beam to Column partial Strength Composite Connections using Trapezoid eb rofiled (T) Sections

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Abstract

In constructions, especially steel, there are commons that the connections are usually designed as pinned which associated with simple construction or rigid. The connections related with continuous construction. However, the actual behavior failures of the structures are between these two extreme cases. The use of partial strength or semi-rigid connections has been encouraged by Eurocode 3 and the studies have proven the significant savings in construction steel weight. The theoretical calculations and experimental work has to do in order to prove the each other for the verification. The objective of this paper is to compare and see the relationships between calculations and experimental works for partial strength composite connections using flush end-plate connection with Trapezoid eb Profile (T P) as a beam and Universal Column (UC). The connection comparison tables were presented base on all possible failure modes that occur in the beam to column connection on major axis. A method proposed by Steel Construction Institute (SCI) which take into account the requirements in Eurocode 3, Eurocode 4, and BS 9 0 0 were adopted to predict the moment capacity and shear capacity in developing the tables. The theoretical calculations of moment capacity presented in this paper were validated by six full scale testing that have been carried out at Steel Technology Centre, Universiti Teknologi Malaysia. The tests showed good agreement between experimental and theoretical values.

Keywords— Theoretical and Experimental, Partial strength, Composite Connections, Moment Capacity, TWP section, flush end-plate connection, Universal Column (UC) section.

II. Methodology

Trapezoid web profile is a corrugated web with the angle of 45 degree as shown in Fig. 1 and welded to the two flanges by automated machine. The web and the flanges composed of different steel grade. The steel grade of the flange is designed for S-355 to increase the flexural capacity and the steel grade of the web is designed for S-275 to reduce the cost of steel material. The capacity of shear is usually not that critical in the design of the beam. Typical flush and plate connection types can be seen in Fig. 2.

1. I TROD CTIO

Composite beam in buildings has known to increase the loading capacity and stiffness of the composite construction. The benefit of composite beam action result will give significant savings in steel weight and reduce the depth of the beam. The composite action took place due to the interaction of steel beam and concrete slab with shear connectors increases the load-carrying capacity and stiffness of composite beam.

Traditionally, steel frames are designed either as pin or rigid. In pin joint, the beam connection are assumed as
2.1 Analysis of Trapezoid Web Profiled (TWP) Composite and Connections

2.1.1. Concept of TWP Composite

Trapezoid Web Profile Composite beam sections can be performed in composite with the studs connected to the top flange of the girder by using automated welding machines as seen in Fig. 3.

![Figure 3. Installation of Studs, metal decking, and composite](image)

The size of the studs varies from 10 mm to 30 mm thick and 90 mm to 140 mm long depending on the need of shear between TWP section and concrete slab. The studs are installed to resist longitudinal shear forces. The size of the flange can vary from 10 mm to 60 mm thick with its width ranging from 100 mm to 500 mm used to determine the moment capacity of TWP beam. The depth of TWP beam varies from 200 mm to 1600 mm.

2.1.2. Benefit of TWP Composite

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. The advantages will be more significant when there is a need for a column free, long span in structural systems, such as portal frames for warehouses, girders for bridges, floor and roof beam for high-rise buildings, and portal frame for factory.

2.2 Aspect of partial strength connections

In the design of braced multi-storey steel frames, the steel weight of the connections may account for less than 3% of the frame weight; however, the cost of the fabrication is in the range of 30% to 50% of the total. Fig 2 shows the typical TWP section with studs.

The benefits of overall cost saving of the partial strength connections are more than simple connections. It is reported that partial strength connection saves the steel weight when used in non-composite 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.

2.3 Composite connection designs

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 tensile of top bolt row and the tensile capacity of the reinforcement anchored inside the concrete slab as shown in Fig. 4. The composite connection capacity was checked to meet the requirement of BS 5950:2000. Moment resistance of the connection was developed from tension force of the reinforcements and the upper bolts with the compression of the bottom flange.

2.3.1 Tension Zone

2.3.1.1 Reinforcement bar

The distance of the reinforcement to the compression flange of the beam will be used to determine the moment capacity of the connection. The tests and models have shown that connection rotation capacity decreases as the area of reinforcement increases. The limitation of reinforcement area and elongation up to 10% as suggested by SCI is needed to ensure that the connection can undergo sufficient rotation to strain the reinforcement to yield and to determine moment capacity. The contribution of wire mesh should be ignored.

