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BEHAVIOR OF FLUSH END-PLATE ON TRAPEZOID WEB PROFILED STEEL BEAM WITH PARTIAL STRENGTH CONNECTIONS

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ABSTRAK

In steel structures, the connections will be designed whether simple, semi-continuous, or continuous construction. Many constructions are usually designed as simple which associated with simple construction or rigid which is associated with continuous construction. However, the actual behaviour between these two extreme cases took placed between simple and rigid, which is semi-rigid connection. 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 present the behaviour of flush endplate of steel beam with partial strength connections using Trapezoid Web Profiled (TWP) steel sections and Universal Beam (UB). The TWP steel section is a built up section where the flange is designed using S355 steel section and the web is designed using S275 steel section. Some tests have been carried out for beam with flush endplate connections as partial strength connections. The use of partial strength will also reduce the deflection of the beam as suggested by the Steel Construction Institute (SCI). The moment resistances and the deflection of the beam presented in this paper, and the behaviour of endplate connections have shown good agreement between experimental values and the predicted values. The results have shown that the partial restrained of the connections has contributed to the reduction in the deflection and the increasing in the moment resistance of the beam. The results also showed that the flush endplate failure mode has formed similar to the failure mode stated in the codes. It can be concluded that the use of partial strength connections has contributed to significant reduction to the deflection and significant increase to the moment resistance of the design of TWP steel section as a beam.

***Keywords:** Behaviour, Flush End-Plate, Trapezoid Web Profiled Section, Partial Strength Connections*

1. PENDAHULUAN

Steel building structures are widely used right now. The technology for using steel is also broadening. The study of using the connection between the beam and column in a frame is also studied significantly. The connections 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 as semi-rigid using the partial strength connection. 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. 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 has been studied. The purpose of using TWP sections is to take advantage of the benefits offered by the sections in general.

2. BACKGROUND

Trapezoid Web Profiled (TWP) Steel Sections.

A trapezoid web profile is a built-up steel section made up of two flanges connected together by a thin corrugated web usually in the range of 2 mm to 8 mm. The web is corrugated at an angle of 45 degree and welded to the two flanges by 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 as shown in figure 2.1 in comparison to a hot-rolled section of the same weight lead to larger load carrying capacity and greater beam span.

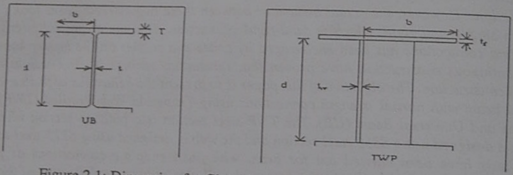
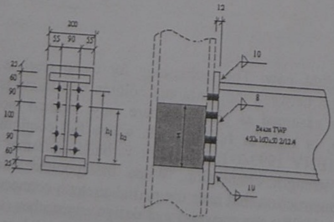


Figure 2.1: Dimension for Classification of UB and TWP Sections

Flush End-Plate

Typically, flush endplate connections are the most suitable for use in semi-continuous braced frames due to the advantages that they possessed. Annex J in EC3 (1992) in conjunction with Section 6.9 describes the types of connections, design methods and procedures to be adopted for beam-to-column connections. A traditional triangular distribution is modified to acquire a more accurate distribution of bolt forces using a plastic distribution approach. Specifically, the model followed and used by EC3 is called the component method of which a particular connection is divided into three critical areas or zones. These zones are (SCI, 1996):



Zone	Ref	Failure mode
Tension	a	Bolt tension
	b	End plate bending
	c	Column flange bending
	d	Beam web tension
	e	Column web tension
	f	Flange to end plate weld
	g	Web to end plate weld
Horizontal shear	h	Column web panel shear
Compression	j	Beam flange compression
	k	Beam flange weld
	l	Column web crushing
	m	Column web buckling
Vertical shear	n	Web to end plate weld
	p	Bolt shear
	q	Bolt bearing (plate or flange)

Figure 2.2: Arrangement for isolated tests

The resistance of these zones will determine the moment resistance of a beam-to-column connection. Figure 2.2 shows an end plate connection with the critical zones affected during loading.

Partial Strength Connections

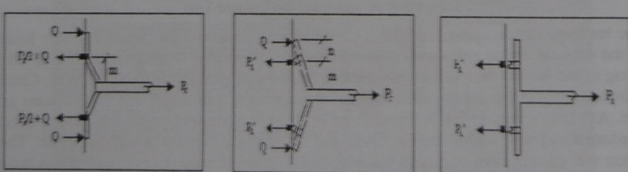
There are two types of partial strength connections that are commonly used are the flush endplate connection and extended endplate connection. These two connections are made of a plate, which is welded to the beam's end in the workshop, and then bolted to the column on site. For the connection of extended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

In designing 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. The use of the proposed connections will be an added advantage. It has been 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%. The overall cost saving was up to 10% of the construction cost.

