BOLTED END PLATE BEAM TO COLUMN CONNECTIONS – ARE THEY SEMI RIGID?

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Thesis submitted in partial fulfillment of the requirements for the degree of Master of Engineering

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Abstract

The most commonly used moment resisting connections are bolted end plate beam to column connections. Connections are usually designed as simple or continuous although the actual behavior is known to fall between these two extreme cases. The use of semi- continuous connection results substantial savings in steel weight of the overall construction. Extended endplate, Flush end plate and partial depth connections are the widely used type of connections in steel frame construction. To understand the real behavior of semi- continuous connection, full scale laboratory test is the most accurate approach, but it is time consuming and costly to undertake. Therefore other methods were developed to predict the capacity of connections.

Thus, in this study 48 extended end plate and 48 flush end plate connections are analyzed to find the connection's behavior with variations in bolt diameter, end plate thickness, and grade (4.6/8.8) and bolt gauge length. A method proposed by Steel Construction Institute (SCI) is taken into account for analyzing. The analyzed connections are classified based on strength and stiffness.

Specially dedicated to my beloved family and friends…

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First of all, I would like to express gratitude to my supervisors, Dr.(Mrs) M.T.P Hettiarachchi and Dr.K.Baskaran for their valuable advices and guidance during this research study. I would also like to express my sincere appreciation to staff members of civil engineering department of University of Moratuwa for their kind assistances.

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LIST OF ABBREVIATIONS

Abbreviation	Description
A_e	Sum of the effective net areas of all the elements of the cross section
A_{eff}	Effective cross-sectional area
A_g	Gross cross-sectional area
A_n	Total net area
A_s	Shear area of a bolt
A_t	Tensile stress area as specified in the appropriate bolt standard
A_v	Shear area of member
a_p	Weld throat thickness
b_1	Stiff bearing length
b_e	Distance to the nearer end of the member from the end of the stiff bearing
d	Nominal diameter of the bolt
e	End distance
F_{vp}	Column web panel zone the local shear force
P_{bb}	Bearing capacity of the bolt
p_{bs}	Bearing strength of the connected part
p_c	Compressive strength
p_s	Shear strength of a bolt
P_T	Transverse capacity per unit length of weld

Pt	Tension strength of the bolt
P_{bw}	Bearing capacity of the web
Py	Yield strength of the connected part.
P_v	Shear capacity
p_{yw}	Design strength of the web
r	Root radius
s	Leg length of a fillet weld
T	Thickness of a flange
T	Thickness of a web
t_p	Thickness of the connected part

LIST OF APPENDICES

1. INTRODUCTION

1.1 General

Steel is one of the most widely used construction material for structural systems in modern construction. The successful performance of every structural steel frame in terms of both in service behavior and economy of fabrication and erection is dependent as much on its connections and size of structural members.

In all kind of steel structures, the selection of the type of connection is a matter of great importance to the economy of the structure. A type of connection which in recent years has attracted ever increasing interest is the bolted end plate connection. This connection is very suitable with steel skeleton buildings for beam to column and beam to beam connections.

1.2Bolted end plate connection

The most commonly used moment resisting connections are bolted end plate beam to column connections. The primary advantage of such connections is simplicity of their on-site assembling. In addition, the end plate connections make it possible to erect a steel framework in any climatic condition or to dismount the framework without making any damage to its structural members. Bolted end plate connections include their high reliability under dynamic loadings and facility of the connection supervision. Compared with welded connection, end plate connections have considerable advantages with respect to fabricationand erection, since all elements may be completed in the steel workshop and thus field welding may be avoided. This is the special importance where structures are to be erected in areas lacking skilled labor.

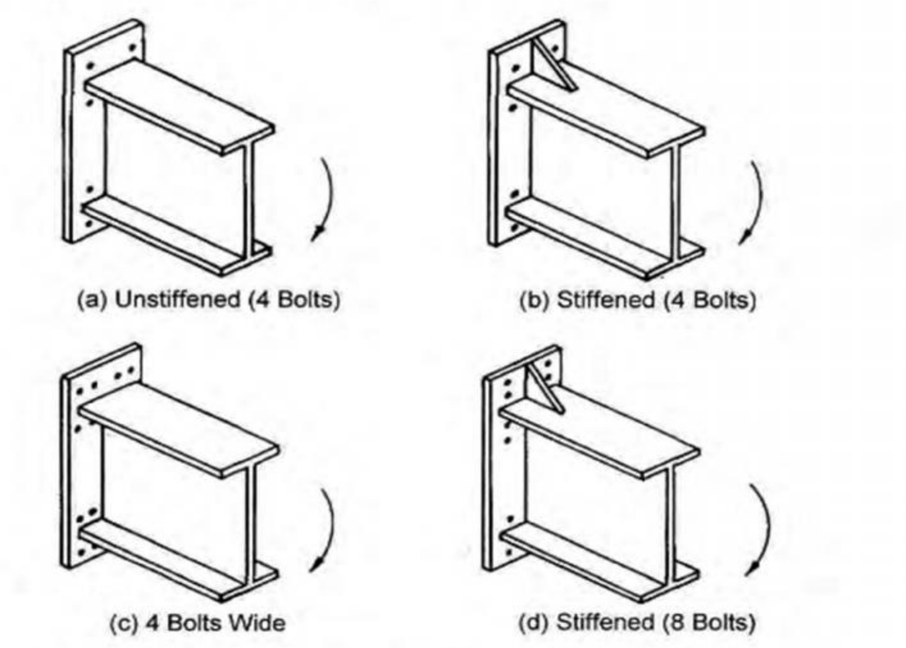

Figure 1.1 Extended end plate connections

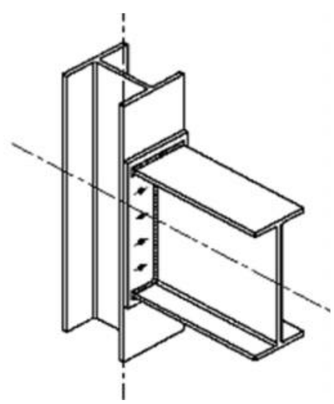

Figure 1.2Flush end plate connections

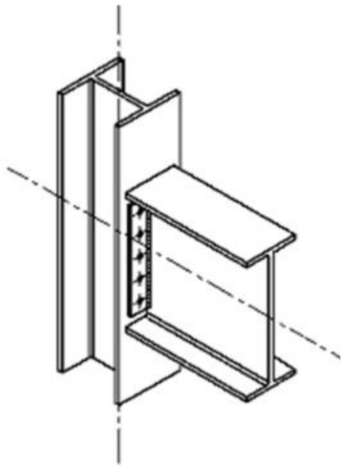

Figure 1.3Partial depth end plate connections

A typical end-plate connection is composed of a steel plate welded to the end of a beam section with attachment to an adjacent member using rows of fully tensioned bolts. The connection may join two beams (splice plate connection) or a beam and a column. Bolted end-plate connections are classified as extended end plate, flush end plate or partial depth end plate, with or without stiffeners as shown in Figure 1.1, Figure 1.2 and Figure 1.3 respectively.

Extended end plates are used for beam-to-column moment connections. Extended end plate connection is detailed such that the end plate extends beyond the tension flange a sufficient distance to allow a location of bolts. The purpose of this extended end plate is to increase the lever arm of bolts and the ability to carry more loads.

A flush end plate connection is detailed such that the end plate does not extend beyond the beam flanges and all the bolts are located between the flanges.

A partial depth end plate connection is detailed that the end plate is within the beam web.

In practice, the end plate is often detailed to extend the full depth of the beam and welded to the bottom flange. This makes the connection relatively stiffer than a partial depth end plate.

Column flange and web thickness, end plate thickness, beam depth, bolt size and grade affect the behavior of the end plate connection. So, these parameters should be taken into account when analyzing the end plate connection. This makes analyzing process more complicated.

This connection is relatively inexpensive but has the disadvantage that there is no room for site adjustment.

Extended end plate connections are widely used in steel structures as moment resistant connections and as an alternative to fully welded ones that has been considered for use in steel frame structures.

1.3 Moment – Rotation (M-ϴ) curve

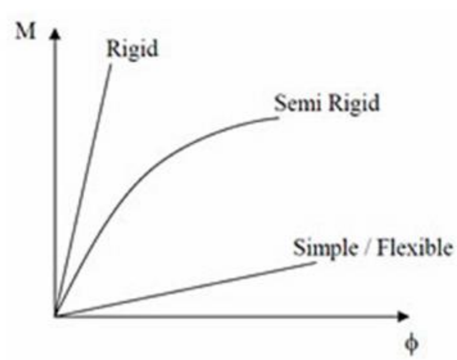

Figure 1.4: Comparison of M-ϴ curves for different type of Connections.

The (M-ϴ) curve is one of the essential behavior of the steel joint. The characteristic of (M-ϴ) curve will represent the connection’s stiffness, strength and ductility. All steel connections have a unique (M-ϴ) Relationship. The characteristic of the (M-ϴ) curve depends on a lot of parameters such as the thickness of end plate, size and number of bolts, configuration of connection, etc. Figure 1.4 shows the comparison of M-ϴ curve for different type of connections.

Accurate modelling of their moment rotation M-ϴ relationship is necessary if the effects of connections are to be considered in structural analysis. The moment-rotation characteristics will have to be determined based on experiments conducted for the specific design. Attention is focused on moment rotation characteristics as this is the most important influence on the response of either individual members or entire frames.

As the connection moment and rotation increase, the slope of the curve decreases resulting in ever decreasing stiffness. It is nonlinear relationship that makes connection behavior difficult to incorporate into structural design.

It is impossible to analyze connections accurately as it depends on all the elements involved in the connection including bolts, welds, beam and column sections, connection geometry and end plate. The nonlinear behavior of these components makes the situation more complex. The most accurate method is to carry out experimental analysis or a finite element model to analyze the connections.

Numerous effort and hard work have been put in the field of research to estimate the actual behavior of steel connections. Several mathematical models of the M-Θ relationship have been developed as a result, but each model has its own advantages and drawbacks.

1.4 Classification of joints based on Eurocode 3 (EC3,2005)

BS EN 1993-1-8:2005requires that joints are classified by stiffness or by strength. The stiffness classification is relevant for elastic analysis of frames while the strength classification is for frames analysed plastically. The standard defines joint models as simple, semi-continuous or continuous, depending on stiffness and strength.

1.4.1 Classification by stiffness

As defined in Clause 5.2.2 of EN 1993-1-8:2005, joint may be classified as rigid, nominally pinned or semi rigid according to its rotational stiffness, by comparing its initial rotational stiffness $S_{j, ini}$ with the classification boundaries shown in Figure 1.5.

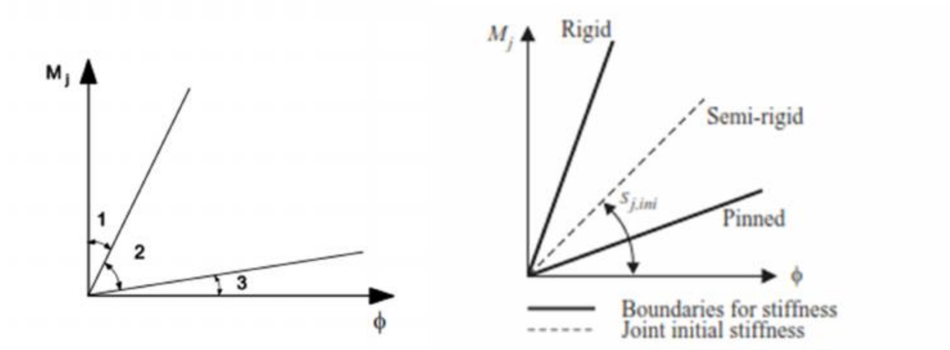

Figure 1.5 Classification of joints by stiffness according to Eurocode 3

In Figure 1.5, zone 3 represents nominally pinned connection, which should be capable of transmitting internal forces without developing significant moments which might adversely affect members or the structure as a whole.

Zone 3; nominally pinned, if $S_{j,ini} \leq 0,5\, EI_b / L_b$

E- Elastic modulus of steel section

I_b – Second moment of area of a beam

L_b - span of a beam (centre to centre of columns)

$S_{j,ini}$– initial rotational stiffness

In Figure 1.5, zone 1, rigid connection, which have sufficient rotational stiffness.

Zone 1 – rigid, if $S_{j,ini} \geq k_b EI_b / L_b$

where kb=8 for frames where the bracing system reduces the horizontal displacement by at least 80%.

K_b = 25 for other frames provided that in every storey

$K_b/K_c \geq 0,1$

Semi rigid connections (zone 2) fall between simple and rigid connections. Semi rigid joints should be capable of transmitting the internal forces and moments.
All joints in zone 2 should be classified as semi rigid.
Beam span mainly affects the stiffness classification.

1.4.2 Classification by strength

As defined in Clause 6.3 of EN 1993-1-8:2005, a joint may be classified as full strength, nominally pinned or partial strength as shown in Figure 1.6 by comparing its design moment resistance $M_{j,Rd}$ with the design moment resistances of the members that it connects.
A joint may be classified as nominally pinned if its design moment resistance $M_{j,Rd}$ is not greater than 0.25 times the design moment resistance required for a full strength joint.
The design resistance of a full strength joints should be not less than that of the connected members.
Partial strength joints fall between full strength and nominally pinned connection.

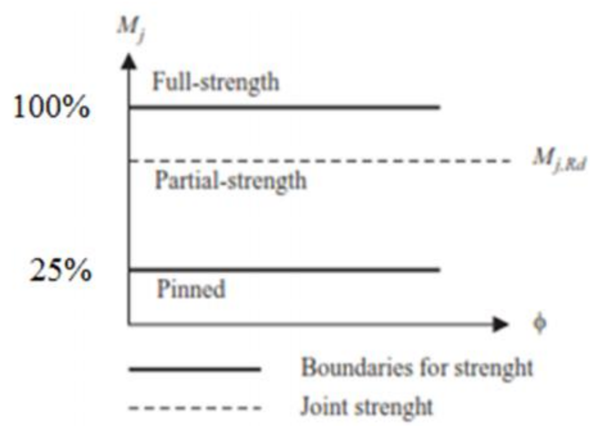

Figure 1.6 Classification of joints by strength according to Eurocode 3

1.5 Classification of connections

Joints need to be modeled for the global frame analysis and that three different types of joint modeling are introduced; simple, semi-continuous and continuous as shown in Figure 1.7 .

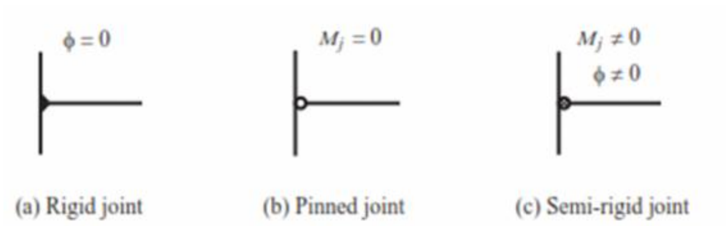

Figure 1.7 Modeling of joints (elastic global analysis)

The selection of the more appropriate one by the user is a quite important step which will strongly influence the number and the nature of the checks. Joint behavior affects the structural frame response and shall therefore be modeled for the frame analysis and design.

Classification criteria are used to define the stiffness class and the strength class to which the joint belongs and also to determine the type of joint modeling which shall be adopted for analysis.

Global frame analysis is aimed at deriving the values of the internal forces and of the displacements in the considered structure when this one is subjected to a given set of loads.

Effects to consider in global analysis

1. deformability and stiffness of the structure and supports;
2. stability of the structure (global, members and local);
3. behaviour of cross-sections (classification of sections);
4. behaviour of joints (strength and stiffness);
5. Imperfections (global and in members).

There are four types of global analysis

Analysis Type	Initial Geometry	Deformed Geometry	Linear material behaviour	Non-linear material behaviour
First-order elastic	✓		✓	
Second-order elastic		✓	✓	
First-order plastic	✓			✓
Second-order elastic		✓		✓

Table 1.1 Joint modeling and frame analysis

Modelling	**Type of frame analysis**		
	Elastic analysis	Rigid-plastic analysis	Elastic-perfectly plastic and elasto-plastic analysis
Continuous	Rigid	Full-strength	Rigid/full-strength
Semi-continuous	Semi-rigid	Partial-strength	Rigid/partial-strength Semi-rigid/full-strength Semi-rigid/partial-strength
Simple	Pinned	Pinned	Pinned

In the case of an elastic global frame analysis, only the stiffness properties of the joint are relevant for the joint modeling. In the case of a rigid plastic analysis, the main joint feature is the resistance. In all other cases, both the stiffness and resistance properties govern the joints. These possibilities are illustrated in Table 1.1.

1.6 Research Objective

Designing connections can be a long and tedious process. The objective of the research is to provide guideline to designer on the impact of changes of selected parameters (bolt diameter, end plate thickness, bolt grade and bolt gauge length) on moment resistance (M_R) and initial stiffness ($S_{j,ini}$) of steel bolted end plate connection.

1.7 Scope of work

The bolted end plate connections areanalyzed based on EC3 to find the connection's behavior with variations in bolt diameter (16mm, 20mm and 24mm), end plate thickness (12mm, 15mm, 20mm and 25mm), bolt grade (4.6/8.8) and bolt gauge length (100mm and 140mm).Then, the analyzed connections are classified based on strength and stiffness.

2. LITERATURE REVIEW

In order to understand the actual behaviour of beam-to-column connection, full scale experiments are the reliable method to determine the moment – rotation characteristics of semi rigid connections. From the survey of experimental work, it is predicted that all types of connection present a nonlinear and inelastic behavior as a semi rigid connection. Since experimental study is expensive and time-consuming, three dimension finite element analysis is the best method to study the connection behaviour. It offers more accurate result, time-saving and cost-saving in terms of material.

An experimental investigation of eight statically loaded extended end plate moment connections was undertaken by Coelho, Bijlaard & Silva (2004) to provide insight into the behaviour of this joint type up to collapse. The specimens were designed to confine failure to the end plate and bolts without development of the full plastic moment capacity of the beam considering the effect on end plate thickness and steel grade. The joint moment resistance increases with an increase of end plate thickness and with the yield stress of the plate and the joint rotational stiffness also increases with the end plate thickness.

Maggi, Goncalves, Leon & Ribeiro (2005) present the parametric analyses on the behaviour of bolted extended end plate connections using finite element modelling tools. The analytical models took into account material nonlinearities, geometrical discontinuities and large displacements. Six numerical models and associated experimental specimen’s overall stiffness, displacements of the end plate and axial forces in the bolts were discussed. T stub failure models were also used for the calculations of the flexural strength for the end plate. Comparison between numerical and experimental data for moment rotation curves, displacements of the end plate and forces on bolts showed satisfactory agreement.

Eight experimental tests have been carried out for extended end plate connections by Tahir & Hussein (2008) consists of variable parameters such as size and thickness of end plate, size and number of bolts, size of columns and beams. Geometric parameters such as the thickness of end-plate, the number and size of bolts and the use of deeper beam have contributed significantly to the increase in moment resistance andstiffness of the connections. The study concluded that all the tested extended end-plate connections are eligible to be classifiedas partial strength connections and the results of maximum moment resistance showed good agreement between experimentaland predicted component method in most cases.

Series of tests on beam to-column connections have been carried out for flush end-plate (FEP) connections by Tahir, Husseein, Sulaiman, &Mohamed (2009) to validate the component method introduced by Steel Construction Institute based on EC3. The moment versus rotation of the test results have been plotted and compared with the moment resistancederived from the component method. Tests on the flush end-plate connections revealed that the behavior of the connections satisfied the requirements for partial strength connections with moment resistance of the connections recorded more than 25% of the moment resistance of the connected beam and the rotation capacity of the connections recorded more than 20 mrad.

Sebastian-Lician Buzuleac (2013) investigates the characteristics of beam to column bolted end plate connections using ANSYS software with various types and details. The interaction between the end plate and column flange, geometric and material nonlinearities have been considered in the finite element analysis. From finite element analysis, moment resistance, rotational stiffness and tension force distribution of bolts have been presented. Column stiffeners and end plate stiffeners mainly influence the joint rotational stiffness. The end plate thickness influences the joint stiffness more than bolt diameter.

Shaker & Elrahman (2014) presents the behaviour of flush and extended end plate beam to column joints under bending and axial force. This paper is to discuss the behaviour of flush and extended end plate beam to column joints under bending and axial force. The analytical investigation utilizes nonlinear finite element modelling techniques using ANSYS program, considering both geometric and material nonlinearities. The level of axial forces in the joint may be significant in the case of sway frames under horizontal loads, pitched roof portal frames and irregular frames.

Ismail, Fahmy, Khalifa &Mohamed (2015) presents a three dimension finite element model using ABAQUS software to study the effect of different geometrical parameters on the ultimate behaviour of the end plate steel connections subject to monotonic loading. The finite element results are verified with experimental results. Using verified FEM, parametric study is carried out to study the ultimate behaviour with variations in bolt diameter, end plate thickness, and length of column stiffener and angle of rib stiffener. Bolt diameter and end plate thickness have significant influence on the ultimate moment and rotation capacity. Column tension stiffener having moderate end-plate thickness, increasing the bolt diameter from 20 to 24 mm increases the ultimate moment and rotation capacity by about 14% and 63%, respectively. Increasing bolt diameter from 24 to 27mm increases the ultimate moment and rotation capacity by about 3.5% and 3.5%, respectively.Column tension stiffeners slightly increases the ultimate moment but decreases the rotation capacity.

Jaspart J-P and K.Weynand (2015) presents design of moment resisting connections in steel structures. The Eurocodes give new and advanced options to design efficient and economic steel structures. The design of joints plays a major role in that process. Thus the detailing of joints and the methods of considering the joints properties in the frame analysis will significantly influence the costs of a steel structure. This has been demonstrated by various investigations. The results of the investigations presented in this paper is the use of semi rigid joints will become more and more interesting to find economical solutions for steel structures.

Faridmehr, Tahir & Lahmer (2016) attempt to recognise an adequate classification for a semi rigid beam to column connection by investigating strength, stiffness and ductility. An experimental test was carried out to investigate the moment rotation features of flush end plate connections including variable parameters. The initial elastic stiffness and ultimate moment capacity of connections were determined by an extensive analytical procedure using AISC and EC3. The results showed that EC3 provided a more reliable classification index for flush end plate connections and the actual response of semi rigid connections are non-linear.

3. ANALYSIS AND DESIGN OF CONNECTIONS ACCORDING TO EC3

3.1 Analysis and design of connections

The standards are not user friendly and the majority of connections designs are to be conducted with the aid of handbooks, such as SCI/BCSA, Joints in steel construction.

