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Standardization of composite connections for trapezoid web profiled steel sections

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Abstract. Connections are usually designed either as pinned usually associated with simple construction or rigid normally is associated with continuous construction. However, the actual behaviour falls in between these two extreme cases. The use of partial strength or semi-rigid connections has been encouraged by Euro-code 3 and studies on semi-continuous construction have shown substantial savings in steel weight of the overall construction. Composite connections are proposed in this paper as partial or full strength connections. Standardized connection tables are developed based on checking on all possible failure modes as suggested by "component method" for beam-to-column composite connection on major axis. Four experimental tests were carried out to validate the proposed standardised connection table. The test results showed good agreement between experimental and theoretical values with the ratio in the range between 1.06 to 1.50. All tested specimens of the composite connections showed ductile type of failure with the formation of cracks occurred on concrete slab at maximum load. No failure occurred on the Trapezoidal Web Profiled Steel Section as beam and on the British Section as column.

Keywords: computational mechanics; finite element method (FEM); functionally graded; numerical methods; parametric analysis

1. Introduction

The benefits of composite beam action can be realized by reducing steel weight and depth of the beam. To obtain more economical structural design for bare steel beams, composite beam is designed by taking advantage of incorporating concrete slab strength by using headed studs. The composite action increases the load-carrying capacity and stiffness of composite beam due to interaction of steel beam and concrete slab with shear connectors. These advantages of composite beam have contributed to its dominance in steel construction industry. The advantages of composite construction have been further extended by using composite connection. This composite connection further enhances stiffness of the connection as well as weight saving of the composite beam design (SCI P213 1996). Traditionally, steel frames are designed either as pinned jointed or rigidly jointed.

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The beams are simply supported with pin jointed connections and columns are assumed to sustain axial and nominal moment (moment from the eccentricities of beam's end reactions) only. Though connection is simple but beam size obtained from this approach, results in heavy and deep beam. On the other extreme, rigidly jointed frame results in heavy columns due to end moments transmitted through the connection. Hence, a more complicated fabrication of the connection could not be avoided.

One approach which creates a balance between the two extreme has been introduced. This approach, termed as partial strength connection is usually associated with a connection having a moment resistance less than the connected beam (SCI P207 1996). However, in composite connection the moment resistance of the connection is compared to moment resistance of bare steel beam as suggested by Steel Construction Institute (SCI P207 1996). Partial strength connection is the term used for connection associated with design of semi-continuous construction for multi-storey steel frames by Eurocode 3 (EC3: BS EN 1993-1-1 2005). However, for composite connection, the strength of reinforcement embedded inside the concrete slab is taken into account to improve moment resistance and stiffness of the connection. Composite connection has been applied to a special moment frame where the systems utilized area with low and high seismic loads as reported by (Habashizadeh-Asl *et al*. 2013). The study demonstrated that such connection has resulted to a stable response of up to first cycles (equal to 4.0% drift), where a considerable buckling was found in upper flange of beams. Whether the proposed composite connection is classified as partial strength or full strength will be discussed in this paper.

In semi-continuous frame, the degree of continuity between beam and column is greater than that of simple construction but less than that of continuous construction. The degree of continuity in partial strength connection of beam-to-column can be predicted to produce an economical beam section that represents the section between pin and rigid joints (Chen and Kishi 1989). By adopting this approach, studies conducted on the use of partial strength connection in multi-storey steel frame have proven substantial savings in overall steel weight (Tahir 1995). Couchman's study on semi-continuous composite frame with partial strength connections, concluded that the weight and depth savings in an individual beam could reach up to 25% (Couchman 1997). This is possible as the use of partial strength has contributed to the benefits on both the ultimate and serviceability limit states design. To enhance the study for TWP sections by proposed standardized composite connections, their tables need to be established. This paper proposes standardized composite connections tables for TWP sections by adopting component method as proposed by Steel Construction Institute (SCI) (SCI P213 1996).

2. Trapezoidal web profiled (TWP) steel sections

A trapezoid web profiled steel section (TWP) or plate girder is a built-up section made up of two flanges connected together by a thin corrugated web (Fig. 1) (Osman 2001). Corrugated web is employed to eliminate the use of stiffener along the beam due to load bearing and point load (Luo 1995). However, further checks need to be carried out for heavier load bearing. The web is corrugated at an angle of 45 degrees and welded to the two flanges by automated machine. The web and flanges are comprised of different steel grades depending on design requirements. TWP steel section is also classified as a hybrid steel section as two different types of steel grades were used. The steel grade of flanges is designed for S355 whereas the web is designed for S275. The flanges are purposely designed for S355 so that flexural capacity of the beam can be increased

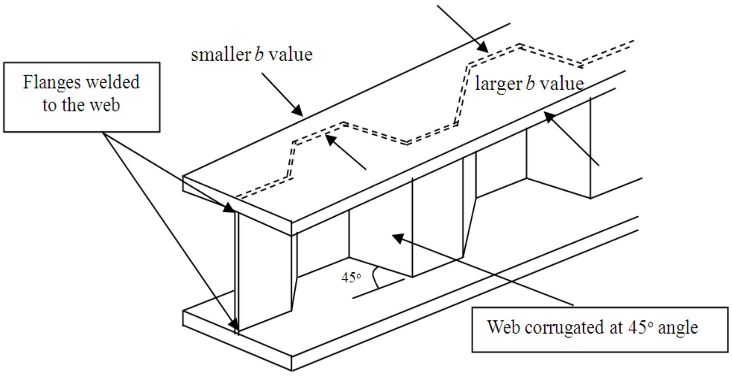


Fig. 1 Typical shape of a TWP steel section and position of "*b*" used to determine the ratio of b/T_f ratio

whereas the web is designed for S275 so as to reduce the cost of steel material. The reduction in TWP cost can be achieved by using state or the art automated machine cut, formed into corrugated shape and welded together to form the TWP section (Osman 2001).

The capacity of shear is usually not that critical in the beam design (Hussein 2001). The use of different steel grades in the fabrication of TWP section leads to further economic contribution in steel frames design along with the use of partial strength connection. The use of thick flanges, thin corrugated web and deeper beam for the TWP section as compared with hot-rolled section of same steel weight have contributed to heavier load capacity and greater beam span (Osman 2001). The difference between TWP section and hot-rolled universal beam (UB) section are the section's geometrical configurations and steel grade used. The TWP section depth can reach up to 1600 mm whereas UB section depth is limited to 914 mm with special order. However, limitation of the beam depth is very much on the floor height.

