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ECONOMIC ASPECTS OF THE USE OF PARTIAL AND FULL STRENGTH JOINTS ON MULTI-STOREY UNBRACED STEEL FRAMES

(ASPEK EKONOMI KEGUNAAN SAMBUNGAN SEPARA TEGAR DAN TEGAR UNTUK KERANGKA KELULI BERTINGKAT TAK DIREMBAT)

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TAJUK PROJEK :
ECONOMIC ASPECTS OF THE USE OF PARTIAL AND FULL STRENGTH JOINTS ON MULTI-STOREY UNBRACED STEEL FRAMES

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Dedicated to *Arizu Sulaiman*, *Anis Saggaff*, *Tan CS* and *Shek PN*.

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ABSTRACT

Connections are usually designed as pinned or rigid although the actual behaviour is known to fall between these two extreme cases. The use of partial strength or semi-rigid connections has been encouraged by codes and studies on the matter known as semi-continuous construction have proven that substantial savings in steel weight of the overall construction. The objective of this research is to develop a series of standardized partial strength connections tables of flush end-plate connections and extended end plate for unbraced steel frames with the use of trapezoidal web profiled steel (TWP) sections. The range of standard connections presented in tabulated form is limited to fourteen tables comprised of different geometrical aspects of the connections. The moment capacity, shear capacity, geometrical aspects of the connections, the size of beams, and columns that are suitable with the connections are included in the standardized tables. A method proposed by Steel Construction Institute(SCI) which take into account the requirements in Eurocode 3 and BS 5950:2000 Part 1 were adopted to predict the moment capacity and shear capacity in developing the tables. Although the use of the proposed method is intended for hot rolled section, it has been proven via experimental tests that to apply the same proposed method to TWP section, provided that the predicted failure modes should comply with the requirements of Eurocode 3 and BS 5950:2000 Part 1. The TWP section should at least classified as compact section. The moment capacity and shear capacity in the standard tables presented in this report showed good agreement with the requirement of Eurocode 3 and BS 5950:2000 Part 1.

ABSTRAK

Sambungan biasanya direkabentuk sebagai pin dan ikat tegar walaupun sifat sebenarnya jatuh di antara kedua-dua jenis sambungan ini. Penggunaan sambungan separa tegar telah digalakkan oleh piawaian dan penyelidikan dalam pembinaan separa-bersambungan, di mana ia telah dibuktikan penjimatan dalam berat keluli untuk kos keseluruhan pembinaan. Objektif penyelidikan ini adalah untuk membangun satu siri jadual-jadual rekabentuk sambungan separa tegar berbentuk Plat Hujung untuk kerangka tak dirembat dengan pemakaian keratan keluli TWP. Rangka sambungan piawaian disusun dan dihadkan jumlahnya empat belas jadual yang terdiri daripada aspek geometri yang berbeza bagi sambungan. Catatan rintangan moment, rintangan ricih, aspek geometri sambungan, size rasuk and size tiang yang bersesuaian dengan sambungan digolong bersama dalam jadual piawaian. Satu kaedah dicadang oleh Steel Construction Institute (SCI) yang mengambilkira peraturan dalam Eurocode 3 dan BS5950:2000 Part 1 telah dipakai untuk menjangka rintangan moment dan rintangan ricih sambungan dalam pembangunan jadual. Walaupun penggunaan asal yang dicadang adalah pada keratan keluli tergelek panas, tetapi ia telah dibuktikan melalui ujian eksperimen untuk memakai kaedah yang sama pada keratan keluli TWP, dengan syarat bentuk-bentuk kegagalan mesti sama dengan peraturan Eurocode 3 dan BS5950:2000 Part 1. Kelas keratan keluli TWP mestilah sekurang-kurangnya keratan kompat. Rintangan moment dan rintangan ricih yang ditunjukkan dalam jadual piawaian dalam laporan ini telah mematuhi keperluan dalam Eurocode 3 dan BS5950:2000 Part 1.

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A	-	Area
A_c	-	Area of cross section
A_g	-	Gross area of section
A_e	-	Effective area of section
B_b	-	Breadth of beam
B_c	-	Breadth of column
B_t	-	Design tension resistance of a single bolt-plate assembly
C_a	-	Size effect factor for external pressure
$\mathrm{C_p}$	-	Net pressure coefficient for wind load
$\mathrm{C_{pe}}$	-	External pressure coefficient for wind load
$\mathrm{C_{pi}}$	-	Internal pressure coefficient for wind load
C_r	-	Dynamic augmentation factor
D	-	Overall depth
$\mathrm{D_b}$	-	Overall depth of beam
$\mathrm{D_{bolt}}$	-	Overall depth of bolt
$\mathrm{D_c}$	-	Overall depth of column
F_R	-	Compression force
E	-	Modulus of elasticity
F_c	-	Applied axial load due to vertical loading, or a combination of vertical loads and wind loads
F_i	-	Force in component i of the connection due to the moment M
$F_{i,Rd}$	-	Design resistance of component I
$F_{ti.Rd}$	-	Design value of the effective resistance of an individual row of bolts
G_k	-	Dead Load
H	-	lever arm moment
$\mathrm{H_r}$	-	Building reference height
I	-	Second moment of area

I_b	-	Second moment of area for beam
I_{xx}	-	Second moment of area at x-x (major) axis
I_{yy}	-	Second moment of area at y-y (minor) axis
K	-	Coefficients
K_b	-	Building type factor
K_{bb}	-	Stiffness coefficient for bottom beam
K_{bt}	-	Stiffness coefficient for top beam
K_c	-	Stiffness coefficient for column
L	-	Length
L_{eff}	-	Effective length
M	-	Moment
M_b	-	Lateral torsional buckling resistance moment
M_{bs}	-	Lateral torsional buckling resistance moment for simple design
M_{bp}	-	Lateral torsional buckling resistance moment for end plate
M_c	-	Moment capacity
M_{cx}	-	Moment capacity at major axis (x-x axis)
M_j	-	Moment resistance of joint
M_p	-	Moment resistance of end plate
M_{Rd}	-	Moment resistance
$\overline{M}$	-	Equivalent uniform moment
M_x	-	Applied moment about the major axis due to appropriate combination of vertical loading, notional horizontal forces and wind loads
M_y	-	Applied moment about the minor axis due to appropriate combination of vertical loading
M_u	-	Ultimate moment
P	-	Load pressure
P-Δ	-	Load-deflection behaviour
P_c	-	Compressive resistance
P_t	-	Tension resistance
P_v	-	Shear load
Q_k	-	Live Load
S	-	Plastic modulus

S_a	-	Altitude factor for wind load
S_b	-	Terrain and building factor for wind load
S_d	-	Directional factor for wind load
S_j	-	Limiting stiffness
S_j	-	Rotational stiffness
$S_{j,\,unl}$	-	Unloaded stiffness
$S_{j,ini}$	-	Initial stiffness
S_s	-	Seasonal factor
S_p	-	Probability factor
T	-	Flange thickness
T_c	-	Column flange thickness
U_f	-	Ultimate tensile strength of bolt
U_f	-	Ultimate tensile strength flange of a steel section
V_b	-	basic wind speed obtained from figure 6 in BS 6399-2
V_e	-	Effective wind speed
V_s	-	Site wind speed
W_k	-	Wind load
Z_y	-	Elastic modulus about the minor axis
a	-	Robertson coefficient
$b_{eff,t,wc}$	-	Effective width of column’s web depth in compression
d	-	Clear depth / inner depth
d_b	-	Clear depth / inner depth of beam’s web
d_c	-	Clear depth / inner depth of column’s web
e	-	Eccentricity
e_p	-	Eccentricity of plate
h_{bolt}	-	Distance of between bolts
h_i	-	Distance from that bolt row to the centre of resistance of the compression zone
h_{nut}	-	Distance between nuts
h_r	-	Distance between bolt-row *r* and the centre of compression
h_T	-	Total height of the multi-storey frame
k_i	-	stiffness factor for component *I*

l_{eff}	-	smallest value of effective lengths of column’s flange without web stiffeners
m	-	Moment factor
$\mathrm{p_s}$	-	Design pressure
p_{yc}	-	Design strength of column
p_{yp}	-	Design strength of endplate
r	-	Radius
t	-	Thickness
t_f	-	Thickness of flange
t_{fb}	-	Thickness of beam flange
t_{fc}	-	Thickness of column flange
t_w	-	Thickness of web
t_{washer}	-	Thickness of washer
t_{wc}	-	Thickness of the web of column
t_p	-	Thickness of end plate
p_y	-	yield stress
q	-	Pressure
$\mathrm{q_s}$	-	Dynamic pressure
r_{xx}	-	Radius of gyration at x-x (major) axis
r_{yy}	-	Radius of gyration at y-y (minor) axis
V	-	wind velocity
z	-	lever arm (refer to Clause 6.2.5 in EC 3)
z_{eq}	-	Equivalent lever arm
μ_i	-	Modification factor
ϕ	-	Rotation / Diameter
ϕ_b	-	Rotation of beam
ϕ_c	-	Rotation of column
$Ø_{LT}$	-	Rotation of lateral torsional buckling
θ_k	-	Rotation coefficient
μ	-	Stiffness ratio of $\frac{S_{j,ini}}{S_j}$
μ_i	-	Stiffness modification factor

δ	-	Deflection
λ	-	Slenderness ratio
λ_{xx}	-	Slenderness ratio at x-x axis
λ_{yy}	-	Slenderness ratio at x-x axis
λ_{LT}	-	Slenderness ratio for lateral torsional buckling

CHAPTER I

INTRODUCTION

1.1 General

Construction of structures using steel as the construction material, nowadays, has becomes one of the major alternatives to the conventional reinforced concrete. Its popularity has increased and can be seen by the erection of many major structures around the world even in Malaysia such as the Kuala Lumpur International Airport, Kuala Lumpur City Centre and the Bukit Jalil National Stadium. The interest in the use of this material since the beginning has triggered many researches and development works to be carried out with the aim of improving the design methods to allow more economical, practical and strong constructions. Despite of the increasing in usage and the advent of new development, most structures especially the conventional steel buildings are still using the methods of simple (pinned) design and rigid (continuous) design. However, it is well known that the true behaviour lies between these two extremes; and connections, on the other hand, play major roles in transmitting required actions between the individual members. Simple design results in a more conservative but utilising heavier sections whereas the continuous design requires more rigorous non-economical connection to ensure enough moment resistance.

Hence, an alternative method of design called the semi-rigid (semi-continuous) design is more suitable which can be chosen to produce the most economic balance between the primary benefits associated with the two conventional designs. Moreover, the introduction of partial-strength connection in this semi-rigid design only slightly increases the complexity compared to the simple design but yet is able to produce significant reduction in beam depths and weight. Many researchers in the past have shown significant reduction in the economical aspect of semi-continuous construction even though the real benefits may vary among structures, location, and relative costs of materials and labour in a particular country

In this study, Trapezoid Web Profiled (TWP) sections will be used as beam elements since it is believed to be able to produce even more reduction in beam depths and weight. Figure 1.1 shows the typical TWP section, which formed by welding the flanges to the trapezoidal-shaped web of 2 mm to 8 mm thickness. Since the use of TWP sections in the real semi-continuous construction has not been utilised yet, studies on the connections and sub-assemblages of frames have to be conducted in order to fully understand the behaviour and to incorporate the findings in the design of semi-rigid steel frames. This may include the aspects of moment resistance, rotational stiffness and rotational capacity of the connections, and the overall behaviour associated with the multi-storey unbraced frames construction.

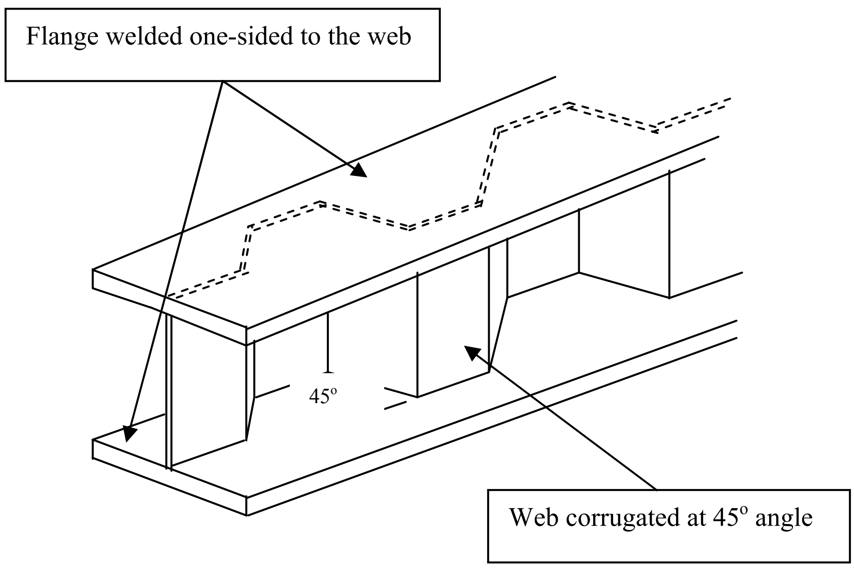


Figure 1.1: Typical shape of TWP section showing the trapezoidal corrugated web

1.2 Braced and Unbraced Steel Frames

Multi-storey frames may be divided into two distinct categories for the purpose of design: sway and non-sway frames. In BS 5950-1: 2000 (BSI, 2000), a multi-storey frame may be classified as "non-sway" if its sway deformation is small for the resulting secondary forces and moments to be negligible. In Eurocode 3 (DD ENV 1993-1-1: 1992 and BS EN 1993-1-8: 2005), the frame is classified as braced when the bracing system reduces the horizontal displacement by at least 80%. A steel frame which does not satisfy the criterion for a braced frame is classified as unbraced. The picture of both braced and unbraced frames are given in Figure 1.2.

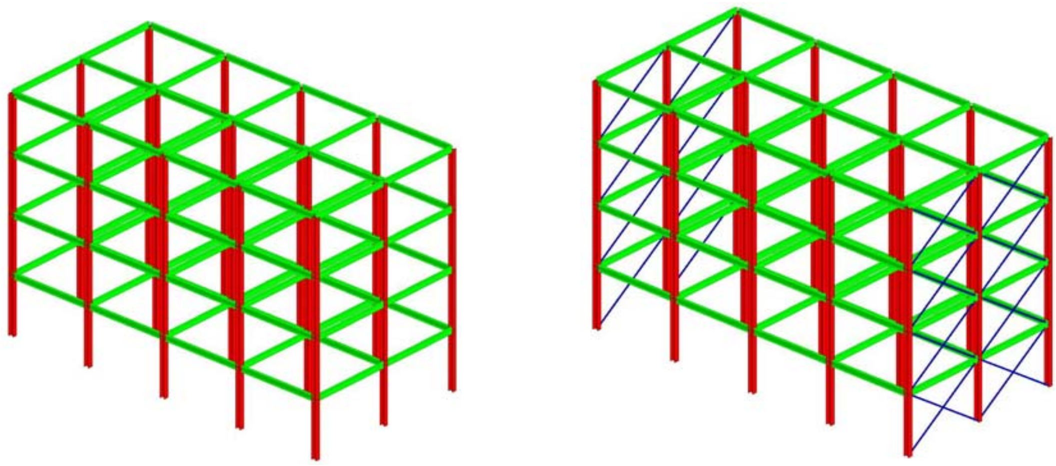


Figure 1.2: Multi-storey Unbraced (left) and braced (right) steel frames

For an unbraced frame, the main consideration is to limit sway, to control the inter-storey drifts and to avoid premature collapse by frame instability. To meet this requirement, it is usual to rely on the bending resistance and stiffness of the connections to resist horizontal loads. For ultimate limit state, it is important to make sure that the structural members are capable of transferring the factored loads to the columns and down to the foundations. In practice, unbraced frame usually designed by assuming that the connections are rigid in order to provide adequate stiffness to resist horizontal loads. In rigid frame analysis and design, the internal moments and forces are distributed among the columns and beams according to their stiffness coefficients (K). The stiffness coefficient is a function of the length (L), the second moment of area (I) and the modulus of elasticity (E).

1.3 Trapezoidal Web Profiled (TWP) Steel Section

A trapezoid web profile plate girder is a built-up section made up of two flanges connected together by a thin corrugated web as shown in Figure 1.1. The web and the flanges comprised of different steel grade depending on design requirements. TWP section is also classified as a prefabricated steel section as the section is comprised of two different types of steel grade. The steel grade of the flanges is designed for S355 and the steel grade of the web is designed for S275. The steel grade of the flanges is purposely designed for S355 so that the flexural capacity of the beam can be increased. The steel grade of the web is designed for S275 so as to reduce the cost of steel material and the capacity of shear is not that critical in the design of the beam. The use of different steel grades in the fabrication of TWP section leads to further economic contribution to steel frames design besides the use of partial strength connection. The use of thick flanges, thin web and deeper beam for TWP section compared with hot-rolled section of the same steel weight leading to heavier load capacity and greater beam span that can be achieved.

The advantages of TWP beam as compared to the conventional plate girder or hot rolled steel section include the following:

- Utilization of very thin web which is light weight and reduce the tonnage of the steel.
- Elimination of the need of stiffeners which reduced the fabrication cost.
- The use of high strength steel S355 for flanges and deep beam which lead to higher flexural capacity, wider span and less deflection.

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

1.4 Standardized Partial Strength Connections

In the design of braced multi-storey steel frames, the steel weight of the connections may account for less than 5% of the frame weight. However, the cost of the fabrication is in the range of 30% to 50% of the total cost. The increase in the fabrication of the connections is due to the difficulty in selecting the type of connection, the grades and sizes of fittings, bolt grades and sizes, weld types and sizes, and the geometrical aspects. Therefore, a standardized partial strength connections tables are introduced to cater for the problems arise due to so many uncertainties in the fabrication of the connections.

1.4.1 Advantages of standardized partial strength connections.

The advantages of the partial strength approach are that it utilizes the moment resistance of connections to reduce beam depth and weight, while avoiding the use of stiffening in the joints. This practice will reduce the cost of fabrication and ease the erection of steel member in the construction of multi-storey steel frames. The potential benefits of using this approach can be listed as follows:

A. Lighter beams

In the design of semi-continuous braced steel frame, the required beam plastic modulus is less than those required in simple frame for the same frame. This reduction is possible as the partial strength connection reduced the design moment of the beam due to the partial restraint effect of the connection as illustrated in Figure 1.3. The design moment which a beam must resist, decreases as the moment capacity of the connection increases. As a result, a lighter beam can be selected for the design of the beam.

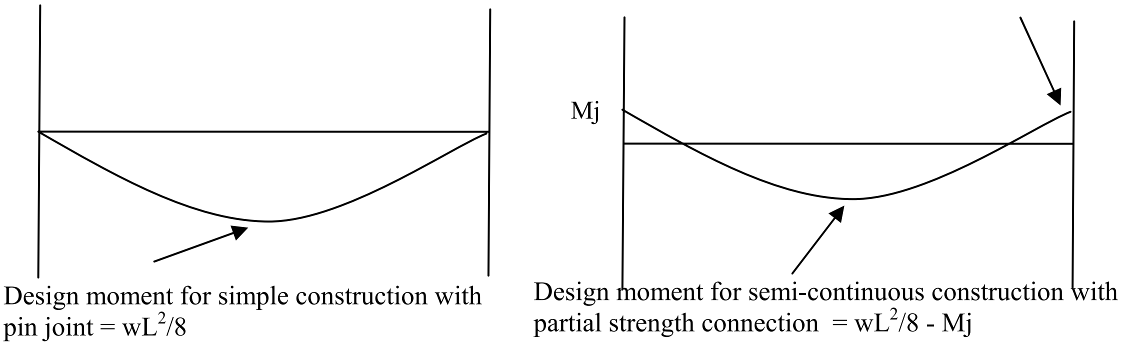


Figure 1.3: Design moment for beams due to different support conditions

B. Shallower beams

The partial restraint of the connection will also result in shallower beams. This is due to the increase in stiffness of the connection, which contributes to the decrease in deflection. The use of partial strength connection will reduce the constant coefficient β in the formulae of deflection ($\beta_w L^4/384EI$) in simple construction with uniform load from β equal to 5 to 2 for internal beam and to 3 for external beam. The partial strength connection acts as restrained to the deformation of the beam due to applied load. As a result, a reduction in the deflection of the beam can be achieved which lead to the shallower beam. The relationship between connection stiffness and deflection coefficient "Beta" for uniform load on beam is shown in Figure 1.4.

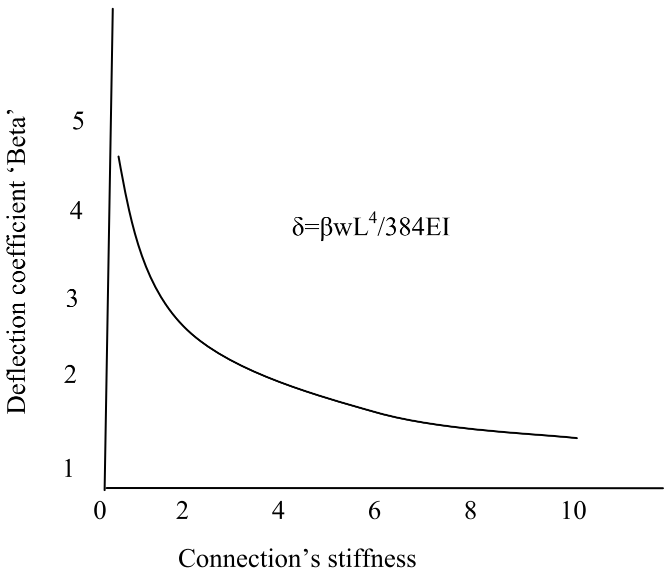


Figure 1.4: Deflection coefficient 'Beta' as a function of relative stiffness of connection

C. Greater stiffness and more robust structure

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

D. Lower overall cost.

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

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

aspects of the connection have already been set. This leads to fast and quality fabrication.

Although the advantages or benefits of using the partial strength connections are quite significant, the disadvantages of this approach should also be addressed. The disadvantage in this approach is that it may be marginally more expensive to fabricate partial-strength connection rather than simple connections. However, the benefit of overall cost saving of the partial strength connections have proven to be more than simple connections (Md Tahir, 1995)(Couchman, 1997). It is reported that the savings in steel weight of using partial strength connection in multi-storey braced steel frames using British hot-rolled section was up to 12%. The overall cost saving was up to 10% of the construction cost which is quite significant (Md Tahir, 1995).

1.4.2 Range of Standard Flush End-plate Connections.

The use of partial strength connection for hot-rolled British sections has well established by SCI & BCSA (1995). A series of tests at the University of Abertay, Dundee has been successfully been carried out to verify the predicted moment and shear capacity with the experimental tests capacities (Bose, 1993). The results confirmed with the predicted values and the standardized tables for the connection have been published by SCI & BCSA (1995). In the development of standard flush end-plate connections tables for TWP sections, only six tables are presented in this study based on the proposed method. Although the best validation of the results presented in the tables is by comparing the predicted results with the actual experimental tests results, however, the presented standard connection tables for TWP section can still be use by adopting the same failure modes of the hot-rolled section as tested by SCI. A few tests have been carried out to support the predicted moment resistance of the connection using TWP section as a beam. Some of the results are presented later in this paper. The proposed standard connections have the following attributes which in some cases the attributes are not exactly the same as the one described by SCI in hot-rolled section.

- 12mm thick end plates in conjunction with the use of M20 bolts.
- 15mm thick end plates in conjunction with the use of M24 bolts.
- Strength of end plates was maintained as S275 steel.
- Width of the end plate was kept at 200mm and 250mm with the vertical height of the end-plate was kept at the beam depth plus 50mm.
- Full strength of flange welds with size of weld proposed at 10mm
- Full strength of web welds with size of weld proposed at 8mm
- The vertical and horizontal distance between the bolts was maintained at 90mm.

Figure 1.5, shows a typical flush end plate connection for TWP section as beam connected to British hot-rolled section as column. British section is selected for the column as it is very good in compression which is not the case for TWP section as the web of TWP is too thin to carry axial load. TWP section is proposed for beam as the corrugated web section is very effective to cater for buckling and bearing resistance. The minimum thickness for corrugated web is 3mm for shallow beam and the maximum thickness is 6mm for deeper beam. The ratio of beam depth versus web thickness is kept not to exceed the limit for compact section as described by BS5950:2000 Part 1.(BSI, 2000)

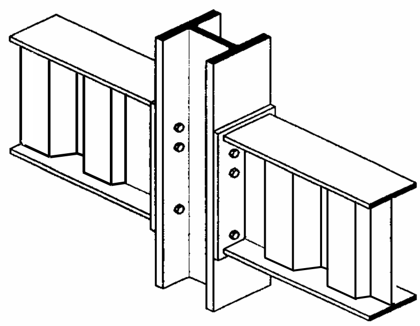


Figure 1.5: Typical flush end-plate connection of TWP beam section connected to British Hot-Rolled section.

1.5 Problem Statement

In some structural design cases, especially of the minor axis of unbraced frame system, or in some architectural requirements, the utilization of rigid connection that assumed as an expensive alternative cannot be avoided. Since this method being known that its price is higher than the simple and semi-rigid connection, there has no further initiative taken to identify the actual cost difference with other common alternatives.

Substantial works have been carried out and results have been published on the matter, but most of the works concentrated on the typical rolled sections. Since TWP sections are proposed, the work on the behaviour of standardised connections has to be carried out. Results obtained from experimental evidences and theoretical have to be obtained before any attempt to incorporate the semi-continuous method to the braced frames design is possible. Capacity tables for standardised bolted beam to column connections: Flush End Plate and Extended End Plate, are to be produced. In addition, the moment and rotation capacity for the above-mentioned connections in terms of Moment-Rotation curves are to be obtained as well.

To date, not much work has been done on the matter utilising TWP sections, therefore its behaviour which is believed to be different from the typical rolled sections needs to be studied extensively. Percentage of saving in materials could then be obtained for the same moment and rotation capacity.

Furthermore, the behaviour of unbraced frames form using the moment connections specified is to be studied. Analysis will be conducted using the Wind Moment Method. Results obtained can then represent the overall performance of the semi-continuous braced frames utilising TWP sections.

The importance of the study is further stressed by the statements made in the introductory remarks from the British Constructional Steelwork Association and Steel Construction Institute publication, *Joints in Simple Construction : Moment Connections*, on page 1 (1995):

> *“ Historically, moment connections have been designed for strength only with little regard to other characteristics i.e stiffness and ductility. There is growing recognition that in certain situations this practice is questionable and so guidance is given to help designers ”*

1.6 Objectives of the Study

The advent of a more economical design and innovative use of steel structures has encouraged thorough studies on the behaviour of connections and frame responses, which leads to better and clear understanding of semi-continuous construction. The study concentrates mainly on experimental investigations of partial strength moment connections. However, analytical solutions based on SCI guidelines and parametric studies on design of braced-frames are also carried out.

In order to solve the problems as mentioned above, the objectives of this study are:

1) To identify economic comparison between semi-rigid and rigid joints for multi-storey unbraced steel frame structures.
2) To develop an optimum design for unbraced steel frame structures by partial and full strength joints
3) To standardize the connections for unbraced steel frames.

1.7 Scope of Work

The research carried out was concentrated mostly on the experimental investigations on moment connections by utilising TWP sections as beams. The type of beam-column connections were studied are Flush End Plate and Extended End Plate. The connections were fabricated as partial strength and were meant to be used in semi-continuous construction of multi-storey braced frames. The works involved in this study can be divided into 4 main parts:

Part 1 covers the general introduction for the subject including basic information about the study to be conducted, main objectives and the scope of the research. All of these are mentioned in Chapter 1. Chapter 2 covers the literature review on the subject which describe the background information about the investigations into the behaviour of moment connections and semi-rigid construction of frames. Previous testing arrangements are also reviewed so as to find a standardised and suitable arrangement for the reliability and accuracy of data. Part 2 covers two chapters: 3 and 4. Chapter 3 described the analytical investigation in producing standardised tables for the capacity of moment connections utilising TWP sections. Design procedures employed are in accordance with the BCSA and SCI guidance. Chapter 4, on the other hand, concerned with the parametric studies of designing multi-storey unbraced frames in semi-rigid constructions. Variation of bays (up to 4 bays) and storeys (up to 8 storeys) were employed according to the minimum weight design or the minimum depth design. Results obtained are expressed in term of percentage of savings in material.

Part 3 of the study contains the details of experimental investigations carried out. Chapter 5 described the experimental works carried out for individual connections. Includes also in this chapter is the works done prior to the actual tests in bringing the testing rig, testing procedures and data recording facilities up to the standard required for the tests plus the testing done on the materials to be tested.

Finally Chapter 6 and Chapter 7 are grouped in Part 4 of which Chapter 6 provides the results obtained and discussions on the results, and Chapter 7 provides the conclusions and recommendations for future works.

CHAPTER 2

REVIEW ON BACKGROUND
OF RELATED WORKS

2.1 General

The development of metal structures began as early as 1770's in England with cast iron used as the material (Salmon, 1980). Cast iron was then replaced by wrought iron after 1840 until 1890 when steel replaced wrought iron as the principal building material. In Malaysia, as can be observed from the number of buildings constructed, reinforced concrete is preferable as the structural material than any other materials such as steel or timber. However, the tendencies of using steel including the composite of steel and concrete have increased thus attract many researches and development works to be carried out within the field. Elsewhere around the world, many major structures have been erected using steel such as the Sears Tower in the United States, Nagoya Dome in Japan and Jin Mao Tower in Shanghai, China, to name a few. Closest to home, the Petronas Twin Tower, which is recently became the second tallest building in the world after losing the title of tallest building to the Taipei 101 in Taiwan, has also been built using steel. (The Petronas Twin Tower houses 88 stories and stands 452 m while the Taipei 101 houses 101 storeys and rises 509 m above the street level (Emporis, 2004)). In lieu of this, it can be said that the development of a country runs parallel with the amount of steel used in construction and can be considered as an indicator to the level of development of that particular country (Hussein, A. F. *et. al.,* 1996). The importance of using steel as the construction material is justified in the seismic regions where it is preferable and believed to be able to sustain earthquake loads. As an example, in the aftermath of the Northridge earthquake in January 1994, no collapses of steel-framed buildings were reported despite the extremely strong ground motions (Chen and Yamaguchi, 1996), even though later, it was found out that more than one hundred moment frame buildings sustained major damages in terms of cracking at the welded connections. Connections and subsequently the structural frame systems, therefore, play a very important role in making sure that structures will be able to sustain the intended loadings.

Connections whether welded connections or bolted connections possess certain degrees of resistance against moments and, stiffness and ductility against rotational. In practice, connections are usually being designed as pinned or rigid. Structural frames, on the other hand, are typically categorised into two systems, which are the Braced Frames and the Unbraced Frames or Moment-Resisting Frames.

To date, emphasis is more on obtaining the balance between the two extremes of connection designs (pinned and rigid) in aspects of serviceability and economy. Connections designed as pinned are much simpler to construct but tends to result in more heavier sections used whereas connections designed as rigid can produce lighter sections but expensive to construct. This situation gives rise to an alternative approach to the design of connections called semi-rigid design.

2.2 Connections

Basically, a beam-to-column connection can be identified by understanding the behavioural characteristics of the particular connection. Conveniently, these behavioural characteristics can be represented by a relationship between the joint moment and the rotation of the connected member. This useful and important relationship can be depicted by a curve called a Moment versus Rotation (M-ϕ) Curve. Figure 2.1 shows a typical moment-rotation curve for a bolted connection suitable for a semi-continuous construction, while Figure 2.2 shows the exaggerated deformation of a joint through an arbitrary moment. Based on the moment-rotation curve, a connection can be classified typically by three characteristics, which are the Moment Resistance (Strength), the Rotational Stiffness (Rigidity), and the Rotational Capacity (Ductility) (SCI and BCSA, 1995).

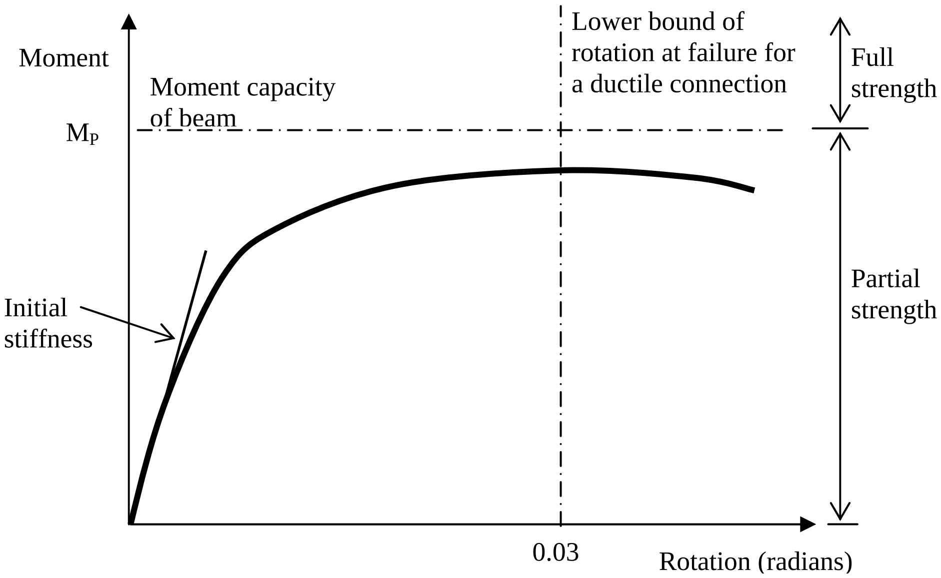


Figure 2.1: Moment-rotation behaviour for a connection suitable for semi-continuous construction *(Adopted from SCI and BCSA, 1995)*

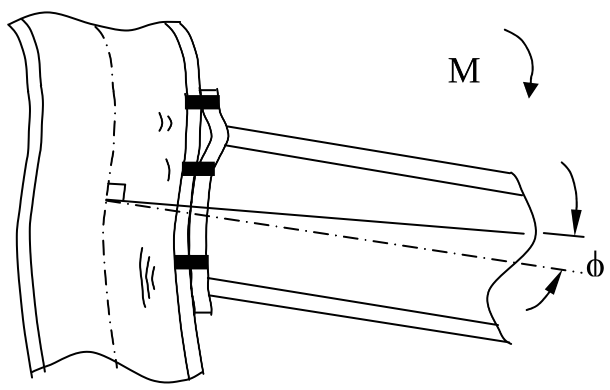
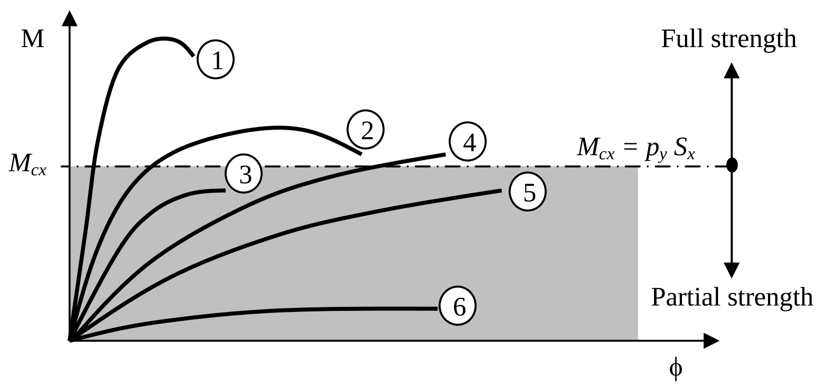


Figure 2.2: Typical deformation and rotation of a semi-rigid joint *(Adopted from SCI and BCSA, 1995)*

With respect to strength, a connection can be further classified as Full Strength, Partial Strength or Nominally Pinned of which when applied to the construction of frames, the system is known as Continuous Construction, Semi-

Continuous Construction and Simple Construction respectively. A full strength connection is defined as a connection with a moment resistance, M_j at least equal to the moment capacity of the connected member (beam), M_{cx} or M_p. A partial strength connection, on the other hand, is defined as a connection with moment resistance less than the moment capacity of the connected member. Whereas, a nominally pinned is defined as a connection, that is sufficiently flexible with a moment resistance not greater that 25% of the moment capacity of the connected member. Figure 2.3(a) shows the classification of a connection by strength based on the moment-rotation curves for six typical types of connections. (Types of connections are discussed in details in Section 2.3). The second behavioural characteristic mentioned above is called rigidity. A connection is termed as rigid when it is stiff enough for the effect of its flexibility on frame bending moment diagram to be neglected and with minimum deformation and rotation. A semi-rigid connection is regarded as a connection that is too flexible to quantify as rigid but is not a pin to be considered as a nominally pinned. Figure 2.3(b) shows the classification of a connection by rigidity. The third behavioural characteristic is the ductility. In this respect, a ductile connection is termed as a connection that has a capacity to rotate sufficiently to form a plastic hinge at some stage of the loading cycle without failure. Figure 2.3(c) shows the classification of connections by ductility.



