SPS-PGSS
UNIVERSITI TEKNOLOGI MALAYSIA

IGCES 2008

e-proceeding

ISSN 1823-3287

eam to Column artial Strength Composite Connections sing Trape oid eb rofiled (T) Sections

Anis Saggaff[1], Mahmood Md. Tahir[2], Shahrin Muhammad[2], Arizu Sulaiman[3]
[1]PhD, Civil Engineering Department, Faculty of Engineering, Sriwijaya University, Palembang - Indonesia
[2]Prof, Steel Technology Centre (STC), Faculty of Engineering, Universiti Teknologi Malaysia (UTM), Johor
[3]PhD, Steel Technology Centre (STC), Faculty of Engineering, Universiti Teknologi Malaysia (UTM), Johor
Email: anissaggaf@yahoo.com

Abstract

In constructions, especially steel, there are commons that the connections are usually designed as pinned which associated with simple construction or rigid. The connections related with continuous construction. However, the actual behavior failures of the structures are between these two extreme cases. The use of partial strength or semi-rigid connections has been encouraged by Eurocode 3 and the studies have proven the significant savings in construction steel weight. The theoretical calculations and experimental work has to do in order to prove the each other for the verification. The objective of this paper is to compare and see the relationships between calculations and experimental works for partial strength composite connections using flush end-plate connections with Trapezoid eb Profile (T P) as a beam and Universal Column (UC). The connection comparison tables were presented base on all possible failure modes that occur in the beam to column connection on major axis. A method proposed by Steel Construction Institute (SCI) which take into account the re uirements in Eurocode 3, Eurocode 4, and BS 9 0:2000 were adopted to predict the moment capacity and shear capacity in developing the tables. The theoretical calculations of moment capacity presented in this paper were validated by six full scale testing that have been carried out at Steel Technology Centre, Universiti Teknologi Malaysia. The tests showed good agreement between experimental and theoretical values.

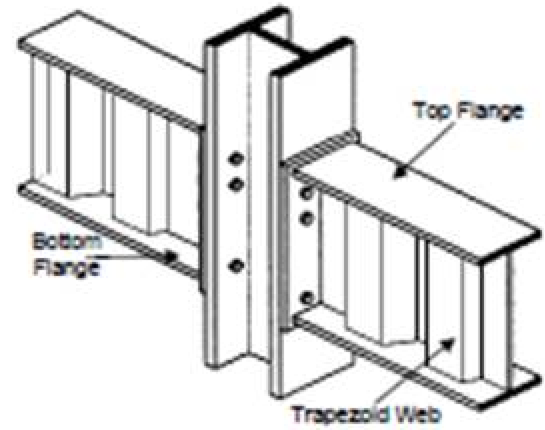
Keywords— Theoretical and Experimental, Partial strength, Composite Connections, Moment Capacity, TWP section, flush end-plate connection, Universal Column (UC) section.

1. I TROD CTIO

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

Traditionally, steel frames are designed either as pin or rigid. In pin joint, the beam connections are assumed as simple supported and the columns are assumed to sustain axial and nominal moment. Partial strength composite connection is used for connection in the design of semi-continuous construction for multi-storey steel frames as mentioned by Eurocode 3.

The strength of reinforcement embedded inside the concrete slab was taken into account in improving the moment capacity and stiffness of the connection. In semi-continuous frame the degree of continuity between the beams and columns is greater than that in simple construction design but less than that in continuous construction. The studies conducted on the use of partial strength connection have proven substantial savings in overall steel weight.



II. Methodolgy

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

2.1 Analysis of Trapezoid Web Profiled (TWP) Composite and Connections

2.1.1. Concept of TWP Composite

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

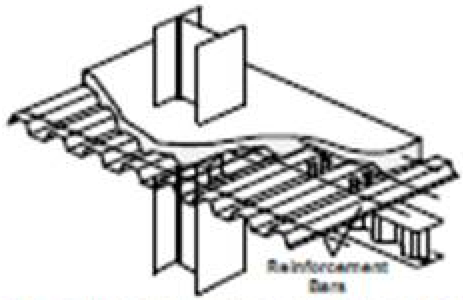


Figure 2. Typical flush-end plate connection for T P section with studs connected to hot-rolled universal column (UC) section.

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

Figure 3. Installation of Studs, metal decking, and composite

2.1.2. Benefit of TWP Composite

Based on the configuration of the structure, TWP beam can offer substantial saving in the steel usage, and in some cases up to 40% as compared to conventional rolled sections. The advantages will be more significant when there is a need for a column free, long span in structural system, such as portal frames for warehouses, girder for bridges, floor and roof beam for high-rise buildings, and portal frame for factory.