2.1 Advantages of TWP Beam

The advantages of TWP beam as compared to conventional plate girder or conventional hot rolled steel section are listed as follows:- (Saggaff 2007)

• Utilization of a very thin web (minimum of 3 mm) resulting in light weight steel section.
• Eliminating the need of stiffeners – reduce fabrication cost.
• Use of much slender or deep sections- high flexural capacity, wider span and less deflection.
• Increase in fatigue strength.
• Increase in lateral torsion buckling resistance.

TWP beam can offer substantial saving in the steel usage and in some cases can go up to 40% as compared to conventional rolled sections (Saggaff 2007).

2.2 Structural behaviour of TWP sections

Structural behaviour of the TWP section is presented based on the geometrical configurations which affect performance on moment resistance, shear resistance, and ability of corrugated web to resist bearing. The beam's moment resistance is based on the classification of section. This approach is applicable for depth-to-web ratio not greater than 72ε and the beam is not to susceptible to shear buckling. Otherwise, the beam should be checked for shear buckling use based on section 5 of EN 1993-1-5 (2005). As for the TWP section web which is thin, the determination of moment capacity of the section can be simplified using a "flange only method" as suggested by BS 5950:2000 Part 1 for a built-up section

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 focused on the numerical analysis to investigate buckling modes of the corrugated steel web of I-steel girders. The numerical model was studied to determine the effect of moment resistance on I-steel section with corrugated web girders. The study concluded that corrugated web has no contribution to moment resistance of the beam. (Sayed-Ahmad 2007) reported that shear stress that caused an element of a corrugated web to yield is defined as $\tau_y = \frac{f_y}{\sqrt{3}} \cong 0.577\, f_y$ which is consistent with $0.577p_y$ as suggested by EN 1993-1-1. Sayed Ahmad found that behaviour of corrugated web when subjected to a pure shear stress state, was controlled by shear buckling and steel yielding. The shear buckling failures were associated to local and global buckling failure. The study concluded that web failures are in the form of local, global, and combination of local and global failures. In lateral torsional buckling of I-girders with corrugated web, Sayed (2007) also concluded that the resistance to lateral torsional buckling of such girders was in the range of 12% to 37% higher than that of plate girder with plane web.

In a conventional I-section, the web is located symmetrically along the longitudinal length of flange with a single value of out-stand width, *b* as shown in Fig. 1. The trapezoidal and corrugated shape of the web has resulted in developing larger *b* and smaller *b* values. According to EC 3 (BS EN 1993-1-1 2005), the relevant properties of a compression flange are yield strength f_y, thickness T_f and outstanding width *b*. The question is then whether or not the shape of compression flange element due to corrugated web (in the TWP case) will have some effects on classification of the section. A study done by (Johnson and Cafolla 1997) showed that slenderness should be based on outstanding mean or the maximum; a more conservative value could be obtained if outstanding maximum was used. A recent study carried out by Sayyed (2007) using numerical model concluded that the flange outstanding-to-thickness ratio should be based on the larger out-stands element**.** of the corrugated web girder's flange.

3. Aspects of partial strength connections

In braced multi-storey steel frames design steel weight of the connections may account for less than 5% of the frame weight. However, the cost of fabrication (include cost of planning, designing and detailing) is in the range of 30% to 50% of the total cost of construction (SCI P212 2009). A study about cost reported by (Steel Insight 2012) revealed that fabrication cost in steel construction should include from planning to fabrication cost which was estimated to be in the range of 30% to 40% of the total cost of construction. The increase in fabrication of the connections is due to the difficulty in selecting the type of connection, grades and sizes of fittings, bolt grades and sizes, weld types and sizes, and the geometrical aspects. For composite connection to classify as partial strength connection the behaviour of this connection needs to satisfy the criteria of partial strength connection where moment resistance of the connection should be greater than 25% but less than 100% of connected beam's moment resistance (SCI P212 2009). The geometrical parameters need to be established for the standardized composite connection tables such as quantifying size of

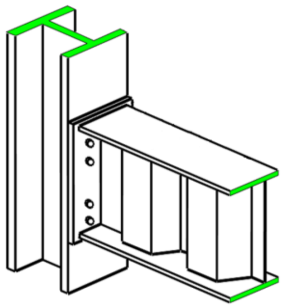


Fig. 2 Typical flush end plate connection for TWP beam section connected to hot-rolled steel column

bolts, size of beam, and size of end-plate. The main advantage of establishing these connection tables is to make it easy for engineers to use the proposed composite connections in their design. Tedious steps to calculate composite connections' moment resistance can be avoided by referring to the standardised tables. Therefore, standardized tables for composite connections should be available with geometrical parameter details to ease the design process. Although the advantages or benefits of composite connections are quite significant, the disadvantages of this approach should also be addressed. The disadvantage in this approach is that it may be more expensive marginally, depending on cost of labour paid; it varies between Europe and Asian countries to fabricate than that of simple connections. In Malaysia, where the cost of labour is relatively low as compared with European countries, use of proposed composite connections will be an added advantage. The benefits of overall cost savings in composite connections have proven to be more than that of simple connections (Tahir 1995, Couchman 1997). It has been reported that savings in using partial strength connection steel weight alone (non-composite) in multi-storey braced steel frames using British hot-rolled section was up to 12% (Tahir 1995). The overall saving cost was up to 10% of the construction cost, which is quite significant according to labour cost in the United Kingdom (Tahir 1995). However, further savings are expected from the use of composite connection, as moment capacity and stiffness of the connection are expected to be higher than non-composite connection. Fig. 2 shows a typical flush end-plate connection of TWP beam section connected to hot-rolled steel column section used in composite connection design.

4. Design of composite connection

The design philosophy presented in this paper is adopted from 'component approach' described in Steel Construction Institute (SCI P207 1997, SCI P213 1998) based on Eurocode 3 (EC 3: BS EN 1993-1-8 2005). The moment resistance of the connection is determined by considering the capacity of each relevant component, tension of the top bolt row and tensile capacity of reinforcement bar anchored to the column and embedded in the concrete slab. Tension forces are developed from reinforcement tension and the top bolts only; whereas compression force is developed from the bottom flange of the beam as shown in Fig. 3. Tension forces of the bolts are usually taken as linear elastic behaviour. However, in 'component method', where failure of connection is based on the yield of end-plate and connected part (not solely on tension of bolts) distribution of forces is modified by the failure loads as shown in Fig. 3.