Failure Mode of the Connections

There are three kinds of failure of the connections during loading applied to the structures as shown in figure 2.3;

- Complete flange yielding mode; the strength of the flange is weaker than to the strength of the bolts. The flange will yield but the bolts are still intact shown in figure 2.3 (a)
- Bolt failure with flange yielding; the strengths of the flange and the bolts are about the same. As a result, both the flange and the bolts will yield together upon failure. This mode of failure is shown in figure 2.3 (b).
- Bolt failure; the strength of the bolts is weaker than the strength of the flange. Upon failure, the bolts will yield (or even break), the flange is still intact, shown in figure 2.3 (c)



(a) Complete flange yielding (b) Bolt failure with flange yielding (c) Bolt failure

Figure 2.3: Modes of failure of equivalent T-stubs (Adapted from SCI (1995)

In designing 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

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3. EXPERIMENTAL PROGRAMME AND PROCEDURE (REVISE)

The inspiration 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 bolted to form loading frames, which were subsequently secured to the laboratory strong floor as shown in Figure 3.1 for the isolated tests. The column height was 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 whilst the beam was restrained to avoid lateral movement as shown in the figures. In the isolated tests, the load was applied using a hydraulic jack 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. This distance was still within the standard distance of one fourth of the length of the beam so that a bending situation was assured.

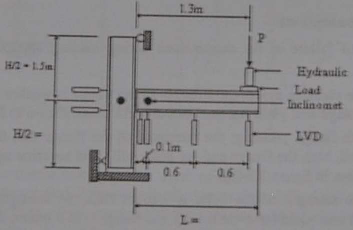

Figure 3.1: Arrangement for Isolated Test

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 properly 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.

In the isolated joint tests, four 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 a different type of partial strength connections. The arrangement was a flush endplate connection (FEP) as shown in Figure 3.1. The geometry of the connections was identical of the endplate connection. Details of the specimens for the arrangements in the isolated joint tests are as shown in Table 1.

Table 3.1: Details of specimens

Test Specimen	Beam Size TWP	Column Size UC	Connection Type	Bolt Row (Top-Bot)	End Plate	Beam Length
FT1B1-1	300x130x37/12/6	254x254x107 UC S-275	FEP	1(2-2) M20	200x12	1.5 m
FT2B1-1	400x170x49/12/6		FEP	2(4-2) M20	200x12	1.5 m
FT2B2-1	530x210x83/16/8		FEP	2(4-4) M20	200x12	1.5 m
FT2B2-1	680x250x117/20/ 8		FEP	2(4-4) M20	200x12	1.5 m

4. DISCUSSION AND RESULTS

The experiments were focused on the behavioural characteristics of the flush endplate connections in isolated tests with the applied loads of the beam in the tests. The deflection of beam was compared with the deflection limit suggested by BS5950: 2000, Part 1. On the other hand, the moment resistance and rotation of the connections that associated with the deflection and load were also considered. As The results, the behaviour of endplate in term of the moment-rotation relationship of the connections was obtained from the isolated tests results.

Modes of Failure of Endplate.

At the very initial stage of loading there were definitely no apparent visual deformations observed in all the tests. This was expected since the application of loads was intended for all components of the joint to be 'embedded' in the arrangement (or to be in equilibrium) prior to the commencing of the actual test. 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, and so the test was brought to a stop. During the tests, there was no occurrence of any vertical slip at the interface between the endplate and the column. This was mainly due to the adequate tightness of the bolts carried out during the installation and after the initial loading.

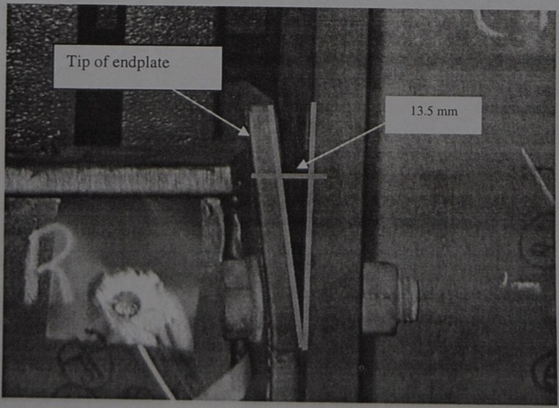

Figure 4.1: Deformation of flush end plate connection specimen 1 (FT1B1-1)

Started with yielding of the end plate, further increase of the applied load had not only deformed the end plate more but it deformed the rows of bolts above and below the top flange of the beam. The applied load was then released at about one third of the predicted load for all specimens to ensure that the behaviour of the connections was in a linearly elastic state at that particular range. It was found out that the recovery of the loads in all specimens was in a linearly elastic manner, which corresponded to the initial stiffness of the connections. Even after failure, when releasing the applied loads, the slope of the drop of the loads was still corresponded to the initial stiffness of the connections.

