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EXPERIMENTAL TEST ON BEAM-TO-COLUMN OF PARTIAL STRENGTH COMPOSITE CONNECTIONS USING TRAPEZOID WEB PROFILED STEEL SECTIONS

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

[1]Assoc. Prof./Director, Steel Technology Centre, Faculty of Civil Engineering, Universiti Teknologi Malaysia, 81310 UTM Skudai, Johor
[2]Researcher, Steel Technology Centre, Faculty of Engineering, Universiti Teknologi Malaysia, 81310 UTM Skudai, Johor

INTRODUCTION

The use of composite beam in buildings has known to increase the loading capacity and stiffness of the composite construction. The benefits of composite beam action result in significant savings in steel weight and reduce the depth of the beam. To obtain more economical structural design against the bare steel beams, composite beam was designed by taking the advantage of incorporating the strength of concrete slab by the use of shear connectors. The composite action due to the interaction of steel beam and concrete slab with shear connectors increases the load-carrying capacity and stiffness of composite beam. These advantages of composite beam contributed to the dominance of composite beam in the commercial building in steel construction industry. The advantages of composite construction have been extended with the use of composite connection. This partial strength composite connection enhanced the stiffness of the connection and increased the weight saving of the design of composite beam.

The termed partial strength composite connection is usually associated with a connection having a moment resistance less than the moment resistance of the connected beam. Partial strength composite connection is the term used for connection in the design of semi-continuous construction for multi-storey steel frames by Eurocode 3 where the strength of reinforcement embedded inside the concrete slab was taken into account in improving the moment resistance and the stiffness of the connection. In semi-continuous frame the degree of continuity between beam and column 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 represents the section between pin joints and rigid joints. By adopting this approach, studies conducted on the use of partial strength connection have proven substantial savings in overall steel weight. 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 composite connections for Trapezoidal Web Profiled(TWP) sections has not been established yet. Therefore, this paper intends to establish the standardized tables for partial strength composite connections for TWP sections based on the proposed method by SCI.

DEVELOPMENT OF TRAPEZOIDAL WEB PROFILED STEEL SECTION

A trapezoid web profiled steel section is a built-up section comprised of two flanges connected together to thin corrugated web usually between 3 to 8mm thick by a fillet weld as shown in Figure 1. The web is corrugated at an angle of 45 degree and welded to the two flanges using automated welding machine. The web and the flanges comprised of different steel grade depending on the design requirements. TWP section is also classified as a hybrid steel section as two different types of steel grade were 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 for maximum flexural capacity and the web is designed for S275 so as to reduce the cost of steel material. The shear capacity and bearing capacity are usually not that critical in the design of the beam as the web is corrugated. The shape of trapezoid web is designed to accommodate shear forces and to increase the crushing and buckling resistance of the TWP web. The size of the flange varies from 10mm to 60 mm thick with the width in the range of 100 mm to 500 mm. The size of flange and the depth of beam will determine the moment capacity of the TWP beam. The depth of the TWP beam varies from 200 mm to 1600mm. The web and the flanges comprised of different steel grade depending on design requirements. The depth of the beam which can reach 1600mm deep is an added advantage to TWP section with greater moment resistance and longer beam span as compared with limited depth of hot-rolled section which can only reach up to 900mm deep. 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.

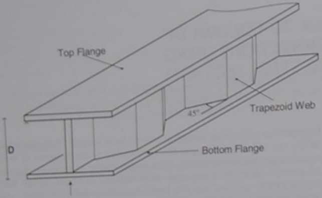

Figure 1: Geometrical configuration of Trapezoidal Web Profiled Steel Section.

ADVANTAGES OF TWP SECTION

Based on the configuration of the structure, TWP beam can offer substantial saving in the steel usage, and in some cases up to 40% as compared to conventional rolled sections according to research done by Osman M. H. (1998). The advantages of using TWP sections are more significant when there is a need for a column free area, long span structural system such as portal frames for warehouses, girder for bridges, floor and roof beam for high-rise buildings, and portal frame for factory. The advantages of TWP beam as compared to the conventional plate girder or conventional hot rolled steel section can be listed as follows (Wail, 2001):

1. The corrugated web will eliminate or minimize the need of stiffeners which result in stronger web compression capacity that can provide lighter section weight, optimizing of steel used, and reduction of fabrication cost.
2. The use of much deeper section will increase the flexural capacity that will also result in longer span and lesser deflection.
3. It increases lateral torsion buckling resistance due to corrugated web.
4. The manufacturing of TWP is fully automated production line which ensures high quality product and reduces time for fabrication.
5. The manufacturing of TWP beam is based on the design required according to the size needed or 'tailor made', thus eliminating any wastage of steel.
6. The production line is capable of manufacturing up to 1.60m depth which is not provided for hot rolled section. This advantage will offer range of choice for most structural usage especially for long span structures.

However, there are some disadvantages of TWP profile. This profile is quite complicated to fabricate due to its trapezoid web shape which means that the use of state of the art machine cannot be avoided; therefore, the initial production of TWP section is quite expensive. TWP section is usually connected to the column as a pin jointed connection in composite beam design. In this study, the proposed connection is a partial strength composite connection. The definition and identification of connections types are discussed below.