The design philosophy of the composite connections can be well addressed by understanding

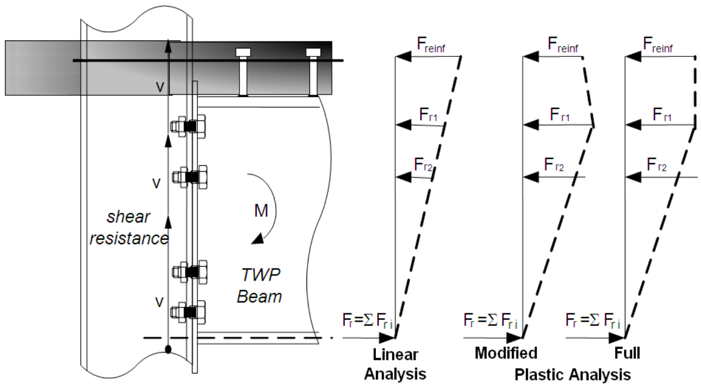


Fig. 3 Plastic analysis of bolt forces distribution in composite connection

the possibilities of failure zones developed in the composite connection. These failure zones include: tension zone, compression zone, and shear zone as explained in details elsewhere (SCI P213, 1998). In this study however, compression zone is the most critical zone as compression stress relied very much on the strength of beam's lower flange. The beam web of the beam should not be included as it is very thin.

4.1 Tension zone

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

4.1.1 Reinforcement bar

Distance between the reinforcement bar and beam's compression flange is considered as a lever arm, to determine the moment resistance of the connection. Tests and models (EC 1996) have shown that connection rotation capacity increases as area of reinforcement increases, which also means that stiffer connection leads to greater rotation capacity. Minimum area for reinforcement bars is needed to ensure that the connection can undergo sufficient rotation to strain the reinforcement yield (SCI P213 1998). The minimum area suggested by SCI is 500 mm^2 for S275 steel. Determination of moment resistance is based on the assumption that reinforcement bar yields (SCI P213 1998). Tension force in reinforcement is calculated as follows: - $P_{re\inf} = \dfrac{f_y A_{re\inf}}{\gamma_m}$

where f_y is the design yield of reinforcement strength, $A_{re\,inf}$ is area of reinforcement within the effective width of slab and γ_m is partial safety factor for reinforcement taken as 1.05.

The value of reinforcement bar's force for four numbers of steel bars of 16mm size in diameter is calculated as $P_{re\inf} = \dfrac{f_y A_{re\inf}}{\gamma_m} = \dfrac{495.7 \times 804 \times 10^{-3}}{1.05} = 379.6kN$. Contribution to moment resistance of any reinforcement mesh and profiled metal decking used in the concrete slab should be ignored, since it fails at lower values of elongation than the reinforcement bars. This assumption has been used to develop standardized composite connections' tables for hot-rolled steel section by Steel

Construction Institute (SCI P213 1998). This can also be adopted for TWP steel sections.

4.1.2 Tensile force in bolts

Composite connection usually uses flush end-plate connection where the tension bolts are positioned underneath the upper beam flange. Most of the connection's moment capacity is developed by the contribution of tension reinforcement bar. However, the contribution of upper tension bolts row still needs to be considered to ensure that compression zone is not according to design specification. That could lead to premature failure due to non-ductile compression zone (BSI 1994). The bolts row furthest from the beam compression flange tends to attract more tension force than the lower bolts. The force permitted in bolts row is based on its potential resistance which relies on size of bolts and thickness of the end-plate as well as lever arm of the bolts. The potential resistance of each row of bolts in the tension zone is governed by bending of the end-plate or column flange. The values of F_{r1} and F_{r2} as shown in Fig. 3 are calculated by checking the top row working downward based on component method as explained elsewhere (Saggaff 2007).

4.1.3 Longitudinal shear force

The formation of full tensile force in reinforcement depends on longitudinal shear force being transferred from the beam to slab by means of shear connectors. According to Lawson (1990), full shear connectors should be provided in the region of negative moment. Reinforcement used in the connection should be extended beyond the negative region of the span and anchored sufficiently into the compression region of slab to satisfy requirements of anchorage length between reinforcement bars and concrete slab as suggested by British Standard Institute (BS8110 1997). For Type 2 deformed bar, anchorage length should be at least 40 times the bar diameter for concrete strength of 30 N/mm^2 (BS 8110 1997).

5. Flange and lower beam web

Compression resistance of compression zone in beam depends on direct force developed in the lower beam flange applied perpendicularly to the column's face. The component method in SCI publication (SCI 1998) has proposed that the applied compression stress should not exceed 1.4 f_y where f_y is the steel's strength in standardized composite connection tables for hot-rolled steel section. This 1.4*fy* factor is to allow for hardening strain and some dispersion into the web at section root. However, in this study as proposed by Saggaff (2007), applied compression stress is taken not to exceed 1.2 f_y as TWP section is a built-up section where the connection between the flange and web is welded, unlike hot-rolled section formed as one section.

5.1 Comparison of forces in tension and compression zones

In tension zone, forces leading to strength developed by the reinforcement bars combined with the bolts row, whereas in compression zone the force the resist the tension force developed from the bottom beam flanges. To ensure that no premature connection failure due to the failure of bottom beam flange, a comparison between tension forces and compression force is presented in Table 1. Results presented in Table 1, shows that compression forces are greater than tension forces for all connections. This shows that moment resistance of the connection is governed by

Table 1 Theoretical and experimental values of moment resistance for composite connections

Specimens	No. of bolts rows	Size of bolts	Tension Zone kN kN P_{reinf} F_{r1} F_{r2}	Tension Zone Total kN	Compression Zone kN	Moment Resistance (kN.m) $M_{R\ theo}$.
CF-5	1	M20 8.8	379 208 0	587	835	253.0
CF-6	1	M24 8.8	379 306 0	685	1379	363.3
CF-7	2	M20 8.8	379 208 167	754	953	330.4
CF-8	2	M24 8.8	379 306 229	914	1532	522.5

tension force developed from the reinforcement bars and the bolts row tension. In order to predict the compressive stress proposed value of 1.2 *fy* is quite reasonable, as the value is not greater than combination of tension force. Determination of moment resistance of composite connection is based on the calculated value of the tension bolts and compression force of bottom beam flange as explained later in this paper. The least value between tension and compression forces is used to determine moment resistance of the connection.

6. Standardization of composite connections

The use of partial strength connection for hot-rolled British sections is well established by SCI (SCI 1996). A series of experimental capacities test at the University of Alberta, Dundee has successfully been carried out to verify the predicted moment and shear capacity (Bose 1993). The results confirming the predicted values as well as standardized tables for the connection have been published by Steel Construction Institute (SCI 207 1996) and Jaspart (2000). The tests were carried out for hot-rolled steel section and non-composite connection.

Shi *et al.* (2007) tested composite joints with flush end plate connection as partial strength connection under cyclic loading. The composite joints with flush end plate connection showed large strength resistance and good ductility. The slippage between concrete slab and steel beam was very small, proving that full interaction between concrete slab and steel beam could be obtained when sheer connectors are properly designed. Further study of partial strength connection with TWP steel section was carried out by (Tahir *et al*. 2008). Two full scales tests were carried out with beam set-up as sub-assemblage and beam-to-column connections set-up as flush and extended end-plate connections respetively. It was concluded that the use of extended end-plate connection had contributed to significant reduction in deflection and significant increase in moment resistance of the beam as compared to flush end-plate connection.

