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# Experimental Tests on Composite and Non-composite Connections Using Trapezoid Web Profiled Steel Sections

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**Abstract**

The use of partial strength or semi-rigid connections has been encouraged by Eurocode 3 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 performance of the strength, the rotational stiffness, and the ductility of the composite connections and non-composite connection using trapezoidal web profiled steel (TWP) sections. Eight full scales testing of beam-to-column connections comprised of four specimens for composite and four for non-composite connection with different geometrical configurations have been carried out. The tests results showed good agreement between the experimental and the predicted values. The test also concluded that composite connections have higher moment resistance, higher stiffness, and less ductile compared with the non-composite connections. The size of TWP steel section should be limited to 500mm deep for both composite and non-composite connections.

**Keywords:** Composite connections, moment rotation curves, partial strength, semi-continuous construction, trapezoid web profiled steel section

## 1. Composite Construction

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 has known to increase the load-carrying capacity and stiffness of composite beam. These advantages of composite beam have contributed to the dominance of composite beam in the commercial building in steel construction industry. To enhance further the advantages of composite construction, this paper has further extended the research work on composite connection. The proposed composite connections are expected to enhance further the moment resistance and the stiffness of the non-composite connections. However, the moment resistance and the stiffness of the connection can only be understand by carry out full scale testing of beam-to-column connections for both composite and non-composite connections. Four composite specimens and four non-composite specimens were fabricated and tested. The results of moment resistance, initial stiffness, and the ductility of the connections were compared and discussed in details between composite and non-composite specimens in this paper.

The termed partial strength connection is usually associated with a connection having a moment resistance less than the moment resistance of the connected beam. However, in composite connection, the moment resistance of the connections needs to be established and compared with the moment resistance of the connected beam ($M_{cx}$). If the moment resistance of the composite connection is less than the $M_{cx}$ value and greater than 25% of $M_{cx}$ value as suggested by Steel Construction Institute (SCI, 1996) for partial strength connection, than the composite connections can be classified as partial strength connection which also be named as partial strength composite connections. 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 (EC3, 2005) where the strength of reinforcement embedded inside the concrete slab was taken into account to improve the moment resistance and the stiffness of the connection. In semi-continuous frame the degree of continuity between beam and column is

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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 representing 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 (Tahir, 1997). This is possible as the use of partial strength has contributed to the benefits at both the ultimate and serviceability limit states design as reported by Steel Construction Institute (SCI, 1996). However, the comparison on the partial strength composite and non-composite connections for Trapezoidal Web Profiled (TWP) sections has not been established yet. Therefore, this paper intends to establish the comparison based on the strength, rotational stiffness, and the ductility of the connection.

## 2. Trapezoidal Web Profiled Steel Sections

A trapezoid web profiled steel section is a built-up section comprised of two flanges connected together to thin corrugated web usually between 3 mm to 8 mm thick by a fillet weld as shown in Fig. 1. This fillet web is strong enough to hold the web and the flange together. 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 with two different types of steel grade was used to form 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 resistance 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 can varies from 10 mm to 60 mm thick with the width in the range of 100 mm to 500 mm. The depth of the TWP

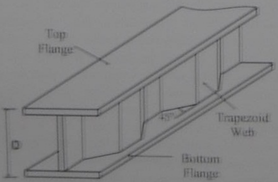



**Figure 1.** TWP Steel Section.

beam can also varies from 200 mm to 1,600 mm. The depth of the beam which can reach 1,600 mm 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 900 mm 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. The selected TWP beam size for specimens tested in this paper have the width of the flange in the range of 140 mm to 160 mm with the thickness in the range of 12 mm to 16 mm thick, the thickness of the web was maintained at 4 mm with the depth of the beam in the range between 400 mm to 600 mm.

### 2.1. Advantages of TWP Sections

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, 2001). The advantages on the use of 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. Increase lateral torsion buckling resistance due to corrugated web.
4. The manufacturing of TWP is fully automated production line which ensure high quality product and reduce the 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 1.60 m depth which is not provided for hot rolled section. These advantages will offer the range of choice for most structural usage especially for long span structures.

However, there are some disadvantages of TWP section. This section is quite complicated to fabricate due to its trapezoid web shape which means that the need to use the state of the art machine. As a result, the initial production of TWP section is quite expensive. A study on the effect of corrugated web steel section in plate girder has been carried out by Sayed-Ahmed (Sayed-Ahmad, 2007). The study was focused on the numerical analysis to investigate the buckling modes of the corrugated steel web of I-steel girders. The numerical model was carried out to determine the effect of moment resistance of I-steel section with corrugated web girders. The study has concluded that the corrugated web has no contribution to the moment resistance of the beam. 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 composite connection. The definition and identification of types of connections are discussed below.

## 3. Design of Composite Connection

The design philosophy presented in this paper was adopted from 'component method' as described by Steel Construction Institute (SCI, 1996). This component method takes into consideration the failure mode of each component that interacts together to the formation of the connection. The failure mode of each component is checked base on the failure zone that divided into three major zones namely tension, shear, and compression zone as shown in Fig. 2. 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. The tension forces are developed from the tension of the reinforcement and the top bolts, whereas the compression force is developed from the bottom flange of

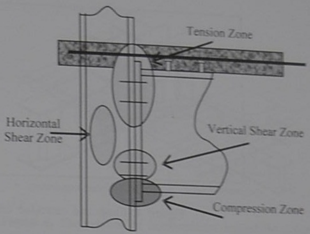



**Figure 2.** Zones of failure modes.

the beam as shown in Fig. 3. The tension forces of the bolts are usually taken as a linear elastic behaviour. However, in 'component method', where the failure of the connection is based on the yield of the end-plate and the connected part, not solely on the tension of the bolts, the distribution of forces was modified to a full plastic failure load as shown in Fig. 3. Details of the calculations using 'component method' for composite connection are presented elsewhere (Anis, 2007; SCI, 1998).

