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# Performance of partial strength connection connected by thick plate between column flanges

Mahmood M. Tahir*[1], Irwan Juki[2], Mohd Y. Ishak[1], Shahrin Mohammad[1], Abdullah Z. Awang[1] and Roger Plank[3a]

*[1]UTM Construction Research Centre (UTM CRC), Faculty of Civil Engineering, Universiti Teknologi Malaysia, 81310 Johor Bahru, Johor, Malaysia*
*[2]Faculty of Civil Engineering, Universiti Tun Hussein Onn, Beg Berkunci 101, Parit Raja, 86400 Batu Pahat, Johor, Malaysia*
*[3]University of Sheffield, Western Bank, Sheffield, South Yorkshire S10 2TN, UK*

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**Abstract.** Traditional beam connections to the minor axis of a column have relatively low strength and stiffness. A modified detail, using a plate welded between the toes of the column flange - referred to as a toe plate connection - is examined in this paper. The results of an experimental investigation for both flush and extended end-plate connections connected to a 25 mm thick end-plate are presented. The tests are complemented by finite element modelling which compares very well with the test observations. The results show a significant increase in both moment resistance and initial stiffness for this connection detail compared with connections made directly to the column web. This offers the prospect of more optimal solutions taking advantage of partial strength frame design for the minor axis as well as major axis.

**Keywords:** flush end-plate connection; extended end-plate connection; minor axis; partial strength; finite element analysis

## 1. Introduction

Steel frames for building are traditionally designed on the basis that beam-to-column joints are either pinned or rigid. However, the actual joint usually falls between these extremes, giving what is generally termed semi-rigid behaviour. A joint usually has a moment resistance less than that of the connected beam; such joints are termed ‘partial-strength’ by Eurocode 3 (BS EN 1993-1-1 2005), which allows designers to include the actual joint strength in an equivalent plastic hinge analysis of the frame. However, even relatively substantial connection details such as flush and extended end-plates may have a low moment resistance when connected to the column web - i.e., as a minor-axis connection. This is because the web of the column is generally thin compared with the beam end-plate, limiting joint strength and also leading to large deformations (Mahmood *et al.* 2011). A common form of such joints is to connect beam end-plates to the web of the column but despite this, both experimental and theoretical investigation of such joints has been limited for

∗Corresponding author, Professor, E-mail: mahmoodmdtahir@gmail.com
[a]Emeritus Professor


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practical application for moment connection.

There have been many studies on beams connecting to the major axis of columns, but relatively little has been published for minor axis connections. Previous studies on the behaviour of minor axis joints were reported by Rentschler *et al.* (1982), Chen and Lui (1988), Janss *et al.* (1987), Gomes *et al*. (1996), Davison *et al*. (1987 ), Kim (1988) and Mahmood *et al.* (2011). The studies provide clear evidence of the crucial contribution of the column web panel out-of-plane deformation to the joint response. A large deformation and large displacement of the column web are the typical behaviour of this type of connection. A series of 22 cantilever tests was reported by Kim (1988) who examined flush end-plate connections with different geometries. The results showed that the column web suffered large deformations, and that the moment resistance of the connection was highly dependent on the thickness of the column web. Mahmood *et al.* (2011) tested five minor axis beam-to-column flush end-plate connections in a cantilever arrangement, with beams framing into the column web, and confirmed that the connections developed large deformation and rotation. Cabrero and Bayo (2007) carried out experimental and theoretical studies to investigate the extended end-plate connection in both major and minor column axes. Three-dimensional joint configuration was proposed where the minor axis joints are not bolted to the web of column, but to an additional plate welded to the column flanges. This arrangement prevents the minor axis beams to get into the column and the joints to hinder each other. In addition, this welded plate could act as a stiffener for the joint in the major axis. It was concluded that the strength and stiffness of the joints depend on the end-plate thickness. Also, it was found that Eurocode method need to be improved to predict the minor axis and three-dimensional joints. Thus, new models were proposed.

Other test results have been conducted by Neves *et al.* (2002), while de Lima *et al*. (2002) reported a series of tests using angle cleats in various configurations. Mechanical models based on component method and reflection photoelasticity techniques are proposed to predict the connection’s structural respond (de Lima *et al.* 2002, Luciano 2009). The results of the tests were then modelled in finite element model (FEM) where the column was stiffened with various thicknesses. The FEM results agreed well with the equation proposed by (de Lima *et al*. 2002) using a polynomial approximation method.

A number of researchers have used finite element modelling to predict the behaviour of the connection Mohammed and Archibald (1996), Choi and Chung (1996), Bursi and Jaspart (1998), Yorgun *et al.* 2004). Mohammed and Archibald (1996) used an inelastic finite element model to evaluate stiffness and strength characteristics of steel bolted end-plate connections. Choi and Chung (1996) had employed finite element analysis to investigate the behaviour of end-plate connections. Bursi and Jaspart (1998) have conducted a series of numerical simulations of steel bolted connections with finite element models. Yorgun *et al.* (2004) developed a series of finite element models for double channel beam-to-column connections subjected to in-plane bending moment and shear. The simulation was carried out using ANSYS 5.5.1 version and the results were compared with the experimental tests conducted at the Istanbul Technical University. The comparisons were in good agreement with the difference between the experimental and finite element curves was about 6% in non-linear phrase of curves.

