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Editorial

Welcome from the Editor

The Editorial welcomes all readers to this second issue of Malaysian Construction Research Journal (MCRJ). Special thanks to all contributing authors for their technical papers. The Editorial would also like to express their acknowledgement to all reviewers for their invaluable comment and suggestion. This issue highlights seven titles which focus on topics related to Industrialised Building Systems (IBS).

In this issue, ***Hamid et. al.*** highlights the current state of IBS in Malaysia and its related R&D initiatives. The study addresses the current scenario of IBS adoption and identified the difficulties in its implementation. The authors also stress the important role of R&D and proposed the strategic approach to be taken on board by Construction Research Institute of Malaysia (CREAM).

Mahmood and Arizu discuss on the development of a standardised partial strength connection tables of extended end-plate connections for trapezoidal web profiled (TWP) steel sections. These tables will assist designers and improve the design of semi-continuous construction of multi-storey braced steel frames. Laboratory tests were carried out to validate the results and presented them in the standardised tables.

Ahmad Baharuddin et. al. writes on the comparative study of monolithic and precast concrete beam-to-column connection. The response of the connection subjected to incremental loading was studied. Specimens of monolithic and precast concrete beam-to-column were tested to evaluate the ultimate loading capacity, moment rotation characteristic as well as their crack response.

Mahyuddin evaluates the structural performance of ferrocement sandwich panel used in Industrialised Building Systems. Experimental investigation was carried out to assess the load-deflection characteristics, crack resistance and moment curvature of ferrocement elements that were exposed to air and salt water environment.

In his second paper, ***Mahyuddin*** reports on the permeability of polymer-modified cement system for structural applications. The durability enhancement of the cement system is achieved by reducing the permeability of the material through polymer modification. He had also investigated and reported the intrinsic properties, mechanical properties and the durability performance of the polymer-modified cement system.

Doh et. al. discusses the findings of their research works on the use of oil palm shell as structural topping for semi-precast concrete slab. Their works focus on the strength characteristic and flexural behaviour of concrete slab and to check its compliance to the requirement specified by the Code of Practice.

Finally, ***Mohd Al Amin et. al.*** investigates the response of ceramic foam core sandwich composite under flexural loading. Their study focuses on the determination of a range of sandwich properties which include shear modulus and bending stiffness through conducting a series of bending tests. They concluded that the ceramic foam core sandwich composite were comparable to those of polymeric foams core materials and has a high potential to be used as core material for sandwich structure construction.

Editorial Committee

STANDARDISATION OF PARTIAL STRENGTH CONNECTIONS OF EXTENDED END-PLATE CONNECTIONS FOR TRAPEZOID WEB PROFILED STEEL SECTIONS

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Abstract:

Traditionally, connections are usually classified as pinned or rigid although the actual behaviour is known to fall between these two extreme cases. The use of partial strength or semi-rigid connections has been encouraged by codes and studies on the matter known as semi-continuous construction have proven that substantial savings in steel weight of the overall construction. The objective of this paper is to develop a series of standardised partial strength connections tables of extended end-plate connections for trapezoidal web profiled steel (TWP) sections. The range of standard connections presented in tabulated form is limited to eight tables comprised of different geometrical aspects of the connections. These tables could enhance the design of semi-continuous construction of multi-storey braced steel frames. The connections are presented in the form of standardised tables which include moment capacity and shear capacity after considering all possible failure modes. A method proposed by Steel Construction Institute (SCI) which take into account the requirements in Eurocode 3 and BS 5950:2000 Part 1 were adopted to predict the moment capacity and shear capacity in developing the tables. A series of tests have been carried out to validate the results of the standardised tables. The test results showed good agreement between theoretical and experimental values. It can be concluded that the proposed standardised tables for TWP sections is suitable to be used in the design of semi-continuous construction.

Keywords: Partial Strength Connection; Extended End Plate; Beam-to-Column Connection; Trapezoid Web Profiled Section; Semi-Continuous Construction

INTRODUCTION

In the design of steel frame, connections play an important part in the determination of the types of construction. Pinned jointed connection is usually associated with simple construction and rigid jointed connection is usually associated with continuous construction. When designed as pinned jointed, the beams are assumed as simply supported and the columns are assumed to sustain axial and nominal moment only. The associated connection such as flexible end plate connection is simple and relatively easy to erect but the sizes of the beams obtained from this approach are relatively heavy and deep. On the other hand, rigidly jointed frame results in heavy columns due to the end moments transmitted through the connection. Hence, a more complicated fabrication of the connection could not be avoided.

Eurocode 3 (1992) introduces designer to use semi-rigid connection or partial strength connection which creates a balance between the two extreme approaches. This alternative semi-rigid or partial strength connection is usually associated with a connection having a moment capacity less than the moment capacity of the connected beam (Peter, *et. al.* 1996). Partial strength connection or semi-rigid connection is the term used for connection in the design of semi-continuous construction for multi-storey steel frames by Eurocode 3(1992).

In semi-continuous construction design the degree of continuity between the beams and columns is greater than that in simple construction design but less than that in continuous construction design. The degree of continuity in the use of partial strength connection of beam to column can be predicted to produce an economical beam section that representing the section between pin joint and rigid joint. By adopting this approach, studies conducted on the use of partial strength connection in hot-rolled steel section have proven substantial savings in overall steel weight (Md Tahir, 1997 & 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. However, the use of partial strength connections for Trapezoid Web Profiled sections has not been established yet. To enable the use of TWP sections with partial strength connection, standardised partial strength connection tables need to be established first. Therefore, this paper intends to establish the standardised tables for partial strength connections for TWP sections based on the proposed method by SCI.

TWP SECTION AS A BUILT-UP SECTION

A trapezoid web profiled section is a built-up plate girder comprised of two flanges connected together by a thin corrugated web as shown in Figure 1 (Osman, 2001 & Hussein, 2001). The web and the flanges are welded together with different steel grade depending on the design requirements. The steel grade of the flanges is designed for S355 and the steel grade of the web is designed for S275. The steel grade of the flanges is purposely designed for S355 so that the flexural capacity of the beam can be increased. The steel grade of the web is designed for S275 so as to reduce the cost of steel material and the capacity of shear is not that critical in the design of the beam (Hussein, 2001). The use of different steel grades in the fabrication of TWP section leads to further economic contribution to steel frames design besides the use of partial strength connection. The use of thick flanges, thin web and deeper beam for TWP section compared with hot-rolled section of the same steel weight leading to heavier load capacity and greater beam span that can be achieved.

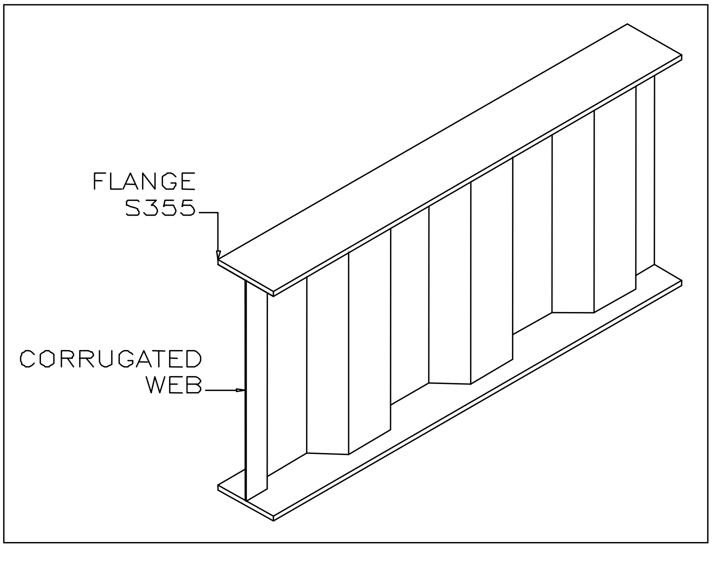


Figure 1. Configuration of Trapezoidal Web Profiled Section

Benefits of using TWP steel sections

The benefits of a TWP beam as compared to the conventional plate girder or hot rolled steel section include the following (Tan, 2004):

- Reduce the steel weight by utilizing thin web.
- Eliminate the need of stiffeners which reduced the fabrication cost.
- The use of high strength steel S355 for flanges and deep beam which lead to higher flexural capacity, wider span and less deflection.

Based on the configuration of the structure, TWP beam can offer substantial saving in the steel usage, and in some cases of up to 40% as compared to conventional rolled sections (Osman, 2001& Hussein, 2001). It is more significant when there is a need for a column free; long span structural system, such as portal frames for warehouses, girder for bridges, floor and roof beam for high-rise buildings, and portal frame for factory.

ADVANTAGES OF STANDARDIZED 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 (Peter A, et al 1996). However, the cost of the fabrication is in the range of 30% to 50% of the total cost depending on the difficulty of the fabrication (Peter, *et. al.* 1996). The increase in the fabrication of the connections is due to the difficulty in selecting the type of connection, the grades and sizes of fittings, bolt grades and sizes, weld types and sizes, and the geometrical aspects. Therefore, a standardized partial strength connections tables are introduced to cater for the problems arise due to so many uncertainties in the fabrication of the connections. The advantages of the partial strength approach are that it utilizes the moment resistance of connections to reduce beam depth and weight, while avoiding the use of stiffening in the joints. This practice will reduce the cost of fabrication and ease the erection of steel member in the construction of multi-storey steel frames. The potential advantages of using this approach can be listed as follows (Peter & Mike,1992)(Peter & Mike, 1993):

Lighter beams

In the design of semi-continuous braced steel frame, the required beam plastic modulus is less than those required in simple frame for the same frame. This reduction is possible as the partial strength connection reduced the design moment of the beam due to the partial restraint effect of the connection as illustrated in Figure 2 (Couchman, 1997). The design moment which a beam must resist, decreases as the moment capacity of the connection increases. As a result, a lighter beam can be selected for the design of the beam.