## 4. Design of Non-composite Connections

The design philosophy of the non-composite connection is the same as the composite connection except that the reinforcement bars are replaced by one bolt row. This type of connection is known as extended end-plate connection. The proposed composite connection should not be used together with the extended end-plate connection as the extended bolt on top of the flange developed a tension force which supposed to be carried out by the reinforcement bars in composite connection.

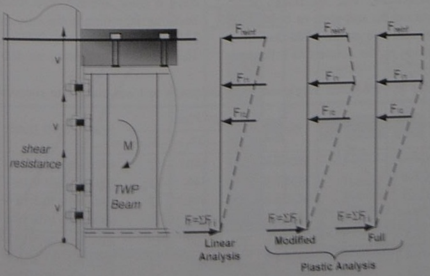
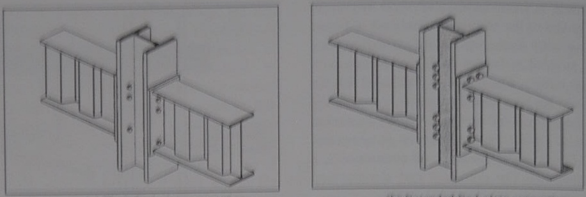



**Figure 3.** Elastic and Plastic Analysis of Bolt Forces Distribution in Composite Connection.



(a) Flush End-plate connection (b) Extended End-plate connection

**Figure 4.** Flush and extended end-plate connected to TWP steel section.

Therefore, the flush end-plate connection is strongly recommended for composite connection. However, for non-composite connection, both flush and extended end-plate connections can be used as shown in Fig. 4. The moment resistance for the extended end-plate is greater than the flush end-plate as the bolts are extended outside the top flange.

## 5. Experimental Tests

The use of partial strength connection for hot-rolled British sections has well established by SCI. A series of tests at the University of Albertay, Dundee was successfully been carried out to verify the predicted moment and shear capacity with the experimental tests capacities (Bose, 1993). The results show good agreement between the numerical values and experimental values. (Abdalla *et al.*, 2007) have carried six full-scale testing on beam-to-columns of extended end-plate connection with stiffened and unstiffened columns. High tensile bolts of M20 grade 8.8 were used together with the 15mm thick end-plate. It was concluded that the maximum difference for the tension bolts between stiffened and unstiffened were in the range of −3.3% to 6.6%. (Coelho *et al.*, 2004) have carried out experimental tests on eight statically loaded end-plate moment connections. The specimens were designed to cause failure on the end-plate or bolts. The study concluded that an increase in end-plate thickness has resulted in an increase to the connection's flexural strength and stiffness. However, these tests were carried out for hot-rolled steel section and the connection is a non-composite connection. Shi, Li, Ye, and Xiao (2007) have tested on composite joints with flush end plate connection under cyclic loading. The composite joints with flush end plate connection show large strength resistance and good ductility, and the slippage between the concrete slab and steel beam is very small, which shows that between the concrete slab and steel beam, the full composition can be obtained by the proper design for the shear connectors. In this paper, the proposed connections are composite and non-composite with TWP sections as a beam. Both types of composite and non-composite connections were tested and compared to understand the capacities and the mode of failure of the connections. The test specimens are listed in Table 1, from which it can be seen that the parameters were varied by different geometrical configuration. For composite connection tests, four number of 16 mm diameter reinforcement bar were used and the thickness of the slab was 125 mm with concrete of grade 30 N/mm$^2$. The size of beam for composite and non-composite was the same but the size of column was not the same. However, the difference on the size of columns has not affected the moment resistance of the connection as both columns have a thick column web which will not fail due to buckling according to the numerical analysis done earlier (Anis, 2007; Sulaiman, 2007). The size of the TWP sections was designated as 400×140×39.7/12/4 where

**Table 1.** Geometrical configuration of composite and non-composite connections

| Type of connection | Specimen No. | Bolt Rows | Bolts | End Plates | | | Size of TWP Beam | Size of Column |
|---|---|---|---|---|---|---|---|---|
| | | | | W | D | T | | |
| Composite connection | CF-5 | 1 | M20 8.8 | 200 | 440 | 12 | 400×140×39.7/12/4 | 305×305×118 |
| | CF-6 | 1 | M24 8.8 | 250 | 540 | 15 | 500×180×61.9/16/4 | 305×305×118 |
| | CF-7 | 2 | M20 8.8 | 200 | 490 | 12 | 450×160×50.2/12/4 | 305×305×118 |
| | CF-8 | 2 | M24 8.8 | 250 | 640 | 15 | 600×200×80.5/16/6 | 305×305×118 |
| Non-composite connection | N-5 | 2 | M20 8.8 | 200 | 500 | 12 | 400×140×39.7/12/4 | 254×254×107 |
| | N-6 | 2 | M24 8.8 | 250 | 600 | 15 | 500×180×61.9/16/4 | 254×254×107 |
| | N-7 | 3 | M20 8.8 | 200 | 550 | 12 | 450×160×50.2/12/4 | 254×254×107 |
| | N-8 | 3 | M24 8.8 | 250 | 700 | 15 | 600×200×80.5/16/6 | 254×254×107 |

**Table 2.** Material properties of TWP steel beam and universal column members

| Sample taken from | Mean value of $P_y$ (N/mm$^2$) |
|---|---|
| TWP beam flange | 414 |
| TWP beam web | 378 |
| Column flange | 405 |
| Column web | 407 |
| End-plate | 372 |

400 was the depth in mm, 140 was the width in mm, 39.7 was the weight in kilogram per metre, 12 was the thickness of flange in mm, and 4 was the thickness of web in mm. Coupon tests have been carried out for the flange and web of TWP beam and the column and also the end-plate. Three coupon tests have been carried for each of the samples and the mean values of yield strength $P_y$ are recorded as shown in Table 2. The results of $P_y$ in Table 2 have shown that the experimental results were higher than the theoretical values of $P_y$. The higher $P_y$ values in the actual tests are the most likely the reason why the experimental values are greater than the theoretical values. All possible components that affect the moment resistance such as size of bolt, size of end-plate, and size of beam were kept constant so that the behaviour of composite and non-composite connection could be compared. Two sizes of high tensile bolts were used namely M20 and M24 so as to understand the effect of changing the size of bolts to the behaviour of the connections.