All of the above works have been concerned with beams connecting directly to the web of the column. An alternative beam-to-column (minor axis) joint detail is to weld a plate – referred to herein as a toe plate - between the toes of the column flanges. This provides the face to which the beam end plate can connect, and potentially offers the benefit of enhanced joint stiffness and strength. In this paper the term toe plate connection is used to describe such a detail, which may offer the added benefit of easier erection, because it avoids the problems caused by the restricted space between the column flanges characteristic of traditional minor axis.

This paper presents the results of an experimental study on connections in which the minor axis connection uses a 25mm thick toe plate welded between column flanges. Both flush and extended end plate details are considered and the results are compared with previously published work on minor axis connections attached directly to the column web. Eurocode 3: Part 1-8 (2005) does not cover the empirical design for such connection details. Designers wishing to adopt such details will therefore need some guidance based on research data. Whilst the experimental results presented in this paper will contribute to this, such testing is expensive. A finite element model is therefore included and validated against the test results, providing a basis for generating a wider range of data without the need for expensive testing.

## 2. Test procedures

The connection details used in this study were adapted from those proposed by the Steel Construction Institute for major axis connections (SCI 1995). A total of ten tests were conducted on partial strength toe plate connections, of which five were flush end-plate connections and five were extended end-plate connections. In all cases the end plates were connected to 25mm thick toe plates welded between the column flanges as shown in Fig. 1. Two column sizes, four beams sizes and two thicknesses of beam end plates – 12 mm and 15 mm – were used, and details of the test configurations are listed in Table 1. M20 and M24 Grade 8.8 bolts were used for the 12mm and 15mm thick end-plate connections respectively. For the flush end plate connections, four bolts were used, arranged in two rows, the top row in tension and the bottom row in compression. The extended end-plate connections used an additional row of bolts in tension, as shown in Fig. 1. All steelwork, including the column and beam sections and the endplates, were grade S275, supplied by Perwaja Steel Section, Malaysia (PSS).

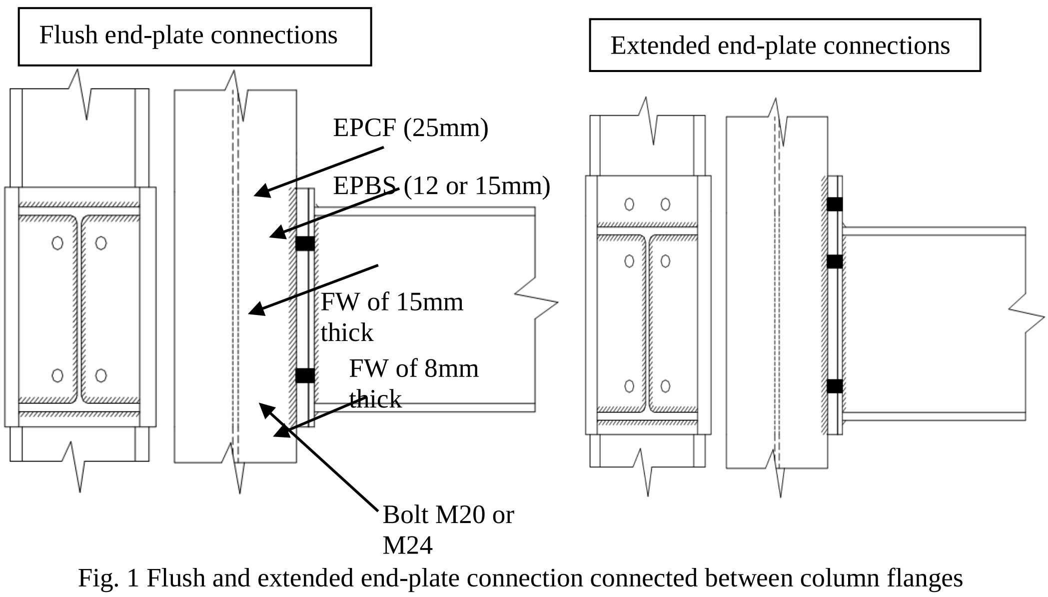



Fig. 1 Flush and extended end-plate connection connected between column flanges

Table 1 Test specimens with various parameters

| Test No. | Symbol | Column | Beam | End-plate thickness EPCF (mm) | Width of end plate EPCF (mm) | Diameter of bolt $D_b$ (mm) |
|---|---|---|---|---|---|---|
| 1 | FEP 6 | HB 300×300×83.5 | HB 450×200×74.9 | 12 | 200 | 20 |
| 2 | FEP 7 | HB 300×300×83.5 | HB 450×200×74.9 | 15 | 200 | 24 |
| 3 | FEP 8 | HB 300×300×83.5 | HB 500×200×102 | 15 | 200 | 24 |
| 4 | FEP 9 | HB 250×250×63.8 | HB 400×200×65.4 | 12 | 200 | 20 |
| 5 | FEP 10 | HB 250×250×63.8 | HB 400×200×65.4 | 15 | 200 | 24 |
| 6 | EEP 11 | HB 300×300×83.5 | HB 450×200×74.9 | 12 | 200 | 20 |
| 7 | EEP 12 | HB 300×300×83.5 | HB 450×200×74.9 | 15 | 200 | 24 |
| 8 | EEP 13 | HB 300×300×83.5 | HB 500×200×102 | 15 | 200 | 24 |
| 9 | EEP 14 | HB 250×250×63.8 | HB 400×200×65.4 | 12 | 200 | 20 |
| 10 | EEP 15 | HB 250×250×63.8 | HB 400×200×65.4 | 15 | 200 | 24 |