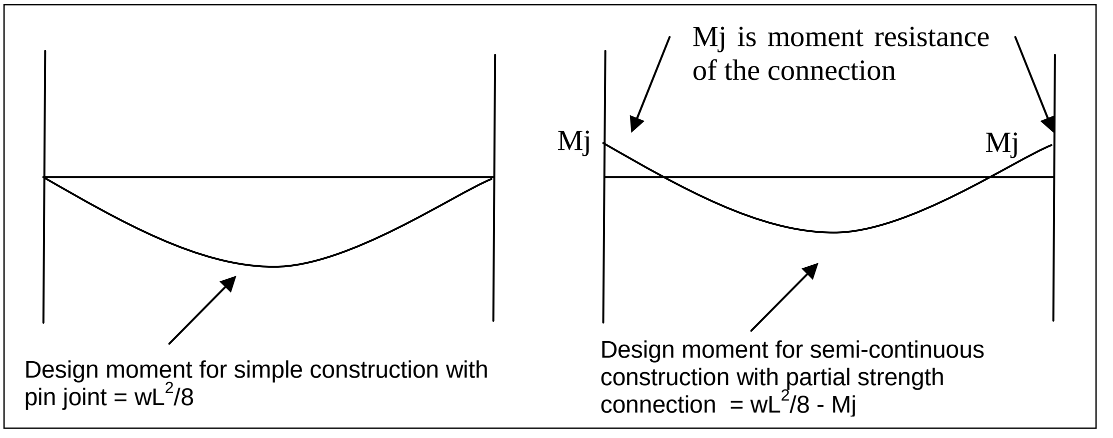


Figure 2. Design moment for beams due to different support conditions

Shallower beams

The partial restraint of the connection will also result in shallower beams. This is due to the increase in stiffness of the connection, which contributes to the decrease in deflection. The use of partial strength connection will reduce the constant coefficient β in the formulae of deflection ($\beta wL^4/384EI$) in simple construction with uniform load from β equal to 5 to β equal to 2 for internal beam and β equal to 3 for external beam (Couchman, 1997). The partial strength connection acts as restrained to the deformation of the beam due to applied load. As a result, a reduction in the deflection of the beam can be achieved which lead to the shallower beam. The relationship between connection stiffness and deflection coefficient “Beta” for uniform load on beam is shown in Figure 3.

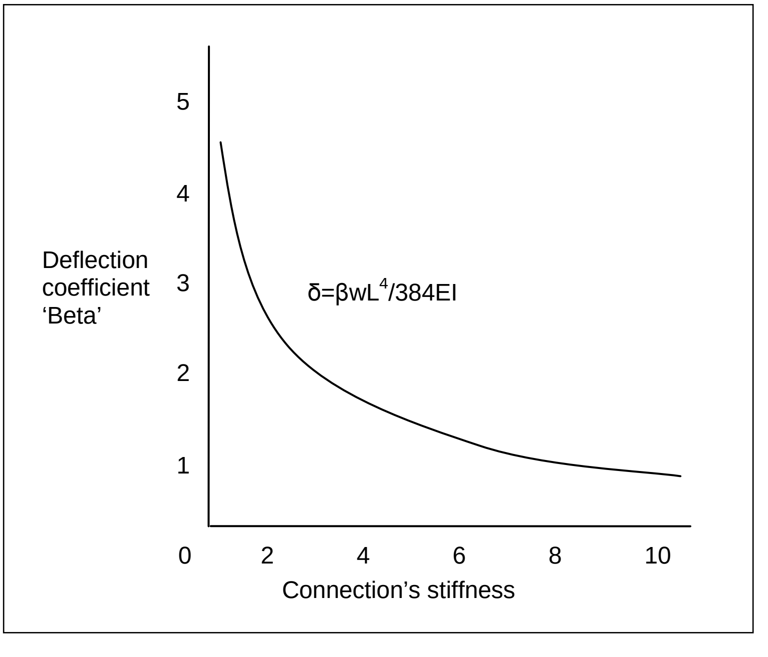


Figure 3. Deflection coefficient ‘Beta’ as a function of relative stiffness of connection

Greater stiffness and more robust structure

Connection stiffness means that the ends of a beam are restrained against rotation. Partial strength connection has higher capacity to restrain against rotation, shear, moment, and tying force. The rotation capacity should be in the range of 0.02 to 0.03 radians at failure for the connection to be considered as ductile and stiff enough to be categorised as partial strength (Peter, *et. al*., 1996). The shear capacity of the connection is designed in such a way that the capacity is higher than the shear capacity of the connected beam, and the moment capacity of the connection can resist up to 50% of the moment capacity of the connected beam (M_{cx}) depending on the size and number of bolts for the proposed standard tables. The tying force of the connection is two to three times greater than the tying force required by (BS 5950:2000 Part 1) that is 75kN. Therefore, the connection can be categorized as strong, stiff, and robust connection.

Lower overall cost

Good connection should be the one which can ease the design process, the preparation of detailing, the fabrication process, and the erection works. It should also consider the most cost effective in the development of the connection. The saving in the overall cost can be achieved due to the following reasons (Peter & Mike, 1992)(Peter & Mike, 1993):

- A reduction in the number of connection types may lead to a better understanding of the cost and type of connection by all steel players such as fabricator, designer, and erector.
- A standardised connection can enhance the development of design procedures and encourage in the development of computer software.
- The use of limited standardised end-plates or fittings can improves the availability of the material leading to reduction in material cost. At the same time, it will improve the order procedures, storage problems and handling time.
- The use of standardised bolts will reduce the time of changing drills or punching holes in the shop which lead to faster erection and less error on site. The drilling and welding process can be carried out at shop as the geometrical aspects of the connection have already been set. This leads to fast and quality fabrication.

Although the advantages or benefits of using the partial strength connections are quite significant, the disadvantages of this approach should also be addressed. The disadvantage in this approach is that it may be marginally more expensive to fabricate partial-strength connection rather than simple connections. However, the benefit of overall cost saving of the partial strength connections has proven to be more than simple connections. It is reported that the savings in steel weight of using partial strength connection in multi-storey braced steel frames using British hot-rolled section was up to 12% (Md Tahir, 1997). The overall cost saving was up to 10% of the construction cost which is quite significant (Md Tahir, 1997). This is quite significant as the proposed partial strength connection is able to satisfy the requirement of the design code without implicate any problems with the safety of the structure.

PROPOSED STANDARDIZED EXTENDED END-PLATE CONNECTIONS

The use of partial strength connections for hot-rolled British sections has well established by SCI (Peter, *et. al*., 1996). A series of tests at the University of Abertay, Dundee has been successfully been carried out to verify the predicted moment and shear capacity with the experimental tests capacities (Bose, 1993). The results confirmed with the predicted values and the standardized tables for the connection have been published by SCI (Peter, *et. al.*, 1996). In partial strength connections, two types of connections are preferred: flush end plate connection and extended end plate connection. The development of standard flush end plate connections tables has been reported in another publication by the author (Md Tahir, *et. al.*, 2006). In the development of standard extended end-plate connections tables for TWP sections, only eight tables are presented in this study based on the proposed method. Although the best validation of the results presented in the tables is by comparing the predicted results with the actual experimental tests results, however, the presented standard connection tables for TWP section can still be used by adopting the same failure modes of the hot-rolled section as tested by SCI. A few tests have been carried out to support the predicted moment resistance of the connection using TWP section as a beam. Some of the results are presented later in this paper. The proposed standard connections have the following attributes, which in some cases the attributes are not exactly the same as the one described by SCI in hot-rolled section.

- The end plates are extended at the tension side only since no reversal of moments is expected as shown in Figure 4. This type of connection is recommended for braced steel frame.
- 12mm thick end plates in conjunction with the use of M20 bolts.
- 15mm thick end plates in conjunction with the use of M24 bolts.
- Strength of end plates was maintained as S275 steel.
- Width of the end plate was kept at 200mm and 250mm with the vertical height of the end-plate was kept at the beam depth plus 90mm.
- Only one row of bolts is used in the extended part of end plates.
- Full strength of flange welds with size of weld proposed at 10mm but an 8 mm weld is also adequate.
- Full strength of web welds with size of weld proposed at 8mm but a 6 mm weld is also adequate.
- The vertical and horizontal distance between the bolts was maintained at 90mm.

Figure 4, shows a typical extended end plate connection for TWP section as beam connected to British hot-rolled section as column. British section is selected for the column as it is very good in compression which is not the case for TWP section as the web of TWP is too thin to carry axial load. TWP section is proposed for the beam as the corrugated web section is very effective to cater for buckling and bearing resistance. 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 increased to at least semi-compact even though the suggested limit is compact as described by (BS5950:2000 Part 1). The limit is increased based on the observation from previous study using Flush End Plate connections that are capable to sustain higher moment capacities (Peter, *et. al.*, 1996).

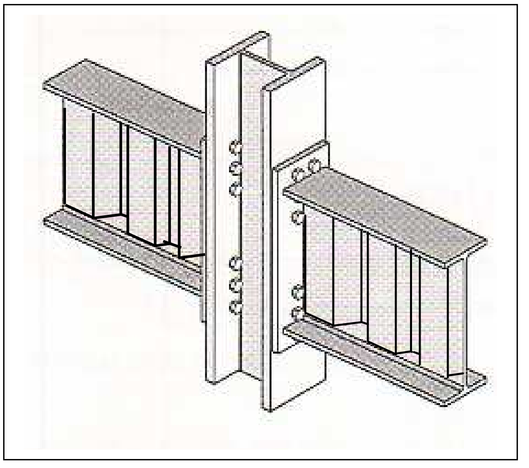


Figure 4. Typical extended end-plate connection of TWP beam section connected to British Hot-Rolled section.

DEVELOPMENT OF THE STANDARD CONNECTION TABLES

Unlike simple and rigid connections, the design of partial strength connections involves more complex and rigorous procedures. Therefore, Steel Construction Institute published a reference guide in designing moment connections, which includes sections on the standardised capacity tables for bolted end plate connections (Peter, *et. al.* 1996). The design model presented in the SCI's guide is in accordance to the procedures in Annex J of EC3, which is based on the plastic distribution of bolt forces. Traditionally, the bolt forces are taken as a triangular distribution but plastic distribution is 'accurately' representing the actual behaviour of bolt forces as shown in Figure 5 (Eurocode 3, 1992). The tension force F_{reinf} noted as the reinforcement was calculated as follows:-

$F_{reinf} = \dfrac{f_y A_{reinf}}{\gamma_m}$ where f_y is the design yield strength of reinforcement, A_{reinf} is the area of reinforcement within the effective width of the slab and γ_m is the partial safety factor for reinforcement taken as 1.05. The forces of the tension bolts are noted as F_{r1} and F_{r2} as shown in Figure. 5 are calculated by checking on the top row and working downward. This means that each of bolt rows is checked first in isolation which is then combined with the lower row to get the potential force for that particular bolt. The potential force for the bolts can be summarised as follows:-

F_{r1} = (resistance of row 1 alone)

F_{r2} = minimum of (resistance of row 2 alone or (resistance of rows 2 + 1) – F_{r1})

In the SCI's guide, the beam-to-column arrangements constitute of conventional hot rolled sections for both the beams and the columns. In this study, TWP sections are used as beams; therefore, the tables provided in the design guide for hot rolled British sections are not applicable to the TWP sections as the section properties of TWP sections are not similar.