(a) CLASSIFICATION BY STRENGTH

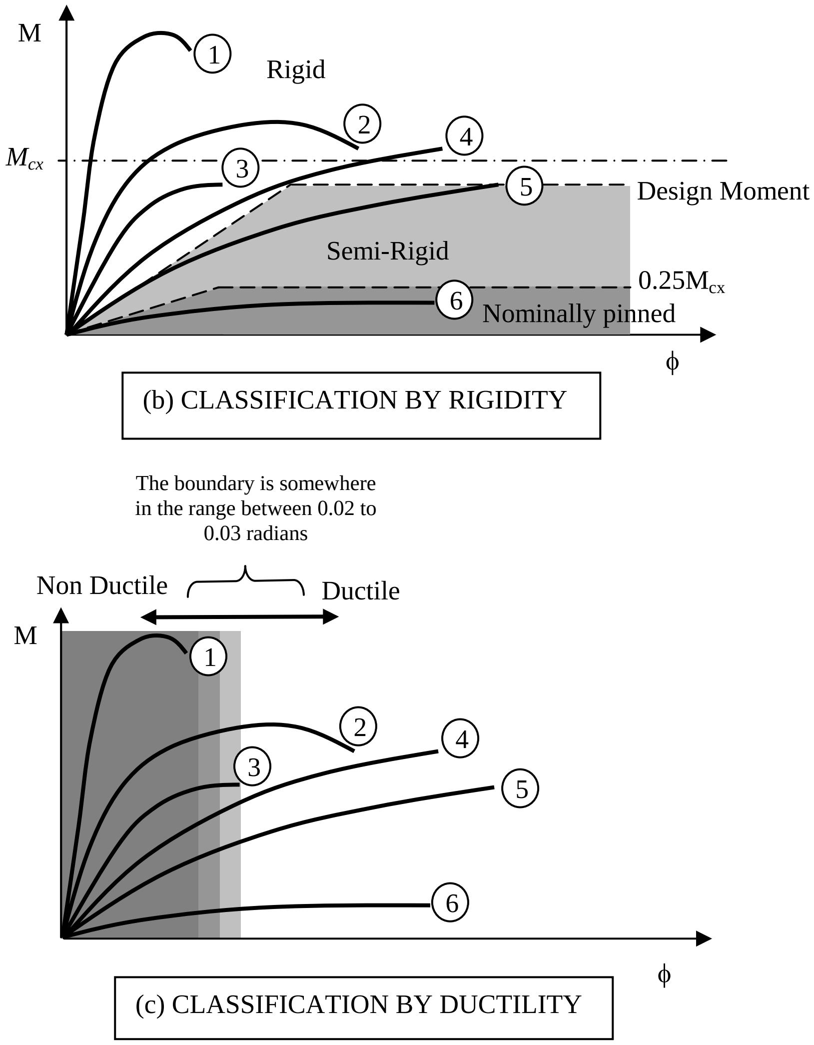


Figure 2.3: Classification of a connection by a) Strength, b) Rigidity, and c) Ductility ***(Adopted from SCI and BCSA, 1995)***

With regard to the design of frames, there are three major design methods depending on the types of connections used and the assumptions of how the connections behave. A rigid design (continuous construction) is a design of frame where connections are considered as fully rigid joints for elastic analysis and full strength joints for plastic analysis. For normal bolted joints, rigidity of that level can be expensive due to the complexity of the connection. A semi-rigid design (semi-continuous construction), on the other hand, is a design of frame where semi-rigid connections are modelled as rotational springs and partial strength connections are modelled as plastic hinges. Here, the moment-rotation characteristics of the connections are determined and required for the analysis. A simplification of this design method called a Wind-Moment method can provide a safe and quick solution for low-rise unbraced frames. The third method, which is the most conservative method of frame design, is the simple design (simple construction) of which the connections are assumed not to develop moments that affects the connected members. In this method, beams are generally designed as pin ended and columns are designed for the axial and moments which results from beam reactions and nominal eccentricities (SCI and BCSA, 1993).

2.3 Types of Connections

As mentioned in the previous section, connections can be classified by the rigidity and associate closely to the frame design methods. Discussed below are the typical types of connections under the categories of simple, rigid and semi-rigid connections.

2.3.1 Simple Connections

Simple connections are the connections that transmit an end shear only resulted from the end reaction of a connecting beam. Also known as flexible connections, they possess sufficiently low stiffness and thus are incapable of transmitting moments at ultimate limit state (SCI and BSA, 1993). In the United Kingdom, the connections are assumed to transmit some nominal moments resulting from the 'eccentricities' of the beam reactions to the columns, but as described in BS 5950-1: 2000, the effects of these moments are somewhat offset by using the column buckling length less than the true length. As a result of assuming pin-ended, the design is a bit conservative thus increasing the beam size obtained. A column, in contrast, since designed only for axial load and 'eccentricity' moments, is obtained a little lighter. Under simple connections, there are three common types of connections that are frequently been used:

a) Web Angle Cleat

This type of connections usually comprises of two angle cleats bolted at the web of a beam though single angle cleat is also used normally for small connections or due to limited access around the connection area. Shown in Figure 2.4 are the typical double web angle cleats for a beam connected to the major and minor axis of a column.

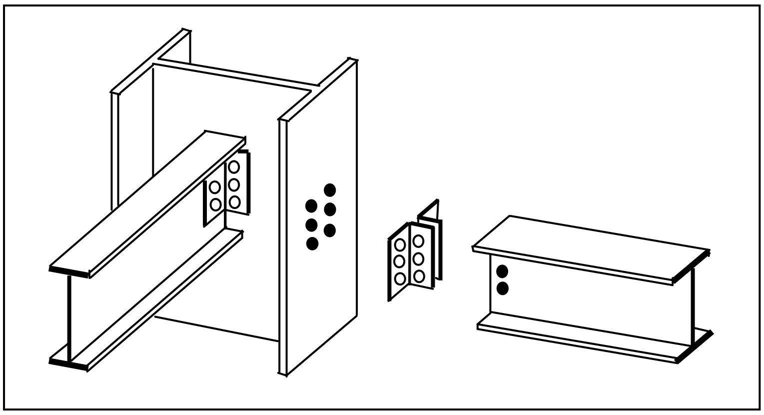


Figure 2.4: Web Angle Cleats

b) Flexible Endplate

Typical flexible endplate connections are shown in Figure 2.5 of which can be attached at both the major and minor axis of a column. For this type of connections, an endplate is fillet welded to the end of a beam and bolted at site to web or flange of a column. Sometimes it is necessary to trim the beam flanges to suit the width of the endplate.

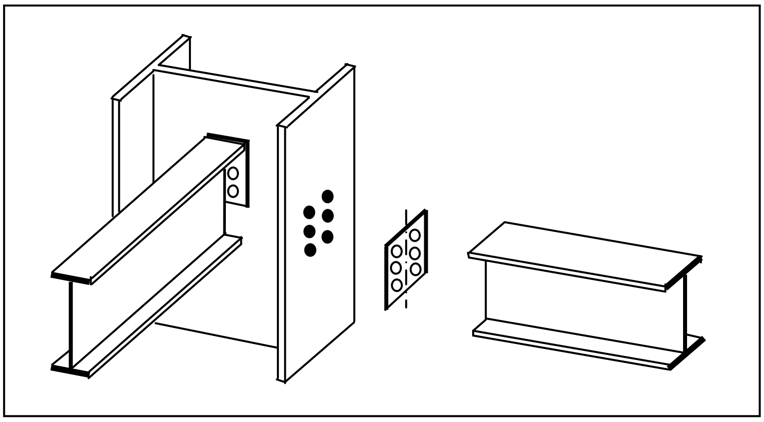


Figure 2.5: Flexible Endplates

c) Fin Plate

As the name implies, a fin plate connection comprises of a thin plate sticking out like a fin from the web or flange of a column depending on whether the intended connection is at the major or minor axis. The fin plate is welded to the column and bolted at site to the beam. They are simple to fabricate and can be considered as the easiest type of connections to fabricate on site. Figure 2.6 shows the typical fin plate connections.

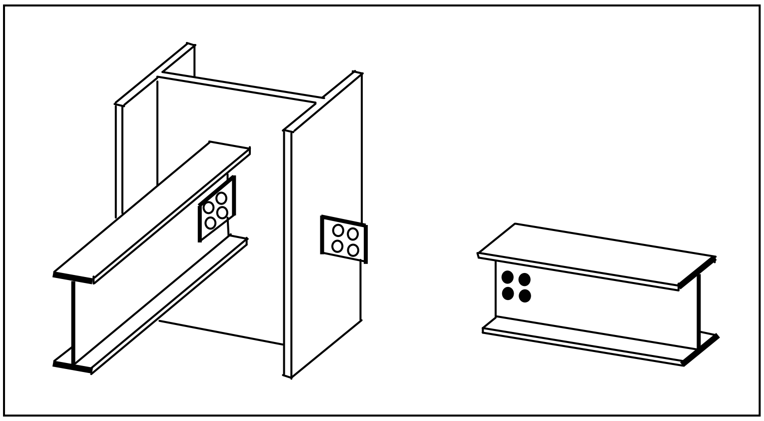


Figure 2.6: Fin Plates

The SCI & BCSA publication titles Joint in Simple Construction Volume 1: Design Methods, 2nd Edition (1993) described in details the checks for the design procedure that are to be carried out for simple connections. These checks, applied to beams connected either to the column flange or web, are listed as follows:

Table 2.1: Design procedure checks recommended for common simple connections

CHECK NO.	DESCRIPTION OF CHECKS		
	Web Angle Cleat	**Flexible End Plate**	**Fin Plate**
1	Recommended detailing requirements	Recommended detailing requirements	Recommended detailing requirements
2	Bolts to beam	Bolt capacity	Bolt capacity
3	Cleat leg adjacent to beam	End plate shear and bearing	Beam weld
4	Beam weld	Beam weld	Fin plate
5	Bolts to column	Beam to plate weld	Additional check for long fin plates
6	Cleat leg adjacent to column	Column flange or web	Fin plate to column weld
7	Column flange or web	*Structural integrity*- end plate tension	Column flange or web
8	*Structural integrity*- cleat tension	*Structural integrity*- beam web tension	*Structural integrity*- fin plate tension
9	*Structural integrity*- beam web tension and bearing	*Structural integrity*- weld tension	*Structural integrity*- beam web tension
10	*Structural integrity*- bolt tension	*Structural integrity*- bolt tension	*Structural integrity*- beam web and fin plate bearing
11			*Structural integrity*- column web in bending

2.3.2 Rigid Connections

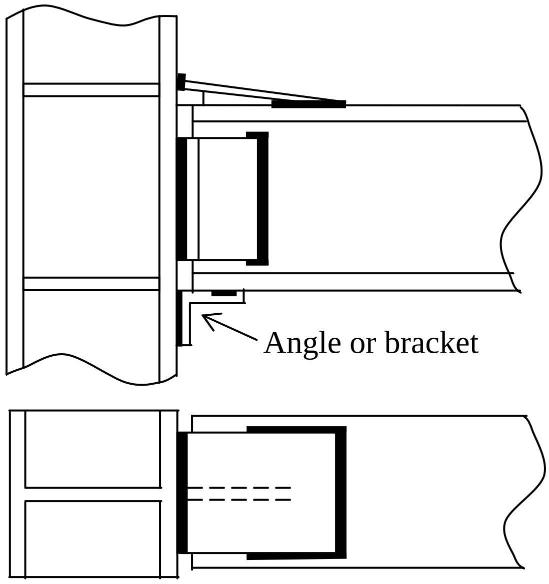
Another category of connections that is commonly used in the design and construction of frames is the rigid connection; and normally, the usage of this category of connection is referred to as a continuous construction. A rigid connection is assumed to be able to transmit fully the end moments from a beam to a column. As the name implies, in theory, there is no relative rotation of members within the joint due to a very high stiffness characteristic possessed. The mid-length moment of the beam is reduced significantly, thus resulting in a lighter and smaller section. However, the column, since designed for both the axial load and the end moments, has resulted in a much heavier section. Usually, the joint is stiffened in order to achieve the fully rigidity state and is accomplished by using either stiffeners, backing plates or haunches in addition to welding. This makes a rigid connection quite expensive to fabricate and construct. Typical examples of rigid connections are in the types of welded attachments and bolted attachments. (Salmon and Johnson, 1980)

a) Welded Attachments

These types of rigid connections use weld to connect the ‘extra’ components to the joint. Since the compression zone at the bottom flange of the beam is substantial, stiffeners for column whether horizontal or vertical are required to prevent sudden column flange buckling. For preventing beam rotation, brackets, plates or haunches are usually welded to the beam. Figure 2.7 shows an example of a welded attachment rigid connection.

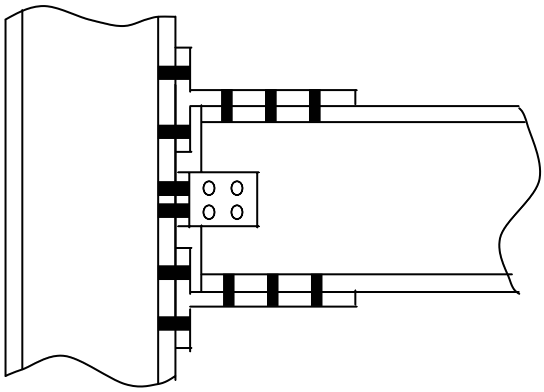
b) Bolted Attachments

To obtain the rigid connections of these forms, ‘extra’ components such as plates are bolted to the members within the joints. Top and bottom plates, top plate and seat angle, split tees, with or without web angles and welded end plates are the typical examples of rigid connections with bolted attachments. An example of a bolted attachment rigid connection is as shown in Figure 2.8.



Seat and top plate

Figure 2.7: Welded attachment rigid connection



Split tees with web angles

Figure 2.8: Bolted attachment rigid connection

2.3.3 Semi-Rigid Connections

Conventionally, in designing steel building's frames, the method that is usually being chosen is based on either the joints are pinned or rigid. Ironically, both design methods do not represent the actual behaviour of the joints where it falls somewhere between these two 'ideal' categories. The behaviour of joints can be clearly seen by referring to the moment versus rotation curve where the *y*-axis represents the fully rigid and the *x*-axis represents the fully pinned condition. Figure 2.1 in Section 2.2 shows that for most commonly used connections, the moment-rotation curve lies between the two conditions. Hence, this type of connections is termed as the semi-rigid connections. EC 3 specifies the semi-rigid connections further by the value of the moment resistance possessed. A full-strength connection refers to a connection that has a moment resistance equal to or greater than the moment capacity of the connecting beam. A partial-strength connection, on the other hand, is a connection that has a moment-resistance less than the moment capacity of the connecting beam. Besides the stiffness and strength, semi-rigid connections are generally very ductile.

With regard to the construction of frames, the type of constructions using semi-rigid connections is generally referred to as the semi-continuous construction by the codes (BS 5950 and EC 3). Listed below are six groups of connections that might be categorised under semi-rigid connections (Chen, 1993).

1. Single web angle
2. Double web angle
3. Header plate
4. Top and seating cleat
5. End plate
6. T-Stub

The typical moment-rotation curves for the six groups of connections mentioned above are as depicted in Figure 2.9. By looking at the curves, it is evident that the Endplate connection definitely can be categorised as the semi-rigid connection. In reference to the vertical axis in this figure as the ‘perfect’ rigid connections and the horizontal axis as the ‘perfect’ pinned connections, a T-stub connection exhibits a rather rigid condition whereas a single web angle exhibits a very flexible condition. Furthermore, it is also noticed that a T-stub connection possessed a high value of stiffness, which corresponds to a high degree of rigidity. On the other hand, Single web angle had a low degree of rigidity due to a low value of stiffness.

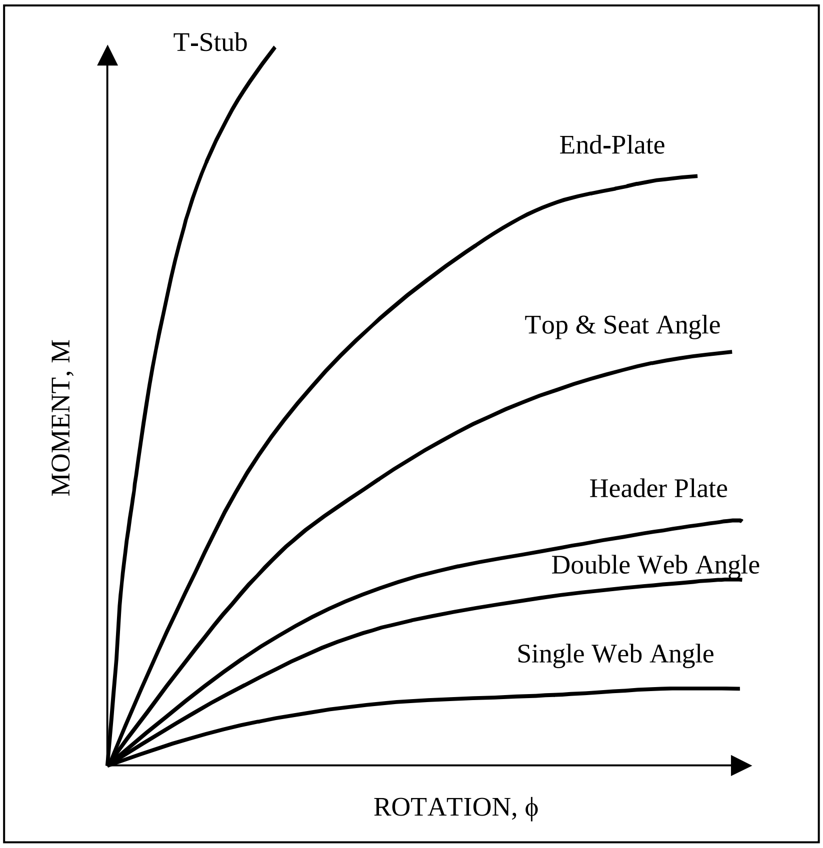


Figure 2.9: Typical moment-rotation curves for connections *(Adopted from Chen et. al., 1996)*

Since only two types of semi-rigid connections are studied in the scope of this research, details about the two connections are given below.

a) Flush Endplate

In general, for a flush endplate connection, a full-depth end plate is welded to the cross-sectional face of a beam and bolted to the column at site. A typical flush endplate connection is shown in Figure 2.10.

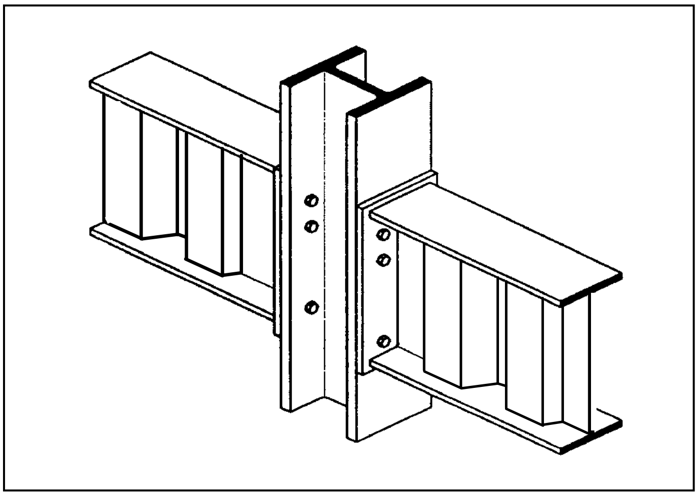


Figure 2.10: Flush end plate connection

b) Extended Endplate

For an extended endplate connection, the endplate is made extended out of the beam flange whether on the tension side only or on both tension and compression sides. In most loading cases, the endplate extended on the tension side only is adequate. Exception to this situation is when there is a reversal of moment such as during the earthquake. Compare to the flush endplate connection, extended endplate connection usually has a higher stiffness and moment resistance. A typical extended endplate connection is shown in Figure 2.11.

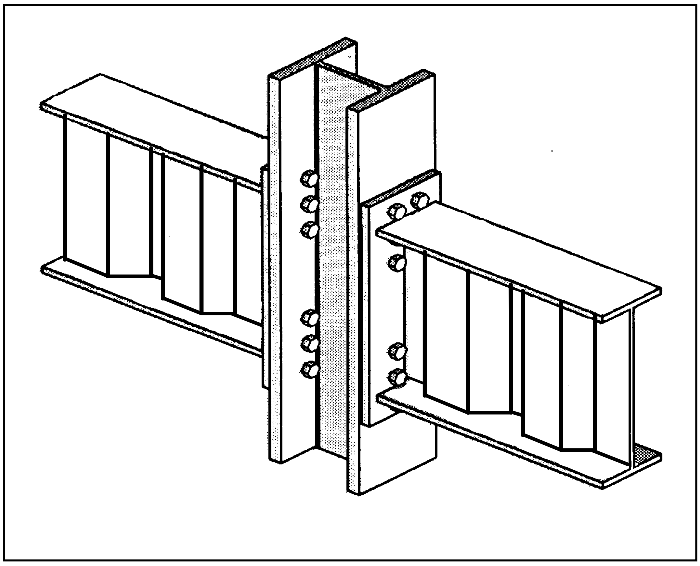
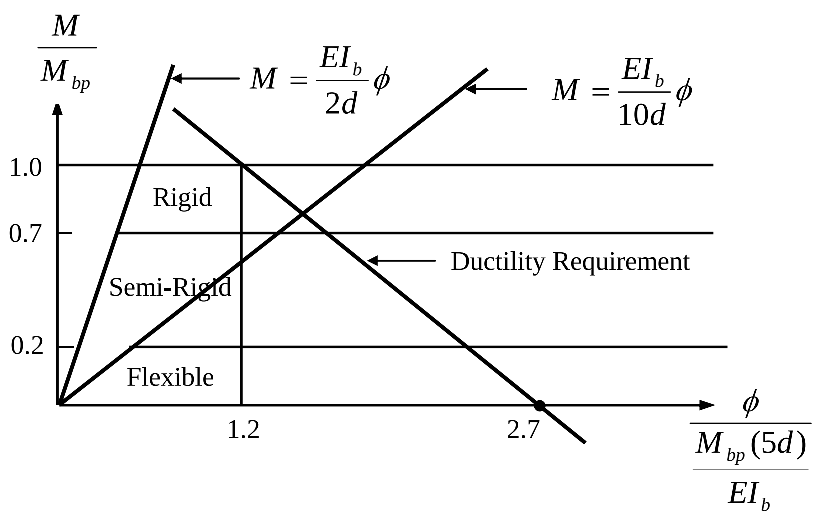


Figure 2.11: Extended end plate connection

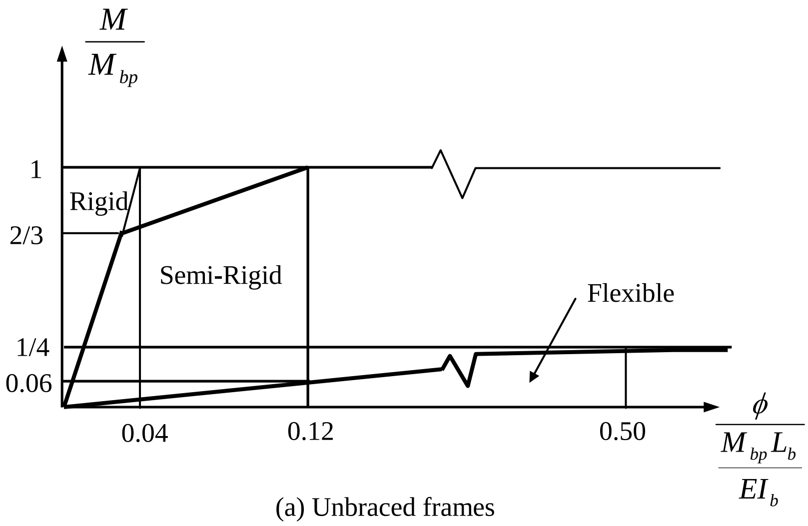
2.4 Classification of Connections

In practice, it is very importance to be able to recognise the category of connections whether it is rigid, pinned (flexible) or semi-rigid. To date, there are two approaches used for classifying the connections. One is the Bjorhovde, Brazetti and Colson (BBC) classification system (Bjorhovde, Brazetti and Colson, 1990) and the other is the EC 3 classification system (Chen *et. al.*, 1996). Figure 2.12 shows the BBC classification system while Figure 2.13 (a) and 2.13(b) show the EC 3 classification system for unbraced frames and braced frames respectively. In using the BBC classification system, prior information on the behaviour of a frame is not needed. EC 3 classification does, however, distinguish between the braced and unbraced frames of which makes the classification system more justifiable.

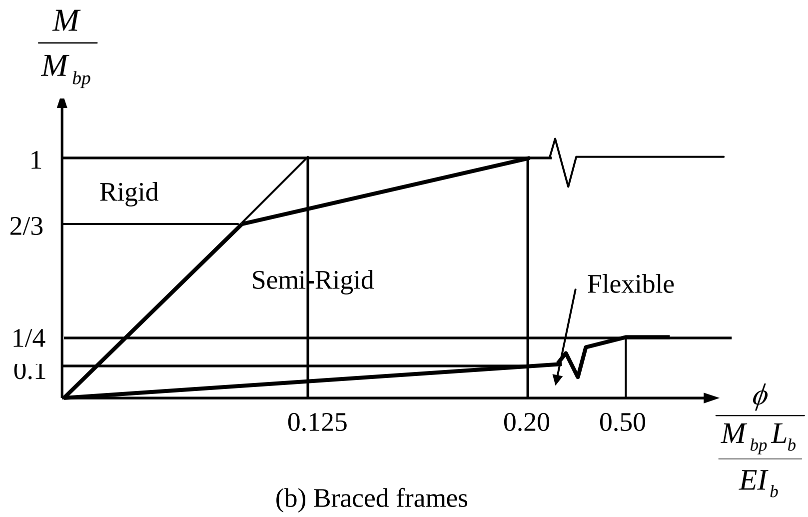


EI_b = bending rigidity of connected beam
d = beam depth
M_{bp} = plastic moment capacity of connected beam

Figure 2.12: Bjorhovde, Brazzetti, Colson (BBC) classification system *(Adopted from Chen et. al., 1996)*



(a) Unbraced frames



(b) Braced frames

Figure 2.13: EC 3 classification system for (a) Unbraced frames, and (b) Braced frames *(Adopted from Chen, et. al., 1996)*

From the moment-rotation curves, the initial stiffness, $S_{j,ini}$, taken as the slope of a secant line drawn from the origin of the curve to an arbitrary rotation, is typically used. The stiffness is then compared to the limiting stiffnesses, S_j that define the rigid, semi-rigid and simple. Table 2.2 shows the limiting stiffnesses for the case of unbraced frames and braced frames derived from the EC 3 classification system. Under the unbraced frames, the limiting stiffness for rigid connections is

$$\frac{M_{bp}}{0.04\left(\dfrac{M_{bp}L_b}{EI_b}\right)} = \frac{25EI_b}{L_b},$$

and the limiting stiffness for simple connections is

$$\frac{0.25M_{bp}}{0.5\left(\frac{M_{bp}L_b}{EI_b}\right)} = \frac{EI_b}{2L_b}.$$

Carrying out the same derivation for braced frames, the limiting stiffnesses for rigid and simple are

$$\frac{M_{bp}}{0.125\left(\frac{M_{bp}L_b}{EI_b}\right)} = \frac{8EI_b}{L_b},$$

and

$$\frac{0.25M_{bp}}{0.5\left(\frac{M_{bp}L_b}{EI_b}\right)} = \frac{EI_b}{2L_b}$$

respectively.

Table 2.2: Limiting stiffnesses according to EC 3

	Rigid	**Simple**
Unbraced Frames	$\frac{25EI_b}{L_b}$	$\frac{EI_b}{2L_b}$
Braced Frames	$\frac{8EI_b}{L_b}$	$\frac{EI_b}{2L_b}$

2.5 Moment versus Rotation Data

In understanding the behaviour of any connection, data on the moment and rotation of the connection has to be collected. Usually, the data is obtained through experimental works. Observation and important values are then determined from a plot of moment versus rotation. Modelling of the connection can be carried out analytically by calibrating to the experimental results.

2.5.1 Moment-Rotation (M-ϕ) Curves

The moment-rotation curve is by far the most important representation of the behavioural characteristics of connection. Moment, M, in this case, is the resultant load acting a connection through the in-plane bending of a beam. Rotation, ϕ, on the other hand, is defined as the relative movement of a beam through an angle in radian with respect to a column. The curve can be obtained by means of experimental (which is more justifiable due to the real interaction among components of a joint) or analytical. Analytically, there are four methods, as described by Nethercot *et.al.* (1989) and Jaspart (2000), that can be used to generate moment-rotation curves. These methods are:

1. Curve fitting
2. Simplified analytical models
3. Mechanical models
4. Finite element Analysis

In general, a moment - rotation curve of a connection carries some traits or characteristics that can be summarised as follows (Aggarwal,*et.al.* (1986), Jones, *et. al.* (1980), and SCI and BSCA (1995)):

a) The stiffness of a connection is indicated by the slope of the M-ϕ curve.

b) In general, the joint behaviour is non-linear of which stiffness decreases as the rotation increases.

c) In theory, the initial stiffness, $S_{j,ini}$ has the same value to the unloaded stiffness, $S_{j,\,unl}$.

d) The strength of a joint is indicated by the value of the moment capacity of which could be taken as the peak value on the moment-rotation curve. However, a method called a 'Knee-joint' is usually employed. (Extensive usage of this method is demonstrated in Chapter 3 and 4).

e) Ductility of a joint is indicated by the rotational capacity that can be achieved by the joint before a significant loss in strength occurs. Ductility increases as the limit of the rotational increases. A connection is generally considered as ductile if the rotation is greater than 0.03 miliradians (SCI and BCSA, 1995).

Chen *et.al.* (1993) had listed out the moment-rotation data that were considered 'useful' to the studies of semi-rigid connections. Since in this study only the flush endplate and extended endplate connections that were considered, listed in Table 2.3 are the available moment versus rotation data for the two connections as reviewed by Chen *et. al* (1993) up until 1992.

Table 2.3: Moment versus rotation data for endplate connections (*Adopted from Chen et. al., 1993*)

Flush End Plate Connection				
Reference (date, country)	**Number of tests**	**Fastening arrangements**	**Number of 'useful' *M-ϕ* curves**	**Comments**
Zoetemeijer and Kolstein (1975, Netherlands)	12	M20 gr. 8.8 bolts	12	
Bose (1981, U.K)	1	M20 gr. 8.8 bolts	1	
Morris and Newsome (1981, U.K)	4	7/8-in preloaded bolts	4	
Zoetemeijer(1981, Netherlands)	23	M24 gr. 8.8 bolts	23	16 curves available in Zoetemejeir and Munter (1984), others supplied privately
Phillips and Packer (1981, Canada)	5	M22 A325 bolts	5	
Jenkins, et. al. (1984, U.K)	3	M20 gr. 8.8 bolts	3	
Zoetemeijer (1984, Netherlands)	6	M20 gr. 8.9 bolts	6	Four tests for joints in frames
Ostrander (1970, Canada)	13		13	
Extended End Plate Connection				
Reference (date, country)	**Number of tests**	**Fastening arrangements**	**Number of 'useful' *M-ϕ* curves**	**Comments**
Sherbourne (1961, U.K)	5	3/4 and 7/8-in. HT bolts	5	
Johnson, et. a. (1960, U.K)	1	3/4-in. HT bolts	1	
Bailey (1970, U.K)	13	1-in HSFG bolts	26	13 pairs of *M-ϕ* curves provided
Surtees and Mann (1970, U.K)	6	3/4, 1 and 1.5-in. HSFG bolts		
Zoetemeijer (1974, Netherlands)	8	M20 and M22gr. 10.9 bolts	4	
Grundy et. al.(1980, Australia)	2	7/8-in HSFG bolts	1	
Packer and Morris (1977, U.K)	3	M16 HSFG bolts	3	
Tarpy and Cardinal (1981, U.S)	16	3/4, 7/8 and 1-in. A325 bolts	2	
Bahia et. al. (1981, U.K)	20	M16 HSFG bolts	20	Provided for one test in Phan and Mansell(1982)
Jenkins et. al. (1984, U.K)	3	M20 gr. 8.8 bolts	3	
Moore and Sims (1986, U.K)	4	M16 gr. 8.8 bolts	2	
Zoetemeijer and Munter (1984, Netherlands)	4	M20 gr. 8.8 bolts	4	

Zoetemeijer (1981, Netherlands)	10	M20 and M24 gr. 8.8 bolts	10	
Zoetemeijer (1981, Netherlands)	5	M20 and M24 gr. 8.8 bolts	5	
Ioannides (1978, U.S)	6	3/4, and 7/8-in. A325 bolts	6	Extended on both edges
Johstone and Walpole (1981, New Zealand)	4	M20 and M24 gr. 8.8 bolts	8	Extended on both edges
Yee (1984, Australia)	16	M23 and M30 torqued bolts	16	Four pairs of M-ϕ provided,cyclic loading, extended on both edges

2.5.2 Modelling of Connections

Since the actual behaviour of semi-rigid connections is more complex than the conventionally assumed rigid and pinned behaviour, the M-ϕ data is often needed to be modelled. This information (M-ϕ data) is essential if analysis on the semi-rigid frames is to be conducted.