In this paper, the connection is categorized as composite connection, while sections used are of TWP type only one table is presented from standard composite connections tables for TWP sections, only one table is presented in this study. Other standardized tables are published elsewhere (Saggaff 2007). Although best validation of the results presented in the tables is by comparing predicted results with extensive experimental test results. Due to higher cost involved in extensive full scale testing, only four tests were carried out to validate standard connection tables for TWP sections. Details of those tests results are explained later in this paper. The proposed standardized composite connection tables comprise of attributes which in some cases are not exactly the same as the one described by SCI (SCI P213 1998). In hot-rolled section the proposed beams are a built-up of hybrid TWP sections. This paper presents only one attribute of

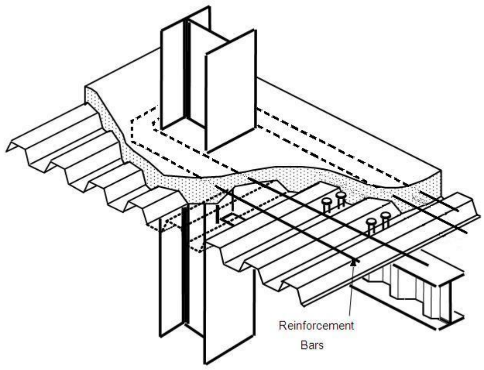


Fig. 4 Composite connection with the TWP beam section

Flush End-Plate, size 200×12 with M20 grade 8.8 bolts, from the standardized composite connection table. All procedures and calculation details needed to establish the standardized composite connection tables for all attributions proposed, are presented elsewhere (Saggaff 2007).

Fig. 4 shows a typical composite connection where TWP section is used as beam and steel decking with reinforcement bar arranged to form a slab. The proposed minimum thickness for corrugated web is 3 mm for shallow beam, while maximum thickness for deeper beam is 6mm in order to develop standardized composite connection tables.

6.1 Development of standard tables

Unlike simple and rigid connections, the design of composite connections involves more complex and rigorous procedures. Therefore, this paper proposed a reference guide for designing moment connections, which included sections in standardized capacity tables for bolted end plate composite connections (SCI P207 1996). The design model presented in the SCI's guide is in accordance with the 'component approach' adopted in Euro Code 3 Part 1-8 (EC3) and Euro code 4 (EN 1994). This approach considers the strength of each related component to form composite connection, and to determine moment resistance of the connection. Traditionally, the bolt forces are of triangular distribution but plastic distribution form; represents the actual behaviour of bolt forces accurately, (SCI 1996) as shown in Fig. 3. In SCI's guide, beam-to-column arrangements represent conventional hot rolled sections for both the beams and columns.

6.2 Notation of standard table

A spread sheet was developed to calculate the moment and shear resistance of the standardized connections. Critical zones check and component method proposed by SCI were adopted. Formulations and calculations are described in detail elsewhere (SCI P207 1996) (Saggaff 2007). The development of standard tables for connections in Table 2(a) represents beam side check, whereas Table 2(b) represent checking of column side. The compression value at the bottom flange (P_c) is equal to the area of cross section of the bottom flange of the connected beam multiplied by $1.2P_y$. This calculated compression value should be bigger than summation of forces in the tension zone so that premature failure can be avoided. Tension forces coming from reinforcements noted as (F_{reinf}), and forces of bolts noted as (F_{r1}+F_{r2}), appear in the tables. Moment resistance arising from the proposed table was calculated from the summation of each reinforcement and bolt row, multiplied by the lever arm of the connection. The lever arm for reinforcement bars tension is defined as the distance measured from the centre of compression beam flange bottom to reinforcement bar tension noted as '*Ar*' in the tables. Bolts row tension was measured as the distance between first bolt tension row and compression beam flange centre denoted as '*A*' in the tables. The moments that developed from reinforcement tension and bolts row tension were added together to establish moment resistance of composite connection. This moment resistance is designated as M_{cc} in the tables. If the summation of tension forces is greater than the compression forces, the moment resistance of connection is designated as dashed (-) in the tables, denoting the need to stiffen the connection. However, moment resistance for the stiffened connections is not available.

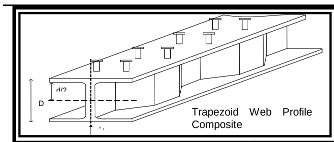
6.2.1 Geometrical configurations of connections

Dimensions of the end-plate noted are B_p for width and T_p for thickness. All flanges and web of beam attached to the end-plate were fully welded with minimum weld filler (size 10 mm for flange and 8mm for web). If ensured that mode of failure won't occur at welding. The design strength of flange and TWP section web is designated in brackets (F_t – Web – F_b) where F_t and F_b are S355 steel grade and Web is S275 steel grade. The size of TWP sections is designated as D for depth, B for width, W for weight per metre length, T_f for thickness of flange, t_w for thickness of web. For the reinforcement used in the Table 1(a), the design strength of the steel is denoted as T460 for steel grade of 460 N/mm^2. The diameters of the reinforcement are given as 12 mm and 16 mm. The

Table 2(a) Standardised Table for CFEP-1BRM20-EP200-12 (beam side)

Standardised Table-TWP steel Composite Connections (CFEP-1 BRM20-EP200-12)

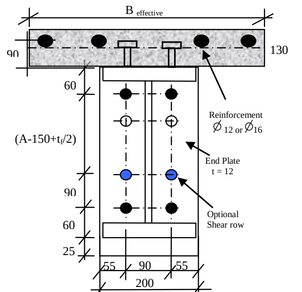
Beam Side



Moment Resistance of Composite Connection (Mcc)

