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EXPERIMENTAL TEST ON STEEL BEAM WITH PARTIAL STRENGTH CONNECTIONS USING TRAPEZOID WEB PROFILED STEEL SECTIONS

Arizu Sulaiman[1], Mahmood Md Tahir[1] and Anis Saggaff[2]

[1]Steel Technology Centre, Faculty of Civil Engineering, Universiti Teknologi Malaysia, 81310 Skudai, Johor, Malaysia
[2]Civil Engineering Department, Sriwijaya University, Jalan Raya Inderalaya, Palembang, Sumatera, Indonesia

INTRODUCTION

For a typical steel building frame, 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)[1] allows building frames to be designed as semi-rigid using the partial strength connection, provided that the moment resistance of the connection can be quantified. When incorporated into the construction of a whole frame, the type of construction that uses the partial strength connection is referred to as a semi-continuous construction, due to the partial continuity that exists between the beam and column. Unlike the conventional design approaches (simple and rigid), semi-rigid design requires the moment-rotation relationships of partial strength connection, which includes the moment resistance and rotational stiffness (rigidity), to be established prior to its usage in design. In this research, the behaviour of partial strength connections with TWP sections as beams was studied. The purpose of using TWP sections is to take advantage of the benefits offered by the sections in general and the effect of using partial strength connection in the design of the beam.

BACKGROUND

Partial Strength Connections

By definition, if the moment resistance or capacity of a connection is lower than the moment capacity of the connected beam, the connection is referred to as the partial strength connection. Two types of partial strength connections that are commonly used are the flush endplate connection and extended endplate connection. These connections consist of a plate, which is welded to the beam's end in the workshop, and then bolted to the column on site. In the case of extended endplate, the plate is extended above the flange of a beam and with one row of bolt in case of extended endplate connection.

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 . The fabrication of partial strength connections may be marginally more expensive since some degree of rigidity has to be provided. However, by using partial strength connections instead of simple connections, beam sizes could be reduced and significant overall savings of frame weight could be acquired . In Malaysia where the cost of labour is relatively low compare to Europe, the use of the proposed connections will be an added advantage. It was 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%. 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.

Trapezoid Web Profiled (Twp) Steel Sections

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. Since the web and flanges may comprise of different steel grades, TWP section is also classified as a hybrid steel section. The steel grade of the flanges is usually designed for S355 so that the flexural capacity of the beam can be increased, whilst the steel grade of the web is usually designed for S275, so that the cost of steel material can be reduced since the shear capacity is usually not critical. The use of different steel grades in the fabrication of TWP section leads to further economic contribution in addition to the contribution from using partial strength connections. The thick flanges, thin web and deeper beam of a TWP section in comparison to a hot-rolled section of the same weight lead to larger load carrying capacity and greater beam span.

EXPERIMENTAL TEST PROCEDURES

The aim of the experiment is to study the effects of partial restraint provided by the partial strength connections on the ultimate and serviceability of the TWP beam. A series of two isolated bare steel beam-to-column joints and two bare steel sub-assemblage beam-to-column joints were tested on a full-scale basis. A purpose-built test rig was designed and erected to accommodate a column height of 3 m and a beam span of up to 6 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(a) for the isolated tests and Figure 1(b) for the sub-assemblage tests.

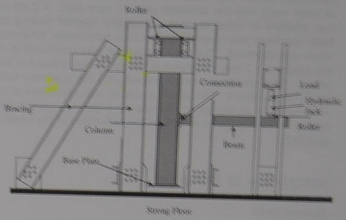

Figure 1(a): Arrangement for isolated tests

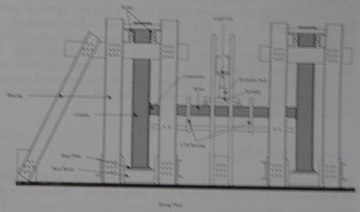

Figure 1(b): Arrangement for sub-assemblage tests

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 rotating at both ends whilst the beam was restrained from lateral movement as shown in the figures. In the isolated tests, the load was applied at a distance of 1.3 m from the face of the column. This distance was deemed adequate to cover the distance of the contra flexural point between the negative end moment of the joint and the positive moment of the beam. As for the sub-assemblage tests, the load was applied on the 6 m beam using a hydraulic jack at the mid-span, and was converted into a two-point load using a spreader of 1.8 m spreading distance. This distance was still within the standard distance of one third of the length of the beam so that a bending situation was assured.

After the instrumentation system had been set-up and the specimen had been securely located in the rig, the data collection software in the computer was checked to make sure that all channels connecting to the instruments on the specimen indicated a proper working condition. Correction factors from calibration and gauge factors from manufacturer were input into the software prior to each test. A 5 kN increment was adopted in order to have a gradually applied loading condition. Each specimen was then loaded until a substantial deflection of the beam was observed. The load application was continued at this point but adopting a 2 mm increment in the deflection instead of the load as before. This procedure was continued until the specimen had reached its failure condition. The failure condition was considered to have been reached when an abrupt or significantly large reduction in the applied load or when a large increment in the deflection of the beam has been attained.

DESCRIPTION OF SPECIMENS

Isolated Tests

In the isolated joint tests, two sets of beam-to column joints, which consisted of a 1.5 m beam, were tested. Each of the beam specimens was connected to the column flange using different types of partial strength connections. The first arrangement was a flush endplate connection (FEP) as shown in Figure 2(a), whilst the other arrangement was an extended endplate connection (EEP) as shown in Figure 2(b). The geometry of the two connections was identical except that there were two additional bolts on the extended part of the extended endplate connection. Details of the specimens for the two arrangements in the isolated joint tests are as shown in Table 1.