2.2 Aspect of partial strength connections

In the design of braced multi-storey steel frames, the steel weight of the connections may account for less than 5% of the frame weight; however, the cost of the fabrication is in the range of 30% to 50% of the total. Fig 2 shows the typical TWP section with studs.

The benefits of overall cost saving of the partial strength connections are more than simple connections. It is reported that partial strength connection saves the steel weight when used in non-composite multi-storey braced steel frames using British hot-rolled section was up to 12%. The overall cost saving was up to 10% of the construction cost which is quite significant.

2.3 Composite connection designs

The design philosophy presented in this paper was adopted from 'component approach' described in SCI. The moment capacity of the connection was determined by considering the capacity of each relevant component such as tensile of top bolt row and the tensile capacity of the reinforcement anchored inside the concrete slab as shown in Fig. 4. The composite connection capacity was checked to meet the requirement of BS 5950:2000. Moment resistance of the connection was developed from tension force of the reinforcements and the upper bolts with the compression of the bottom flange.

2.3.1 Tension Zone.

2.3.1.1 Reinforcement bar

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

2.4 Tensile Bolts.

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

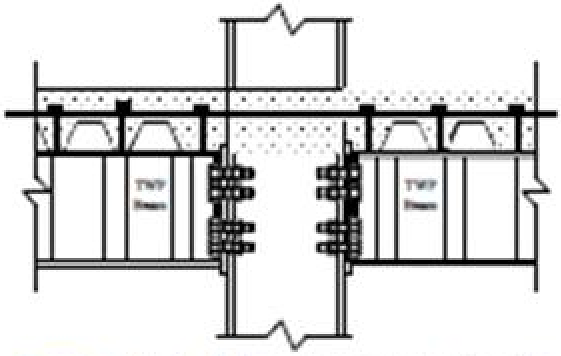


Figure 4. The details of T P beam to universal column connection

2.5 Longitudinal shear forces.

The tensile force in the reinforcement depends on the longitudinal shear force transferred from the TWP beam to the slab as shown in Fig. 5. According to BS5950; full shear connectors should be provided in the region of positive moment. The reinforcement used in the connection should be extended beyond the negative region of the span until the compression region of the slab to satisfy the requirements of BS 8110.

2.5.1 Compression Zone.

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

2.5.1.1 Flange and web of the lower beams.

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

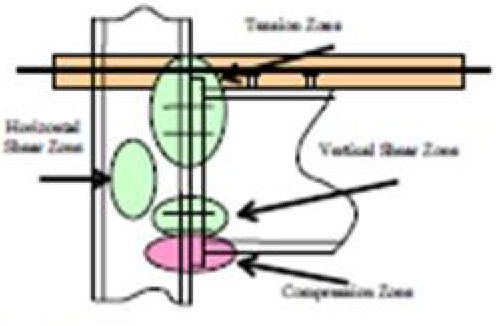


Figure . View of the tension area, compression area, and shear values of the connection area

2.5.1.1 Column web.

The compression zone of column web should be checked for the buckling and bearing resistance. The compression resistance should not be over the limit of the buckling and bearing resistance of the column web so that failure of column web as non-ductile can be avoided.

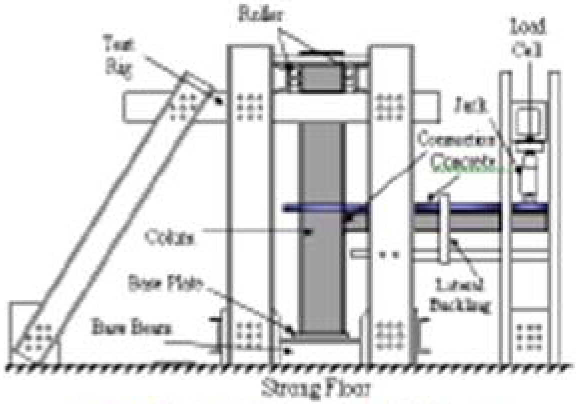


Figure 6. Test rig for Experiments in the Laboratory

III. R S TS

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

3.1 Test procedures.

Test specimens were set-up by connecting two 3 m length columns and a 6 m long beam as shown in Fig. 6 and Fig. 7. A metal decking with 1.5 m width which acts as a permanent formwork for the slab was attached to the top flange of the column using pairs of shear stud with diameter 19 mm and height 95 mm. The thickness of the slab was taken as 130 mm with concrete strength grade 30.

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

TABLE 1
TEST SPECIMENTS WITH VARIOUS PARAMETERS.