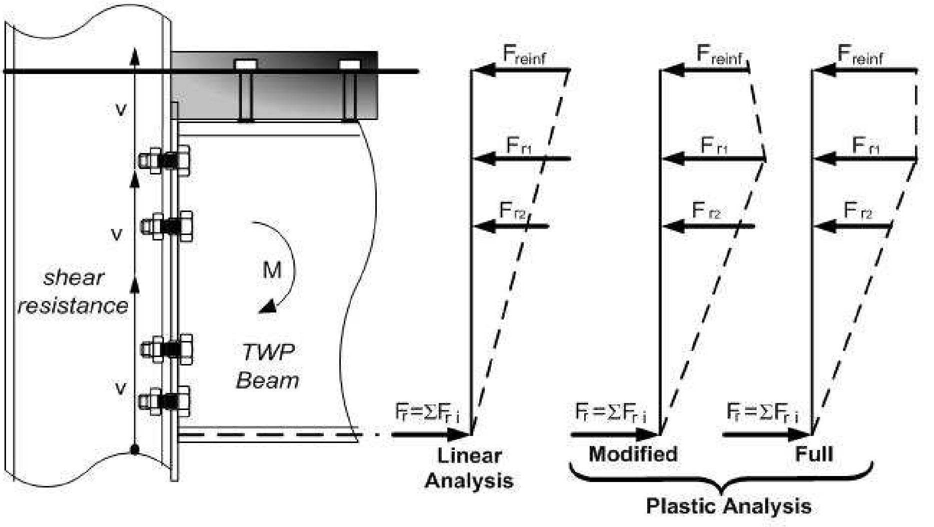


Figure 5. Forces in connection and corresponding distribution

Design philosophy of the connections

The design model adopted in this study is actually presented in Annex J of Eurocode 3: Part 1.1 (1992). For checking the details of strength on the bolts, welds, and steel section, modification to suit (BS 5950:2000 Part 1) have been done. The checking on the capacity of the connections is classified into three zones namely tension zone, compression zone, and shear zone as shown in Figure 6 (Peter, *et. al.*, 1996). The basic principles of the distribution of bolt forces need to be addressed first before details of the checking on all possible modes of failures can be discussed.

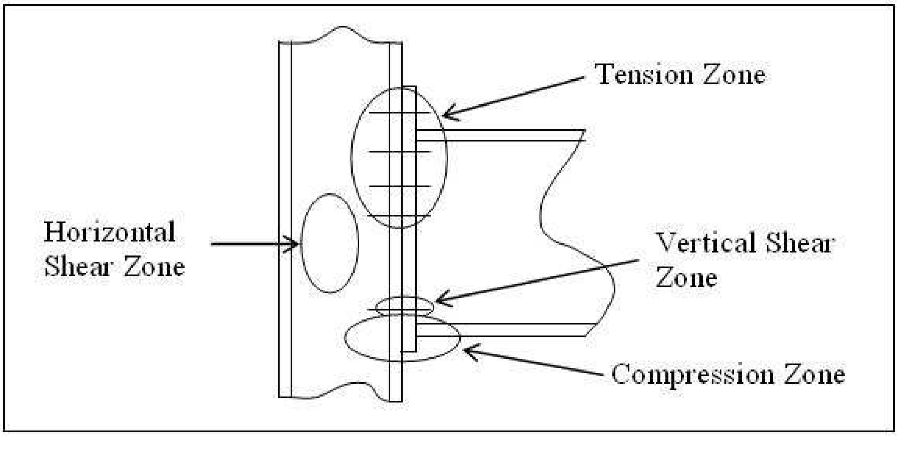


Figure 6. Critical zones that need to be checked for failure

Distribution of Bolt Forces

The moment resistance of a connection transmitted by an end plate connection is through the coupling action between the tension forces in bolts and compression force at the centre of the bottom flange. Each bolt above the neutral axis of the beam produced tension force whereas the bolts below the neutral axis are dedicated to shear resistance only. Eurocode 3(1992) suggests that the bolt forces distribution should be based on the plastic distribution instead of the traditional triangular distribution. Figure 5 shows the forces in the connection and the corresponding distributions. The forces of the bolt are based on the plastic distribution which is the actual value calculated from the critical zones in Figure. 6. The force from the top bolt row transmits to the end-plate connection as tension force which balanced up by the compression force at the bottom flange of the beam to the column. The end-plate is connected to web and both of the flanges by welding. The formation of tension at the top and compression at the bottom contributes to the development of moment resistance of the connection. Tests on the connections have showed that the centre of compression flange which bears against the column was found to be the centre of rotation of the connection (Bose, 1993). The force permitted in any bolt row is based on its potential resistance and not just the length of the lever arm.

Tension zone

The resistance at each bolt row in the tension zone may be limited due to bending of column flange, end-plate, column web, beam web, and bolt strength. Column flange or end-plate bending was checked by using Eurocode 3(1992) which converts the complex pattern of yield lines around the bolts into a simple ‘equivalent tee-stub’. Details of the procedures are illustrated in SCI publication (Peter, *et. al.*, 1996).

Compression zone

The checking in the compression zone are the same procedures as mention in (BS 5950:2000 Part 1) which requires checks on web bearing and web buckling. The compression failure modes can be on the column side or on the beam side. The column side should be checked for web buckling and web bearing due to the compression force applied to the column. The use of stiffener or the effect of having other beam connected to the web of the column is not included so as to reduce the cost of fabrication and simplified the calculation. The compression on the beam side can usually be regarded as being carried entirely by the beam flange, however when large moments combine with axial load, the compression zone will spread to the web of the beam which will effect the centre of compression. Therefore, the stiffening of the web of the beam needs to be done. However, in this study the moment resistance of the connection is not considering the use of stiffener in order to reduce the cost of fabrication.

Shear zone

The column web can fails due to the shearing effect of the tension and compression force applied to the web of the column. The failure to the shearing of the web is most likely to happen before it fails due to bearing or buckling. This is possible because the thickness of the flange is more than the thickness of the web. Again in this shear zone, stiffer is not needed so as to reduce the cost of fabrication.

Welding

Fillet weld is preferred than the butt welds as the welding of beam to the end-plate is positioned at 90 degree which is suitable for fillet weld to be used. The end-plate is connected to the web of the beam by an 8 mm fillet weld, whereas a 10 mm fillet weld is suggested for connecting the end-plate to the flange. The weld is designed is such a way that the failure mode of the connection is not on the welding. This is to ensure the ductility of the connection which is necessary for partial strength connection.

Validation of the standardised connections tables

The validation of the standardised connections tables for TWP is best presented by comparing the predicted values in the table with full scale testing of the connections. Therefore, a series of full scale testing on TWP girder sections comprised of four specimens was conducted by the Steel Technology Centre, Universiti Teknologi Malaysia. Although the tests did not cover the whole ranged of the proposed connections, the comparison of the tests and the predicted values can still be established. Figure 7(a, b, c, and d) show some of the results of the experiment by plotting the moment on the connection versus the rotation of the connection. The curves show that the moment resistance of the connection was linear at initial stage followed by non-linear stage. The tests results of moment resistance, M_R were determined when a "knee" formed in each of the M-Φ curves plotted in Figure 7(a to d). This knee technique has been used by many researchers to predict the moment resistance of the connection from the M-Φ curves drawn from the tests results (Tahir, 1997, Sulaiman, 2007 & Anis, 2007). The formation of 'knee' which determine the moment resistance of the connection was developed by drawing two straight lines; a straight line drawn from linear region and intersected to another straight line drawn from a non-linear region that formed almost a plateau in the M-Φ curves. By adopting this technique, the test values of moment resistance, M_R for the overall joint for the tests were established from the point of intersection which identified as 'knee'. This technique takes into account the deformation of the connection due to the formation of elasto-plastic, a region between elastic and plastic regions.

The moment resistance of the beam should be based on the slenderness of the section and the stress block as shown in Fig. 8. This approach is applicable of web-to-depth ratio not greater than 62ε where the beam is assumed not to be susceptible to shear buckling. Otherwise, the beam should be checked for shear buckling using clause 4.4.4.2 in BS 5950:2000 Part 1. As the web of TWP sections is thin, the determination of moment resistance of the sections can be simplified by using a "flange only method" as suggested by BS 5950:2000 Part 1 for built-up section. This method assumed that the flanges fully yielded and with d/t of the thin web is more than 62ε, the contribution of the web is ignored to ease engineers in design calculation. The failure modes of end-plate of the connections are shown in Figure 9 as expected from the calculation. Details of the method of testing and the discussion of the result have been published elsewhere (Sulaiman, 2007).