Eurocode3(EN 1993-1-8:2005) clearly describes the connection classification, design methods and procedures for beam-to-column connections.
In modeling this connection, the idealized T stub consists of the end plate and the beam web to which it is welded as one T stub, which is bolted to the column flange which with the column web forms the other T stub.

The beam reaction is transferred by weld shear to the end plate, by shear and bearing to the bolts, and by shear and bearing to the supporting member.
End plate connection transmits moment by coupling tension in the bolts with compression in the bottom flange. The bolt row furthest from the compression flange may attract the most tension and has been to assume a triangular distribution of bolt forces.
As described in SCI/BCSAhandbook (SCI/BCSA, 1995),a typical end plate connection can be split in to four zones as shown in Figure 3.1– the tension zone, the compression zone, horizontal shear and vertical shear.

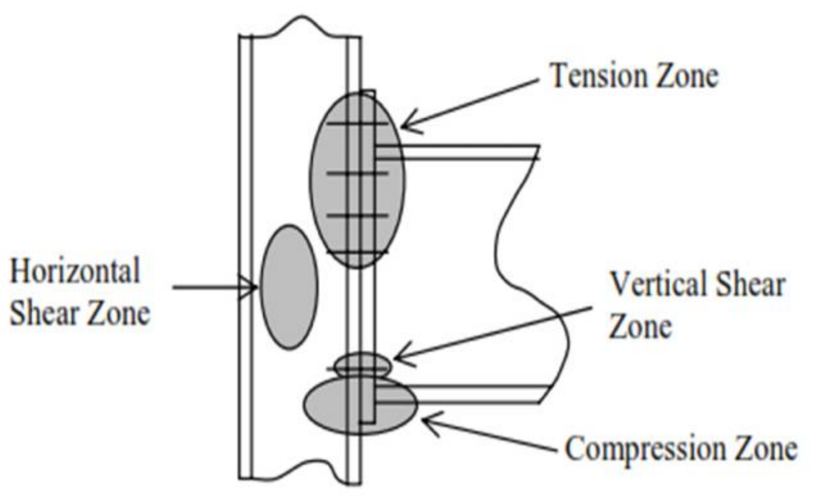

Figure 3.1 Critical zones of a Joint (SCI/BCSA, 1995).

According to SCI' guide, for a satisfactory design of the connection, following checks need to be done for the beam, the column, the bolts and the welds as shown in Figure 3.2.

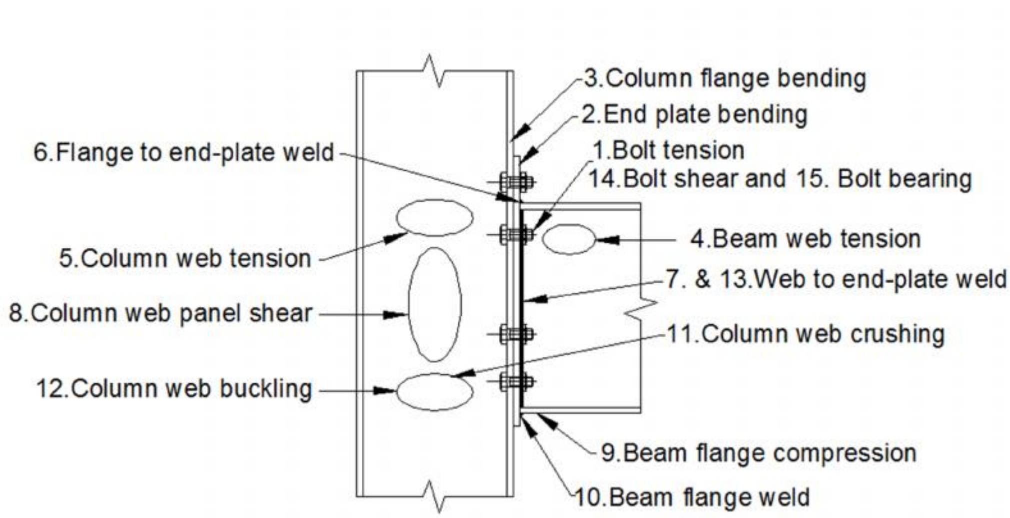

Figure 3.2 Critical checks of a Joint (SCI/BCSA, 1995).

As outlined in SCI handbook, a particular connection shall be checked against the following checks.

1. Bolt tension

As defined in Clause 3.6.1 (1)of EN 1993-1-8:2005 the design resistance for an individual fastener subjected to shear and/or tension is given in Table 3.4. For a bolt resistance to tension;

$$F_{t,Rd} = \frac{k_2\ f_{ub}\ A_s}{\gamma_{M2}}$$

2. End-plate in bending

End-plate bending capacity should be calculated by Clause6.2.6.5of EN 1993-1-8:2005. The design resistance and failure mode of an end-plate in bending, together with the associated bolts in tension, should be taken as similar to those of an equivalent T-stub flange, for both:

– each individual bolt-row required to resist tension;

– each group of bolt-rows required to resist tension.

3. Column flange in bending

As described in Clause 6.2.6.4.1 of EN 1993-1-8:2005;The design resistance and failure mode of an unstiffened column flange in transverse bending, together with the associated bolts in tension, should be taken as similar to those of an equivalent T-stub flange, see 6.2.4, for both:

– each individual bolt-row required to resist tension;

– each group of bolt-rows required to resist tension.

4. Beam web in tension

According to the Clause 6.2.6.4.3of EN 1993-1-8:2005; in a bolted end-plate connection, the design tension resistance of the beam web should be obtained from:

$$F_{t,wb,Rd} = b_{eff,t,wb}\, t_{wb}\, f_{y,wb} / \gamma_{M0}$$

5. Column web tension

As defined in Clause6.2.6.3of EN 1993-1-8:2005; the design resistance of an unstiffened column web subject to transverse tension should be determined from:

$$F_{t,wc,Rd} = \frac{\omega\, b_{eff,t,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M0}}$$

.

6. Column web panel shear

As defined in Clause6.2.6.1of EN 1993-1-8:2005; For a single-sided joint, or for a double-sided joint in which the beam depths are similar, the design shear resistance Vwp,Rd of an unstiffened column web panel, subject to a design shear force $V_{wp,Ed}$, see5.3(3), should be obtained using:

$$V_{wp,Rd} = \frac{0,9\, f_{y,wc}\, A_{vc}}{\sqrt{3}\, \gamma_{M0}}$$

Where,

A_{vc} is the shear area of the column, see EN 1993-1-1. (Clause: 5.4.6)

7 &8. Column web in compression

According to the Clause6.2.6.2of EN 1993-1-8:2005; transverse compression capacity of Column web shall be determined from:

$$F_{c,wc,Rd} = \frac{\omega\, k_{wc}\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M0}} \quad \text{but} \quad F_{c,wc,Rd} \leq \frac{\omega\, k_{wc}\, \rho\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M1}}$$

9. Bolt Shear

As defined in Clause 3.6.1 (1)of EN 1993-1-8:2005 the design resistance for an individual fastener subjected to shear is given in Table 3.4.

For a bolt resistance to shear;

$$F_{v,Rd} = \frac{\alpha_v \ f_{ub} \ A}{\gamma_{M2}}$$

- where the shear plane passes through the threaded portion of the bolt (A is the tensile stress area of the bolt As):

  - for classes 4.6, 5.6 and 8.8: $a_v = 0.6$
  - for classes 4.8, 5.8, 6.8 and 10.9: $a_v = 0.5$

- where the shear plane passes through the unthreaded portion of the bolt (A is the gross cross section of the bolt): $a_v = 0.6$

10. Bolt Bearing

As defined in Clause 3.6.1 (1)ofEN 1993-1-8:2005 the design resistance for an individual fastener subjected to bearing is given in Table 3.4. For a bolt resistance to bearing;

$$F_{b,Rd} = \frac{k_1 \, a_b \, f_u \, d \, t}{\gamma_{M2}}$$

3.2 Classification of connection

As defined in Clause 5.2 of EN 1993-1-8:2005, connections are classified by their stiffness and strength.

3.2.1 Classification by stiffness

As defined in Clause 5.2.2 of EN 1993-1-8:2005, joint may be classified as rigid, nominally pinned or semi rigid according to its rotational stiffness, by comparing its initial stiffness $S_{j,\,ini}$ with the classification boundaries.

3.2.1.1 Initial stiffness

As defined in Clause 6.3 of EN 1993-1-8:2005, rotational stiffness of a joint should be determined from the flexibilities of its basic components, each represented by an elastic stiffness coefficient ki obtained from clause 6.3.2.

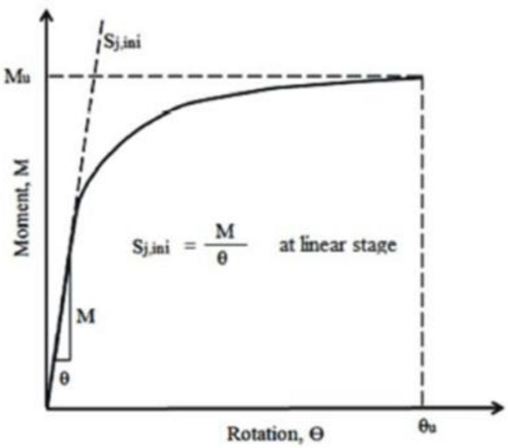

Figure 3.3 Initial stiffness of a Joint

Initial rotational stiffness as shown in Figure 3.3 Sj,ini of the joint is given by

$$S_j = \frac{Ez^2}{\mu \sum_i \frac{1}{k_i}}$$

where

k_i is the stiffness coefficient for basic joint component i;

z is the lever arm,

μ is the stiffness ratio $S_{j,ini}/S_j$,

The basic components that should be taken into account when calculating the stiffness for a bolted end plate connection are given in Table 6.10 and Table 6.11 of EN 1993-1-8:2005.

Beam-to-column joint with bolted end-plate connections	Number of bolt-rows in tension	Stiffness coefficients k_i to be taken into account
Single-sided	One	k_1; k_2; k_3; k_4; k_5; k_{10}
	Two or more	k_1; k_2; k_{eq}

For end plate connections with two or more bolt rows in tension, the basic components related to these entire bolt rows should be represented by a single equivalent stiffness coefficient keq determined from Clause 6.3.3 of EN 1993-1-8:2005.

$$k_{eq} = \frac{\sum_r k_{eff,r} h_r}{z_{eq}}$$

h_r - distance between bolt row r and the centre of compression

$k_{eff,r}$ - Effective stiffness coefficient for bolt row r

Z_{eq}– equivalent lever arm

As defined in Clause 6.3.3 of EN 1993-1-8:2005, The effective stiffness coefficient $k_{eff,r}$ for bolt row r should be determined from;

$$k_{eff,r} = \frac{1}{\sum_i \frac{1}{k_{i,r}}}$$

$K_{i,r}$—stiffness coefficient representing component i relative to bolt row r

The equivalent lever arm Z_{eq} should be determined from;

$$z_{eq} = \frac{\sum_r k_{eff,r} h_r^2}{\sum_r k_{eff,r} h_r}$$

As defined in Clause 6.3.3 of EN 1993-1-8:2005, beam to column joint with an end plate connection, K_{eq} should be based upon the stiffness coefficient K_i for;

- Column web in tension (K_3)
- Column flange in bending (K_4)
- End plate in bending (K_5)
- Bolts in tension (K_{10})

3.2.1.2 Classification by strength

As defined in Clause 6.3 of EN 1993-1-8:2005, a joint may be classified as full strength, nominally pinned or partial strength by comparing its design moment resistance $M_{j,Rd}$ with the design moment resistances of the members that it connects.

The moment resistance of the connection is calculated from

$$M_{j,Rd} = \sum_r h_r F_{tr,Rd}$$

where:

$F_{tr,Rd}$ is the effective design tension resistance of bolt-row r ;

h_r is the distance from bolt-row r to the centre of compression;

r is the bolt-row number.

In annex A and annex B, detailed calculation of extended end plate and flush end plate connections are mentioned.

4. ANALYSIS, RESULTS AND DISCUSSION

4.1 General

In this study, several configurations of extended end plate and flush end plate steel beam-to-column connections are selected to study the effect of different geometrical parameters such as bolt diameter, end plate thickness, bolt grade and bolt gauge length on the behaviour of the connection. Moment of resistance (M_R) and initial stiffness (Sj,ini) for the each connection is calculated based on Eurocode 3 (EC3, 2005) standards. The ultimate behavior of the bolted end plate steel connection is discussed in detail and recommendations for the design purpose are made.

4.2 Configurations

As shown in Table 4.2 and Table 4.3, extended end plate connection configurations and flush end plate connection configurations are selected for parametric study. Configurations are designated as EEP 1a to EEP 3d and FEP1a to FEP3d.

The typical geometrical configuration of the extended end plate and flush end plate connections are shown in Figure 4.1 and Figure 4.2.

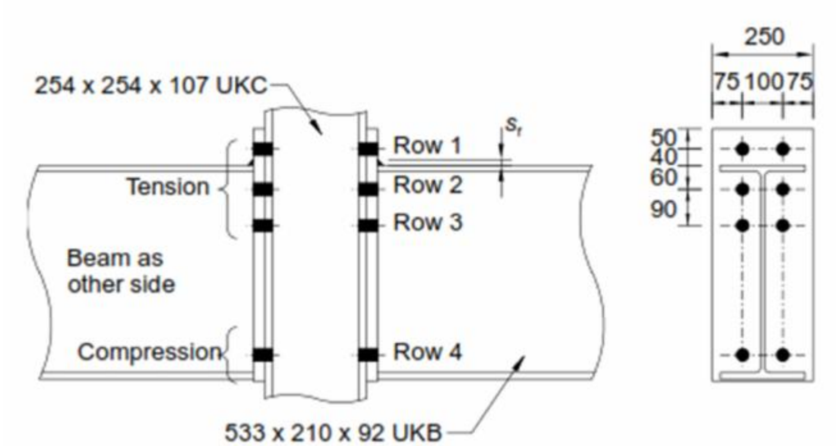

Figure 4.1: Geometrical configuration of extended end plate connection

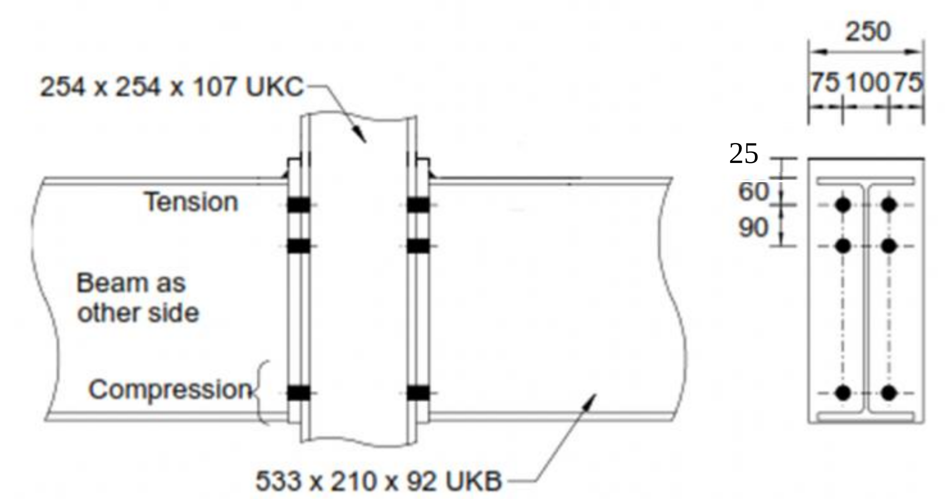

Figure 4.2: Geometrical configuration of flush end plate connection

4.3 Selection of variable parameters for parametric study

To conduct parametric study, the determination of the range of input variables is very important. Therefore, the geometric parameters are selected and varied within the practical ranges of extended end plate connections and flush end plate connections.

Table 4.1 parameters selected for numerical analysis

Parameter	Range of selected parameter
End plate thickness	12mm,15mm,20mm,25mm
Bolt diameter	16mm,20mm,24mm
Bolt gauge length	100mm,140mm
Bolt grade	4.6/8.8

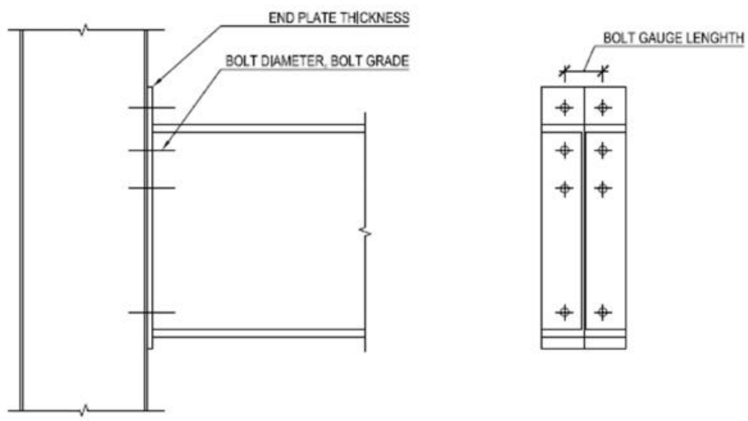

Figure 4.3 variable parameters of extended end plate connection

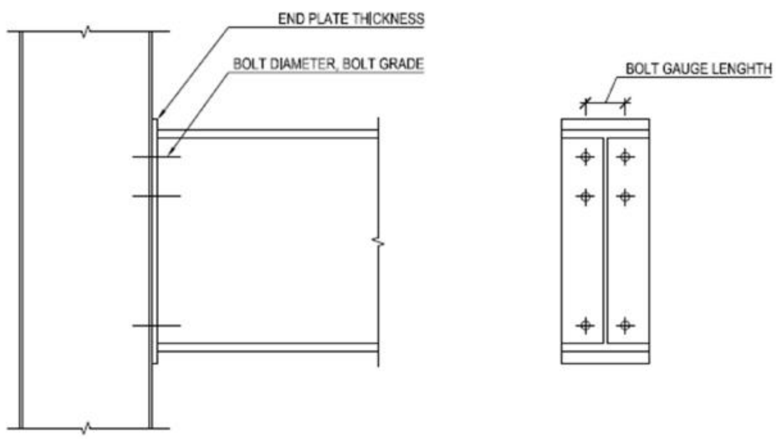

Figure 4.4 variable parameters of flush end plate connection

Table 4.1 shows the selected parameters for numerical analysis. Figure 4.3 and Figure 4.4 show the variable parameters of the connection. Only one parameter is changed at a time so as to clarify its effect. They are considered to be the most influential factors for the considered connections. Several configurations are analyzed to study the behavior of the connection.

Table 4.2 Geometrical configuration of the extended end plate connection (EEP)

specimen	No. of bolt row in tension	Size of end-plate			Size of bolt (mm)
		Width (mm)	Depth (mm)	End plate thickness(mm)	
EEP 1a	3	250	670	12	M16
EEP 1b				15	
EEP 1c				20	
EEP 1d				25	
EEP 2a	3	250	670	12	M20
EEP 2b				15	
EEP 2c				20	
EEP 2d				25	
EEP 3a	3	250	670	12	M24
EEP 3b				15	
EEP 3c				20	
EEP 3d				25	
Note : Column section S275 HB 254 x 254 x107, flange thickness 20.5mm, web 12.8mm Beam section S275 HB 533 x 210 x 92 flange thickness 15.6mm, web 10.1mm					

Table 4.3 Geometrical configuration of the flush end plate connection (FEP)

specimen	No. of bolt row in tension	Size of end-plate			Size of bolt (mm)
		Width (mm)	Depth (mm)	End plate thickness(mm)	
FEP 1a	2	250	605	12	M16
FEP 1b				15	
FEP 1c				20	
FEP 1d				25	
FEP 2a	2	250	605	12	M20
FEP 2b				15	
FEP 2c				20	
FEP 2d				25	
FEP 3a	2	250	605	12	M24
FEP 3b				15	
FEP 3c				20	
FEP 3d				25	
Note : Column section S275 HB 254 x 254 x107, flange thickness 20.5mm, web 12.8mm Beam section S275 HB 533 x 210 x 92 flange thickness 15.6mm, web 10.1mm					

4.4 Numerical Results

The Ultimate moment of resistance (M_R) and initial stiffness (Sj,ini) of connection based on Eurocode 3 are calculated for each bolted end plate connection configuration. These results are graphically presented and compared with each other to find out the variations due to bolt diameter, end plate thickness, bolt grade and bolt gauge length.

Here, weld is expected not to fail. So, welding failure calculations are not included.

