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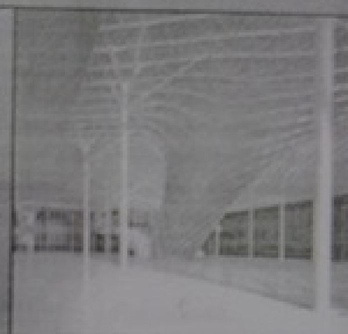

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Partial Strength Connections Using Trapezoid Web Profiled (TWP) Bare-Steel Beam Section

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ABSTRACT

Semi-rigid connection concept has been studied in the latest decade either with the theoretical analysis or with the experimental works. The publications are starting to grow up significantly however most of the researches are limited to typical hot-rolled sections as beams. The study on partial-strength connection in semi-continuous design is not much proposed due to its complicated behavior and analysis compared to simple connection design, however it gives more economics beams in size and weights. The analysis of 'semi-continuous' connections including the moment resistance of 'partial-strength' connections in plastic hinge has been mentioned by the Eurocode 3 for designing steel frames. Joints comprising of semi-rigid and partial strength connections with a new design of beam profile section known as Trapezoid Web Profiled (TWP) sections were proposed and the behavior characteristics of connections were studied. Some regular joints that always be used for the connections of TWP beam sections are simple joints; fin, double angel web cleat, and partial depth flexible end-plate while flush end plate and extended end plate are the most probable connection to be used for partial strength joints. Compared to simple joint, partial strength joint contributes to initial moment capacity that can make the deflection of beam becomes smaller and decrease in sagging moment of the mid span up to 30 % because of hogging moments on the connections. In addition, using partial strength connections in Trapezoid Web Profiled beam section will enhance further the moment capacity of the joints. In this paper, two full-scale sub-assemblage tests of semi-rigid beam-to-column connections are reported. Trapezoid Web Profiled sections were adopted as beams connected to Universal Column (UC) whilst Flush End Plate (FEP) and Extended End Plate (EEP) connections were used as partial strength joints. The experimental set-up and instrumentations are explained and described in detail based on Eurocode 3 and British Standard (BS) 5950 - 2000, the experimental behavior is analyzed based on the test results and the structural behavior of these semi-rigid connections and partial strength joints are discussed.

Keywords: Full scale testing, partial strength joints, semi-rigid connection, Trapezoid Web Profiled (TWP) section, pin joint, Flush End Plate (FEP), Extended End Plate (EEP), Universal Column (UC) section.

1. INTRODUCTION

Conventional building frames, either braced or un-braced, usually comprised of beams and columns assembled and arranged in 'rectangular box' manner. The connection between the beam and column is either assumed as pinned, where no moment from the beam is transferred to the column, or rigid, where full continuity of moment transfer exists. Alternatively, as has been emphasized in the Eurocode (EC) 3 (ENV 1993-1-1: 2002) [1], building frames could be designed as semi-rigid, a type of design that utilized a condition between the simple and rigid design. When incorporated into the construction of a whole frame, the term semi-continuous is often adopted to show that some continuity does exist between the beam and column. Unlike the traditional design approaches, simple and rigid, semi-rigid design requires extra information to characterize the joint to be used for the frame. The most important characteristics of a joint in semi-continuous construction are the rotational stiffness or rigidity, the strength, and the ductility (rotation capacity).

In simple design, connections are assumed not to have any moment, whilst in rigid design, connections are assumed to be able to achieve full moment continuity. However, it is recognized that the real behavior of a joint of a frame is located between the pinned and rigid joints. In designing a multi-storey braced frame for a semi-continuous construction, guides and references are based on the EC 3 specification [1] where it offers the possibility to account partial strength joints in designing multi-storey frames, BS 5950-1: 2000 specification [2] and Steel Construction Institute publications P183, P205, and P334 [3].

2. RESEARCH DEVELOPEMENTS

Many researches have been done in the field of beam to column connections behavior. The use of semi-rigid connections to connect beams to columns can be illustrated from the yielding of smaller sizes of beams [3]. This is due to the decrease of moment in the mid-span as a result from the end moment developed. The subject of semi-rigid connections has been extensively researched upon since 1980's; and noticeable works on their behavior were carried out mostly by researchers such as Nathercot, Chen, Aggarwal, Bjorhovde and Jaspart. It was in 1992 that EC3 has been published with a section specially dedicated to semi-rigid connections. M. Md Tahir in 1995 [4] has carried out a study on the economic comparison of multi-storey braced steel frame using Universal Beam (UB) sections in simple and semi-continuous construction. The result of the study indicates that by using semi-continuous design, the percentage of savings in weight of steel used ranging from 2.38% to 11.95%. Weynand *et. al* in 1998 has published a paper on the economy studies done in various countries. Those studies have shown some possible benefits from the use of the concept of semi-rigid joints. The results indicate that possible savings due to semi-rigid design can be 5% to 9% in case of braced frames [5]. Further more, the price of plates to build up the section at the moment is far cheaper than the price of rolling the steel billet as in producing Universal sections.