DESIGN OF COMPOSITE CONNECTION

The design philosophy presented in this paper was adopted from 'component approach' described in SCI. The moment capacity of the connection was determined by considering the capacity of each relevant component such as the tensile of the top bolt row and the tensile capacity of the reinforcement bar anchored inside the concrete slab. The composite connection capacity was also checked to meet the requirement of BS 5950:2000 Part 1. The moment resistance of the connection was developed by coupling tension force in the reinforcement and the upper bolt with the compression of the beam flange to the column at the lower part of the beam. The lever arm to calculate the moment capacity was established by considering the distance between the components of the tension zone and the compression zone.

TENSION ZONE

The tension zone comprises of three components that govern the magnitude of the tensile force which contributes to the moment capacity of the connection. These three components are listed as the reinforcement bar, the upper row of bolts and the longitudinal shear force.

TENSILE FORCE IN THE REINFORCEMENT BAR

The position of the reinforcement bar is very important in contributing to the moment capacity of the connection. The distance of the reinforcement bar to the compression flange of the beam will be used in the determination of the moment capacity of the connection. Tests and models have shown that connection rotation capacity increases as the area of reinforcement increases. The minimum area for reinforcement bars is needed to ensure that the connection can undergo sufficient rotation to strain the reinforcement to yield. The rebar elongation limit suggested by SCI is up to 10%. The determination of moment capacity is based on the assumption that the reinforcement bar yields. The contribution to moment capacity of any mesh reinforcement used in the concrete slab should be ignored, because it fails at lower values of elongation than the reinforcement bars.

TENSILE FORCE IN THE BOLTS

The composite connection usually uses flush end-plate connection where the tension bolt is underneath the upper beam flange. Most of the moment capacity of the connection is developed by the contribution of tension reinforcement bar. However, the contribution of upper bolt row still needs to considered to ensure that the compression zone is not under design which can lead to be premature failure due to non-ductile compression zone. The bolt row furthest from the beam compression flange tends to attract more tension than the lower bolts. The force permitted in the bolt row is based on its potential resistance based on the size of bolt and the thickness of the end-plate, not only the lever arm of the bolt. Details of the formulation and calculation to predict the tension values of the reinforcement and bolts are shown elsewhere.

LONGITUDINAL SHEAR FORCE

The formation of full tensile force in the reinforcement depends on the longitudinal shear force being transferred from the beam to the slab by the shear connectors and TWP section as shown in Figure 2. According to the requirement of BS5950; Part 3, full shear connectors should be provided in the region of positive moment. The reinforcement used in the connection should be extended beyond the negative region of the span and anchored sufficiently into the compression region of the slab to satisfy the requirements of BS 8110.

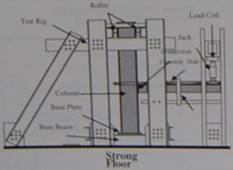

Figure 2: Arrangement of tested specimen on the test rig.

COMPRESSION ZONE

The strength of the compression capacity of the connection relies on the flange and web of the lower beam and on the resistance of the column web. The failure modes on compression zone are due to crushing or buckling of the column web or on the flange or web of the lower beam. This compression zone however, is not designed for the use of stiffener at the web of the column so as to reduce the cost of fabrication. Therefore, the calculated value of moment resistance in the tables developed for TWP section is only for unstiffened column web.

EXPERIMENTAL TESTS

The use of partial strength connection for hot-rolled British sections is well established by SCI. A series of tests at the University of Albertay, Dundee were successfully carried out to verify the predicted moment and shear capacity with the experimental tests capacities (Allen, P, 1994). The results confirmed with the predicted values and the standardized tables for the connection published by SCI. However, these tests were carried out for hot-rolled steel section and the connection was a non-composite connection. In this paper, the connection was categorized as composite connection and the sections used were TWP sections. The standard flush end-plate composite connections tables for TWP sections were developed in accordance to Eurocode 3 and SCI procedures. The best validation of the results presented in the tables is by comparing the predicted results with extensive experimental tests results. However, due to higher cost to conduct an extensive full scale testing, four tests have been carried out to validate the presented standard connection tables for TWP section. The test specimens are listed in Table 1, from which it can be seen that the parameters were varied in systematic manner. In the experiment, four number of 16mm diameter reinforcement bar were used and the thickness of the slab is 125mm with the concrete grade of G-30.

Table 1: Test specimens with various parameters.