Table 4.4 Summary of EEP Connection (Grade 8.8, Gauge Length = 100)

Design moment resistance of beam M_{beam} = 569.8kNm E = 210 x 10^3 N/mm^2 I_b = 55227 x 10^4 mm^4

Specimen	Bolt Dia. (mm)	Plate thickness (mm)	$M_{connect}$ (kNm)	$\frac{M_{con}}{M_{beam}}$	Classification by strength	$S_{j,ini}$ x 10^7 kNmm/rad	$S_{j,ini}/EI_b/L_b$			Classification by stiffness
							4m span	6m span	10m span	
EEP 1a	16	12	200	0.35	Partial strength	7.67	2.64	3.97	6.6	Semi rigid
EEP 1b		15	227	0.4	Partial strength	7.99	2.75	4.13	6.89	Semi rigid
EEP 1c		20	242	0.42	Partial strength	8.22	2.83	4.25	7.09	Semi rigid
EEP 1d		25	242	0.42	Partial strength	8.33	2.87	4.31	7.19	Semi rigid
EEP 2a	20	12	279	0.49	Partial strength	7.95	2.74	4.11	6.85	Semi rigid
EEP 2b		15	312	0.55	Partial strength	8.29	2.86	4.29	7.15	Semi rigid
EEP 2c		20	352	0.62	Partial strength	8.54	2.95	4.42	7.36	Semi rigid
EEP 2d		25	360	0.63	Partial strength	8.66	2.99	4.48	7.47	Semi rigid
EEP 3a	24	12	328	0.58	Partial strength	8.12	2.8	4.2	7	Semi rigid
EEP 3b		15	374	0.66	Partial strength	8.48	2.92	4.39	7.31	Semi rigid
EEP 3c		20	399	0.7	Partial strength	8.73	3.01	4.52	7.53	Semi rigid
EEP 3d		25	417	0.73	Partial strength	8.86	3.05	4.58	7.64	Semi rigid

Table 4.5 Summary of EEP Connection (Grade 8.8, Gauge length = 140)

Design moment resistance of beam M_{beam} = 569.8kNm   E = 210 x 10^3 N/mm^2 I_b = 55227 x 10^4 mm^4

Specimen	Bolt Dia. (mm)	Plate thickness (mm)	$M_{connect}$ (kNm)	$\frac{M_{con}}{M_{beam}}$	Classification by strength	$S_{j,ini}$ x 10^7 kNmm/rad	$S_{j,ini}$ / EI_b/L_b			Classification by stiffness
							4m span	6m span	10m span	
EEP 1a	16	12	184	0.32	Partial strength	7.4	2.55	3.83	6.38	Semi rigid
EEP 1b		15	211	0.37	Partial strength	7.74	2.67	4	6.69	Semi rigid
EEP 1c		20	224	0.39	Partial strength	7.95	2.74	4.11	6.86	Semi rigid
EEP 1d		25	224	0.39	Partial strength	8.05	2.78	4.16	6.94	Semi rigid
EEP 2a	20	12	255	0.45	Partial strength	7.72	2.66	3.99	6.66	Semi rigid
EEP 2b		15	287	0.5	Partial strength	8.07	2.78	4.18	6.96	Semi rigid
EEP 2c		20	310	0.54	Partial strength	8.28	2.86	4.29	7.14	Semi rigid
EEP 2d		25	304	0.53	Partial strength	8.38	2.89	4.33	7.22	Semi rigid
EEP 3a	24	12	296	0.52	Partial strength	7.92	2.73	4.1	6.83	Semi rigid
EEP 3b		15	355	0.62	Partial strength	8.28	2.85	4.28	7.14	Semi rigid
EEP 3c		20	375	0.66	Partial strength	8.49	2.93	4.39	7.32	Semi rigid
EEP 3d		25	389	0.68	Partial strength	8.57	2.96	4.44	7.39	Semi rigid

Table 4.6 Summary of EEP Connection (Grade 4.6, Gauge Length = 100)

Design moment resistance of beam M_{beam} = 569.8kNm E = 210 x 10^3 N/mm^2 I_b = 55227 x 10^4 mm^4

Specimen	Bolt Dia. (mm)	Plate thickness (mm)	$M_{connect}$ (kNm)	$\frac{M_{con}}{M_{beam}}$	Classification by strength	$S_{j,ini}$ x 10^7 kNmm/rad	$S_{j,ini}$ / EI_b/L_b			Classification by stiffness
							4m span	6m span	10m span	
EEP 1a	16	12	121	0.21	Nominally pinned	7.67	2.64	3.97	6.6	Semi rigid
EEP 1b		15	121	0.21	Nominally pinned	7.99	2.75	4.13	6.89	Semi rigid
EEP 1c		20	121	0.21	Nominally pinned	8.22	2.83	4.25	7.09	Semi rigid
EEP 1d		25	121	0.21	Nominally pinned	8.33	2.87	4.31	7.19	Semi rigid
EEP 2a	20	12	166	0.29	Partial strength	7.95	2.74	4.11	6.85	Semi rigid
EEP 2b		15	188	0.33	Partial strength	8.29	2.86	4.29	7.15	Semi rigid
EEP 2c		20	188	0.33	Partial strength	8.54	2.95	4.42	7.36	Semi rigid
EEP 2d		25	188	0.33	Partial strength	8.66	2.99	4.48	7.47	Semi rigid
EEP 3a	24	12	218	0.38	Partial strength	8.12	2.8	4.2	7	Semi rigid
EEP 3b		15	245	0.43	Partial strength	8.48	2.92	4.39	7.31	Semi rigid
EEP 3c		20	272	0.48	Partial strength	8.73	3.01	4.52	7.53	Semi rigid
EEP 3d		25	272	0.48	Partial strength	8.86	3.05	4.58	7.64	Semi rigid

Table 4.7 Summary of EEP Connection (Grade 4.6, Gauge Length = 140)

Design moment resistance of beam M_{beam} = 569.8kNm   E = 210 x 10^3 N/mm^2 I_b = 55227 x 10^4 mm^4

Specimen	Bolt Dia. (mm)	Plate thickness (mm)	$M_{connect}$ (kNm)	$\frac{M_{con}}{M_{beam}}$	Classification by strength	$S_{j,ini}$ x 10^7 kNmm/rad	$S_{j,ini}/EI_b/L_b$			Classification by stiffness
							4m span	6m span	10m span	
EEP 1a	16	12	113	0.2	Nominally pinned	7.4	2.55	3.83	6.38	Semi rigid
EEP 1b	16	15	114	0.2	Nominally pinned	7.74	2.67	4	6.69	Semi rigid
EEP 1c	16	20	114	0.2	Nominally pinned	7.95	2.74	4.11	6.86	Semi rigid
EEP 1d	16	25	114	0.2	Nominally pinned	8.05	2.78	4.16	6.94	Semi rigid
EEP 2a	20	12	154	0.27	Partial strength	7.72	2.66	3.99	6.66	Semi rigid
EEP 2b	20	15	177	0.31	Partial strength	8.07	2.78	4.18	6.96	Semi rigid
EEP 2c	20	20	178	0.31	Partial strength	8.28	2.86	4.29	7.14	Semi rigid
EEP 2d	20	25	178	0.31	Partial strength	8.38	2.89	4.33	7.22	Semi rigid
EEP 3a	24	12	200	0.35	Partial strength	7.92	2.73	4.1	6.83	Semi rigid
EEP 3b	24	15	228	0.4	Partial strength	8.28	2.85	4.28	7.14	Semi rigid
EEP 3c	24	20	242	0.42	Partial strength	8.49	2.93	4.39	7.32	Semi rigid
EEP 3d	24	25	242	0.42	Partial strength	8.57	2.96	4.44	7.39	Semi rigid

Table 4.8 Summary of FEP Connection (Grade 8.8, Gauge Length = 100)

Design moment resistance of beam $M_{beam} = 569.8kNm$ $E = 210 \times 10^3 N/mm^2$ $I_b = 55227 \times 10^4 mm^4$

Specimen	Bolt Dia. (mm)	Plate thickness (mm)	$M_{connect}$ (kNm)	$\frac{M_{con}}{M_{beam}}$	Classification by strength	$S_{j,ini} \times 10^7$ kNmm/rad	$S_{j,ini}/ EI_b/L_b$			Classification by stiffness
							4m span	6m span	10m span	
FEP 1a	16	12	129	0.23	Nominally pinned	5.59	1.93	2.89	4.82	Semi rigid
FEP 1b		15	145	0.26	Partial strength	5.98	2.06	3.09	5.15	Semi rigid
FEP 1c		20	152	0.27	Partial strength	6.26	2.16	3.24	5.4	Semi rigid
FEP 1d		25	152	0.27	Partial strength	6.39	2.21	3.31	5.51	Semi rigid
FEP 2a	20	12	179	0.31	Partial strength	5.86	2.02	3.03	5.05	Semi rigid
FEP 2b		15	202	0.35	Partial strength	6.28	2.17	3.25	5.42	Semi rigid
FEP 2c		20	235	0.41	Partial strength	6.59	2.27	3.41	5.68	Semi rigid
FEP 2d		25	237	0.42	Partial strength	6.73	2.32	3.48	5.81	Semi rigid
FEP 3a	24	12	218	0.38	Partial strength	6.02	2.08	3.11	5.19	Semi rigid
FEP 3b		15	263	0.46	Partial strength	6.46	2.23	3.34	5.57	Semi rigid
FEP 3c		20	301	0.53	Partial strength	6.79	2.34	3.51	5.86	Semi rigid
FEP 3d		25	304	0.53	Partial strength	6.94	2.39	3.59	5.99	Semi rigid

Table 4.9 Summary of FEP Connection (Grade 8.8, Gauge Length = 140)

Design moment resistance of beam M_{beam} = 569.8kNm E = 210 x 10^3 N/mm^2 I_b = 55227 x 10^4 mm^4

Specimen	Bolt Dia. (mm)	Plate thickness (mm)	$M_{connect}$ (kNm)	$\frac{M_{con}}{M_{beam}}$	Classification by strength	$S_{j,ini}$ x 10^7 kNmm/rad	$S_{j,ini}/EI_b/L_b$			Classification by stiffness
							4m span	6m span	10m span	
FEP 1a	16	12	120	0.21	Nominally pinned	4.45	1.54	2.3	3.84	Semi rigid
FEP 1b		15	136	0.24	Nominally pinned	5.16	1.78	2.67	4.45	Semi rigid
FEP 1c		20	152	0.27	Partial strength	5.73	1.98	2.96	4.94	Semi rigid
FEP 1d		25	152	0.27	Partial strength	5.98	2.06	3.09	5.15	Semi rigid
FEP 2a	20	12	165	0.29	Partial strength	4.62	1.59	2.39	3.98	Semi rigid
FEP 2b		15	187	0.33	Partial strength	5.39	1.86	2.79	4.65	Semi rigid
FEP 2c		20	213	0.37	Partial strength	6	2.07	3.11	5.18	Semi rigid
FEP 2d		25	213	0.37	Partial strength	6.28	2.17	3.25	5.42	Semi rigid
FEP 3a	24	12	180	0.32	Partial strength	4.72	1.63	2.44	4.07	Semi rigid
FEP 3b		15	243	0.43	Partial strength	5.52	1.9	2.86	4.76	Semi rigid
FEP 3c		20	281	0.49	Partial strength	6.17	2.13	3.19	5.32	Semi rigid
FEP 3d		25	281	0.49	Partial strength	6.46	2.23	3.34	5.57	Semi rigid

Table 4.10 Summary of FEP Connection (Grade 4.6, Gauge Length = 100)

Design moment resistance of beam M_{beam} = 569.8kNm E = 210 x 10^3 N/mm^2 I_b = 55227 x 10^4 mm^4

Specimen	Bolt Dia. (mm)	Plate thickness (mm)	$M_{connect}$ (kNm)	$\frac{M_{con}}{M_{beam}}$	Classification by strength	$S_{j,ini}$ x 10^7 kNmm/rad	$S_{j,ini}/EI_b/L_b$			Classification by stiffness
							4m span	6m span	10m span	
FEP 1a	16	12	76	0.13	Nominally pinned	5.59	1.93	2.89	4.82	Semi rigid
FEP 1b		15	76	0.13	Nominally pinned	5.98	2.06	3.09	5.15	Semi rigid
FEP 1c		20	76	0.13	Nominally pinned	6.26	2.16	3.24	5.4	Semi rigid
FEP 1d		25	76	0.13	Nominally pinned	6.39	2.21	3.31	5.51	Semi rigid
FEP 2a	20	12	106	0.19	Nominally pinned	5.86	2.02	3.03	5.05	Semi rigid
FEP 2b		15	119	0.21	Nominally pinned	6.28	2.17	3.25	5.42	Semi rigid
FEP 2c		20	119	0.21	Nominally pinned	6.59	2.27	3.41	5.68	Semi rigid
FEP 2d		25	119	0.21	Nominally pinned	6.73	2.32	3.48	5.81	Semi rigid
FEP 3a	24	12	140	0.25	Partial strength	6.02	2.08	3.11	5.19	Semi rigid
FEP 3b		15	157	0.28	Partial strength	6.46	2.23	3.34	5.57	Semi rigid
FEP 3c		20	171	0.3	Partial strength	6.79	2.34	3.51	5.86	Semi rigid
FEP 3d		25	171	0.3	Partial strength	6.94	2.39	3.59	5.99	Semi rigid

Table 4.11 Summary of FEP Connection (Grade 4.6, Gauge Length = 140)

Design moment resistance of beam M_{beam} = 569.8kNm E = 210 x 10^3 N/mm^2 I_b = 55227 x 10^4 mm^4

Specimen	Bolt Dia. (mm)	Plate thickness (mm)	$M_{connect}$ (kNm)	$\frac{M_{con}}{M_{beam}}$	Classification by strength	$S_{j,ini}$ x 10^7 kNmm/rad	$S_{j,ini}/EI_b/L_b$			Classification by stiffness
							4m span	6m span	10m span	
FEP 1a	16	12	75	0.13	Nominally pinned	4.45	1.54	2.3	3.84	Semi rigid
FEP 1b		15	76	0.13	Nominally pinned	5.16	1.78	2.67	4.45	Semi rigid
FEP 1c		20	76	0.13	Nominally pinned	5.73	1.98	2.96	4.94	Semi rigid
FEP 1d		25	76	0.13	Nominally pinned	5.98	2.06	3.09	5.15	Semi rigid
FEP 2a	20	12	100	0.17	Nominally pinned	4.62	1.59	2.39	3.98	Semi rigid
FEP 2b		15	118	0.21	Nominally pinned	5.39	1.86	2.79	4.65	Semi rigid
FEP 2c		20	119	0.21	Nominally pinned	6	2.07	3.11	5.18	Semi rigid
FEP 2d		25	119	0.21	Nominally pinned	6.28	2.17	3.25	5.42	Semi rigid
FEP 3a	24	12	130	0.23	Nominally pinned	4.72	1.63	2.44	4.07	Semi rigid
FEP 3b		15	147	0.26	Partial strength	5.52	1.9	2.86	4.76	Semi rigid
FEP 3c		20	171	0.3	Partial strength	6.17	2.13	3.19	5.32	Semi rigid
FEP 3d		25	171	0.3	Partial strength	6.46	2.23	3.34	5.57	Semi rigid

4.4.1 Extended end plate connection

Parametric study is carried out to investigate the effect of bolt diameter, end plate thickness, bolt grade and bolt gauge length on moment resistance and initial stiffness of extended end plate connection as shown in Table 4.4, Table 4.5, Table 4.6 and Table 4.7 respectively. The results are summarised and presented graphically below.

1. **Effect of bolt diameter**

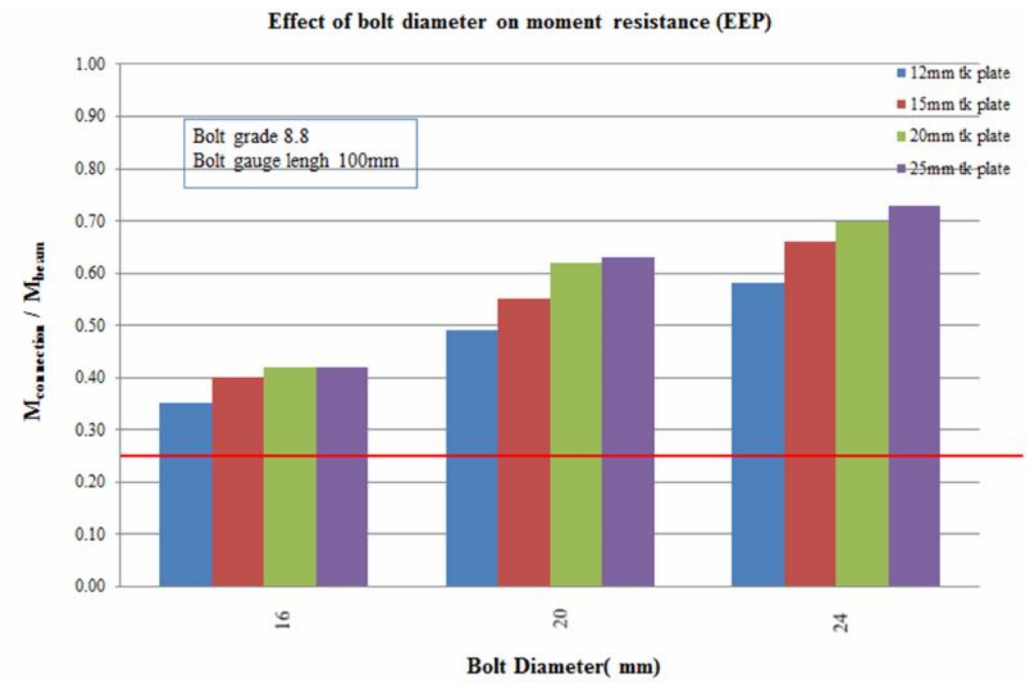

Figure 4.5 Effect of bolt diameter on moment resistance

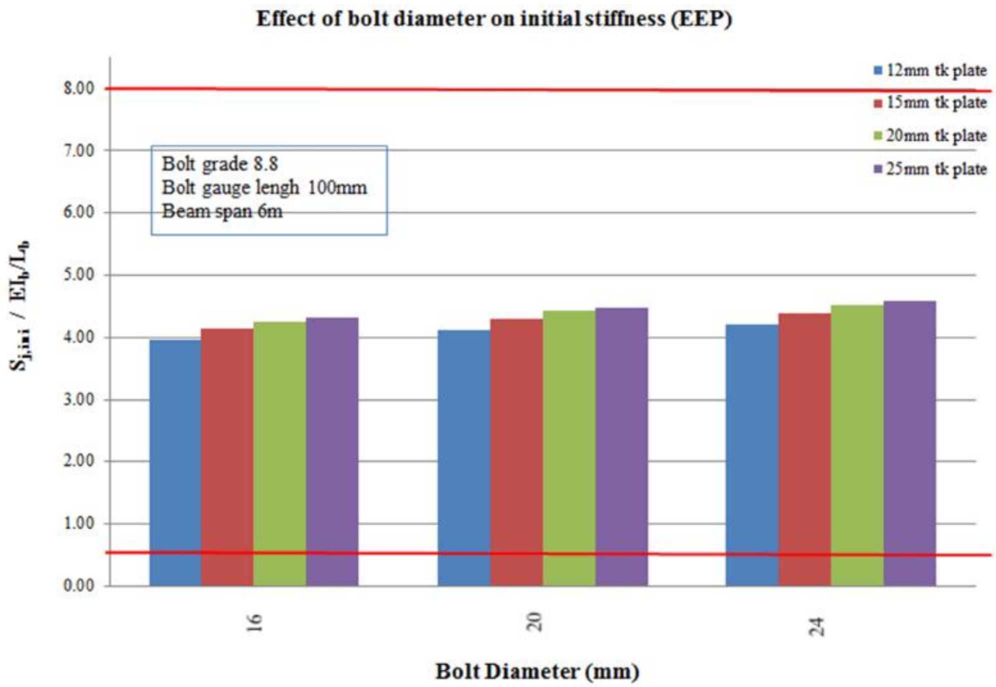

Figure 4.6 Effect of bolt diameter on initial stiffness

Figure 4.5 and Figure 4.6 show the effect of different bolt diameters on the initial stiffness and moment of resistance of extended end plate connections with end plate thickness 12, 15, 20 and 25mm. It can be observed that increasing of both bolt diameter and thickness of end plate increases the initial stiffness only marginally and ultimate moment of connections.

The lowest initial stiffness and ultimate moment are found in the case when bolt diameter of 16mm is used. Increasing the bolt diameter from 16 to 20mm increases the initial stiffness and ultimate moment by about 4% and 43% respectively. But increasing the bolt from 20 to 24mm increases the initial stiffness and ultimate moment by about 2% and 17% respectively.

2. Effect of bolt gauge length

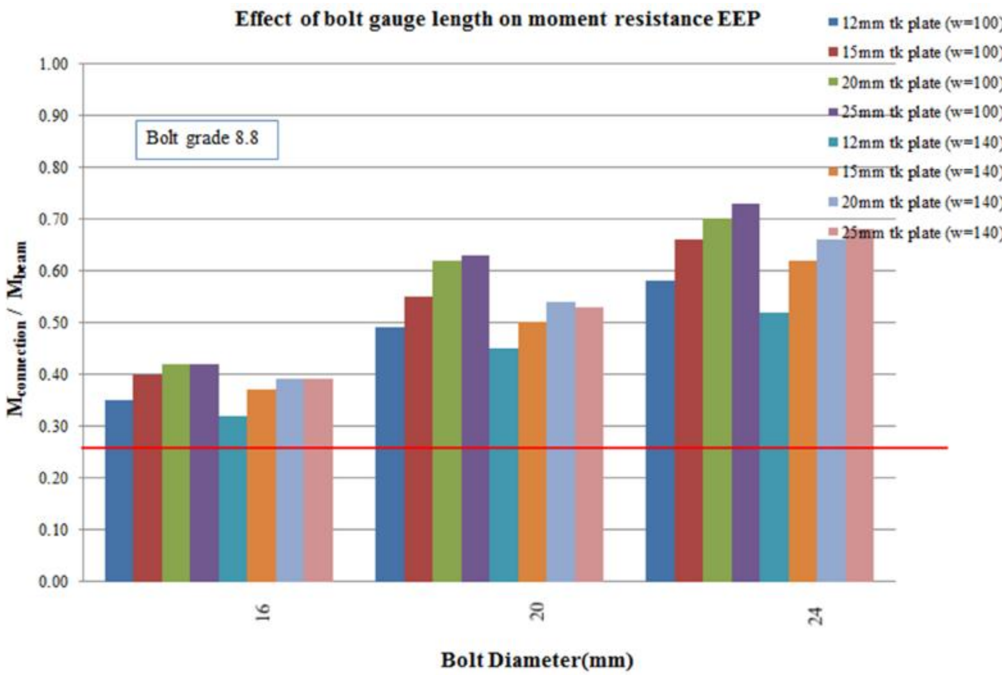

Figure 4.7 Effect of bolt gauge length increasing 100 to 140mm on moment resistance

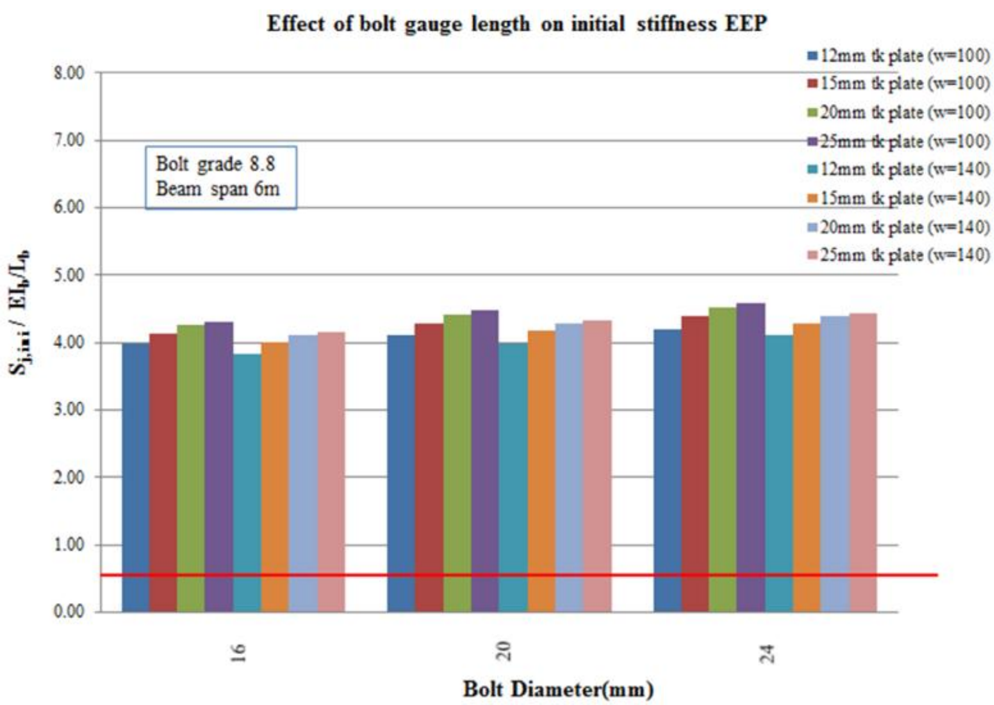

Figure 4.8 Effect of bolt gauge length increasing 100 to 140mm on initial stiffness

Figure 4.7 and Figure 4.8 show the effect increase of bolt gauge length from 100 to 140mm on the initial stiffness and moment of resistance of extended end plate connections with end plate thickness 12, 15, 20 and 25mm. The results showed that increasing the bolt gauge length decreases the initial stiffness and ultimate moment of connections by 3% and 9% respectively.