As proposed in this study, Trapezoid Web Profiled (TWP) section can be considered as a new innovative product of steel section. Being a hybrid section fabricated from plates, this section offers several advantages including lighter sections due to the thinner corrugated web. The significant works on its behavior (trapezoidal shaped web) was started by Sherman. D and Fisher J [6] in 1971. Elgaaly and Hamilton [7] in 1996 studied the shear and buckling capacities of the sections. Luo and Edlund. B [8] also carried out some works on the capacities and strength of the sections. Current researches on the matter revolve around the application of the sections in combination with other element.

Based on the advantages offered by the semi-rigid connections and TWP sections, this paper is presented to explain and to describe the behavior of semi-rigid and partial strength joints using Trapezoid Web Profiled section as a beam. Particular attention was taken at the effects of mid-span deflection of a long span beam and its influence on the moment resistance of the joints.

3. TESTING ARRANGEMENTS AND PROCEDURES

Full-scale tests with the test rig was designed and erected with using columns 3 m height and beam spans of 6 m. Serious cares were taken and careful consideration was made in setting the rig due to a total occupied length of over nine meters. The rig consists of channel sections pre-drilled with 22 mm holes for bolting purposes. The sections were fastened and bolted to form loading frames, which were subsequently secured to the laboratory strong floor. Figure 1 shows the arrangement of experimental work for the sub-frame tests, illustrating the scale of the works was carried out. The height of the column was proposed 3 m with the beam span of 6 m to represent the height of a storey in a typical braced steel frame. The load was applied on the 6 m beam using a hydraulic jack at the mid-span and was converted into a two-point load using a spreader of 1.8 m spreading distance. This distance was still within the standard distance of one third of the length of the beam so that a bending situation was assured.

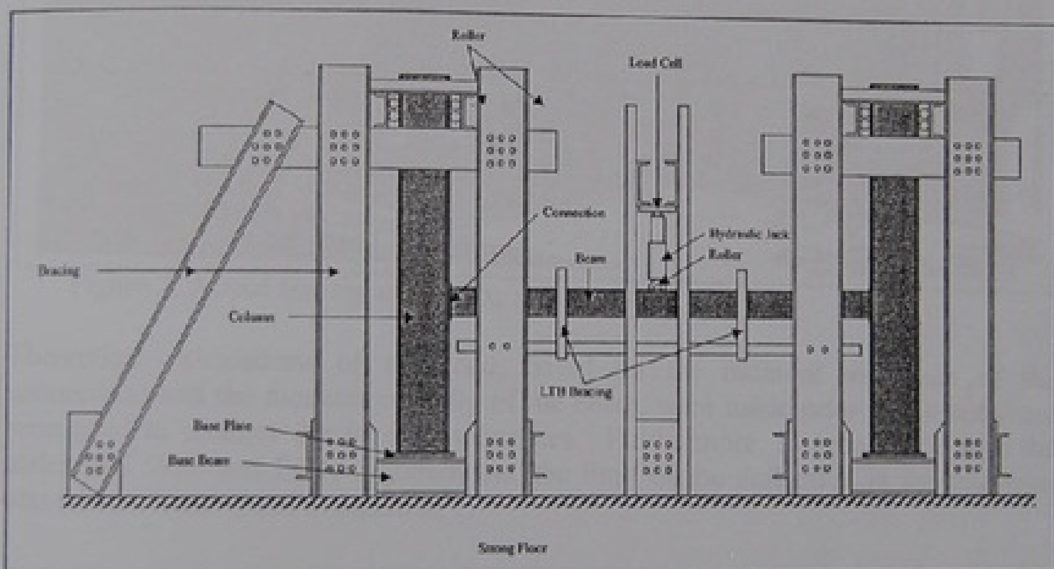


Figure 1: Testing arrangemen

In placing the test specimen for each arrangement, the column at one end was placed first by bolting the base plate to the strong base on the strong floor. Tightening of the bolts was done using a torque wrench and maintained throughout the experimental programs for consistency. Then, the 6 m beam with the welded end plates was lifted and bolted to the column's flange. Next, the other column was placed and bolted to the end plate welded on the other end of the beam. Finally, the second column was securely fastened to the strong floor by bolting the base plate of the column to the strong base. During the process, careful action was taken in making sure that all specimens were in alignment by using a bubble leveler. Figure 2 shows the actual full-scale sub-assemblage test in the rig with the instruments and equipments.