Sub-Assemblage Tests

For the sub-assemblage tests, two new arrangements, which consisted of a beam with a typical length of 6 m, were tested. The beam in each arrangement was connected to the column flange at both ends using the identical FEP connection and EEP connection as in the isolated joint tests.

The full-scale tests were conducted using a 6 m length beam so as to observe the influences of the mid-span deflection on the behaviour of the connections. Those influences can be listed as followed:

i) On the moment resistance
ii) On the rotational rigidity
iii) On the ductility

Table 1: Details of specimens

Model Name	Beam Size TWP	Column Size UC	Connection Type	Bolt Row (Top-Bot)	End Plate // Bolt	Beam Length (m)
	i) Isolated Test					
FSR20P11 (N9)	450x160x50/22/24	305x305x118	FEP	2(4+4)	200x12 // M20	1.5
EER20P11 (N7)			EEP	3(6+4)	200x12 // M20	1.5
	ii) Sub-assemblage Test					
FS-FSP24520 (N10)	450x160x50/22/24	305x305x118	FEP	2(4+4)	200x12 // M20	6.0
FS-EER20P11 (N12)			EEP	3(6+4)	200x12 // M20	6.0

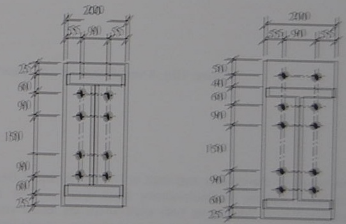

Figure 2: Geometry configuration of partial strength connections

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iii) On the ductility

Table 1: Details of specimens

Model Name	Beam Size TWP	Column Size UC	Connection Type	Bolt Row (Top-Bot)	End Plate / Bolt	Beam Length (m)
	i) Isolated Test					
F2R20P1 (N9)	450x160x50.2/12/4	305x305x118	FEP	2(4-4)	300x12 / M20	1.5
E3R20P1 (N7)			EEP	3(6-4)	200x12 / M20	1.5
	ii) Sub-assemblage Test					
FS-FSF2R20 (N10)	450x160x50.2/12/4	305x305x118	FEP	2(4-4)	300x12 / M20	6.0
FS-E3R20P1 (N12)			EEP	3(6-4)	200x12 / M20	6.0

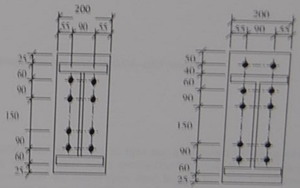

Figure 2: Geometry configuration of partial strength connections

DISCUSSION OF RESULTS

The results of the experiments were focused on the behavioural characteristics of the flush endplate and extended endplate connections in isolated tests, and the deflection at mid-span and the applied loads of the beam in sub-assemblage tests. The deflection at mid-span was compared with the deflection limit suggested by BS5950: 2000, Part 1 as L/360 for brittle material and L/200 for other than brittle.the moment resistance and rotation of the connections that associated with the mid-span deflection and load were also observed. The behaviour in term of the moment-rotation relationship of both types of connections was obtained from the isolated tests results.

Results of all tests are shown in Table 2. Figure 3 shows the plots of moment versus rotation for the FEP and EEP, whereas Figure 4 and Figure 5 show the plots of load versus mid-span deflection and load versus rotation for the two sub-assemblage tests. It was noticed that although both specimens of the sub-assemblage tests failed due to the buckling of the top flange at the centre of the beam as shown in Figure 6(a and b) and Figure 7. The connections possessed a ductility characteristic with a rotation capacity of 33.5 mRad for FEP and 32.8 mRad for EEP. The moment capacities, on the other hand, are 95 kNm for FEP, which are 0.28Mp and 225 kNm for EEP, which is 0.67Mp. These values are between 25% and 100% of the capacity of the connected beam, which categorise both connections as partial strength connections. Theoretically, the plastic moment, Mp or capacity of the beam is calculated to be 335.8 kNm.

Table 2: Test results

Reference	F2R20P1	E3R20P1
Moment Capacity (kNm)	95	225
Rotation Capacity (mRad)	33.5	32.8
Initial Stiffness (kNm)	30770	30770
Max Load (kN)	235	380
Mid-Span Deflection at Max Load (mm)	23	34
Rotation at Max Load (mRad)	5.20	6.10
Moment Resistance at Max Load (kNm)	85	150
Failure Mode of Isolated Specimens	Endplate yielding **(Figure 6(a))**	Endplate yielding plus bolt slipping **(Figure 6(b)**
Failure Mode of Sub-Assemblage Specimens	mid-span **(Figure 7)**	Buckling of top flange at mid-span followed by end plate yielding **(Figure 7)**

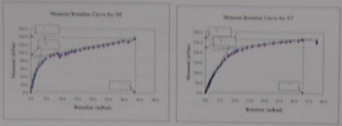

Figure 3: Moment versus rotation of FEP and EEP

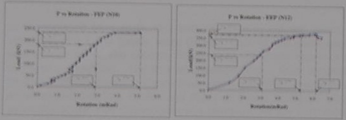

Figure 4: Load versus rotation of FEP and EEP

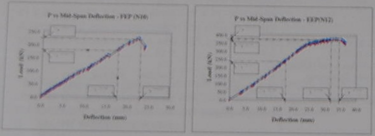

Figure 5: Load versus mid-span deflection of FEP and EEP

(a) (b)

Figure 6: Failure mode of a) FEP and b) EEP

Figure 7: Failure mode of beam on sub-assemblage tests

CONCLUSIONS

Based on the results obtained, several observations have been made which lead to the following conclusions:

i) Both types of connections have the same value of initial stiffness; however the moment capacity of the extended endplate connection is more than double the moment capacity of the flush endplate connection.

ii) The failure mode for the flush endplate in the isolated tests is endplate yielding, whilst for the extended endplate, the failure mode