3. Effect of bolt grade

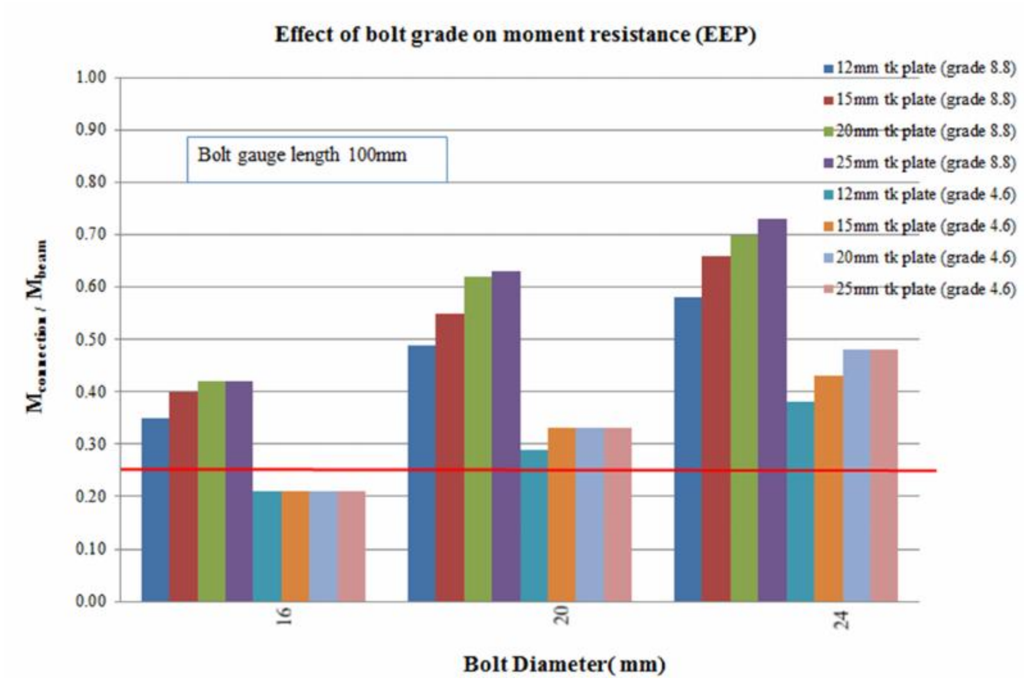

Figure 4.9 Effect of bolt grade on moment resistance

Figure 4.9 showed that the effect of increasing bolt grade on moment of resistance of extended end plate connections with end plate thickness 12, 15, 20 and 25mm. It can be observed that increasing the bolt grade 4.6 to 8.8 increases the ultimate moment of connections by 88% in 16mm bolt, 78% in 20mm bolt and 51% in 24mm bolt and no effect on the initial stiffness as it is no impact from calculations.

4. Effect of end plate thickness

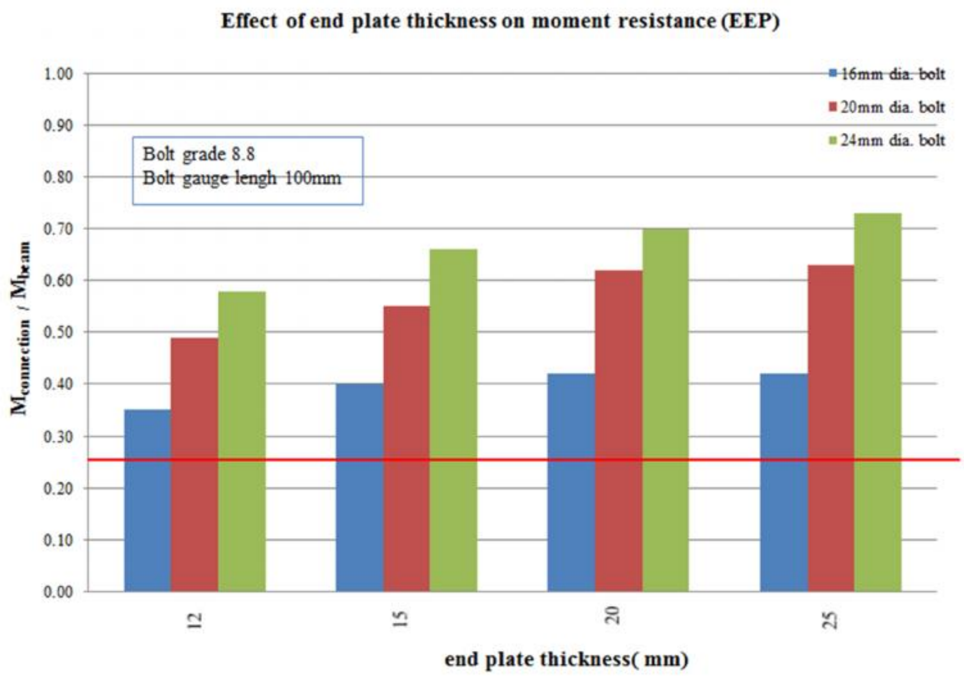

Figure 4.10 Effect of end plate thickness on moment resistance

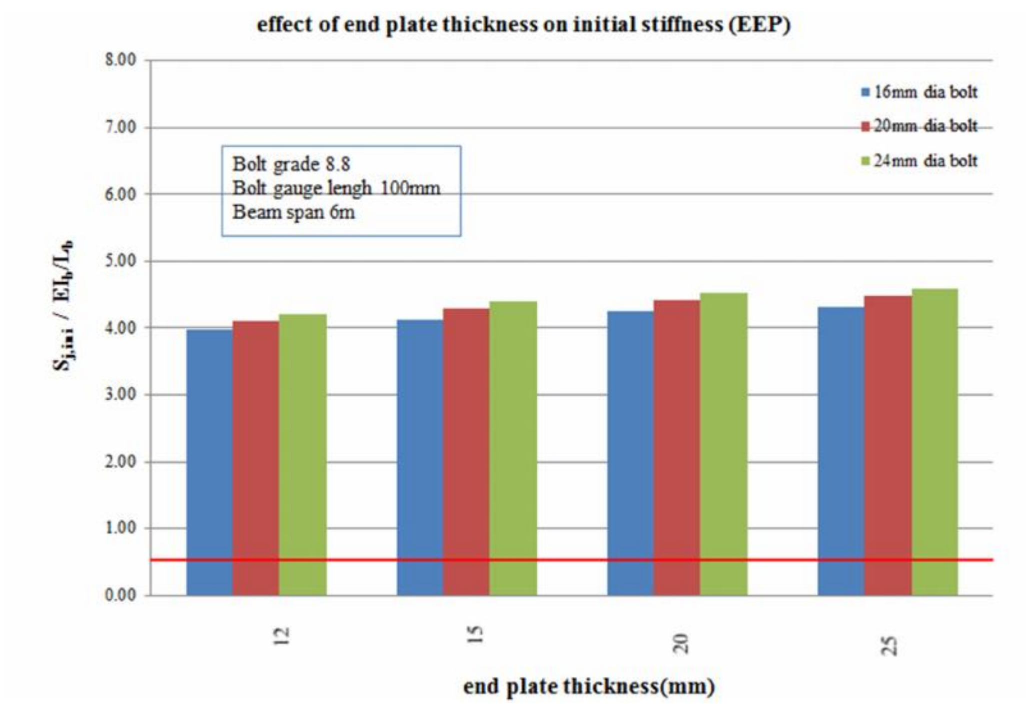

Figure 4.11 Effect of end plate thickness on initial stiffness

Figure 4.10 and Figure 4.11 show the effect of different end plate thickness on the initial stiffness and ultimate moment. End plate thicknesses are 12, 15, 20 and 25mm. Generally it can be observed that the initial stiffness increases with the increase of end plate thickness 12 to 15 mm by 4%, 15mm to 20mm by 3% and 20mm to 25mm by 1%, while moment resistance increases with the increase of end plate thickness 12 to 15mm by 13%, 15 to 20mm by 9% and 20 to 25mm by 2%. In addition, it can be observed that the bolt diameter and end plate thickness have significant influence on the ultimate moment capacity.

4.4.2 Flush end plate connection

As shown in Table 4.8, Table 4.9, Table 4.10 and Table 4.11, flush end plate connection results are summarised and presented graphically below to express the effect of bolt diameter, end plate thickness, bolt grade and bolt gauge length on moment resistance and initial stiffness

1. **Effect of bolt diameter**

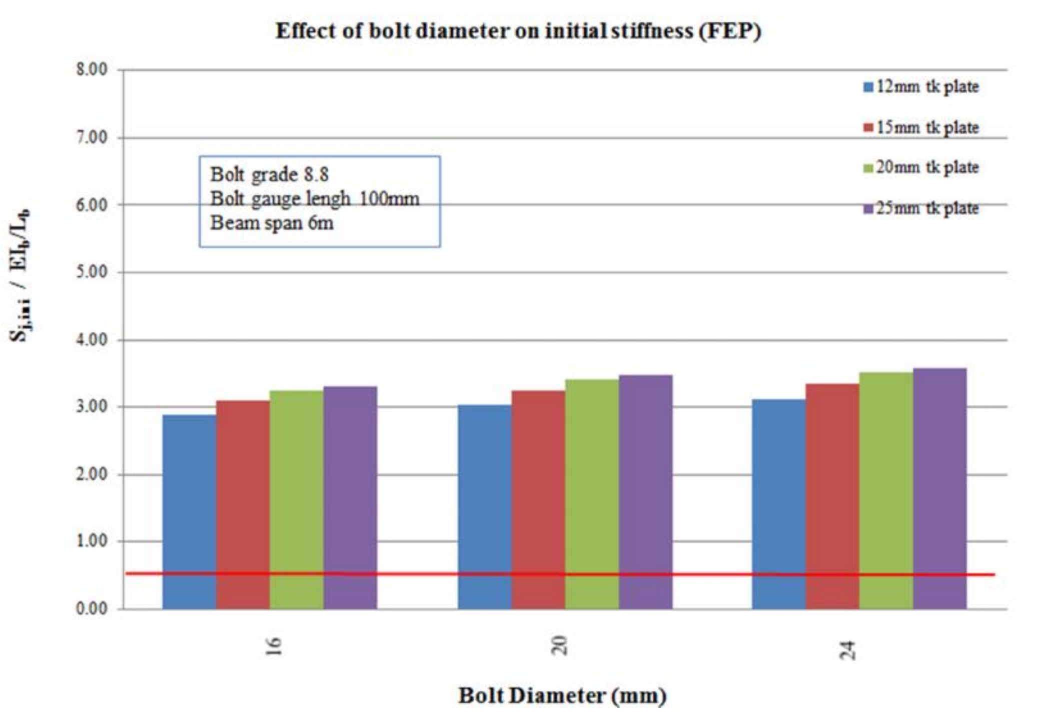

Figure 4.12 Effect of bolt diameter on initial stiffness

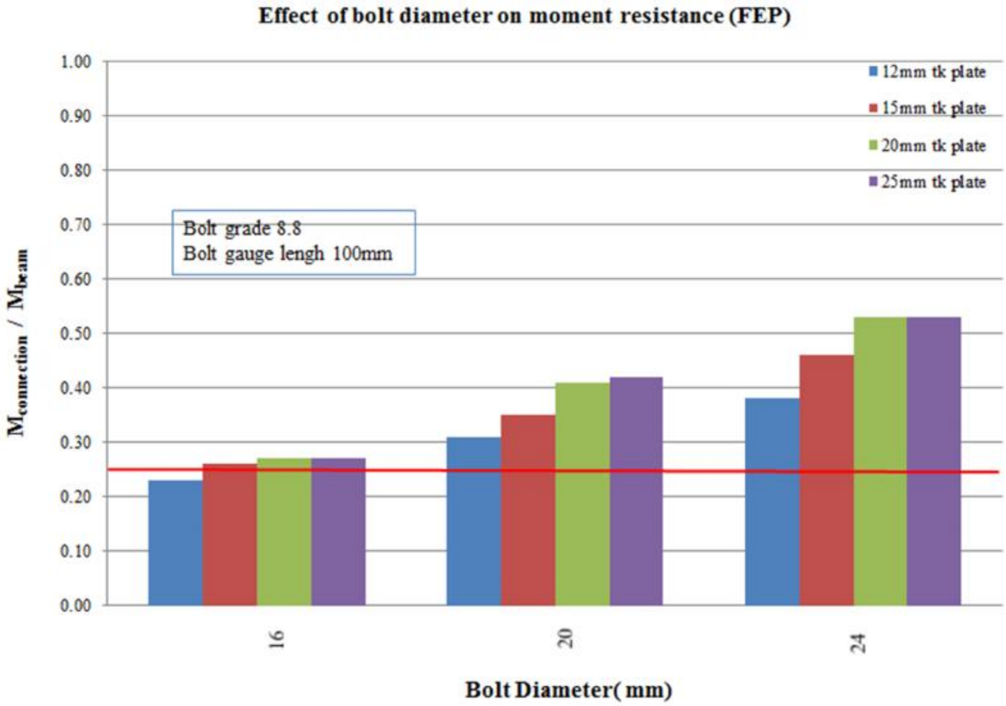

Figure 4.13 Effect of bolt diameter on moment resistance

Figure 4.12 and Figure 4.13 show the effect of different bolt diameters on the initial stiffness and moment of resistance of flush end plate connections with end plate thickness 12, 15, 20 and 25mm. It can be observed that increasing of both bolt diameter and thickness of end plate increases the initial stiffness and ultimate moment of connections.

Increasing the bolt diameter from 16 to 20mm increases the initial stiffness and ultimate moment by about 5% and 47% respectively. But increasing the bolt from 20 to 24mm increases the initial stiffness and ultimate moment by about 3% and 27% respectively.

2. Effect of end plate thickness

Figure 4.14 Effect of end plate thickness on moment resistance

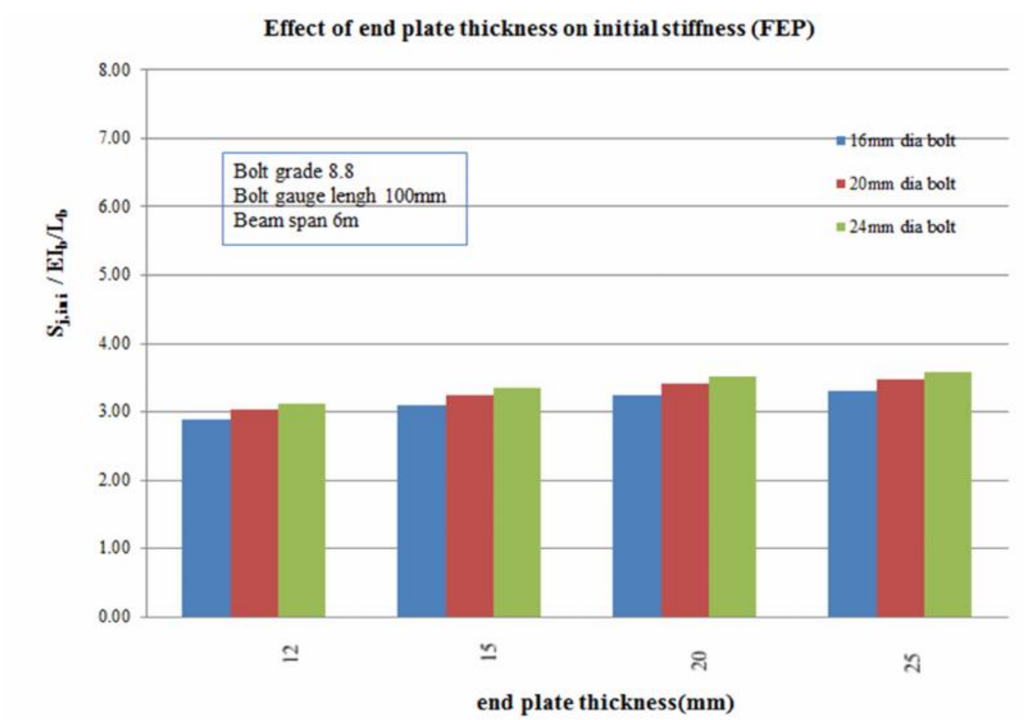

Figure 4.15 Effect of end plate thickness on initial stiffness

Figure 4.14 and Figure 4.15 show the effect of different end plate thickness on the initial stiffness and ultimate moment. It can be observed that the initial stiffness increases with the increase of end plate thickness 12 to 15 mm by 7%, 15mm to 20mm by 5% and 20mm to 25mm by 2%, while moment resistance increases with the increase of end plate thickness 12 to 15mm by 15%, 15 to 20mm by 12% and 20 to 25mm 1%. In addition, the results that the bolt diameter and end plate thickness have significant influence on the ultimate moment capacity.

3. Effect of bolt grade

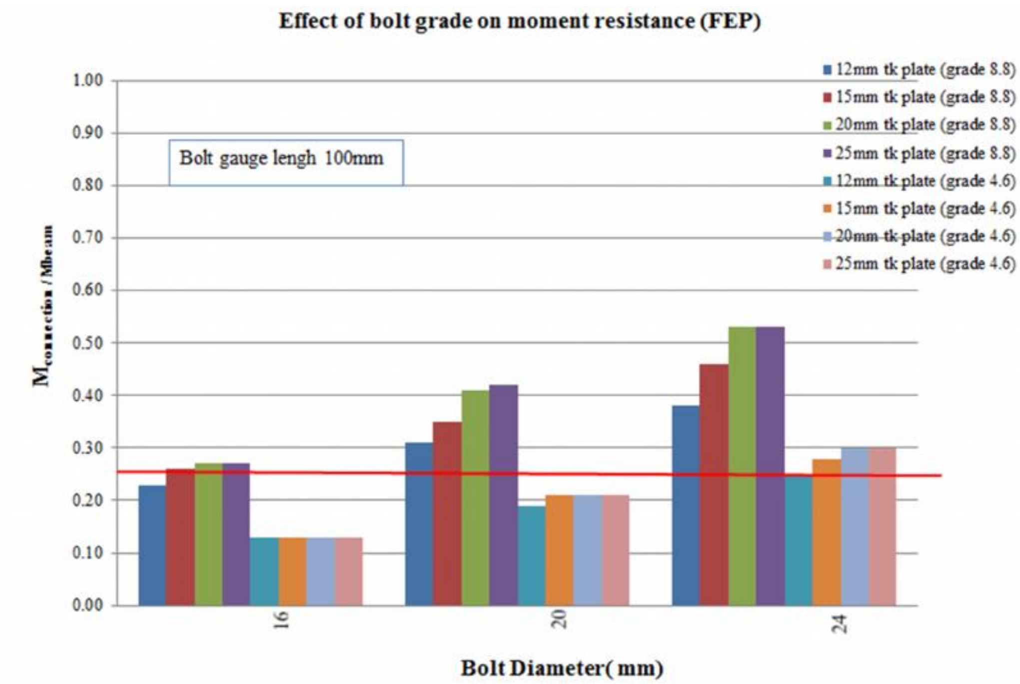

Figure 4.16 Effect of bolt grade on moment resistance

Figure 4.16 shows the effect of increasing bolt grade on moment of resistance of flush end plate connections with end plate thickness 12, 15, 20 and 25mm. It can be observed that increasing the bolt grade 4.6 to 8.8 increases the ultimate moment of connections by 90% in 16mm bolt, 84% in 20mm bolt and 69% in 24mm bolt and no effect on the initial stiffness as there is no impact from calculations.