1 ROW M20 8.8 B0LTS
FLUSH END PLATE 200 X 12
GRADE S-275

A_{bolt} = 585 mm^2
f_y = 355 N/mm^2
$\sum Fr_1$ = 208 kN
$\sum Fr_2$ = 0 kN



Vertical Shear Capacity
Without shear row = 258 kN
With shear row = 442 kN

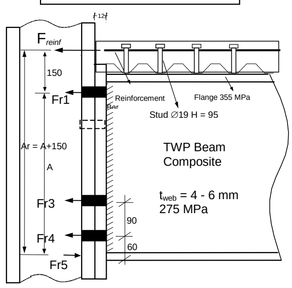


Table 3.2 a			**Flush End Plate**		**Dimension**		**Reinforcement**									
TWP - Beam Size (F_t – Web – F_b) (355 – 275 – 355)							**T12 f_y = 460**					**T16 f_y = 460**				**Bottom Flange Comp Capacity**
							A	**B**	**C**	**D**		**E**	**F**	**G**	**H**	
							4 D-12	**6 D-12**	**8 D-12**	**10 D-12**		**4 D-16**	**6 D-16**	**8 D-16**	**10 D-16**	
							452 mm	679 mm	905 mm	1131 mm		804 mm	1206 mm	1608 mm	2011 mm	
							0.35 %	0.52 %	0.70 %	0.88 %		0.62 %	0.94 %	1.25 %	1.57 %	
							198 kN	297 kN	396 kN	495 kN		352 kN	528 kN	705 kN	881 kN	
D x B x W	**T_f**	**t_w**	**B**	**T_p**	**'A'**	**'A_r'**	**M_{cc}**	**M_{cc}**	**M_{cc}**	**M_{cc}**		**M_{cc}**	**M_{cc}**	**M_{cc}**	**M_{cc}**	**P_c=Fr_5**
mm x mm x kg	mm		mm		mm											**kN**
300 x 120 x 26.3	10	4	200	12	235	**385**	**125**	**163**	**201**	**240**	**Concrete Grade 30**	**-**	**-**	**-**	**-**	**511.20**
300 x 140 x 33.7	12	4	200	12	234	**384**	**125**	**163**	**201**	**239**		**184**	**-**	**-**	**-**	**715.68**
350 x 120 x 27.6	10	4	200	12	285	**435**	**145**	**188**	**232**	**275**		**212**	**-**	**-**	**-**	**511.20**
350 x 140 x 35.1	12	4	200	12	284	**434**	**145**	**188**	**231**	**274**		**212**	**-**	**-**	**-**	**715.68**
400 x 140 x 39.7	12	5	200	12	334	**484**	**165**	**213**	**261**	**309**		**240**	**-**	**-**	**-**	**715.68**
400 x 160 x 48.4	14	5	200	12	333	**483**	**165**	**213**	**261**	**308**		**239**	**324**	**409**	**-**	**954.24**
450 x 160 x 50.2	14	5	200	12	383	**533**	**185**	**238**	**291**	**344**		**267**	**361**	**455**	**548**	**954.24**
450 x 180 x 60.1	16	5	200	12	382	**532**	**185**	**237**	**290**	**343**		**267**	**360**	**454**	**-**	**1226.9**
500 x 160 x 52.0	14	6	200	12	433	**583**	**205**	**263**	**321**	**379**		**295**	**398**	**501**	**602**	**954.24**
500 x 180 x 61.9	16	6	200	12	432	**582**	**205**	**263**	**320**	**378**		**295**	**397**	**500**	**495**	**1226.9**
550 x 200 x 73.3	16	6	200	12	482	**632**	**225**	**288**	**351**	**413**		**323**	**434**	**545**	**657**	**1363.2**
600 x 200 x 80.5	16	6	200	12	532	**682**	**246**	**313**	**381**	**448**		**351**	**471**	**591**	**711**	**1363.2**

Table 2(b) Standardised Table for CFEP-1BRM20-EP200-12 (column side)

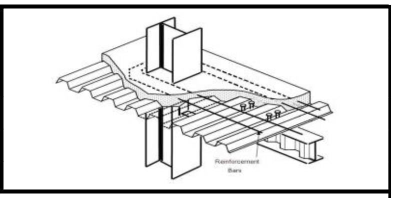
Standardised Table-TWP steel Composite Connections
(CFEP-1 BRM20-EP200-12)

Moment Resistance of Composite Connection (Mcc)

Column Side — Table 3.2 b

Column Property — f_y: 275 N/mm^2; 355 N/mm^2

1 ROW M20 8.8 B0LTS
FLUSH END PLATE 200 X 12
GRADE S-275



S – 275												Column Size	S – 355											
Panel Shear Capacity	Web Compression Capacity	Tension Zone Fr		Compression Zone Reinforcement Option								D x B x W	Compression Zone Reinforcement Option								Tension Zone Fr		Web Compression Capacity	Panel Shear Capacity
(kN)	(kN)	Fr_1	Fr_2	A	B	C	D	E	F	G	H		A	B	C	D	E	F	G	H	Fr_1	Fr_2	(kN)	(kN)
				T12 f_y = 460				T16 f_y = 460				356 X 368 X	T12 f_y = 460				T16 f_y = 460							
1000	1141	√		√	√	√	√	√	√	√	√	202	√	√	√	√	√	√	√	√	√		1486	1302
849	935	√		√	√	√	√	√	√	√	√	177	√	√	√	√	√	√	√	√	√		1217	1105
725	766	√		√	√	√	√	√	√	√	S	153	√	√	√	√	√	√	√	√	√		947	944
605	605	√		√	√	√	√	√	√	√	S	129	√	√	√	√	√	√	√	S	√		788	787
												305 X 305 X												
1037	1432	√		√	√	√	√	√	√	√	√	198	√	√	√	√	√	√	√	√	√		1865	1350
816	1051	√		√	√	√	√	√	√	√	√	158	√	√	√	√	√	√	√	√	√		1368	1062
703	858	√		√	√	√	√	√	√	√	S	137	√	√	√	√	√	√	√	√	√		1116	915
595	692	√		√	√	√	√	√	√	S	S	118	√	√	√	√	√	√	√	S	√		909	774
503	553	√		√	√	√	S	√	S	S	S	97	√	√	√	√	√	√	S	S	√		713	649
												254 X 254 X												
882	1384	√		√	√	√	√	√	√	√	√	167	√	√	√	√	√	√	√	√	√		1802	1149
685	992	√		√	√	√	√	√	√	√	S	132	√	√	√	√	√	√	√	S	√		1292	892
551	744	√		√	√	√	√	√	√	S	S	107	√	√	√	√	√	√	S	S	√		969	717
434	557	√		√	√	√	S	√	S	S	S	89	√	√	√	√	√	S	S	S	√		725	566
360	436	√		√	√	S	S	S	S	S	S	73	√	√	√	S	S	S	S	S	√		583	465
												203 X 203 X												
459	701	√		√	√	S	S	S	S	S	S	86	√	√	√	√	√	S	S	S	√		913	598
353	512	√		√	√	S	S	S	S	S	S	71	√	√	√	S	S	S	S	S	√		666	460
322	440	√		√	S	S	S	S	S	S	S	60	√	√	S	S	S	S	S	S	√		568	415
272	360	√		√	S	S	S	S	S	S	S	52	√	S	S	S	S	S	S	S	√		464	351
245	313	√		S	S	S	S	S	S	S	S	46	S	S	S	S	S	S	S	S	√		404	316