After the instrumentation system had been set-up and the specimen had been securely located in the rig, the data collection software in the computer was checked to make sure that all channels connecting to the instruments on the specimen indicated a properly working condition. The correction factors from calibration and gauge factors from manufacturer were input into the software prior to each test. A further check on the instrumentation was then carried out by loading the specimen to a load of about 20 kN to 30 kN (22 kNm to 33 kNm), and then unloading the specimen back down. In addition to making sure that the values from the instruments were received and recorded satisfactorily, this procedure was taken to enable all the components in the connection arrangement to be embedded in prior to commencing the test. The load increment at this stage was taken as 5 KN.

Figure 2: Actual test rig and testing arrangements.

Theoretical calculations of the load, based on the moment resistance of the connections and the moment capacity of the beam, were made prior to the tests, and were used to monitor the loading sequences. Furthermore, in accordance with the codes (BS 5950 and EC3) requirements, the limit of the deflection in the mid-span was taken as mentioned in equation 1:

$$\frac{L}{200}=\frac{(6000+300)}{200}=31.5mm \qquad (1)$$

The specimen was then loaded up to two-thirds of the theoretically calculated moment of resistance, and was expressed in term of point load applied for easier monitoring. Increments of 5 kN were adopted since a not so high value of moment of resistance would be expected. After reaching the two-third value, the specimen was unloaded back.

After re-initializing the instrumentation system, the specimen was loaded as described above, but the load applied was not restricted to the two-thirds value. Instead, the specimen was father loaded until there was a significantly large deflection of the beam observed. The load application was continually applied after this point but the increments were controlled by the deflection instead of the load as before. A deflection of 2 mm was adopted as a suitable increment for this stage. This procedure was continued until the specimen had reached its 'failure' condition, or until the existing of the situation that required the test to be concluded. The 'failure' condition was deemed to be reached when any of the situations occurred; *a significant reduction of the applied load* being attained or *a noteworthy increase in the deflection* of the beam being loaded.

The failure mode that might cause the above situations to occur could be analyzed as indicated in the followings;

1. The development of shear deformation on the web of the beam in the surrounding area of the connection.
2. The development of local buckling on the bottom flange of a beam in the around the connection due to the compressive action along the flange.
3. The development of local buckling on the top flange of the beam around the mid-span zone due to the compressive action along the top flange.
4. Sudden yielding of the end plate around the top flange of a beam due to the tensile action along the flange as mentioned in the SCI publication, mode 1.
5. Sudden yielding of the end plate around the top flange of a beam due to the tensile action along the flange coupled with the yielding of the critical bolts. The critical bolts are the bolts below the top flange of a beam as in the case of flush end plate connections, and above and below the top flange of a beam as in the case of extended end plate connections. This failure mode is referred to as Mode 2 in the SCI publication.
6. Sudden yielding of the critical bolts only. This failure mode is the worst ones that should be avoided and known as Mode 3 in the SCI publication.
7. The deformation of the web in the vicinity of the contra flexural points of negative and positive moment due to trapezoid form of web.

Due to the longer beam used in these sub-assemblage frame tests, the following failure modes could be occurred.

1. The development of local buckling on the bottom flange of a beam in the around the connection due to the compressive action along the flange.
2. Sudden yielding of the end plate around the top flange of a beam due to the tensile action along the flange coupled with the yielding of the critical bolts, mode2.

3.1 Specimens

There was no specific requirement available with the length of a TWP beam to be used as the specimen. However, in order to make certain that the mid-span deflection could have significant impacts on the connections at both ends, the length of six meter was chosen besides the length represents a standard span in general practice of the construction of multi-storey braced frame. As mentioned earlier, there were two arrangements of full-scale sub-assemblage tests conducted in this study. The first arrangement was utilizing a flush end plate connection, and the other arrangement was utilizing an extended end plate connections. The beam size, end plate size and the diameter of bolts for both tests were kept the same for the purpose of comparison. Table 1 shows the test matrix for the two arrangements in these sub-assemblage frame tests.