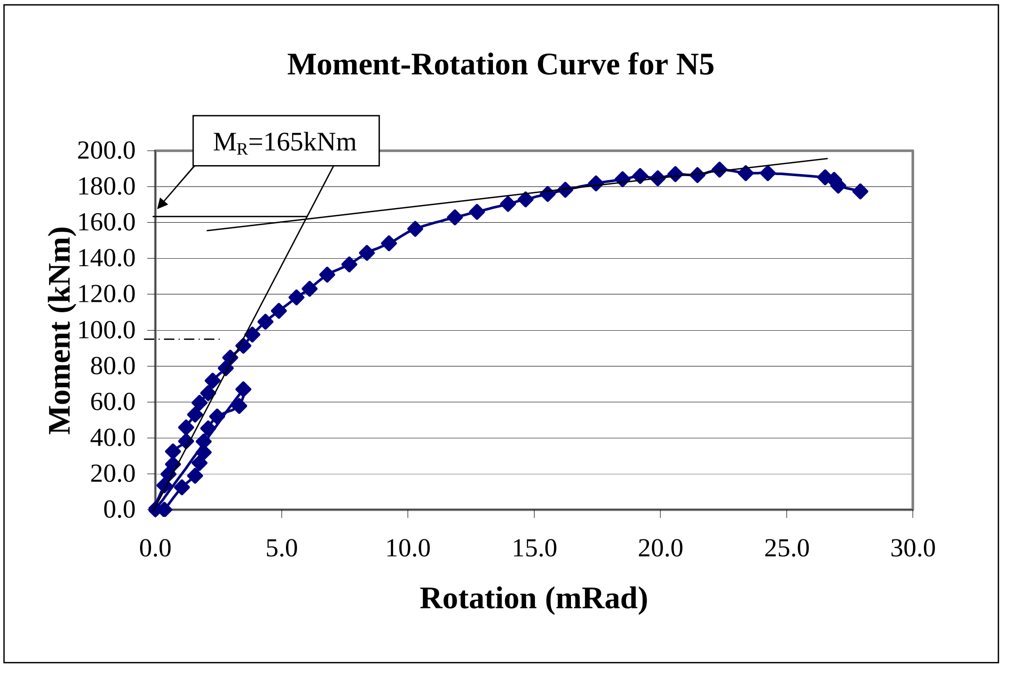


Figure 7(a). Moment versus rotation for specimen N5 (E2R20P1)

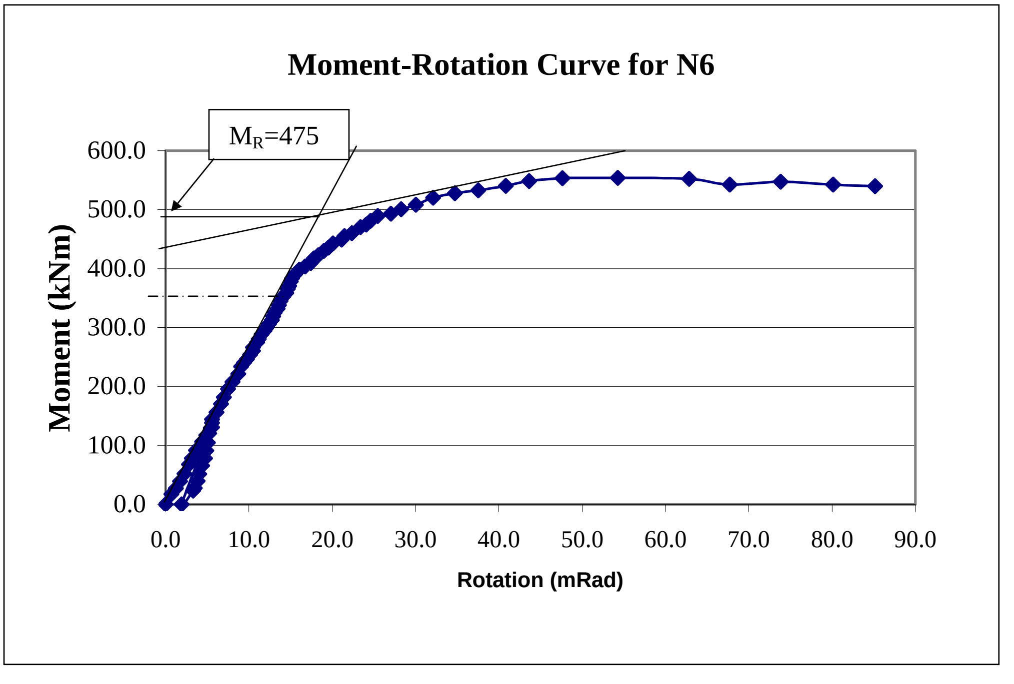


Figure 7(b). Moment versus rotation for specimen N6 (E2R24P2)

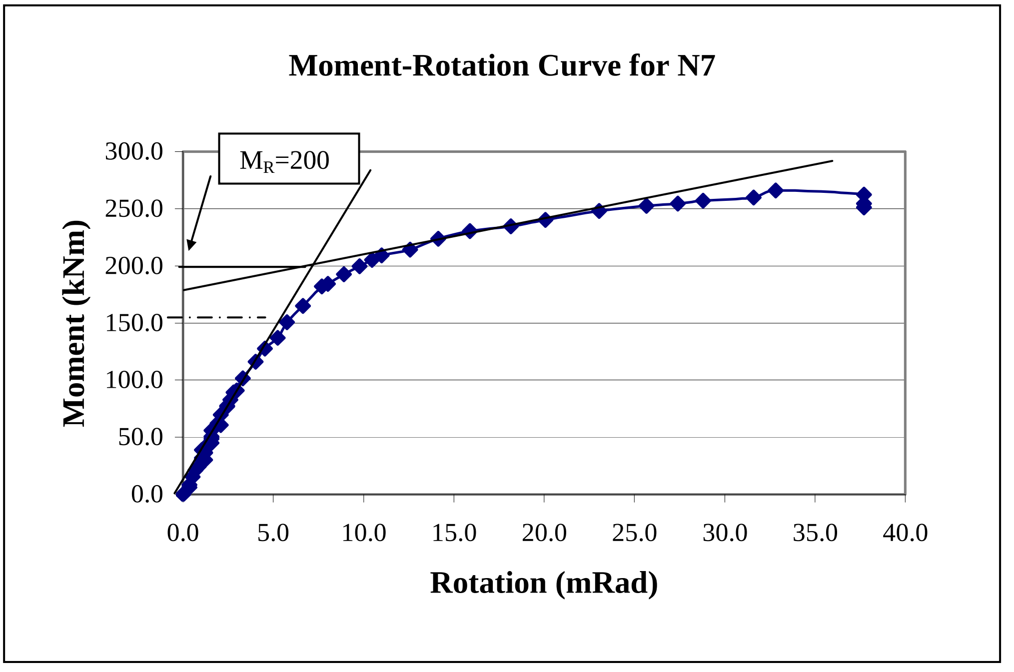


Figure 7(c). Moment versus rotation for specimen N7 (E3R20P1)

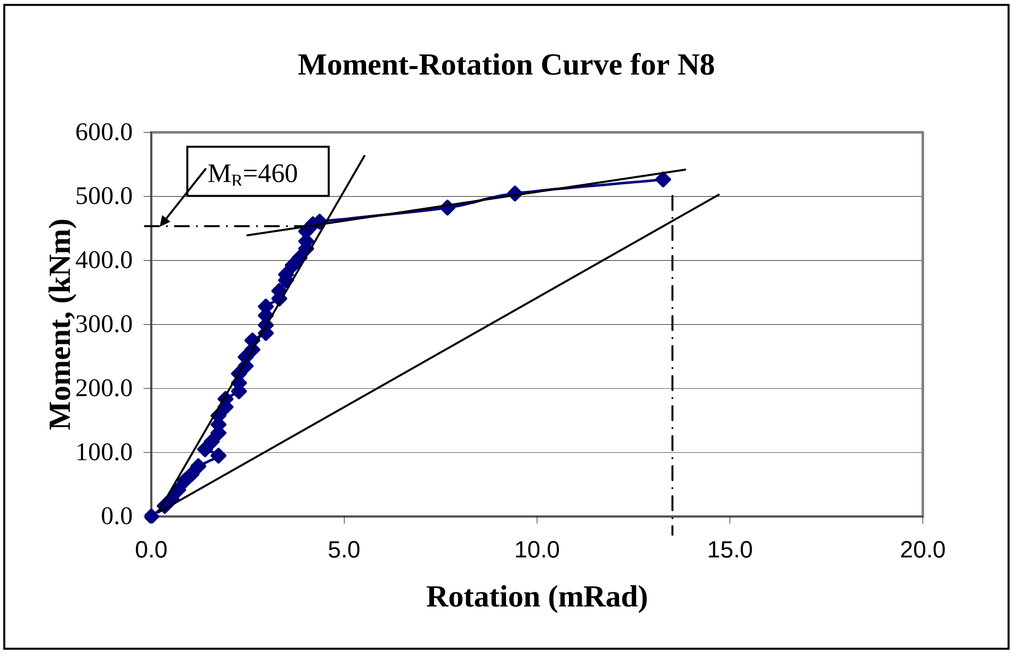
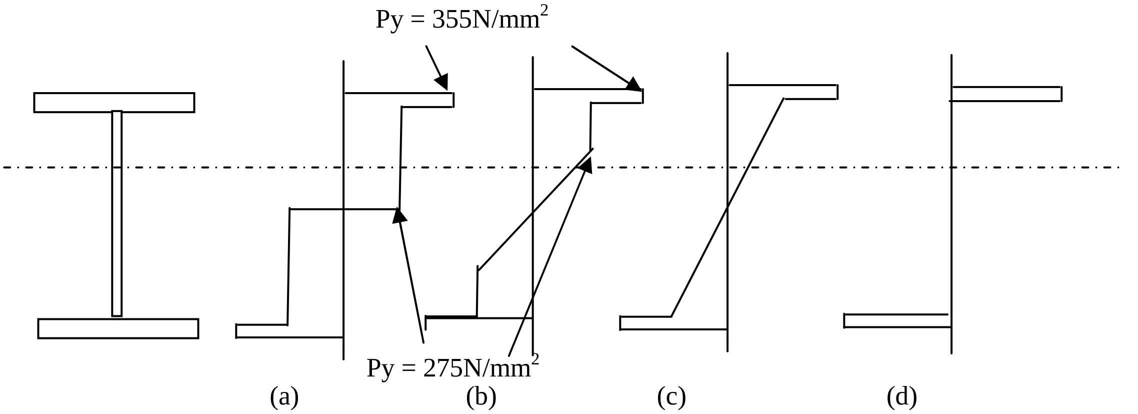


Figure 7(d). Moment versus rotation for specimen N8 (3R24P2)



(a) class 1 (plastic) (b) class 2 (elasto-plastic) (c) class 3 (elastic). (d) flange only method

Figure 8. Stress block on the cross section with different design strength.