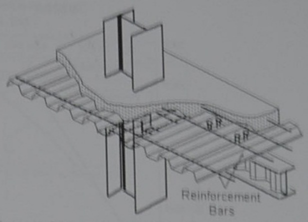



**Figure 6.** The position of reinforcement around the column and embedded to the slab.

### 5.1. Test procedures

Test specimens were set-up by connecting a 3 m long column with a 1.3 m long beam as shown in Fig. 5. A metal decking with 1.3 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 were measured as 19 mm diameter and 95 mm height was used. The thickness of the slab was taken as 125 mm thick with concrete strength of grade C30. Two reinforcement bars of size 16 mm diameter was installed around the column and embedded to the slab as shown in Fig. 6. The reinforcement bars should also be checked for the anchorage length which supposed to be at least 40 times the diameter of the bars (BS 8110, 1997). As the bars were installed along the cantilever beam, the length of anchorage was 1,300 mm which is higher than the required length of 640 mm. Therefore, the anchorage length is enough to prevent any slippage of the bars. The type of test arrangement employed for the beam-to-column connections in this study was the cantilever arrangement of which the bending in the beam was produced by the load applied at the end of the cantilever. No axial load was applied to the column as there is less likely that the

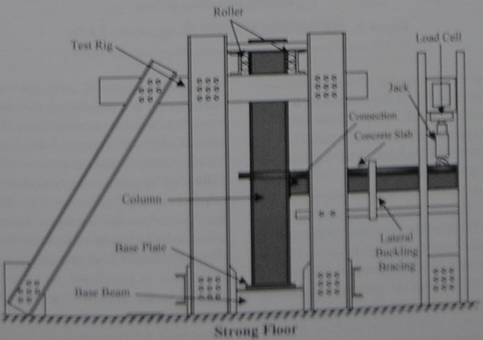



**Figure 5.** Arrangement for tested specimens on the test rig.

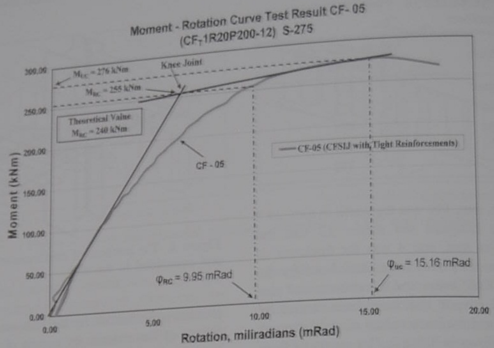



**Figure 7.** Moment-rotation curves for specimens CF-05.

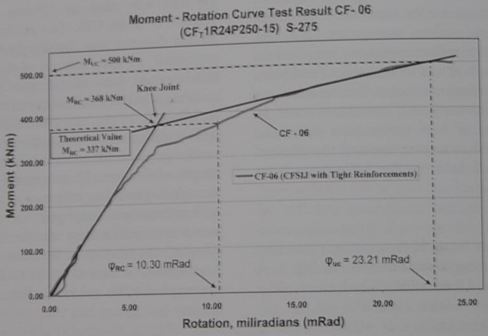



**Figure 8.** Moment-rotation curves for specimens CF-06.

axial load exceed the elastic capacity of the column and influence the moment capacity of the connection. Many researchers (Azizinamini *et al.*, 1987; Nethercot and Zandonini, 1989; Aggarwal, 1994; De Carvalho *et al.*, 1998) in the past have not applied axial load to the column due to the same reason.

The top and bottom part of the column was restrained from any movement. A point load was applied at 1.3 m from the centre of column flange. 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 rotation of the connection between the beam and column, the displacement of the beam, and the load applied. A small load was applied and then removed, to check the stability of the rig. About a third of an expected failure load was then gradually applied, sufficient enough to cause inelastic deformation and to establish the connection in the state of equilibrium before a complete applied load response was carried out. To determine the complete response until failure, each connection was later subjected to the following sequence. An increment of about 5 kN was applied to the specimen. The readings applied load, displacement, and rotation were recorded after two minutes had elapsed so as 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 3 mm. 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 behaviour of the connected column. A graph of

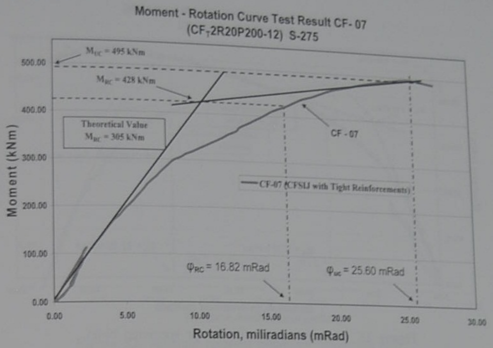



**Figure 9.** Moment-rotation curves for specimens CF-07.

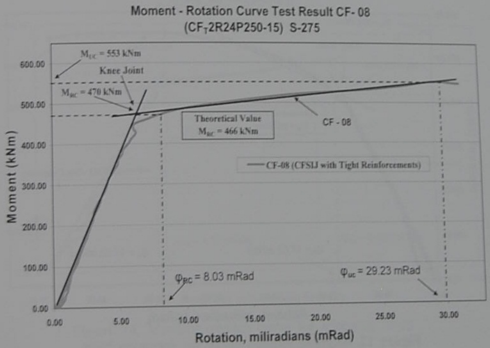



**Figure 10.** Moment-rotation curves for specimens CF-08.

moment versus rotation was plotted to predict the moment resistance of the connection. The value of applied moment was calculated by multiplying the applied load to the lever arm of the cantilever beam which was measured from the position of applied load to the face of the column flange. The rotation of the connection was measured as the difference between the rotation at the centre of the beam and the rotation at the centre of the column recorded using inclinometer.