There are several models that can be used to generate the M-ϕ curves analytically. These available models are summarised in Table 2.4 (Abdalla and Chen, 1995).

Table 2.4: Models for generating *M*-ϕ curves analytically

Model	Proposed by	Year	Comments
1. Linear			
• Linear	• Bartho • Rathbun • Baker	1931, 1934, 1936 1936 1934	
• Bi-Linear	• Melchers & Kaur • Romstad & Subramaniam • Lui & Chen	1982 1970 1983	
• Piecewise Linear	• Razzaq	1983	
2. Polynomial	• Frye & Morris	1975	$\boxed{\theta_r = C_1(KM)^1 + C_2(KM)^3 + C_3(KM)^5}$
3. Cubic B-Spline		1972, 1981, 1982	
4. Power	• Batho & Lash • Krishnamurthy • Colson & Louveau • Golberg & Richard • Richard & Abbott	1936 1979 1983 1963 1975 1987	$\boxed{\theta_r = aM^b;\ a > 0, b > 1}$ $\boxed{\theta_r = \frac{\lvert M \rvert}{K_i} \frac{1}{1 - \lvert M / M_u \rvert^n}}$

	• Kishi & Chen • Ang & Morris	1984	$\boxed{\theta_r = \dfrac{M}{K_i\left[1-(M/M_u)^n\right]^{1/n}}}$ $\boxed{\dfrac{\theta}{(\theta_r)_0} = \left\|\dfrac{KM}{(KM)_0}\right\|\left[1+\left(\left\|\dfrac{KM}{(KM)_0}\right\|\right)^{n-1}\right]}$
5. Exponential	• Chen & Lui • Kishi & Chen • Yee & Melchers	1986 1986 1986	$\boxed{M = \sum_{j=1}^{m} C_j\left[1-\exp\left(-\dfrac{\|\theta_r\|}{2ja}\right)\right] + M_0 + R_{kf}\|\theta_r\|}$ $\boxed{M = M_0 + \sum_{j=1}^{m} C_j\left[1-\exp\left(-\dfrac{\|\theta_r\|}{2ja}\right)\right] + M_0 + \sum_{k=1}^{n} D_k(\theta_r-\theta_k)H[\theta_r-\theta_k]}$ $H[\theta]=1 \quad \theta\ge 0$ $H[\theta]=0 \quad \theta<0$ $\boxed{M = M_p\left\|1-\exp\left\|-\dfrac{(K_i-K_p+C\theta)}{M_p}\right\|\right\| + K_p\theta}$

2.6 Multi-Storey Frames Incorporating Semi-Rigid Connections

Steel building frames are typically divided into two types depending upon whether or not there is a bracing system on the frames to prevent the lateral movement. The first type is referred to as the braced frame. In this type of frame, beams are assumed to resist only gravity loads while the lateral loads are resisted by the bracings (usually placed diagonally). An unbraced frame, on the other hand, does not require any bracings instead lateral loads are resisted by the moment capacity of the beam to column connections. Unbraced frames with these moment capacities are sometimes referred to as moment resisting frames. According to EC 3 (1992), the frame is braced if the bracing system able to reduce the lateral displacement by at least 80 %. However, sway can still occur in a frame whether braced or unbraced even though lateral loads are resisted.

2.7 Experimental Study

Basically, the researches carried out on semi-rigid joints can be divided into two interrelated subjects. The first is the work conducted on isolated connections. Here, most of the experimental works concentrated on the moment-rotation behaviour of individual joints using any of the ‘standard’ arrangements as follows:

a) Cantilever – moment and shear
b) Cantilever – with variable moment / shear ratio
c) Cantilever – with axial column load
d) Cruciform – pure moment (moment and shear also possible)
e) Cruciform – with axial column load

Another subject that is studied within the periscope of semi-rigid joint is the analysis and design of frames. This includes the responses of the frames towards the use of semi-rigid connections.

2.7.1 Full-Scale Isolated Joints

Wilson and Moore conducted the first experiment on the subject of connections in 1917 in the United States. The work carried out was on the flexibility and rigidity of riveted structural connections. Before 1950, connections were mostly riveted, but after 1950, high strength bolts were used instead. In 1958, Bell, Chesson and Munse perfomed some static tests on riveted and bolted connections of which can be considered as the first experiment conducted on bolted connections. According to Abdalla and Chen (1995), Nethercot had published in 1985 databases after examined and evaluated more than 800 individual tests from open literature. Since then (1985), several researchers such as Yee and Melchers (1986), and Kishi and Chen (1989) have added a number of tests in creating a computerized databank system. In 1995, Abdalla and Chen expanded the database by adding another 46 experimental test data.

2.7.2 Analysis and Design of Semi-Rigid Frames

In the analysis of steel frames, the behaviour of connection plays a major and very important role. Semi-rigid connections experience some rotational deformation caused by the in-plane bending moment of a beam. Hence, the stability of the frame will be affected by the deformation in terms of reducing the connections stiffness and, thus, creating additional drift to the frame. The P-Δ effect experienced by the frame will be intensified by this additional drift of which will affect its overall stability.

The subject of analysis and design of semi-rigid steel frames has received a lot of attentions by many researchers in the past. The areas of interest within the subject include in-plane monotonic loading, in-plane cyclic loading, and in-plane dynamic and earthquake loading. Rathburn in 1936 has modified the slope deflection and moment distribution method to include the effect of semi-rigid connections in the analysis of frames. Other researchers who contributed significantly to the development of the analysis including:

i) Monforton and Wu (1963) – introduced the semi rigid concept into the analysis by using the matrix stiffness method.
ii) Lionberger and Weaver (1969) – investigated the dynamic response of 2-D frames with non-rigid connections using a bi-linear moment-rotation representation.
iii) Moncarz and Gestle (1981) – proposed a non-linear analysis for flexibly connected frames.
iv) Stelmack (1986) – predicted frame behaviour by using the non-linear analysis for flexibly connected frames.
v) S. Mohammad (2000) – proposed a non-linear finite element analysis incorporating geometric, material and connection non-linearities for flexibly connected frames subjected to variable loadings.

A general picture of researches conducted on the subject of analysis and design of semi-rigid frames is best represented by a 'mind map' of S. Mohammad (2000), as illustrated in Figure 2.14.

2.7.2.1 Types of Analysis

The development of methods used for analysing frames has dated back since 1930's with the work of Baker. However, only after 1956 (since the publication of a paper by Livesley), the development has been closely related to the advances in computer capabilities. In general, there are two most important aspects within the numerous analysis methods available. These aspects are (Nethercot, 2000):

i) Elastic or inelastic
ii) Geometrically linear or non-linear

As a comparison, Figure 2.15 illustrated the types of analytical models that can be used for analysing frames. The behaviour of the frame is examined by observing the load versus deflection curve for each analytical model.

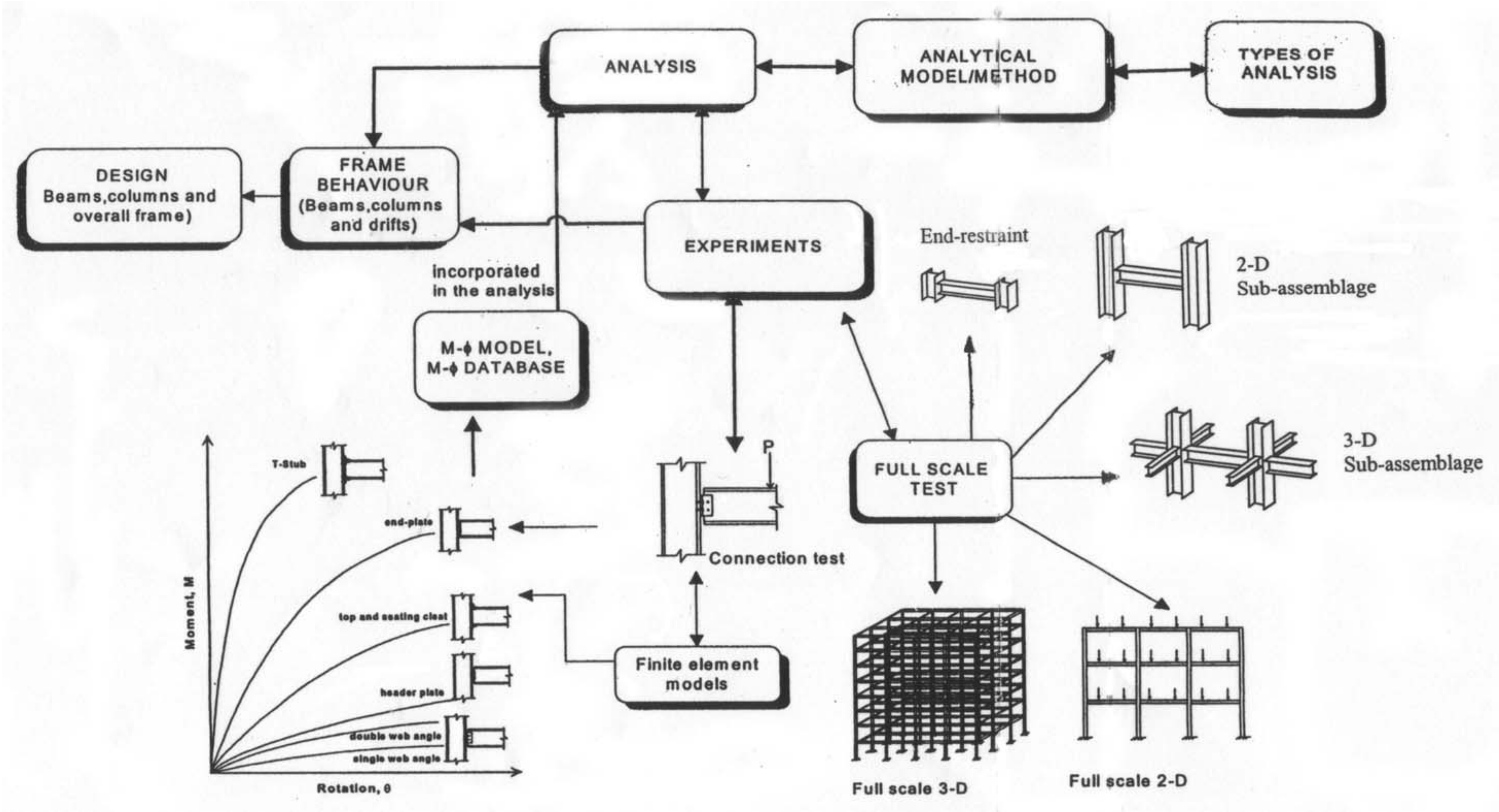


Figure 2.14: General picture on the scope of analysis and design of semi-rigid frames

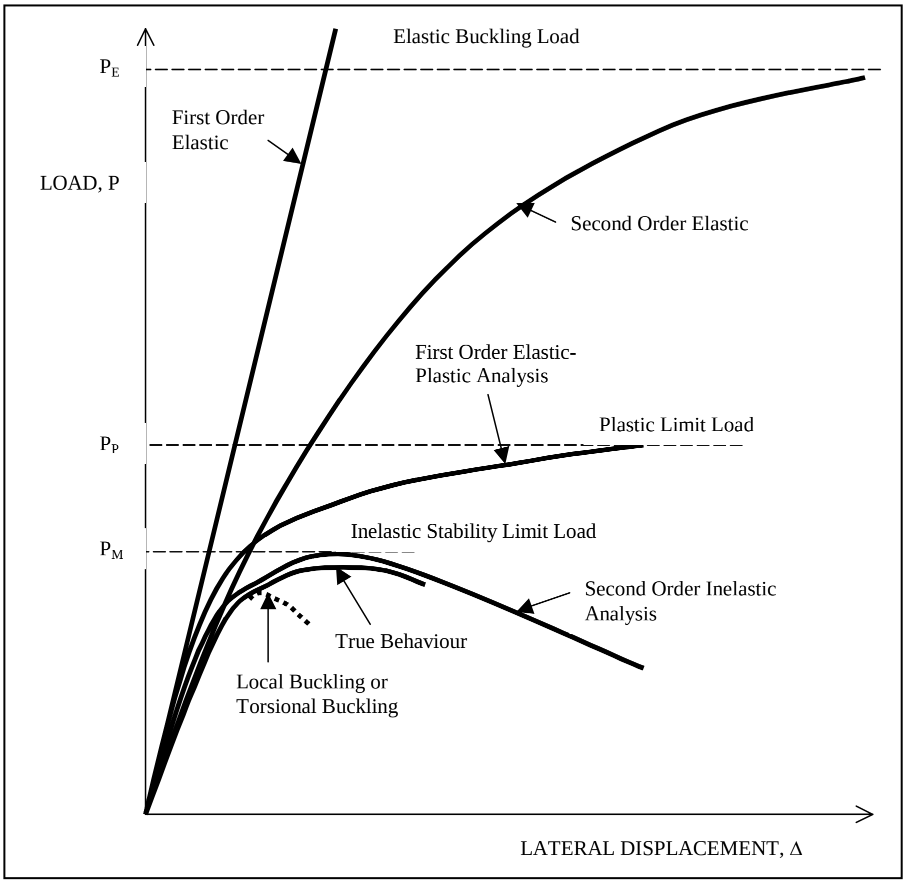


Figure 2.15: Comparison of analytical models

2.8 Endplate Connections Details

Typically, endplate connections are the most suitable for use in semi-continuous braced frames due to the characteristics that they possessed. Annex J in EC 3 (1992) in conjunction with Section 6.9 describes the types of connections, design methods and procedures to be adopted for beam-to-column connections. A traditional triangular distribution was modified to acquire a more accurate representation of distribution of bolt forces using a plastic distribution approach. Specifically, the model followed and used by EC 3 is called the Component method of which a particular connection is divided into three critical areas or zones. These zones , which were described in details in Chapter 5, are as follows (SCI and BCSA, 1995):

1. Tension zone.

   In this tension zone, several critical areas are taken into account in determining the resistance. These areas are column web in tension, column flange in bending, bolts in tension, end plate in bending and beam web in tension.

2. Compression zone.

   The compression zone of which is located around the bottom flange of a beam comprises of several critical areas namely column web crushing, column web buckling and beam flange in compression.

3. Shear zone.

   The shear zone, on the other hand only comprises of column web panel in shear.

The resistances of these three zones would determine the moment resistance of a beam-to-column connection. For column flange or endplate bending, the approach taken is representing the yield line patterns that occur around the bolts by using equivalent T-stubs approach. This approach results in checking against three modes of failure as follows:

a) Mode 1: Complete flange yielding
b) Mode 2: Bolt failure with flange yielding
c) Mode 3: Bolt failure.

Details explanation on each zone and the three modes of failure mentioned above is covered in Section 5.4 of Chapter 5.

2.8.1 Moment Resistance

EC3 (2005) outlines the procedures for determining the moment resistance, M_{Rd}, of a beam-to-column connection in Procedure J.3.1. In the procedures, resistances of all the components in the critical zones are determined of which the resistance of the weakest zone is used after series of iteration. EC3 (2005) specifies that the moment resistance, M_{Rd}, can be obtained using:

$$M_{Rd} = \sum \left[F_{ti.Rd} \; h_i \right] \qquad \ldots(2.1)$$

where

$F_{ti.Rd}$ is the design value of the effective resistance of an individual row of bolts.

h_i is the distance from that bolt row to the centre of resistance of the compression zone.

As mentioned in previous section, the tension resistance of a column flange and an end plate is modelled as an equivalent T-stubs of which, according to EC 3, may be governed by the resistance of the flange, bolts, web and web-to-flange welds (in the case of welded T-stub). The design tension resistance of T-stub flange, then, should be taken as the smallest value from the three modes of failure as follows:

Mode 1: Complete yielding of flange

$$F_{t.Rd} = \frac{4M_{pl}}{m} \qquad \ldots(2.2)$$

Mode 2: Bolt failure with yielding of flange

$$F_{t.Rd} = \frac{2M_{pl} + n\sum B_t}{m+n} \qquad \ldots(2.3)$$

Mode 3: Bolt failure only

$$F_{t.Rd} = \sum B_t \qquad \ldots(2.4)$$

where

$M_{pl} = 0.25 L_{eff}^{2} f_u$

B_t is the design tension resistance of a single bolt-plate assembly

n = e_{min} but $n \leq 1.25m$

m and e_{min} are as indicated in Figure J.3.1 (EC3, 1992).

L_{eff} is

f_u is

As in the case of connections with more than one row of bolts (normally done in practice), the tension resistance of each row of bolts and the combination of rows of bolts have to be checked for the smallest value.

2.8.2 Stiffness

Generally, the moment-rotation curve of a semi-rigid connection exhibits a non-linear characteristic. According to EC 3(2005), the rotational stiffness, S_j, shall be taken as the secant stiffness which is the slope of a straight line up to the M_{Sd}($<M_{Rd}$) in the case of non-linear or tri-linear characteristic or M_{Rd} in the case of bi-linear characteristic. The initial rotational stiffness, if based on the classification boundaries of EC 3, is taken as the slope of an elastic limit up to a value of 2/3 of the design momen resistance, M_{Rd}. After that the rotational stiffness is decreasing as the slope decreasing until it reaches M_{Rd} where it is assumed no to have any stiffness (constant plateau). In Annex J (J.3.7), the rotational stiffness may be approximated by:

$$S_j = \frac{Eh^2 t_{wc}}{\sum \frac{\mu_i}{k_i}\left[\frac{F_i}{F_{i,Rd}}\right]^2} \qquad \ldots(2.5)$$

where

h	lever arm moment
μ_i	modification factor
k_i	stiffness factor for component *I*
F_i	force in component *i* of the connection due to the moment *M*
$F_{i,Rd}$	design resistance of component *I*
t_{wc}	thickness of the web of column

2.9 Trapezoid Web Profiled

The interest in steel section or profile with corrugated webs has dated back quite some times ago with one of the earliest experiment was conducted by A. F Fraser at Langley Aeronautical Laboratory, Virginia, United States in 1956. The investigation was carried out to determine the strength of multiweb beams with corrugated webs. Isolated researches have been conducted since then but the development in this type of profile is noticeable some 20 years ago. The main factor lies in the inability to fabricate corrugated web profiles until recently with the advent in the welding technology that the uses of relatively thin corrugated webs have been possible.

The profile is generally a 'built-up' steel plate girder of which the web could be made corrugated into several shapes such as sinusoidal or trapezoidal. The profile that is going to be studied upon is called the Trapezoid Web Profile (TWP) which is first introduced in Malaysia in late 1997 by Spelten Consulting GmbH, Germany and manufactured commercially by Trapezoid Web Profile Sdn Bhd. TWP can offer several advantages compared to the conventional plate girder or hot rolled section. Among those are (TWP Sdn Bhd, 1997, and Osman, 2001):

a) The TWP sections could be fabricated with a very thin web usually in the range of 2 mm to 8 mm of which would result in lightweight sections.

b) The trapezoidal corrugated web of a TWP section provides extra stiffening to the section in bending, thus eliminate the need of stiffeners as in plate girders. As a result, lightweight sections could be produced and fabrication cost could be reduced.

c) The nature of the trapezoidal corrugated web of a TWP permits the use of a much slender / deep section. With this, a higher flexural capacity could be achieved, and a wider span and a less deflection member could be utilised.

d) The fatigue strength of a TWP section could also be increased.

e) The lateral torsional buckling resistance of a TWP section could also be increased.

It is claimed by TWP Sdn. Bhd. (1997) that due to the nature of its configuration, TWP sections can offer substantial saving in the steel usage, and in some cases of up to 40 % as compared to conventional rolled sections. TWP sections can be used for any structural elements as well, for instance portal frames, floor and roof beams, girders for crane rails and bridges, and domes. Being a built-up section, the size and the grade to be used for the web and flange element of a TWP is determined by the structural and/or architectural necessity. On the other hand, the flanges can also be formed by any of the typical sections as shown in Figure 2.16. In term of flexibility, TWP could be made tapered, cambered or curved according to the structural and aesthetical aspects in the construction. Figure 2.17 shows several examples of flexibilities that TWP can offer. Details geometrical properties of a typical TWP section, and also the section to be used in this study, are as shown in Figure 2.18.

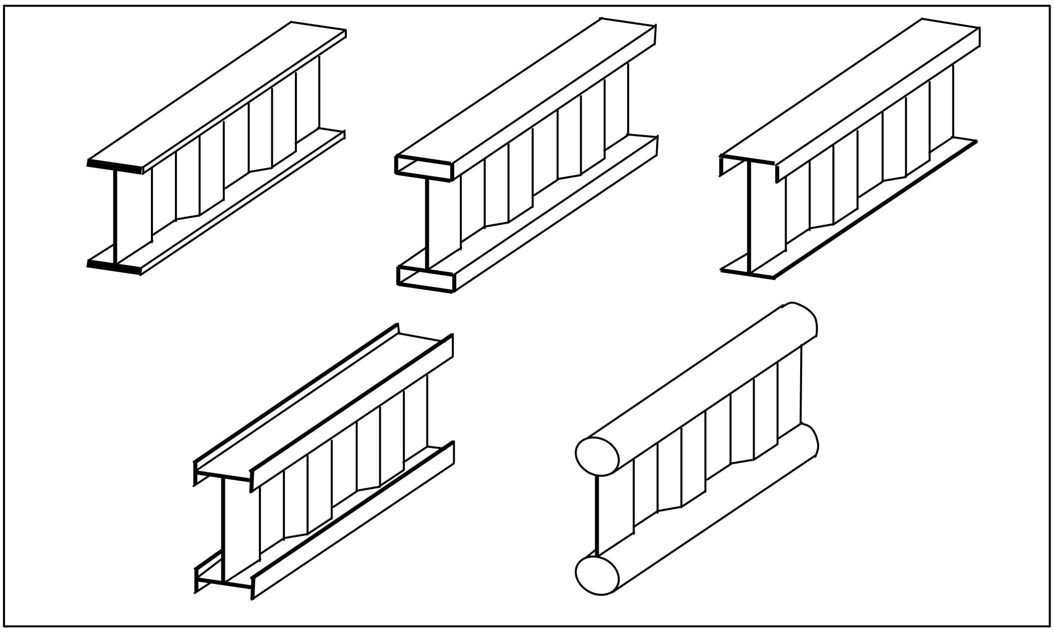


Figure 2.16: Sections with corrugated web and various types of flanges

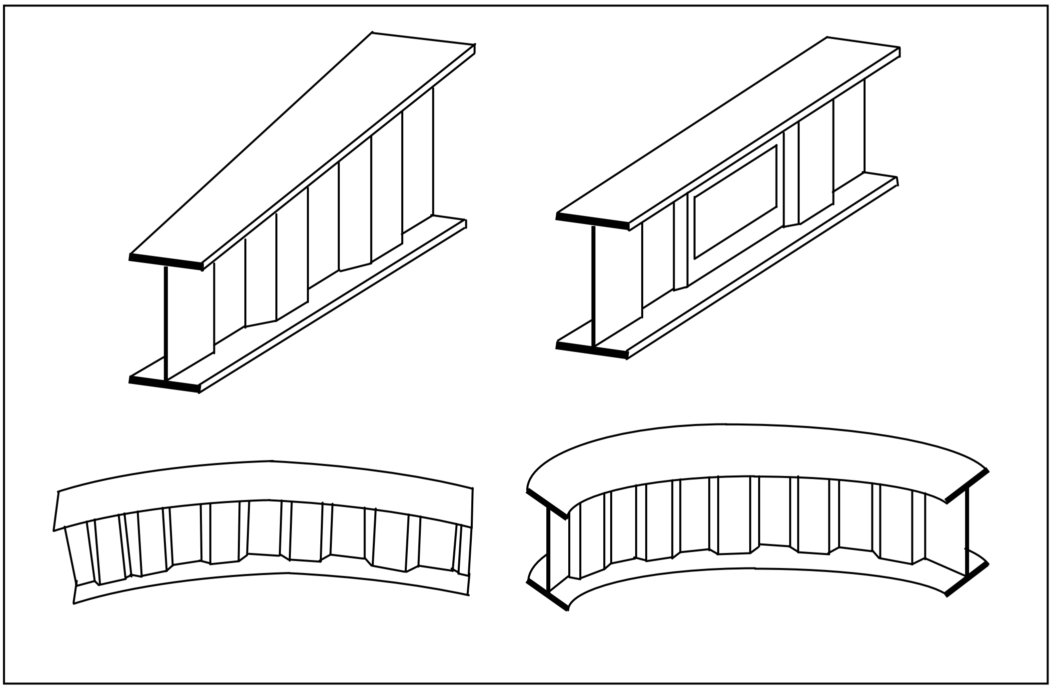


Figure 2.17: Flexibility of shapes of TWP sections

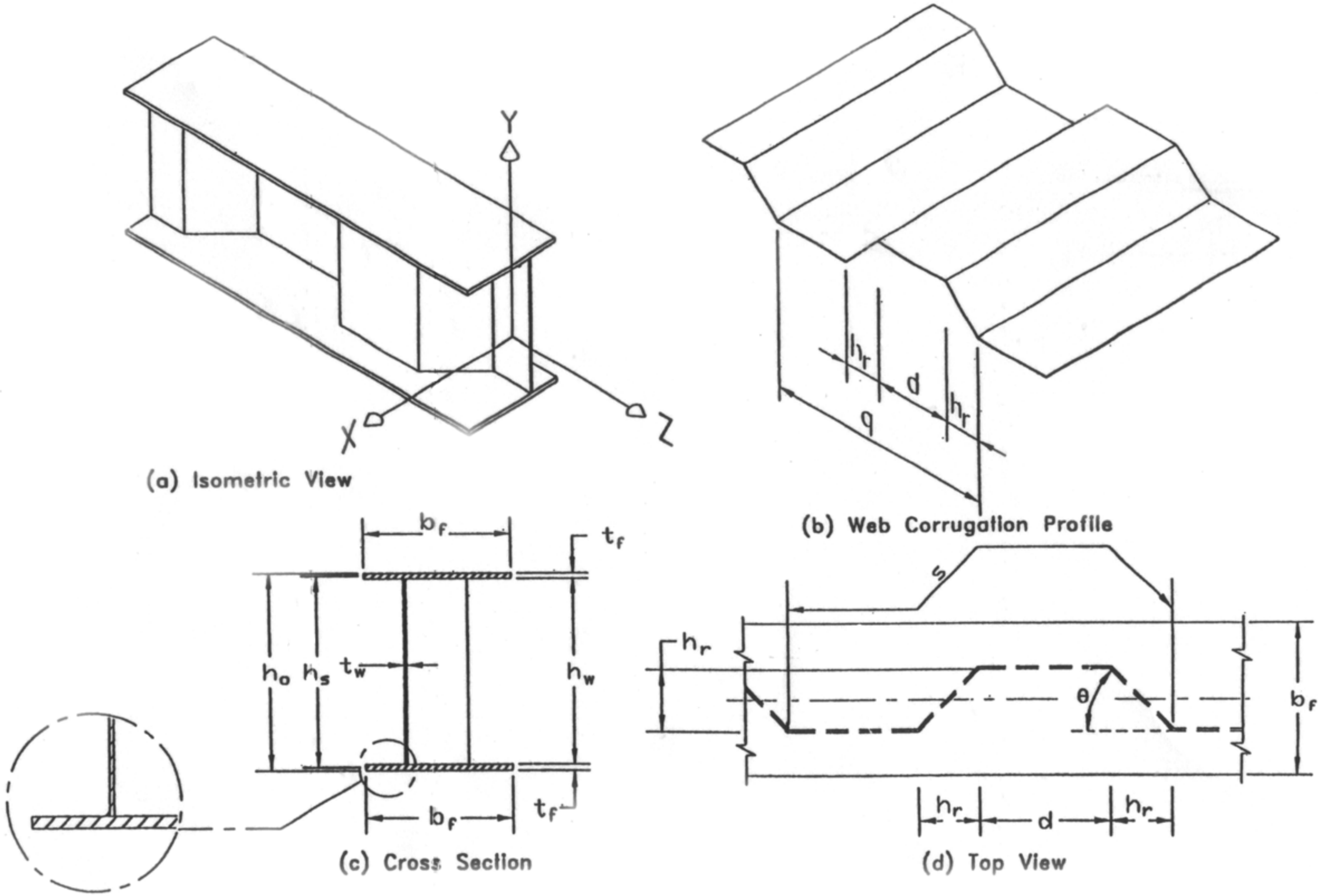


Figure 2.18: Geometrical properties of a typical TWP section

2.9.1 Past Researches

Studies on the behaviour and performance of TWP sections are relatively low since the profile itself is only manufactured commercially and popularised some 20 years ago by a few companies but the trend of using this type of profile for structural usages has gained ever since. Spelten Consulting of Netatal of Germany, TSP Corporation of Florida, Borga Corporation of California and TWP Sdn Bhd of Malaysia to name a few companies that manufacture and fabricate beams with corrugated webs.

One of the earliest experiments on corrugated web profiles was carried out in 1956 by Fraser who investigated the strength of multiweb beams. Harrison (UK) in 1965 published a paper on the fatique behaviour of beams with corrugated webs. When comparing to the conventionally stiffened webs, it was found out that the amount of steel used was about the same due to the ability to weld the web and the flange at that time. However, it was noticed that the fabrication time was significantly improved and the work was much more prone to automation. In 1969, Easley and McFarland made several experimental investigations on shear diaphragms of panel with trapezoidal corrugated webs. Sherman and Fisher of Germany in early 1970's published a work that showed that welding of only the flat portions of the web, only on one side, produced virtually the same results as welding on all parts of the corrugation. In Sweden, the studies on buckling behaviour of corrugated webs were carried out at Chalmers University of Technology by Leiva (1983), Bergfelt and Leiva (1984), Simula and Jonsson (1984), Dahlen and Krona (1984), Bergfelt, Edlund and Leiva (1985), and Leiva (1987). The studies were then extended by Luo(1991), and Luo and Edlund (1990, 1991, 1992 and 1995) by adding the simulation and analysis using the finite strip method.

Beams and girders with corrugated webs have also being investigated at the University of Maine, USA by Hamilton in 1993. Elgaaly, Hamilton and Sesdhari then published the investigation on the shear strength of corrugated beams in 1996. Subsequently, Elgaaly, Sesdhari and Hamilton also published the investigation on the bending strength of corrugated beams in 1997. Johnson and Cafolla studied local flange buckling in plate girders with corrugated webs in 1997. Recently, Wang (2003) has carried out a study on the behaviour of steel members with corrugated webs and tubular flanges subjected to shear, bending and axial compression. In Malaysia, researches on the matter (trapezoid corrugated webs) were mostly carried out by Osman *et. al.* since 1998 but concentrated only on the individual capacities. Among those are shear, bending, local flange buckling, lateral beam buckling, stability of column, foundation pile capacity, fatigue, and composite floor system.

2.10 Concluding Remarks

The application of semi-rigid concepts to the construction of steel frames is already being accepted and proved to be capable of providing a safe and economical design. Its applicability with the conventional rolled sections has reached a stage where design guides and advanced methods of analysis have been produced. However, the need to search for a more innovative and economical construction (but not compromising on the strength and capacity) has lead to the use of a different type of section. Steel profiles with corrugated webs are believed and proved to be able to provide the strength and capacity needed but at the same time reducing the amount of material used due to its thin webs.

This study, therefore, intends to look into the behaviour of beam-to-column connections that are partial strength and semi-rigid. The joints are formed using Flush Endplate and Extended Endplate connections connected to several sizes of TWP beams and one size of conventional hot-rolled column. The results obtained focus on the behaviour of partial strength connections with TWP beams in the semi-continuous construction. Furthermore, parametric studies on the economic aspect of designing multi-storey braced frames with various bays and storeys were conducted for semi-continuous construction using Universal beams and TWP beams as well.

CHAPTER 3

STANDARDISED PARTIAL STRENGTH CONNECTION CAPACITY TABLES

3.1 General

Unlike simple and rigid connections, the design of partial strength connections involves more complex and rigorous procedures. Therefore, the SCI in association with the BCSA of United Kingdom had published in 1995 a reference guide (referred to as SCI's guide herein) in designing moment connections, which includes sections on standardised capacity tables for bolted end plate connections. The design model presented in the SCI's guide is in accordance to the procedures outlined in Annex J of EC 3 (DD ENV 1993-1-1: 1992 and BS EN 1993-1-8: 2005), which is based on the plastic distribution of bolt forces. Traditionally, the bolt forces are taken as a triangular distribution but plastic distribution is considered more accurate and realistic in representing the actual behaviour of bolt forces, as shown in Figure 3.1 (SCI and BSCA, 1995).

In the SCI's guide, the beam-to-column arrangements constitute of conventional hot rolled sections for both the beams and columns. In this study, however, Trapezoid Web Profiled (TWP) sections were to be used as the beam elements. Therefore, the connection capacity tables provided in the guide could not be used directly. New connection capacity tables have to be produced instead. Basically, the procedures outline in the EC 3 and the SCI's guide as well can still be followed. Some modification, however, are needed if there exist some significant contribution from the corrugated and thinner web nature of the TWP.

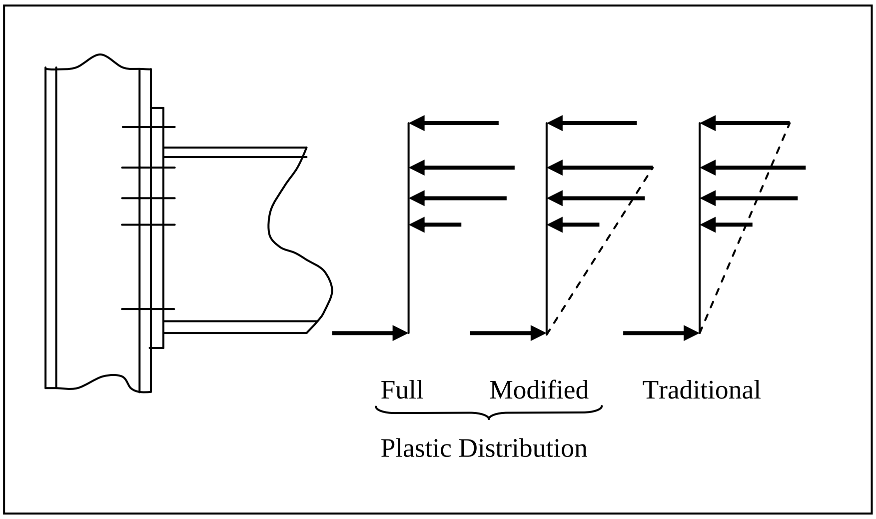


Figure 3.1: Distribution of bolt forces

In this chapter, the analytical procedures of determining the moment capacity, and thus generating the standardised connection capacity tables were explained in details. All of the checks required were calculated using programs done on Excel Worksheet for flush end plate and extended end plate connections. The sizes of the connections were kept the same while the size of the beams were chosen almost the same as in SCI's guide for the purpose of comparison.