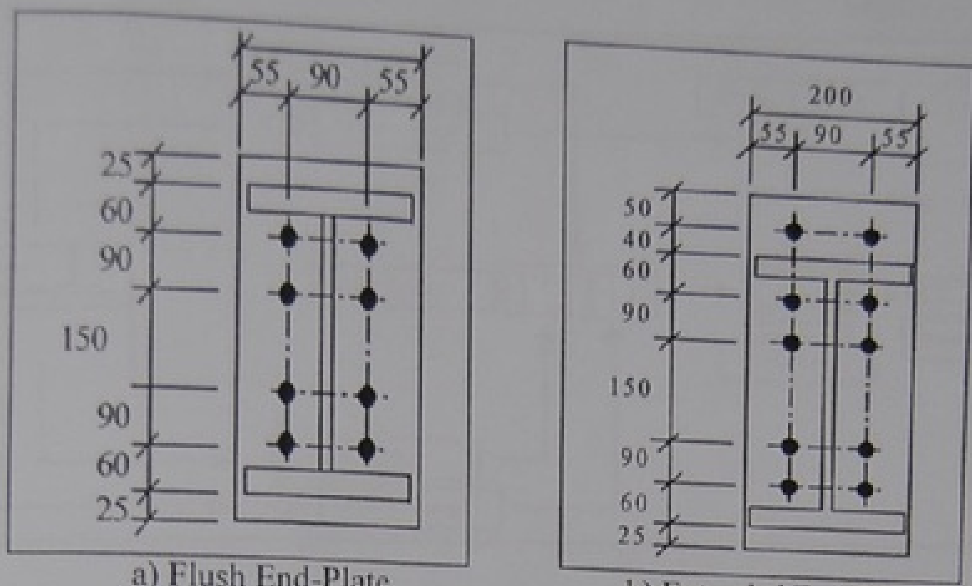
Table 1: Test matrix for the sub-assemblage frame tests

Model Name	Beam Size TWP	Column Size UC	Connection Types	Bolt Row (T-B)	End Plate	Bolt
FS-E3R20P1	450x160x50.2/12/4	305x305x118	EEP	3(6-4)	200x12	M20
FS-F2R20P1			FEP	2(4-4)	200x12	M20

The information obtained from isolated tests of both types of connections was used as the control behavior. Subsequently, the results obtained from these full-scale tests could be compared and correlated for thorough understanding. Careful consideration was taken in choosing the arrangements since the experiment could only be managed to be conducted one each. The selection and design of both arrangements were based on the following aspects;

1) The length of the beam was taken to be six meters. This length was believed suitable to sustain and provide a moment of resistance to the joint as predicted without jeopardizing the mid-span deflection limit of L/200 to L/360 (as specified in SCI).
2) The section chosen for the beam was TWP 450 x 160 x 50.2/14/4. The moment capacity M_{cx} of this section was calculated to be 375.6 kNm of which brought about a mid-span point load approximately 341.4 kN over a 0.7L free span (sagging region). This load was considered reasonable as the limit of the applied load of the beam.
3) The geometry of the flush and extended end plate connections selected was deemed suitable to be connected to the width and depth of the beam section chosen. In addition, the top and bottom rows of bolts of the connections were deemed adequate for providing the resistances (especially the moment resistance) required.

In both tests, the beams were supported from lateral torsional buckling by using an intermediate bracing placed at every quarter point along the beam. The size of the column is UC 305 x 305 x 118 kg / m. The geometry of the flush end plate connection and the extended end plate connection are illustrated in figure 3 (a) and (b) and designated as FS-F2R20P1 and FS-E2R20P1 respectively as mentioned in Table 1.



a) Flush End-Plate.　　b) Extended End-Plate

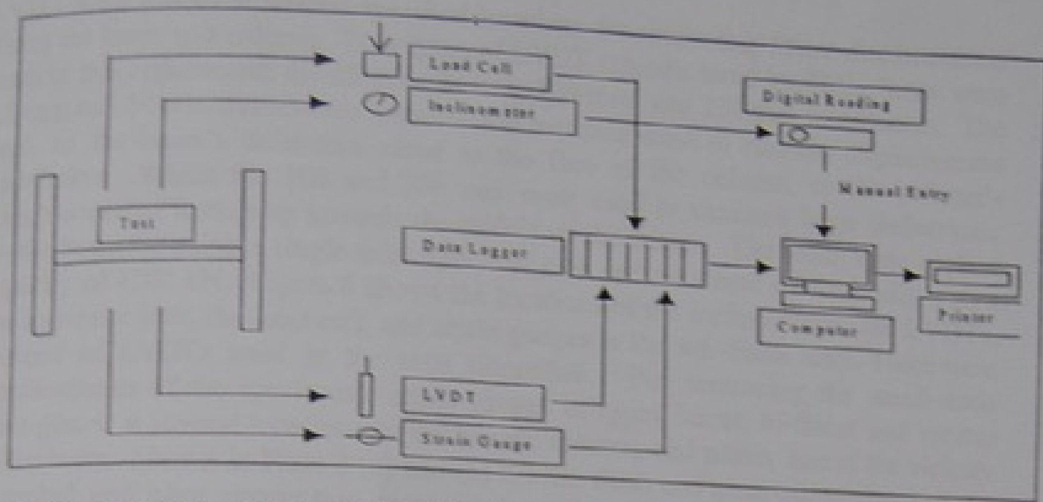
Figure 3: Geometry of connections: (a) Flush end plate and (b) Extended end plate