### 5.2. Test results

The behaviour of the connections is very much 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 beam, the number, size and distance of the bolt and the thickness of the end-plate may significantly affect the moment resistance and the rotation stiffness of the connection. However, the contributions of the profiled metal decking and the wire mesh have been ignored as they failed at lower values of elongation than the reinforcement bars (SCI, 1998). To understand the effect of these geometrical configurations of the connection to the moment resistance, rotational stiffness, and the ductility of the composite and non-composite connection, the results from the M-Φ curve should be presented in a tabulated form and compared.

#### 5.2.1. Moment resistance ($M_R$)

The moment resistance of the connection is very much

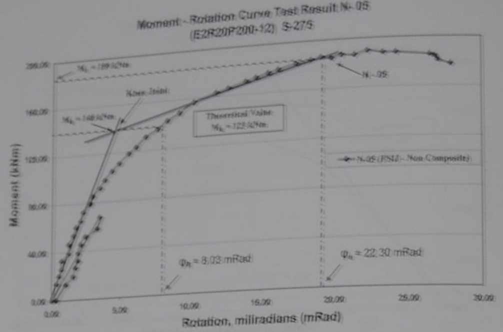



**Figure 11.** Moment rotation curve for specimen N-05.

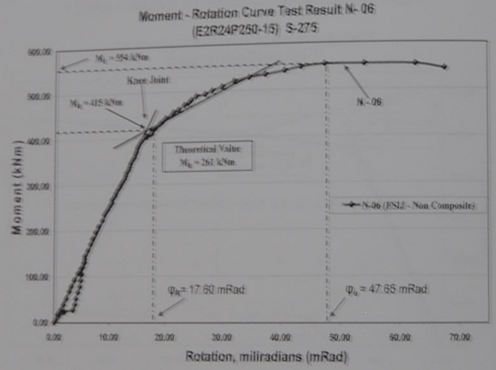



**Figure 12.** Moment rotation curve for specimen N-06.

dependent on the connected members and the types of joints. Beam-to-column connections generally behave as linear followed by non-linear in moment-rotation curves. The structural analysis needs to account for this non-linearity of joint response to predict accurately the moment resistance as the joint behaviour exhibits a form of material non-linearity. The moment resistance versus rotation of the connection, (M-Φ) curves are shown in Fig. 7 to Fig. 10 for composite connection and Fig. 11 to Fig. 14 for non-composite connection. The tests results of moment resistance, $M_R$ listed in Table 3 were determined when a "knee joint" was formed in each of the M-Φ curves plotted in Fig. 7 to Fig. 14. This "knee joint method" technique has been used by many researchers to predict the moment resistance of the connection from the M-Φ curves drawn from the tests results (Tahir, 1997; Azman, 2001; Anis, 2007; Sulaiman, 2007). The formation of "knee joint" which determine the moment resistance of the connection was developed by drawing two straight lines; a straight line drawn from linear region and intersected to another straight line drawn from a non-linear region that formed almost a plateau in the M-Φ curves. By adopting this technique, the test values of moment resistance, $M_R$ for the overall joint for the tests were established from the point of intersection. The overall results showed that the experimental values of moment resistance were greater than the predicted values with the ratio in the range between 1.01 to 1.40 for composite connections and between 1.09 to 1.60 for non-composite connections as shown in Table 3. The theoretical moment resistance of the extended end-plate connection was established by adopting the "component

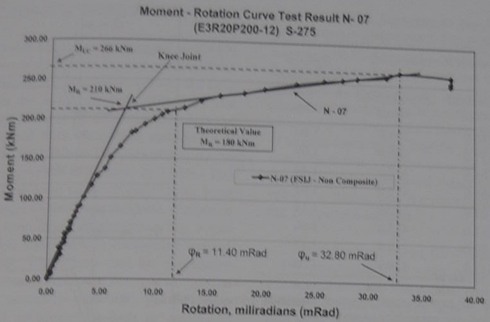



**Figure 13.** Moment rotation curve for specimen N-07.

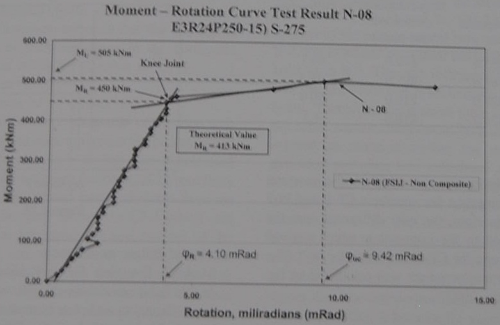



**Figure 14.** Moment rotation curve for specimen N-08

method" proposed by Steel Construction Institute (SCI, 1996, SCI, 1998) as explained earlier.

The results in Table 3 show that the moment resistance of the experimental values have a good agreement with the theoretical values. Theses results showed that the component method suggested by Steel Construction Institute is not limited to the application of hot-rolled section only. The component method is suitable to be applied to TWP section. The ratio between experimental and theoretical values do not show a linear increment as he moment resistance of the connection is influenced by he component of the connections such as the size of beam, columns, end-plate and bolts.

To clearly understand the level of moment resistance of he connections, the results of the experimental moment esistance of composite connection were compared with he experimental moment resistance of non-composite partial strength connection of extended end-plate connection as shown in Table 4. From the results, the use of proposed composite connection for TWP section has shown the range of percentage difference from negative 10.1% to positive 50.9% depending on the geometrical configuration of the connections. The comparison of the results is best explained by comparing the composite and non-composite connections with the same geometrical configuration.