3.2 Hypotheses

Even though the cross section of a TWP section comprised of a thin and usually deep web (as a result, a larger value of d/t_w ratio), the buckling would not likely to occur because of the stiffening strength provided by the corrugated-shaped web. The deep web (which means the longer lever arm), therefore, will result in an increase in the moment capacity of the connection as compared to the conventional section of the same weight. Furthermore, the flanges of a TWP section are normally made of S355 grade steel of which is stronger than the usual S275 grade steel. Figure 3.2 shows the critical dimensions of a conventional hot-rolled section UB and the built-up hybrid section TWP.

As far as the calculation of the value of moment capacity is concerned, the procedures outlined by SCI and BCSA (1995) could be adopted. However, it is recommended that the shear capacity based on the local buckling and global buckling could be checked as well.

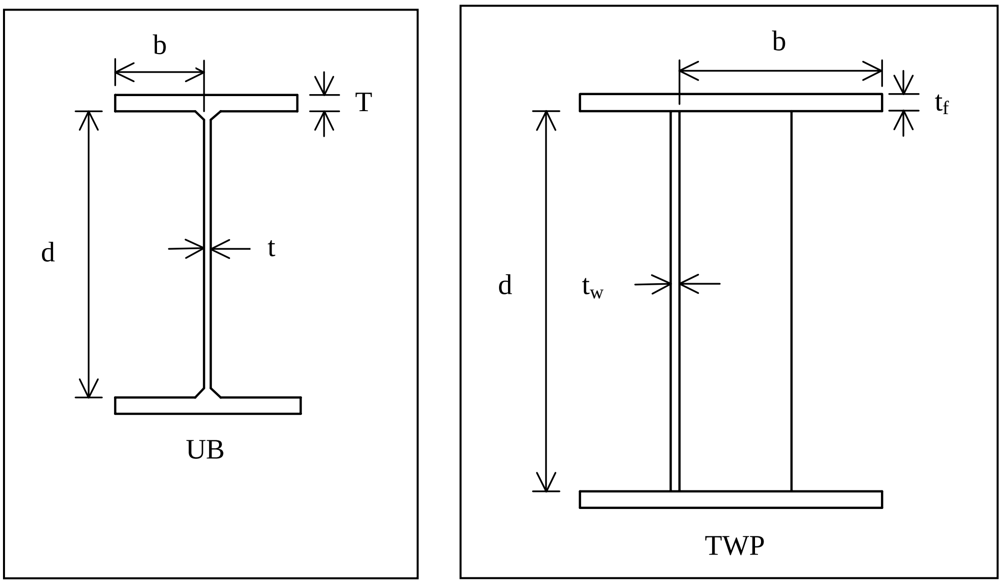


Figure 3.2: Critical dimensions for classification of UB and TWP sections

3.3 Advantages of Standardised Partial Strength Connections

In a typical braced steel frame, connections accounts for 5% of the weight of the frame, yet the cost of fabrication is 30% or more of the total construction cost (Md Tahir, 1995). Therefore, standardisation of connections is one way of reducing the total cost by offerings several advantages. The benefits that results from using standardised connections can be listed as follows:

a) The number of connection types could be reduced since only selected connection types that are tabulated.

b) Few standard parts for fitting are usually needed due to the limited amount of connections configuration. As a result, better availability of the connection, reduced material costs, reduced buying, storage and handling time, could be achieved.

c) The standardised connections could be fabricated using one grade, one diameter of bolt and limited range of length. Therefore, the fabricator could save time changing drills. Erection of frames could also be faster, and fewer mistakes could be encountered on site.

d) The use of small and single pass fillet welds for connecting the endplates to the beam could avoid the edge preparation and the amount of NDT required

3.4 Capacity Checks

As mentioned in Chapter 2, capacity checks are required to be conducted for three critical zones of a joint. These zones are tension zone, compression zone and shear zone (horizontal and vertical). Each zone comprises of several main checks depending on the potential failure on the beam, the column, or the bolts. According to SCI's guide, altogether there are fifteen principal checks to be made; however, not all checks are necessary since a connection may have a different configuration. In particular, the checks could be separated into the beam side checks and the column side checks. Figure 3.3 shows an extended end plate connection with the critical zones mentioned above.

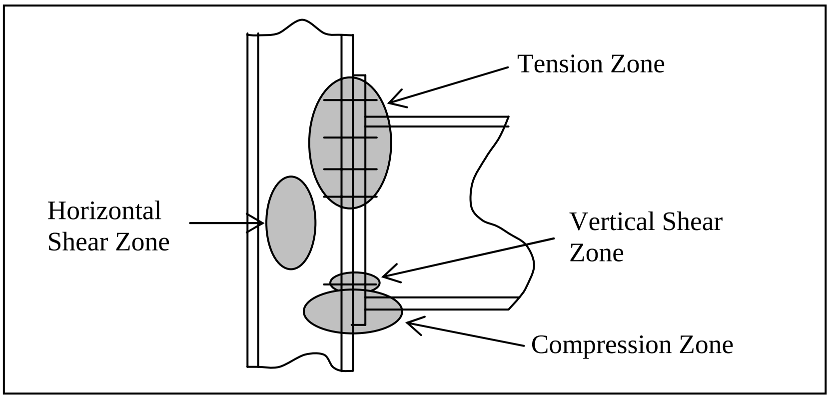


Figure 3.3: Critical zones of a joint

3.4.1 Tension Zone

In the tension zone, resistances of each bolt row are determined and may be limited by any of these potential failures:

a) Beam side:

- Beam web tension
- Endplate bending and bolt strength

b) Column side:

- Column web tension
- Column flange bending and bolt strength

For column flange bending or endplate bending, EC 3 approach uses a method called ’equivalent T-stub’ to simulate the yield line patterns which occur around the bolts. The T-stub is then checked, using some formula developed by taking into account the prying force on the bolts, against the three possible modes of failure as depicted in Figure 3.4.

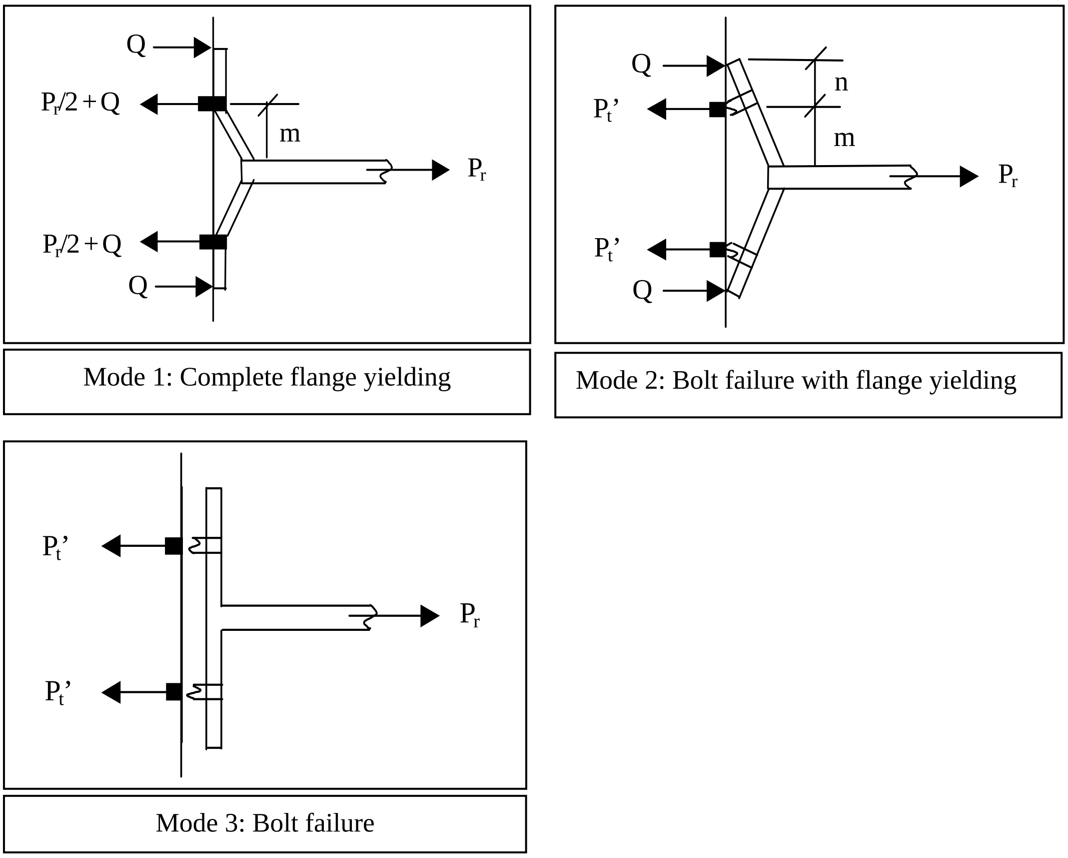


Figure 3.4: Modes of failure for equivalent T-stub

The steps involve the calculation of the resistance in each row of bolts starting from the top row, Row 1, to the next. At each row, the resistance is calculated for the particular row alone and the combination of the particular row with the row above less the previous calculated resistance. The control value of bolt resistance is then taken as the least of all the values calculated for all the rows and their combinations. Figure 3.5 shows the details steps in the process of calculating the resistance of bolts for each row.

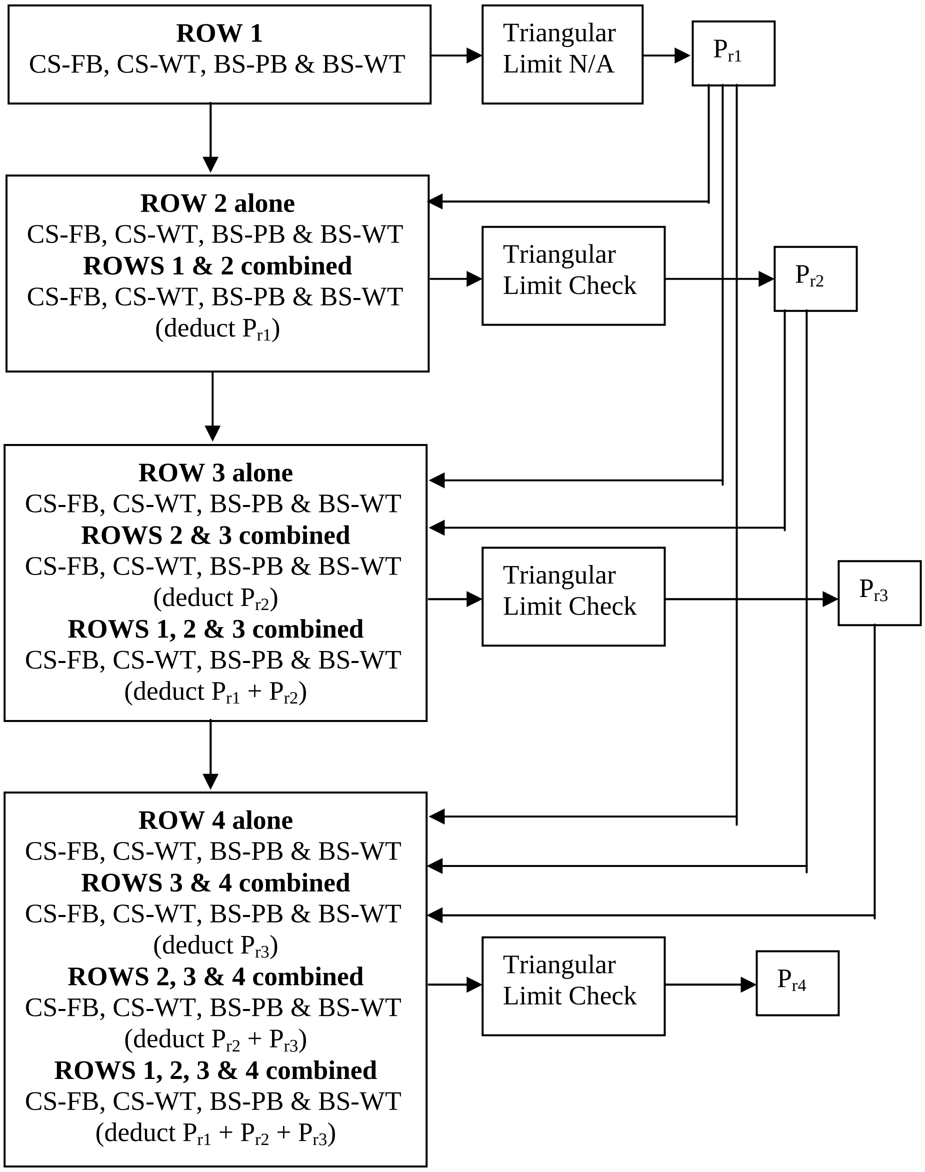
The resistances of bolts obtained from the above-mentioned steps resulted in a plastic distribution (refer Figure 3.1). This plastic distribution of bolt forces must be modified unless either the endplate thickness or the column flange thickness is less than the limit determined using Equation. 3.1 for beam side and Equation 3.2 for column side respectively. In other words, the triangular distribution only needs to be imposed if both sides of the connection (beam and column) exceed their respective thickness limits.

Beam side: $$t_p < \frac{d}{1.9}\sqrt{\frac{U_f}{p_{yp}}} \qquad \ldots(3.1)$$

Column side: $$T_c < \frac{d}{1.9}\sqrt{\frac{U_f}{p_{yc}}} \qquad \ldots(3.2)$$

where

t_p	= endplate thickness
T_c	= column flange thickness
d	= bolt diameter
U_f	= ultimate tensile strength of bolt
p_{yp}	= design strength of endplate
p_{yc}	= design strength of column



Notes:
CS-FB : Column Side Flange Bending
CS-WT : Column Side Web Tension
BS-PB : Beam Side Plate Bending
BS-WT : Beam Side Web Tension

Figure 3.5: Steps in the process of calculating resistance of bolts

3.4.2 Compression Zone

For bolted connections, the compression zone concentrates around the bottom flange of the beam assuming typical deformation due to the clockwise rotation. Three possible failures may occur and have to be checked for capacities, which are:

a) Beam flange compression
b) Column web crushing
c) Column web buckling

Generally, it is assumed that the compression force is being carried entirely by the bottom flange and, hence, the point of action is taken at the centre of the flange. The checks for the compression zone then can be applied by using the approach as in BS 5950.

3.4.3 Shear Zone

In shear zone, the shear could be acting in two directions: horizontal and vertical. However, the horizontal shear, which affects the column web panel, is usually the most critical condition. The value of the horizontal shear depends upon the forms of connection whether it is one-sided or two sided. For a one-sided connection, the horizontal shear force is equal to the compression force at the bottom flange of the beam whereas in two-sided connection, the value is the addition of the two compression forces on both sides. In situations where column web panel shear is the connection's failure mechanism, stiffening elements are needed to reinforce the column web panel. The stiffeners could be placed parallel to beam flanges, diagonally or as supplementary web plates.

3.5 Standardisation

In producing standardised partial strength connection capacity tables, standard components were proposed. Size of bolts, dimension of end plates and size of welds were chosen as to represent the most suitable configuration for the partial strength connections. For this reason and for the purpose of comparison, standard components as used by SCI (see Table 3.1) were adopted and retained.

Table 3.1: Standard components used in the standardised capacity tables

Elements	Preferred Option	Notes
Bolts	M24 8.8 M20 8.8	Bigger beams Smaller beams
End Plates	200 mm x 12 mm 250 mm x 15 mm	
Welds	10 mm 8 mm	Flange to end plate Web to end plate

For all of the configurations, the preferred size of bolts was the M24 8.8, but for smaller size of beams, the M20 8.8 was adequate. Two sizes of endplates were adopted for the connections: the 200 mm x 12 mm and 250 mm x 15 mm. The 200 mm x 12 mm endplate was used for the smaller size of beams, whereas the 250 mm x 15 mm endplate was used for the bigger size of beams. The type of weld used for the connecting the endplate to the beam was the fillet weld of size 8 mm for connecting the web and 10 mm for connecting the flanges. However, other sizes of fillet weld may also be used if appropriate checks for their adequacy have been carried out.

3.6 Design Procedures

As outlined in the SCI's guide, there are altogether fifteen principal checks to be carried out in designing a moment connection. However, for typical bolted partial strength connections, only five main steps or procedures need to be performed. These steps are listed as follows:

- STEP 1

  - Determining the potential resistances of bolt rows in the tension zone.

  This step is further divided into three sub-steps:

  STEP 1A: End plate or column flange bending or bolt yielding

  STEP 1B: Web tension in beam or column

  STEP 1C: Modification of bolt row force distribution

- STEP 2

  - Determining the potential resistances in the compression zone.

  This step consists of two steps, one for the column and one for the beam:

  STEP 2A: Resistance of the column web

  STEP 2B: Resistance of the beam flange and web

- STEP 3

  - Determining the potential resistance of the column web panel in shear.

- STEP 4

  - Calculation of moment capacity.

- STEP 5

  - Design for vertical shear forces.

A spreadsheet program using Microsoft Excel was written by the author based on the above-mentioned steps. By using the program, standardised capacity tables for partial strength connections were generated. In order to suit with the standardised tables in the SCI's publication, six tables for the flush endplate connections and eight tables for the extended endplate connections were developed. The standardised tables obtained for both types of connections were described and discussed in details in Chapter 6.

3.6.1 Effects of Corrugated and Thin Web of TWP

Between the hot-rolled UB section and the built-up TWP section, the main different lies in the configuration of the web. The web of a TWP section is made corrugated in a trapezoidal shape and is usually very thin (in the range of 2 mm to 8 mm). This thin and corrugated web might have some effects on the value of the potential resistance. Based on the procedures outlined by SCI, the only check where the web might play a significant role is the Beam Side Web Tension. However, the governing resistance depends upon whether or not the potential resistance from this check is the most critical.

3.6.2 Worked Example

For the purpose of illustrating the steps involved in designing and thus determining the capacity of a bolted partial strength connection, a worked example on a flush end plate connection was described in detail below. The connection was made of a 200 mm x 12 mm endplate welded by a 10 mm and 8 mm fillet weld to the flange and web of a TWP beam respectively. The TWP beam was of a size 900 x 250 x 109.2 /20/4 and the column was of a size 356 x 368 x 202 UC. Grade of material for all elements of the connection was S275 (Grade 43). Figure 6.6 shows the geometry of the illustrated flush endplate connection with two rows of tension bolts (four numbers of top bolts) and two rows of shear bolts (four numbers of bottom bolts).

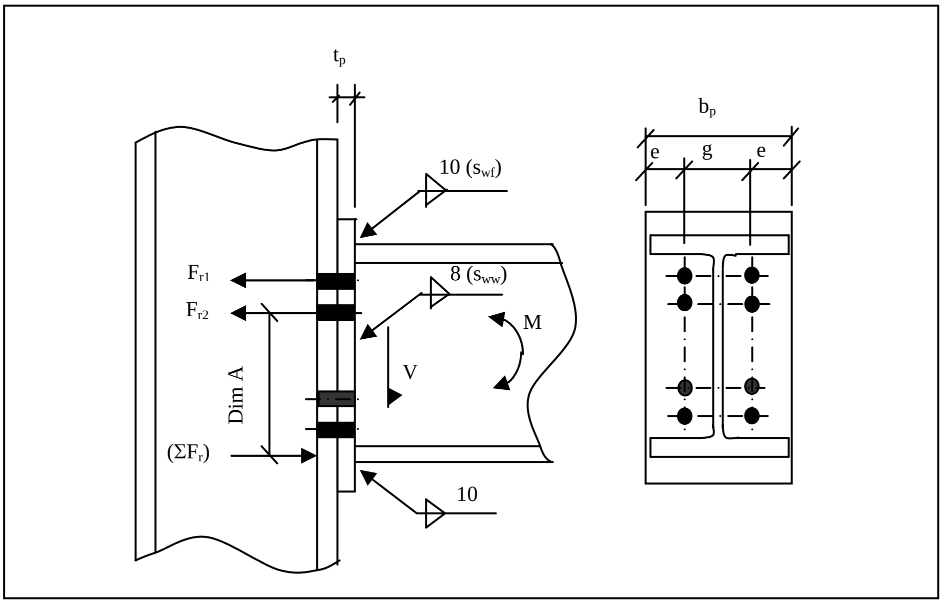


Figure 3.6: Geometry of the illustrated flush end plate connection.

MOMENT CAPACITY FOR FLUSH END PLATE CONNECTION

	TWP:	**UC:**	**End-plate:**
D_b=	900mm	D_c=374.5mm	g=90mm
B_b=	250mm	B_c=374.4mm	e=55mm
T_b=	20mm	T_c=27mm	b_p=200mm
t_b=	4mm	t_c=16.8mm	t_p=12mm
d_b=	860mm	r_c=15.2mm	n_s =4
gred=	43	d_c=290.2mm	n_t =4
P_{yb}=	265kN/mm^2	gred=43	D_{bolt}=20mm
s_{ww}=	8mm	P_{yc}=265kN/mm^2	PsAs=91.9kN
s_{wf}=	10mm		P_t=137kN
			P_{yp}=275kN/mm^2

Result summary:

P_c	=1224.3kN			
P_{r1}	= 207.5214kN	P_v	=1000.364kN	column web buckling:
P_{r2}	= 138.7020kN	Mc	=274.8823kNm	λ = 43.18452

Sum F_{ri} = 346.2234kN P_v =514.64kN p_c = 220N/mm^2

Connection Geometry

Column side:

$m = g/2 - tc/2 - 0.8r_c$ =24.44mm

$e = B_c/2 - g/2$ =142.2mm

n = smallest of e (column flange), e (end-plate) or 1.25m (column flange)

n =30.55mm

Beam side:

$m = g/2 - t_b/2 - 0.8s_w$ = 36.6mm (assume 8 FW)

$e = b_p/2 - g/2$ = 55mm

n = smallest of e (column flange), e (end-plate) or 1.25m (end-plate)

n = 45.75mm

POTENTIAL RESISTANCE OF BOLTS IN TENSION ZONE

BOLT ROW 1

Column flange bending:

Calculate effective length of T-stub. The bolt row is not influenced by a stiffener or a free end.

From table 2.5 and 2.4, L_{eff} is the minimum of:

$2 * \pi * m$ = 153.581mm

or $4m + 1.25e$ = 275.51mm

Thus, Leff = 153.5810mm

Calculate M_p for the column flange:

$M_p = (L_{eff} * T_c^2 * P_{yc})/4 = 7417.384kNmm$

Find the critical mode. This is the minimum of the following three formulae:

Mode 1: $P_r = (4*Mp)/m$ = 1213.975

Mode 2: $P_r = (2*Mp+n\sum P'_t)/(m+n)$ = 421.9943kN

Mode 3: $P_r = \sum P't$ = 274kN **274**-------------- 1

Column web tension:

$P_t = L_t * t_c * P_{yc}$

L_t is the tensile length of web assuming a spread of load of 1:1.73 from the bolts

$L_t = 1.73*2*g/2$ = 155.7mm

Thus, P_t = 693.1764kN **693.1764**------- 2

End plate bending:

Calculate effective length of T-stub. Row 2 is below the beam flange of an extended end plate. From table 2.4 and 2.5, L_{eff} is given by:

Min{Max(pattern ii,pattern iii),pattern i}

Pattern ii: 4m+1.25e = 215.15mm

Pattern iii: αm_1

α is obtained from Figure 2.16 using the following parameters:

$m_1 = m = 36.6mm$

$m_2 = 60 - Tb - 0.8swf = 32mm$

$\lambda_1 = m_1/(m_1+e) = 0.204698mm$

$\lambda_2 = m_2/(m_1+e) = 0.178971mm$

The chart show $\alpha = 2\pi = 6.284$

Thus, $\alpha m_1 = 229.9944mm$

Pattern i: $2\pi m = 229.9944mm$

$0.7Bb = 175mm$ $0.8tp = 9.6mm$

Thus, $L_{eff} = 229.9944mm$

$M_p = (L_{eff} * t_p^2 * P_{yp})/4 = 2276.945kNmm$

Mode 1: $P_r = (4*Mp)/m = 248.8464kN$

Mode 2: $P_r = (2*Mp+n\sum P'_t)/(m+n) = 207.5214kN$

Mode 3: $P_r = \sum P't = 274kN$ **207.5214**-------- 3

Beam web tension:

Row 1 is situated below the beam flange, the underside of which is only 44mm from the bolt row. This would place the flange within the tensile length and therefore beam web tension can be discounted. **N/A**------------- 4

Triangular Limit:

Does not apply. **N/A**------------- 5

Therefore the potential resistance of row 1, P_{r1} = 207.5214kN

BOLT ROW 2

Row 2 alone

Column flange bending:

P_r is calculated as for row 1.

Therefore, $P_r = 274kN$ **274**--------------- 7

Column web tension:

As before, $P_t = 693.1764kN$ **693.1764**-------- 8

End plate bending:

Calculate effective length of T-stub. The bolt row is not influenced by a stiffener or a free end.

From tables 2.5 and 2.4, L_{eff} is the minimum of:

$2 * \pi * m = 229.9944mm$

or $4m + 1.25e = 215.15mm$

Thus, $L_{eff} = 215.15mm$

$$M_p = (L_{eff} * t_p^2 * P_{yp})/4 = 2129.985kNmm$$

Mode 1: $P_r = (4*M_p)/m = 232.7852kN$

Mode 2: $P_r = (2*Mp+n\sum P'_t)/(m+n) = 203.9523kN$

Mode 3: $P_r = \sum P't = 274kN$

9

P_r = 203.9523kN **203.9523--------**

Beam web tension:

$P_t = L_t * t_b * P_{yb}$

L_t = 1.73*2*g/2 = 155.7mm

P_t = 165.042kN **165.042-------** 10

Row 1+2 combined

Column flange bending:

Calculate effective length of T-stub.
Neither row is influenced by stiffener or a free edge. From tables 2.6 and 2.4, L_{eff} for the group is given by:

L_{eff} = 2{ ii/2 + p/2 } = 2 * (2m + 0.625e + p/2 }
= 365.51mm

$M_p = (L_{eff} * T_c^2 * P_{yc})/4$ =17652.76kNmm

Mode 1: $P_r = (4*M_p)/m$ = 2889.159kN

Mode 2: P_r = (2*Mp+n∑P'$_t$)/(m+n) = 946.4798kN

Mode 3: P_r = ∑P't = 548kN **548------------** 11

Thus, P_r = (min Mode 1 to 3) - P_{r1} =340.4786kN **340.4786------** 15

Column web tension:

$P_t = L_t * t_c * P_{yc}$

$L_t = \{ g/2 * 1.73 * 2\} + p = 245.7mm$

$P_{t(1+2)} = 1093.856kN$ **1093.856**------ 12

For row 2, $P_t = P_{t(1+2)} - P_{r1} = 886.335kN$ **886.335**-------- 16

End plate bending:

Calculate effective length of T-stub. Row 1 is adjacent to beam flange. Row 2 is not influenced by a stiffener or free edge. From tables 2.6 and 2.4, L_{eff} is given by:

Max { ii/2, (iii - ii/2) } + p/2 + ii/2 + p/2

$L_{eff} = (4m + 1.25e)/2 = (4m + 1.25e)/2 + p$

$= 4m + 1.25e + p$

$= 305.15mm$

or $L_{eff} = \{\alpha m_1 - (4m+1.25e)/2 \} + p/2 + (4m+1.25e)/2 + P/2$

$=\alpha m_1 + p$ (α as for row 2 alone)

$= 319.9944mm$

Thus, $L_{eff} = 319.9944mm$

$M_p = (L_{eff} * t_p^2 * P_{yp})/4 = 3167.945kNmm$

Mode 1: $P_r = (4*M_p)/m = 346.2234kN$

Mode 2: $P_r = (2*M_p+n\sum P'_t)/(m+n) = 381.383kN$

Mode 3: $P_r = \sum P't = 548kN$ **346.2234**------ 13

P_r for row 2 is taken as the minimum from Mode 1 to 3 minus P_{r1}

17

Therefore, P_r = 138.702kN **138.702--------**

Beam web tension:

Not applicable **N/A------------** 14

The potential resistance for row 2, P_{r2} is the smallest values from boxes 7 to 10, 15 to 19. Therefore, the potential resistance of row3,

P_{r3} = 138.702kN **138.7020------** 42

$d/1.9 * (U_f/P_{yp})^{\wedge 1/2}$ =18.11708

$d/1.9 * (U_f/P_{yc})^{\wedge 1/2}$ =17.78463

If triangular limit apply, P_{r3} = 185.0191kN

RESISTANCE OF THE COLUMN WEB AND BEAM FLANGE IN THE COMPRESSION ZONE

The compressive resistance, P_c is the minimum of the following values:

1) Column web crushing:

$P_c = (b_1 + n_2) * t_c * P_{yc}$

$b_1 = T_b + 2*FW + 2*t_p$ = 64mm

$n_2 = 2 * \{ 2.5 * (T_c + r_c) \}$ = 211mm

Thus, P_c = 1224.3kN

2) Column web buckling:

$P_c = (b_1 + n_1) * t_c * p_c$

n_1 = depth of column = 374.5mm

p_c is obtained from the table 27(c) of BS 5950 using:

$\lambda = 2.5d_c/t_c$ =43.18452

From the table, p_c =220N/mm^2

Thus, P_c = 1620.696kN

3) Beam flange crushing:

$P_c = 1.4 * P_{yb} * T_b * B_b$ = 1855kN

Therefore the resistance in the compression zone, P_c = 1224.3kN

RESISTANCE OF THE COLUMN WEB PANEL IN SHEAR

$P_v = 0.6 * P_{yc} * A_v$

$A_v = t_c * D_c$ = 6291.6mm^2

Therefore, P_v = 1000.364kN

CALCULATION OF MOMENT CAPACITY

Horizontal equilibrium is satisfied by:

$$\sum F_{ri} + N = F_c$$

where Fc is the smallest of:

$\sum F_{ri} = N = 346.2234kN$ (N=0)

$P_c = 1224.3kN$

$P_v = 1000.364kN$

In this example of two sided connection with equal and opposite moments, the column web panel shear is zero, and P_v is not critical.

Thus, Pc is critical. $\sum F_{ri}$ must be equal to 346.2234kN

Reduce $\sum F_{ri}$ by a total of = 0kN

Starting with the lowest row, $F_{r3} = 138.702kN$

Moment capacity:

The moment capacity of the connection is:

$$M_c = \sum(F_{ri} * h_i)$$

where $h_1 = 830$

$h_2 = 740$

Thus, $M_c = 274.8823kNm$

DESIGN FOR VERTICAL SHEAR FORCE

$$Pv = n_s P_{ss} + n_t P_{ts}$$

P_{ss} is the shear capacity of the single bolt in the shear zone and is the lesser of:

$p_s A_s = 91.9kN$

$d t_p P_b = 110.4kN$

dT_cp_b = 248.4kN

P_{ss} = 91.9kN

P_{ts} is the shear capacity of the single bolt in the tension zone and is the lesser of:

$0.4p_sA_s$ = 36.76kN
dt_pP_b = 110.4kN
dT_cp_b =248.4kN

P_{ts} = 36.76kN

Thus, Pv = 514.64kN

WELD DESIGN

Assuming that the size of fillet welds at the flange and web are not critical. The size of 10mm FW for connecting the flange to the end plate and 8mm FW for connecting the web to the end plate are adequate.

3.6.3 Remarks on the Capacity Tables

The capacity tables produced were divided into two major elements within a joint, which are the beam and the column. On the beam side, distribution of tension force on each bolt row (F_{R1}, F_{R2} and so forth) and compression force (ΣF_R) were determined and indicated on the connection diagram provided. Moment capacity was then calculated based on the forces using the lever arm length. If happens that the bolt forces were reduced due to the lesser capacity of column in the tension zone, a new moment capacity had to be calculated. Grade of beams to be used was normally taken as S275. The size of fillet weld for the flange was 10 mm whereas for the web the size was 8 mm.

On the column side, checks were made for the tension zone, the compression zone and the web panel shear capacity. If the capacity of column in the tension zone was less than the bolt forces, the value of bolt forces should be reduced or tension stiffeners were to be provided. If the capacity of the column in the compression zone was less than the compression force, compression stiffeners were to be provided.

3.7 Initial Rotational Stiffness

Although inside the standardised capacity tables, initial rotational stiffness of a connection is not provided, its value is important and essential for the analysis and design of semi-rigid frames. In this section, the steps involve in determining the initial stiffness of a semi-rigid connection analytically were illustrated by a worked example as described below.

According to Annex J 3.7 of EC 3 (DD ENV 1993-1-1: 1992), the rotational stiffness, S_j of a semi-rigid connection can be approximated using the mathematical expression of:

$$S_j = \frac{E h_1^2 t_{wc}}{\sum \frac{\mu_i}{k_i}\left[\frac{F_i}{F_{i.Rd}}\right]^2} \qquad \ldots(2.5)(\text{repeated})$$

where

h_1	= the lever arm measured from the first bolt row below the tension flange to the centre of compression
μ_i	= modification factor
k_i	= stiffness factor for component i
F_i	= force in component i due to applied moment M

$F_{i.Rd}$ = design resistance of component *i* of the connection.

By using this expression, the rotational stiffness obtained is the secant stiffness with respect to a specific value of the applied moment, particularly at two thirds of the connection's design moment of resistance or M_{Sd} (for the case of non-linear and tri-linear characteristics) and at the design moment of resistance, M_{Rd} (for the case of bi-linear characteristic). It is noticed that the stiffness depends predominantly by the parameter k_i and the ratio of force carried in each component to its design resistance. The parameter k_i is calculated for each component in the critical zones as follows:

a) column web in the shear, tension and compressive zones (k_1, k_2 and k_3 respectively)
b) column flange in the tension zone (k_4)
c) bolts in the tension zone (k_5)
d) end plate in the tension zone (k_6)

However, due to the complex nature of the above-mentioned mathematical expression and the recent advancement of the matter, the Annex J was revised in 1998. In this revised version, the major difference occurs in determining the rotaional stiffness of which a new mathematical expression is proposed. It has been simplified and aims at obtaining the approximate value of the connection's initial stiffness. The expression is further reduced as published in the recent draft versions of EC 3 (EN 1993-1-8: 2002 and BS EN 1993-1-8: 2005). Hence, the expression for calculating the rotational stiffness can be written as follows:

$$S_j = \frac{Ez^2}{\mu \sum_i \frac{1}{k_i}} \qquad \ldots(3.3)$$

where

z = lever arm (refer to Clause 6.2.5 in EC 3)

μ = stiffness ratio of $\frac{S_{j,ini}}{S_j}$

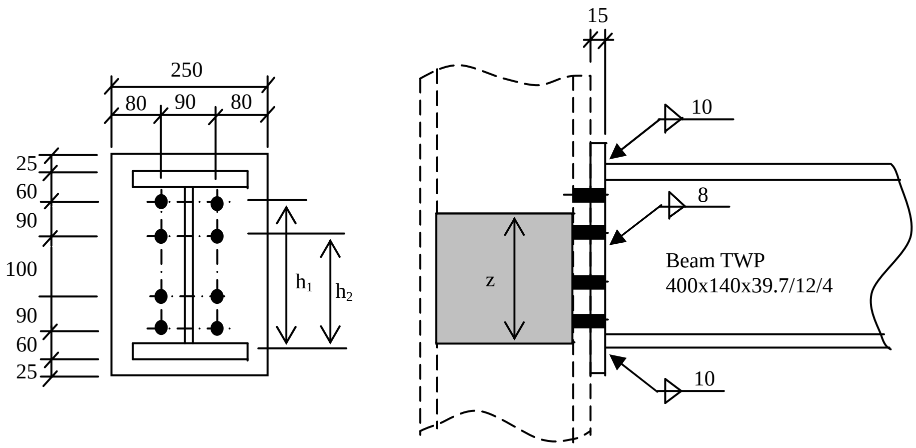
k_i = stiffness coefficient for basic joint component *i*,

$S_{j.ini}$ = initial rotational stiffness of the joint.