*Nomenclature: FEP represents for Flush End-Plate connection and EEP represents Extended End-Plate connection.

For these full-scale tests, a test rig was specially designed and erected to accommodate a column height of 3m and a cantilever beam span of 1.5 m. The test rig was constructed from twin channel sections to act as a guide for the test specimens with a separate loading frame as shown in Fig. 2. The column was restrained against rotation at both ends and the beam was restrained from lateral movement by bracing it laterally against the loading frame. Vertical loading was applied to the beams at a distance of 1.3 m from the face of the column web using a hydraulic jack. This distance was taken to represent the approximate length of the hogging moment region in the beam in frames with partial strength connection. Inclinometers were positioned at the centre line (mid-depth) of the beam and at the centre of the column as shown in Fig. 2. The rotation of the connection is defined as the difference between the rotations of the centre line of the beam to the centre line of the column measured by the inclinometer.

After the instrumentation system had been set-up and the specimen securely located in the test rig, data collection software in the computer is used to check the reading of all connected channels to the instruments on the specimen. Correction factors from calibration and gauge factors from manufacturer are used in the software prior to each test. The specimen is then loaded up to one-third of the predicted value. The reading of load is taken as point load applied for easier monitoring. After reaching the one-third of the predicted load capacity, the specimen is unloaded back and re-initialised. This procedure was carried out so as to enable the specimen to be in the state of equilibrium prior to the actual test. An increment of about 5 kN is adopted so that a uniform data and gradual failure of the specimen can be monitored. After re-initialising the instrumentation system, the specimen is loaded again; however, the applied load was not restricted to the one-third capacity. Instead, the specimen was further loaded until substantial deflection of the beam could be observed. At this point, the loading sequence is controlled by the increment of the deflection as a small increment of load has resulted to substantial increase in the deflection. Therefore, the load is continuously applied but each increment of the load interval is limited to the deflection of 2 mm of the beam instead. This procedure is continued until the specimen had reached its failure condition. The failure condition is categorized as an abrupt or significantly large reduction in the applied load had reached or when a large rotation of the connection due to

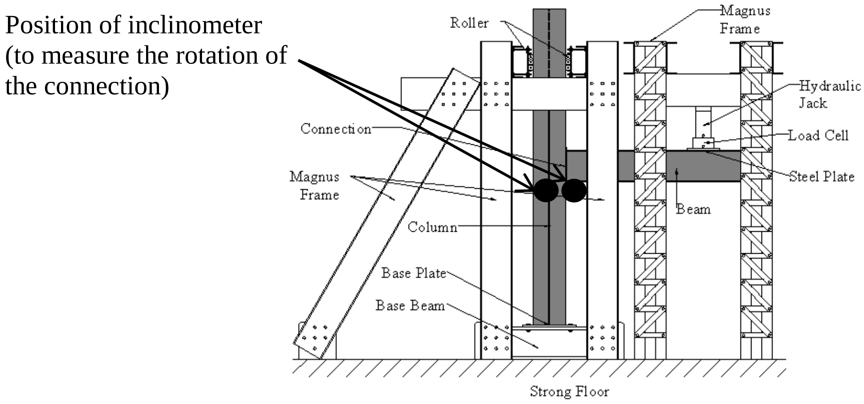
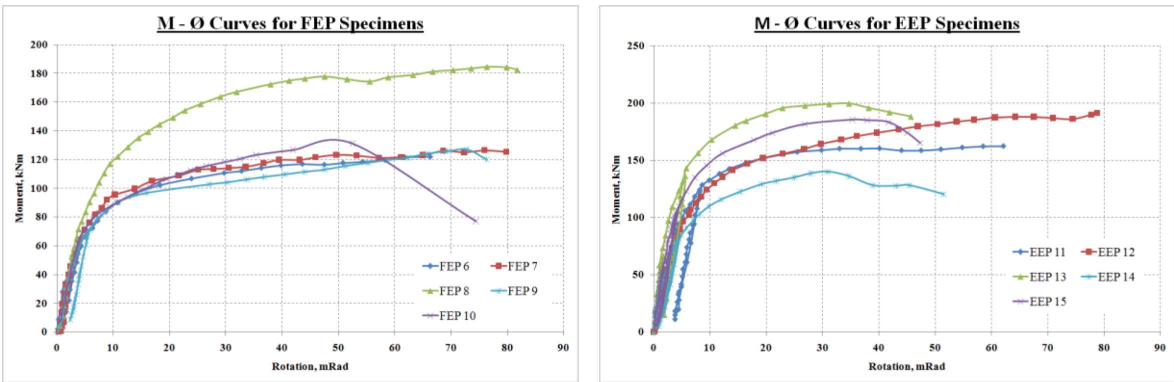



Fig. 2 Laboratory testing arrangement




(a) FEP specimens (b) EEP specimens

Fig. 3 Moment rotation curves for FEP and EEP specimens

deformation of the tested specimen had occurred. For each loading, a set of reading is taken for deflections, rotations, and applied load.