3.2 Instrumentation System

The instrumentation system adopted for the experimental investigation was designed to acquire all the necessary measurements and important data that would be required to determine the behavior characteristics of the connections. Table 2 lists the instruments that were used in the tests along with the respective amounts and nearest units. The locations and arrangement of the instrumentation system for data acquisition are shown in the schematic representation of figure 4.

Table 2: Details of Instruments used in the sub-frame tests

Name	Type	Amount	Accuracy	Remarks
1. Load Cell	- TML 1MN	1 unit	0.01 kN	Can measure up to 1000 kN
2. Linear Displacement Transducer	- 50 mm -100 mm -200 mm	6 units 6 units 1 unit	0.01 mm	
3. Rotational Inclinometer	- Digital	2 units	0.01°	Converted to miliradian
4. Strain Gauges	- FRA - FCA - FLA	3 pieces 5/6 pieces 6 pieces	0.01 με	- 3 directions - 2 directions - 1 direction



With the exception of the rotational inclinometer, all of the other devices were connected directly to and in turn monitored by the 'heart' of the instrumentation system named Kyowa Data Logger. Capable of monitoring up to 50 channels, the data logger was controlled through a desktop computer. Readings from the load cell, strain gauges and Low Voltage Displacement Transducers (LVDT) were recorded via the data logger on to the hard disk of the computer.

However, the rotational values of the beam and column were recorded manually from the digital display unit of the Lucas Rotational Inclinometers. This is because the instrument does not have the capability of connecting to the data logger. At the time of experiment, the new inclinometers ordered with that capability have not been delivered yet. One inclinometer was mounted midway at the web of the beams at a distance of about 100 mm from the face of the column flange. This inclinometer provided the rotational values of the beam, ϕ_b, upon loading. The other inclinometer was placed at the centre of the column panel shear thus provided the rotational values of the column, ϕ_c. The overall rotation of the joint, ϕ, was then taken as the difference between ϕ_b and ϕ_c. The default units for the measured rotation of the inclinometers were degree, and therefore, the values had to be converted to the standard units of m-radians using the conversion factor;