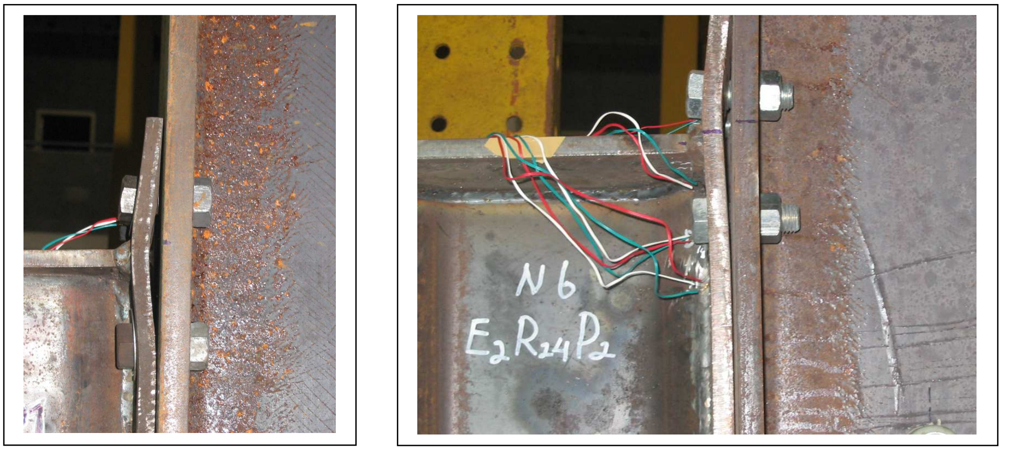


Figure 9. Failure modes of extended end-plate connection during test

Explanation on the notation used in the proposed capacity tables

Eight configurations of extended end plate connections tables have been developed as shown in Table 1. A computer programming based on spread sheet has been developed to calculate and predict the moment capacity and shear capacity of the standardised connections proposed in Table 1 based on the critical zones checks and method proposed by SCI as described earlier. The details of the capacities of the standard tables for the connections are tabulated in Table 2 to 9. The moment capacity from the proposed table is calculated from the summation of each bolt row multiply by the lever arm of the connection. The lever arm for the first tension bolt row, which is defined as ‘dimension A’ measured from the centre of compression capacity to the lowest row of tension, bolts. The lever arm for the second tension bolt row is measured as ‘dimension A’ plus the distance first tension bolt row to the second tension bolt row, in this case 90mm. All flange welds are to be fully welded with minimum fillet weld of size 10mm for flange and 8mm for web. A tick in the table indicates that the column flange and web in tensions have a greater capacity than the beam force as indicated in the beam table. If the column has a smaller capacity, the reduction of bolt force is shown in the table. A modified moment resistance that has been reduced can be determined from these lower forces. A tick in compression zone indicates that the column web has a greater compression capacity than the sum of the bolt row forces. A vertical shear capacity is the shear resistance of the bolt due to shearing, bearing to the bolt and bearing to the plate.

Table 1. Configurations of end plate connections used to generate standardised tables

Type of Connections	Row of Bolts	Type of Bolts	Size of End Plates
EEP,2BRM20,200W12TEP	2	M20 8.8	200 x 12
EEP,2BRM20,250W12TEP	2	M20 8.8	250 x 12
EEP,2BRM24,200W15EP	3	M20 8.8	200 x 12
EEP,2BRM24,250W15TEP	3	M20 8.8	250 x 12
EEP,3BRM20,200W12TEP	2	M24 8.8	200 x 15
EEP,3BRM20,250W12TEP	2	M24 8.8	250 x 15
EEP,3BRM20,200W15TEP	3	M24 8.8	200 x 15
EEP,3BRM24,250W15TEP	3	M24 8.8	250 x 15

DISCUSSION OF RESULTS

The standard tables as shown in Table 2 to 9 illustrate the geometrical configuration of the suggested connections and the capacities of the connections. The suggested size of column and beam used for the proposed connection is listed in the designated table. The moment capacity of the connection is listed base on the size of the beam. The smallest suggested size of beam (in the low capacity connection table) is taken as 350x140 whilst the largest suggested size of beam (in the high capacity connection table) is taken as 750x250. Although TWP section can be produced for up to 1600mm deep, the limited suggested size for partial strength connection is up to 750mm deep. This is to maintain the ductility of the connection that is crucial for partial strength connection. The shear capacity of the connection is based on the shear capacity of the tension bolt row and lower bolt rows. However, the lower bolt row will carry most of the shear force. The increase in moment capacity depends on the size of bolt, the number of bolt, the size of end-plate, and the thickness of end-plate. The notation used for the designated connection such as (EEP,2BRM20,200W12TEP) meaning that the connection is an extended end-plate with two bolt rows of M20 (one row in the extended part of end plate and one row beneath the flange) grade 8.8, and an end-plate size of 200mm wide and 12mm thick. The comparison of the moment capacity of the connection based on different geometrical configuration of the connections is discussed below:

Effect of increasing the number of bolt row from two rows to three rows

Table 2, 3, 4 and 5 show the moment capacity of the connection for double bolt rows. Table 6, 7, 8 and 9 shows the moment capacity of the connection for triple bolt rows. The results of percentage increase in moment capacity for two and three bolt rows are shown in Table 10. The results showed that by increasing the number of bolt row from two to three, moment capacity of the connection is increased by an average of 30.1% for M20 bolt with 12mm thick and 200mm wide end plate, 30.8% for M20 bolt with 12mm thick and 250mm wide end plate, 32.8% for M24 bolt with 15mm thick and 200mm wide end plate, 29.4% for M24 bolt with 15mm thick and 250mm wide end plate. The combination of M24 with 15mm thick end plate has contributed to the increase in the moment capacity of the connection. The increment however is not that significant. The increase in moment capacity is very much linear to the depth of the beam. This shows that the moment capacity of the connection depends on the depth of the beam, the number and size of bolt, and the thickness of the end plate.

The vertical shear capacity of connection in Table 2 and 3 is increased from 331kN without optional shear bolt row to 515kN with shear row. The vertical shear capacity of connection in Table 4 and 5 is increased from 475kN without optional shear bolt row to 739N with shear row. The increment of the vertical shear capacity is not exactly double as the determination of the shear capacity depends on the number of row of the tension bolt too. The vertical shear capacity of the connection in Table 6 and 7 is 588kN with optional shear bolt row. The vertical shear capacity of the connection in Table 8 and 9 is 845kN with optional shear bolt row. These values are about twice the vertical shear capacities of the connections in Table 2 and 3, and Table 4 and 5 respectively without optional shear bolt row. This is because the number of bolt row at the tension zone in Table 6, 7, 8 and 9 is three rows. Panel shear capacity for all the connections is the same as the size of the columns is the same and the force of tension and compression that exert on the column web is not high enough to change the calculated values.

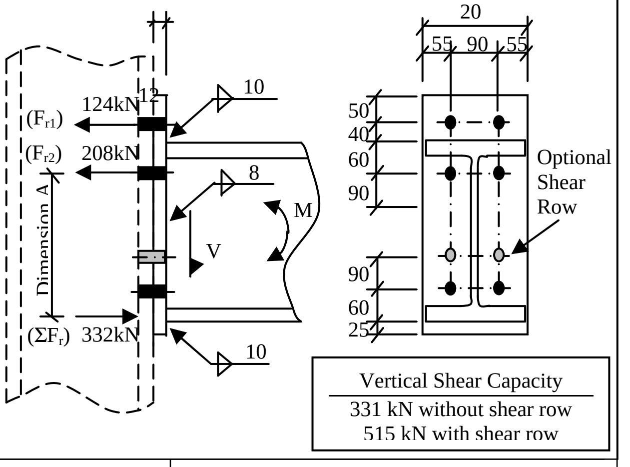
Table 2. Standard table for extended end-plate for 2 row M20 8.8 bolts, 200x12 end-plate (EEP,2BRM20,200W12TEP)

2 ROW M20 8.8 B0LTS
200 x 12 DESIGN GRADE 43 EXTENDED END PLATE

Beam Side

BEAM – FLANGE S355
WEB S275

Beam Serial Size D x B x kg/m (T/ t)	Dimension 'A' (mm)	Moment Capacity (kNm)
350 x 140 x 35.1 (12/3)	284	106
400 x 140 x 39.7 (12/4)	334	123
400 x 160 x 48.4 (14/4)	333	123
450 x 160 x 50.2 (14/4)	383	139
450 x 180 x 60.1 (16/4)	382	139
500 x 160 x 52.0 (14/4)	433	156
500 x 180 x 61.9 (16/4)	432	156
550 x 200 x 73.3 (16/5)	482	172
600 x 200 x 80.5 (16/6)	532	189
350 x 140 x 35.1 (12/3)	284	106



Column Side

DESIGN GRADE S275				COLUMN	DESIGN GRADE S355			
Panel Shear Capacity (kN)	Tension Zone F_{R1} (kN)	Tension Zone F_{R2} (kN)	Compn. Zone	Serial Size	Compn. Zone	Tension Zone F_{R1} (kN)	Tension Zone F_{R2} (kN)	Panel Shear Capacity (kN)
1000	√	√	√	356 x 368 x 202	√	√	√	1302
849	√	√	√	177	√	√	√	1105
725	√	√	√	153	√	√	√	944
605	√	√	√	129	√	√	√	787
1037	√	√	√	305 x 305 x 198	√	√	√	1350
816	√	√	√	158	√	√	√	1062
703	√	√	√	305 x 305 x 137	√	√	√	915
595	√	√	√	118	√	√	√	774
503	√	√	√	97	√	√	√	649
882	√	√	√	254 x 254 x 167	√	√	√	1149
685	√	√	√	132	√	√	√	892
551	√	√	√	107	√	√	√	717
434	√	√	√	89	√	√	√	566
360	√	√	√	73	√	√	√	465
459	√	√	√	203 x 203 x 86	√	√	√	598
353	√	√	√	71	√	√	√	460
322	√	√	√	60	√	√	√	415
272	√	√	√	52	√	√	√	351
245	√	181	√	46	√	√	√	316

Tension Zone:

√	Column satifactory for bolt row tension values shown for the beam side.
xxx	Calculate reduced moment capacity using the reduced bolt row values.

Compression Zone:

√	Column capacity exceeds ΣF_r.
S (xxx)	Column requires stiffening to resist ΣF_r (value is the column web capacity).