5.2.1.1. Comparison of CF-5 and N-5

The component CF-5 was a composite connection with 4T16 reinforcement was used to replace the two extended bolt in extended end-plate in N-5. The size of bolt used was M20 grade 8.8 with end-plate thickness of 12 mm thick and 200 mm width. The position of the reinforcement was the same position as the extended bolt in the extended end-plate in N-5. This was to ensure that the

**Table 3.** Theoretical and experimental values of moment resistance for composite and non-composite connections

| Specimen | Moment Resistance, $M_R$ (kNm) | | Ratio of Experimental vs. Theoretical values |
|---|---|---|---|
| | Theoretical values | Experimental values | |
| CF-5 | 240.0 | 255.0 | 1.06 |
| CF-6 | 337.0 | 368.0 | 1.09 |
| CF-7 | 305.0 | 428.0 | 1.40 |
| CF-8 | 466.0 | 470.0 | 1.01 |
| N-5 | 123.0 | 140.0 | 1.14 |
| N-6 | 261.0 | 415.0 | 1.60 |
| N-7 | 180.0 | 210.0 | 1.17 |
| N-8 | 413.0 | 450.0 | 1.09 |

**Table 4.** Comparison of experimental moment resistance between composite and non-composite connections

| Specimen | $M_R$ (kNm) | | % Difference |
|---|---|---|---|
| | Composite | Non-composite (extended end-plate) | |
| CF-5 vs N-5 | 255.0 | 140.0 | 45.1 |
| CF-6 vs N-6 | 368.0 | 405.0 | -10.1 |
| CF-7 vs N-7 | 428.0 | 210.0 | 50.9 |
| CF-8 vs N-8 | 470.0 | 450.0 | 4.3 |

same distance of lever arm to calculate the moment resistance of the connection for specimen CF-5 and N-5 was maintained. Therefore, the only difference was the tension force applied to the connection which derived from the reinforcement for CF-5 and bolts for N-5. The tension force in the reinforcement was calculated in Eq. (1) (SCI, 1998; Anis, 2007).

$$P_{reinf} = \frac{f_y A_{reinf}}{\gamma_m} \quad (1)$$

where

$f_y$ is the design yield strength of reinforcement

$A_{reinf}$ is the area of reinforcement within the effective width of the slab.

$\gamma_m$ is the partial safety factor for reinforcement taken as 1.05.

In CF-5, the $P_{reinf} = \frac{f_y A_{reinf}}{\gamma_m} = \frac{460 \times 402 \times 10^{-3}}{1.05} = 193.2$ kN

On the other hand, the tension force developed in N-5 from the two M20 grade 8.8 bolts was calculated based on the mode of failure as described in component method (SCI, 1998). For column flange or end-plate bending, the approach taken in 'component method' as suggested by Steel Construction Institute is representing the yield line patterns that occur around the bolts by using equivalent T-stubs. This approach results in checking against three

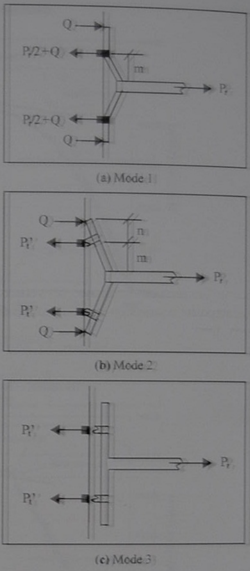



**Figure 15.**

modes of failure as follows:

Mode 1: Complete flange yielding

In this failure mode, the strength of the flange (beam or column flange) or end-plate is weaker than to the strength of the bolts. Upon failure, the flange or end-plate will yield but the bolts are still intact as shown in Fig. 15(a). As a result, a ductile failure can be achieved. This type of failure is the most preferred failure mode in the semi-continuous construction as suggested by SCI (SCI,1996).

Mode 2: Bolt failure with flange yielding

In this failure mode, the strengths of the flange or the end-plate and the bolts are about the same. As a result, both the flange or the end-plate and the bolts will yield together upon failure. This mode of failure is shown in Fig. 15(b). This type of failure can be used in the design of semi-continuous construction provided that the moment resistance of the connection can be quantified and the connection can be classified as ductile connection.

Mode 3: Bolt failure

In this failure mode, the strength of the bolts is weaker than the strength of the flange. Upon failure, the bolts will yield (or even break) but the flange is still intact. Shown in Fig. 15(c) is the failure of Mode 3. This type of failure is not suitable for semi-continuous connection and should be avoided as the connections possess an abrupt type of failure.

Therefore the $P_t$ value for N-5 was calculated as 124 kN with mode 1 failure which indicate the deformation of the end-plate of the connection. As the lever arm for both CF-5 and N-5 was the same, the only difference that affects the moment resistance of the connection was the tension force calculated from component method (SCI, 1998, Anis, 2007) $P_t$ for CF-5 as 193.2kN and $P_t$ for N-5 as 124 kN which give the difference of 55.8%. The contribution of tension force of the bolts, Fr1 for bolts beneath the top flange of the beam for both composite and non-composite connections as mention in Fig. 3 was calculated using yield line theory in component method as suggested by SCI. This means that the percentage difference of the moment resistance between composite and non-composite connection came from the difference of tension force of reinforcement bars in composite connection and tension force of extended bolts in non-composite connection. The result of 55.8% was quite close to the experimental results in Table 4 recorded as 50.9%. The result however, indicates that the increase in the percentage of moment resistance in the composite connection was due to higher $P_t$ value of CF-5.

