Name	Experimental $M_R(Exp)$	Theoretical $M_R(The)$	Ratio $M_R(Exp)/M_R(The)$
N1	FB1P1-1	135	103	1.31
N2	FB1P1-2	100	115	0.87
N3	E2R20P1	152	123	1.23
N4	E3R20P1	210	180	1.17

COMPARISON OF BEHAVIOUR OF FEP AND EEP CONNECTIONS.

The Comparison behaviour of FEB and EEP is best presented by comparing the moment resistance, the initial stiffness of the connection and the ductility of the connection. The results of moment resistance are tabulated in Table 1 whereas the initial stiffness and the ductility of the connection are tabulated in Table 2. The data in Table 1 and 2 are drawn by plotting M-Fcurves in Figure 6 to 9 as mentioned earlier. The behaviour of the connections depends on the geometrical configurations of the connection. However, not all geometrical configurations of the connection have significant effect to the behaviour of the connections. The size of the depth of the beam, the number, the size and the distance of the tension bolt, the thickness of the end-plate and the size of column may affect the moment resistance and the rotation stiffness of the connection. In this paper, the comparison of FEP and EEP connections is based on variables such as the changing in the depth of the beam, the number of tension bolts and the distance of the tension bolt.

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Table 2: Test result based on the moment versus rotation curves

Specimen	Name of model	Beam Size TWP	Moment Resistance M_R, (kNm)	Rotation, Φ (in mrad)	Initial Stiffness, $S_{j,ini} = M_R/\Phi$ (kNm/rad)
N1	FB1P1-2	400x170x49/12/6	135	5.0	27000
N2	F2R20P1	450x160x50.2/12/4	100	4.8	20833
N3	E2R20P1	400x140x39.7/12/4	152	4.0	38000
N4	E3R20P1	450x160x50.2/12/4	210	7.5	28000

MOMENT RESISTANCE OF CONNECTION FEP VERSUS EEP

The moment resistance of the connections depends on the depth of the beam and the number and size of the bolt. In these tested connection, the variables are the size of the beam and the number of tension bolts. Table 3 shows the increment in moment resistance of the connection by comparing the specimens FEP with the specimens EEP. Table 3 shows that the increment of 12.6% by comparing N1 with N3 and 55.5% by comparing N1 with N4. The result shows that the increment of 12.6% for N1 to N3 was due to the extra of two tension bolts for the N3 specimen. The increment increases to 55.5% as the extra number of tension bolts was increased to four numbers and at the same time the size of the beam increases from 400 to 450mm deep. Therefore the impact of the combination of extra tension bolt and the increase in the depth of the beam has resulted in a 110.0% increment of moment resistance of the connection.

Specimen N2 is compared with specimens N3 and N4, the increment of moment resistance of the connection are recorded in Table 3 as 52.0% and 110.0% respectively. The comparison between N2 and N3 has shown an increment of 52.0% even though the depth of the beam in N2 has been reduced to 400mm deep. However, due to the contribution of an extra of two numbers of tensions bolt in EEP specimen of N3 has resulted to a significant contribution to the moment resistance of the connection. The increment is very significant by comparing N2 with N4 as the connection has four number of extra tension bolts combined with deeper beam.

Table 3: Percentage of increment of moment by comparing FEP and EEP

Specimen	Moment Resistance M_R, (kNm)	N1(FEP) versus N3 (EEP) %	N1(FEP) versus N4(EEP) %	N2(FEP) versus N3(EEP) %	N2(FEP) versus N4(EEP) %
N1	135	N/A	N/A	N/A	N/A
N2	100	N/A	N/A	N/A	N/A
N3	152	12.6	N/A	52.0	N/A
N4	210	N/A	55.5	N/A	110.0

INITIAL STIFFNESS OF CONNECTION FEP VERSUS EEP

The overall result of comparison of initial stiffness for FEP versus EEP is shown in Table 4. The results in Table 4 show that the extended end-plate connections are stiffer than the flush end-plate connections in the range of increment from 3.8% to 82%. The significant increase of 40.7% by comparing N1 with N3 was due to extra bolt row that stiffen the connection. However, the increment of the stiffness start to reduce as the depth of the beam increases from 400mm to 450mm. The same behaviour also was recorded by comparing N2 with N3 and N2 with N4. The stiffness increases as the number of tension bolts increases but reduces as the depth of the beam increases.

Table 4: Percentage of increment of initial stiffness by comparing FEP and EEP

Specimen	Initial Stiffness, $S_{j,ini} = M_R/\Phi$ (kNm/rad)	N1(FEP) versus N3 (EEP) %	N1(FEP) versus N4(EEP) %	N2(FEP) versus N3(EEP) %	N2(FEP) versus N4(EEP) %
N1	27000	N/A	N/A	N/A	N/A
N2	20833	N/A	N/A	N/A	N/A
N3	38000	40.7	N/A	82.0	N/A
N4	28000	N/A	3.8	N/A	34.4

CONCLUSION

From the test results, the following conclusions can be drawn:-

i) Increment of moment resistance of the extended end plate is more than 50% the moment resistance of the flush end plate connection by the additional of two tension bolts and is more than 100% if the additional of two tension bolts is combined with the increase in the depth of the beam.

ii) The increment of stiffness of extended end-plate connection is more than the stiffness of flush end-plate connection in the range of 3.8% to 40.7%.