4. Effect of bolt gauge length

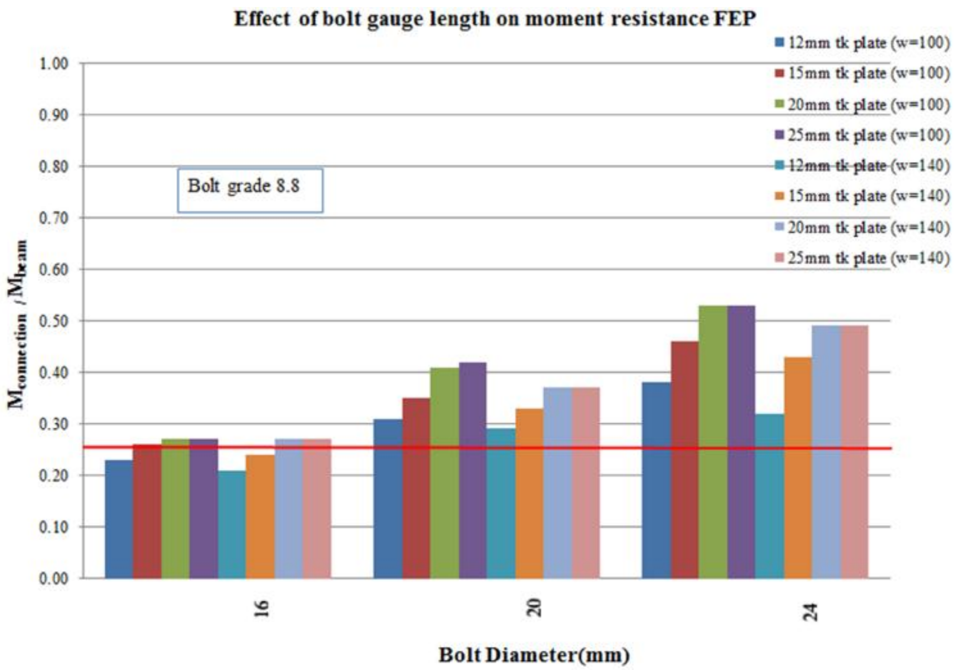

Figure 4.17 Effect of bolt gauge length on moment resistance

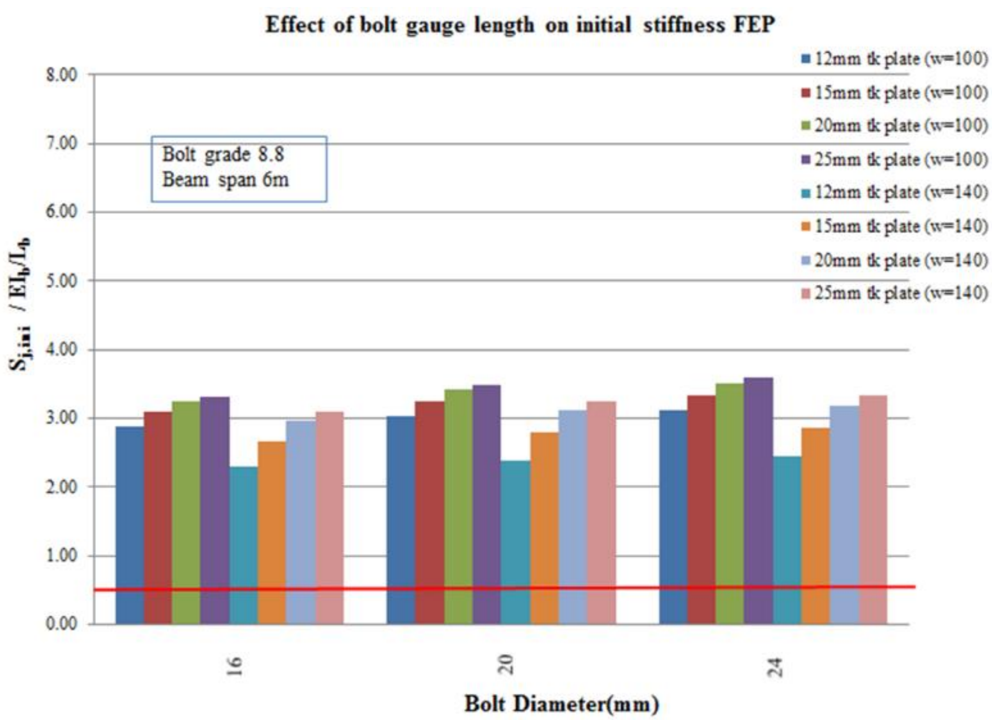

Figure 4.18 Effect of bolt gauge length on initial stiffness

Figure 4.17 and Figure 4.18 show the effect of bolt gauge length increasing 100 to 140mm on the initial stiffness and moment of resistance of flush end plate connections with end plate thickness 12, 15, 20 and 25mm. It can be observed that increasing the bolt gauge length decreases the initial stiffness and ultimate moment of connections by 13% and 8% respectively.

4.5 Comparison between the extended end plate connection and flush end plate connection

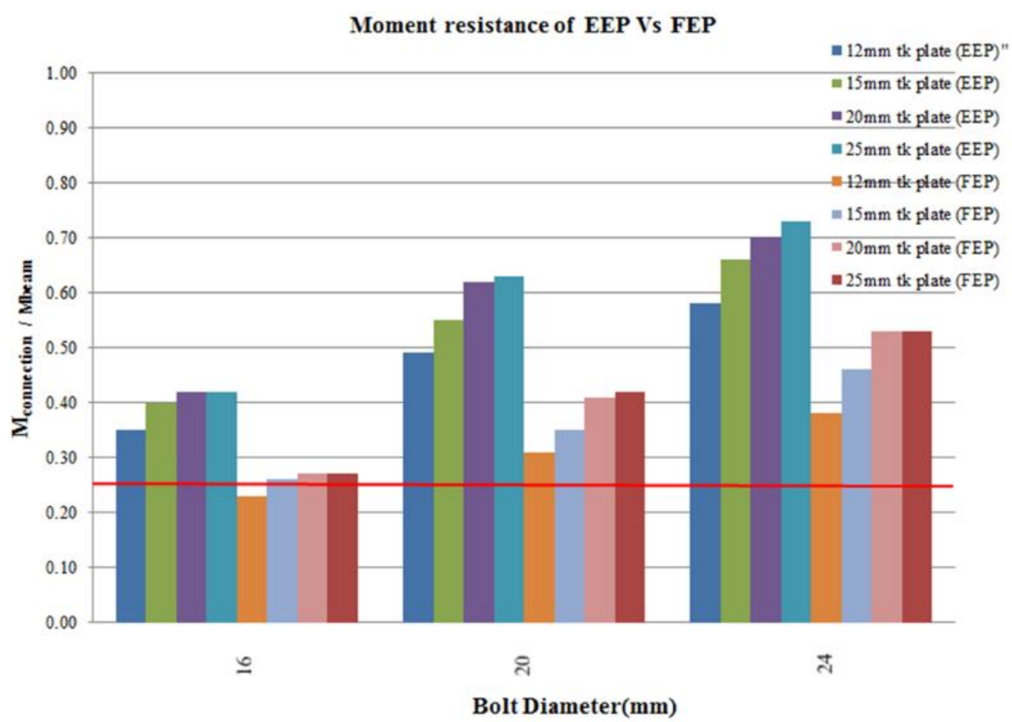

Figure 4.19 Comparison between the extended end plate connection and Flush end plate connection on moment of resistance (bolt grade 8.8, W= 100mm)

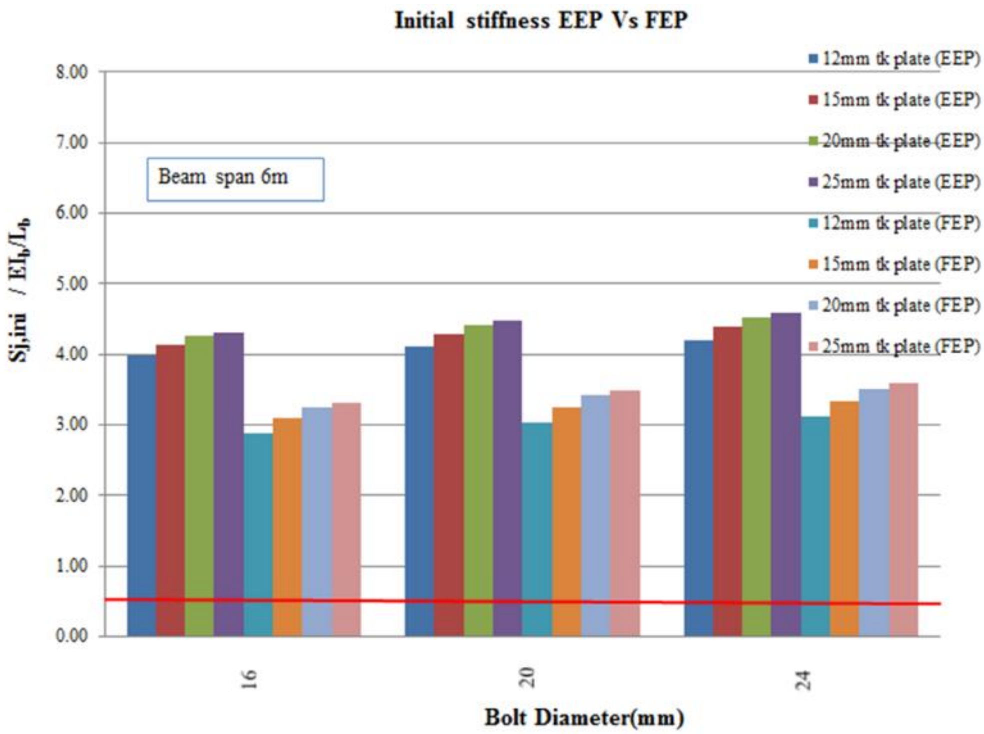

Figure 4.20 Comparison between the extended end plate connection and flush end plate connection on initial stiffness (bolt grade 8.8, W= 100mm)

Figure 4.19 and Figure 4.20 show the comparison between extended end plate connection and flush end plate connection on initial stiffness and moment of resistance. The results showed that the moment resistance of extended end plate connection increased by 58% in 16mm bolt, 53% in 20mm bolt and 41% in 25mm bolt than flush end plate connection. Initial stiffness of extended end plate connection increased by 33% in 16mm bolt, 31% in 20mm bolt and 31% in 25mm bolt than flush end plate connection.

4.6 Classification of joints based on EC3

As defined in EC3, connection is classified by their strength and stiffness.

EC3 classification categorised connections based on their moment resistance, where maximum moment resistance of the connection is more than 25% of the moment resistance of the connected beam but less than 100%, the connection is classified as partial strength connection. The connection that possess moment resistance smaller than 25%, are considered as nominally pinned. As shown in Table 4.4, Table 4.5, Table 4.6 and Table 4.7, parametric study of extended end plate connections revealed that the behaviour of the connections has satisfied the requirement for partial strength connections by their strength except 16mm 4.6 bolt.

Extended end plate connection has satisfied the requirement for semi rigid connections by their stiffness.

16mm grade 4.6 bolt with 12mm, 15mm, 20mm and 25mm thick plate, EEP connection falls to nominally pinned based on strength.

EEP connections are classified as semi rigid (beam span 4m to 10m) based on stiffness. When beam span is 12m, connection moves to rigid region.

As shown in Table 4.8, Table 4.9, Table 4.10 and Table 4.11, 16mm bolt with 12mm, 15mm, 20mm and 25mm thick plate, Flush end plate connection falls to nominally pinned based on strength. FEP connection with bolt grade 4.6 is also classified as nominally pinned. Rest of the connections are partial strength.

All FEP connections are classified as semi rigid (beam span 4m to 12m) based on stiffness.

SCI guideline limits the range of bolt grades and sizes in moment connections as shown in Table 4.12. Fully threaded M24 8.8 bolts should be the first choice for beam sections 400mm deep or greater and M20 8.8 bolts for shallower beams. End plates 250 x 25mm for M24 bolts and 200 x 20mm for M20 bolts are recommended. For the majority of sections, cross centres of 100mm with M24 bolts and 90mm with M20 bolts are recommended.

SCI guideline recommend M20 8.8 bolts , end plate S275 150 x 8mm with bolt gauge 90mm for beam sections up to 457mm deep and end plate 200 x 10mm with bolt gauge 140mm for beam sections 533mm deep or above in simple connections as shown in Table 4.12.

Table 4.12 Recommended connection details based on SCI guidelines

Connection	**Bolt (Grade 8.8)**	**Plate size (mm)**		**Bolt gauge length (mm)**	**Beam depth (mm)**
		Thickness	**Width**		
Moment Connection	M 20	20	200	90	<400
	M 24	25	250	100	≥400
Simple Connection	M 20	8	150	90	≤457
		10	200	140	≥533

From parametric study, M20 8.8 bolts and M24 8.8 bolts with 12mm, 15mm, 20mm and 25mm thick plates for beam section 533mm deep with bolt gauge 100mm and 140mm, both EEP and FEP connections are classified as semi rigid connections as shown in Table 4.13 and Table 4.14.

Table 4.13 Extended end plate connection details from parametric study

Bolt (Grade 8.8)	**Plate size (mm)**		**Bolt gauge length (mm)**	**Beam depth (mm)**	**Connection (Output)**
	Thickness	**Width**			
M 20	12/15/20/25	250	100	533	Semi rigid connection
M 24			140		

Table 4.14 Flush end plate connection details from parametric study

Bolt (Grade 8.8)	**Plate size (mm)**		**Bolt gauge length (mm)**	**Beam depth (mm)**	**Connection (Output)**
	Thickness	**Width**			
M 20	12/15/20/25	250	100	533	Semi rigid connection
M 24			140		

4.7 Effect of beam span on initial stiffness (Sj,ini) of connection

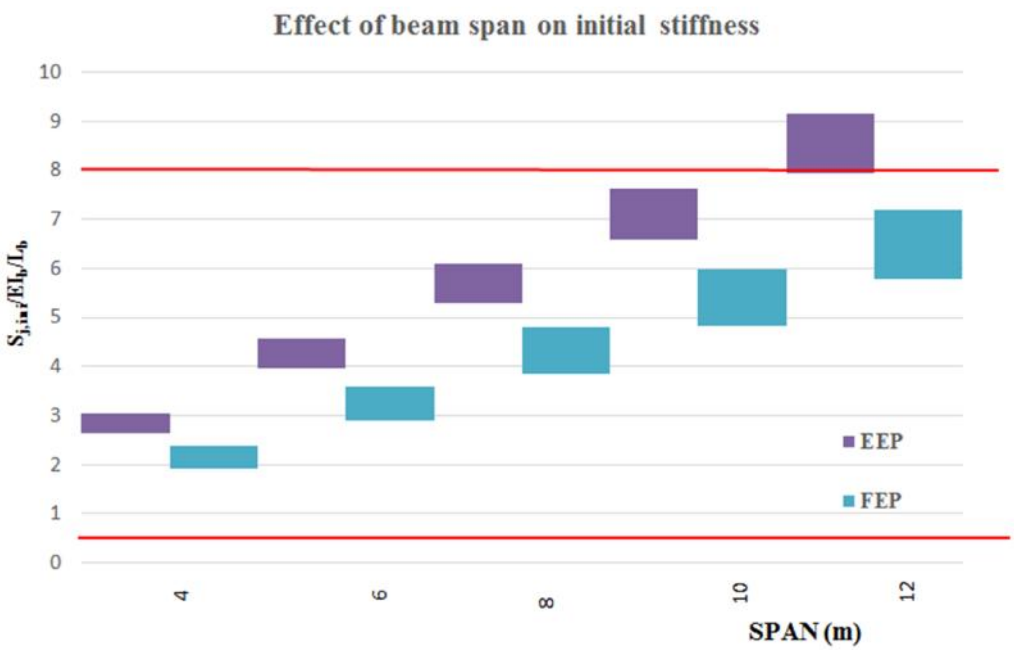

Figure 4.21 Effect of beam span on initial stiffness of EEP and FEP

Figure 4.21 show the effect of beam span on initial stiffness of extended end plate connection and flush end plate connection. Beams span are 4m, 6m, 8m, 10m and 12m.

It can be observed that the initial stiffness of EEP connection increases with the increase of beam span 4m to 6m by 50%, 6m to 8m by 33%, 8m to 10m by 25% and 10m to 12m by 20%. The initial stiffness of FEP connection increases with the increase of beam span is also same as EEP connection. The initial stiffness of EEP connections are greater than the initial stiffness of FEP connections by 32% for all beam span cases.

Graph clearly revealed that the behaviour of EEP connection has satisfied the requirement for semi rigid connection based on stiffness for beam span 4m to 10m. When beam span is 12m, EEP connection falls to rigid region.

In all cases, FEP connection is classified as semi rigid.

4.8 Classification based on moment resistance of EEP and FEP connections

As shown in Figure 4.22, it can be observed that the moment resistance of EEP and FEP connection is in the range 0.1 to 0.8. Most of the connections are classified as partial strength connection. Rest of the connections are nominally pinned.

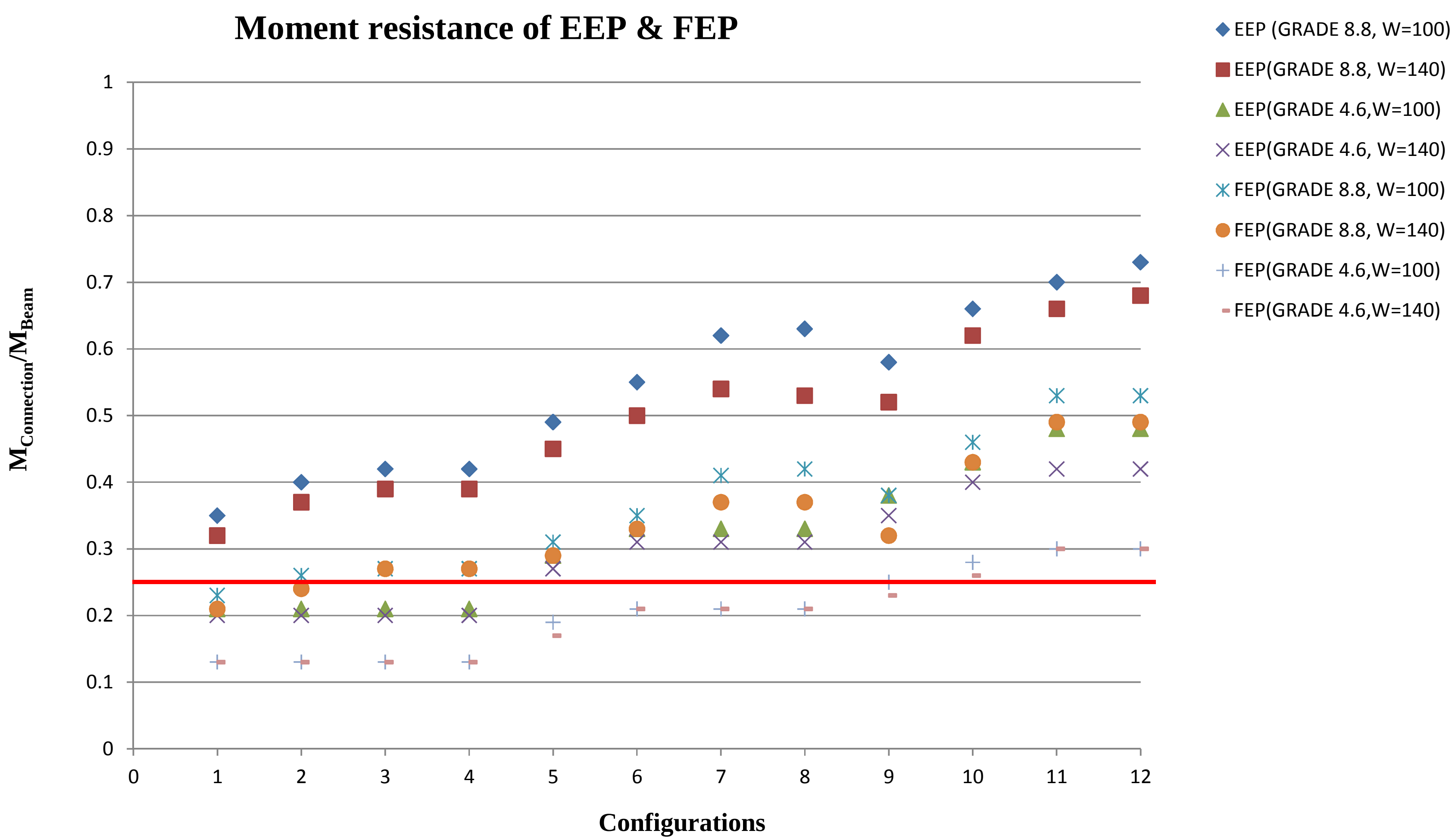

Figure 4.22 Moment resistance of EEP & FEP

5. CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

A parametric study of 48 extended end plate and 48 flush end plate steel beam-to-column connections (column S275 HB 254x254x107, Beam S275 HB 533x210x92) were discussed to find the behaviour with variations in bolt diameter (M16, M20 & M24) end plate thickness (12mm, 15mm, 20mm & 25mm), bolt grade (4.6/8.8) and bolt gauge length (100mm & 140mm). The initial elastic stiffness (Sj,ini) and ultimate moment capacity of connections (M_R) were determined.

Based on the parametric study, the following conclusions can be drawn.

- While the moment resistance of the extended end plate and flush end plate connections significantly increased as the size of bolt increased by 13-50%, the initial stiffness only increased by 2-4%. Effect on moment resistance is much greater than on initial stiffness.
- While the moment resistance of the extended end plate and flush end plate connections increased as the end plate thickness increased by 2-21%, the initial stiffness only increased by 1-7%.
- Bolt grade also increased the moment resistance by 47-100% but there was no effect on initial stiffness as there is no impact from calculations. The increase in the bolt gauge length decreased the moment resistance by 5-18% and initial stiffness of connection by 2-22%.
- The initial stiffness increased with the increase of end plate thickness, while it slightly increased with the increase of bolt diameter.
- Parametric study of extended end plate connections revealed that the behaviour of the connections has satisfied the requirement for partial strength connections based on strength except 16mm 4.6 bolts.
- EEP connections are classified as semi rigid (beam span 4m to 10m) based on stiffness. When beam span is 12m, connection falls to rigid region.
- 16mm bolt with 12mm, 15mm, 20mm and 25mm thick plate, Flush end plate connection falls to nominally pinned based on strength. FEP connection with

bolt grade 4.6 is also classified as nominally pinned. Rest of the connections are partial strength.

- All FEP connections are classified as semi rigid (beam span 4m to 12m) based on stiffness.

5.2 Recommendations

- Carryout more parametric study for different beam and column combinations.
- Analyze the connection Using finite element softwares and easy method to be developed.