2.4 Tension Bolt

In composite connection, the tension bolt is underneath the upper beam flange. The moment capacity of the connection is developed by the contribution of tension reinforcement and the contribution of upper bolt row to avoid premature failure due to non-ductile.
2.5 Longitudinal shear forces.

The tensile force in the reinforcement depends on the longitudinal shear force transferred from the TWP beam to the slab as shown in Fig. 5. According to BS5950, 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 until the compression region of the slab to satisfy the requirements of BS 8110.

2.5.1 Compression Zone.

The compression capacity of the connection is located on the flange and web of the lower beam and on the resistance of the column web. The failure modes on compression zones are due to crushing or buckling of the column web or on the beam flange or web.

2.5.1.1 Flange and web of the lower beams.

The compression resistance of the beam depends on the direct response from the lower beam flange. SCI has proposed that the applied compression force should not exceed 1.4 of design strength $p_y$ to limit the compression resistance of the lower beam flange and web of the beam. In this study, the applied compression force was proposed not to exceed $1.2 p_y$ as the TWP is a built-up section.

III. RESULTS

A series of tests at the University of Albertay, Dundee was successfully carried out to verify the predicted moment. However, these tests were carried out for hot-rolled steel section non-composite connection. In this study, four TWP composite connections are presented. The minimum thickness for corrugated web is 3mm for shallow beam and the maximum thickness is 6mm for deeper beam. The ratio of beam depth versus web thickness is kept not to exceed the limit for compact section as described by BS5950:2000.

3.1 Test procedures.

Test specimens were set-up by connecting two 3 m length columns and a 6 m long beam as shown in Fig. 6 and Fig. 7. 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 using pairs of shear studs with diameter 19 mm and height 95 mm. The thickness of the slab was taken as 130 mm with concrete strength grade 30.

Four reinforcement bars with the diameter of 12 mm or 16 mm was installed around the column and embedded to the slab as shown in Fig. 4 and Fig. 7. The top and bottom part of the column was restrained from any movement. Two point loads was applied at 1.200 mm from the centre of column using an automatic hydraulic jack and monitored with a load cell 100 tons.
### Table 1

**TEST SPECIMENTS WITH VARIOUS PARAMETERS.**

<table>
<thead>
<tr>
<th>Test</th>
<th>Bolt Rows</th>
<th>Bolts</th>
<th>End Plates</th>
<th>Size of TWP</th>
<th>UC</th>
</tr>
</thead>
<tbody>
<tr>
<td>C5</td>
<td>M20 R5</td>
<td>200</td>
<td>640</td>
<td>12</td>
<td>400x40x60</td>
</tr>
<tr>
<td>C6</td>
<td>M24 R8</td>
<td>250</td>
<td>540</td>
<td>15</td>
<td>500x100x100</td>
</tr>
<tr>
<td>C7</td>
<td>M24 R8</td>
<td>250</td>
<td>640</td>
<td>12</td>
<td>450x100x100</td>
</tr>
<tr>
<td>C8</td>
<td>M24 R8</td>
<td>250</td>
<td>640</td>
<td>15</td>
<td>600x200x150</td>
</tr>
</tbody>
</table>

### Figure 8

*Forces diagram for elastic and plastic Analysis*

### Table 3

**COMPARISON OF MOMENT CAPACITIES FOR COMPOSITE AND NON-COMPOSITE CONNECTION.**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>M&lt;sub&gt;x&lt;/sub&gt; (kNm)</th>
<th>Composite</th>
<th>Non-composite (EEP)</th>
<th>% Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>C5</td>
<td>221.0</td>
<td>218.0</td>
<td>7.24</td>
<td></td>
</tr>
<tr>
<td>C6</td>
<td>305.0</td>
<td>181.0</td>
<td>67.0</td>
<td></td>
</tr>
<tr>
<td>C7</td>
<td>337.0</td>
<td>235.0</td>
<td>43.4</td>
<td></td>
</tr>
<tr>
<td>C8</td>
<td>465.0</td>
<td>432.0</td>
<td>7.9</td>
<td></td>
</tr>
</tbody>
</table>

The graph shows that the connections behaved linearly in the first stage followed by non-linear behavior 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 ranging between 1.01 to 1.23 as shown in Table 2.