Tension Zone :
√ = Column Satisfactory for bolt row tension values shown for the beam side

Compression Zone :
√ = Column Capacity exceeds : $\sum F = F_{reinf} + Fr_1$
S = need Compression Stiffener.

number of reinforcement assigned as 4D-16 is designated as 4 numbers of 16mm diameters of reinforcement bars. The area of reinforcement is also given and converted to percentage area of steel in concrete as shown in Table 1(a). Beneath this percentage row is the values given for the tension force of the reinforcement. The tension force of the bolts is denoted as F_{r1} and F_{r2}. The number and size of reinforcement will determine the moment resistance of the connection. Therefore, Table 2(a) is divided into column *A* to *H* to differentiate the moment resistance of the connection due to the change in size and number of reinforcement. If the column has a smaller capacity, a stiffener is needed on the column side. The need of the stiffener is denoted as "*S*" in Table 2(b). The value given in Table 2(a) presented in this paper is the moment resistance of composite connection where no stiffener is needed on the column web. Table 2(a) also presents the vertical shear capacity of the connection with shear bolts row and with optional shear bolts row.

6.3 Validation of the standard tables

Standard table's validation for TWP composite connections is best presented by comparing predicted values with full scale testing of the connections. A series of full scale testing on TWP girder sections were conducted at UTM Construction Research Centre, Universiti Teknologi

Table 3 Geometrical configuration of composite connections

Specimen No.	No. of bolts rows	Size of bolts	Size of end plates			Size of TWP Beam	Size of Column
			Width	Depth	Thickness		
CF-5	1	M20 8.8	200	440	12	400x140x39.7/12/4	305x305x118
CF-6	1	M24 8.8	200	540	16	500x180x61.9/16/4	305x305x118
CF-7	2	M20 8.8	250	490	12	450x160x50.2/12/4	305x305x118
CF-8	2	M24 8.8	250	640	16	600x200x80.5/16/6	305x305x118

Malaysia. Four full scale testing are conducted to validate the standardized tables, as well to understand the behaviour of proposed TWP composite connections. Although the tests Table 3) did not cover the whole range of proposed beam-to-column connections as in Table 2(a) and (b), comparing test results with predicted values can still be established. Actual strength of steel and concrete used can evolve calculations using component method to predict the moment resistance of the connections. Test specimen's comparison using component method to establish standardized composite connection tables, are discussed later in this paper.

6.3.1 Experimental tests

Table 3 showed that the parameters varied with different geometrical configurations. TWP beam section sizes varied from 400 mm to 600 mm in depth; beams width varied from 140 to 160 mm. The size of TWP sections is designated as 400×140×39.7/12/4 where 400 is the depth in mm, 140 is the width in mm, 39.7 is the weight in kilogram per metre. Thickness of flange (12 mm) and the thickness of web was 4 mm. The hybrid TWP beam section was fabricated in such a way that both flanges are of grade S355 and the web's grade S275. Four numbers of 16 mm diameter reinforcement bars with yield strength of 460 N/mm^2 were used to develop tension force of the connection. Thickness of the slab was maintained at 125 mm, while concrete grade was 30 N/mm^2 for all specimens. Column size with steel grade of S275 was maintained at 305×305×118UC for all tested specimens categorized as a heavy section. This was purposely done so that the column will not fail due to buckling or shearing. High tensile bolts of 2 different sizes were used namely, M20 and M24 of Grade 8.8 bolts. M20 bolts size was decided in conjunction with 12 mm thick end-plate while the size of M24 bolts was used with 15mm thick end-plate. They were matched together on purpose so deformation of end-plate and failure, act together to develop full strength without any premature failure. Yield strength of the end-plate was steel grade S275 recommended for ductile types of connections by SCI (SCI P207 1996).

6.3.2 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 (1.3 m width) which acts as a permanent formwork for the slab was attached to the top flange of TWP beam by a pair of shear studs on each. A total of 4 pairs comprised of 8 numbers of shear studs were installed. The shear studs measuring as 19 mm in diameter and 95 mm in height were used. These studs should be able to resist tension force developed from reinforcement tension of the bars. Four reinforcement bars of size 16 mm diameter were installed around the column and embedded in the slab as shown in Fig. 4. Earlier on the tension force developed from these bars was calculated to be 379.6 kN. The resistance of one shear stud of 19 mm diameter embedded in normal concrete strength (30 N/mm$^{2)}$ is regarded as 80 kN per stud based on U.K Annex of Euro-code 4. This resulted to a total of 640 kN for 8 studs

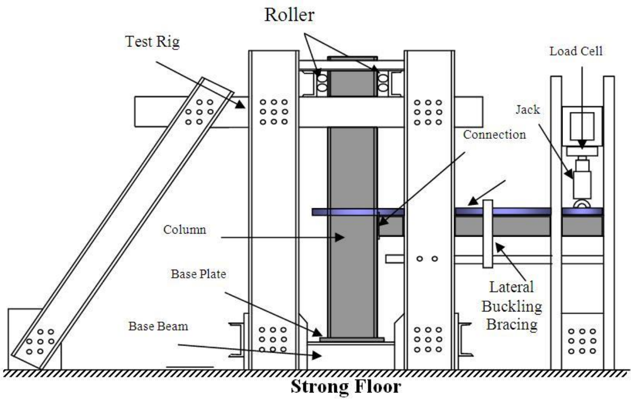


Fig. 5 Arrangement of tested specimen on the test rig

installed, it is greater than 379.6 kN, enough to carry the tension force developed by the reinforcement bars.

The reinforcement bars should also be checked for minimum required area in concrete slab. The total area required for four bars is 804 mm^2 which is greater than the minimum required area of 500 mm^2 as suggested by (SCI P213 1998). Three tensile tests were carried out as to understand the material properties of the reinforcement bars. The average value of yield strength of bars was recorded as 496 N/mm^2 which is higher than the expected design strength of 460 N/mm^2. The average elongation of bars was recorded as 6.98% at maximum load which is higher than 5% (SCI P213 1998). The reinforcement bars should also be checked for the anchorage length which is supposed to be at least 40 times the bar's diameter. As the bars were installed along the cantilever beam, the length of anchorage was 1300 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 top and bottom parts of the column were 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 beam, and load applied. A small load was gradually applied up to a third of expected failure load, sufficient enough to cause non-elastic deformation. That established the connection in a 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 of applied load, displacement, and rotation of beam and column were recorded after a lapse of two minutes 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 specimen was then controlled by deflection increments of 2 mm. The test continued till failure, when large deformation occurred or the load decreased significantly. The response of a joint in those phases may have governed the buckling behaviour of connected column. 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 column flange. Rotation of the connection was measured as the difference between rotation at the centre of beam as well as centre of the column, which was recorded using an inclinometer.