5.2.1.2. Comparison of CF-6 and N-6

The component CF-6 was a composite connection with 4T16 reinforcement was used to replace the two extended bolt in extended end-plate in N-6. The size of bolt used is M24 grade 8.8 with end-plate thickness of 15 mm thick and 250 mm width. The tension force calculated from the reinforcement was the same force as in CF-5 which was equal to 193.2 kN as the same size and number of the reinforcement bars was used. However, for N-6, the extended bolts are depends on the thickness and the size of the bolts. In N-6 the extended bolts used has been increased from M20 to M24 and the end-plate thickness has been increased from 12 mm thick to 15 mm thick with the width of the end-plate also increases from 200 mm to 250 mm. Therefore, the tension force of the bolt should be increased. By using component method as mention earlier, the result of Pt for the extended end-plate was calculated as 242.0 kN (Sulaiman, 2007). As the lever arm for both CF-6 and N-6 was the same, the only difference that affects the moment resistance of the connection was the tension force with $P_t$ for CF-6 was 193.2kN and $P_t$ for N-6 was 242 kN which give the difference of −20.2%. As compared with the ratio given in Table 4, which was −10.1%, the −20.2% was a little bit higher than the experimental value but the results do indicate that the moment resistance of the composite connection does not always greater than the extended end-plate connection. A study carried out by (Abdallah, 2007) has showed that the theoretical values of tension bolt row beneath the top flange derived from linear distribution method are higher than the experimental values. The linear distribution method has shown that the tension bolts were calculated in the range of 54.0 kN to 73.5 kN whereas in experimental tests the tension bolts were recorded in the range of 39.3 kN to 72.7 kN. From the study, it can be concluded that the linear distribution method has underestimate the experimental values. In component method, the bolt underneath the top flange is assumed to be linear which has resulted to the assumption of tension force lesser than the experimental value. As a result, there was a difference in the tension force between theoretical and experimental values where the experimental value was higher than the theoretical value derived from the component method.

5.2.1.3. Comparison of CF-7 and N-7

The component CF-7 was a composite connection with 4T16 reinforcement was used to replace the two extended bolt in extended end-plate in N-7. The size of bolt used was M20 grade 8.8 with end-plate thickness of 12 mm thick and 200 mm width. The tension force calculated from the reinforcement was the same tension force as in CF-7 which was equal to 193.2 kN as the size and number of the reinforcement bars was used. However, for N-7, the extended bolts used the same number and size of bolts as in N-5. The only difference between N-7 and N5 was the extra two bolt rows beneath the top flange of the beam. Therefore, the tension force of the bolt should be the same tension force as in N-5. By using component method as mention earlier, the result of Pt for the extended end-plate was calculated as 124.0 kN. As the lever arm for both C-6 and N-6 was the same, the only difference that affects the moment resistance of the connection was the tension force with $P_t$ for C-7 is 193.2 kN and $P_t$ for N-7 is 124.0 kN which give the difference of 57.1%. As compared with the ratio of moment given in Table 4, which is 50.9%, the 57.1% was a little bit higher than the comparison between C-5 and N-5 which was 50.9%. This is because the specimens N-7 have two bolt rows beneath the top flange that were in tension. As a result, the tension force applied to the extended bolt row was distributed to these two bolt rows. This result indicates that the comparison of moment resistance between composite and non-composite connection did not only depends on the comparison between extended bolt rows and the reinforcement of the bars but also the number of bolt row underneath the beam flange. As the tension force of the reinforcement bars in composite connection was not the same as the tension force of the bolts in non-composite connection, the distribution of force to the bolts underneath the top flange was not the same also as shown by the difference in moment resistance in Table 4 for TWP sections. The assumption of the yield line theory in component method to calculate the force beneath the top flange of the beam which gives equal value of $F_{r1}$ for both composite and non-composite connections in hot-rolled section (SCI, 1996; SCI 1998) could give a conservative result.

5.2.1.4. Comparison of CF-8 and N-8

The component for specimen CF-8 was a composite connection with 4T16 reinforcement was used to replace the two extended bolt in N-8 specimen. The size of bolt used was M24 grade 8.8 with end-plate thickness of 15 mm thick and 250 mm width. The different between specimens CF-6 and CF-8 were the extra bolt rows below the top flange of CF-8 which has two bolt rows instead of one in CF-6. The tension force calculated from the reinforcement was the same force as in CF-5 which was equal to 193.2 kN as the size and number of the reinforcement bars were the same. However, for N-8, the extended bolts were increased from M20 to M24 but the extra two bolt rows below the top flange has resulted to the distribution of tension force from the extended bolt to the two bolts. This will reduce the deformation of the end-plate of the connection at the extended bolt which results to an increase in the moment resistance of the connection for specimen N-8. By using component method as mention earlier, the result of Pt for the extended end-plate was calculated as 242.0 kN (Sulaiman, 2007). As the lever arm for both CF-8 and N-8 was the same, the only different that affect the moment resistance of the connection was the tension force with $P_t$ for CF-8 was 193.2 kN and $P_t$ for N-8 was 242 kN which give the difference of −20.2%. As compared with the ratio given in Table 4, which is 4.3%, the −20.2% is a little bit higher than the experimental value but the results do indicate that the moment resistance of the composite connection increases as the depth of the beam increases. The possible explanation of this difference could be related to the depth and stiffness of the connection. As a result, there was a difference in the tension force between theoretical and experimental values where the experimental value was higher than the theoretical value derived from the component method as explained earlier in the study carried out by Abdallah (2007).

This result also indicated that the depth of the beam has influenced the moment resistance of the connection. As the lever arm of the connection increases due to deeper beam, the extended end-plate tends to deform which was not happened in composite connection. As a result the composite connection showed a higher moment resistance than the non-composite connection. The stiffer bolts M24 used in N-8 also contributed to the redistribution of tension bolt force to the two bolt rows beneath the top flange of the beam. The assumption of the yield line theory in component method to calculate the force beneath the top flange of the beam which gives equal value of $F_{r1}$ for both composite and non-composite connections in hot-rolled section (SCI, 1996; SCI, 1998) could give a conservative result.