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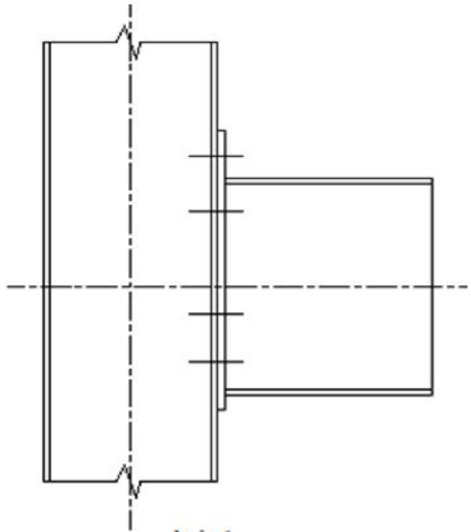
Appendix A:

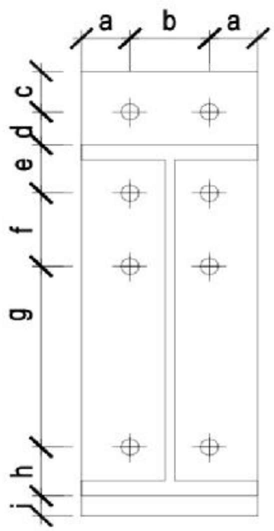
Design Calculations for Extended End Plate Connection- EEP 3d

Reference	Calculation	Output

Connection Detail - EEP 3d



Joint



Row 1	a	=	75	mm
	b	=	100	mm
Row 2	c	=	50	mm
	d	=	40	mm
	e	=	60	mm
Row 3	f	=	90	mm
	g	=	345	mm
	h	=	60	mm
	i	=	25	mm

Beam 533x210x92

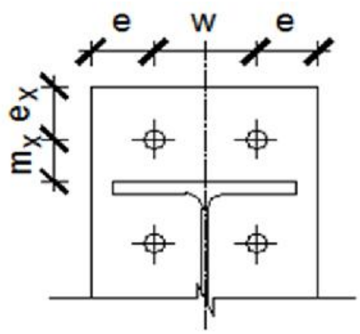
Beam height	=	533.1	mm
Flange width	=	209.3	mm
Mass per 1m length	=	92	kg/m
Flange thickness	=	15.6	mm
Web thickness	=	10.1	mm
Radius of gyration	=	12.7	mm

Column 254x254x107

column height	=	266.7	mm
Flange width	=	258.8	mm
Mass per 1m length	=	107	kg/m
Flange thickness	=	20.5	mm
Web thickness	=	12.8	mm
Radius of gyration	=	12.7	mm

Yield strength	$f_{y,p}$	=	265	N/mm^2
Ultimate tensile strength	$f_{u,p}$	=	410	N/mm^2

Reference	Calculation	Output
	End plate	
	End plate thickness t_p = 25 mm	
	End plate height h_p = 670 mm	
	End plate width w_p = 250 mm	
	Yield strength $f_{y,p}$ = 265 N/mm^2	
	Ultimate tensile strength $f_{u,p}$ = 410 N/mm^2	
	Bolts (class 8.8)	
	Bolt Diameter = 24 mm d_w = 39.6 mm	
	Tensile stress area = 353 mm^2	
	Total no of bolts = 8	
	no of bolts in tension = 6	
	no of bolts in shear = 8	
T 3.1	Yield strength f_{yb} = 640 N/mm^2	
	Ultimate tensile strength f_{ub} = 800 N/mm^2	
	fillet weld thickness	
	Beam flange to end plate weld thickness = 12 mm	
	Beam web to end plate weld thickness = 8 mm	

Reference	Calculation	Output
	Design Calculation according to EC3 for **EEP 3d**	
	Partial factors for Resistance	
	Structural Steel	
EN 1993-1-8	γ_{MO} = 1.0	
N.A.2.15		
T NA.1	γ_{M1} = 1.00 (Resistance of a member to buckling)	
T NA.1	γ_{M2} = 1.10 (plates in bearing in bolted connections)	
	For tring resistance verification $\gamma_{M,u}$ = 1.10	
T NA.1	**Bolts** γ_{M2} = 1.25	
T NA.1	**Welds** γ_{M2} = 1.25	
EN 1993-1-8 :2005	**1. Bolts Tension**	
Cl.3.6.1 (1) T 3.4	$F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}}$	
	For non countersunk Bolts , k_2 = 0.9	F_1 =
	$F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}} = \frac{0.9 \times 800 \times 353}{1.25}$ = 203.328 kN	203.33 kN
Cl.6.2.6.5	**2. End plate in bending**	
Cl.6.2.6.5 (1)	Bolt row 1 - Bolt row outside tension flange of beam	
T 6.6	Effective length for an end plate	
	for circular patterns, $\ell_{eff,cp}$ = Min ($2\pi m_x$, $\pi m_x + w$, $\pi m_x + 2e$)	
	for extended part of end plate	
	w = 100 mm	
	m_x = 30.40 mm	
	e = 75 mm	
	e_x = 50 mm	
	e_{min} = 75 mm	
	bp = 250 mm	
		
	$2\pi m_x$ = 190.912 mm	
	$\pi m_x + w$ = 195.456 mm	
	$\pi m_x + 2e$ = 245.456 mm	
	$\ell_{eff,cp}$ = 190.91 mm	
	for non circular patterns,	
	$\ell_{eff,nc}$ = Min ($4m_x + 1.25\,e_x$, $e + 2m_x + 0.625\,e_x$, 0.5bp , $0.5w + 2m_x + 0.625\,e_x$)	
	$4m_x + 1.25\,e_x$ = 184.1 mm	
	$e + 2m_x + 0.625\,e_x$ = 167.05 mm	
	0.5bp = 125.0 mm	
	$0.5w + 2m_x + 0.625\,e_x$ = 142.05 mm	
	$\ell_{eff,nc}$ = 125.00 mm	
	Mode 1 - Complete failure of the T-stub flange	
	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$	
	$\ell_{eff,1}$ = 125.00 mm	

Reference	Calculation	Output
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	e_w = 9.8875 mm	
	n = e_{min} but n ≤ 1.25m (38)	
	n = 38.00 mm	
	$M_{p,1,Rd} = \frac{0.25 \sum \ell_{eff} t_p^2 f_y}{\gamma_{MO}} = \frac{0.25 * 125.00 * 25^2 * 265}{1.0}$	
	$M_{p,1,Rd}$ = 5175.8 kNmm	
	$F_{T,1,Rd}$ = 900 kN	
	Mode 2 - Bolt failure with yielding of the T-stub flange	
T 6.4	$\ell_{eff,2}$ = $\ell_{eff,nc}$ = 125.00 mm	
T 6.2	$F_{T,2,Rd} = \frac{2 M_{p,2,Rd} + n\sum F_{t,Rd}}{m + n}$	
	$M_{p,1,Rd} = \frac{0.25 \sum \ell_{eff} t_p^2 f_y}{\gamma_{MO}} = \frac{0.25 * 125.00 * 25^2 * 265}{1.0}$	
	$M_{p\,2,Rd}$ = 5175.8 kNmm	
	n = e_{min} but n ≤ 1.25m (38)	
	n = 38.00 mm	
	$F_{t,Rd} = \frac{0.9 f_{ub} A_s}{\gamma_{M2}} = \frac{0.9 \times 800 \times 353}{1.25}$ = 203.328 kN	
	$F_{T,2,Rd}$ = 377.26 kN	
	Mode 3	
	$F_{T,3,Rd} = \sum F_{t,Rd}$ = 2 * 203.33 = 406.656 kN	
	Resistance only from Row 1 bolts = 377.26 kN	
Cl.6.2.6.5 (1)	Bolt row 2 - First Bolt row below tension flange of beam	
T 6.6	Effective length for an end plate,	
	for circular patterns, $\ell_{eff,cp} = 2\pi m$	
	m = 38.55 mm	
	$\ell_{eff,cp}$ = 242.09 mm	
	for non circular patterns, $\ell_{eff,nc} = \alpha m$	
	m_2 = 34.8 mm	
Figure 6.11	$\lambda_1 = \frac{m}{m + e}$ = 0.34 , $\lambda_2 = \frac{m_2}{m + e}$ = 0.31	
	α = 7.5	
	$\ell_{eff,nc}$ = 289.13 mm	
T 6.2	**Mode 1** - Complete failure of the T-stub flange	
	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$	
	$\ell_{eff,1}$ = 242.09 mm	
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	e_w = 9.8875 mm	
	n = e_{min} but n ≤ 1.25m (48.2)	
	n = 48.19 mm	

Reference	Calculation	Output
	$M_{p,1,Rd} = \frac{0.25 \sum \ell_{eff} t_p^2 f_y}{\gamma_{MO}} = \frac{0.25 * 242.09 * 25^2 * 265}{1.0}$	
	$M_{p,1,Rd} = 10024.2$ kNmm	
	$F_{T,1,Rd} = 1283$ kN	
	Mode 2 - Bolt failure with yielding of the T-stub flange	
T 6.4	$\ell_{eff,2} = \ell_{eff,nc} = 289.13$ mm	
T 6.2	$F_{T,2,Rd} = \frac{2 M_{p,2,Rd} + n\sum F_{t,Rd}}{m + n}$	
	$M_{p,2,Rd} = \frac{0.25 \sum \ell_{eff} t_p^2 f_y}{\gamma_{MO}} = \frac{0.25 * 289.13 * 25^2 * 265}{1.0}$	
	$M_{p\,2,Rd} = 11971.6$ kNmm	
	$n = e_{min}$ but $n \leq 1.25m$ (48.2)	
	$n = 48.1875$ mm	
	$F_{t,Rd} = 203.33$	
	$F_{T,2,Rd} = 501.96$ kN	
	Mode 3	
	$F_{T,3,Rd} = \sum F_{t,Rd} = 2 * 203.33 = 406.656$ kN	
	Resistance only from Row 2 bolts = 406.66 kN	
Cl.6.2.6.5 (1)	Bolt row 3 - Other end bolt row	
T 6.6	Effective length for an end plate,	
	for circular patterns, $\ell_{eff,cp} = 2\pi m$	
	$m = 38.55$ mm	
	$\ell_{eff,cp} = 242.09$ mm	
	for non circular patterns, $\ell_{eff,nc} = 4m + 1.25e$	
	$= 247.95$ mm	
	$\ell_{eff,nc} = 247.95$ mm	
T 6.2	**Mode 1** - Complete failure of the T-stub flange	
	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$	
	$\ell_{eff,1} = 242.09$ mm	
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	$e_w = 9.8875$ mm	
	$n = e_{min}$ but $n \leq 1.25m$ (48.2)	
	$n = 48.19$ mm	
	$M_{p,1,Rd} = \frac{0.25 \sum \ell_{eff} t_p^2 f_y}{\gamma_{MO}} = \frac{0.25 * 242.09 * 25^2 * 265}{1.0}$	
	$M_{p,1,Rd} = 10024.2$ kNmm	
	$F_{T,1,Rd} = 1282.91$ kN	
	Mode 2 - Bolt failure with yielding of the T-stub flange	
T 6.4	$\ell_{eff,2} = \ell_{eff,nc} = 247.95$ mm	
T 6.2	$F_{T,2,Rd} = \frac{2 M_{p,2,Rd} + n\sum F_{t,Rd}}{m + n}$	

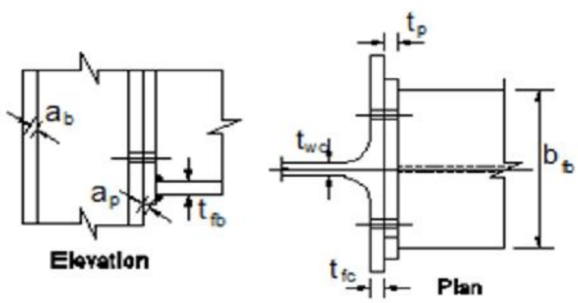
Reference	Calculation	Output
	$M_{p,2,Rd} = \frac{0.25 \sum \ell_{eff} t_p^2 f_y}{\gamma_{MO}} = \frac{0.25 * 247.95 * 25^2 * 265}{1.0}$	
	$M_{p2,Rd}$ = 10266.7 kNmm	
	n = e_{min} but n ≤ 1.25m (48.2)	
	n = 48.1875 mm	
	$F_{t,Rd}$ = 203.33	
	$F_{T,2,Rd}$ = 462.65 kN	
	Mode 3	
	$F_{T,3,Rd} = \sum F_{t,Rd}$ = 2 * 203.33 = 406.656 kN	
	Resistance only from Row 3 bolts = 406.66 kN	
Cl.6.2.6.5 (1)	Bolt row 2 & 3 - combined	
	resistance of row 3 may be limited by the resistance of rows 2 & 3 as a group	
	row 2 first bolt row below the tension flange of the beam	
	row 3 other end bolt row	
T 6.6	Effective length for an end plate,	
	row 2	
	for circular patterns, $\ell_{eff,cp} = \pi m + p$	
	m = 38.55 mm p = 90 mm	
	$\ell_{eff,cp}$ = 211.05 mm	
	for non circular patterns, $\ell_{eff,nc}$ = 0.5p + αm-(2m + .625e)	
Figure 6.11	$\lambda_1 = \frac{m}{m+e}$ = 0.34 , $\lambda_2 = \frac{m_2}{m+e}$ = 0.31	
	α = 7.5	
	$\ell_{eff,nc}$ = 210.15 mm	
	row 3	
	for circular patterns, $\ell_{eff,cp} = \pi m + p$	
	m = 38.55 mm p = 90 mm	
	$\ell_{eff,cp}$ = 211.05 mm	
	for non circular patterns, $\ell_{eff,nc}$ = 2m + .625e+0.5*p	
	= 169 mm	
	total effective length for this group of rows	
	$\Sigma \ell eff,cp$ = 211 + 211 = 422 mm	
	$\Sigma \ell eff,nc$ = 210 + 169 = 379 mm	
T 6.2	**Mode 1** - Complete failure of the T-stub flange	
	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$	
	$\Sigma \ell_{eff,1}$ = 379.13 mm	
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	e_w = 9.8875 mm	
	n = e_{min} but n ≤ 1.25m (48.2)	
	n = 48.19 mm	

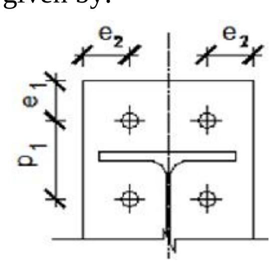
Reference	Calculation	Output
	$M_{p,1,Rd} = \dfrac{0.25 \sum \ell_{eff} t_p^2 f_y}{\gamma_{MO}} = \dfrac{0.25 * 379.13 * 25^2 * 265}{1.0}$	
	$M_{p,1,Rd}$ = 15698.1 kNmm	
	$F_{T,1,Rd}$ = 2009.07 kN	
	Mode 2 - Bolt failure with yielding of the T-stub flange	
T 6.4	$\ell_{eff,2} = \ell_{eff,nc}$ = 379.13 mm	
T 6.2	$F_{T,2,Rd} = \dfrac{2 M_{p,2,Rd} + n \sum F_{t,Rd}}{m + n}$	
	$M_{p,2,Rd} = \dfrac{0.25 \sum \ell_{eff} t_p^2 f_y}{\gamma_{MO}} = \dfrac{0.25 * 379.13 * 25^2 * 265}{1.0}$	
	$M_{p\ 2,Rd}$ = 15698.1 kNmm	
	$n = e_{min}$ but $n \le 1.25m$ (48.2)	
	n = 48.1875 mm	
	$F_{t,Rd}$ = 203.33	
	$F_{T,2,Rd}$ = 813.81 kN	
	Mode 3	
	$F_{T,3,Rd} = \sum F_{t,Rd}$ = 4 * 203.33 = 813.312 kN	
	Resistance only from 2 & 3 bolts = 813.31 kN	
		F_2 =
	Resistance for End plate in bending = 377.26 kN	377.26 kN
Cl.6.2.6.4	**3.Column flange in transverse bending**	
Cl.6.2.6.4 (1)	- each individual bolt-row required to resist tension	
Cl.6.2.6.4 (1)	Bolt row 1 - end bolt row	
T 6.4	Effective length of an unstiffened column flange	
	for circular patterns, $\ell_{eff,cp}$ = smaller of $2\pi m$ and $\pi m + 2 e_1$	
	for welded end plate narrower than column flange	
	r_c = 12.7 mm	
	m = 33.44 mm	
	e = 79.4 mm	
	e_1 is large so it will not be critical	
	e_{min} = 75 mm	
	$2\pi m$ = 210.003 mm	
	$\ell_{eff,cp}$ = 210.00 mm	
	for non circular patterns,	
	$\ell_{eff,nc}$ = smaller of $4m + 1.25e$ and $2m + 0.625e + e_1$	
	$4m + 1.25e$ = 233.01 mm	
	$\ell_{eff,nc}$ = 233.01 mm	
	Mode 1	
	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \le \ell_{eff,cp}$	
	$\ell_{eff,1}$ = 210.00 mm	
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	e_w = 9.8875 mm	
	$n = e_{min}$ but $n \le 1.25m$ (41.8)	
	n = 41.80 mm	
	$M_{p,1,Rd} = \dfrac{0.25 \sum \ell_{eff} t_f^2 f_y}{\gamma_{MO}} = \dfrac{0.25 * 210.00 * 20.5^2 * 265}{1.0}$	

Reference	Calculation	Output
	$M_{p,1,Rd}$ = 5846.8 kNmm	
	$F_{T,1,Rd}$ = 896.62 kN	
	Mode 2	
T 6.4	$\ell_{eff,2} = \ell_{eff,nc}$ = 233.01 mm	
T 6.2	$F_{T,2,Rd} = \dfrac{2M_{p,2,Rd} + n\Sigma F_{t,Rd}}{m+n}$	
	$M_{p,2,Rd} = \dfrac{0.25\,\Sigma\ell_{eff}\,t_f^2\,f_y}{\gamma_{MO}} = \dfrac{0.25 * 233.01 * 20.5^2 * 265}{1.0}$	
	$M_{p\,2,Rd}$ = 6487.4 kNmm	
	$n = e_{min}$ but $n \leq 1.25m$ (41.8)	
	n = 41.8 mm	
	$F_{t,Rd}$ = 203.33	
	$F_{T,2,Rd}$ = 398.36 kN	
	Mode 3	
	$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 2 * 203.33 = 406.656 kN	
	Resistance only from Row 1 bolts = 398.36 kN	
Cl.6.2.6.4 (1)	Bolt row 1 and 2 combined	
	Bolt row 1 - end bolt row	
	Bolt row 2 - end bolt row	
T 6.4	Effective length of an unstiffened column flange	
	for circular patterns, $\ell_{eff,cp} = (\pi m + p)$	
	for welded end plate narrower than column flange	
	r_c = 12.7 mm	
	m = 33.44 mm	
	e = 79.4 mm	
	p = 100 mm	
	$2(\pi m + p)$ = 410.003 mm	
	$\Sigma\ell_{eff,cp}$ = 410.00 mm	
	for non circular patterns,	
	$\ell_{eff,nc} = 2*(2m + 0.625e + 0.5p)$	
	$2(2m + 0.625e + 0.5p)$ = 333.01 mm	
	$\Sigma\ell_{eff,nc}$ = 333.01 mm	
	Mode 1	
	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$	
	$\Sigma\ell_{eff,1}$ = 333.01 mm	
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	e_w = 9.8875 mm	
	$M_{p,1,Rd} = \dfrac{0.25\,\Sigma\ell_{eff}\,t_f^2\,f_y}{\gamma_{MO}} = \dfrac{0.25 * 333.01 * 20.5^2 * 265}{1.0}$	
	$M_{p,1,Rd}$ = 9271.52 kNmm	
	$F_{T,1,Rd}$ = 1421.81 kN	

Reference	Calculation	Output
	Mode 2	
T 6.4	$\ell_{eff,2} = \ell_{eff,nc} = 333.01$ mm	
T 6.2	$F_{T,2,Rd} = \frac{2M_{p,2,Rd} + n\Sigma F_{t,Rd}}{m+n}$	
	$M_{p,2,Rd} = \frac{0.25\Sigma\ell_{eff} t_f^2 f_y}{\gamma_{MO}} = \frac{0.25 * 333.01 * 20.5^2 * 265}{1.0}$	
	$M_{p2,Rd} = 9271.52$ kNmm	
	$n = e_{min}$ but $n \leq 1.25m$	
	$n = 41.8$ mm	
	$F_{t,Rd} = 203.33$	
	$F_{T,2,Rd} = 698.29$ kN	
	Mode 3	
	$F_{T,3,Rd} = \Sigma F_{t,Rd} = 4 * 203.33 = 813.312$ kN	
	Resistance from Bolt Row 1 & Row 2 combination = 698.29 kN	
Cl.6.2.6.4 (1)	Bolt row 1,2 and 3 combined	
	Bolt row 1 ,3 - end bolt row	
	Bolt row 2 - inner bolt row	
T 6.4	Effective length of an unstiffened column flange	
	for circular patterns, $\Sigma\ell_{eff,cp} = 2p + (\pi m + p1)+(\pi m + p2)$	
	for welded end plate narrower than column flange	
	$r_c = 12.7$ mm	
	$m = 33.44$ mm	
	$e = 79.4$ mm	
	$p1 = 100$ mm $p = 95$ mm	
	$p2 = 90$ mm	
	$\Sigma\ell_{eff,cp} = 2p + (\pi m + p1)+(\pi m + p2)$	
	$= 590.003$ mm	
	for non circular patterns,	
	$\Sigma\ell_{eff,nc} = p + (2m + 0.625e + 0.5p1) + (2m + 0.625e + 0.5p2)$	
	$\Sigma\ell_{eff,nc} = 423.01$ mm	
	Mode 1	
	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$	
	$\Sigma\ell_{eff,1} = 423.01$ mm	
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	$e_w = 9.8875$ mm	
	$M_{p,1,Rd} = \frac{0.25\Sigma\ell_{eff} t_f^2 f_y}{\gamma_{MO}} = \frac{0.25 * 423.01 * 20.5^2 * 265}{1.0}$	
	$M_{p,1,Rd} = 11777.3$ kNmm	
	$F_{T,1,Rd} = 1806.07$ kN	
	Mode 2	
T 6.4	$\ell_{eff,2} = \ell_{eff,nc} = 423.01$ mm	
T 6.2	$F_{T,2,Rd} = \frac{2M_{p,2,Rd} + n\Sigma F_{t,Rd}}{m+n}$	

Reference	Calculation	Output
	$M_{p,2,Rd} = \frac{0.25 \Sigma\ell_{eff} t_f^2 f_y}{\gamma_{MO}} = \frac{0.25* \; 423.01 \; * \; 20.5^2 * \; 265}{1.0}$	
	$M_{p\,2,Rd}$ = 11777.3 kNmm	
	n = e_{min} but n ≤ 1.25m	
	n = 41.8 mm	
	$F_{t,Rd}$ = 203.33	
	$F_{T,2,Rd}$ = 990.82 kN	
	Mode 3	
	$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 6 * 203.33 = 1219.97 kN	
	Resistance from Bolt Row 1,2 & 3 combination = 990.82 kN	
Cl.6.2.6.4 (1)	Bolt row 2 and 3 combined	
	Bolt row 2,3 - end bolt row	
T 6.4	Effective length of an unstiffened column flange	
	for circular patterns, $\Sigma\ell_{eff,cp}$ = 2*(πm + p)	
	for welded end plate narrower than column flange	
	r_c = 12.8 mm	
	m = 33.44 mm	
	e = 79.4 mm	
	p = 90 mm	
	$\Sigma\ell_{eff,cp}$ = 390.003 mm	
	for non circular patterns,	
	$\ell_{eff,nc}$ = 2*(2m + 0.625e + 0.5p)	
	$\Sigma\ell_{eff,nc}$ = 323.01 mm	
	Mode 1	
	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$	
	$\Sigma\ell_{eff,1}$ = 323.01 mm	
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	e_w = 9.8875 mm	
	$M_{p,1,Rd} = \frac{0.25 \Sigma\ell_{eff} t_f^2 f_y}{\gamma_{MO}} = \frac{0.25* \; 323.01 \; * \; 20.5^2 * \; 265}{1.0}$	
	$M_{p,1,Rd}$ = 8993.10 kNmm	
	$F_{T,1,Rd}$ = 1379.11 kN	
	Mode 2	
T 6.4	$\ell_{eff,2} = \ell_{eff,nc}$ = 323.01 mm	
T 6.2	$F_{T,2,Rd} = \frac{2 M_{p,2,Rd} + n\Sigma F_{t,Rd}}{m + n}$	
	$M_{p,2,Rd} = \frac{0.25 \Sigma\ell_{eff} t_f^2 f_y}{\gamma_{MO}} = \frac{0.25* \; 323.01 \; * \; 20.5^2 * \; 265}{1.0}$	
	$M_{p\,2,Rd}$ = 8993.10 kNmm	
	n = e_{min} but n ≤ 1.25m	
	n = 41.8 mm	
	$F_{t,Rd}$ = 203.33	
	$F_{T,2,Rd}$ = 690.89 kN	