The first visible deformation observed was around the endplate of the connection; and this deformation was limited to the tension region of the joint due to the tension forces exerted through the top bolt rows. The form of the deformation flush endplate connection was the translation of the tip of the endplate away from the face of the column. This corresponded to the first sign of yielding of the endplate, which could lead to the Mode 1 failure. 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 endplate. The flush endplate connection specimens that experienced this type of failure were the FT1B1-1, FT2B1-2, FT2B2-3, and FT2B2-4. Figure 4.1 shows the deformation of the flush endplate of specimen FT1B1-1 that brought about the failure mode of the connection 13.5 mm. Figure 4.2 shows the deformation of the flush endplate connection of specimen FT2B1-2, which brought to the deformation of endplate 12.5 mm.

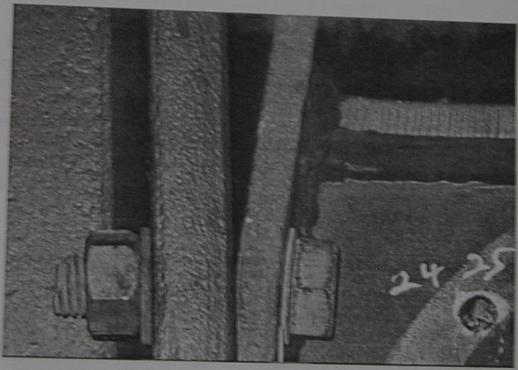

Figure 4.2: Deformation of flush end plate connection specimen 2 (FT2B1-2)

As for the specimens FT2B2-3, and FT2B2-4, the form of deformation was the translation of the tip of the endplate away from the face of the column followed by a slight elongation of the top row bolts and a slight buckling of the web around the tension region of the connections. This failure mode is usually referred to failure mode. Figure 4.3 has shown the failure of the FT2B2-3. Bigger beam of specimen FT2B2-4 might have caused the small deformation around the endplate.

For column flange or endplate bending, the approach taken (by EC 3 and SCI) is representing the yield line patterns that occur around the bolts by using equivalent T-stubs. This approach results in checking against three modes of failure as follows:

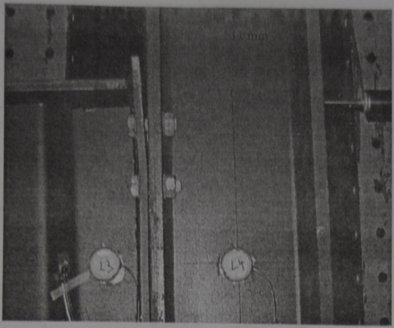

Figure 4.3: Deformation of flush end plate connection specimen FT2B2-3

Table 4.1: Test result based on the moment versus rotation plots

Specimen Name	Moment, M_U (kNm)	Rotational, ϕ_C (mRad)	$\frac{M_U}{M_R}$	FAILURE MODE
FT1B1-1	98	23.0	1.1	Endplate Yielding
FT2B1-2	168	14.7	1.2	Endplate Yielding
FT2B2-3	330	11.5	1.2	Endplate Yielding and Slight Bolt Slipping
FT2B2-4	511	10.8	1.2	Slight Endplate Yielding and Bolt Slipping

Table 4.2: Ratio between the loads and flush endplate deformation for experimental

Specimen Name	Flush Endplate Knee-Joint			Flush Endplate value Ultimate		
	Load', P_j (kN)	Deflection, δ_j EP (mm)	P_j/δ_j EP (kN/mm)	Load, P_u (kN)	Deflection, δ_u EP (mm)	P_U/δ_U EP (kN/mm)
FT1B1-1	67.7	12	5.64	75.4	13.5	5.89
FT2B1-2	109.2	10.5	10.40	129.2	12.5	10.34
FT2B2-3	207.7	9.5	21.86	253.8	11	23.07
FT2B2-4	334.6	8	41.82	393.1	9	43.68

As for the specimens FT2B2-3, and FT2B2-4, the form of deformation was the translation of the tip of the endplate away from the face of the column followed by a slight elongation of the top row bolts and a slight buckling of the web around the tension region of the connections. This failure mode is usually referred to failure mode. Figure 4.3 has shown the failure of FT2B2-3. Bigger beam of specimen FT2B2-4 might have caused the small deformation around the endplate.