**5.2.2. Rotation stiffness and ductility**

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 non-linear deformation of the connections' components or those of members of the frame in the immediate vicinity of the joint which are beam and column. 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 account the joint response to predict accurately both rotational stiffness and ductility of the joint behaviour. The stiffness of the connections is presented as an initial stiffness by drawing straight line along the linear region in the M-Φ curve. The value of initial stiffness was calculated as the moment resistance divided by the rotation of the connection at that particular moment in the linear region. The results of the initial stiffness derived from the M-Φ curves are tabulated in Table 5. The comparison of composite and non-composite connections for initial stiffness is shown in Table 6. From Table 6, the results showed that the most of the composite connections are stiffer than the non-composite connections except for CF-8 and N-8. The result showed that CF-8 is less stiff than N-8. The use of M24 bolts with 15mm thick end-plate in N-8 has resulted to stiffer connection as the tension force for the extended end-plate in N-8 was greater than the tension force developed from the reinforcement bar. Moreover, the redistribution of tension force to the bolt beneath the top beam flange has contributed to stiffer connection in N-8. The combination of higher tension force of extended bolt in N-8 than the tension force in CF-8 with the use of deep beam

**Table 5.** Test result based on initial stiffness

| Specimen | Size of TWP Beam | Moment Resistance $M_R$, (kNm) | Rotation (mrad) | Initial Stiffness, $S_{j,ini} = M_R/\Phi$ (kNm/mrad) |
|---|---|---|---|---|
| CF-5 | 400×140×39.7/12/4 | 255.0 | 9.95 | 25.63 |
| CF-6 | 500×180×61.9/16/4 | 368.0 | 10.30 | 35.73 |
| CF-7 | 450×160×50.2/12/4 | 428.0 | 16.82 | 25.45 |
| CF-8 | 600×200×80.5/16/6 | 470.0 | 8.03 | 58.53 |
| N-5 | 400×140×39.7/12/4 | 140.0 | 8.03 | 17.44 |
| N-6 | 500×180×61.9/16/4 | 415.0 | 17.60 | 23.58 |
| N-7 | 450×160×50.2/12/4 | 210.0 | 11.40 | 18.42 |
| N-8 | 600×200×80.5/16/6 | 450.0 | 4.10 | 109.76 |

**Table 6.** Comparison of experimental initial stiffness between composite and non-composite connections

| Specimens | Composite (kNm/mrad) | Non-composite (extended end-plate) (kNm/mrad) | % difference |
|---|---|---|---|
| CF-5 vs N-5 | 25.63 | 17.44 | 31.95 |
| CF-6 vs N-6 | 35.73 | 23.58 | 34.01 |
| CF-7 vs N-7 | 25.45 | 18.42 | 27.62 |
| CF-8 vs N-8 | 58.53 | 109.76 | -87.52 |

**Table 7.** Rotation of connections at maximum moment for tested connections

| Specimens | Experimental moment resistance, $M_R$ (kNm) | Rotation of connection at maximum moment (mrad) |
|---|---|---|
| CF-5 | 255.0 | 15.16 |
| CF-6 | 368.0 | 23.21 |
| CF-7 | 428.0 | 25.60 |
| CF-8 | 470.0 | 29.23 |
| N-5 | 140.0 | 22.30 |
| N-6 | 415.0 | 47.05 |
| N-7 | 210.0 | 32.80 |
| N-8 | 450.0 | 9.42 |

(600 mm) has resulted to a stiffer connection for N-8 specimen. This result shows that the use of deeper beam has resulted to a stiffer connection as the lever arm of the connection that measured from tension to compression zone is longer which resulted to less rotation.

The ductility of the connection is measured as the ability of the connection to form a plastic hinge which can be recognized from the M-Φ curve by the formation of non-linear region without any abrupt failure. The ability of the connection to rotate to form a ductile connection is an important criterion to satisfy the requirement in the design of semi-continuous construction (Couchman, 1997). In semi-continuous construction, the connection should fail as a ductile connection and possess a rotation that allows the deformation of the connection instead of the connected members. The connections are considered as ductile connection if the rotation of the connection can achieve at least 20 mrad to form a plastic hinge without any sudden failure as suggested by Steel Construction Institute (SCI, 1996). Table 7 shows the rotation of the tested connections at maximum moment. The results however showed that not all specimens possess the rotation of the connections with at least equal to 20 mrad. For C-5 specimen, the rotation was recorded as 15.16 mrad which is quite close to the suggested value of 20 mrad by SCI. The connection however, did not failure abruptly and the connection still deformed and behaved as ductile type of failure. For N-8 specimen, the rotation was recorded as 9.42 mrad which is lower than the suggested value of 20 mrad. This is due to the use of deep beam (600 mm) to limit the rotation of the connection. The combination of the deep beam and the use of M24 bolts with 15 mm thick end-plate have contributed to lesser rotation. SCI has suggested that for hot-rolled section with M24 bolts in conjunction with 15 mm thick end-plate, the size of beam should be limited up to 533 mm deep (SCI, 1996). It looks like the same limitation should be applied to TWP as the rotation for 600 mm deep beam has shown less ductile connection. The overall comparison of rotation between composite and non-composite connection have shown that the non-composite connections were more ductile than the composite connection. This is because the failure mode of the composite connection is limited due to the restrained of the connected slab. On the other hand, in non-composite connections, the end-plate is free to deform.

### 5.2.3. Mode of failures

The mode of failure of the connections is focused on three zones, namely the tension, shear, and compression zone. These failure zones have taken into account the failure of all components of the joint which includes the section of beam and column. However, the mode of failures in the tested connections is focused on the tension zone only. The compression zone did not show any sign of failure as the size of column used was a heavy section and the flange of the bottom beam was strong enough to prevent from crushing due to the applied load. In shear zone, no sign of failure could be detected as the thickness of the column was very thick. The shearing of the bolt was not possible as the maximum applied load (426 kN) was lesser than the minimum shear capacity (698 kN) of the tested specimens (Anis, 2007; Sulaiman, 2007). For the fillet weld used to connect the beam to the end-plate, the welded was strong enough to carry out the tension force. A fillet weld of size 10mm thick was used to weld the flange of the beam to the end-plate and 8mm thick was used to weld the web of the beam to the end-plate. No sign of failure occurs on the weld. Other components of connections that are most likely to fail are the end-plate and the bolts which fail due the tension force which explained separately as non-composite and composite connections.