$$\phi = \phi_b - \phi_c \qquad (2)$$

$$1^\circ = 17.46032\ mrad \qquad (3)$$

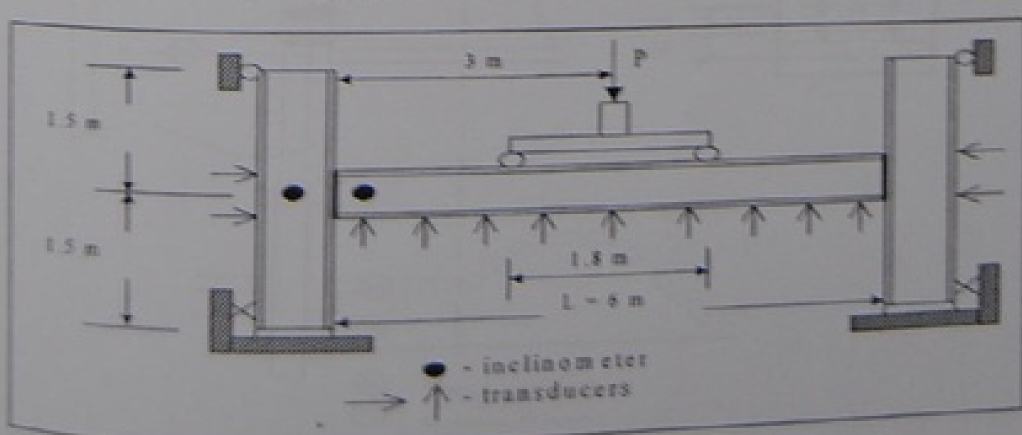


Figure 5: Locations of equipment for data acquisition

LVDT were placed at several specified locations for measuring linear displacements along the beam and column. Four types of LVDT manufactured by TML, Japan were used in this experiment: the 25 mm, 50 mm, 100 mm, and 200 mm transducers. The 25 mm and 50 mm transducers were used to measure small to medium displacements such as the beam's deflection close to the face of the column, or the column's translation. Whilst the 100 and 200 mm were used to measure large deflections, which occurred further up towards the middle of the beams. The loads were applied on the beam by using a single inlet hydraulic jack and measured by load cells with a capacity of 1000 kN. Figure 5 shows the locations of the inclinometers, the LVDTs, the hydraulic jack, the load cell, and strain gauges in the sub-frame tests. There were thirteen of LVDTs used in the tests altogether. For measuring the small-scale displacements of the specimens, strain gauges of types linear, bi-linear and rosette were placed at several locations on the beams, column, end plates, and at the vicinity of the bolts. Shown in Plate 2 (a) and (b) are the flush end plate connection and the extended end plate connection respectively with strain gauges and inclinometers attached. Figure 6 show the drawing location of bolts and flush end-plate that are used in the experiments.

3.3 Parameters

In order to make a significant comparison, the type of connections selected for the tests were the flush end plate and the extended end plate. The geometry of the two connections was identical except that there were two additional bolts on the extended part of the extended end plate connection.

The full-scale tests were conducted using a six meter length beams so as to observe the influences of the mid-span deflection on the behavior of the connections which is influences by the moment resistance, the rotational rigidity, and the ductility.