Test.	Bolt Rows	Bolts	End Plates W	End Plates D	End Plates T	Size of TWP Beam	UC
C5	1	M20 8.8	200	440	12	400x140x39.7	305x305x118
C6	1	M24 8.8	250	540	15	500x180x61.9	
C7	2	M20 8.8	200	490	12	450x160x50.2	
C8	2	M24 8.8	250	640	15	600x200x75.3	

The data logger system was set-up to read displacements from inclinometer in millimeters and load in kN. An increment of 5 kN loading was applied to the specimen. The readings for loads, displacement, and rotation that were recorded after two minutes had elapsed. The loading on the specimen was then controlled by deflection increments of 3 mm. The test was continued until the failure of specimens; when large deformation or the load decreases significantly.

Figure 7. Experimental works during the test in the Laboratory

3.2 Prediction of Moment Resistance (MR).

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

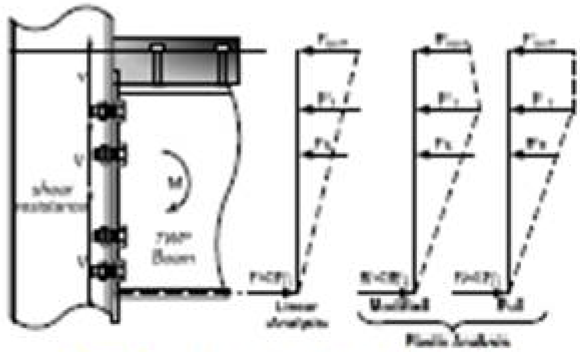


Figure 8. Forces diagram for elastic and plastic Analysis

TABLE 3
COMPARISON OF MOMENT CAPACITIES FOR COMPOSITE AND NON-COMPOSITE CONNECTION.

Specimen	MR (kNm) Composite	MR (kNm) Non-composite (EEP)	% Difference
C-5	212.0	123.0	72.4
C-6	305.0	181.0	67.0
C-7	337.0	235.0	43.4
C-8	466.0	432.0	7.9

The graphs showed that the connections behaved linearly in the first stage followed by non-linear behavior and gradually losing the stiffness with the increase in rotation. The overall results showed that the experimental values of moment resistance were greater than the theoretical values with the ratio ranged in between 1.01 to 1.23 as shown in Table 2.

TABLE 2
THEORITICAL AND EXPERIMENTAL VALUES OF MOMENT RESISTANCE FOR EACH SPECIMEN

Specimen	Moment Resistance (k m) Theoretical	Moment Resistance (k m) xperimental	Ratio of Theoretical vs. xperimental values
C-5	212.0	220.0	1.04
C-6	305.0	330.0	1.08
C-7	337.0	430.0	1.23
C-8	466.0	470.0	1.01

I . DISC SSIO

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

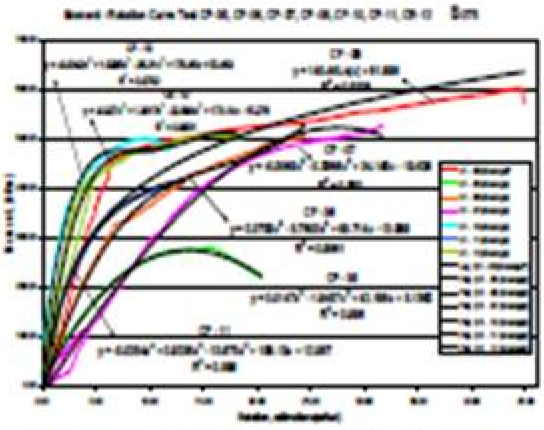


Figure 9. Moment Rotation Curve Test Result C- to C-12

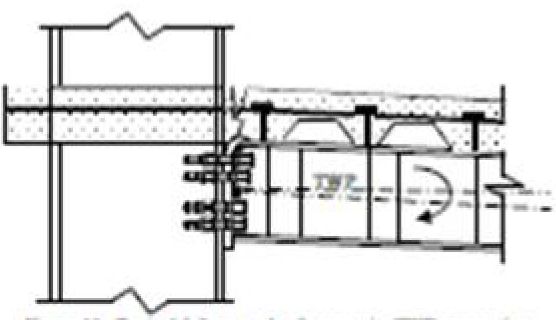


Figure 10. Typical failure mode of composite TWP connection

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

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

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

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

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

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

. CO C SIO

The moment capacity of flush end plate connections connected to a column flange for TWP composite can be determined by adopting the method proposed by SCI. The capacities of the connection depends on the geometrical aspects of the connection such as the size of bolt, number of bolt, size of end-plate, thickness of end-plate, size of beam and size of column. The increase in the number of bolt row from one row to two rows has contributed to an increase in the moment capacity in the range of 50% to 59% which is quite significant. The increase the size of bolt from M20 with 12mm thick end-plate to M24 with 15mm thick end-plate has contributed to an increase in the moment capacity in the range of 48% to 55% which is about the same as the effect of increasing the number of bolt from one to two bolt rows. The shear capacity of the connection contributed the shear capacity of the connection by an increment of 71% with the addition of optional shear bolt row. The proposed tables can be used in the design of semi-continuous construction in multi-storey steel frames.