#### 5.2.3.1. Non-composite connections

For the extended end plate in non-composite 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 end plate, the deformation of the

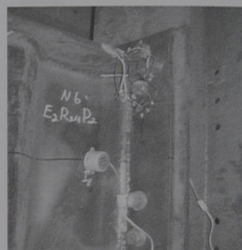
**Figure 16.** 'Y-shape' deformation of extended end plate.

connection translated the end-plate away from the face of the column in a 'Y-shape' form. Again, this deformation corresponded to the Mode 1 failure as in Fig. 16 or 'yielding of the end plate'; and appeared to be symmetrical on both sides of the bolts when looking from the plan view of the connection. The extended end plate connection specimens that experienced this type of failure were the specimens N5 and N7. Fig. 16 shows the deformation of the extended end plate in the form of a 'Y-shape' deformation of specimen N5 at failure which is similar to specimen N7.

As the yielding of the end plate in progress, further increase of the applied load had deformed the end plate even more and started to deform the extended bolts row and the bolt row below the top flange of the beam. The extended end plate connection specimens that experienced this Mode 2 type of failure were the N6 and N8 specimens. The deformation of the end-plate and the elongation of the bolts were then followed by some buckling on the web of the beam as shown in Fig. 17(a) and (b). However, this occurred at load close to maximum load which was not in the region where the moment resistance of the connection was predicted using a knee joint method. For the specimen N8, the elongation of the bolts was more dominant than the specimen N6 which might be due to the use of deep beam. Fig. 17(a) shows the yielding of the end plate and the elongation of the bolts of specimen N6, whilst Fig. 17(b) shows the buckling of the beam web that associated with the deformation.

There was hardly any deformation of the columns throughout the experimental tests for all specimens. This was expected since the columns for all specimens (UC 305×305×118) for the extended end plate connections were designed to adequately sustain the panel shear and the compression action along the bottom flange of the beam. However, bigger beams especially in the extended end plate connection tests tend to exert more compressive force along the bottom flange of the beam towards the face of the column. This was evidenced through the noticeable lines of 'skin tearing' on the web of the column along the bottom flange of the beam. Fig. 18(a) shows the 'skin-tearing' of column web of the extended end plate connection specimen N6. The same effect was also occurred on the column web of the extended end plate connection specimen N8.

(a) yielding of end plate



(b) buckling of beam web

**Figure 17.**

**Figure 18.** 'Skin-tearing' on the column web.

5.2.3.2. Composite connections

The main difference between composite and non-composite connection is the use of reinforcement bars which act compositely with the slab of the concrete. Concrete is known to behave weak in tension. However,

**Figure 19.** Crack on concrete slab.

**Figure 20.** Failure due to cracks on concrete slab.

in composite connection, the reinforcement bars located at the tension zone embedded in concrete slab prevent the pre-mature failure of the concrete. This will reduce significantly the possibility of failure due to concrete cracking in tension. In the cracking trend on the top of the concrete was the most probable failure for the composite connection specimens. Typical allocations of the crack lines of the slab are shown in Fig. 19. The cracking of the concrete slab started at the column corners where the high stress occurs due to the discontinuity and spread out transversely to the left edge and right edge of the slab. The cracks roughly have the same width and the length of cracks was extended towards the end of the slab, with the cracking patterns in all tests being spread up to 400-600 mm length on both sides of the column. These cracks however were barely seen by our own eyes. The cracks were considered modest and occurred only in the top of the composite slabs surroundings the universal column. The visible cracks occurred when the load applied almost reached maximum load. The width of the main crack was about 10-15 mm at the time of failure when the tests were stopped as shown in Fig. 20. The pattern of cracks was very much related to the stiffness of the connection. The stiffer the connection the less will be the appearance of the cracks. This can be seen in the reflection of the inclined pattern of cracks. The less stiff connection as in CF-8 has led to almost straight cracks running transversely across the slab. The connection moments resistance under the loads should be limited to values of 220 kNm and 380 kNm for the specimens CF-05 and CF-06, respectively. Therefore, the use of 'knee joint method' to predict the moment resistance of the partial strength composite connection can be used without any problem with the formation of cracks on the slab as the cracks occurred close to maximum load.

## 6. Conclusions

The conclusions drawn from this study were based on the comparison of the composite and non-composite connection. The use of component method as suggested by SCI can be adopted to calculate the moment resistance of the connections for both composite and non-composite connections for TWP steel sections. The use of 'knee joint method' to predict the moment resistance from M-Φ curves showed good agreement with the component method. The comparison of composite and non-composite connections which is limited to this study can be concluded base on the moment resistance, the initial stiffness, the ductility and the mode of failure of the connections as follows:

1. The results of the moment resistance for composite connection were higher than the non-composite connection for most of the compared specimens. However, the non-composite connection showed a higher moment resistance than the composite connection by about 10% as the use of bolt was changed from M20 to M24 and the end-plate was changed from 12 mm to 15 mm thick. The increase in moment resistance due to the use of M24 in conjunction with 15 mm thick end-plate has reduced as the depth of the beam increases from 500 mm to 600 mm deep. It is suggested that the size of the beam for TWP steel section should be limited to 500 mm deep.
2. The initial stiffness of the connections showed that most of the composite connections were stiffer than the non-composite connections. However, N-8 showed stiffer result than CF8. This is due to the combination of higher tension force of extended bolt in N-8 than the tension force in CF-8 with the use of deep beam (600 mm) has resulted to a stiffer connection for N-8 specimen.
3. The ductility of the connection showed that most of the connections possess more than 20 mrad as suggested by SCI. However, for C-5 the rotation of the connection was recorded at 15.16 mrad which was less than 20 mrad but can still be considered as ductile as the mode of failure was not an abrupt failure. For N-8 specimen, the rotation was too stiff to be considered as ductile connection. It is suggested that the TWP section should avoid the use of beam

which has the depth more than 500 mm.

4. The failure mode for composite connections was the cracks of the top slab which occurred very close to the maximum load. No cracks on the slab at the region where moment resistance of the connection was determined in the M-Φ curves by 'knee joint method'
5. No failures occurred at the compression and shear zones during the tests.
6. The pattern of cracks was very much related to the stiffness of the connection. The stiffer the connection the less will be the appearance of the cracks.

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