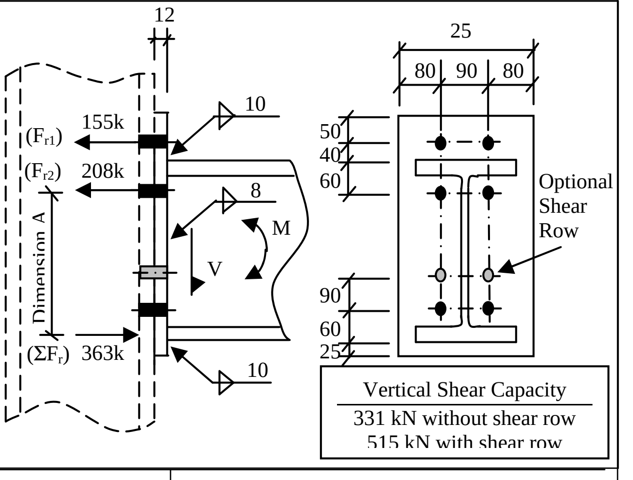
Table 3. Standard table for extended end-plate for 2 row M20 8.8 bolts, 250x12 end-plate (EEP,2BRM20,250W12TEP)

2 ROW M20 8.8 B0LTS
250 x 12 DESIGN GRADE 43 EXTENDED END PLATE

Beam Side

BEAM – FLANGE S355
WEB S275

Beam Serial Size D x B x kg/m (T/ t)	Dimension 'A' (mm)	Moment Capacity (kNm)
400 x 160 x 48.4 (14/4)	333	136
450 x 160 x 50.2 (14/4)	383	154
450 x 180 x 60.1 (16/4)	382	154
500 x 180 x 61.9 (16/4)	432	172
550 x 200 x 73.3 (16/5)	482	190
600 x 200 x 80.5 (16/6)	532	208
650 x 250 x 103.4 (18/6)	581	226



Column Side

DESIGN GRADE S275				COLUMN	DESIGN GRADE S355			
Panel Shear Capacity (kN)	Tension Zone F_{R1} (kN)	Tension Zone F_{R2} (kN)	Compn. Zone	Serial Size	Compn. Zone	Tension Zone F_{R1} (kN)	Tension Zone F_{R2} (kN)	Panel Shear Capacity (kN)
1000	√	√	√	356 x 368 x 202	√	√	√	1302
849	√	√	√	177	√	√	√	1105
725	√	√	√	153	√	√	√	944
605	√	√	√	129	√	√	√	787
1037	√	√	√	305 x 305 x 198	√	√	√	1350
816	√	√	√	158	√	√	√	1062
703	√	√	√	305 x 305 x 137	√	√	√	915
595	√	√	√	118	√	√	√	774
503	√	√	√	97	√	√	√	649
882	√	√	√	254 x 254 x 167	√	√	√	1149
685	√	√	√	132	√	√	√	892
551	√	√	√	107	√	√	√	717
434	√	√	√	89	√	√	√	566
360	√	√	√	73	√	√	√	465
459	√	√	√	203 x 203 x 86	√	√	√	598
353	√	√	√	71	√	√	√	460
322	√	√	√	60	√	√	√	415
272	√	√	√	52	√	√	√	351
245	√	150	√	46	√	√	√	316

Tension Zone:
√ Column satifactory for bolt row tension values shown for the beam side.
xxx Calculate reduced moment capacity using the reduced bolt row values.
Compression Zone:
√ Column capacity exceeds ΣF_r.
S (xxx) Column requires stiffening to resist ΣF_r (value is the column web capacity).

Effect of increasing the size of end-plate from 200mm to 250mm

Table 4 and Table 6 show the moment resistance of the connection for end-plate width of 200mm. Table 5 and Table 7 show the moment resistance of the connection for end-plate width of 250mm. The idea of comparison is to know the percentage increase due to increment of the width of the end-plate. The results of percentage increase in moment capacity for 200mm and 250mm wide of the end-plate are tabulated in Table 9. The results showed that by increasing the size of end-plate width from 200mm to 250mm, moment capacity of the connection is increased by an average of about 5.1% for M20 bolt with 12mm thick end-plate and an average of 2.7% for M24 bolt with 15mm thick end-plate. The results show that the increment of the plate size from 200 to 250mm has contributed to a marginal amount of moment capacity to the connection. For M24 bolt, the increment in moment capacity is reduced by almost half of M20 bolt. This shows that the moment capacity of the connection depends on the strength of the bolt more than the strength of the end-plate.

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

The need to compare the result is to know the percentage increase due to increment of the size of bolt and thickness of the end-plate. The results of percentage increase in moment capacity for M20 with 12mm thick end-plate and M24 with 15mm thick end-plate are tabulated in Table 10. The results showed that by increasing the size of bolt from M20 with 12mm thick end-plate to M24 with 15mm 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 result show that the moment capacity of the connection depends on the strength of the bolt more than the strength of the end-plate.

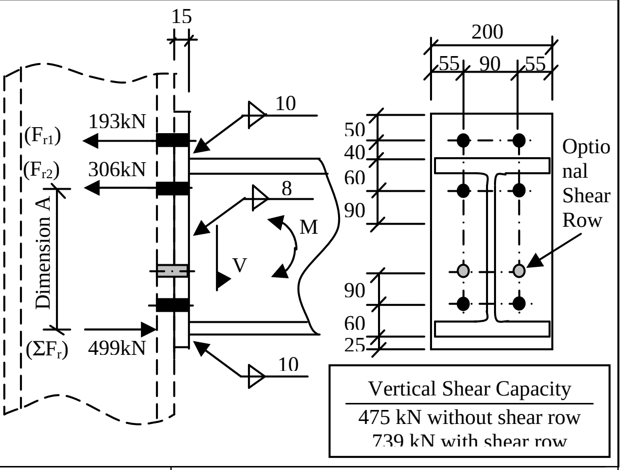
Table 4. Standard table for extended end-plate for 2 row M24 8.8 bolts, 200x15 end-plate (EEP,2BRM24,200W15EP)

2 ROW M24 8.8 B0LTS
200 x 15 DESIGN GRADE 43 EXTENDED END PLATE

Beam Side

BEAM – FLANGE S355
WEB S275

Beam Serial Size D x B x kg/m (T/ t)	Dimension 'A' (mm)	Moment Capacity (kNm)
400 x 160 x 48.4 (14/4)	333	186
450 x 160 x 50.2 (14/4)	383	211
450 x 180 x 60.1 (16/4)	382	210
500 x 160 x 52.0 (14/4)	433	236
500 x 180 x 61.9 (16/4)	432	235
550 x 200 x 73.3 (16/5)	482	260
600 x 200 x 80.5 (16/6)	532	285



Column Side

DESIGN GRADE S275				COLUMN	DESIGN GRADE S355			
Panel Shear Capacity (kN)	Tension Zone F_{R1} (kN)	Tension Zone F_{R2} (kN)	Compn. Zone	Serial Size	Compn. Zone	Tension Zone F_{R1} (kN)	Tension Zone F_{R2} (kN)	Panel Shear Capacity (kN)
1000	√	√	√	356 x 368 x 202	√	√	√	1302
849	√	√	√	177	√	√	√	1105
725	√	√	√	153	√	√	√	944
605	√	√	√	129	√	√	√	787
1037	√	√	√	305 x 305 x 198	√	√	√	1350
816	√	√	√	158	√	√	√	1062
703	√	√	√	305 x 305 x 137	√	√	√	915
595	√	√	√	118	√	√	√	774
503	√	√	√	97	√	√	√	649
882	√	√	√	254 x 254 x 167	√	√	√	1149
685	√	√	√	132	√	√	√	892
551	√	√	√	107	√	√	√	717
434	√	√	√	89	√	√	√	566
360	√	297	S(474)	73	√	√	√	465
459	√	√	√	203 x 203 x 86	√	√	√	598
353	√	√	√	71	√	√	√	460
322	√	297	S(481)	60	√	√	√	415
272	√	203	S(395)	52	√	√	296	351
245	√	111	√	46	√	√	199	316

Tension Zone:

√ Column satifactory for bolt row tension values shown for the beam side.

xxx Calculate reduced moment capacity using the reduced bolt row values.

Compression Zone:

√ Column capacity exceeds ΣF_r.

S (xxx) Column requires stiffening to resist ΣF_r (value is the column web capacity).

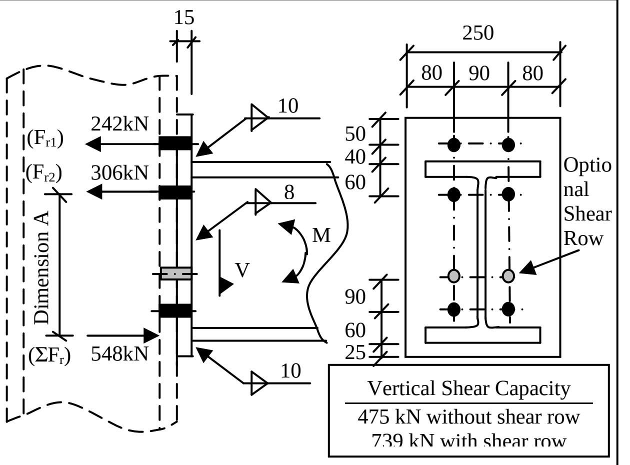
Table 5. Standard table for extended end-plate for 2 row M24 8.8 bolts, 250x15 end-plate (EEP,2BRM24,250W15TEP)

2 ROW M24 8.8 B0LTS
250 x 15 DESIGN GRADE 43 EXTENDED END PLATE

Beam Side

BEAM – FLANGE S355
WEB S275

Beam Serial Size D x B x kg/m (T/ t)	Dimension 'A' (mm)	Moment Capacity (kNm)
450 x 180 x 60.1 (16/4)	382	234
500 x 180 x 61.9 (16/4)	432	261
550 x 200 x 73.3 (16/5)	482	288
600 x 200 x 80.5 (16/6)	532	316
650 x 250 x 103.4 (18/6)	581	343
750 x 250 x 108.7 (18/6)	681	397



Column Side

DESIGN GRADE S275				COLUMN	DESIGN GRADE S355			
Panel Shear Capacity (kN)	Tension Zone F_{R1} (kN)	Tension Zone F_{R2} (kN)	Compn. Zone	Serial Size	Compn. Zone	Tension Zone F_{R1} (kN)	Tension Zone F_{R2} (kN)	Panel Shear Capacity (kN)
1000	√	√	√	356 x 368 x 202	√	√	√	1302
849	√	√	√	177	√	√	√	1105
725	√	√	√	153	√	√	√	944
605	√	√	√	129	√	√	√	787
1037	√	√	√	305 x 305 x 198	√	√	√	1350
816	√	√	√	158	√	√	√	1062
703	√	√	√	137	√	√	√	915
595	√	√	√	118	√	√	√	774
503	√	√	√	305 x 305 x 97	√	√	√	649
882	√	√	√	254 x 254 x 167	√	√	√	1149
685	√	√	√	132	√	√	√	892
551	√	√	√	107	√	√	√	717
434	√	√	√	89	√	√	√	566
360	√	297	S(479)	73	√	√	√	465
459	√	√	√	203 x 203 x 86	√	√	√	598
353	√	√	√	71	√	√	√	460
322	√	276	S(486)	60	√	√	√	415
272	√	154	√	52	√	√	269	351
245	204	100	√	46	√	√	151	316

Tension Zone:

√ Column satifactory for bolt row tension values shown for the beam side.

xxx Calculate reduced moment capacity using the reduced bolt row values.