## 3. Experimental test results

The test results are summarised as moment-rotation curves (M-Φ) for both the flush and extended end-plate specimens and these are shown in Figs. 3(a)-(b) respectively. The rotation of the connection was measured as the difference between the rotation of the column and the rotation of the beam, which was largely due to the deformation of the end-plate. The results clearly show that the proposed connections developed reasonable partial strength with high moment resistance. The initial behaviour was approximately linear with high stiffness; at a certain point the deflections then started to increase a little more rapidly and the response became non-linear, and increasingly less stiff. At higher loading, the curves form a plateau, effectively defining the moment resistance of the connection. The curves are also characterised by significant plastic deformation, demonstrating good ductility of the connection. Details of the experimental tests

Table 2 Experimental tests results

| Test specimen | Moment ($M_R$) at $S_{j,initial}$ kNm | Rotation $(\Phi)_R$ at $S_{j,initial}$ mrad | Initial Stiffness, $S_{j,initial}=(M_R/\Phi_R)_{initial}$ kNm/mrad | Moment resistance ($M_{Max}$) kNm | Rotation $(\Phi)_{rotation}$ at $M_{Max,}$ mrad |
|---|---|---|---|---|---|
| Flush end-plate | | | | | |
| FEP 6 | 100.0 | 5.6 | 17.9 | 122.1 | 66.3 |
| FEP 7 | 100.0 | 5.3 | 18.7 | 126.5 | 75.9 |
| FEP 8 | 100.0 | 5.3 | 19.0 | 184.5 | 76.3 |
| FEP 9 | 100.0 | 5.2 | 19.2 | 128.1 | 52.8 |
| FEP 10 | 100.0 | 5.5 | 18.2 | 135.3 | 72.8 |
| Extended end-plate | | | | | |
| EEP 11 | 100.0 | 4.3 | 23.3 | 162.4 | 62.1 |
| EEP 12 | 100.0 | 4.4 | 22.7 | 184.4 | 67.7 |
| EEP 13 | 100.0 | 3.2 | 31.3 | 185.5 | 34.7 |
| EEP 14 | 100.0 | 3.7 | 27.0 | 140.1 | 30.9 |
| EEP 15 | 100.0 | 3.0 | 33.3 | 199.7 | 36.3 |

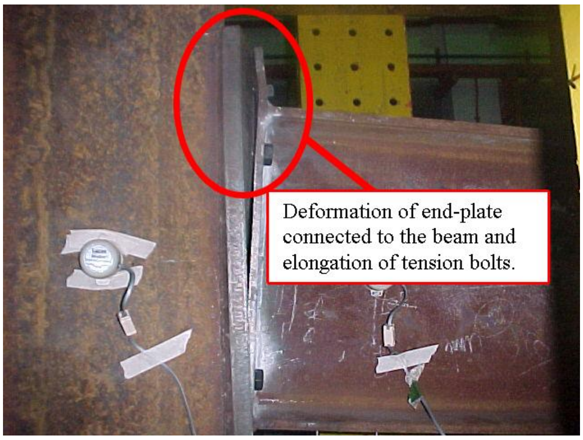



Fig. 4 Deformation of the beam end-plate

results are tabulated in Table 2. The initial stiffness of the connection is derived by dividing the moment ($M_R$) at $S_{j,initial}$ with rotation at $S_{j,initial}$.

The connection detail comprises two plates, namely the end-plate connected to the beam section and the toe-plate connected between the toes of the column section. The beam end-plates used in the tests were either 12 mm or 15mm thick, whilst the toe-plate was substantially thicker at 25 mm as shown in Fig. 1. Which of the two plates contributes most to the deformation of the connection depends on their relative strength and stiffness, and the details of the forces on each. It is therefore not simply that the thinner of two, namely the beam end-plate in this case, will be the critical component. However, as can be seen from Fig. 4, in all tests it was the beam end plate which deformed most. The deformation of the beam's end plate was more evident in the case of the flush end-plate detail than for the extended end-plate connection, in which the tension force from the bolts and the balancing compression force are lower.  The toe-plate also suffered some deformation but this was consistently small compared with that of the beam end-plate. In all tested