(The initial rotational stiffness is the slope of the elastic range of the design moment rotation characteristic and is given by the above expression with μ = 1.0)

The components of which contribute to the calculation of paramater k_i remain unchanged. However, the expression for each k is different slightly to reflect the changes made to the previous approach. Outlined in details below are two worked examples for calculating the initial rotational stiffness of a flush endplate connection (F2R24P2) and an extended endplate connection (E2R20P1).

a) Flush End Plate (F2R24P2)



__PRELIMINARY CALCULATION__

Column:

d_c = 246.7 mm

A_{vc} = column's shear area = $A_c - 2bt_{fc} + (t_{wc} + 2r)t_{fc}$

= 15000 – (2 x 306.8 x 18.7) + (11.9 + 2 x 15.2) x 18.7 = 4316.7 mm^2

β = transformation parameter (Clause 5.3(7)) = 1.0 (for one-sided joint configurations)

$m = \dfrac{w - t_{wc}}{2} - 0.8r$ = (90 – 11.9)/2 – 0.8 x 15.2 = 26.9 mm

$e = \dfrac{b - w}{2}$ = (306.8 – 90)/2 = 108.4 mm

Beam:

z = 400 – 60 – 90/2 – 12/2 = 289 mm

End plate:

$m_1 = \frac{w - t_b}{2} - 0.8\sqrt{2}a_w$ = (90 – 4)/2 – 0.8 x √2 x 5.6 = 36.7 mm

$m_2\,(Row1) = (60 - t_f) - 0.8\sqrt{2}a_f$ = (60 – 12) – 0.8 x √2 x 7.0 = 40.1 mm

$m_2\,(Row2) = (150 - t_f) - 0.8\sqrt{2}a_f$ = (150 – 12) – 0.8 x √2 x 7.0 = 130.1 mm

e_p = 80 mm

Row 1:

$\lambda_1 = \frac{m_1}{m_2 + e_p}$ = 36.7/(40.1 + 80) = 0.31

$\lambda_2 = \frac{m_2}{m_2 + e_p}$ = 40.1/(40.1 + 80) = 0.33

α = 8.0 (according to chart)

Row 2:

$\lambda_1 = \frac{m_1}{m_2 + e_p}$ = 36.7/(130.1 + 80) = 0.17

$\lambda_2 = \frac{m_2}{m_2 + e_p}$ = 130.1/(130.1 + 80) = 0.62

α = 8.0 (according to chart)

Bolts:

$L_b = t_{fc} + t_p + t_{washer} + 0.5(h_{bolt} + h_{nut})$ = (18.7 + 15 + 6 + 0.5 x (16 + 12) = 53.7 mm

A_s = tensile area of bolt = 353 mm^2

h_1 = 334 mm

h_2 = 244 mm

COLUMN WEB PANEL IN SHEAR

$k_1 = \frac{0.38A_{vc}}{\beta z}$ = 0.38 (4316.7)/1(289) = **5.68 mm**

COLUMN WEB IN COMPRESSION

$$k_2 = \frac{0.7 b_{eff,t,wc} t_{wc}}{d_c}$$ = (0.7 x 200.30 x 11.9)/246.7 = **6.76 mm**

$b_{eff,t,wc}$ = effective width of column's web depth in compression (6.2.4.2)

t_{wc} = thickness of web of column

d_c = depth of column's web

$$b_{eff,t,wc} = t_{fb} + 2\sqrt{2}a_p + 5(t_{fc} + s) + s_p \quad ; \quad s_p = t_p + s$$

= 12 + 2√2 x 7 + 5 x (18.7 + 10) + (15 + 10) = 200.30 mm

For a joint with two or more rows of bolts in tension, according to Table 6.10, the stiffness coefficients k_i to be taken into account are k_1, k_2 *and* k_{eq}. k_{eq} should be based upon (and replace) the stiffness coefficients k_i for:

- the column web in tension, k_3
- the column flange in bending, k_4
- the end plate in bending, k_5
- the bolts in tension, k_{10}

From 6.3.3,

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

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

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

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

<u>ROW 1:</u>

<u>COLUMN WEB IN TENSION</u>

$$k_3 = \frac{0.7 b_{eff,t,wc} t_{wc}}{d_c}$$ = (0.7 x 168.93 x 11.9)/246.7 = **5.70 mm**

$b_{eff,t,wc}$ = effective length of column's web depth in tension (Table 6.4)

$= \min[2\pi m\,; 4m + 1.25e]$ = min[2 x 3.14 x 26.9 ; 4 x 26.9 + 1.25 x 108.4]

= min[168.93 ; 243.1]

= 168.93 mm

COLUMN FLANGE IN BENDING

$k_4 = \frac{0.9 l_{eff} t_{fc}^{\ 3}}{m^3}$ = (0.9 x 168.93 x 18.7^3)/26.9^3 = **51.08 mm**

l_{eff} = smallest value of effective lengths of column's flange without web stiffeners (Table 6.4)

= $b_{eff,t,wc}$ = 168.93 mm

END PLATE IN BENDING

$k_5 = \frac{0.9 l_{eff} t_p^{\ 3}}{m^3}$ = (0.9 x 230.48 x 15^3)/36.7^3 = **14.16 mm**

l_{eff} = smallest value of effective lengths (Table 6.6)

$= \min[2\pi m\,; \alpha m]$ = min[2 x 3.14 x 36.7 ; 8 x 36.7]

= min[230.48 ; 293.60]

= 230.48 mm

BOLTS IN TENSION

$k_{10} = \frac{1.6 A_s}{L_b}$ = (1.6 x 353)/53.7 = **10.52 mm**

Effective stiffness coefficient for Row 1:

$k_{eff,1} = \frac{1}{\sum_r \frac{1}{k_{i,r}}} = \frac{1}{(1/5.70 + 1/51.08 + 1/14.16 + 1/10.52)}$ = **2.77 mm**

ROW 2:

COLUMN WEB IN TENSION

$k_3 = \frac{0.7 b_{eff,t,wc} t_{wc}}{d_c}$ = (0.7 x 168.93 x 11.9)/246.7 = **5.70 mm**

$b_{eff,t,wc}$ = effective length of column's web depth in tension (Table 6.4)

= $\min[2\pi m;4m+1.25e]$ = min[2 x 3.14 x 26.9 ; 4 x 26.9 + 1.25 x 108.4]

= min[168.93 ; 243.1]

= 168.93 mm

COLUMN FLANGE IN BENDING

$k_4=\dfrac{0.9l_{eff}t_{fc}^{\;3}}{m^3}$ = (0.9 x 168.93 x 18.7^3)/26.9^3 = **51.08 mm**

l_{eff} = smallest value of effective lengths of column's flange without web stiffeners (Table 6.4)

= $b_{eff,t,wc}$ = 168.93 mm

END PLATE IN BENDING

$k_5=\dfrac{0.9l_{eff}t_p^{\;3}}{m^3}$ = (0.9 x 230.48 x 15^3)/36.7^3 = **14.16 mm**

l_{eff} = smallest value of effective lengths (Table 6.6)

= $\min[2\pi m;4m+1.25e]$ = min[2 x 3.14 x 36.7 ; 4 x 36.7 + 1.25 x 80]

= min[230.48 ; 246.80]

= 230.48 mm

BOLTS IN TENSION

$k_{10}=\dfrac{1.6A_s}{L_b}$ = (1.6 x 353)/53.7 = **10.52 mm**

Effective stiffness coefficient for Row 2:

$$k_{eff,1}=\frac{1}{\sum_r \frac{1}{k_{i,r}}}=\frac{1}{(1/5.70+1/51.08+1/14.16+1/10.52)}$$ = **2.77 mm**

z_{eq} = (2.77 x 334^2)/(2.77 x 244) = 457.2 mm

k_{eq} = (2.77 x 334) + (2.77 x 244)/457.2 = **3.50 mm**

STIFFNESS OF THE JOINT

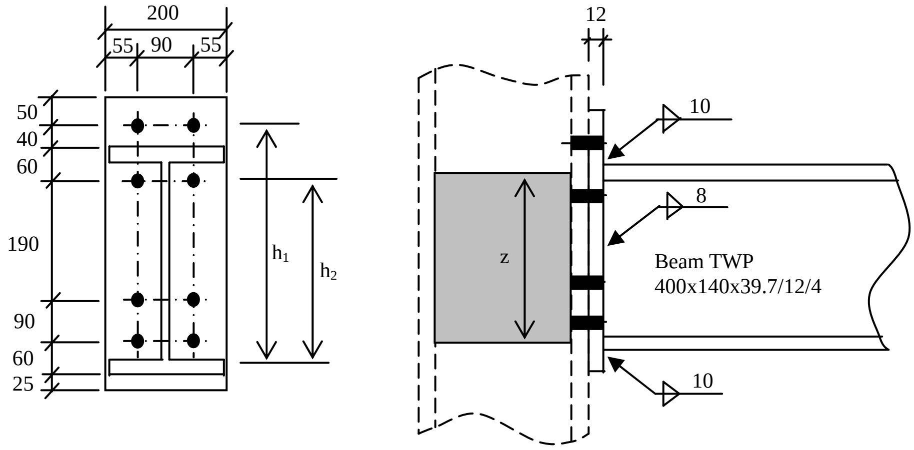
Initial Stiffness:

$$S_{j,ini} = \frac{Ez^2}{\mu \sum_i \frac{1}{k_i}} = \frac{205000\,x\,289^2}{1.0(1/5.68 + 1/6.76 + 1/3.50)} \text{ x } 10^{-6} = \mathbf{28082\ kNm}$$

Secant Stiffness:

$$S_j = \frac{S_{j,ini}}{2} = 28082/2 = \mathbf{14041\ kNm}$$

b) Extended End Plate (E2R20P1)



<u>**PRELIMINARY CALCULATION**</u>

<u>*Column:*</u>

d_c = 246.7 mm

A_{vc} = column's shear area = $A_c - 2bt_{fc} + (t_{wc} + 2r)t_{fc}$

= 15000 – (2 x 306.8 x 18.7) + (11.9 + 2 x 15.2) x 18.7 = 4316.7 mm^2

β = transformation parameter (Clause 5.3(7)) = 1.0 (for one-sided joint configurations)

$m = \frac{w - t_{wc}}{2} - 0.8r$ = (90 – 11.9)/2 – 0.8 x 15.2 = 26.9 mm

$e = \frac{b - w}{2}$ = (306.8 – 90)/2 = 108.4 mm

<u>*Beam:*</u>

z = 400 – 2 x (12/2) = 388 mm

End plate:

$m_1 = \frac{w - t_b}{2} - 0.8\sqrt{2}a_w$ = (90 – 4)/2 – 0.8 x √2 x 5.6 = 36.7 mm

$m_2 = \left(60 - t_f\right) - 0.8\sqrt{2}a_f$ = (60 – 12) – 0.8 x √2 x 7.0 = 40.1 mm

e_p = 55 mm

$m_x = u_1 - 0.8\sqrt{2}a_f$ = 40 – 0.8 x √2 x 7.0 = 32.1 mm

e_x = 50 mm

Row 1:

$\lambda_1 = \frac{m_1}{m_2 + e_p}$ = 36.7/(32.1 + 55) = 0.42

$\lambda_2 = \frac{m_2}{m_2 + e_p}$ = 32.1/(32.1 + 55) = 0.37

α = 6.6 (according to chart)

Row 2:

$\lambda_1 = \frac{m_1}{m_2 + e_p}$ = 36.7/(40.1 + 55) = 0.39

$\lambda_2 = \frac{m_2}{m_2 + e_p}$ = 40.1/(40.1 + 55) = 0.32

α = 7.1 (according to chart)

Bolts:

$L_b = t_{fc} + t_p + t_{washer} + 0.5(h_{bolt} + h_{nut})$ = 18.7 + 12 + 6 + 0.5 x (16 + 12) = 50.7 mm

A_s = tensile area of bolt = 245 mm^2

h_1 = 434 mm

h_2 = 334 mm

COLUMN WEB PANEL IN SHEAR

$k_1 = \frac{0.38A_{vc}}{\beta z}$ = 0.38 (4316.7)/1(388) = **4.23 mm**

COLUMN WEB IN COMPRESSION

$$k_2 = \frac{0.7 b_{eff,t,wc} t_{wc}}{d_c}$$ = (0.7 x 197.30 x 11.9)/246.7 = **6.66 mm**

$b_{eff,t,wc}$ = effective width of column's web depth in compression (6.2.4.2)

t_{wc} = thickness of web of column

d_c = depth of column's web

$$b_{eff,t,wc} = t_{fb} + 2\sqrt{2}a_p + 5(t_{fc} + s) + s_p \quad ; \quad s_p = t_p + s$$

= 12 + 2√2 x 7 + 5 x (18.7 + 10) + (12 + 10) = 197.30 mm

For a joint with two or more rows of bolts in tension, according to Table 6.10, the stiffness coefficients k_i to be taken into account are k_1, k_2 *and* k_{eq}. k_{eq} should be based upon (and replace) the stiffness coefficients k_i for:

- the column web in tension, k_3
- the column flange in bending, k_4
- the end plate in bending, k_5
- the bolts in tension, k_{10}

From 6.3.3,

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

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

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

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

<u>ROW 1:</u>

<u>COLUMN WEB IN TENSION</u>

$$k_3 = \frac{0.7 b_{eff,t,wc} t_{wc}}{d_c}$$ = (0.7 x 168.93 x 11.9)/246.7 = **5.70 mm**

$b_{eff,t,wc}$ = effective length of column's web depth in tension (Table 6.4)

= $\min[2\pi m\,; 4m + 1.25e]$ = min[2 x 3.14 x 26.9 ; 4 x 26.9 + 1.25 x 108.4]

= min[168.93 ; 243.1]

= 168.93 mm

COLUMN FLANGE IN BENDING

$k_4 = \dfrac{0.9 l_{eff} t_{fc}^{\ 3}}{m^3}$ = (0.9 x 168.93 x 18.7^3)/26.9^3 = **51.08 mm**

l_{eff} = smallest value of effective lengths of column's flange without web stiffeners (Table 6.4)

= $b_{eff,t,wc}$ = 168.93 mm

END PLATE IN BENDING

$k_5 = \dfrac{0.9 l_{eff} t_p^{\ 3}}{m^3}$ = (0.9 x 100.00 x 12^3)/36.7^3 = **3.15 mm**

l_{eff} = smallest value of effective lengths (Table 6.6)

1) $\min[2\pi m_x\,; 4m_x + 1.25e_x]$

= min[2 x 3.14 x 32.1 ; 4 x 32.1 + 1.25 x 50] = min[201.59 ; 190.90]

2) $\min[\pi m_x + w\,; e + 2m_x + 0.625e_x]$

= min[3.14 x 32.1 + 90 ; 55 + 2 x 32.1 + 0.625 x 50]

= min[190.79 ; 150.45]

3) $\min[\pi m_x + 2e\,; 0.5b_p]$

= min[3.14 x 32.1 + 2 x 55 ; 0.5 x 200] = min[210.79 ; 100.00]

4) $[0.5w + 2m_x + 0.625e_x]$

= [0.5 x 90 +2 x 32.1 + 0.625 x 50] = 140.45

= 100.00 mm

BOLTS IN TENSION

$k_{10} = \dfrac{1.6 A_s}{L_b}$ = (1.6 x 245)/50.7 = **7.73 mm**

Effective stiffness coefficient for Row 1:

$$k_{eff,1} = \frac{1}{\sum_r \frac{1}{k_{i,r}}} = \frac{1}{(1/5.70 + 1/51.08 + 1/3.15 + 1/7.73)} = \mathbf{1.56\ mm}$$

ROW 2:

COLUMN WEB IN TENSION

$$k_3 = \frac{0.7 b_{eff,t,wc} t_{wc}}{d_c} = (0.7 \text{ x } 168.93 \text{ x } 11.9)/246.7 = \mathbf{5.70\ mm}$$

$b_{eff,t,wc}$ = effective length of column's web depth in tension (Table 6.4)

$= \min[2\pi m ; 4m + 1.25e]$ = min[2 x 3.14 x 26.9 ; 4 x 26.9 + 1.25 x 108.4]

= min[168.93 ; 243.1]

= 168.93 mm

COLUMN FLANGE IN BENDING

$$k_4 = \frac{0.9 l_{eff} t_{fc}^{\ 3}}{m^3} = (0.9 \text{ x } 168.93 \text{ x } 18.7^3)/26.9^3 = \mathbf{51.08\ mm}$$

l_{eff} = smallest value of effective lengths of column's flange without web stiffeners (Table 6.4)

= $b_{eff,t,wc}$ = 168.93 mm

END PLATE IN BENDING

$$k_5 = \frac{0.9 l_{eff} t_p^{\ 3}}{m^3} = (0.9 \text{ x } 230.48 \text{ x } 12^3)/36.7^3 = \mathbf{7.25\ mm}$$

l_{eff} = smallest value of effective lengths (Table 6.6)

$= \min[2\pi m ; \alpha m]$ = min[2 x 3.14 x 36.7 ; 7.1 x 36.7]

= min[230.48 ; 260.57]

= 230.48 mm

BOLTS IN TENSION

$$k_{10} = \frac{1.6 A_s}{L_b} = (1.6 \text{ x } 245)/50.7 = \mathbf{7.73\ mm}$$

Effective stiffness coefficient for Row 2:

$$k_{eff,1} = \frac{1}{\sum_r \frac{1}{k_{i,r}}} = \frac{1}{(1/5.70 + 1/51.08 + 1/7.25 + 1/7.73)} = \mathbf{2.16\ mm}$$

z_{eq} = (1.56 x 434^2)/(2.16 x 334) = 407.29 mm

k_{eq} = (1.56 x 434) + (2.16 x 334)/407.29 = **3.43 mm**

<u>STIFFNESS OF THE JOINT</u>

Initial Stiffness:

$$S_{j,ini} = \frac{Ez^2}{\mu \sum_i \frac{1}{k_i}} = \frac{205000\,x\,388^2}{1.0(1/4.23 + 1/6.66 + 1/3.43)} \text{ x } 10^{-6} = \mathbf{45512\ kNm}$$

Secant Stiffness:

$$S_j = \frac{S_{j,ini}}{2} = 45512/2 = \mathbf{22756\ kNm}$$

3.8 Concluding Remarks

In general, the moment capacity of standard partial strength connections with TWP sections can be determined analytically using the procedures as described by SCI. The moment capacities obtained were then tabulated which included the panel shear capacities, and the checks for tension and compression zones for columns. Altogether there were six standardised capacity tables generated for flush endplate connections and eight standardised capacity tables generated for extended endplate connections as shown in Chapter 6 along with the details discussion.

The initial rotational stiffness of a connection, which is very important especially in determining whether the connection is pinned, rigid or semi-rigid, can also be determined analytically as outlined in detail by EC 3. However, the steps involved in the calculation were quite complex and cumbersome.

CHAPTER IV

PARAMETRIC STUDY ON ANALYSIS AND DESIGN OF SEMI-CONTINUOUS MULTI-STOREY UNBRACED STEEL FRAMES USING WIND MOMENT METHOD

4.1 General

Multi-storey frames may be divided into two distinct categories for the purpose of design: sway and non-sway frames. In BS 5950-1: 2000 (BSI, 2000), a multi-storey frame may be classified as "non-sway" if its sway deformation is small for the resulting secondary forces and moments to be negligible. In Eurocode 3 (DD ENV 1993-1-1: 1992 and BS EN 1993-1-8: 2005), the frame is classified as braced when the bracing system reduces the horizontal displacement by at least 80%. A steel frame which does not satisfy the criterion for a braced frame is classified as unbraced. For an unbraced frame, the main consideration is to limit sway, to control the inter-storey drifts and to avoid premature collapse by frame instability. To meet this requirement, it is usual to rely on the bending resistance and stiffness of the connections to resist horizontal loads. For ultimate limit state, it is important to make sure that the structural members are capable of transferring the factored loads to the columns and down to the foundations. In practice, unbraced frame usually designed by assuming that the connections are rigid in order to provide adequate stiffness to resist horizontal loads. In rigid frame analysis and design, the internal moments and forces are distributed among the columns and beams according to their stiffness coefficients (K). The stiffness coefficient is a function of the length (L), the second moment of area (I) and the modulus of elasticity (E).

One alternative, a simple design method, termed the wind moment method is often used in the U.K. for the design of unbraced frame. Wind moment method, also known as wind connection method, assuming that the structure is statically determinate and allow the structure to be analyzed using manual techniques. The designed method proposed in wind moment method assuming that the connections act as pins under gravity load and rigid under horizontal loads. These assumptions allow the beams and columns to be designed using simple construction methods and sway deflections are calculated using the simple graphical method assuming connections is rigid. As the beam in wind moment design usually governed though by mid-span gravity moment, the connections are designed to a lower moment than the beam sections and are therefore termed as partial strength in the context of Eurocode 3 Part 1.1. The partial strength connections proportioned in wind moment design have some degree of strength and stiffness, but insufficient to develop full continuity as in rigid connection. The standard tables for these types of connections have been produced by the Steel Construction Institute (Joints in steel construction: Moment connections). Both rigid and partial strength joints can be applied in wind moment design where the controlling parameter is the sway limit at serviceability limit state. The calculated rigid frame deflections will be increased by 50% as an approximate allowance for partial strength connections as suggested by SCI. The main advantage of the wind moment method is its simplicity. The frame is treated as statically determinate, thus the internal moments and forces are not dependent on the relative stiffness of the frame members. The need of iterative analysis and design procedure is therefore avoided.

4.2 Range of Application

The range of the study is for two and four bays with heights of two, four, six and eight storeys. In recognition of unlikelihood of the frame consisting of only one longitudinal bay, the minimum number of bays in the out of plane framing was taken as two. Each longitudinal bay was assumed to be 6m in length and all beams assumed to be fully restrained. The limitations on frame dimensions conformed to those specified in the existing guide for wind moment design. The summary of the frame dimension and loading are given in Table 4.1 and Table 4.2.

For ultimate limit states, all loadings are in accordance with the values suggested in wind moment design for unbraced frame. Two cases were considered in the design; minimum wind load combined with maximum gravity load and maximum wind load combined with the minimum gravity load, by choosing appropriate load values and column lengths. Basic wind speeds were taken as the hourly mean speed estimated to be exceeded on average once in 50 years. Wind forces were calculated in accordance with BS6399-2: 1997. Wind forces were considered as horizontal point loads acting on the windward external columns at each floor level. In design, account was taken of the compressive axial forces in the leeward columns, contributed by the horizontal wind. No account was taken of wind uplift on the roof, as this would relieve the compressive axial forces in the columns.

For serviceability limit states, the sway-deflection limit is taken as h_T / 450 for partial strength connections and h_T / 300 for full strength connections, where h_T is the total height of the multi-storey frame.

Table 4.1: Frame dimension

Scope		Description
Number of bay		2 and 4
Number of storey		2, 4, 6, 8 storeys
Bay width		6m and 9m
Longitudinal Bay width		6m
Storey height:	Ground	5m
	Elevated	4m

Table 4.2: Loading

	Case 1 Minimum wind and maximum gravity	Case 2 Maximum wind and minimum gravity
Gravity Load:		
Dead Load (DL) - Roof	4.00 kN/m^2	3.75 kN/m^2
Floor	5.00 kN/m^2	3.50 kN/m^2
Live Load (LL) - Roof	1.50 kN/m^2	1.50 kN/m^2
Floor	7.50 kN/m^2	4.00 kN/m^2
Wind load: basic wind	20 m/s	28 m/s

speed

In this study, frames were analyzed under three load combinations as follows:

1. 1.4 dead load plus 1.6 imposed load plus factored notional horizontal force
2. 1.2 dead load plus 1.2 imposed load plus 1.2 wind load
3. 1.4 dead load plus 1.4 wind load

In structural section design, the universal beam sections were used for horizontal members and universal column sections were used for verticals members. All sections were orientated such that loads in the plane of the frame tend to cause bending about the major axis for major axis frame and bending about the minor axis for minor axis frame. All columns are rigidly connected to foundations.

4.3 Wind Moment Method

4.3.1 Portal Method of Analysis

In wind moment method analysis, the frame reaction is calculated base on portal method. Referring to Figure 4.1, each bay of the multi-storey frame is assumed to act as a single portal and the horizontal load:

$$H_1 = \frac{L_1 w_1}{\sum L}$$

The horizontal force is assumed to be divided equally between the two columns on a portion of one bay, thus the shear force:

$$S_1 = \frac{H_1}{2}$$

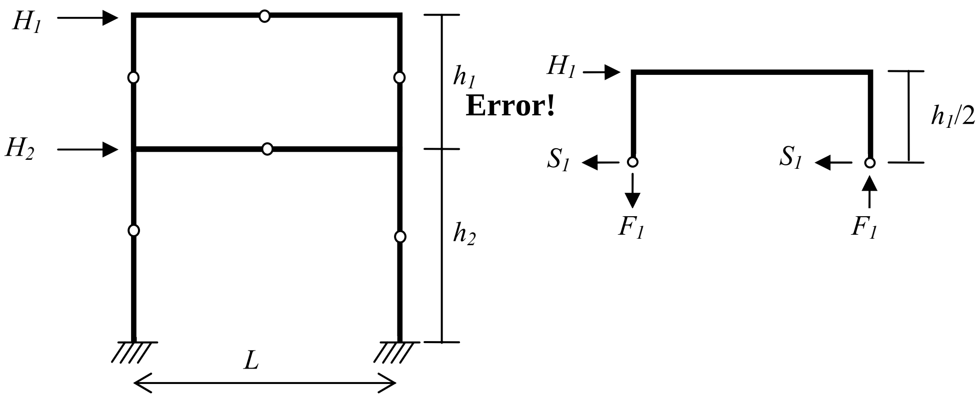


Figure 4.1: Portal Method of Analysis

The vertical forces and moments at the column are therefore:

$$F_1 = \frac{H_1 h_1}{2L}$$

$$M_1 = \frac{S_1 h_1}{2}$$

The internal moment at each end of the beam equals to $M_1 + M_2$. The shear force for the beam is given by:

$$V_1 = \frac{M_1}{L/2} \text{ and } V_2 = \frac{(M_1 + M_2)}{L/2}$$

The portal method analysis simplified the calculation procedures for moment distribution due to horizontal forces. Moments and shear forces obtained from the analysis then combined with the moments calculated from the gravity load. These values then used to design the frames with specific load combination.

4.3.2 Design of Major Axis Frame

In the design of major axis frame, it is assuming that the frames are effectively braced at the roof and each floor level to prevent sway about the minor axis of the columns but are unbraced about the major axes columns (see Figure 4.2). The prevention of sway about the minor axes can be achieved by cross bracing or by other systems such as attachment to a rigid core.

For beam design, the moment capacity and classification for beam section are in accordance to Steelwork Design Guide to BS 5950: Part 1: 2000 publication of Steel Construction Institute. The floor details are to be such that the beam is effectively restrained against lateral and lateral-torsional buckling. Therefore, no check was done for lateral-torsional buckling. However, when the wind speed is too high while the design of beam is controlled by the moment generated by wind, lateral-torsional buckling should be checked. In this case, *m* is taken to be equal to 0.44 due to the double curvature effect. The studies have shown thought that lateral-torsional buckling is not critical.

In column design, the moments in the columns due to vertical load alone are given in the algebraic sum of 10% end restraint moments from the beams and nominal moments due to eccentricity of the beam reactions. Additional internal moments and forces due to horizontal forces (wind load and notional horizontal force) are calculated from the portal method analysis as proposed in wind moment design. The graphical method of Woods is applied to determine the sway-deflection for frames in wind moment design. In wind moment method, the frames were analysed as an elastic rigid-jointed frame, therefore the sway-deflection limit are:

Sway-deflection limit, $\Delta = \dfrac{h_T}{300}$ mm

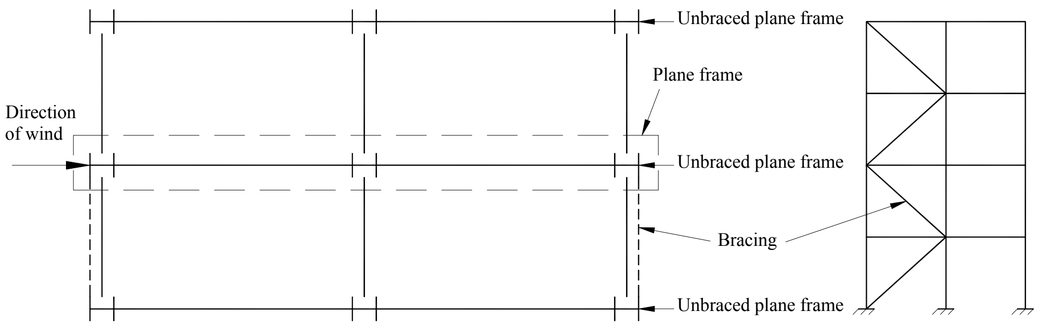


Figure 4.2: Plane frames braced against out-of-plane sway.

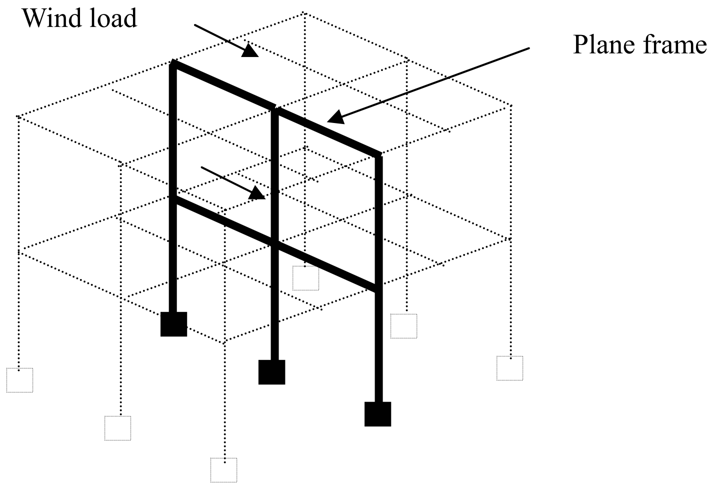


Figure 4.3: Typical plane frame of two bays two-storey structure.

4.3.3 Design of Beams

The internal moments and forces in the design of beam are in accordance with the requirements of BS 5950-1: 2000 for simple construction. The beams are assumed to be simply supported and the design normally governed by the maximum sagging moment at the mid length of the beam. Sections used in the design of beam are either plastic or compact sections which in accordance with the recommendation in wind moment method. The design moment (M) should be smaller than the plastic moment resistance (M_c) of the section in order to provide sufficient rotational restraint to the column. The plastic moment resistance is an equation of design strength of the steel (p_y) multiplied by the plastic modulus of the beam section (S). For parts of beams that are effectively unrestrained according to BS 5950-1: 2000, the equivalent uniform moment ($\overline{M}$) should be checked with the lateral torsional buckling resistance moment (M_b). The calculation for equivalent uniform moment is in accordance with the BS 5950-1: 2000 Clause 4.3.5.3.

4.3.4 Design of columns

The sections in the design of columns should be classified as plastic or compact sections. As the frame is unbraced about the major axis but braced on minor axis, the columns are designed to have buckling about the major axis. The frame is a sway frame on major axis; therefore the effective length of the column should be taken as 1.5*L* for major axis and 1.0*L* for minor axis where *L* is the height of the column. The compression resistance and buckling resistance moment are calculated based on the Steelwork Design Guide as mentioned above. The buckling resistance moment is that for "simple" design as stated in BS 5950-1: 2000 Clause 4.7.7. As for column design, the following relationship needs to be satisfied:

$$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} + \frac{M_y}{P_y Z_y} \leq 1.0$$

where

F_c is the applied axial load due to vertical loading, or a combination of vertical loads and wind loads

M_x is the applied moment about the major axis due to appropriate combination of vertical loading, notional horizontal forces and wind loads

M_y is the applied moment about the minor axis due to appropriate combination of vertical loading

P_y is the design strength of steel

Z_y is the elastic modulus about the minor axis

P_c is the compressive resistance

M_{bs} is the lateral torsional buckling resistance moment for simple design.

4.3.5 Designs at Serviceability Limit State

The designs at the serviceability limit state consist of various requirements in BS 5950-1: 2000, but horizontal deflection is the only consideration in wind moment method. The deflection limits given in BS 5950-1: 2000 with the purpose to ensure that the resistance and in-service performance of the structure are not impaired. A sensible limit on horizontal deflection for low-rise frames is height / 300. The vertical deflections of beams should generally be calculated using unfactored imposed loads assuming that the beams are simply supported. The limits on imposed load deflection should generally be in accordance with BS 5950-1: 2000 span / 360 for beams carrying plaster or other brittle finishes.

The frames are checked for sway using the unfactored wind loads. Full analysis of frames taking into account connection flexibility shows that partial strength connections deflect significantly more under horizontal loading than those with fully rigid connections. This increased sway can be allowed for by the designer by means of a simple amplification factor applied to the sway deflection. The simple graphical Wood method as recommended in wind moment method sufficiently calculate rigid frame deflections, without taking into account the sway due to asymmetric vertical loads. The calculated rigid frame deflections then increased by 50% as an approximate allowance for the flexibility of partial strength connections. If the deflections are unacceptable, the size of the member will be increased to obtain the required stiffness for frame stability.