### Table 2

**THEORETICAL AND EXPERIMENTAL VALUES OF MOMENT RESISTANCE FOR EACH SPECIMEN.**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Moment Resistance (kNm)</th>
<th>Ratio of Theoretical vs. Experimental Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>C5</td>
<td>221.0</td>
<td>1.04</td>
</tr>
<tr>
<td>C6</td>
<td>305.0</td>
<td>1.06</td>
</tr>
<tr>
<td>C7</td>
<td>337.0</td>
<td>1.13</td>
</tr>
<tr>
<td>C8</td>
<td>465.0</td>
<td>1.01</td>
</tr>
</tbody>
</table>

### 3.2 Prediction of Moment Resistance (MR)

The prediction of moment resistance and the lost of stiffness is very much dependent on the stiffness of the connected members, types of joints, and orientation. The deformations are needed to be accounted because of substantial contribution to the frame displacements and affect internal forces distribution as described in Fig. 8. The M-<sup>Φ</sup> curve from the experiment results are shown in Fig. 9. The experimental values of moment resistance MR are listed in Table 2. Typical failure mode of TWP composite with partial strength connection and flush-end plate connection is shown in Fig. 10. The moment resistance of the EEP connection was established by applying the method proposed by Steel Construction Institute. The comparison was shown in Table 3. The use of composite connection has shown an increment up to 72.4% moment resistance of the connection.

### DISCUSSION

The tables as shown in Table 2 to 3 present connections and the capacities of the connections theoretically and experimentally. The ratios between both works are almost closed. The smallest ratio is shown in C-08. The largest ratio is shown in C-07. The connection with beam size larger than this will have smaller angle of rotation which result in connection categorized as non-ductile. Although TWP section can be produced for up to 1600 mm deep, the limited suggested size for partial strength connection is up to 650 mm deep. This is to maintain the ductility of the connection and crucial to meet the design requirement by EC3 for semi-continuous
construction. The shear capacity of the connection is based on shear capacity of the tension bolt row and lower bolt rows. The increase of moment resistance depends on the size of bolt, the number of bolt, the size of end-plate, and the thickness of end-plate.

![Figure 9. Moment Rotation Curve Test Results C-1 to C-12](image)

**Figure 9.** Moment Rotation Curve Test Results C-1 to C-12

![Figure 10. Typical failure mode of composite TWP connection](image)

**Figure 10.** Typical failure mode of composite TWP connection

4.1 Effect of increasing bolt rows from one to two rows.

The Experimental works have shown the moment capacity of the connection for single bolt and double bolts. The moment capacity is increased for one and two bolt rows size M20 size M24. The increasing number of bolt row from one to two increased moment capacity average of 50% for M20 and average of 59% for M24. The moment capacity of the connection is influenced by the depth of beam, number of bolt, and size of bolt.

4.2 Effect of increasing the bolt size from M20 with 12 mm thick end-plate to M24 with 15 mm thick end-plate.

The results of percentage increase in moment capacity for M20 with 12 mm thick end-plate and M24 with 15 mm thick end-plate showed that by increasing the size of bolt from M20 with 12 mm thick end-plate to M24 with 15 mm thick end-plate, the moment capacity of the connection is increased by an average about 48% for one bolt row and 55% for two bolt rows. The results show that the moment capacity of the connection depends on the strength of the bolt.

4.3 Effect of increasing the number of shear bolt from one bolt row to two bolt rows.

The vertical shear capacity of connection in Table 5 is increased from 258 kN without optional shear bolt row to 442 kN with shear row. The increment of the vertical shear capacity is not exactly double as the determination of the shear capacity is also depends on the number of row of the tension bolt. The vertical shear capacity of the connection is 515 kN with optional shear bolt row.

**CONCLUSION**

The moment capacity of flush end-plate connections connected to a column flanges for TWP composite can be determined by adopting the method proposed by SCI. The capacities of the connection depends 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. The increase in the number of bolt row from one row to two rows has contributed to an increase in the moment capacity in the range of 50% to 59% which is quite significant. The increase in 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 in the range of 48% to 55% which is about the same as the effect of increasing the number of bolt from one to two bolt rows. The shear capacity of the connection contributed the shear capacity of the connection by an increment of 71% with the addition of optional shear bolt row. The proposed tables can be used in the design of semi-continuous construction in multi-story steel frames.

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REFERENCES