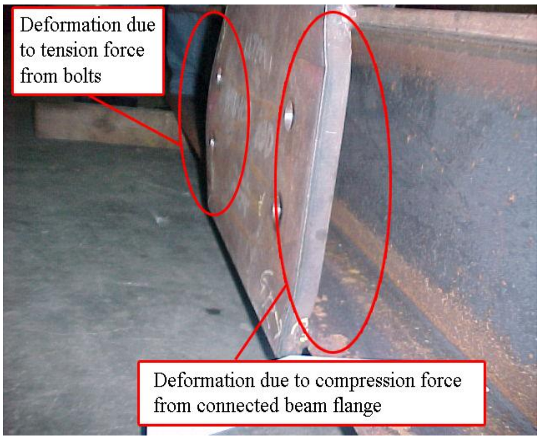



Fig. 5 Deformation of the toe plate

Table 3 Comparison of specimens FEP with EEP connections

| Specimen | Connection's End-plate Thickness (mm) | parameter Size of bolts (mm) | Moment resistance from experimental tests, $M_{Max}$ (kNm) | Percentage difference % | Initial Stiffness, $S_{j,ini}$ = $(M_{j,Ed}/\Phi)_{ini}$ (kNm/mrad) | Percentage difference % |
|---|---|---|---|---|---|---|
| FEP 6 | 12 | M20 | 122.1 | | 17.9 | |
| vs | | | | 33.0% | | 30.2% |
| EEP 11 | 12 | M20 | 162.5 | | 23.3 | |
| FEP 7 | 15 | M24 | 126.5 | | 18.7 | |
| vs | | | | 45.8% | | 20.3% |
| EEP 12 | 15 | M24 | 184.4 | | 22.7 | |
| FEP 9 | 12 | M20 | 128.1 | | 19.2 | |
| vs | | | | 9.4% | | 40.1% |
| EEP 14 | 12 | M20 | 140.1 | | 27.0 | |
| FEP 8 | 15 | M24 | 184.5 | | 19.0 | |
| vs | | | | 0.5% | | 40.1% |
| EEP 13 | 15 | M24 | 185,5 | | 31.3 | |
| FEP 10 | 15 | M24 | 135.3 | | 18.2 | |
| vs | | | | 47.6% | | 83.0% |
| EEP 15 | 15 | M24 | 199.7 | | 33.3 | |

*Connections with the same end-plate thickness, size of bolts, size of beam and size of column

specimens, a gap developed between the beam end-plate and the thicker toe-plate, indicating significant elongation of the tension bolts (see Fig. 4).

The welds were designed to be stronger than the applied tension force of the bolts so as to avoid any failure in this part of the connection. As well as providing specific data for the particular configurations tested, the results also show some general trends. As might be expected, thicker end plates resulted in higher moment resistance, although this was relatively small (approximately 5%) for the connections using FEPs. In contrast the use of thicker end plates for the EEP connections resulted in an increased moment resistance of up to 40%. The stiffness of the connection was only affected to a much smaller degree.

The connection stiffness was surprisingly consistent across all the FEP tests, but showed a little more variation for the connections using EEPs, indicating increased stiffness for deeper beam and smaller column sections. The test results suggest that column size may have relatively little effect on the connection characteristics, which may be associated with the small level of deformation observed in the columns section. However, it should be noted that the range of column sizes considered was very narrow.

The response of the flush (FEP) and extended (EEP) end-plate connections is compared in Table 3. As expected this shows that in general extended end-plate connections have significantly higher moment resistance and initial stiffness than the equivalent flush end-plate. However, for FEP 8 versus EEP 13 specimens, the percentage difference of moment resistance is very close (0.5%); as the increased in the tension zone forces due to the use of M24 bolts and 15 mm thick end-plate together with deep beam (500 mm) for both specimens have led to the deformation of the toe plate (25 mm thick) (see Fig. 5) instead of the end-plate (15 mm thick) connected to the beam.

## 4. Connection between column flanges vs connection on column web

Beam connections to the minor axis of a column commonly take the form of a beam end plate attached directly to the web of the column as shown in Fig. 6. These can be treated as partial strength semi-rigid connections although they tend to have less strength and stiffness than the equivalent major-axis connection. Mahmood *et al*. (2011) recently conducted a study to examine the relevant characteristics of such connections with similar proportions to those currently under investigation. This included full scale testing of five Flush End-Plate beam-to-column web connections and the details of the specimens are summarised in Table 4. The test results are listed in Table 5, recording the moment resistance and initial stiffness for each test. It should be noted that in these tests the deformation of the column web was very large so the moment resistance of the connection ($M_{Rd}$) was taken from the $M$-$\Phi$ curve as the moment at a rotation of 50mrad as suggested by other researchers (Chung and Lau 1999, Wong and Chung 2002).

Whilst these details are not directly equivalent to those currently reported, there are sufficient similarities to enable some useful comparisons, and these are summarised in Table 6.

The most directly comparable pair of tests is FEP 4 and FEP 6 which uses the same section sizes (HB300×300×83.5 and HB450×200×74.9 for the column and beam respectively. All principal characteristics of the two connections are therefore identical with the exception that in FEP4 the beam is connected to the column web which has a thickness of 12 mm, whilst FEP6 uses a 25 mm thick toe-plate. This results in an increased moment resistance from 85.5 kNm to 122.1 kNm, a difference of 42.8%.