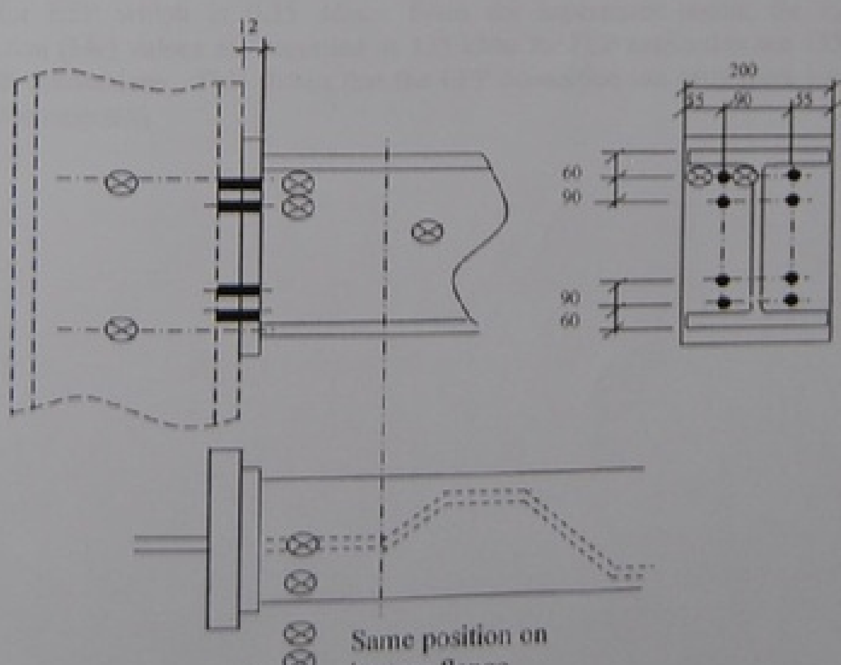


Figure 6: Locations and types of strain gauges

4. DISCUSSION OF THE RESULTS

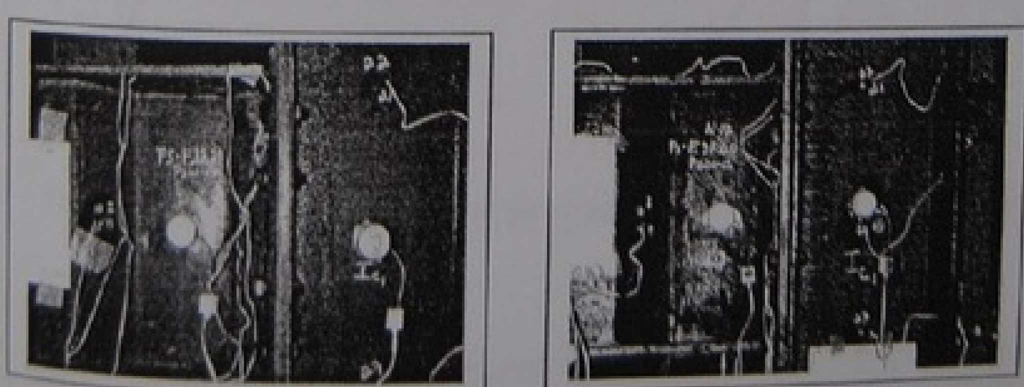
The results of the experiments were focused on the deflection at mid-span and the loads applied to the beam. The maximum results of deflection are shown in Table 3. For FEP connection, the maximum deflection recorded from experiment at mid-span was 23 mm at applied maximum load of 230 kN, whereas for EEP connection, the maximum deflection at mid-span was recorded as 31 mm at applied load of 380 kN. The ratio of applied load versus deflection for FEP is 10 whereas for EEP is 12.2. The result showed that the EEP connection which known to be stiffer than the FEP connection has contributed to a lesser deflection than the FEP connection as expected [8]. The result of the experiment was also focused on the moment resistance and rotation of the connections that associated with the mid-span deflection and applied load. Results of the two full-scale tests are shown in Table 3, whereas figure 7 shows the connection of FEP and EEP used in the specimens. The results in Table 3 are recorded from the graph of load versus deflection of mid-span as shown in figure 8 (a) and (b) and the graph of load versus rotation of the connection as shown in figure 9 (a) and (b) where the graph was plotted up to failure of the specimens.

The failures of the full-scale testing specimens were due to the buckling on the bottom flange of the beam around the connection as discussed in this paper. The moment resistance of the FEP and EEP connections has been recorded earlier in other tests [7][8]. These tests were conducted for an isolated beam-to-column connection where the purpose of the tests is to determine the moment resistance and rotation of the connections. The results of these tests are shown in Table 3 and the graphs of moment versus rotation are shown in figure 10 (a) and (b) for FEP and EEP connection respectively. The connections possessed a ductility characteristic with a rotation capacity of 25 mrad for FEP and 30 mrad for EEP. The minimum moment connection capacities, on the other hand, are 79.7 kNm for FEP which is 0.21 Mcx and 131.7 kNm for EEP which is 0.35 Mcx. From the experiment results, the moment connection (Mc) values are recorded as 135 kNm for FEP connection and 253 kNm for EEP connections. This shows that the EEP connection can carry more load than the FEP connection.

Table 3: Test Result of Full-Scale Sub-Assemblage Trapezoid Web Profiled (TWP) beam to Universal Column Connection

Reference	FS-F2R20P1	FS-E2R20P1
Type of Connection	FEP	EEP
Universal Column (UC)	305 x 305 x 118	305 x 305 x 118
Trapezoid Web Profiled Beam	450 x 160 x 50.2	450 x 160 x 50.2
Moment Capacity (kNm)	375.6	375.6
Beam Load Capacity (kN)	341.4	341.4
Defection Capacity (mm)	31.5	31.5
Rotation Capacity (mrad)	25	30
Moment Conn minimum (kNm)	79.7	131.7
Rotation at Failure Exp (mrad)	5.80	6.10
Maximum Load Exp (kN)	230	380
Deflection Maximum Exp (mm)	23	31
Ratio Load to Deflection	10	12.3
Spreader Load to Column (m)	2.2	2.2
Moment Connection Exp (kNm)	135	253
Moment at mid-span (kNm)	(379.5-135) = 244.5	(627 -253) = 374
Max Strain* (ue)	4700	11000
Failure Mode	End-plate buckling on top flange at connection	End-plate buckling on top flange at connection and yielding of end-plate

* At one location only as checking place.



(a) Flush End-Plate (b) Extended End-Plate

Figure 7: a) Flush end-plate connection and b) Extended end-plate connection with strain gauges and inclinometers installations.