Compression Zone:

√ Column capacity exceeds ΣF_r.

S (xxx) Column requires stiffening to resist ΣF_r (value is the column web capacity).

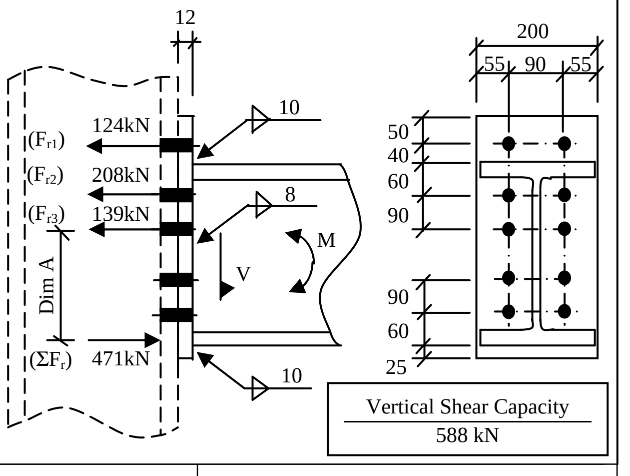
Table 6. Standard table for flush end-plate for 3 row M20 8.8 bolts, 200x12 end-plate (EEP,3BRM20,200W12TEP)

Beam Side

3 ROW M20 8.8 B0LTS
200 x 12 DESIGN GRADE 43 EXTENDED END PLATE

BEAM – FLANGE S355
WEB S275

Beam Serial Size D x B x kg/m (T/ t)	Dimension 'A' (mm)	Moment Capacity (kNm)
400 x 160 x 48.4 (14/4)	243	156
450 x 160 x 50.2 (14/4)	293	180
450 x 180 x 60.1 (16/4)	292	180
500 x 180 x 61.9 (16/4)	342	203
550 x 200 x 73.3 (16/5)	392	227
600 x 200 x 80.5 (16/6)	442	251
650 x 250 x 103.4 (18/6)	491	274
750 x 250 x 108.7 (18/6)	591	321



Column Side

DESIGN GRADE S275					COLUMN	DESIGN GRADE S355				
Panel Shear Capacity (kN)	Tension Zone F_{R1} (kN)	Tension Zone F_{R2} (kN)	Tension Zone F_{R3} (kN)	Compn. Zone	Serial Size	Compn. Zone	Tension Zone F_{R1} (kN)	Tension Zone F_{R2} (kN)	Tension Zone F_{R3} (kN)	Panel Shear Capacity (kN)
1000	√	√	√	√	356 x 368 x 202	√	√	√	√	1302
849	√	√	√	√	177	√	√	√	√	1105
725	√	√	√	√	153	√	√	√	√	944
605	√	√	√	√	129	√	√	√	√	787
1037	√	√	√	√	305 x 305 x 198	√	√	√	√	1350
816	√	√	√	√	158	√	√	√	√	1062
703	√	√	√	√	305 x 305 x 137	√	√	√	√	915
595	√	√	√	√	118	√	√	√	√	774
503	√	√	√	√	97	√	√	√	√	649
882	√	√	√	√	254 x 254 x 167	√	√	√	√	1149
685	√	√	√	√	132	√	√	√	√	892
551	√	√	√	√	107	√	√	√	√	717
434	√	√	√	√	89	√	√	√	√	566
360	√	√	√	S(465)	73	√	√	√	√	465
459	√	√	√	√	203 x 203 x 86	√	√	√	√	598
353	√	√	√	√	71	√	√	√	√	460
322	√	√	√	S(471)	60	√	√	√	√	415
272	√	√	√	S(386)	52	√	√	√	√	351
245	√	181	90	S(337)	46	S(435)	√	√	√	316

Tension Zone:
√ Column satifactory for bolt row tension values shown for the beam side.
xxx Calculate reduced moment capacity using the reduced bolt row values.
Compression Zone:
√ Column capacity exceeds ΣF_r.
S (xxx) Column requires stiffening to resist ΣF_r (value is the column web capacity).

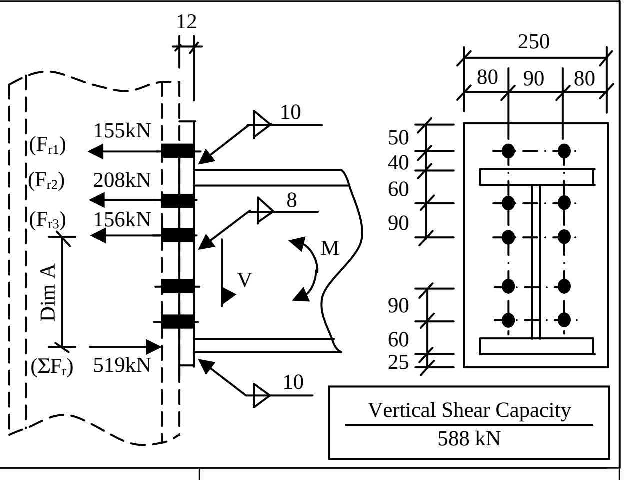
Table 7. Standard table for extended end-plate for 3 row M20 8.8 bolts, 250x12 end-plate (EEP,3BRM20,250W12TEP)

3 ROW M20 8.8 B0LTS
250 x 12 DESIGN GRADE 43 EXTENDED END PLATE

Beam Side

BEAM – FLANGE S355
WEB S275

Beam Serial Size D x B x kg/m (T/ t)	Dimension 'A' (mm)	Moment Capacity (kNm)
400 x 160 x 48.4 (14/4)	243	174
450 x 160 x 50.2 (14/4)	293	200
450 x 180 x 60.1 (16/4)	292	200
500 x 180 x 61.9 (16/4)	342	225
550 x 200 x 73.3 (16/5)	392	252
600 x 200 x 80.5 (16/6)	442	278
650 x 250 x 103.4 (18/6)	491	303
750 x 250 x 108.7 (18/6)	591	355



Column Side

Panel Shear Capacity (kN)	Tension Zone F_{R1} (kN)	F_{R2} (kN)	F_{R3} (kN)	Compn. Zone	COLUMN Serial Size	Compn. Zone	Tension Zone F_{R1} (kN)	F_{R2} (kN)	F_{R3} (kN)	Panel Shear Capacity (kN)
DESIGN GRADE S275						**DESIGN GRADE S355**				
1000	√	√	√	√	356 x 368 x 202	√	√	√	√	1302
849	√	√	√	√	177	√	√	√	√	1105
725	√	√	√	√	153	√	√	√	√	944
605	√	√	√	√	129	√	√	√	√	787
1037	√	√	√	√	305 x 305 x 198	√	√	√	√	1350
816	√	√	√	√	158	√	√	√	√	1062
703	√	√	√	√	305 x 305 x 137	√	√	√	√	915
595	√	√	√	√	118	√	√	√	√	774
503	√	√	√	√	97	√	√	√	√	649
882	√	√	√	√	254 x 254 x 167	√	√	√	√	1149
685	√	√	√	√	132	√	√	√	√	892
551	√	√	√	√	107	√	√	√	√	717
434	√	√	√	√	89	√	√	√	√	566
360	√	√	√	S(465)	73	√	√	√	√	465
459	√	√	√	√	203 x 203 x 86	√	√	√	√	598
353	√	√	√	√	71	√	√	√	√	460
322	√	√	√	S(471)	60	√	√	√	√	415
272	√	√	151	S(386)	52	S(484)	√	√	√	351
245	√	150	90	S(337)	46	S(435)	√	√	147	316

Tension Zone:
√ Column satifactory for bolt row tension values shown for the beam side.
xxx Calculate reduced moment capacity using the reduced bolt row values.
Compression Zone:
√ Column capacity exceeds ΣF_r.
S (xxx) Column requires stiffening to resist ΣF_r (value is the column web capacity).

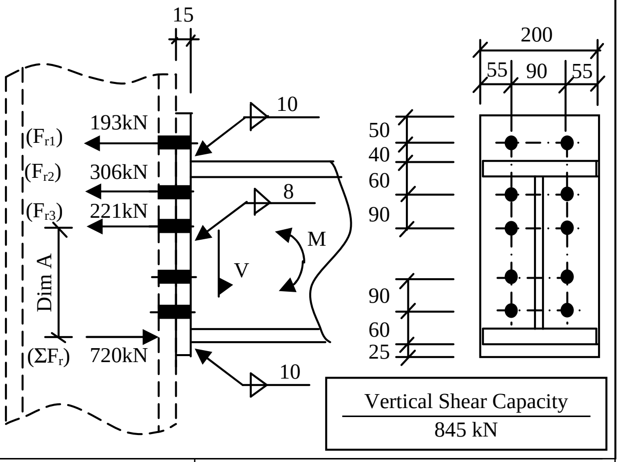
Table 8. Standard table for extended end-plate for 3 row M24 8.8 bolts, 200x15 end-plate (EEP,3BRM20,200W15TEP)

3 ROW M24 8.8 B0LTS
200 x 15 DESIGN GRADE 43 EXTENDED END PLATE

Beam Side

BEAM – FLANGE S355
WEB S275

Beam Serial Size D x B x kg/m (T/ t)	Dimension 'A' (mm)	Moment Capacity (kNm)
450 x 180 x 60.1 (16/4)	292	275
500 x 180 x 61.9 (16/4)	342	311
550 x 200 x 73.3 (16/5)	392	347
600 x 200 x 80.5 (16/6)	442	383
650 x 250 x 103.4 (18/6)	491	418



Column Side

DESIGN GRADE S275					COLUMN	DESIGN GRADE S355				
Panel Shear Capacity (kN)	Tension Zone F_{R1} (kN)	Tension Zone F_{R2} (kN)	Tension Zone F_{R3} (kN)	Compn. Zone	Serial Size	Compn. Zone	Tension Zone F_{R1} (kN)	Tension Zone F_{R2} (kN)	Tension Zone F_{R3} (kN)	Panel Shear Capacity (kN)
1000	√	√	√	√	356 x 368 x 202	√	√	√	√	1302
849	√	√	√	√	177	√	√	√	√	1105
725	√	√	√	√	153	√	√	√	√	944
605	√	√	√	S(651)	129	√	√	√	√	787
1037	√	√	√	√	305 x 305 x 198	√	√	√	√	1350
816	√	√	√	√	158	√	√	√	√	1062
703	√	√	√	√	137	√	√	√	√	915
595	√	√	√	√	305 x 305 x 118	√	√	√	√	774
503	√	√	√	S(596)	97	√	√	√	√	649
882	√	√	√	√	254 x 254 x 167	√	√	√	√	1149
685	√	√	√	√	132	√	√	√	√	892
551	√	√	√	√	107	√	√	√	√	717
434	√	√	√	S(601)	89	√	√	√	√	566
360	√	297	√	S(474)	73	S(612)	√	√	√	465
459	√	√	√	√	203 x 203 x 86	√	√	√	√	598
353	√	√	√	S(555)	71	S(723)	√	√	√	460
322	√	297	183	S(481)	60	S(621)	√	√	√	415
272	√	203	118	S(395)	52	S(479)	√	√	√	351
245	√	111	90	S(345)	46	S(443)	√	199	116	316

Tension Zone:

√ Column satifactory for bolt row tension values shown for the beam side.

xxx Calculate reduced moment capacity using the reduced bolt row values.