AC O D M T

This study is part of a research towards a PhD by one of the authors. The authors would like to acknowledge special thanks and gratitude to CIDB for funding this project. Special thanks is also addressed to Steel Technology Centre (STC) and all staffs of Structure Laboratory, Universiti Teknologi Malaysia for some efforts conducting with this research that could bring this experiment awarded gold medal on Malaysian Technology Expo (MTE) 2006 and International Invention Exhibition 2006 in Geneva.

REFERENCES

[1] Steel Construction Institute and British Constructional Steelwork Association. (1996). *Joints in Steel Construction. Volume 1: Moment Connections.* , London.

[2] Eurocode 3. (1992). *Design of Steel Structures: ENV 1993-1-1: Part 1.1: General Rules and Rules for Buildings.* CEN, Brussels.

[3] Md Tahir, M (1995). *Structural and Economic Aspects of The Use of Semi-Rigid Joints in Steel Frames* . PhD Thesis. University of Warwick, United Kingdom.

[4] G. H. Couchman, 1997, *Design of Semi-continuous Braced Frames,* Steel Construction Institute Publication 183, Silwood Park, Ascot, Berkshire SL5 7QN, U.K.

[5] Osman, M. H. (2001). *Performance Test and Research on Trapezoid ab Profile* , Presentation in Design of Steel Structure Short Course, Universiti Teknologi Malaysia.

[6] Hussein, Wa' il Q. (2001). *Design Guide for Steel Plate Girder with Corrugated ebs (T P)* , Presentation in Design of Steel Structure Short Course, TWP Sdn Bhd.

[7] Steel Construction Institute and British Constructional Steelwork Association, SCI P213 (), Joints in Steel Construction Composite Connections, Silwood Park, Ascot, Berks SL 7QN, London.

[8] Steel Construction Institute and British Constructional Steelwork Association. (1994). *Joints in Simple Construction. Volume 1: Design Methods* , Second Edition, Silwood Park, Ascot, Berks SL 7QN, London.

[9] British Standards Institute BS 5950-1. (2000). *Structural Use of Steelwork in Building Part 1: Code of Practice for Design – Rolled and elded Sections.* British Standards Institution, London.

[10] Bose, B. *"Tests to verify the performance of standard ductile connections,"* Dundee Institute of Technology, 1993.

[11] M. Md. Tahir, *Design of Semi-Continuous Construction for Multi-Storey Braced Steel Frames Using T P sections* , Technical report, Steel Technology Centre, Universiti Teknologi Malaysia, Skudai, Johor, Malaysia, 2003.

[12] Abdalla, K. M. and Chen, W. F. (1995). "Expanded Database of Semi-Rigid Steel Connections." *Computer And Structures,* Vol. 56, No. 4, pp 553-564.

[13] Chen, W. F. et al. (1993). *Semi-rigid Connections in Steel Frames.* Council on tall Buildings and Urban Habitat, Committee 43, Mc Graw-Hill, New York.

[14] Chen, W. F. and Kishi, N. (1989). "Semi-rigid Steel Beam-to-Column Connections: Database and Modelling." *Journal of Structural Engineering.* Vol. 115, No. 1, pp 105-119.

[15] Luo, R. (1995). "Load Carrying Capacity of Steel Girders and Panels with Thin-Walled Trapezoidally Corrugated Webs." Compilation of Papers. Chalmers University of Technology, Sweden.

[16] BRITISH STANDARDS INSITUTION, DD-ENV 1994-1-1: 1994 Eurocode 4: Design of composites steel and concrete structures Part 1.1 General rules for buildings EC 1994.

[17] EUROPEAN COMMISSION, COST C1, Semi-rigid behaviour of civil engineering structural connections, composite steel-concrete joints in braced frames for buildings EC 1994.

[18] BRITISH STANDARDS INSTITUTION, BS 5950: Structural use of steelwork in building Part 3: Design in composite construction Section 3.1: 1990: Code of practice for design of simple and continuous construction: hot rolled sections BSI, 1990.

[19] BRITISH STANDARDS INSTITUTION, BS 8110: Structural use of concrete. Part 1: Code of practice for design and construction, 1997.

[20] Aziz Saggaf, *"Economic aspects of composite partial-strength connection in semi-continuous construction for multi-storey braced steel frame using T P sections"*, Civil Engineering Faculty, Universiti Teknologi Malaysia, Skudai, Johor, Malaysia, Ph.D Thesis, 2007.

[21] Joint in Steel Construction, Composite Connections, The Steel Construction Institute Publication 213, Silwood Park, Ascot, Berkshire, SL5 7QN, 1998.