4.4 Wind Forces

Basic wind speeds were taken as the hourly mean speed estimated to be exceeded on average once in 50 years. Wind forces were calculated in accordance with BS 6399-2: 1997, Code of practice for wind loads. The formula for site wind speed V_s for particular direction is given in BS 6399-2 clause 2.2.2 as:

$$V_s = V_b S_a S_d S_s S_p$$

Where

V_b is the basic wind speed obtained from figure 6 in BS 6399-2

S_a is the an altitude factor

S_d is a directional factor

S_s is a seasonal factor

S_p is a probability factor

The design wind speed is converted to dynamic pressure q (N/m^2) using the relationship

$q_s = 0.613V_e^2$

Where

V_e is the effective wind speed from the equation $V_e = V_sS_b$

S_b is the terrain and building factor

The wind force on a surface is then given by: $P = 0.85(\Sigma q_sC_pC_a)(1 + C_r)$ N/m^2

Where

C_p is the net pressure coefficient

C_a is the size effect factor for external pressure

C_r is the dynamic augmentation factor

Wind forces were considered as horizontal point loads acting on the windward external columns at each floor level. In design, account was taken of the compressive axial forces in the leeward columns, contributed by the horizontal wind. No account was taken of wind uplift on the roof, as this would relieve the compressive axial forces in the columns.

4.4.1 Calculation of the Minimum and Maximum Wind Pressure

A. Minimum Wind Pressure

The follow calculation of minimum wind pressure was made in accordance to BS 6399-2: 1977 (inc. Amd. 1 2002), and by using standard method (since $H < B$ division by parts is not applicable and frictional drag is neglected as having little effect):

General information:

Basic wind speed V_b = 20 m/s

Building type: 4 bay 8 storeys

Building length L = 24 m

Building width W = 24 m

Building wall height H = 33 m

Building reference height H_r = 33 m

Building type factor K_b = 1 (open plan office)

Dynamic augmentation factor $C_r = 0.038 < 0.25$, therefore BS 6399-2 can be used.

Altitude factor S_a = 1.00

Directional factor S_d = 1.00

Seasonal factor S_s = 1.00

Probability factor S_p = 1.00

Site wind speed $V_s = V_bS_aS_dS_sS_p$ = 20 m/s

Distance to sea = 100 km

Terrain = town

Terrain & building factor S_b = 1.93 (Table 4)

Effective wind spped $V_e = V_sS_b$ = 38.65 m/s

Note: Normally, either all wind directions should be checked to establish the highest effective wind speed or a conservative approach may be taken by using a value of S_d = 1.0 together with the shortest distance to sea irrespective of direction. A lower value of S_b will be obtained for sites in town by using the hybrid approach.

Dynamic pressure $q_s = 0.613\ V_e^2$ = 915.8 N/m^2

Breadth B = 24 m

In-wind depth D = 24 m

Ratio B/D = 1

Span ratio = D/H = 0.727 ≤ 1

Net pressure coefficient $C_p = C_{pe} - C_{pi} = 0.8 - (-0.3) = 1.1$

Diagonal dimension, a = 10 x $\sqrt[3]{24x33x24}$ = 40.804 m

Size effect factor for external pressure C_a = 0.842 (Figure 4.4)

For net wind load to building $P = 0.85\ (\sum q_sC_pC_aA)(1 + C_r)$ Clause 2.1.3.6 NOTE 3

Simplifying P = <u>0.83 kN/m^2</u>

B. Maximum Wind Pressure

Using standard method (since H<B division by parts is not applicable and frictional drag is neglected as having little effect):

Basic wind speed Vb = 28 m/s

General Information:

Building type: 4 bay 8 storeys

Building length L = 24 m

Building width W = 24 m

Building wall height H = 33 m

Building reference height H_r = 33 m

Building type factor K_b = 1 (open plan office)

Dynamic augmentation factor $C_r = 0.038 < 0.25$, therefore BS 6399-2 can be used.

Altitude factor S_a = 1.00

Directional factor S_d = 1.00

Seasonal factor S_s = 1.00

Probability factor S_p = 1.00

Site wind speed $V_s = V_b S_a S_d S_s S_p$ = 28 m/s

Distance to sea = 100 km

Terrain = town

Terrain & building factor S_b = 1.87 (Table 4)

Effective wind speed $V_e = V_s S_b$ = 53.32 m/s

Note: normally either all wind directions should be checked to establish the highest effective wind speed or a conservative approach may be taken by using a value of S_d = 1.00 together with the shortest distance to sea irrespective of direction. A lower value of S_b will be obtained for sites in town by using the hybrid approach.

Dynamic pressure $q_s = 0.613\ V_e^2$ = 1691.1 N/m^2

Breadth B = 24 m

Inwind depth D = 24 m

Ratio B/D = 1

Span ratio = D/H = 0.73 ≤ 1

Net pressure coefficient $C_p = C_{pe} - C_{pi} = 0.8 - (0.3) = 1.1$

Diagonal dimensions, $a = 10 \times 3\sqrt{24 \times 33 \times 24} = 40.8$ m

Size effect factor for external pressure $C_a = 0.842$ (Figure 4)

For net wind load to building $P = 0.85\,(\Sigma q_s C_p C_a A)(1 + Cr)$ Clause 2.1.3.6 NOTE 3

Simplifying P = 1.637 kN/m^2

4.4.2 Summary

Summary of the design wind pressure (*P*) for different building height is shown in the Table below. Table 4.3 shows the wind pressure for a 20 m/s wind speed and Table 4.4 shows the wind pressure for a 28 m/s wind speed.

Table 4.3: Wind pressure for a basic wind speed of 20 m/s.

No of storey	Building height (m)	C_r	S_b	V_e m/s	q_s N/m^2	C_a	C_{pe}	C_{pi}	a	P N/m^2
2	9	0.019	1.550	30.930	586.5	0.905	0.689	0.189	15.00	465.7
4	17	0.026	1.740	34.720	739.0	0.892	0.773	0.273	20.00	689.1
6	25	0.033	1.810	36.280	806.9	0.871	0.800	0.300	27.73	772.7
8	33	0.038	1.870	37.370	856.2	0.853	0.800	0.300	35.00	803.2

Table 4.4: Wind pressure for a basic wind speed of 28 m/s.

No of storey	Building height (m)	C_r	S_b	V_e m/s	q_s N/m^2	C_a	C_{pe}	C_{pi}	a	P N/m^2
2	9	0.019	1.550	43.47	1158.4	0.905	0.689	0.189	15.00	1049.25
4	17	0.026	1.740	48.80	1459.7	0.892	0.773	0.273	20.00	1390.88
6	25	0.033	1.810	50.99	1593.7	0.871	0.800	0.300	27.73	1496.84
8	33	0.038	1.870	52.52	1691.1	0.853	0.800	0.300	35.00	1565.38

4.5 Worked Example for the Design of 2-Bay 4-Storey Unbraced Steel Frame

An example of analysis and design of an unbraced steel frame using wind moment method was illustrated here. In the design, maximum Wind Load in Conjunction with Minimum Gravity Load

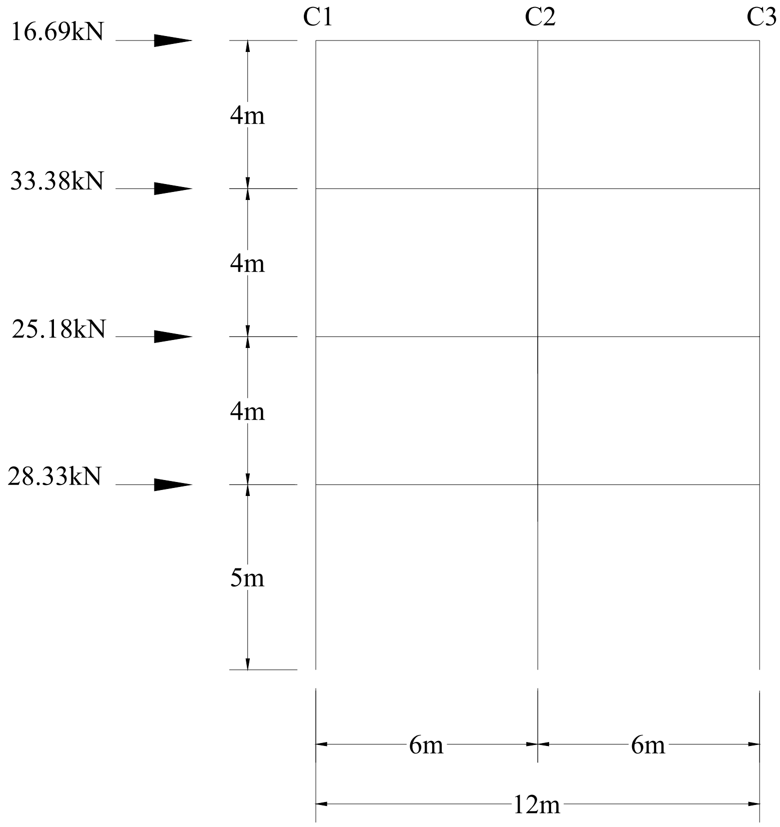
Loading

Roof	G_k	3.75kN/m^2	22.5kN/m
	Q_k	1.50kN/m^2	9.00kN/m
Floor	G_k	3.50kN/m^2	21.0kN/m
	Q_k	4.00kN/m^2	24.0kN/m

Notional Horizontal Force

NHF = $0.005(1.4G_k + 1.6Q_k)$
NHF on roof = 0.005 (1.4 x 22.5 + 1.6 x 9.00) x 12 = 2.75kN
NHF on floor = 0.005 (1.4 x 21.0 + 1.6 x 24.0) x 12 = 4.07kN

Wind Analysis

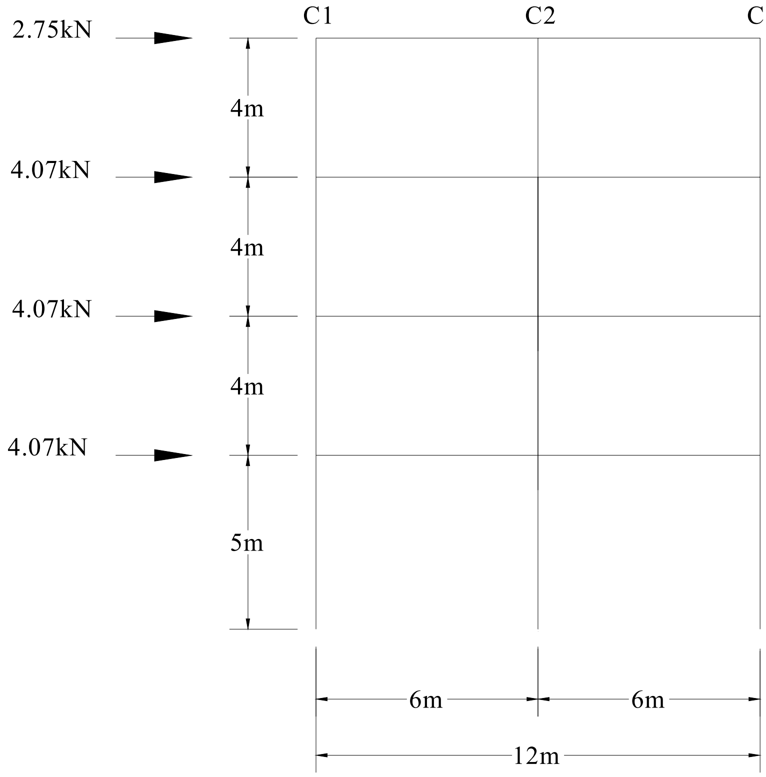


Storey	Total Wind (kN)	Shear Force in Column (kN)			Bending Moment in Column (kNm)		
		C1	C2	C3	C1	C2	C3
4	16.69	4.17	8.35	4.17	8.35	16.69	8.35
3	50.07	12.52	25.04	12.52	25.04	50.07	25.04
2	75.25	18.81	37.63	18.81	37.63	75.25	37.63
1	103.58	25.90	51.79	25.90	64.74	129.48	64.74

Floor Level	Bending Moment in Ext Column (kNm)		Bending Moment in Beam (kNm)
	Upper Column	Lower Column	
Roof	0.00	8.35	8.35
3	8.35	25.04	33.38
2	25.04	37.63	62.66
1	37.63	64.74	102.37

Storey	Moments about point of contraflexure at mid-height					Fc (kN)
	SL	h1	h2	h3	h4	
4	12	2				2.8
3	12	6	2			13.9
2	12	10	6	2		34.8
1	12	14.5	10.5	6.5	2.5	68.9

Notional Horizontal Force Analysis



Storey	Total NHF (kN)	Shear Force in Column (kN)			Bending Moment in Column (kNm)		
		C1	C2	C3	C1	C2	C3
4	2.754	0.69	1.38	0.69	1.38	2.75	1.38
3	6.822	1.71	3.41	1.71	3.41	6.82	3.41
2	10.89	2.72	5.45	2.72	5.45	10.89	5.45
1	14.958	3.74	7.48	3.74	9.35	18.70	9.35

Floor Level	Bending Moment in Ext Column (kNm)		Bending Moment in Beam (kNm)
	Upper Column	Lower Column	
Roof	0	1.38	1.4
3	1.377	3.41	4.8
2	3.411	5.45	8.9
1	5.445	9.35	14.8

Storey	Moments about point of contraflexure at mid-height					Fc (kN)
	SL	h1	h2	h3	h4	
4	12	2				0.5
3	12	6	2			2.1
2	12	10	6	2		5.0
1	12	14.5	10.5	6.5	2.5	9.9

4.5.1 Beam Design

A. Roof Beam

Case 1 $1.4G_k + 1.6Q_k$ + NHL
Beam Length = 6m

Design Load, W = (1.4 x 22.5 + 1.6 x 9.00) x 6
= 275.40kN

Maximum Moment, M_x = $0.9WL/8$ = 0.9 x 275.40 x 6 / 8
= 185.90kNm

Maximum Shear Force, F_y = W / 2 = 275.40 / 2
= 137.7kN

Try 356 x 171 x 45 UB S275 steel

Section classification
t = 7mm < 16mm
T = 9.7mm < 16mm, $P_y = 275\text{N/mm}^2$
$\varepsilon = (275/275)^{0.5} = 1$

$b/T = 8.82 < 9\varepsilon$
$d/t = 44.5 < 80\varepsilon$, Class 1 Plastic

Moment capacity, $0.9M_{cx}$ = 0.9 x p_y x S_{xx} = 0.9 x 275 x 775 / 1000
= 191.81kNm
Shear capacity, P_V = $0.6p_yA_v$ = 0.6 x 275 x 351.4 x 7 / 1000
= 405.9kN

$0.9M_{cx} > M_x$, $P_v > F_y$ Section is acceptable.

Case 2 1.2 $(G_k + Q_k + W_k)$

Design moment at end of beam due to wind, M_x = 1.2 x 8.35
= 10.01kNm

$M_{cx} > M_x$ This load combination is not critical.

Case 3 1.4 $(G_k + W_k)$

Design moment at end of beam due to wind, M_x = 1.4 x 8.35
= 11.68kNm

$M_{cx} > M_x$ This load combination is not critical.

Serviceability limit state

Design imposed load, W = 9.0 x 6.0 = 54kN

Second moment of area, $I = 12100\text{cm}^4$

Deflection of beam, $\delta \quad = \dfrac{5WL^3}{384EI} = \dfrac{5 \times 54 \times 6000^3}{384 \times 205 \times 12100 \times 10^4} = 6.12\text{mm}$

Deflection limit $= L/360 = 16.67\text{mm} > \delta$ OK!

B. Floor Beam - 3rd Floor

Case 1 $1.4G_k + 1.6Q_k$ + NHL

Beam Length = 6m

Design Load, W = (1.4 x 21.00 + 1.6 x 24.00) x 6
= 406.80kN

Maximum Moment, M_x = $0.9WL/8$ = 0.9 x 406.80 x 6 / 8
= 274.59kNm

Maximum Shear Force, F_y = W / 2 = 406.80 / 2
= 203.40kN

Try 406 x 178 x 60 UB S275 steel

Section classification

t = 7.9mm < 16mm

T = 12.8mm < 16mm, $P_y = 275\text{N/mm}^2$

$\varepsilon = (275/275)^{0.5} = 1$

b/T = 6.95 < 9ε

d/t = 45.6 < 80ε, Class 1 Plastic

Moment capacity, $0.9M_{cx}$ = 0.9 x p_y x S_{xx} = 0.9 x 275 x 1200 / 1000
= 297kNm

Shear capacity, P_V = $0.6p_yA_v$ = 0.6 x 275 x 406.4 x 7.9 / 1000
= 530kN

$0.9M_{cx} > M_x$, $P_v > F_y$ Section is acceptable.

Case 2 $1.2\ (G_k + Q_k + W_k)$

Design moment at end of beam due to wind, M_x = 1.2 x 33.38
= 40.06kNm

$M_{cx} > M_x$ This load combination is not critical.

Case 3 $1.4\ (G_k + W_k)$

Design moment at end of beam due to wind, M_x = 1.4 x 33.38

= 46.73kNm

$M_{cx} > M_x$ This load combination is not critical.

Serviceability limit state

Design imposed load, W = 24.0 x 6.0 = 144kN

Second moment of area, I = 21600cm^4

Deflection of beam, δ $= \dfrac{5WL^3}{384EI} = \dfrac{5 \times 144 \times 6000^3}{384 \times 205 \times 21600 \times 10^4}$ = 9.15mm

Deflection limit = L/360 = 16.67mm > δ OK!

C. Floor Beam - 1st and 2nd Floor

Case 1 $1.4G_k + 1.6Q_k$ + NHL

Beam Length = 6m

Design Load, W = (1.4 x 21.00 + 1.6 x 24.00) x 6
= 406.80kN

Maximum Moment, M_x = 0.9WL/8 = 0.9 x 406.80 x 6 / 8
= 274.59kNm

Maximum Shear Force, F_y = W / 2 = 406.80 / 2
= 203.40kN

Try 457 x 152 x 67 UB S275 steel

Section classification
t = 9mm < 16mm
T = 15mm < 16mm, P_y = 275N/mm^2
$\varepsilon = (275/275)^{0.5} = 1$

b/T = 5.13 < 9ε
d/t = 45.3 < 80ε, Class 1 Plastic

Moment capacity, 0.9M_{cx} = 0.9 x p_y x S_{xx} = 0.9 x 275 x 1450 / 1000
= 360kNm

Shear capacity, P_V = 0.6p_yA_v = 0.6 x 275 x 458 x 9 / 1000
= 680kN

0.9$M_{cx} > M_x$, $P_v > F_y$ Section is acceptable.

Case 2 1.2 $(G_k + Q_k + W_k)$

Design moment at end of beam due to wind, M_x = 1.2 x 102.37

= 122.84kNm

$M_{cx} > M_x$ This load combination is not critical.

Case 3 1.4 ($G_k + W_k$)

Design moment at end of beam due to wind, M_x = 1.4 x 102.37
= 143.31kNm

$M_{cx} > M_x$ This load combination is not critical.

Serviceability limit state

Design imposed load, W = 24.0 x 6.0 = 144kN

Second moment of area, I = 28900cm^4

$$\text{Deflection of beam, } \delta = \frac{5WL^3}{384EI} = \frac{5 \times 144 \times 6000^3}{384 \times 205 \times 28900 \times 10^4} = 6.84\text{mm}$$

Deflection limit = L/360 = 16.67mm > δ OK!

4.5.2 Column Design Using UC

Storey	Beam reaction		10% restraint moment		Moment due to horizontal loads			
	Dead	Imposed	Dead	Imposed	Notional loads		Wind loads	
	(kN/m)	(kN/m)	(kNm)	(kNm)	External	Internal	External	Internal
4	22.5	9.0	10.1	4.1	1.38	2.75	8.35	16.69
3	21.0	24.0	9.5	10.8	3.41	6.82	25.04	50.07
2	21.0	24.0	9.5	10.8	5.45	10.89	37.63	75.25
1	21.0	24.0	9.5	10.8	9.35	18.70	64.74	129.48

The values for the 10% restraint moment are calculated from the unfactored floor loads.

Dead load = 0.1 x 21 x 6^2 / 8 = 9.5 kNm
Imposed load = 0.1 x 24 x 6^2 / 8 = 10.8 kNm

A. Internal Column Design

The columns will be spliced above the second storey floor beams, where change in section size may take place. Therefore, design calculations will be required for storey 3 and 1.

Storey	Loading (kN)		Sw of column (kN)	Total Load		Reduction in imposed load (kN)	Reduced imposed load (kN)
	Dead (kN/m)	Imposed (kN/m)		Dead (kN)	Imposed (kN)		
4	22.5	9.0	4	139	54	0	54
3	21.0	24.0	4	268	198	10%	178
2	21.0	24.0	6	400	342	20%	274
1	21.0	24.0	7	533	486	30%	340

The reduction in imposed load for the number of storeys carried is given by BS 6399-1: Table 2.

Storey 3

Case 1 $1.4G_k + 1.6Q_k$ + NHL

Design load at ultimate limit state: $F_c = 1.4 \times 268 + 1.6 \times 178 = 661$kN
Design moment at ultimate limit state: $M_x = 6.82$kNm
(due to notional loads)

Moments due to eccentric reactions and the 10% restraint moment balance and produce no net moment about the major axis. By inspection, pattern imposed load (i.e. omitting imposed load on one beam at third floor level) will not be critical.

$L = 4$m
$L_{ex} = 1.5\,L = 1.5 \times 4 = 6$ m
$L_{ey} = 1.0\,L = 1.0 \times 4 = 4$ m

Try 254 x 254 x 89 UC S275 steel

Section classification

$T = 17.3\text{mm} > 16\text{mm}$, $P_y = 265\text{N/mm}^2$
$\varepsilon = (275/265)^{0.5} = 1.02$

b/T = 7.41 < 9ε
d/t = 19.40 < 40ε, Class 1 Plastic

Second moment of area, $I_{xx} = 14300\text{cm}^4$
Second moment of area, $I_{yy} = 4860\text{cm}^4$

Radius of gyration, $r_{xx} = 11.20$cm
Radius of gyration, $r_{yy} = 6.55$cm

Gross area of section, $A_g = 113.0\text{cm}^2$

$\lambda_{xx} = L_{ex} / r_{xx} = 6000 / 112 = 53.6$
$\lambda_{yy} = L_{ey} / r_{yy} = 4000 / 65.5 = 61.1$

E = 205000 N/mm^2

$\lambda_0 = 0.2\ (\pi^2\ E\ /\ p_y)^{\ 0.5} = 17.5$

$a_x = 3.5$, $a_y = 5.5$

$n_x = a_x\ (\lambda_{xx} - \lambda_0)\ /1000 = 0.126 > 0$
$n_y = a_y\ (\lambda_{yy} - \lambda_0)\ /1000 = 0.240 > 0$

$PE_x = \pi^2\ E\ /\ \lambda_{xx}^{\ 2} = 705.0\ N/mm^2$
$PE_y = \pi^2\ E\ /\ \lambda_{yy}^{\ 2} = 542.5\ N/mm^2$

$Ø_{xx} = (p_y + (n_{xx} + 1)\ PE_x)\ /2 = 529.5$
$Ø_{yy} = (p_y + (n_{yy} + 1)\ PE_y)\ /2 = 468.8$

$$p_{cx} = \frac{PE_x p_y}{\phi_{xx} + (\phi_{xx}^{\ 2} - PE_x p_y)^{0.5}} = 223.6N/mm^2$$

$$p_{cy} = \frac{PE_y p_y}{\phi_{yy} + (\phi_{yy}^{\ 2} - PE_y p_y)^{0.5}} = 193.1N/mm^2$$

$P_{cx} = A_g p_{cx} = 2527\ kN$
$P_{cy} = A_g p_{cy} = 2182.1\ kN$

$\lambda_{LT} = 0.5L\ /\ r_{yy} = 0.5 \text{ x } 4000\ /\ 65.5 = 30.53$

$\lambda_{L0} = 0.4\ (\pi^2\ E\ /\ p_y)^{\ 0.5} = 35$

$n_{LT} = a_{LT}\ (\lambda_{LT} - \lambda_{L0})\ /1000 = -0.01 < 0$, the n_{LT} is taken as 0

$PE = \pi^2\ E\ /\ \lambda_{LT}^{\ 2} = 2170.1\ N/mm^2$

$Ø_{LT} = (p_y + (n_{LT} + 1)\ PE)\ /2 = 1217.5$

$$p_b = \frac{PE\ \ p_y}{\phi_{LT} + (\phi_{LT}^{\ 2} - PE\ \ p_y)^{0.5}} = 265N/mm^2$$

Plastic Modulus, S_{xx} = 1220 cm^3

$M_{bs} = p_b S_{xx}$ = 323.3 kNm

$$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} = \frac{661}{2182.1} + \frac{6.82}{323.3} = 0.32 < 1 \qquad OK!$$

<u>Case 2 1.2 ($G_k + Q_k + W_k$)</u>

Design load at ultimate limit state: F_c = 1.2 x 268 + 1.2 x 178 = 535.74kN
Design moment at ultimate limit state: M_x = 1.2 x 50.07 = 60.09kNm
(due to wind loads)

Moments due to eccentric reactions and the 10% restraint moment balance and produce no net moment about the major axis.

$$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} = \frac{535.74}{2182.1} + \frac{60.09}{323.3} = 0.43 < 1 \qquad \textit{OK!}$$

Case 3 1.4 (G_k + W_k)
Design load at ultimate limit state: F_c = 1.4 x 268 = 376kN
Design moment at ultimate limit state: M_x = 1.4 x 50.07 = 70.10kNm
(due to wind loads)

$$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} = \frac{376}{2182.1} + \frac{70.10}{323.3} = 0.39 < 1 \qquad \textit{OK!}$$

Use 254 x 254 x 89 UC S275 steel

Storey 1

Case 1 1.4G_k + 1.6Q_k + NHL

Design load at ultimate limit state: F_c = 1.4 x 533 + 1.6 x 340 = 1290kN
Design moment at ultimate limit state: M_x = 18.7kNm
(due to notional loads)

Moments due to eccentric reactions and the 10% restraint moment balance and produce no net moment about the major axis. By inspection, pattern imposed load (i.e. omitting imposed load on one beam at third floor level) will not be critical.

L = 5m
L_{ex} = 1.5 L = 1.5 x 5 = 7.5 m
L_{ey} = 1.0 L = 1.0 x 5 = 5.0 m

Try 305 x 305 x 137 UC S275 steel

Section classification

T = 21.7mm > 16mm, P_y = 265N/mm^2
$\varepsilon = (275/265)^{0.5} = 1.02$

b/T = 7.12 < 9ε
d/t = 17.9 < 40ε, Class 1 Plastic

Second moment of area, I_{xx} = 32800cm^4
Second moment of area, I_{yy} = 10700cm^4

Radius of gyration, r_{xx} = 13.70cm

Radius of gyration, r_{yy} = 7.83cm

Gross area of section, A_g = 174cm^2

$\lambda_{xx} = L_{ex} / r_{xx}$ = 7500 / 137 = 54.7
$\lambda_{yy} = L_{ey} / r_{yy}$ = 5000 / 78.3 = 63.9

E = 205000 N/mm^2

$\lambda_0 = 0.2\ (\pi^2\ E / p_y)^{0.5} = 17.5$

$a_x = 3.5$, $a_y = 5.5$

$n_x = a_x\ (\lambda_{xx} - \lambda_0)\ /1000 = 0.130 > 0$
$n_y = a_y\ (\lambda_{yy} - \lambda_0)\ /1000 = 0.255 > 0$

$PE_x = \pi^2\ E / \lambda_{xx}^2 = 675.1\ N/mm^2$
$PE_y = \pi^2\ E / \lambda_{yy}^2 = 496.2\ N/mm^2$

$Ø_{xx} = (p_y + (n_{xx} + 1)\ PE_x)\ /2 = 514.1$
$Ø_{yy} = (p_y + (n_{yy} + 1)\ PE_y)\ /2 = 443.9$

$$p_{cx} = \frac{PE_x p_y}{\phi_{xx} + (\phi_{xx}{}^2 - PE_x p_y)^{0.5}} = 221.9N/mm^2$$

$$p_{cy} = \frac{PE_y p_y}{\phi_{yy} + (\phi_{yy}{}^2 - PE_y p_y)^{0.5}} = 187.9N/mm^2$$

$P_{cx} = A_g p_{cx} = 3860.8\ kN$
$P_{cy} = A_g p_{cy} = 3269.0\ kN$

$\lambda_{LT} = 0.5L / r_{yy}$ = 0.5 x 4000 / 78.3 = 31.93

$\lambda_{L0} = 0.4\ (\pi^2\ E / p_y)^{0.5} = 35$

$n_{LT} = a_{LT}\ (\lambda_{LT} - \lambda_{L0})\ /1000 = -0.01 < 0$, the n_{LT} is taken as 0

$PE = \pi^2\ E / \lambda_{LT}^2 = 1984.7\ N/mm^2$

$Ø_{LT} = (p_y + (n_{LT} + 1)\ PE)\ /2 = 1124.9$

$$p_b = \frac{PE\ p_y}{\phi_{LT} + (\phi_{LT}{}^2 - PE\ p_y)^{0.5}} = 265N/mm^2$$

Plastic Modulus, S_{xx} = 2300 cm^3

$M_{bs} = p_b S_{xx}$ = 609.5 kNm

$$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} = \frac{1290}{3860.8} + \frac{18.70}{609.5} = 0.43 < 1 \qquad OK!$$

Case 2 1.2 ($G_k + Q_k + W_k$)

Design load at ultimate limit state: F_c = 1.2 x 533 + 1.2 x 340 = 1048kN
Design moment at ultimate limit state: M_x = 1.2 x 129.48 = 155.38kNm
(due to wind loads)

Moments due to eccentric reactions and the 10% restraint moment balance and produce no net moment about the major axis.

$$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} = \frac{1048}{3860.8} + \frac{155.38}{609.5} = 0.58 < 1 \qquad OK!$$

Case 3 1.4 ($G_k + W_k$)

Design load at ultimate limit state: F_c = 1.4 x 533 = 746kN
Design moment at ultimate limit state: M_x = 1.4 x 129.48 = 181.27kNm
(due to wind loads)

$$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} = \frac{746}{3860.8} + \frac{181.27}{609.5} = 0.53 < 1 \qquad OK!$$

Use 305 x 305 x 137 UC S275 steel

B. External Column Design

Storey	Loading (kN)		Sw of	Total Load		Reduction in imposed load (kN)	Reduced imposed load (kN)
	Dead (kN/m)	Imposed (kN/m)	column (kN)	Dead (kN)	Imposed (kN)		
4	22.5	9.0	4	71	27	0	27
3	21.0	24.0	4	138	99	10%	89
2	21.0	24.0	5	206	171	20%	137
1	21.0	24.0	6	275	243	30%	170

Values are unfactored

The reduction in imposed load for number of storeys carried is given by BS 6388-1: Table 2.

The values for the 10% restraint moment are calculated from the unfactored floor loads. (The moments due to partial fixity of the beam ends)

Dead load $= 0.1 \times 21 \times 6^2 / 8 = 9.5$ kNm

Imposed load = $0.1 \times 24 \times 6^2 / 8 = 10.8$ kNm

Storey 3

Case 1 $1.4G_k + 1.6Q_k$ + NHL

Design load at ultimate limit state: $F_c = 1.4 \times 138 + 1.6 \times 89 = 335$kN
Design moment at ultimate limit state: M_x

Eccentricity moment (1.4 x 21 + 1.6 x 24) (6 / 2) (0.1 + 0.260 / 2)	= 47.00 kNm
10 % moment (1.4 x 9.5 + 1.6 x 10.8)	= 31.00 kNm
Total	= 78.00 kNm
Divide moment equally between upper and lower column lengths	= 39.00 kNm
Notional horizontal loads	= 3.41 kNm
Total design moment M_x	= 42.41 kNm

By inspection, pattern imposed load (i.e. omitting imposed load on one beam at third floor level) will not be critical.

$L = 4$m
$L_{ex} = 1.5$ L $= 1.5 \times 4 = 6$ m
$L_{ey} = 1.0$ L $= 1.0 \times 4 = 4$ m

Try 254 x 254 x 89 UC S275 steel

Section classification

$T = 17.3\text{mm} > 16\text{mm}$, $P_y = 265\text{N/mm}^2$
$\varepsilon = (275/265)^{0.5} = 1.02$

b/T = 7.41 < 9ε
d/t = 19.40 < 40ε, Class 1 Plastic

Second moment of area, $I_{xx} = 14300\text{cm}^4$
Second moment of area, $I_{yy} = 4860\text{cm}^4$

Radius of gyration, $r_{xx} = 11.20$cm
Radius of gyration, $r_{yy} = 6.55$cm

Gross area of section, $A_g = 113\text{cm}^2$

$\lambda_{xx} = L_{ex} / r_{xx} = 6000 / 112 = 53.6$
$\lambda_{yy} = L_{ey} / r_{yy} = 4000 / 65.5 = 61.1$

$E = 205000$ N/mm^2

$\lambda_0 = 0.2\ (\pi^2\ E / p_y)^{0.5} = 17.5$

$a_x = 3.5$, $a_y = 5.5$

$n_x = a_x\ (\lambda_{xx} - \lambda_0)\ /1000 = 0.126 > 0$
$n_y = a_y\ (\lambda_{yy} - \lambda_0)\ /1000 = 0.240 > 0$

$PE_x = \pi^2\ E\ /\ \lambda_{xx}^2 = 705.0\ N/mm^2$
$PE_y = \pi^2\ E\ /\ \lambda_{yy}^2 = 542.5\ N/mm^2$

$Ø_{xx} = (p_y + (n_{xx} + 1)\ PE_x)\ /2 = 529.5$
$Ø_{yy} = (p_y + (n_{yy} + 1)\ PE_y)\ /2 = 468.8$

$$p_{cx} = \frac{PE_x p_y}{\phi_{xx} + (\phi_{xx}^2 - PE_x p_y)^{0.5}} = 223.6 N/mm^2$$

$$p_{cy} = \frac{PE_y p_y}{\phi_{yy} + (\phi_{yy}^2 - PE_y p_y)^{0.5}} = 193.1 N/mm^2$$

$P_{cx} = A_g p_{cx} = 2527kN$
$P_{cy} = A_g p_{cy} = 2182kN$

$\lambda_{LT} = 0.5L\ /\ r_{yy} = 0.5 \text{ x } 4000\ /\ 65.5 = 30.53$

$\lambda_{L0} = 0.4\ (\pi^2\ E\ /\ p_y)^{0.5} = 35$

$n_{LT} = a_{LT}\ (\lambda_{LT} - \lambda_{L0})\ /1000 = -0.01 < 0$, the n_{LT} is taken as 0

$PE = \pi^2\ E\ /\ \lambda_{LT}^2 = 2170\ N/mm^2$

$Ø_{LT} = (p_y + (n_{LT} + 1)\ PE)\ /2 = 1218$

$$p_b = \frac{PE\ p_y}{\phi_{LT} + (\phi_{LT}^2 - PE\ p_y)^{0.5}} = 265 N/mm^2$$

Plastic Modulus, $S_{xx} = 1220\ \text{cm}^3$

$M_{bs} = p_b S_{xx} = 323.3$ kNm

$$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} = \frac{335}{2182} + \frac{42.41}{323.3} = 0.28 < 1 \qquad OK!$$

<u>Case 2 1.2 ($G_k + Q_k + W_k$)</u>

Design load at ultimate limit state: $F_c = 1.2 \text{ x } 138 + 1.2 \text{ x } 89 = 272$kN
Design moment at ultimate limit state: M_x

Eccentricity moment (1.2 x 21 + 1.2 x 24) (6 / 2) (0.1 + 0.260 / 2)	= 47.00 kNm
10 % moment (1.2 x 9.5 + 1.2 x 10.8)	= 31.00 kNm
Total	= 78.00 kNm
Divide moment equally between upper and lower column lengths	= 39.00 kNm

Wind loads (1.2 x 25.04)	= 30.05 kNm
Total design moment M_x	= 69.05 kNm

Moments due to eccentric reactions and the 10% restraint moment balance and produce no net moment about the major axis.

$$\frac{F_c}{P_c}+\frac{M_x}{M_{bs}} = \frac{272}{2182}+\frac{69.05}{323.3} = 0.33 < 1 \qquad OK!$$

Case 3 1.4 (G_k + W_k)

Design load at ultimate limit state: F_c = 1.4 x 138 = 193kN
Design moment at ultimate limit state: M_x

Eccentricity moment (1.4 x 21) (6 / 2) (0.1 + 0.260 / 2)	= 20.00 kNm
10 % moment (1.4 x 9.5)	= 13.00 kNm
Total	= 33.00 kNm
Divide moment equally between upper and lower column lengths	= 16.50 kNm
Wind loads (1.4 x 25.04)	= 35.06 kNm
Total design moment M_x	= 51.56 kNm

$$\frac{F_c}{P_c}+\frac{M_x}{M_{bs}} = \frac{193}{2182}+\frac{51.56}{323.3} = 0.29 < 1 \qquad OK!$$

Use 254 x 254 x 89 UC S275 steel

Storey 1

Case 1 1.4G_k + 1.6Q_k + NHL

Design load at ultimate limit state: F_c = 1.4 x 275 + 1.6 x 243 = 657kN
Design moment at ultimate limit state: M_x

Eccentricity moment (1.4 x 21 + 1.6 x 24) (6 / 2) (0.1 + 0.3145 / 2)	= 52.00 kNm
10 % moment (1.4 x 9.5 + 1.6 x 10.8)	= 31.00 kNm
Total	= 83.00 kNm
Divide moment equally between upper and lower column lengths	= 41.50 kNm
Notional horizontal loads	= 9.35 kNm
Total design moment M_x	= 50.85 kNm

By inspection, pattern imposed load (i.e. omitting imposed load on one beam at third floor level) will not be critical.