Compression Zone:

√ Column capacity exceeds ΣF_r.

S (xxx) Column requires stiffening to resist ΣF_r (value is the column web capacity).

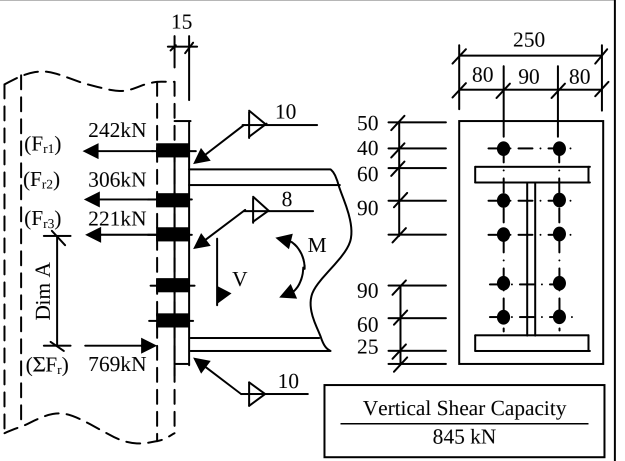
Table 9. Standard table for flush end-plate for 3 row M20 8.8 bolts, 200x12 end-plate (EEP,3BRM20,200W12TEP)

3 ROW M20 8.8 B0LTS
200 x 12 DESIGN GRADE 43 EXTENDED END PLATE

Beam Side

BEAM – FLANGE S355
WEB S275

Beam Serial Size D x B x kg/m (T/ t)	Dimension 'A' (mm)	Moment Capacity (kNm)
450 x 180 x 60.1 (16/4)	292	298
500 x 180 x 61.9 (16/4)	342	337
550 x 200 x 73.3 (16/5)	392	375
600 x 200 x 80.5 (16/6)	442	413
650 x 250 x 103.4 (18/6)	491	451
750 x 250 x 108.7 (18/6)	591	528



Column Side

DESIGN GRADE S275					COLUMN		DESIGN GRADE S355				
Panel Shear Capacity (kN)	Tension Zone			Compn. Zone	Serial Size		Compn. Zone	Tension Zone			Panel Shear Capacity (kN)
	F_{R1} (kN)	F_{R2} (kN)	F_{R3} (kN)					F_{R1} (kN)	F_{R2} (kN)	F_{R3} (kN)	
1000	√	√	√	√	356 x 368 x	202	√	√	√	√	1302
849	√	√	√	√		177	√	√	√	√	1105
725	√	√	√	√		153	√	√	√	√	944
605	√	√	√	S(656)		129	√	√	√	√	787
1037	√	√	√	√	305 x 305 x	198	√	√	√	√	1350
816	√	√	√	√		158	√	√	√	√	1062
703	√	√	√	√		137	√	√	√	√	915
595	√	√	√	S(749)	305 x 305 x	118	√	√	√	√	774
503	√	√	√	S(602)		97	S(777)	√	√	√	649
882	√	√	√	√	254 x 254 x	167	√	√	√	√	1149
685	√	√	√	√		132	√	√	√	√	892
551	√	√	√	S(806)		107	√	√	√	√	717
434	√	√	√	S(607)		89	S(790)	√	√	√	566
360	√	297	214	S(479)		73	S(618)	√	√	√	465
459	√	√	√	√	203 x 203 x	86	√	√	√	√	598
353	√	√	√	S(555)		71	S(730)	√	√	√	460
322	√	276	155	S(481)		60	S(627)	√	√	√	415
272	√	154	118	S(395)		52	S(515)	√	269	152	351
245	204	100	90	S(345)		46	S(451)	√	151	116	316

Tension Zone:
√ Column satifactory for bolt row tension values shown for the beam side.
xxx Calculate reduced moment capacity using the reduced bolt row values.
Compression Zone:
√ Column capacity exceeds ΣF_r.
S (xxx) Column requires stiffening to resist ΣF_r (value is the column web capacity).

Table 10. Percentage increase in moment capacity of the connection by increasing the number of bolt row from two to three bolt rows

Size of TWP beam	EEP,2BRM20,200W12TEP versus EEP,3BRM20,200W12TEP			EEP,2BRM20,250W12TEP versus EEP,3BRM20,250W12TEP			EEP,2BRM24,200W15TEP versus EEP,3BRM24,200W15TEP			EEP,2BRM24,250W15TEP versus EEP,3BRM24,250W15TEP		
	Two bolt rows	Three bolt rows	% increase	Two bolt rows	Three bolt rows	% increase	Two bolt rows	Three bolt rows	% increase	Two bolt rows	Three bolt rows	% increase
400 x 160 x 48.4 (14/4)	123	156	26.8%	136	174	27.9%	N/A	N/A	N/A	N/A	N/A	N/A
450 x 160 x 50.2 (14/4)	139	180	29.5%	154	200	29.9%	N/A	N/A	N/A	N/A	N/A	N/A
450 x 180 x 60.1 (16/4)	139	180	29.5%	154	200	29.9%	210	275	31.0%	234	298	27.4%
500 x 180 x 61.9 (16/4)	156	203	30.1%	172	225	30.8%	235	311	32.3%	261	337	29.1%
550 x 200 x 73.3 (16/5)	172	227	32.0%	190	252	32.6%	260	347	33.5%	288	375	30.2%
600 x 200 x 80.5 (16/6)	189	251	32.8%	208	278	33.7%	285	383	34.4%	316	413	30.7%

Table 11. Percentage increase in moment capacity of the connection by increasing the size of end-plate from 200mm to 250mm wide

Size of TWP beam	EEP,2BRM20,200W12TEP versus EEP,2BRM20,250W12TEP			EEP,2BRM24,200W15TEP versus EEP,2BRM24,250W15TEP			EEP,3BRM20,200W12TEP versus EEP,3BRM20,250W12TEP			EEP,3BRM24,200W15TEP versus EEP,3BRM24,250W15TEP		
	200mm	250mm	% increase	200mm	250mm	% increase	200mm	250mm	% increase	200mm	250mm	% increase
400 x 160 x 48.4 (14/4)	123	136	10.6	N/A	N/A	N/A	156	174	11.5	N/A	N/A	N/A
450 x 160 x 50.2 (14/4)	139	154	10.8	N/A	N/A	N/A	180	200	11.1	N/A	N/A	N/A
450 x 180 x 60.1 (16/4)	139	154	10.8	210	234	11.4	180	200	11.1	275	298	8.4
500 x 180 x 61.9 (16/4)	156	172	10.3	235	261	11.1	203	225	10.8	311	337	8.4
550 x 200 x 73.3 (16/5)	172	190	10.5	260	288	10.8	227	252	11.0	347	375	8.1
600 x 200 x 80.5 (16/6)	189	208	10.1	285	316	10.9	251	278	10.8	383	413	7.8

Table 12. Percentage increase in moment capacity of the connection by increasing the thickness of end-plate from 12mm to 15mm

	EEP,2BRM20,200W12TEP versus EEP,2BRM24,200W15TEP			EEP,2BRM20,250W12TEP versus EEP,2BRM24,250W15TEP			EEP,3BRM20,200W12TEP versus EEP,3BRM24,250W15TEP			EEP,3BRM20,250W12TEP versus EEP,3BRM24,250W15TEP		
Size of TWP beam	M20/EP 12mm	M24/EP 15mm	% increase	M20/EP 12mm	M24/EP 15mm	% increase	M20/EP 12mm	M24/EP 15mm	% increase	M20/EP 12mm	M24/EP 15mm	% increase
400 x 160 x 48.4 (14/4)	123	186	51.2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
450 x 160 x 50.2 (14/4)	139	211	51.8	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
450 x 180 x 60.1 (16/4)	139	210	51.1	154	234	51.9	180	275	52.8	200	298	49.0
500 x 180 x 61.9 (16/4)	156	235	50.6	172	261	51.7	203	311	53.2	225	337	49.8
550 x 200 x 73.3 (16/5)	172	260	51.2	190	288	51.6	227	347	52.9	252	375	48.8
600 x 200 x 80.5 (16/6)	189	285	50.8	208	316	51.9	251	383	52.6	278	413	48.6

CONCLUSIONS

This study concluded that it is possible to determine the moment capacity of extended end plate connections connected to a column flange by adopting the method proposed by SCI, even for different geometric parameters such as the TWP section. The capacities of the connection depend 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. For the size of column, the reduction of moment capacity is due to the effect of compression of the beam flange to the column flange without the need of stiffener. The suggested weld size for flange and web is strong enough to prevent any failure at the weld. The increment of moment capacity of the connection can be concluded as follows:

- 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 in the size of end-plate from 200mm to 250mm has contributed to an increase in the moment capacity in the range of 2.7% to 5.1% which is not 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 depends on number of bolt used in the connection. However, the lower bolt row contributed to most of 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.

The use of 'knee joint method' to predict the moment resistance connection from M-Φ curves showed good agreement with the component method suggested by Steel Construction Institute. It was concluded that the moment resistance of the connection developed in the elastic-plastic region as shown by the knee-joint method.

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