Specimen No.	Bolt Rows	Bolts	End Plates			Size of TWP Beam	Size of Column
			W	D	T		
C5	1	M20 8.8	200	440	12	400x140x39.7	305x305x118
C6	1	M24 8.8	250	540	15	500x180x61.9	305x305x118
C7	2	M20 8.8	200	490	12	450x160x50.2	305x305x118
C8	2	M24 8.8	250	640	15	600x200x73.3	305x305x118

TEST PROCEDURES

Test specimens were set-up by connecting a 3 m long column with a 1.5 m long beam as shown in Figure 2. A metal decking with 1.5 m width which acts as a permanent formwork for the slab was attached to the top flange of the column by a pair of shear stud on each though. The shear studs measured at 19mm diameter and 95mm height was used. The thickness of the slab was taken as 125mm thick with concrete strength of grade 30. Four reinforcement of size 16mm diameters were installed around the column and embedded to the slab as shown in

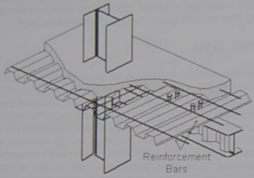

Figure. 3. The position of reinforcement around the col umn and embedded to the slab

Load was applied through an automatic operated hydraulic jack and monitored with a pre-calibrated 100 tonnes capacity load cell. The data logger system was set-up to read displacements from inclinometer in millimeters and load in kN. A small load was applied and then removed, to check the performance of the rig. Significant load was then applied, sufficient to cause extensive inelastic deformation of the connection. To determine the complete response, each connection was later subjected to unloading, followed by reverse loading. The following load sequence was generally used. An increment of 5 kN was applied to the specimen. The readings for loads, displacement, and rotation were recorded after two minutes had elapsed. This time elapse of 2 minutes allowed the specimen to reach an equilibrium state. The incremental load procedure was then repeated until there was a significant increase in deformation. The loading on the specimen was then controlled by deflection increments of 3mm. The test was continued until failure, when large deformation or the load decreases significantly. The response of a joint in these phases may govern the buckling behavior of the connected column. A graph of rotation of the connection was plotted against moment to predict the moment resistance of the connection.

PREDICTION OF MOMENT RESISTANCE (MR) AND DISCUSSION OF RESULTS

The prediction of moment resistance and the lost in stiffness are very much dependent on the stiffness of the connected members, types of joints, and orientation of the column axis(Tahir, Md. M, 1997). 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 significantly affect the internal force distribution. The structural analysis needs to account for 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 the experimental shape results for the M-* curve are shown in Figure 4 to Figure 7.

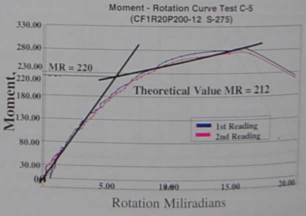

Figure 3: Moment Rotation Curve Test C-5 (Mrad)

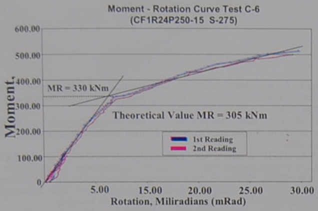

Figure 4: Moment Rotation Curve Test C-6

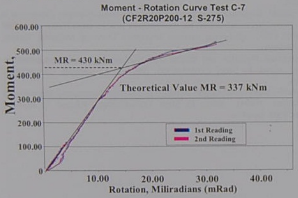

Figure 5: Moment Rotation Curve Test C-7

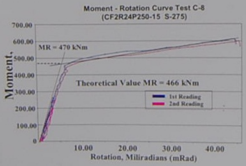

Figure 6: Moment Rotation Curve Test C-8
Article IV.

The experimental values of moment resistance MR listed in Table 2 was determined by estimating when a "knee" formed in each of the M-* curves plotted in Figure 4 to Figure 7. By adopting this technique, the experimental values of moment resistance (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. The overall results showed that the experimental values of moment resistance were greater than the theoretical values with the ratio ranged in between 1.01 to 1.23 as shown in Table 1.

Table 2: Theoretical and experimental values of moment resistance for each specimen.

Specimen	Moment Resistance, *MR* (kNm)		Ratio of Theoretical vs. Experimental values
	Theoretical values	Experimental values	
C-5	212.0	220.0	1.04
C-6	305.0	330.0	1.08
C-7	337.0	430.0	1.23
C-8	466.0	470.0	1.01

The results of the moment resistance of composite connection were also compared with the moment resistance of partial strength connection of extended end-plate connection. The moment resistance of the extended end-plate connection was established by applying the component method proposed by Steel Construction Institute. The comparison of the results was shown in Table 3. From the results, the use of proposed composite connection for TWP section has showed an increment up to 72.4% of the moment resistance of the connection.

Table 3: Comparison of moment capacities for composite and non-composite connections

Specimen	M (kNm)		% Difference
	Composite	*Non-composite* (extended end-plate)	
C-5	212.0	123.0	72.4
C-6	305.0	181.0	67.0
C-7	337.0	235.0	43.4
C-8	466.0	432.0	7.9

CONCLUSIONS

This study concluded that it is possible to determine the moment resistance of composite flush end plate connections linked to a column flange by adopting the method proposed by SCI, even for different geometric parameters such as TWP 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, size of column and the size and number of reinforcement. 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 contributed to an increase in the moment capacity up to 23% which is quite significant.
- The use of reinforcement in composite connection increased the moment resistance of the connection up to 73% compare with the extended end-plate connection for non-composite connection.
- The ductility of the connection in composite connection is suitable for the design in semi-continuous construction as the ductility of the connection is more than 20mrad as proposed by SCI.

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