L = 5m
L_{ex} = 1.5 L = 1.5 x 5 = 7.5 m
L_{ey} = 1.0 L = 1.0 x 5 = 5.0 m

Try 305 x 305 x 118 UC S275 steel

Section classification

T = 18.7mm > 16mm, P_y = 265N/mm^2
$\varepsilon = (275/265)^{0.5} = 1.02$

b/T = 8.22 < 9ε
d/t = 20.6 < 40ε, Class 1 Plastic

Second moment of area, I_{xx} = 27700cm^4
Second moment of area, I_{yy} = 9060cm^4

Radius of gyration, r_{xx} = 13.60cm
Radius of gyration, r_{yy} = 7.77cm

Gross area of section, A_g = 150cm^2

$\lambda_{xx} = L_{ex} / r_{xx} = 7500 / 136 = 55.1$
$\lambda_{yy} = L_{ey} / r_{yy} = 5000 / 77.7 = 64.4$

E = 205000 N/mm^2

$\lambda_0 = 0.2\ (\pi^2\ E / p_y)^{\ 0.5} = 17.5$

$a_x = 3.5$, $a_y = 5.5$

$n_x = a_x\ (\lambda_{xx} - \lambda_0)\ /1000 = 0.132 > 0$
$n_y = a_y\ (\lambda_{yy} - \lambda_0)\ /1000 = 0.258 > 0$

$PE_x = \pi^2\ E / \lambda_{xx}^{\ 2} = 665.3\ N/mm^2$
$PE_y = \pi^2\ E / \lambda_{yy}^{\ 2} = 488.6\ N/mm^2$

$Ø_{xx} = (p_y + (n_{xx} + 1)\ PE_x) /2 = 509$
$Ø_{yy} = (p_y + (n_{yy} + 1)\ PE_y) /2 = 439.8$

$$p_{cx} = \frac{PE_x p_y}{\phi_{xx} + (\phi_{xx}^{\ 2} - PE_x p_y)^{0.5}} = 221.3N/mm^2$$

$$p_{cy} = \frac{PE_y p_y}{\phi_{yy} + (\phi_{yy}^{\ 2} - PE_y p_y)^{0.5}} = 186.9N/mm^2$$

$P_{cx} = A_g p_{cx} = 3319\ kN$
$P_{cy} = A_g p_{cy} = 2804\ kN$

$\lambda_{LT} = 0.5L / r_{yy} = 0.5 \text{ x } 4000 / 77.7 = 32.18$

$\lambda_{L0} = 0.4\ (\pi^2\ E / p_y)^{\ 0.5} = 35$

$n_{LT} = a_{LT}\ (\lambda_{LT} - \lambda_{L0})\ /1000 = -0.01 < 0$, the n_{LT} is taken as 0

$PE = \pi^2 E / \lambda_{LT}^2 = 1954.4\ N/mm^2$

$Ø_{LT} = (p_y + (n_{LT} + 1)\ PE)\ /2 = 1109.7$

$$p_b = \frac{PE\ p_y}{\phi_{LT} + (\phi_{LT}{}^2 - PE\ p_y)^{0.5}} = 265 N/mm^2$$

Plastic Modulus, S_{xx} = 1960 cm^3

$M_{bs} = p_b S_{xx}$ = 519.4 kNm

$$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} = \frac{657}{2804} + \frac{50.85}{519} = 0.33 < 1 \qquad OK!$$

<u>Case 2 1.2 ($G_k + Q_k + W_k$)</u>

Design load at ultimate limit state: F_c = 1.2 x 275 + 1.2 x 170 = 534.33kN
Design moment at ultimate limit state: M_x

Eccentricity moment (1.2 x 21 + 1.2 x 24) (6 / 2) (0.1 + 0.3145 / 2)	= 42.00 kNm
10 % moment (1.2 x 9.5 + 1.2 x 10.8)	= 24.36 kNm
Total	= 66.36 kNm
Divide moment equally between upper and lower column lengths	= 33.18 kNm
Wind loads (1.2 x 64.74)	= 77.69 kNm
Total design moment M_x	= 110.87 kNm

Moments due to eccentric reactions and the 10% restraint moment balance and produce no net moment about the major axis.

$$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} = \frac{534.33}{2804} + \frac{110.87}{519} = 0.41 < 1 \qquad OK!$$

<u>Case 3 1.4 ($G_k + W_k$)</u>

Design load at ultimate limit state: F_c = 1.4 x 275 = 385kN
Design moment at ultimate limit state: M_x

Eccentricity moment (1.4 x 21) (6 / 2) (0.1 + 0.3145 / 2)	= 22.70 kNm
10 % moment (1.4 x 9.5)	= 13.30 kNm
Total	= 36.00 kNm
Divide moment equally between upper and lower column lengths	= 18.00 kNm
Wind loads (1.4 x 64.74)	= 90.64 kNm
Total design moment M_x	= 108.64 kNm

$$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} = \frac{385}{2804} + \frac{108.64}{519} = 0.37 < 1 \qquad OK!$$

Use 305 x 305 x 118 UC S275 steel

4.5.3 Serviceability limit state – sway due to wind

Stiffness in substitute frame

Beam stiffness

Storey	I_b (cm^4)	L_b (cm)	K_b (cm^3)	K_b (cm^3)
4	15700	600	3 x 2 x 15700 / 600	157.0
3	21600	600	3 x 2 x 21600 / 600	216.0
2	28900	600	3 x 2 x 28900 / 600	289.0
1	28900	600	3 x 2 x 28900 / 600	289.0

Column stiffness

Storey	Ext. I_c (cm^4)	Int. I_c (cm^4)	h (cm)	K_c (cm^3)	K_c (cm^3)
4	14300	14300	400	3 x 14300 / 400	107.25
3	14300	14300	400	3 x 14300 / 400	107.25
2	27700	32800	400	(2 x 27700 + 1 x 32800)/400	220.50
1	27700	328.00	500	(2 x 27700 + 1 x 32800) / 500	176.40

Joint stiffness coefficients

Storey	$k_t = \dfrac{K_c + K_u}{K_c + K_u + K_{bt}}$	k_t	$k_b = \dfrac{K_c + K_l}{K_c + K_l + K_{bb}}$	k_b
4	$\dfrac{107.25+0}{107.25+0+157}$	0.41	$\dfrac{107.25+107.25}{107.25+107.25+216}$	0.50
3	$\dfrac{107.25+107.25}{107.25+107.25+216}$	0.50	$\dfrac{107.25+220.5}{107.25+220.5+289}$	0.53
2	$\dfrac{220.5+107.25}{220.5+107.25+289}$	0.53	$\dfrac{220.5+176.40}{220.5+176.40+289}$	0.58
1	$\dfrac{176.40+220.5}{176.40+220.5+289}$	0.58	Fixed base	0.00

Sway deflection

Storey	k_t	k_b	Phi Value	F (kN)	delta (mm)	d x 1.5	Limit (h/300)	
4	0.41	0.50	2.30	16.69	2.33	3.49	13.33	OK!
3	0.50	0.53	2.60	50.07	7.90	11.84	13.33	OK!
2	0.53	0.58	2.90	75.25	6.44	9.66	13.33	OK!
1	0.58	0.00	1.80	103.58	10.74	16.11	16.67	OK!
Total					27.40	41.10	56.67	OK!

4.5.4 Steel weight comparison

4 Storey	Mass per metre	Total Length (m)	Weight (kg)
Roof beam	46	12	552
3rd floor beam	60	12	720
2nd floor beam	67	12	804
1st floor beam	67	12	804
External column (up to roof)	89	16	1424
External column (up to 2nd floor)	118	18	2124
Internal column (up to roof)	89	8	712
Internal column (up to 2nd floor)	137	9	1233
Total			8373

4.6 Parametric Study Results

Parametric study on the design of unbraced steel frames using wind moment method was discussed in the above sections. The worked example of the calculation of wind load and analysis and design of unbraced frame were given in Section 4.4.1 and Section 4.5. The results of the parametric study were listed here from Table 4.5 to Table 4.20.

Table 4.5: Wind-moment design for 2 bay 6m span frames (minimum wind with maximum gravity load) on major axis

Basic Frame Type	Total Deflection (mm)	Width of Bay (m)	Height of Column		No of Longitudinal Bay	Width of Longitudinal Bays (m)	Gravity Load (kN/m)				Basic Wind Speed (m/s)	Wind Moment Design				
							Floor		Roof			Universal Beam			Universal Column	
			Ground (m)	Elevated (m)			DL	LL	DL	LL		Floor	Roof		External	Internal
2 bay 2 storey	14.20											533x210x82	406x140x46	up to 2nd Floor	203x203x60	203x203x60
2 bay 4 storey	31.40											533x210x82	406x140x46	up to 2nd Floor	203x203x86	305x305x97
												533x210x82		2nd to 4th Floor	203x203x46	203x203x52
2 bay 6 storey	54.57											533x210x82	406x140x46	up to 2nd Floor	305x305x97	305x305x137
		6	5	4	2	6	30	45	24	9	20	533x210x82		2nd to 4th Floor	203x203x86	254x254x89
												533x210x82		4th to 6th Floor	203x203x46	203x203x52
2 bay 8 storey	78.59											533x210x82	406x140x46	up to 2nd Floor	254x254x132	356x368x177
												533x210x82		2nd to 4th Floor	305x305x118	305x305x118
												533x210x82		4th to 6th Floor	254x254x89	254x254x89
												533x210x82		6th to 8th Floor	203x203x46	203x203x52

Table 4.6: Wind-moment design for 4 bay 6m span frames (minimum wind with maximum gravity load) on major axis

Basic Frame Type	Total Deflection (mm)	Width of Bay (m)	Height of Column		No of Longitudinal Bay	Width of Longitudinal Bays (m)	Gravity Load (kN/m)				Basic Wind Speed (m/s)	Wind Moment Design				
							Floor		Roof			Universal Beam			Universal Column	
			Ground (m)	Elevated (m)			DL	LL	DL	LL		Floor	Roof		External	Internal
4 bay 2 storey	8.39											533x210x82	406x140x46	up to 2nd Floor	203x203x60	203x203x60
4 bay 4 storey	16.03											533x210x82	406x140x46	up to 2nd Floor	254x254x73	305x305x97
												533x210x82		2nd to 4th Floor	203x203x46	203x203x52
4 bay 6 storey	28.88											533x210x82	406x140x46	up to 2nd Floor	305x305x97	305x305x137
		6	5	4	2	6	30	45	24	9	20	533x210x82		2nd to 4th Floor	254x254x73	254x254x89
												533x210x82		4th to 6th Floor	203x203x46	203x203x52
4 bay 8 storey	43.56											533x210x82	406x140x46	up to 2nd Floor	305x305x118	356x368x177
												533x210x82		2nd to 4th Floor	254x254x89	305x305x118
												533x210x82		4th to 6th Floor	254x254x73	254x254x89
												533x210x82		6th to 8th Floor	203x203x46	203x203x52

Table 4.7: Wind-moment design for 2 bay 6m span frames (maximum wind with minimum gravity load) on major axis

Basic Frame Type	Total Deflection (mm)	Width of Bay (m)	Height of Column		No of Longitudinal Bay	Width of Longitudinal Bays (m)	Gravity Load (kN/m)				Basic Wind Speed (m/s)	Wind Moment Design				
							Floor		Roof			Universal Beam			Universal Column	
			Ground (m)	Elevated (m)			DL	LL	DL	LL		Floor	Roof		External	Internal
2 bay 2 storey	19.90											406x178x60	356x171x45	up to 2nd Floor	203x203x71	254x254x73
2 bay 4 storey	41.10											457x152x67	406x140x46	up to 2nd Floor	305x305x118	305x305x137
												406x178x60		2nd to 4th Floor	254x254x89	254x254x89
2 bay 6 storey	61.14											533x210x92	356x171x45	up to 2nd Floor	356x368x177	356x368x177
		6	5	4	2	6	21.0	24.0	22.5	9.0	28	457x152x82		2nd to 4th Floor	356x368x153	356x368x153
												406x178x60		4th to 6th Floor	254x254x89	254x254x89
2 bay 8 storey	86.81											610x229x113	406x140x46	up to 2nd Floor	356x406x235	356x368x202
												533x210x92		2nd to 4th Floor	356x368x177	356x368x177
												457x152x82		4th to 6th Floor	356x368x153	305x305x158
												406x178x60		6th to 8th Floor	305x305x97	305x305x97

Table 4.8: Wind-moment design for 4 bay 6m span frames (maximum wind with minimum gravity load) on major axis

Basic Frame Type	Total Deflection (mm)	Width of Bay (m)	Height of Column		No of Longitudinal Bay	Width of Longitudinal Bays (m)	Gravity Load (kN/m)				Basic Wind Speed (m/s)	Wind Moment Design				
							Floor		Roof			Universal Beam			Universal Column	
			Ground (m)	Elevated (m)			DL	LL	DL	LL		Floor	Roof		External	Internal
4 bay 2 storey	17.03											406x178x60	356x171x45	up to 2nd Floor	203x203x52	203x203x52
4 bay 4 storey	41.12											406x178x60	356x171x45	up to 2nd Floor	254x254x89	254x254x89
												406x178x60		2nd to 4th Floor	203x203x52	203x203x52
4 bay 6 storey	66.62											406x178x60	356x171x45	up to 2nd Floor	305x305x118	305x305x137
		6	5	4	2	6	21.0	24.0	22.5	9.0	28	406x178x60		2nd to 4th Floor	254x254x107	305x305x118
												406x178x60		4th to 6th Floor	203x203x60	203x203x60
4 bay 8 storey	82.34											610x229x101	356x171x45	up to 2nd Floor	254x254x132	356x368x153
												457x191x98		2nd to 4th Floor	305x305x97	305x305x97
												533x210x82		4th to 6th Floor	254x254x89	254x254x89
												406x178x60		6th to 8th Floor	203x203x52	203x203x52

Table 4.9: Wind-moment design for 2 bay 9m span frames (minimum wind with maximum gravity load) on major axis

Basic Frame Type	Total Deflection (mm)	Width of Bay (m)	Height of Column		No of Longitudinal Bay	Width of Longitudinal Bays (m)	Gravity Load (kN/m)				Basic Wind Speed (m/s)	Wind Moment Design				
							Floor		Roof			Universal Beam			Universal Column	
			Ground (m)	Elevated (m)			DL	LL	DL	LL		Floor	Roof		External	Internal
2 bay 2 storey	7.63											762x267x134	457x191x82	up to 2nd Floor	254x254x73	254x254x73
2 bay 4 storey	15.85											762x267x134	457x191x82	up to 2nd Floor	305x305x97	305x305x137
												762x267x134		2nd to 4th Floor	254x254x73	203x203x71
2 bay 6 storey	28.05											762x267x134	457x191x82	up to 2nd Floor	305x305x137	356x368x177
		9	5	4	2	6	30	45	24	9	20	762x267x134		2nd to 4th Floor	305x305x97	305x305x118
												762x267x134		4th to 6th Floor	254x254x73	203x203x71
2 bay 8 storey	43.00											762x267x134	457x191x82	up to 2nd Floor	356x368x153	356x406x235
												762x267x134		2nd to 4th Floor	305x305x118	356x368x177
												762x267x134		4th to 6th Floor	305x305x97	305x305x118
												762x267x134		6th to 8th Floor	254x254x73	203x203x71

Table 4.10: Wind-moment design for 4 bay 9m span frames (minimum wind with maximum gravity load) on major axis

Basic Frame Type	Total Deflection (mm)	Width of Bay (m)	Height of Column		No of Longitudinal Bay	Width of Longitudinal Bays (m)	Gravity Load (kN/m)				Basic Wind Speed (m/s)	Wind Moment Design				
			Ground (m)	Elevated (m)			Floor		Roof			Universal Beam			Universal Column	
							DL	LL	DL	LL		Floor	Roof		External	Internal
4 bay 2 storey	4.28											762x267x134	457x191x82	up to 2nd Floor	254x254x73	254x254x73
4 bay 4 storey	9.09											762x267x134	457x191x82	up to 2nd Floor	305x305x97	305x305x137
												762x267x134		2nd to 4th Floor	254x254x73	203x203x71
4 bay 6 storey	15.23											762x267x134	457x191x82	up to 2nd Floor	305x305x137	356x368x177
		9	5	4	2	6	30	45	24	9	20	762x267x134		2nd to 4th Floor	305x305x97	305x305x118
												762x267x134		4th to 6th Floor	254x254x73	203x203x71
4 bay 8 storey	22.57											762x267x134	457x191x82	up to 2nd Floor	356x368x153	356x406x235
												762x267x134		2nd to 4th Floor	305x305x118	356x368x177
												762x267x134		4th to 6th Floor	305x305x97	305x305x118
												762x267x134		6th to 8th Floor	254x254x73	203x203x71

Table 4.11: Wind-moment design for 2 bay 9m span frames (maximum wind with minimum gravity load) on major axis

Basic Frame Type	Total Deflection (mm)	Width of Bay (m)	Height of Column		No of Longitudinal Bay	Width of Longitudinal Bays (m)	Gravity Load (kN/m)				Basic Wind Speed (m/s)	Wind Moment Design				
							Floor		Roof			Universal Beam			Universal Column	
			Ground (m)	Elevated (m)			DL	LL	DL	LL		Floor	Roof		External	Internal
2 bay 2 storey	19.38											533x210x101	457x152x82	up to 2nd Floor	203x203x71	203x203x71
2 bay 4 storey	39.85											533x210x101	457x152x82	up to 2nd Floor	305x305x118	305x305x118
												533x210x101		2nd to 4th Floor	203x203x71	254x254x73
2 bay 6 storey	63.51											533x210x101	457x152x82	up to 2nd Floor	356x368x177	356x368x177
		9	5	4	2	6	21.0	24.0	22.5	9.0	28	533x210x101		2nd to 4th Floor	356x368x153	305x305x158
												533x210x101		4th to 6th Floor	254x254x89	203x203x86
2 bay 8 storey	87.26											610x229x125	457x152x82	up to 2nd Floor	356x406x235	356x406x235
												610x229x113		2nd to 4th Floor	356x368x202	356x406x235
												610x229x101		4th to 6th Floor	356x368x153	254x254x132
												533x210x101		6th to 8th Floor	254x254x73	254x254x73

Table 4.12: Wind-moment design for 4 bay 9m span frames (maximum wind with minimum gravity load) on major axis

Basic Frame Type	Total Deflection (mm)	Width of Bay (m)	Height of Column		No of Longitudinal Bay	Width of Longitudinal Bays (m)	Gravity Load (kN/m)				Basic Wind Speed (m/s)	Wind Moment Design				
							Floor		Roof			Universal Beam			Universal Column	
			Ground (m)	Elevated (m)			DL	LL	DL	LL		Floor	Roof		External	Internal
4 bay 2 storey	12.48											533x210x101	457x152x82	up to 2nd Floor	203x203x60	203x203x71
4 bay 4 storey	30.42											533x210x101	457x152x82	up to 2nd Floor	254x254x89	305x305x97
												533x210x101		2nd to 4th Floor	203x203x52	203x203x52
4 bay 6 storey	56.74											533x210x101	457x152x82	up to 2nd Floor	305x305x118	305x305x137
		9	5	4	2	6	21.0	24.0	22.5	9.0	28	533x210x101		2nd to 4th Floor	254x254x89	254x254x89
												533x210x101		4th to 6th Floor	203x203x52	203x203x52
4 bay 8 storey	81.39											533x210x101	457x152x82	up to 2nd Floor	254x254x167	356x368x177
												533x210x101		2nd to 4th Floor	305x3055x137	305x305x137
												533x210x101		4th to 6th Floor	254x254x89	254x254x89
												533x210x101		6th to 8th Floor	203x203x60	203x203x60

Table 4.13: Wind-moment design for 2 bay 6m span frames (minimum wind with maximum gravity load) on minor axis

Basic Frame Type	Total Deflection (mm)	Width of Bay (m)	Height of Column		No of Longitudinal Bay	Width of Longitudinal Bays (m)	Gravity Load (kN/m)				Basic Wind Speed (m/s)	Wind Moment Design				
							Floor		Roof			Universal Beam			Universal Column	
			Ground (m)	Elevated (m)			DL	LL	DL	LL		Floor	Roof		External	Internal
2 bay 2 storey	16.46											533x210x82	406x140x46	up to 2nd Floor	254x254x89	254x254x89
2 bay 4 storey	35.93											533x210x82	406x140x46	up to 2nd Floor	305x305x118	356x368x153
												533x210x82		2nd to 4th Floor	254x254x73	254x254x73
2 bay 6 storey	61.06											533x210x82	406x140x46	up to 2nd Floor	356x368x177	356x368x202
		6	5	4	2	6	30	45	24	9	20	533x210x82		2nd to 4th Floor	305x305x137	305x305x137
												533x210x82		4th to 6th Floor	254x254x73	254x254x73
2 bay 8 storey	80.32											533x210x82	406x140x46	up to 2nd Floor	356x406x287	356x406x287
												533x210x82		2nd to 4th Floor	356x368x202	356x368x202
												533x210x82		4th to 6th Floor	356x368x153	305x305x137
												533x210x82		6th to 8th Floor	254x254x73	254x254x89

Table 4.14: Wind-moment design for 4 bay 6m span frames (minimum wind with maximum gravity load) on minor axis

Basic Frame Type	Total Deflection (mm)	Width of Bay (m)	Height of Column		No of Longitudinal Bay	Width of Longitudinal Bays (m)	Gravity Load (kN/m)				Basic Wind Speed (m/s)	Wind Moment Design				
							Floor		Roof			Universal Beam			Universal Column	
			Ground (m)	Elevated (m)			DL	LL	DL	LL		Floor	Roof		External	Internal
4 bay 2 storey	10.71											533x210x82	406x140x46	up to 2nd Floor	254x254x73	254x254x89
4 bay 4 storey	21.46											533x210x82	406x140x46	up to 2nd Floor	305x305x97	356x368x153
												533x210x82		2nd to 4th Floor	203x203x71	254x254x73
4 bay 6 storey	41.75											533x210x82	406x140x46	up to 2nd Floor	305x305x137	356x368x177
		6	5	4	2	6	30	45	24	9	20	533x210x82		2nd to 4th Floor	254x254x89	305x305x118
												533x210x82		4th to 6th Floor	203x203x71	254x254x73
4 bay 8 storey	58.93											533x210x82	406x140x46	up to 2nd Floor	356x368x177	356x406x235
												533x210x82		2nd to 4th Floor	254x254x132	356x368x153
												533x210x82		4th to 6th Floor	305x305x97	305x305x118
												533x210x82		6th to 8th Floor	203x203x71	254x254x73

Table 4.15: Wind-moment design for 2 bay 6m span frames (maximum wind with minimum gravity load) on minor axis

Basic Frame Type	Total Deflection (mm)	Width of Bay (m)	Height of Column		No of Longitudinal Bay	Width of Longitudinal Bays (m)	Gravity Load (kN/m)				Basic Wind Speed (m/s)	Wind Moment Design				
							Floor		Roof			Universal Beam			Universal Column	
			Ground (m)	Elevated (m)			DL	LL	DL	LL		Floor	Roof		External	Internal
2 bay 2 storey	19.50											406x178x60	356x171x45	up to 2nd Floor	305x305x118	305x305x118
2 bay 4 storey	41.46											457x152x67	406x140x46	up to 2nd Floor	356x406x235	356x406x235
												406x178x60		2nd to 4th Floor	356x368x153	254x254x132
2 bay 6 storey	65.63											533x210x92	356x171x45	up to 2nd Floor	356x406x340	356x406x340
		6	5	4	2	6	21.0	24.0	22.5	9.0	28	457x152x82		2nd to 4th Floor	356x406x287	356x406x28
												406x178x60		4th to 6th Floor	356x368x153	254x254x132
2 bay 8 storey	87.62											610x229x113	406x140x46	up to 2nd Floor	356x406x467	356x406x467
												533x210x92		2nd to 4th Floor	356x406x467	356x406x467
												457x152x82		4th to 6th Floor	356x406x340	356x406x340
												406x178x60		6th to 8th Floor	356x368x153	356x368x153

Table 4.16: Wind-moment design for 4 bay 6m span frames (maximum wind with minimum gravity load) on minor axis

Basic Frame Type	Total Deflection (mm)	Width of Bay (m)	Height of Column		No of Longitudinal Bay	Width of Longitudinal Bays (m)	Gravity Load (kN/m)				Basic Wind Speed (m/s)	Wind Moment Design				
							Floor		Roof			Universal Beam			Universal Column	
			Ground (m)	Elevated (m)			DL	LL	DL	LL		Floor	Roof		External	Internal
4 bay 2 storey	18.23											406x178x60	356x171x45	up to 2nd Floor	254x254x89	254x254x89
4 bay 4 storey	36.72											406x178x60	356x171x45	up to 2nd Floor	356x368x153	356x368x153
												406x178x60		2nd to 4th Floor	254x254x89	305x305x97
4 bay 6 storey	65.79											406x178x60	356x171x45	up to 2nd Floor	356x406x235	356x406x235
		6	5	4	2	6	21.0	24.0	22.5	9.0	28	406x178x60		2nd to 4th Floor	356x368x202	356x368x202
												406x178x60		4th to 6th Floor	254x254x89	305x305x97
4 bay 8 storey	81.76											610x229x101	356x171x45	up to 2nd Floor	305x305x283	356x406x287
												457x191x98		2nd to 4th Floor	305x305x198	356x368x202
												533x210x82		4th to 6th Floor	356x368x153	356x368x153
												406x178x60		6th to 8th Floor	254x254x89	305x305x97

Table 4.17: Wind-moment design for 2 bay 9m span frames (minimum wind with maximum gravity load) on minor axis

Basic Frame Type	Total Deflection (mm)	Width of Bay (m)	Height of Column		No of Longitudinal Bay	Width of Longitudinal Bays (m)	Gravity Load (kN/m)				Basic Wind Speed (m/s)	Wind Moment Design				
			Ground (m)	Elevated (m)			Floor		Roof			Universal Beam			Universal Column	
							DL	LL	DL	LL		Floor	Roof		External	Internal
2 bay 2 storey	10.13											762x267x134	457x191x82	up to 2nd Floor	305x305x97	305x305x118
2 bay 4 storey	25.59											762x267x134	457x191x82	up to 2nd Floor	305x305x137	356x368x177
												762x267x134		2nd to 4th Floor	254x254x89	305x305x97
2 bay 6 storey	42.39											762x267x134	457x191x82	up to 2nd Floor	356x368x177	356x406x287
		9	5	4	2	6	30	45	24	9	20	762x267x134		2nd to 4th Floor	305x305x118	356x368x153
												762x267x134		4th to 6th Floor	254x254x89	305x305x97
2 bay 8 storey	63.26											762x267x134	457x191x82	up to 2nd Floor	356x406x235	356x406x340
												762x267x134		2nd to 4th Floor	356x368x153	356x406x235
												762x267x134		4th to 6th Floor	305x305x118	356x368x153
												762x267x134		6th to 8th Floor	254x254x89	305x305x97

Table 4.18: Wind-moment design for 4 bay 9m span frames (minimum wind with maximum gravity load) on minor axis

Basic Frame Type	Total Deflection (mm)	Width of Bay (m)	Height of Column		No of Longitudinal Bay	Width of Longitudinal Bays (m)	Gravity Load (kN/m)				Basic Wind Speed (m/s)	Wind Moment Design				
							Floor		Roof			Universal Beam			Universal Column	
			Ground (m)	Elevated (m)			DL	LL	DL	LL		Floor	Roof		External	Internal
4 bay 2 storey	5.74											762x267x134	457x191x82	up to 2nd Floor	305x305x97	305x305x118
4 bay 4 storey	13.26											762x267x134	457x191x82	up to 2nd Floor	305x305x137	356x368x177
												762x267x134		2nd to 4th Floor	254x254x89	305x305x97
4 bay 6 storey	22.08											762x267x134	457x191x82	up to 2nd Floor	356x368x153	356x406x287
		9	5	4	2	6	30	45	24	9	20	762x267x134		2nd to 4th Floor	305x305x118	356x368x153
												762x267x134		4th to 6th Floor	254x254x89	305x305x97
4 bay 8 storey	33.36											762x267x134	457x191x82	up to 2nd Floor	356x368x202	356x406x340
												762x267x134		2nd to 4th Floor	356x368x153	356x406x235
												762x267x134		4th to 6th Floor	305x305x118	356x368x153
												762x267x134		6th to 8th Floor	254x254x89	305x305x97

Table 4.19: Wind-moment design for 2 bay 9m span frames (maximum wind with minimum gravity load) on minor axis

Basic Frame Type	Total Deflection (mm)	Width of Bay (m)	Height of Column		No of Longitudinal Bay	Width of Longitudinal Bays (m)	Gravity Load (kN/m)				Basic Wind Speed (m/s)	Wind Moment Design				
							Floor		Roof			Universal Beam			Universal Column	
			Ground (m)	Elevated (m)			DL	LL	DL	LL		Floor	Roof		External	Internal
2 bay 2 storey	16.48											533x210x101	457x152x82	up to 2nd Floor	305x305x118	305x305x118
2 bay 4 storey	41.15											533x210x101	457x152x82	up to 2nd Floor	356x368x202	356x368x202
												533x210x101		2nd to 4th Floor	305x305x118	305x305x118
2 bay 6 storey	65.55											533x210x101	457x152x82	up to 2nd Floor	356x406x393	356x406x393
		9	5	4	2	6	21.0	24.0	22.5	9.0	28	533x210x101		2nd to 4th Floor	356x406x287	356x406x340
												533x210x101		4th to 6th Floor	305x305x118	356x368x153
2 bay 8 storey	86.77											610x229x125	457x152x82	up to 2nd Floor	356x406x551	356x406x467
												610x229x113		2nd to 4th Floor	356x406x467	356x406x551
												610x229x101		4th to 6th Floor	356x406x287	356x406x287
												533x210x101		6th to 8th Floor	305x305x118	356x368x153

Table 4.20: Wind-moment design for 4 bay 9m span frames (maximum wind with minimum gravity load) on minor axis

Basic Frame Type	Total Deflection (mm)	Width of Bay (m)	Height of Column		No of Longitudinal Bay	Width of Longitudinal Bays (m)	Gravity Load (kN/m)				Basic Wind Speed (m/s)	Wind Moment Design				
							Floor		Roof			Universal Beam			Universal Column	
			Ground (m)	Elevated (m)			DL	LL	DL	LL		Floor	Roof		External	Internal
4 bay 2 storey	19.29											533x210x101	457x152x82	up to 2nd Floor	203x203x86	254x254x89
4 bay 4 storey	38.09											533x210x101	457x152x82	up to 2nd Floor	305x305x118	356x368x153
												533x210x101		2nd to 4th Floor	254x254x73	254x254x73
4 bay 6 storey	59.95											533x210x101	457x152x82	up to 2nd Floor	356x368x202	356x406x235
		9	5	4	2	6	21.0	24.0	22.5	9.0	28	533x210x101		2nd to 4th Floor	356x368x153	305x305x137
												533x210x101		4th to 6th Floor	254x254x89	254x254x89
4 bay 8 storey	86.62											533x210x101	457x152x82	up to 2nd Floor	356x406x287	356x406x287
												533x210x101		2nd to 4th Floor	356x406x235	356x406x235
												533x210x101		4th to 6th Floor	356x368x153	305x305x137
												533x210x101		6th to 8th Floor	254x254x107	254x254x107

CHAPTER 5

EXPERIMENTAL INVESTIGATION:
FULL-SCALE ISOLATED JOINT TESTS

5.1 General

To properly manage the experimental investigation in the laboratory, the experimental works involved were divided into two phases. In the first phase of the experiment, four isolated arrangements of flush endplate connections were tested until failure. As for the second phase, four isolated arrangements of extended endplate connections and one isolated arrangement of flush endplate connections were tested until failure. All of the Isolated tests representing the external connections that connect the beam to the major axis of the column; and thus, were configured in a cantilever arrangement.