Although tests on FEP2 and FEP9 used different column sizes (HB300×300×83.5 and HB250×250×63.5 respectively) other details were the same, including the size of beam (HB400×200×65.4). A comparison of the results shows that the moment resistance has increased from 73.0 kNm to 128.1 kNm, a percentage difference of 75.5%. Again the principal difference in the detail is that in FEP2 the beam is connected to a 12 mm thick web whilst FEP9 uses a 25 mm toe-plate.

Similar comparisons can be made between FEP 5 and FEP 8 which adopt the same size of beam (HB 500×200×102) but the sizes of column and beam end plate are different - HB 350×350×104.6 with a 12mm thick end plate for FEP5 and HB300×300×83.5 with a 15 mm end plate for FEP8. In both cases the larger M24 bolts are used. The net effect of this is a very significant increase in moment resistance of the connection of 207%.

A comparison of the initial connection stiffness shows an even more marked increase for the toe-plate connection in all cases. Although these comparisons are not precise because of variations in the details of the respective specimens, they give a very good indication that the toe plate detail can provide a much stronger, stiffer partial strength, semi-rigid joint than the traditional minor axis column connection. This may be largely due to the increased thickness of the toe-plate compared with the thickness of the column web, but even so it provides the designer with an opportunity to take advantage of greater joint strength if appropriate.

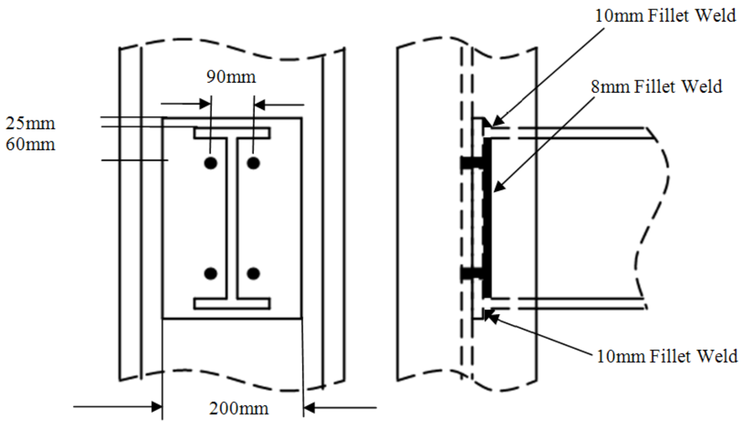



Fig. 6 Flush end-plate connection connected to column web

Table 4 Test specimens for flush end-plate connected to column web (EPCW)

| Test No. | Symbol | Column | Beam | End-plate Thickness (mm) | End Plate Width (mm) | Diameter of bolt (mm) |
|---|---|---|---|---|---|---|
| 1 | FEP 1 | HB350×350×104.6 | HB400×200×65.4 | 12 | 200 | 20 |
| 2 | FEP 2 | HB300×300×83.5 | HB400×200×65.4 | 12 | 200 | 20 |
| 3 | FEP 3 | HB300×300×83.5 | HB350×175×49.4 | 12 | 200 | 20 |
| 4 | FEP 4 | HB300×300×83.5 | HB450×200×74.9 | 12 | 200 | 20 |
| 5 | FEP 5 | HB350×350×104.6 | HB500×200×102 | 12 | 200 | 20 |

Table 5 Test results for flush end-plate connection connected to column web (EPCW)

| Test specimen | Initial stiffness, $S_{j,initial}$ kNm/mRad | Moment resistance ($M_{Rd}$) kNm |
|---|---|---|
| FEP 1 | 4.13 | 60.0 |
| FEP 2 | 3.51 | 73.0 |
| FEP 3 | 3.16 | 55.0 |
| FEP 4 | 4.55 | 85.5 |
| FEP 5 | 5.25 | 60.0 |