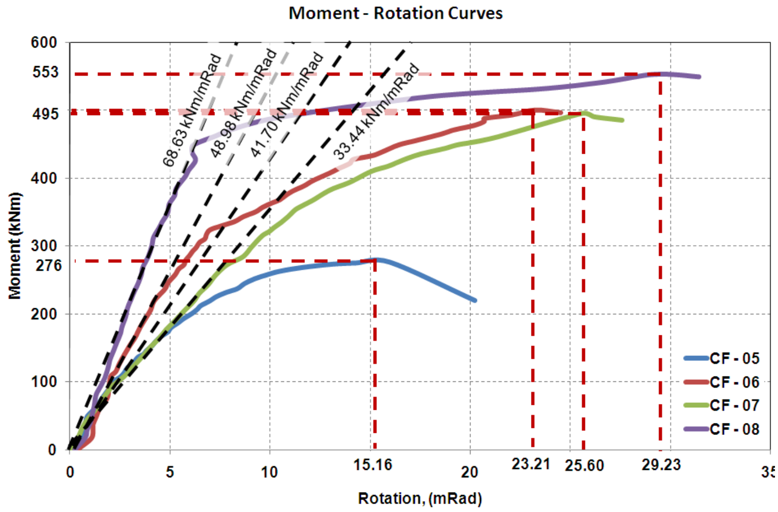


Fig. 6 Moment-rotation curves composite connection

Table 4 Theoretical and experimental values of moment resistance for composite connections

Specimens	Moment Resistance M_R (kNm) $M_{R,\ Theo.}$	$M_{R,\ Exp.}$	Ratio of Experimental vs. Theoretical values	Moment capacity of the TWP beam M_{cx} in kNm	Ratio of $M_{R,\ Exp.}/\ M_{cx}$
CF-5	253.0	276.0	1.09	238.6	1.16
CF-6	363.3	500.0	1.38	511.2	0.98
CF-7	330.4	495.0	1.50	306.7	1.61
CF-8	522.5	553.0	1.06	681.6	0.81

6.3.3 Test results

The behaviour of the connections depends very much on their geometrical configurations with some exceptions have significant effect. The depth of the beam, number, size and distance of the bolts and end-plate thickness may significantly affect moment resistance and rotation stiffness of connection. However, the contributions of the profiled metal decking and the steel mesh were ignored as they failed at lower values of elongation than the reinforcement bars (SCI P213 1998). To understand the effect of these geometrical configurations on moment resistance, the rotational stiffness, and ductility of composite connection should be discussed based on plotted M_R-Φ curve results.

6.3.3.1 Moment (M_R) versus rotation (Φ) curves

Moment resistance of the connection depends very much on the connected members and the types of joints. Beam-to-column connections generally behave linearly followed by non-linear manner in moment-rotation curves joint behaviour exhibits a form of material non-linearity. Since the structural analysis needs to account for this non-linearity of joint response to predict the moment resistance accurately. The curves plotted for non-linear behaviour of moment resistance versus rotation of the connection, (M_R- Φ) for specimens CF 5 to CF 8 respectively (Fig. 6). The Moment resistance, M_R test results are listed in Table 4. They were determined from each of the *M*-Φ curve plotted in Fig. 5 based on the maximum moment. The overall results showed that

Table 5 Material properties of beams, columns, end-plates, and reinforcement bars

No.	Beams and Columns	Yield Strength, f_y (N/mm^2)	Ultimate Strength, f_u (N/mm^2)	Modulus of Elasticity, E (kN/mm^2)
1	400×140×39.7/12 mm (flange)	414	528	194
	400×140×39.7/4 mm (web)	378	447	198
2	450×160×50.2/12 mm (flange)	414	528	194
	450×160×50.2/4 mm (web)	378	447	198
3	500×180×61.9/16 mm (flange)	399	471	193
	500×180×61.9/4 mm (web)	378	447	198
4	600×200×80.5/16 mm (flange)	399	471	193
	600×200×80.5/6 mm (web)	327	449	195
5	305×305×118 (flange)	370	516	202
	305×305×118 (web)	359	510	194
		(avg. 364.5)	(avg. 513)	(avg. 198)
6	End-plate (12 mm)	305	467	203
	P1	308	491	205
	P2	309	470	204
	P3	(avg. 307.3)	(avg. 476)	(avg. 204)
7	End-plate (15 mm)	310	515	204
	P4	311	524	205
	P5	308	507	203
	P6	(avg. 309.7)	(avg. 515.3)	(avg. 204)
8	Reinforcement bars (T16mm)	511	610	205
		490	596	210
		486	601	212
		(avg. 495.7)	(avg. 602.3)	(avg. 209)

experimental values of maximum moment resistance were greater than the predicted values, in terms of the ratio ranging from 1.06 to 1.50 (Table 4). Moment resistance in experimental was greater than of theoretical values. It is consistent with the findings of other researchers for hot-rolled steel section (Sulaiman 2007, Tahir *et al*. 2009, Tahir *et al*. 2008)

These results showed that the component method suggested by Steel Construction Institute is not limited to hot-rolled section only; it is suitable for the TWP section also. The ratio between experimental and theoretical values did not show a linear increment based on beam depth. Connection moment resistance is influenced by failure of connection's components such as beam, columns, end-plate, bolts and cracks in concrete slab. The results in Table 4 also show that the ratios of $M_{R, Exp.}/ M_{cx}$ for CF-5 amd CF-7 specimens were higher than 1.0 and the ratios of $M_{R, Exp.}/ M_{cx}$ for CF-6 and CF-8 specimens were lesser than 1.0 but greater than 0.25. It can be concluded from the results that CF-5 and CF-7 specimens can be classified as partial strength connection while CF-6 and CF-8 specimens as full strength connection based on the classification of connections suggested by SCI (SCI P207 1996). Those results also point out that moment resistance of specimen CF-5 was 276 kNm, found as least of all the tested specimens. It may be the result of development of cracks in concrete slab as the reinforcement bars yielded. Specimen CF-5 with least depth of the connected beam could result to higher tension force developed in tension reinforcement bars than the rest of the tested specimens. As the moment resistance of the connection is based on the lever arm between tension zone and compression zones, shorter lever arm tends to increase the development of tension force in reinforcement bars due to the use of

Table 6 Tests results based on initial stiffness

Specimen	No. and size of bolt	Size of TWP Beam (No. and size of bolts)	Moment Resistance M_R, (kNm)	Actual Rotation **Φ** at M_R (mrad)	Initial Stiffness, $S_{j,ini}=M_R/\Phi$ (kNm/mrad)
CF-5	1M20	400×140×39.7/12/4	255.0	9.95	25.63
CF-6	1M24	500×180×61.9/16/4	368.0	10.30	35.73
CF-7	2M20	450×160×50.2/12/4	428.0	16.82	25.45
CF-8	2M24	600×200×80.5/16/6	470.0	8.03	58.53

shallower beam. As a result, the reinforcement bars tends to yield and develop cracks in concrete slab. The formation of excessive cracks at low level of applied load tends to reduce the moment resistance of the connection.