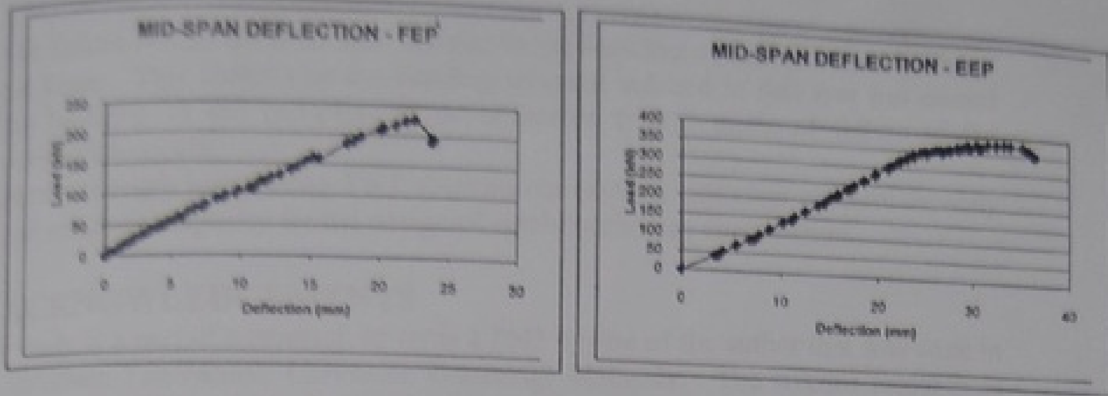


Figure 8 (a) and (b): The Graph of Load versus deflection for FEP and EEP

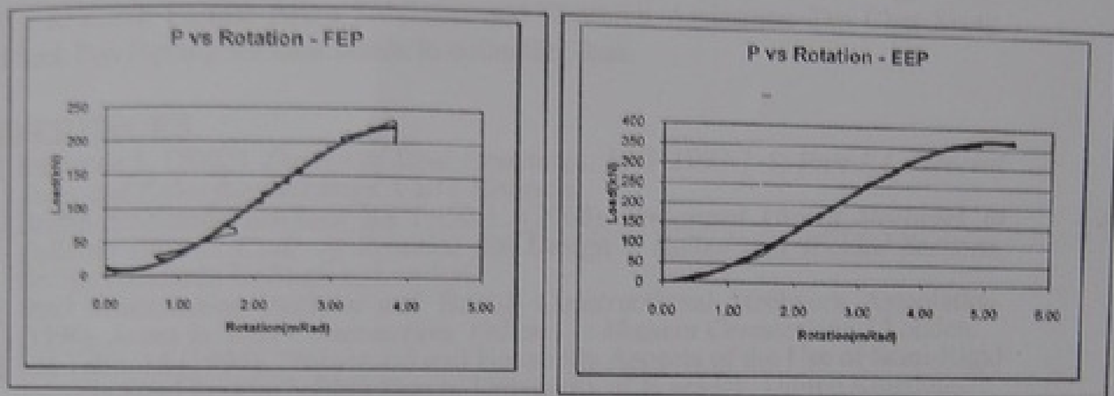


Figure 9 (a) and (b): The Graph of Load versus rotation for FEP and EEP

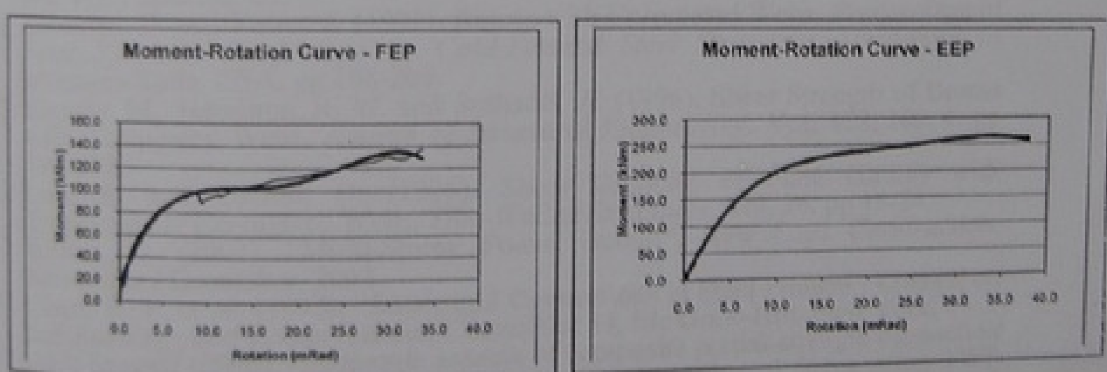


Figure 10 (a) and (b): The Graph of Moment versus rotation for FEP and EEP

5. CONCLUSIONS

In this study, it can be concluded that it is possible to determine the moment capacity of extended end-plate connection is more than flush end plate connection with the conditions as mentioned in the followings:

1. The moment capacity of the extended end plate is almost double the moment capacity of the flush end plate connection because the stiffness of extended end-plate connection is more than the stiffness of flush end-plate connection.
2. The failure of end-plate connection has been reached the deflection limit before the failure of other predicted parts.

3. The failure modes for both tests are due to the buckling of end-plate at top flange the beam. This shows that the bending moment induced in that area has caused the buckling of the end-plate to occur before typical mode of failure of the connection occurs.
4. Partial strength joints using TWP sections can provide sufficient moment capacity and rotational capacity for the design of semi-continuous construction.

6. ACKNOWLEDGEMENTS

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