Table 6 Comparison of specimens FEP (EPBS) with FEP (EPCW) connections

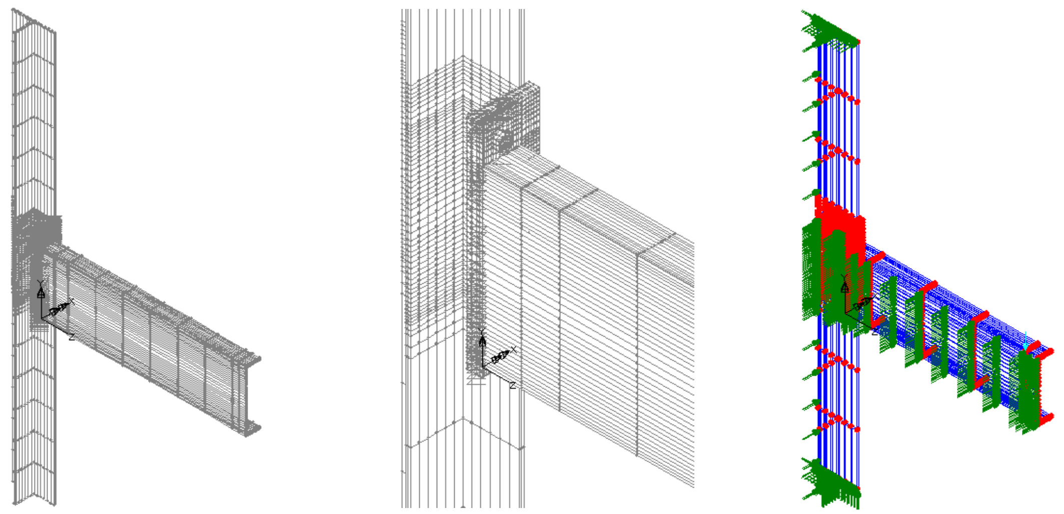
| Specimen | Connection's End-plate Thickness (mm) | parameter Size of bolts (mm) | Moment resistance experiment $M_{\max}$ vs $M_{Rd}$ (kNm) | Percentage difference % | Initial Stiffness, $S_{j,ini} = (M_{j,Rd}/\Phi)_{ini.}$ (kNm/mrad) | Percentage difference % |
|---|---|---|---|---|---|---|
| FEP 4 vs FEP 6 | 12(EPBS) 12(col. web) | M20 | 85.5 | 42.8% | 4.55 | 293% |
| | 12(EPBS) 25mm(EPCF) | M20 | 122.1 | | 17.9 | |
| FEP 2 vs FEP 9 | 12(EPBS) 12(col. web) | M20 | 73.0 | 75.5% | 3.51 | 447.0% |
| | 12 (EPBS) 25mm(EPCF) | M20 | 128.1 | | 19.2 | |
| FEP 1 vs FEP 9 | 12(EPBS) 13(col. web) | M20 | 60.0 | 113.5% | 4.13 | 364.9% |
| | 12(EPBS) 25(EPCF) | M20 | 128.1 | | 19.2 | |
| FEP 3 vs EEP 6 | 12(EPBS) 12(col. web) | M20 | 55.0 | 122.1% | 3.16 | 466.0% |
| | 12(EPBS) 25mm(EPCF) | M20 | 122.1 | | 17.9 | |
| FEP 5 vs FEP 8 | 15(EPBS) 13(col. web) | M24 | 60.0 | 207.3% | 5.25 | 262% |
| | 15(EPBS) 25(EPCF) | M24 | 184.4 | | 19.0 | |

## 5. Finite element model

The connection details tested have also been examined using finite element analysis (FEA), using LUSAS FEA software Version 13.6 (LUSAS 13.6 2002). Although the use of FEA can reduce cost and time compared with testing, the accuracy and reliability of the finite element model depend on a suitable mesh arrangement, balancing the need for accuracy, requiring a fine mesh, against faster analysis, as provided by coarser meshes. The optimal solution is to use a fine mesh in areas of high stress and a coarser mesh in the remaining areas. To further reduce the size of the model and the subsequent processing time, advantage is taken of symmetry and just half of each connection is modelled as shown in Figs. 7 (a)-(c).

The type of element used will also affect both speed and accuracy. Models using only shell elements, only solid elements and a combination of the two were analyzed and compared. In the current study, full solid finite elements were used to achieve optimum. All components in the minor axis of end-plate connections were modelled using the three dimensional solid hexahedral elements (HX8M), comprising 8 nodes each with 3 degrees of freedom. The interface between the end plate and the toe plate were modelled using the non-linear contact gap joint elements JNT4. The contact spring stiffness, K is assigned to the JNT4 elements with the value of 1E9 N/mm$^2$. The purpose of this spring stiffness is to simulate the compression contact between the plates. The spring stiffness is inactive when tension forces are detected. This element property is important to simulate the behaviour of the end plate. To simulate the bending effect of the steel plate components, they are modelled with two layers of solid elements. All bolt components are modelled using HX8M solid element.

The analysis incorporates both geometric and material non-linearity. The material behaviour was represented as a bi-linear stress-strain relationship incorporating an initial linear elastic stage up to yield stress, followed by a plastic region with strain hardening as shown in Fig. 8. In the elastic range Young's Modulus was assumed to be 210 kN/mm$^2$ and Poisson ratio 0.3. For the plastic behaviour, the actual yield stress, $f_y$, was used, based on coupon test results carried out by the authors as shown in Table 7, and the gradient was taken as 0.21 kN/mm$^2$, based on the recommendation by Eurocode 3 Part 1-5.



(a) Mesh discretisation condition (b) Closer view (c) Boundary & Support

Fig. 7 Minor axis of end-plate FEA model with meshed symmetrically into half

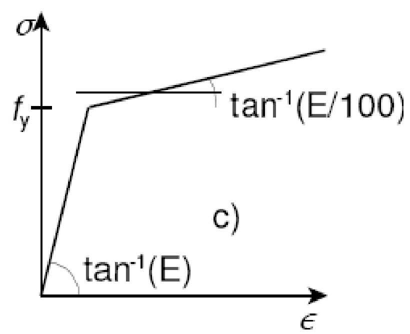
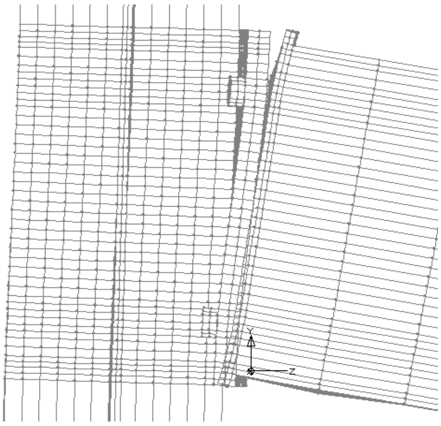