Coupon tests were carried out for the flange and web of TWP beams, the hot-rolled column and the end-plates. Mean value of yield strength (f_y), ultimate strength (f_u), and modulus of elasticity (E) were recorded in Table 5. The f_y results show that the experimental results were higher than those of theoretical values. The predicted values derived from component method were calculated based on the actual yield strength given in Table 5. However, the use of standard design strength of S275 and S355 to develop standardized connections tables and to calculate theoretical values for TWP sections is in accordance with the procedures suggested by SCI for hot-rolled steel sections (SCI P213, 1998). This is done because the proposed standardised tables are used by designers where the standard design strength of steel is necessary to ease the design procedures.

6.3.3.2 Rotation stiffness and ductility

Initially, the connections have a stiff response which is then followed by a second phase of much reduced stiffness. This second phase is due to non-linear deformation of connections' components or those of the frame members in the immediate vicinity of the joint i.e., beam and column. These deformations have to be considered because they contribute substantially to frame displacements and may affect internal force distribution significantly. The structural analysis of joint response is needed for both rotational stiffness and ductility of joint behaviour. The stiffness of connections is presented by drawing straight line along the linear region to show initial stiffness in the M-Φ curve. The value of initial stiffness is calculated as moment resistance divided by rotation of connection at that particular moment in the linear region. The results of initial stiffness derived from the M-Φ curves are tabulated in Table 6. By increasing the number and size of bolts, increase in initial stiffness of the connection is more significant.

The ductility of connection is measured as the ability of the connection to form a plastic hinge. This can be recognized from the M-Φ curve by the non-linear region formation which rotates at maximum load without any abrupt failure. The ability of connection to rotate to form a ductile connection is an important criterion to satisfy the requirement in the design of semi-continuous construction (Chouchman 1997). In semi-continuous construction, the connections fail as a ductile connection and possess a rotation that allows deformation of the connection instead of connected members. The connections are considered as ductile connection if rotation can achieve at least 20 mrad to form a plastic hinge without any sudden failure, as suggested by Steel Construction Institute (SCI 1998). Table 6 showed rotation of tested connections at maximum moment. The results showed that not all specimens possess rotation of connections with at least equal to 20 mrad. In these tests, specimen CF-5 shows that rotation is recorded as 15.16 mrad which is lower

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

Specimens	Exp. Mom. Resistance, M_R (kNm)	Initial Stiffness, $S_{j,ini}=M_R/\Phi$ (kNm/mrad)	Rotation of connection at max. moment (mrad)
C-5	255.0	25.63	15.16
C-6	368.0	35.73	23.21
C-7	340.0	25.45	25.60
C-8	470.0	58.53	29.23

Fig. 7 Crack pattern in concrete slab

than the suggested value of 20 mrad. The connection however, does not fail abruptly but deformed and behaved as a ductile type of failure. Specimen CF-5 possesses the least initial stiffness. However, this behaviour was quite consistent with the test results and models carried out by European Commission which concluded that rotation capacity increased with stiffness of the connection (EC 1996). Table 7 shows that as the stiffness increased the rotation of the connection also increased which was consistent with European Commission findings (EC COST C1 1996).

6.3.3.3 Mode of failures

Connection's failure mode is focused in three zones, namely, tension, shear, and compression. These failure zones take into account failure of all joint components i.e., columns and concrete slab. However, tested connections failure mode in tension zone is considered most critical failure. The compression zone shows no signs of failure in all specimens. In tension zone where concrete is very weak, the possibility of failure is most likely due to cracks in concrete. However, in composite connection, the installation of reinforcement bars in tension zone embedded in concrete slab prevents pre-mature failure. This reduces the possibility of failure significantly due to cracks in concrete. In these tests, all specimens failed due to cracks. No significant sign of deformation of the end-plate or bolts was observed in all specimens. Concrete cracking started at the column corners where high stress occurs due to tension force of the bars that transferred to concrete slab and spread out transversely to left and right edges of slab. The cracks roughly have the same width. The length of cracks was extended towards the end of slab, with cracks pattern in all tests propagated up to 400-600 mm length on both sides of the column as marked in Fig. 7. These cracks however are barely visible. The cracks are considered modest and occurred only at the top of composite slabs surrounding the universal column. Visible cracks occurred when the load

Fig. 8 Failure mode in concrete at maximum crack width

applied was almost maximum. Width of main crack is about 10-13mm at the time of failure when tests were stopped as shown in Fig. 8. The pattern of cracks is very much related to stiffness of the connection. The stiffer the connection lesser the cracks developed. This can be seen in the reflection of inclined pattern of cracks. The least stiff connection as in CF-5 has led to almost straight cracks running transversely across the slab at lower applied load. Therefore in composite connection, formation of cracks in the slab is only visible when maximum load is reached. No visible cracks occurred when moment resistance developed at initial stage of loading.

7. Conclusions

The following conclusions are presented with regard to both the proposed standardized composite connection tables and performance of tested specimens.

(1) The study concluded that it is possible to establish standard composite connection tables using TWP steel sections by adopting the component method as proposed by SCI. The use of component method is not limited to hot-rolled compact section it can be also applied to slender part of TWPS built-up section.

(2) The proposed value of 1.2*fy* to predict applied compressive stress at lower beam flange pointed to the absence of premature failure in compression zone. This finding implies that the use of 1.4 fy for applied compressive stress is quite conservative as suggested by EC 3.

(3) The Moment resistance values calculated in the proposed standardized table were lower than the actual tested values steel strength in the actual experimental tests were higher than the design strength.

(4) Overall results showed that increasing the number and size of bolts, led to significant initial stiffness of connection.

(5) The moment resistance values obtained from the experimental tests plotted as M-Φ curves showed good agreement with the component method. According to the results, experimental values of moment resistance were greater than predicted values with the ratio ranging from 1.09 to 1.50.

(6) The ratio between proposed composite connections to the connected TWP steel beam are in the range of 0.81 to 1.61. It can be concluded that the tested composite connections specimens can be classified as partial strength and full strength connection.

(7) All tested specimens except CF-5 possessed a rotation more than 20 mrad. All specimens showed a ductile type of failure rather than an abrupt failure.

(8) Critical failure of connections was due to the formation of cracks in concrete slab at maximum load for all specimens. No significant deformation occurred at the end-plate or the column flange.

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