Reference	Calculation	Output
	Mode 3 $F_{T,3,Rd} = \Sigma F_{t,Rd} = 4 * 203.33 = 813.312$ kN	
	Resistance from Bolt Row 2 & 3 combination = 690.89 kN	
	Resistance for Column flange in transverse bending = 398.36 kN	F_3 = 398.36 kN
Cl.6.2.6.8 (1) Eq" (6.22)	**4. Beam web in tension** $F_{t,wb,Rd} = \frac{b_{eff,t,wb}\, t_{wb}\, f_{y,wb}}{\gamma_{Mo}}$ $b_{eff,t,wb}$ = 242 mm t_{wb} = 10.1 mm $F_{t,wb,Rd}$ = 672.42 kN	F_4 = 672.42 kN
EN 1993-1-8 :2005 Cl.6.2.6.3 (1) Cl.6.2.6.3 (3)	**5. Column web in tension** $F_{t,wc,Rd} = \frac{\omega\, b_{eff,t,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{Mo}}$ For Bolted connection $b_{eff,t,wc}$ = 233.00 mm t_{wc} = 12.8 mm ω = 1.00 $F_{t,wc,Rd}$ = 790.34 kN	F_5 = 790.34 kN
Cl.6.2.6.1	**6. Column web panel in shear** $V_{wp,Rd} = \frac{0.9\, f_{y,wc}\, A_{vc}}{\sqrt{3}\, \gamma_{Mo}}$ A_{vc} = 267 * 12.8 = 3413.76 mm^2 $V_{wp,Rd}$ = 470.068 kN	F_6 = 470.068 kN
EN 1993-1-8 :2005 Cl.6.2.6.2 T 5.4 T 6.3	**7,8. Column web in compression** $F_{c,wc,Rd} = \frac{\omega\, k_{wc}\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{Mo}} \leq \frac{\omega\, k_{wc}\, \rho\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M1}}$ Transformation factor, $\beta \approx 1$ $\omega = \omega_1 = \frac{1}{[1 + 1.3 (b_{eff,c,wc}\, t_{wc} / A_{vc})^2]^{1/2}}$ For bolted end plate connection $b_{eff,c,wc} = t_{fb} + 2\sqrt{2}\, a_p + 5(t_{fc} + s) + s_p$	
Figure 6.6	 t_{fb} = 15.6 mm a_p = 8.4 mm t_{fc} = 20.5 mm t_p = 25.0 mm t_{wc} = 12.8 mm	
	s_p = 2 * t_p = 50.0 mm For a rolled I or H section column, s = r_c = 12.7 mm $b_{eff,c,wc}$ = 248.4 mm	

Reference	Calculation	Output
	A_{vc} = 3414 mm^2	
	$\omega = \omega_1 = 0.69$	
Cl.6.2.6.2 (2)	Assume longitudinal compressive stress, $\sigma_{com,Ed} < 0.7 f_{y,wc}$	
	k_{wc} = 1.0	
	t_{wc} = 12.8 mm	
	$\frac{\omega k_{wc} b_{eff,c,wc} t_{wc} f_{y,wc}}{\gamma_{Mo}}$ = 842573 N	
	Column web Bearing resistance = 842.573 kN	F_7 = 842.57 kN
Eq^n (6.13c)	$\bar{\lambda}_p = 0.932 \left(\frac{b_{eff,c,wc} d_{wc} f_{y,wc}}{E t_{wc}^2}\right)^{1/2}$	
	$b_{eff,c,wc}$ = 248.4 mm	
	for rolled I or H section column: $d_{wc} = h_c - 2(t_{fc} + r_c)$	
	h_c = 267 mm	
	t_{fc} = 20.5 mm	
	r_c = 12.7 mm	
	d_{wc} = 200.30 mm	
	E = 210.0 kN/mm^2	
	$\bar{\lambda}_p$ = 0.58	
	$\bar{\lambda}_p$ < 0.72	
	ρ = Buckling reduction factor = 1	
	$\frac{\omega k_{wc} \rho b_{eff,c,wc} t_{wc} f_{y,wc}}{\gamma_{M1}}$ = 842573 N	
	Column web Buckling resistance = 842.573 kN	F8 = 842.57 kN
EN 1993-1-8 :2005 T 3.4	**9. Bolt Shear**	
	Resistance of a single bolt in shear ($F_{v,Rd}$) is given by:	
	$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$	
	Where; α_v = 0.6 for class 8.8 bolts	
	$A = A_s$ = 353 mm^2	
	$F_{v,Rd} = \frac{0.6 \times 800 \times 353}{1.25} \times 10^{-3}$ = 135.552 kN	
	$V_{Rd} = n F_{v,Rd}$	
	No: of Bolts in Shear = 8	
	Shear Resistance of the connection = 1084.42 kN	F_9 = 1084.42 kN
EN 1993-1-8 :2005 T 3.4	**10. Bolt Bearing**	
	The bearing Resistance of a single bolt ($F_{b,Rd}$) is given by:	
	$F_{b,Rd} = \frac{k_1 \alpha_b f_{ub} d t_p}{\gamma_{M2}}$	
	Where α_b is the least value of α_d, $\frac{f_{ub}}{f_{y,p}}$ and 1	
		
	For the Direction of load transfer	
	For end Bolts $\alpha_d = e_1$ = 50 = 0.64	

Reference	Calculation	Output
	For inner Bolts $\alpha_d = \frac{p_1}{3d_0} - \frac{1}{4} = \frac{100}{3x\ \ 26} - \frac{1}{4} = 1.03$	
	$\frac{f_{ub}}{f_{u,p}} = \frac{800}{410} = 1.95$	
	$\alpha_b = 0.64$	
	For the perpendicular to the Direction of load transfer	
	For edge bolts k1, is the smaller of $2.8\frac{e_2}{d_0} - 1.7$ or 2.5	
	$2.8\frac{e_2}{d_0} - 1.7 = 2.8 \times \frac{75}{26} - 1.7 = 7.85$	
	Therefore for edge bolts, $k_1 = 2.50$	
	For inner bolts k1, is the smaller of $1.4\frac{p_2}{d_0} - 1.7$ or 2.5	
	$1.4\frac{p_2}{d_0} - 1.7 = 1.4 \times \frac{100}{26} - 1.7 = 3.68$	
	Therefore for inner bolts, $k_1 = 2.50$	
	Therefore the minimum bearing resistance for a bolt is:	
	$F_{b,Rd} = \frac{2.50 \times 0.64 \times 410 \times 24 \times 25}{1.10}$	
	= 358.392 kN	
	bearing resistance of the connection = 8 * 358.392	
	= 2867.13 kN	F_{10} = 2867.13 kN
Cl.6.2.6.7	**11. Beam flange and web in compression**	
Eq^n (6.21)	$F_{c,fb,Rd} = M_{c,Rd} / (h - t_{fb})$	
	$M_{c,Rd}$ = Design resistance of the beam	
	assume that the design shear force in the beam doesn't reduce $M_{c,Rd}$	
	therefore, from P363	
	$M_{c,Rd}$ = 649 kNm	
	$F_{c,fb,Rd} = \frac{649}{533 - 15.6} = 1254.11$ kN	F_{11} = 1254.11 kN

Reference	Calculation	Output

Summary of tension resistance

	Column flange bending	Column web in tension	End plate in bending	beam web in tension	minimum	effective resistance
Row 1, alone	398.36	790	377.26	N/A	377.26	377.26
Row 2,alone	398.36	790	406.66	672.42	398.36	
Row 2, with row 1	698.29		N/A	N/A	698.29	
Row 2					321.03	321.03
Row 3, alone	398.36	790	406.66	672.42	398.36	
Row 3, with row 1 &2	990.82		N/A	N/A	990.82	
Row 3					292.53	
Row 3, with row 2	690.89		813.31		690.89	
Row 3					369.86	292.53

Column web in Transverse compression = 842.57 kN

Beam flange and web in compression is not critical

Moment resistance

Effective resistance of bolt rows

The effective resistance of each of the three bolt rows in tension zone

$F_{t1,Rd}$ = 377.26 kN

$F_{t2,Rd}$ = 321.03 kN

$F_{t3,Rd}$ = 292.53 kN

Effective resistance should be reduced if the resistance of one of the higher rows exceeds

1.9 x $F_{t,Rd}$ = 386.323 kN

Resistance of bolt row 1 & 2 are less than this value.
Hence no reduction is required

Reference	Calculation	Output
EN1993-1-8 :2005 CL5.2.3	Total effective tension resistance $\sum F_{t,Rd}$ = 377.26 + 321.03 + 292.53 = 990.82 kN Compression resistance = 842.57 kN Here, total tension resistance exceeds the compression resistance reduction required = 148.25 kN $F_{t3,Rd}$ = 144.28 kN Moment resistance of the beam to column joint = 565.3 x 377.26 + 465.3 x 321.03 + 375.3 x 144.28 = 417 kNm M_p = Design plastic moment resistance of beam $M_p = (p_y . Z)/\gamma_{m0}$ $= \frac{275 \times 2072 \times 1000}{1.0}$ = 569.8 kNm $M_{con}/M_{p, beam}$ = 0.73147 Hence semi rigid connection	

Reference	Calculation	Output
Table 6.10 Table 6.11 BS EN1993-1-8 :2005	**Determination of rotational stiffness for EEP 3d**	

Stiffness coefficient

1.Column web panel in shear

$k1=(0.38\,A_{vc)/}\beta Z$

Z = Lever arm

β = Transformation parameter

A_{vc} = Shear area of the column

$k1=(0.38\,X\,(266.7x12.8))/(1X\,(670-50-(40+60)/2-25-15.6/2))$

= 2.41

2.Column web in compression

$k2=(0.7\,b_{eff,c,wc}\,X\,t_{wc)/}d_c$

$b_{eff,c,wc}$ = effective width

t_{wc} = column web thickness

d_c = clear depth of column

$k2=(0.7\,X248.4\,X12.8)/200.3$

k2 = 11.11

3. Column web in tension

$k_{3,1}=(0.7\,Xb_{eff,t1wc}\,X\,t_{wc)/}d_c$

$k_{3,1}=(0.7\,X\,166.5\,X\,12.8)/200.3$

$k_{3,1}$ = 7.45

$k_{3,2}=(0.7\,X\,95\,X\,12.8)/200.3$

$k_{3,2}$ = 4.25

$k_{3,3}=(0.7\,X\,161.5\,X\,12.8)/200.3$

$k_{3,3}$ = 7.22

Contribution from bolt row 4 is neglected.

Reference	Calculation	Output

4. Column flange in bending

$k_{4,1} = (0.9 \, X \, l_{eff} \, X \, t_{fc}^{3})/m^3$

$k_{4,1} = (0.9 \, X \, 166.5 \, X 20.5^3)/33.44^3$

$k_{4,1} = 34.52$

$k_{4,2} = (0.9 \, X \, 95 \, X 20.5^3)/33.44^3$

$k_{4,2} = 19.70$

$k_{4,3} = (0.9 \, X \, 161.5 \, X 20.5^3)/33.44^3$

$k_{4,3} = 33.49$

5. End plate bending

$k_{5,1} = (0.9 \, X \, l_{eff} \, X \, t_{p}^{3})/m^3$

$k_{5,1} = (0.9 \, X \, 125 \, X \, 25^3)/30.4^3$

$k_{5,1} = 62.57$

$k_{5,2} = (0.9 \, X \, 210.15 \, X \, 25^3)/38.55^3$

$k_{5,2} = 51.58$

$k_{5,3} = (0.9 \, X \, 169 \, X \, 25^3)/38.55^3$

$k_{5,3} = 41.48$

6. Bolts in tension

$k_{10} = (1.6 \, A_S)/L_b$

L_b - Bolt elongation length

$L_b = 48.5$

$k_{10} = 11.65$

Reference	Calculation	Output
	The effective stiffness coefficient of each bolt row is obtained as follows.	
BS EN1993-1-8 :2005 CL 6.3.3.1	$k_{eff,1}=1/((1/k_{3,1})+(1/k_{4,1})+(1/k_{5,1})+(1/k_{10}))$	
	$k_{eff,1}=1/((1/7.45)+(1/34.52)+(1/62.57)+(1/11.65))$	
	= 3.772	
	$k_{eff,2}=1/((1/4.25)+(1/19.7)+(1/51.58)+(1/11.65))$	
	= 2.555	
	$k_{eff,2}=1/((1/7.22)+(1/33.49)+(1/41.48)+(1/11.65))$	
	= 3.594	
	$h_1=670-25-15.6/2-C$	
	h_1 = 587.2 mm	
	$h_2=670-25-15.6/2-c-(d+e)$	
	h_2 = 487.2 mm	
	$h_3=670-25-15.6/2-c-(d+e)-90$	
	h_3 = 397.2 mm	
	Equivalent lever arm Zeq is	
BS EN1993-1-8 :2005 CL 6.3.3.1	Z_{eq} = 506.24 mm	
	k_{eq} = 9.65	
BS EN1993-1-8 :2005 CL 6.3.1	**The initial joint stiffness**	
	$S_{j,ini}=(E\,Z^2)/(\mu\sum(1/ki))$	
	μ = 1	
	$S_{j,ini}=(210x10^3x506.24^2)/((1/9.65)+(1/2.41)+(1/11.11))$	
	= 8.86E+07 kNmm / rad	

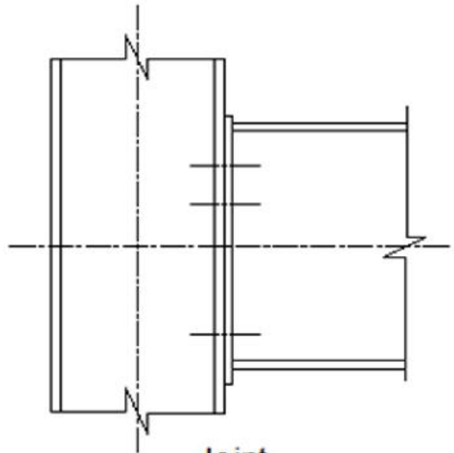
Reference	Calculation	Output
BS EN1993-1-8 :2005 CL5.2.2.5 figure 5.4	$EI_b/L_b = 210\times10^3 \times 55227\times10^4 / 6000\times10^3$	
	= 1.93E+07 kNmm / rad	
	I_b - second moment of area	
	L_b - span of beam	
	$0.5E * I_b/L_b < S_{j,ini} < 8*E*I_b/L_b$	
	$S_{j,ini}/EI_b/L_b =$ 4.58	
	Hence semi rigid connection	

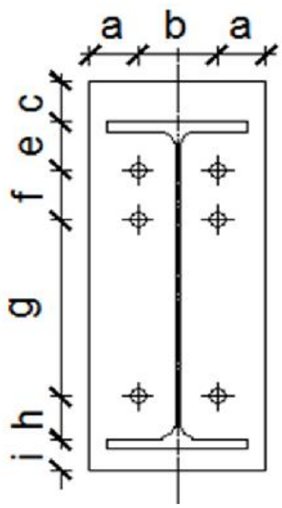
L	EIb/Lb	$S_{j,ini}/EI_b/L_b$	Classification
4000	28994175	3.05	semi rigid
6000	19329450	4.58	semi rigid
8000	14497087.5	6.11	semi rigid
10000	11597670	7.64	semi rigid
12000	9664725	9.16	rigid

Appendix B:

Design Calculations for Flush End Plate Connection- FEP 3d

Reference	Calculation	Output
	Connection Detail - FEP 3d	





Row 1

Row 2

a	=	75	mm
b	=	100	mm
c	=	25	mm
e	=	60	mm
f	=	90	mm
g	=	345	mm
h	=	60	mm
i	=	25	mm

Beam 533x210x92

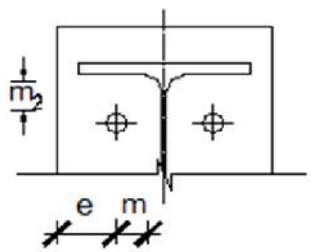
Beam height	=	533.1	mm
Flange width	=	209.3	mm
Mass per 1m length	=	92	kg/m
Flange thickness	=	15.6	mm
Web thickness	=	10.1	mm
Radius of gyration	=	12.7	mm

Column 254x254x107

column height	=	266.7	mm
Flange width	=	258.8	mm
Mass per 1m length	=	107	kg/m
Flange thickness	=	20.5	mm
Web thickness	=	12.8	mm
Radius of gyration	=	12.7	mm

Yield strength	$f_{y,p}$	=	265	N/mm^2
Ultimate tensile strength	$f_{u,p}$	=	410	N/mm^2

Reference	Calculation	Output
	End plate	
	End plate thickness t_p = 25 mm	
	End plate height h_p = 605 mm	
	End plate width w_p = 250 mm	
	Yield strength $f_{y,p}$ = 265 N/mm^2	
	Ultimate tensile strength $f_{u,p}$ = 410 N/mm^2	
	Bolts (class 8.8)	
	Bolt Diameter = 24 mm d_w = 40 mm	
	Tensile stress area = 353 mm^2	
	Total no of bolts = 6	
	no of bolts in tension = 4	
	no of bolts in shear = 6	
T 3.1	Yield strength f_{yb} = 640 N/mm^2	
	Ultimate tensile strength f_{ub} = 800 N/mm^2	
	fillet weld thickness	
	Beam flange to end plate weld thickness = 12 mm	
	Beam web to end plate weld thickness = 8 mm	

Reference	Calculation	Output
	Design Calculation according to EC3 **for FEP**	
	Partial factors for Resistance	
	Structural Steel	
EN 1993-1-8	γ_{MO} = 1.0	
N.A.2.15		
T NA.1	γ_{M1} = 1.00 (Resistance of a member to buckling)	
T NA.1	γ_{M2} = 1.10 (plates in bearing in bolted connections)	
	For tring resistance verification $\gamma_{M,u}$ = 1.10	
T NA.1	**Bolts** γ_{M2} = 1.25	
T NA.1	**Welds** γ_{M2} = 1.25	
EN 1993-1-8 :2005	**1. Bolts Tension**	
Cl.3.6.1 (1) T 3.4	$F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}}$	
	For non countersunk Bolts , k_2 = 0.9	F_1 =
	$F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}} = \frac{0.9 \times 800 \times 353}{1.25}$ = 203.328 kN	203.33 kN
Cl.6.2.6.5	**2. End plate in bending**	
	for flush end plate	
	w = 100 mm	
	e = 75 mm	
	e_{min} = 75 mm	
		
Cl.6.2.6.5 (1)	Bolt row 1 - First Bolt row below tension flange of beam	
T 6.6	Effective length for an end plate,	
	for circular patterns, $\ell_{eff,cp} = 2\pi m$	
	m = 38.55 mm	
	$\ell_{eff,cp}$ = 242.09 mm	
	for non circular patterns, $\ell_{eff,nc} = \alpha m$	
	m_2 = 34.8 mm	
Figure 6.11	$\lambda_1 = \frac{m}{m+e}$ = 0.34 , $\lambda_2 = \frac{m_2}{m+e}$ = 0.31	
	α = 7.5	
	$\ell_{eff,nc}$ = 289.13 mm	
T 6.2	**Mode 1** - Complete failure of the T-stub flange	
	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$	
	$\ell_{eff,1}$ = 242.09 mm	
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	e_w = 10 mm	
	n = e_{min} but n ≤ 1.25m (48.2)	
	n = 48.19 mm	
	$M_{p,1,Rd} = \frac{0.25 \Sigma \ell_{eff} t_p^2 f_y}{\gamma_{MO}} = \frac{0.25 * 242.09 * 25^2 * 265}{1.0}$	

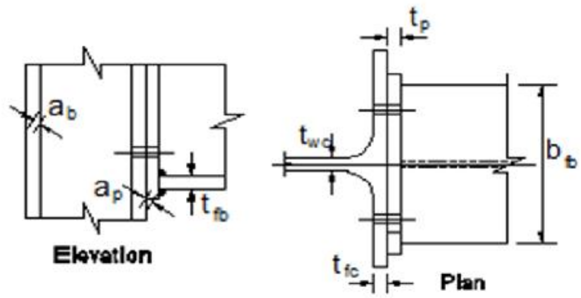
Reference	Calculation	Output
	$M_{p,1,Rd}$ = 10024.2 kNmm	
	$F_{T,1,Rd}$ = 1287 kN	
	Mode 2 - Bolt failure with yielding of the T-stub flange	
T 6.4	$\ell_{eff,2} = \ell_{eff,nc}$ = 289.13 mm	
T 6.2	$F_{T,2,Rd} = \frac{2M_{p,2,Rd} + n\Sigma F_{t,Rd}}{m+n}$	
	$M_{p,2,Rd} = \frac{0.25\Sigma\ell_{eff}t_p^2 f_y}{\gamma_{MO}} = \frac{0.25 * 289.13 * 25^2 * 265}{1.0}$	
	$M_{p2,Rd}$ = 11971.6 kNmm	
	$n = e_{min}$ but $n \leq 1.25m$ (48.2)	
	n = 48.1875 mm	
	$F_{t,Rd}$ = 203.33	
	$F_{T,2,Rd}$ = 501.96 kN	
	Mode 3	
	$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 2 * 203.33 = 406.656 kN	
	Resistance only from Row 1 bolts = 406.66 kN	
Cl.6.2.6.5 (1)	Bolt row 2 - Other end bolt row	
T 6.6	Effective length for an end plate,	
	for circular patterns, $\ell_{eff,cp} = 2\pi m$	
	m = 38.55 mm	
	$\ell_{eff,cp}$ = 242.09 mm	
	for non circular patterns, $\ell_{eff,nc}$ = 4m + 1.25e	
	= 247.95 mm	
	$\ell_{eff,nc}$ = 247.95 mm	
T 6.2	**Mode 1** - Complete failure of the T-stub flange	
	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$	
	$\ell_{eff,1}$ = 242.09 mm	
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	e_w = 10 mm	
	$n = e_{min}$ but $n \leq 1.25m$ (48.2)	
	n = 48.19 mm	
	$M_{p,1,Rd} = \frac{0.25\Sigma\ell_{eff}t_p^2 f_y}{\gamma_{MO}} = \frac{0.25 * 242.09 * 25^2 * 265}{1.0}$	
	$M_{p,1,Rd}$ = 10024.2 kNmm	
	$F_{T,1,Rd}$ = 1286.52 kN	
	Mode 2 - Bolt failure with yielding of the T-stub flange	
T 6.4	$\ell_{eff,2} = \ell_{eff,nc}$ = 247.95 mm	
T 6.2	$F_{T,2,Rd} = \frac{2M_{p,2,Rd} + n\Sigma F_{t,Rd}}{m+n}$	
	$M_{p,2,Rd} = \frac{0.25\Sigma\ell_{eff}t_p^2 f_y}{\gamma_{MO}} = \frac{0.25 * 247.95 * 25^2 * 265}{1.0}$	
	$M_{p2,Rd}$ = 10266.7 kNmm	