Fig. 8 Elastic-plastic stress- strain relationship

Table 7 Material properties of beams, columns, and end-plates

| No. | Beams and columns | Yield Strength, $f_y$ (N/mm$^2$) | Ultimate Strength, $f_u$ (N/mm$^2$) |
|---|---|---|---|
| 1 | 400×200×65.4 (flange)<br>400×200×65.4 (web) | 335<br>312<br>(avg. 323.5) | 405<br>509<br>(avg. 457) |
| 2 | 450×200×74.9 (flange)<br>450×200×74.9 (web) | 356<br>322<br>(avg. 339) | 510<br>556<br>(avg. 533) |
| 3 | 500×200×102.0 (flange)<br>500×200×102.0 (web) | 299<br>357<br>(avg. 328) | 471<br>499<br>(avg. 485) |
| 4 | 300×300×83.5 (flange)<br>300×300×83.5 (web) | 351<br>351<br>(avg. 351) | 510<br>540<br>(avg. 525) |
| 5 | 250×250×63.8 (flange)<br>250×250×63.8 (web) | 379<br>334<br>(avg. 356.5) | 521<br>504<br>(avg. 512.5) |
| 6 | End-plate (12 mm)<br>P1<br>P2<br>P3 | <br>305<br>308<br>309<br>(avg. 307.3) | <br>467<br>491<br>470<br>(avg. 476) |



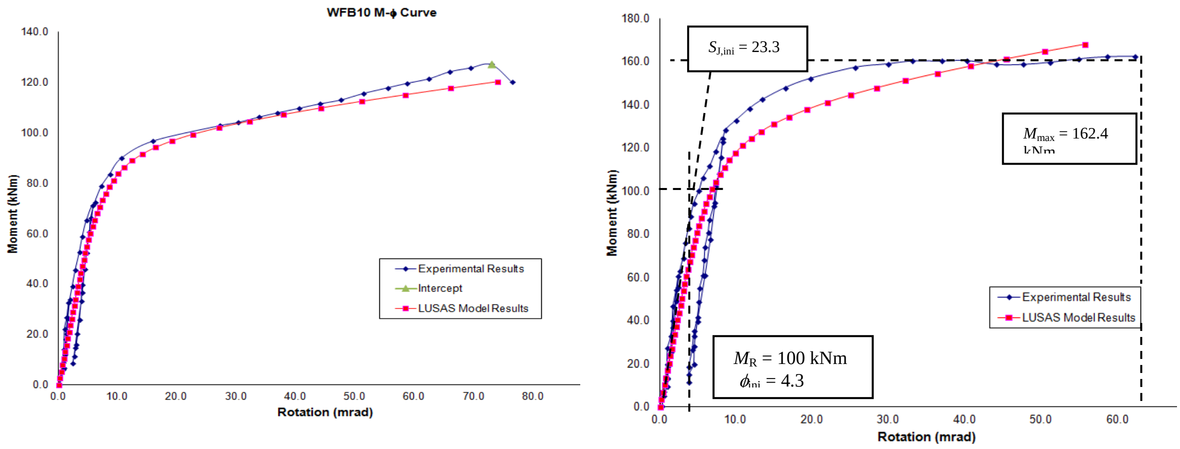
(a) Deformation from FE modelling


(b) Deformation from actual test

Fig. 9 Deformation

### *5.1 Finite element results*

The deformation predicted from the finite element analysis is shown in Fig. 9(a) and compared with those observed in the test in Fig. 9(b), and it is clear that in both the mode of failure is associated principally with large deformations of the beam end-plate in the tension zone. So as to validate the accuracy and reliability of the finite element models further, some of the analytical and experimental results are plotted as ($M$-$\Phi$) curves in Figs. 10(a)-(b). It is clear that there is




(a) Comparison of $M$-Φ curves of FEA vs Experimental test for FEP 10

(b) Comparison of $M$-Φ curves of FEA vs Experimental test for EEP 11

Fig. 10 Comparison of $M$-Φ curves

remarkably good correlation, indicating that the finite element model provides an excellent basis for simulating the actual behaviour of the proposed connection type.

## 6. Conclusions

From the test results and the above discussion of results, the following conclusions can be drawn:

1. The performance of the toe plate connections as characterised by the M-Φ curves show exhibit very good levels of strength and ductility.
2. Compared with the traditional connection detail in which beams are attached directly to the column web, the toe plate connections can offer very significant increases in moment resistance and initial stiffness for both flush and extended end-plates
3. The overall results show that the use of extended end-plate connections affords a significant increase in strength and stiffness compared with a flush end-plate.
4. The finite element model compares very closely with the test results, providing an excellent basis for generating a significant body of additional data for this connection type.

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