All the results of the all test are shown in Table 4.1 and Figure 4.6 shows the plots of moment versus rotation for the FEP, whereas Figure 4.5 and Figure 4.6 show the plots of load versus deflection for the four isolated tests using Flush Endplate connections. It was noticed that although the fourth specimens of the isolated tests failed due to the buckling of the top endplate at the connection of the beam, the connections possessed a ductility characteristic with a rotation capacity. The moment capacities, on the other hand, are 98 s/d 511 kNm for FEP.

These values $\frac{M_U}{M_R}$ are between the 1.1 and 1.2, which categorise the connections as partial strength connections. The plastic moment, Mp or capacity of the beam is calculated to be 560 kNm.

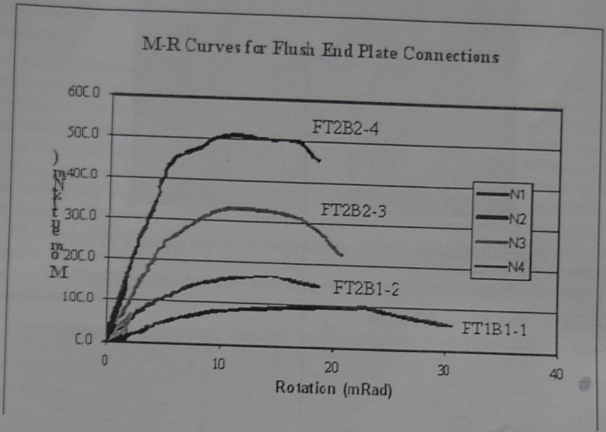

Figure 4.4: Moment versus rotation for all tests on FEP

Figure 4.4 has shown the plots of the moment versus rotation for all tests on flush end plate connections. From these curves, it is observed that the value of the moment resistance increases as the size of the beam increases. The initial stiffness also increases as the size of the beam increases. However, as the size of the beam increases, the connections seem to lose their ductility. This is evidenced through the rotation of the joint that is decreases as the size of the beam increases.

b). Load versus Deflection Curves.

Figure 4.5 and Figure 4.6 show the load versus deflection curves at the location of the maximum deflection on the beam for all nine specimens (labelled as DT4 for specimens FT1B1-1, FT2B1-2, FT2B2-3, and FT2B2-4.).

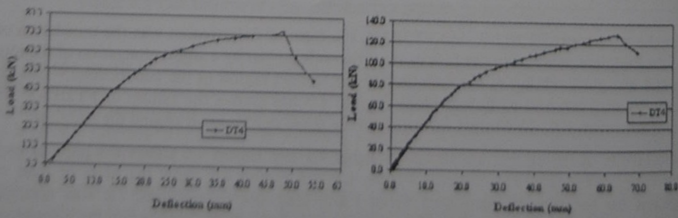

Figure 4.5: Load versus deflection for specimen (FT1B1-1) and (FT2B1-2)

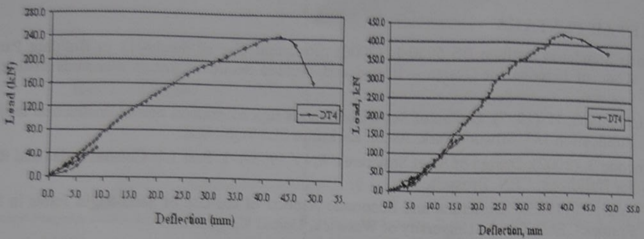

Figure 4.6: Load versus deflection for specimen (FT2B2-3) and (FT2B2-4)

The maximum load of each plot clearly represents the ultimate load that can be sustained by the respective joint. Indicated also in each plot is the load (capacity) based on the 'knee-joint' method. In using this method, a tangential line from the origin is drawn that represents the linear region followed by another tangential line that best represents the non-linear region. The capacity can then be determined by projecting horizontally from the intersecting point between these two lines to the vertical axis. Table 4.2 summarises the results based on the plots of load versus deflection for the entire specimens. From the results, it can be seen that the ratio of P/δ whether at 'knee-joint' or ultimate increases as the beam size increases. In addition, the values of the ratio are higher for the flush end plate connections with bigger beams, which indicate that the connections are stiffer.

5. CONCLUSIONS

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

1. The flush endplate connection has the value of initial stiffness, however the bigger beam with more bolts of the connection has more moment capacity of the connection.
2. The failure mode for the flush endplate in the isolated tests has shown that endplate yielding during failure.
3. The deflection of the isolated specimens has reached its limit at the time the moment capacity of the endplate connections has been reached.
4. The failure modes for fourth specimens are due to the buckling of the top endplate. This shows that the tension forces induced in that area has caused the buckling of the flange to occur before any typical mode of failure of the connection occurs.
5. Partial strength connections with TWP sections can provide sufficient moment capacity and rotational capacity as outlined by the EC3 for semi-continuous construction.

6. ACKNOWLEDGEMENTS

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