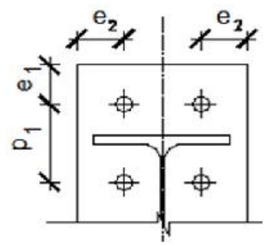
Reference	Calculation	Output
	$n = e_{min}$ but $n \leq 1.25m$ (48.2)	
	n = 48.1875 mm	
	$F_{t,Rd}$ = 203.33	
	$F_{T,2,Rd}$ = 462.65 kN	
	Mode 3	
	$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 2 * 203.33 = 406.656 kN	
	Resistance only from Row 2 bolts = 406.66 kN	
Cl.6.2.6.5 (1)	Bolt row 1 & 2 - combined	
	row 1 first bolt row below the tension flange of the beam	
	row 2 other end bolt row	
T 6.6	Effective length for an end plate,	
	row 1	
	for circular patterns, $\ell_{eff,cp} = \pi m + p$	
	m = 38.55 mm p = 90 mm	
	$\ell_{eff,cp}$ = 211.05 mm	
	for non circular patterns, $\ell_{eff,nc}$ = 0.5p + αm-(2m + .625e)	
Figure 6.11	$\lambda_1 = \dfrac{m}{m + e}$ = 0.34 , $\lambda_2 = \dfrac{m_2}{m + e}$ = 0.31	
	α = 7.5	
	$\ell_{eff,nc}$ = 210.15 mm	
	row 2	
	for circular patterns, $\ell_{eff,cp} = \pi m + p$	
	m = 38.55 mm p = 90 mm	
	$\ell_{eff,cp}$ = 211.05 mm	
	for non circular patterns, $\ell_{eff,nc}$ = 2m + .625e+0.5*p	
	= 169 mm	
	total effective length for this group of rows	
	$\Sigma\ell eff,cp$ = 211 + 211 = 422 mm	
	$\Sigma\ell eff,nc$ = 210 + 169 = 379 mm	
T 6.2	**Mode 1** - Complete failure of the T-stub flange	
	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$	
	$\Sigma\ell_{eff,1}$ = 379.13 mm	
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	e_w = 10 mm	
	$n = e_{min}$ but $n \leq 1.25m$ (48.2)	
	n = 48.19 mm	
	$M_{p,1,Rd} = \dfrac{0.25 \Sigma\ell_{eff} t_p^2 f_y}{\gamma_{MO}} = \dfrac{0.25 * 379.13 * 25^2 * 265}{1.0}$	
	$M_{p,1,Rd}$ = 15698.1 kNmm	

Reference	Calculation	Output
	$F_{T,1,Rd}$ = 2014.72 kN	
	Mode 2 - Bolt failure with yielding of the T-stub flange	
T 6.4	$\ell_{eff,2} = \ell_{eff,nc}$ = 379.13 mm	
T 6.2	$F_{T,2,Rd} = \dfrac{2 M_{p,2,Rd} + n\sum F_{t,Rd}}{m + n}$	
	$M_{p,2,Rd} = \dfrac{0.25 \sum\ell_{eff} t_p^2 f_y}{\gamma_{MO}} = \dfrac{0.25 * 379.13 * 25^2 * 265}{1.0}$	
	$M_{p\ 2,Rd}$ = 15698.1 kNmm	
	n = e_{min} but n ≤ 1.25m (48.2)	
	n = 48.1875 mm	
	$F_{t,Rd}$ = 203.33	
	$F_{T,2,Rd}$ = 813.81 kN	
	Mode 3	
	$F_{T,3,Rd} = \sum F_{t,Rd}$ = 4 * 203.33 = 813.312 kN	
	Resistance only from 1 & 2 bolts = 813.31 kN	
	Resistance for End plate in bending = 406.66 kN	F_2 = 406.66 kN
Cl.6.2.6.4	**3.Column flange in transverse bending**	
Cl.6.2.6.4 (1)	- each individual bolt-row required to resist tension	
Cl.6.2.6.4 (1)	Bolt row 1 - end bolt row	
T 6.4	Effective length of an unstiffened column flange	
	for circular patterns, $\ell_{eff,cp}$ = smaller of $2\pi m$ and $\pi m + 2 e_1$	
	for welded end plate narrower than column flange	
	r_c = 12.7 mm	
	m = 33.44 mm	
	e = 79.4 mm	
	e_1 is large so it will not be critical	
	e_{min} = 75 mm	
	$2\pi m$ = 210.003 mm	
	$\ell_{eff,cp}$ = 210.00 mm	
	for non circular patterns,	
	$\ell_{eff,nc}$ = smaller of 4m + 1.25 e and 2m + 0.625 e + e_1	
	4m + 1.25 e = 233.01 mm	
	$\ell_{eff,nc}$ = 233.01 mm	
	Mode 1	
	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$	
	$\ell_{eff,1}$ = 210.00 mm	
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	e_w = 10 mm	
	n = e_{min} but n ≤ 1.25m (41.8)	
	n = 41.80 mm	
	$M_{p,1,Rd} = \dfrac{0.25 \sum\ell_{eff} t_f^2 f_y}{\gamma_{MO}} = \dfrac{0.25* 210.00 * 20.5^2 * 265}{1.0}$	
	$M_{p,1,Rd}$ = 5846.8 kNmm	
	$F_{T,1,Rd}$ = 899.69 kN	

Reference	Calculation	Output
	Mode 2	
T 6.4	$\ell_{eff,2}$ = $\ell_{eff,nc}$ = 233.01 mm	
T 6.2	$F_{T,2,Rd} = \dfrac{2M_{p,2,Rd} + n\Sigma F_{t,Rd}}{m+n}$	
	$M_{p,2,Rd} = \dfrac{0.25\,\Sigma\ell_{eff}\,t_f^2\,f_y}{\gamma_{MO}} = \dfrac{0.25 * 233.01 * 20.5^2 * 265}{1.0}$	
	$M_{p\,2,Rd}$ = 6487.4 kNmm	
	n = e_{min} but n ≤ 1.25m (41.8)	
	n = 41.8 mm	
	$F_{t,Rd}$ = 203.33	
	$F_{T,2,Rd}$ = 398.36 kN	
	Mode 3	
	$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 2 * 203.33 = 406.656 kN	
	Resistance only from Row 1 bolts = 398.36 kN	
Cl.6.2.6.4 (1)	Bolt row 1 and 2 combined	
	Bolt row 1,2 - end bolt row	
T 6.4	Effective length of an unstiffened column flange	
	for circular patterns, $\Sigma\ell_{eff,cp}$ = 2*(πm + p)	
	for welded end plate narrower than column flange	
	r_c = 12.8 mm	
	m = 33.44 mm	
	e = 79.4 mm	
	p = 90 mm	
	$\Sigma\ell_{eff,cp}$ = 390.003 mm	
	for non circular patterns,	
	$\ell_{eff,nc}$ = 2*(2m + 0.625e + 0.5p)	
	$\Sigma\ell_{eff,nc}$ = 323.01 mm	
	Mode 1	
	$\ell_{eff,1}$ = $\ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$	
	$\Sigma\ell_{eff,1}$ = 323.01 mm	
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	e_w = 10 mm	
	$M_{p,1,Rd} = \dfrac{0.25\,\Sigma\ell_{eff}\,t_f^2\,f_y}{\gamma_{MO}} = \dfrac{0.25 * 323.01 * 20.5^2 * 265}{1.0}$	
	$M_{p,1,Rd}$ = 8993.10 kNmm	
	$F_{T,1,Rd}$ = 1383.84 kN	
	Mode 2	
T 6.4	$\ell_{eff,2}$ = $\ell_{eff,nc}$ = 323.01 mm	
T 6.2	$F_{T,2,Rd} = \dfrac{2M_{p,2,Rd} + n\Sigma F_{t,Rd}}{m+n}$	

Reference	Calculation	Output
	$M_{p,2,Rd} = \frac{0.25 \sum \ell_{eff} t_f^2 f_y}{\gamma_{MO}} = \frac{0.25 * 323.01 * 20.5^2 * 265}{1.0}$	
	$M_{p\,2,Rd}$ = 8993.10 kNmm	
	n = e_{min} but n ≤ 1.25m	
	n = 41.8 mm	
	$F_{t,Rd}$ = 203.33	
	$F_{T,2,Rd}$ = 690.89 kN	
	Mode 3	
	$F_{T,3,Rd} = \sum F_{t,Rd}$ = 4 * 203.33 = 813.312 kN	
	Resistance from Bolt Row 1 & 2 combination = 690.89 kN	
	Resistance for Column flange in transverse bending = 398.36 kN	F_3 = 398.36 kN
Cl.6.2.6.8 (1) Eq" (6.22)	**4. Beam web in tension**	
	$F_{t,wb,Rd} = \frac{b_{eff,t,wb}\, t_{wb}\, f_{y,wb}}{\gamma_{Mo}}$	
	$b_{eff,t,wb}$ = 242 mm	
	t_{wb} = 10.1 mm	
	$F_{t,wb,Rd}$ = 672.42 kN	F_4 = 672.42 kN
EN 1993-1-8 :2005	**5. Column web in tension**	
Cl.6.2.6.3 (1)	$F_{t,wc,Rd} = \frac{\omega\, b_{eff,t,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{Mo}}$	
Cl.6.2.6.3 (3)	For Bolted connection	
	$b_{eff,t,wc}$ = 210.00 mm	
	t_{wc} = 12.8 mm	
	ω = 1.00	
	$F_{t,wc,Rd}$ = 712.33 kN	F_5 = 712.33 kN
Cl.6.2.6.1	**6. Column web panel in shear**	
	$V_{wp,Rd} = \frac{0.9 f_{y,wc} A_{vc}}{\sqrt{3}\, \gamma_{Mo}}$	
	A_{vc} = 267 * 12.8 = 3413.76 mm^2	
	$V_{wp,Rd}$ = 470.068 kN	F_6 = 470.068 kN
EN 1993-1-8 :2005	**7,8. Column web in compression**	
Cl.6.2.6.2	$F_{c,wc,Rd} = \frac{\omega\, k_{wc}\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{Mo}} \le \frac{\omega\, k_{wc}\, \rho\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M1}}$	
T 5.4	Transformation factor, β ≈ 1	
T 6.3	$\omega = \omega_1 = \frac{1}{[1 + 1.3 (b_{eff,c,wc}\, t_{wc} / A_{vc})^2]^{1/2}}$	
	For bolted end plate connection	
	$b_{eff,c,wc} = t_{fb} + 2\sqrt{2}\, a_p + 5(t_{fc} + s) + s_p$	

Reference	Calculation	Output
Figure 6.6		
	t_{fb} = 15.6 mm	
	a_p = 8.4 mm	
	t_{fc} = 20.5 mm	
	t_p = 25.0 mm	
	t_{wc} = 12.8 mm	
	s_p = 2 * t_p = 50.0 mm	
	For a rolled I or H section column, s = r_c = 12.7 mm	
	$b_{eff,c,wc}$ = 248.4 mm	
	A_{vc} = 3414 mm^2	
	ω = ω_1 = 0.69	
Cl.6.2.6.2 (2)	Assume longitudinal compressive stress, $\sigma_{com,Ed}$ < 0.7 $f_{y,wc}$	
	k_{wc} = 1.0	
	t_{wc} = 12.8 mm	
	$\dfrac{\omega\, k_{wc}\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{Mo}}$ = 842573 N	
	Column web Bearing resistance = 842.573 kN	F_7 = 842.57 kN
Eq^n (6.13c)	$\bar{\lambda}p = 0.932 \left(\dfrac{b_{eff,c,wc}\, d_{wc}\, f_{y,wc}}{E\, t_{wc}^2}\right)^{1/2}$	
	$b_{eff,c,wc}$ = 248.4 mm	
	for rolled I or H section column: d_{wc} = h_c - 2 (t_{fc} + r_c)	
	h_c = 267 mm	
	t_{fc} = 20.5 mm	
	r_c = 12.7 mm	
	d_{wc} = 200.30 mm	
	E = 210.0 kN/mm^2	
	$\bar{\lambda}p$ = 0.58	
	$\bar{\lambda}p$ < 0.72	
	ρ = Buckling reduction factor = 1	
	$\dfrac{\omega\, k_{wc}\, \rho\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M1}}$ = 842573 N	
	Column web Buckling resistance = 842.573 kN	F8 = 842.57 kN
EN 1993-1-8 :2005 T 3.4	**9. Bolt Shear**	
	Resistance of a single bolt in shear ($F_{v,Rd}$) is given by:	
	$F_{v,Rd} = \dfrac{\alpha_v\, f_{ub}\, A}{\gamma_{M2}}$	
	Where; α_v = 0.6 for class 8.8 bolts	
	A = A_s = 353 mm^2	
	$F_{v,Rd} = \dfrac{0.6 \times 800 \times 353}{1.25} \times 10^{-3}$ = 135.552 kN	
	V_{Rd} = n $F_{v,Rd}$	
	No: of Bolts in Shear = 6	
	Shear Resistance of the connection = 813.312 kN	F_9 = 813.312 kN

Reference	Calculation	Output
EN 1993-1-8 :2005 T 3.4	**10. Bolt Bearing**	
	The bearing Resistance of a single bolt ($F_{b, Rd}$) is given by:	
	$F_{b, Rd} = \dfrac{k_1 \alpha_b f_{ub} d t_p}{\gamma_{M2}}$	
	Where α_b is the least value of α_d, $\dfrac{f_{ub}}{f_{y,p}}$ and 1	
		
	For the Direction of load transfer	
	For end Bolts $\alpha_d = \dfrac{e_1}{3d_0} = \dfrac{25}{3x \quad 26} = 0.32$	
	For inner Bolts $\alpha_d = \dfrac{p_1}{3d_0} - \dfrac{1}{4} = \dfrac{60}{3x \quad 26} - \dfrac{1}{4} = 0.52$	
	$\dfrac{f_{ub}}{f_{u,p}} = \dfrac{800}{410} = 1.95$	
	$\alpha_b = 0.32$	
	For the perpendicular to the Direction of load transfer	
	For edge bolts k1, is the smaller of $2.8 \dfrac{e_2}{d_0} - 1.7$ or 2.5	
	$2.8 \dfrac{e_2}{d_0} - 1.7 = 2.8 \text{ x } \dfrac{75}{26} - 1.7 = 7.85$	
	Therefore for edge bolts, $k_1 = 2.50$	
	For inner bolts k1, is the smaller of $1.4 \dfrac{p_2}{d_0} - 1.7$ or 2.5	
	$1.4 \dfrac{p_2}{d_0} - 1.7 = 1.4 \text{ x } \dfrac{100}{26} - 1.7 = 3.68$	
	Therefore for inner bolts, $k_1 = 2.50$	
	Therefore the minimum bearing resistance for a bolt is:	
	$F_{b, Rd} = \dfrac{2.50 \text{ x } 0.32 \text{ x } 410 \text{ x } 24 \text{ x } 25}{1.10}$	
	$= 179.196$ kN	
	bearing resistance of the connection: $= 6 * 179.196$	
	$= 1075.17$ kN	F_{10} = 1075.17 kN
Cl.6.2.6.7	**11. Beam flange and web in compression**	
Eq^n (6.21)	$F_{c,fb,Rd} = M_{c,Rd} / (h - t_{fb})$	
	$M_{c,Rd}$ = Design resistance of the beam	
	assume that the design shear force in the beam doesn't reduce $M_{c,Rd}$	
	therefore, from P363	
	$M_{c,Rd} = 649$ kNm	
	$F_{c,fb,Rd} = \dfrac{649}{533 - 15.6} = 1254.11$ kN	F_{11} = 1254.11 kN

Reference	Calculation	Output
	Summary of tension resistance	

	Column flange bending	Column web in tension	End plate in bending	beam web in tension	minimum	effective resistance
Row 1, alone	398.36	712	406.66	N/A	398.36	398.36
Row 2,alone	398.36	712	406.66	672.42	398.36	
Row 2, with row 1	690.89		813.31	N/A	690.89	
Row 2					292.53	292.53

Column web in Transverse compression = 842.57 kN

Beam flange and web in compression is not critical

Moment resistance

Effective resistance of bolt rows

The effective resistance of each of the three bolt rows in tension zone

$F_{t1,Rd}$ = 398.36 kN

$F_{t2,Rd}$ = 292.53 kN

Effective resistance should be reduced if the resistance of one of the higher rows exceeds

1.9 x F t,Rd = 386.323 kN

$F_{t1,Rd}$ = 386.32 kN

Total effective tension resistance

∑ F t,Rd = 386.32 + 292.53

= 678.85 kN

Compression resistance = 842.57 kN

Moment resistance of the beam to column joint

= 487.2 x 386.32 + 397.2 x 292.53

= 304 kNm

Reference	Calculation	Output
	M_p = Design plastic moment resistance of beam	
	$M_p = (p_y . Z)/\gamma_{m0}$	
EN1993-1-8 :2005 CL5.2.3	$= \frac{275 \times 2072 \times 1000}{1.0}$	
	= 569.8 kNm	
	$M_{con}/M_{p, beam}$ = 0.53424	
	Hence semi rigid connection	

Reference	Calculation	Output
Table 6.10 Table 6.11 BS EN1993-1-8 :2005	**Determination of rotational stiffness** **for EEP 3d** **Stiffness coefficient** **1.Column web panel in shear** $k1 = (0.38\ A_{vc})/\beta Z$ Z = Lever arm β = Transformation parameter A_{vc} = Shear area of the column $k1 = (0.38\ X\ (266.7x12.8))/(1X\ (605-25-60-25-15.6/2))$ = 2.66 **2.Column web in compression** $k2 = (0.7\ b_{eff,c,wc}\ X\ t_{wc})/d_c$ $b_{eff,c,wc}$ = effective width t_{wc} = column web thickness d_c = clear depth of column $k2 = (0.7\ X 248.4\ X 12)/200.3$ k2 = 11.11 **3. Column web in tension** $k_{3,1} = (0.7\ Xb_{eff,t1wc}\ X\ t_{wc})/d_c$ $k_{3,1} = (0.7\ X\ 161.5\ X\ 12.8)/200.3$ $k_{3,1}$ = 7.22 $k_{3,2} = (0.7\ X\ 161.5\ X\ 12.8)/200.3$ $k_{3,2}$ = 7.22 **4. Column flange in bending** $k_{4,1} = (0.9\ X\ l_{eff}\ X\ t_{fc}^3)/m^3$ $k_{4,1} = (0.9\ X\ 161.5\ X 20.5^3)/33.44^3$ $k_{4,1}$ = 33.49	

Reference	Calculation	Output
	$k_{4,2}=(0.9\ X\ 161.5\ X 20.5^3\ )/33.44^3$ $k_{4,2} = 33.49$ **5. End plate bending** $k_{5,1}=(0.9\ X\ l_{eff}\ X\ t_p^3)/m^3$ $k_{5,1}=(0.9\ X\ 210.15\ X\ 25^3\ )/38.55^3$ $k_{5,1} = 51.58$ $k_{5,2}=(0.9\ X\ 169\ X\ 25^3\ )/38.55^3$ $k_{5,2} = 41.48$ **6. Bolts in tension** $k_{10}=(1.6\ A_S)/L_b$ L_b - Bolt elongation length $L_b = 48.5$ $k_{10} = 11.65$ The effective stiffness coefficient of each bolt row is obtained as follows.	
BS EN1993-1-8 :2005 CL 6.3.3.1	$k_{eff,1}=1/((1/k_{3,1})+(1/k_{4,1})+(1/k_{5,1})+(1/k_{10}))$ $k_{eff,1}=1/((1/7.45)+(1/34.52)+(1/62.57)+(1/11.65))$ = 3.656 $k_{eff,2}=1/((1/4.25)+(1/19.7)+(1/51.58)+(1/11.65))$ = 3.594 $h_1=605-25-15.6/2-25-60$ $h_1 = 487.2$ mm $h_2=605-25-15.6/2-c-(d+e)$	

Reference	Calculation	Output
	h_2 = 397.2 mm	
	Equivalent lever arm Zeq is	
BS EN1993-1-8 :2005 CL 6.3.3.1	Z_{eq} = 447.16 mm	
	k_{eq} = 7.18	
BS EN1993-1-8 :2005 CL 6.3.1	**The initial joint stiffness**	
	$S_{j,ini} = (E\, Z^{2})/(\mu \sum(1/ki))$	
	μ = 1	
	$S_{j,ini} = (210x10^3 x506.24^2)/((1/9.65)+(1/2.41)+(1/11.11))$	
	= 6.94E+07 kNmm / rad	
BS EN1993-1-8 :2005 CL5.2.2.5 figure 5.4	$EI_b/L_b = 210x10^3x55227x10^4/6000x10^3$	
	= 1.93E+07 kNmm / rad	
	I_b - second moment of area L_b - span of beam	
	$0.5E * I_b/L_b < S_{j,ini} < 8*E*I_b/L_b$	
	$S_{j,ini}/EI_b/L_b =$ 3.59	
	Hence semi rigid connection	

L	EIb/Lb	$S_{j,ini}/EI_b/L_b$	Classification
4000	28994175	2.39	semi rigid
6000	19329450	3.59	semi rigid
8000	14497087.5	4.79	semi rigid
10000	11597670	5.99	semi rigid
12000	9664725	7.18	semi rigid