5.2 Specimen Size and Material

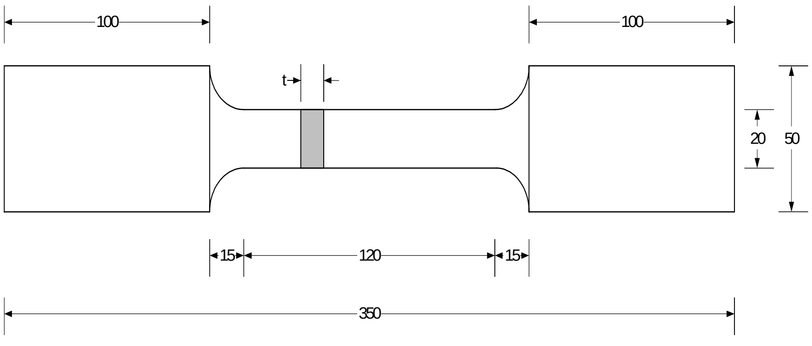
The bulk of the specimen fabrication was undertaken by a steelwork fabricator named Trapezoid Web Profiled Sdn. Bhd. in Pasir Gudang. But, before conducting the Isolated tests, tensile tests of the standard specimen were carried out in accordance with the procedures outlined in the specification (BS EN 10002-1:2001). This was done to determine specifically the properties and the characteristic values of the flanges and webs of the beams and columns, and the endplates used for the connections. Coupon samples in the shape of a bone were initially flame cut and machined to the dimensions shown in Figure 5.1 from each of the components mentioned above. The tensile tests were then carried out using the Universal testing machine (DARTEC) until failure; And important values such as the yield and ultimate stresses were obtained by the means of plotted stress versus strain graphs. Figure 5.2 shows the failure mode for two of the bone-shaped samples after tensile test. The stress versus strain graph as shown in Figure 5.3 is an example of the type of graphs obtained in determining the properties and the characteristic values of the specimen.

For the first phase tests, four sizes of TWP beams were used ranging from the shallow TWP 300 x 130 x 37/12/6 to the deep 680 x 250 x 117/20/8. These four ranges of beams were considered adequate in representing the behaviour of other beam sizes in between. The column, on the other hand, was of one size only that is UC 254 x 254 x 107. The reason of choosing only one size of column for all joints lies in the fact that the focus of the study is to observe the behaviour of TWP in all those joints. From preliminary calculation, this size of column was adequate in resisting the tension flange, compression flange and panel shear actions resulting from the beam's bending. Therefore, column's flange stiffnesses were not needed. The joints were designed and devised so the failures were to occur at the connections. The shear resistance for each connection was provided by one row or two rows of bolts positioned in the compression region of the connection. This is a standard practice outlined by the codes (BS 5950 and EC3) that shear should only be resisted by the bolts adjacent to the bottom flange. The bolts adjacent to the top flange thus were capable to attain full tensile capacity in calculating the moment resistance of the connection. Table 5.1 shows the test matrix for the first phase of the isolated tests conducted.

Table 5.1: Test matrix for the first phase

Test No	Model Name	Beam Size	Column Size	Bolt Row (T-B)	End Plate Size	Bolt
N1	FB1P1-1	300x130x37/12/6 (Eqv. UB 305x127x48)	254x254x107 (UC - S275)	1(2-2)	200 x 12	20
N2	FB1P1-2	400x170x49/12/6 (Eqv. UB 406x178x67)		2(4-2)		
N3	FB1P1-3	530x210x83/16/8 (Eqv. UB 533x210x92)		2(4-4)		
N4	FB1P1-4	680x250x117/20/ 8 (Eqv. UB 686x254x152)		2(4-4)		

In this particular phase, all beams were made of Grade S275 steel though the flange and the web of a TWP beam could be made of different grades (since TWP sections are built-up sections). For instance, Grade S355 for the flange and Grade S275 for the web. The main reason of using a uniform grade is to maintain the condition as close as possible to the one using a hot rolled section so as to highlight the effect of the corrugated web on the connections. This type of fabrication (using different steel grade for flange and web) of course will affect the behaviour and certain capacities of the beam itself.



All dimensions are in mm.

Figure 5.1: Form of a coupon sample for the tensile test

Figure 5.2: Failure mode of bone shaped samples

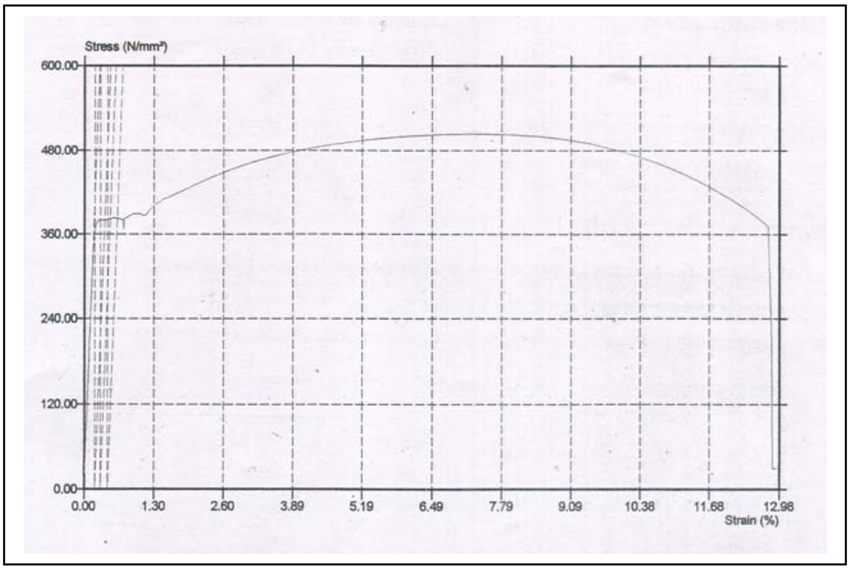


Figure 5.3: Typical stress versus strain curve from a tensile test

As mentioned previously, in the second phase of the Isolated tests, four arrangements of extended endplate connections and one arrangement of flush endplate connections were tested until failure. In order to take into account the higher capacities expected from the extended end plate connections, a bigger size of column was used instead that is UC 305 x 305 x 118. Sizes of beams were also different from the first phase tests and constituted of a S355 flange and S275 web to represent a typical fabrication of TWP sections. Two sizes of end plates were utilised, 200 mm x 12 mm and 250 mm x 15 mm of which 20 mm diameter bolts were used for the former and 24 mm diameter bolts were used for the later. Table 5.2 shows the test matrix for the second phase of the FSIJ tests.

Table 5.2: Test matrix for the second phase

Test No	Model Name	Beam Size	Column Size	Con. Type	Bolt Row (T-B)	End Plate Size	Bolt
N5	E2R20P1	400x140x39.7/12/4	305x305x118 (UC-S275)	EEP	2(4-4)	200x12	20
N6	E2R24P2	500x180x61.9/16/4			2(4-4)	250x15	24
N7	E3R20P1	450x160x50.2/12/4			3(6-4)	200x12	20
N8	E3R24P2	600x200x80.5/16/6			3(6-4)	250x5	24
N9	F2R20P1	450x160x50.2/12/4		FEP	2(4-4)	200x12	20

5.3 Boundary Conditions

All of the connections to be tested were designed as to exhibit a semi-continuous behaviour in accordance with the codes (BS 5950 and EC3). In term of strength, the connections were categorised as partial strength, a category of which the moment capacity of a connection, M_j is between 25% and 100% of the beam's moment capacity, M_{cx} (EC 3, 2005). In term of rigidity, the connections were categorised as semi-rigid, a category of which the initial stiffness of a connection is between $\frac{EI_b}{2L_b}$ and $\frac{8EI_b}{L_b}$ (EC 3, 2005). The base of the column was bolted to a specially devised strong base, which was subsequently secured to the laboratory strong floor. As a result, a pin-supported condition was created for the column at the bottom. At the top, rollers were placed adjacent to the flange towards the expected inclining face. A roller-supported condition at the top will prevent the axial load induced in the column (if any) to affect the connection to be tested.

5.4 Preparation of Specimen

As indicated earlier, all specimens were fabricated by a steelwork fabricator, Trapezoid Web Profiled Sdn. Bhd., according to the detail drawings provided. All together, there were nine connection arrangements fabricated, four in the first phase and five in the second phase. Figure 5.4 illustrates the steelwork fabrication drawing for one of the flush endplate connection arrangements (N1).

Two sizes of standard bolts of grade 8.8 were used in this study, which are the M20 and M24 bolts. The M20 bolts were used for the endplate with a thickness of 12 mm whereas the M24 bolts were used for the endplate with a thickness of 15 mm. The strength of endplates was maintained as S275 steel. Width of the endplate was kept at 200 mm and 250 mm with the vertical height of the endplate was kept at the beam depth plus 50 mm for the flush endplate connections and 90 mm for the extended endplate connections. The end plates for the extended endplate connections were extended on the tension side only since no reversal of moments were expected. One row of bolts was used in the extended parts of the endplates. Fillet weld welded all around was used to connect the endplates to the TWP beams. To make it similar to the SCI's, the sizes of the fillet weld were selected to be 10 mm and 8 mm for connecting to the flanges and the webs of the beams respectively, even though sizes of 2 mm less for both locations were also adequate. The vertical and horizontal distance between the bolts was maintained at 90 mm.

Universal Column (H) sections were selected as the columns whilst TWP sections were selected as the beams for all arrangements. The minimum and maximum thicknesses chosen for the web of the TWP beams were 4 mm and 8 mm respectively. The ratio of beam depth to web thickness was kept not to exceed the limit for compact section in the first phase tests in order to assure that the plastic behaviour of the beam could be attained. However, the ratio was increased and kept not to exceed the limit for semi-compact section in the second phase tests after observing that, in general, buckling of the web was not likely to occur due to the stiffening effect of the trapezoidal corrugated web of the TWP beams.

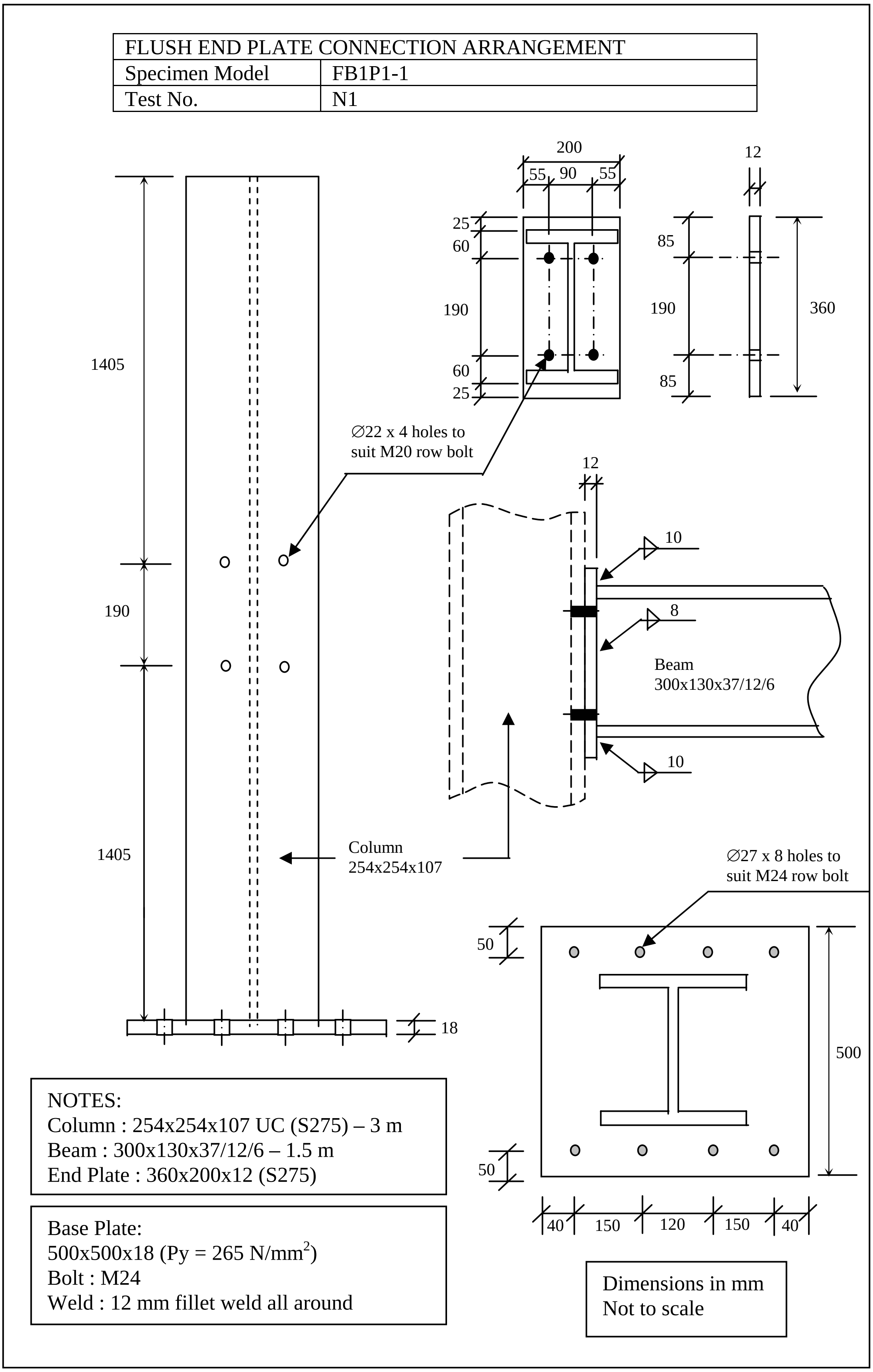


Figure 5.4: Fabrication drawing for a typical connection arrangement

5.5 Data Acquisition

The instrumentation system adopted for the experimental investigation was designed to acquire all the necessary measurements and important data that would be required to determine the behavioural characteristics of the connections. Depicted in Figure 5.5 is the schematic representation of the instrumentation system for data acquisition.

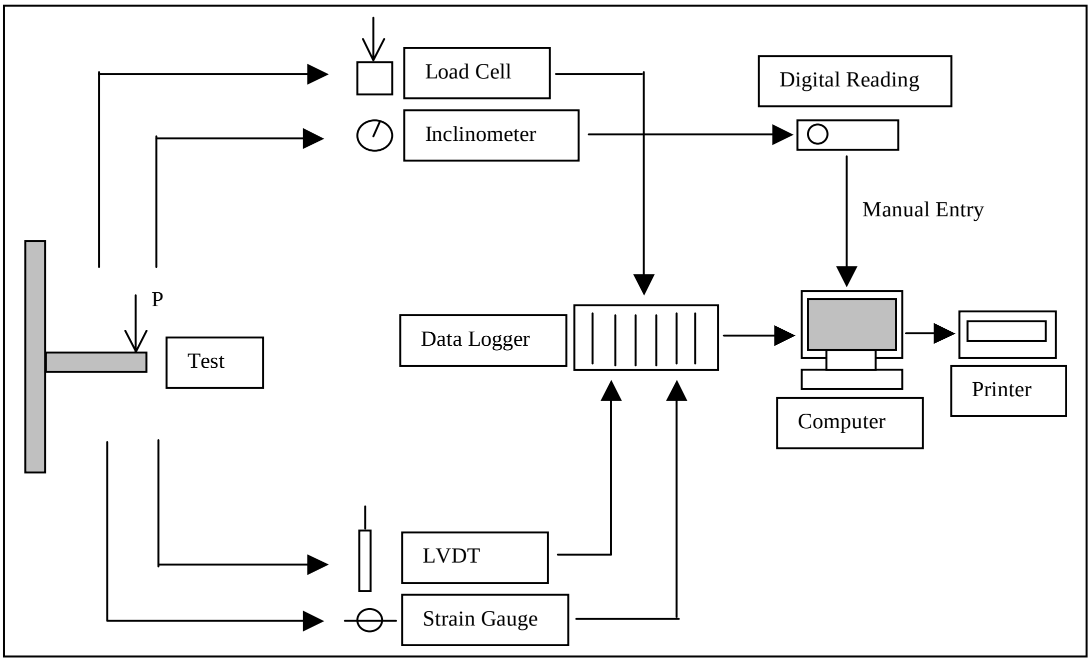


Figure 5.5: Instrumentation system for data acquisition

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

However, the rotational values of the beam and column were recorded manually from the digital display unit of the Lucas Rotational Inclinometers. This is because the instrument does not have the capability of connecting to the data logger and recording the measurements directly. At the time of the experimental programme, the new inclinometers with that capability have already been purchased, but have not been received yet. Settling with the existing ones, one inclinometer was mounted midway at the web of the beams at a distance of about 100 mm from the face of the column flange. This inclinometer provided the rotational values of the beam, ϕ_b, upon loading. The other inclinometer was placed at the centre of the column panel shear thus provided the rotational values of the column, ϕ_c. The overall rotation of the joint, ϕ, was then taken as the difference between ϕ_b and ϕ_c.

$$\phi = \phi_b - \phi_c \tag{3.1}$$

The default unit for the measured rotation of the inclinometers was degree, therefore, the values had to be converted to the standard unit of miliradians using the conversion factor of:

$$1^o = 17.46032\ mrad \tag{3.2}$$

LVDTs were placed at several specified locations for measuring linear displacements along the beam and column. Four types of LVDTs manufactured by TML, Japan were used in this experiment, which are the 25 mm, 50 mm, 100 mm and 200 mm transducers. The 25 mm and 50 mm transducers were used to measure small to medium displacements such as the beam's deflection close to the face of the column, or the column's translation. Whilst the 100 mm and 200 mm transducers were used to measure large deflections, which occurred further up towards the end of the beams.

The loads were applied on the beam by using a single inlet hydraulic jack and measured by load cells with capacities of 300 kN, 500 kN and1000 kN depending upon the expected failure load of the tested specimen.

Figure 5.6 illustrates the layout of the arrangement adopted for the Isolated tests showing the locations of the column, beam, inclinometers, LVDTs, hydraulic jack, and load cell. Six LVDTs were used in the first phase tests but the one located at 50 mm from the face of the column was omitted in the second phase tests since there was another LVDT placed at 100 mm from the face of the column. Strain gauges of types linear, bi-linear and roxette were placed at several locations on the beams, column, end plates, and at the vicinity of the bolts. Figure 5.7 shows the locations of the strain gauges for the first phase tests.

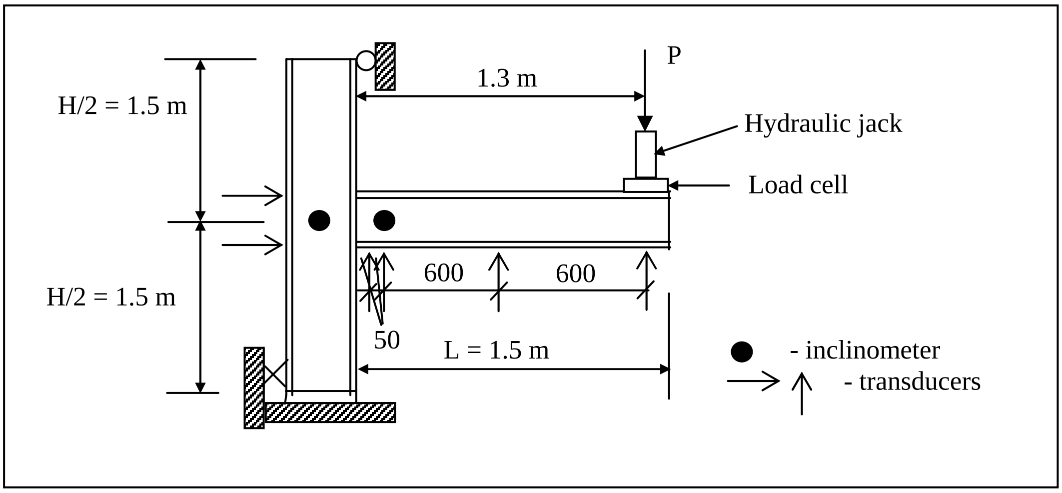


Figure 5.6: Arrangement for Isolated tests

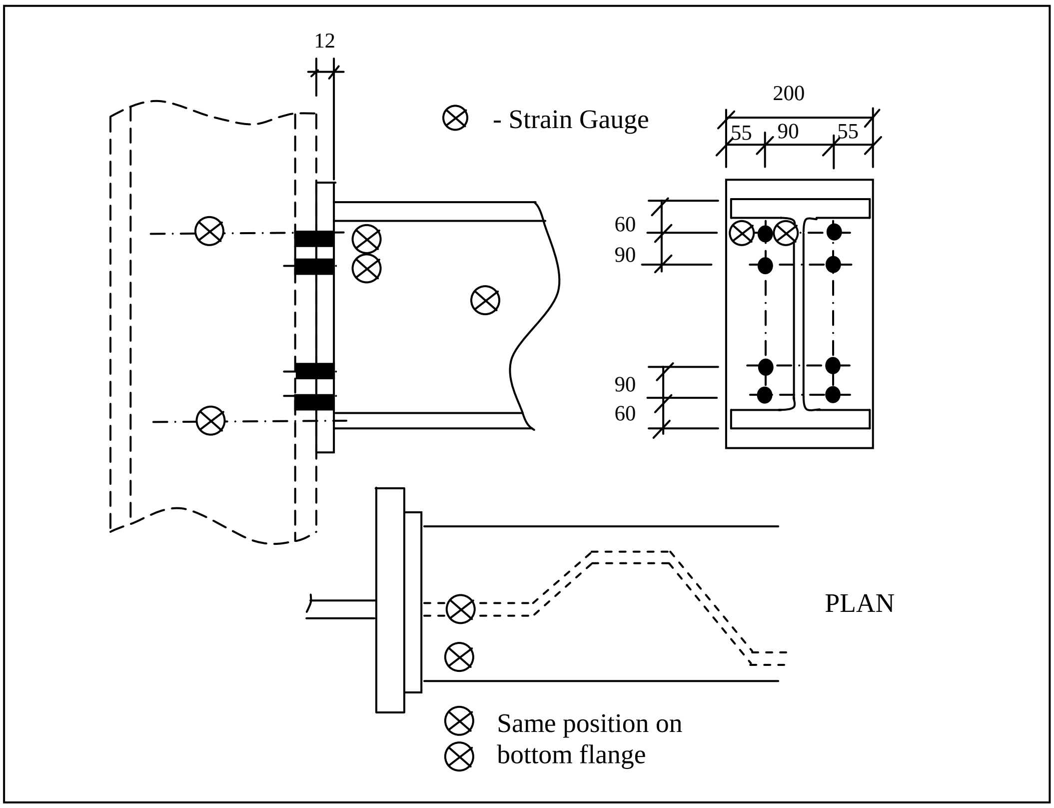


Figure 5.7: Locations of the strain gauges for phase one tests

5.6 Test Set-Up and Procedures

A purpose-built test rig of which was originally used by Md Azman (2001) to study the performance of endplate connections with locally produced sections was adopted. However, in order to suit to the specimens and to enable the experimental investigation to be carried out as designed, certain modifications to the rig were deliberated. Basically, the rig consists of channel sections pre-drilled with 22 mm holes for bolting purposes. The sections were then fastened and bolted to form loading frames of which were subsequently secured to the laboratory strong floor as shown in Figure 5.8. The height of the column was 3 m, chosen as to represent the height of a storey in a typical braced steel frame. The beam was of length 1.5 m of which a point load was applied by using the hydraulic jack at a distance of about 1.3 m from the face of the column. The position of the applied load was carefully chosen so that the connection will experience a moment without jeopardizing the vertical shear capacity. To achieve this, the distance of the applied load from the face of the column should be approximately equal to the corresponding distance of the point of contra flexural between the negative and positive moment from the face of the column. Hence, the distance of about 1.3 m adopted in this experimental investigation was deemed adequate in producing a moment up to failure at the connection, and a plastic mechanism had been reached in the beam.

In placing the test specimen for each arrangement, the column was placed first by bolting the base plate to the strong base on the strong floor. Care was taken in making sure that the column was in alignment by using a bubble leveller. Tightening of the bolts was done using a torque wrench and maintained throughout the experimental programme for consistency. Then, the 1.5 m beam pre-welded with an endplate was lifted and bolted to the column’s flange. The horizontal and vertical positions of the beam were monitored using the bubble leveller during the installation. After the instrumentation system mentioned above had been set-up and the specimen had been securely located in the rig, the data collection software in the computer was checked to make sure that all channels connecting to the instruments on the specimen indicated a properly working condition. Correction factors from calibration and gauge factors from manufacturer were input into the software prior to each test. A further check on the instrumentation was then carried out by loading the specimen to a load of about 20 kN to 30 kN (26 kNm to 39 kNm), and then unloading the specimen back down. In addition to making sure that the values from the instruments were received and recorded satisfactorily, this procedure was taken to enable all the components in the connection arrangement to be embedded in prior to commencing the test. The load increment at this stage was taken as 5 kN.

The specimen was then loaded up to two-thirds of the analytically calculated moment of resistance, and was expressed in term of the point load applied for easier monitoring. For the specimen with low moment of resistance expected, the increments of applied load were taken as 5 kN. However, as the beams became deeper and the endplate connections became stiffer, 10 kN increments were adopted since greater moment of resistance would be expected. After reaching the two-third value, the specimen was unloaded back and re-initialised. This procedure was taken to enable the initial stiffness, S_{ji}, of the joint to be monitored.

After re-initialising the instrumentation system, the specimen was loaded as described above, but the load applied was not restricted to the two-thirds value. Instead, the specimen was further loaded until there was a significantly large deflection of the beam observed. The load application was continually applied after this point but the increments were controlled by the deflection instead of the load as before. A deflection of 2 mm was adopted as a suitable increment for this stage. This procedure was continued until the specimen had reached its ‘failure’ condition, or until there exist a situation that required the test to be concluded. The ‘failure’ condition was deemed to have been reached when any of the following situations mentioned below occurred.

i) An abrupt and significantly large reduction in the applied load being attained.

ii) An abrupt and significantly large increment in the deflection of the beam being attained.

The failure mode that might cause the above situations to occur could be one of the following.

i) The development of shear deformation on the web of the beam in the vicinity of the connection.

ii) The development of local buckling on the bottom flange of a beam in the vicinity of the connection due to the compressive action along the flange.

iii) Sudden yielding of the end plate around the top flange of a beam due to the tensile action along the flange. This failure mode is referred to as Mode 1 in the SCI's publication (1996).

iv) Sudden yielding of the end plate around the top flange of a beam due to the tensile action along the flange coupled with the yielding of the critical bolts. The critical bolts are the bolts below the top flange of a beam as in the case of flush end plate connections, and above and below the top flange of a beam as in the case of extended end plate connections. This failure mode is referred to as Mode 2 in the SCI's publication (1996).

v) Sudden yielding of the critical bolts only. This failure mode is referred to as Mode 3 in the SCI's publication (1996).

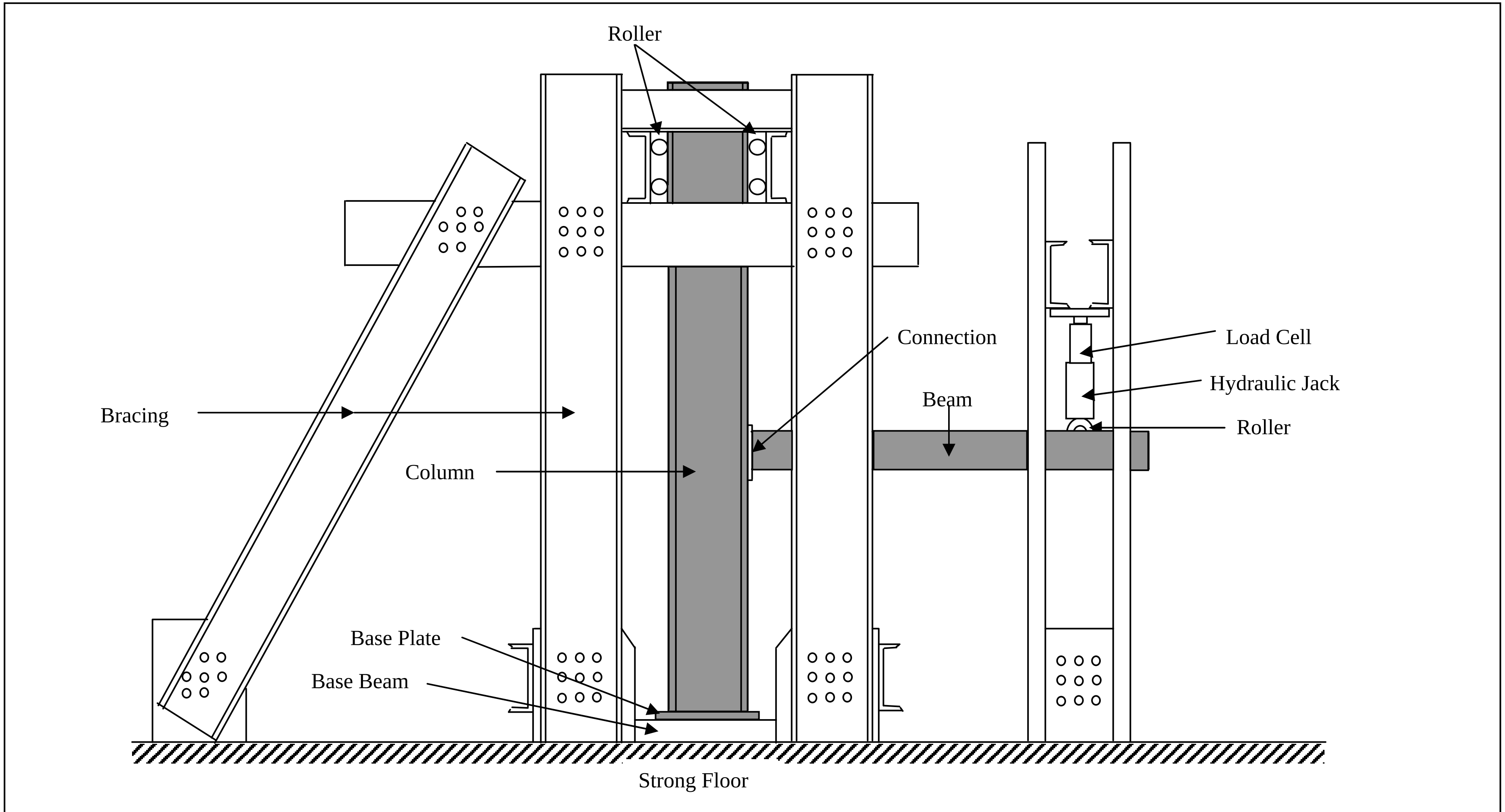


Figure 5.8: Layout of the testing rig for the isolated tests

5.7 Calibration

The reliability of the data acquired from the instrumentation system designed does not only involve using the right and sophisticated instrument, but importantly, the data received from any type of instrument used must be able to be interpreted into accurate and meaningful measurements. In order to achieve this intention, the instrumentation was calibrated by adopting the following procedures:

a) Rotational Inclinometers

The inclinometers were calibrated by using a special device called Total Station of which is capable of reading angles and measuring distances. Each inclinometer was placed at one end of a horizontally positioned steel straight edge. With the aid of the Total Station, the straight edge was levelled and a distance of 1000 mm was marked on the other end of the straight edge. A series of vertical distances of a 5 mm increment was then marked at this end. The straight edge was then pivoted; And the angles measured by the inclinometer and the Total Station were recorded. The $y = mx$ curves were plotted for the measured angles by the inclinometer against the angle obtained from the geometry and form the Total Station. An average value was then taken from the two curves, which is equal to **1.0122**.

b) Displacement Transducers

Four types of transducers were used during the experiment, which are the 200 mm, 100 mm, 50 mm and 25 mm transducers. As for these transducers, calibration of the measured values was made by inserting the correction factors for each transducer into the data collection software. These correction factors were either obtained from the manufacturer's supplied manual or from the direct conversion using the software.

c) Load Cell

Depending on the expected failure load of the specimen, there were three types of load cells used. These load cells are the 300 kN, 500 kN and 1000 kN capacity. Similar to the transducers, the measured values from the load cells were calibrated by using the correction factors from the manufacturer's supplied manual. The corresponding correction factor was then inserted into the software where the recorded values were calibrated automatically.

5.8 Remarks on Experimental Investigation

The experimental programme for the Full-Scale Isolated Joint tests was designed and devised as to closely resemble the actual arrangements of beam-to-column joints in a semi-continuous construction of a typical multi-storey braced steel frame. Since the critical location in a typical construction is at the joint to the external column, the test arrangement chosen was the cantilever. The data and values were acquired and collected as much as possible using an instrumentation system that consists of a data logger , load cells, displacement transducers, inclinometers, and strain gauges. Loads were applied gradually until a noticeable failure mode was obtained, or there existed some abrupt and large increment in the displacements. In addition, visual monitoring was also been carried out for all tests, which includes eye inspection, physical inspection, and video recording for animation purposes. Photographs were taken of every item, location and situation that was considered important and significant.

CHAPTER 6

RESULTS AND DISCUSSION

6.1 General

The presentation and discussion of results of this study are best explained in accordance with the tasks involved of which can be categorised into the following works:

1. The behaviour of partial strength connections for flush endplate and extended endplate investigated through Full-Scale Isolated Joint tests.

2. Standardised capacity tables for partial strength connections with TWP beams. All together, there were six tables for the flush endplate connections and eight tables for the extended endplate connections.

3. A parametric study on the design of multi-storey unbraced steel frames of various bays and storeys incorporating standardised connections generated previously. The economic aspect in term of the total weight savings between simple and semi-rigid construction was the main focus.

Finally, all of the results obtained were correlated and further discussions on its applicability in the design of multi-storey braced steel frames were presented.

6.2 Full-Scale Isolated Joint Tests

The Isolated tests constituted of two phases with Phase 1 consisted of four flush endplate connections, and Phase 2 consisted of four extended endplate connections and one flush endplate connection. All of the specimens underwent the same procedures of testing. At the initial stage of loading (up to about 5 % of the predicted load), there were no apparent visual deformations observed in all of the experiments. This was expected since the application of loads was intended for all of the components of the joint to be embedded in the configuration. In addition, this stage was also meant for checking all of the instrumentation system prior to the actual commencement of the test.

Each specimen was loaded until there was an indication that a 'failure' has been obtained (refer to section 5.6), and so the test was brought to a stop. During all of the tests, there was no occurrence of any vertical slip at the interface between the endplate and the column. This was mainly due to the adequate tightness of the bolts carried out during the installation and after the initial stage of loading.

The unloading of the loads was done at about one third of the predicted loads for all specimens. The recovery of the loads in all specimens was in a linearly elastic manner, which corresponded to the initial stiffness of the connection. Even after failure, when releasing the applied loads, the recovery of the loads still corresponded to the initial stiffness of the connection.

The first visible deformation around the vicinity of the connection was limited to the tension region of the joint. For the flush endplate connection, the form of the deformation was the translation of the tip of the endplate away from the face of the column. This corresponded to the first sign of yielding of the endplate. The deformation of the connection appeared to be symmetrical on both sides of the connection. Upon unloading of the loads at about two third of the predicted loads, the deformation of the connection was not completely recovered. This indicated that a permanent deformation has occurred. Further loading of the specimens has resulted into more translation of the tip of the endplate followed by a slight elongation of the top row bolts and a slight buckling of the web around the tension region of the connection. Figure 6.1 shows the deformation of the flush endplate of specimen N1 that brought about the failure mode of the connection. Figure 6.2(a) and 6.2(b), show the deformation of the flush endplate connection of specimen N2 which was identical to the deformation of specimen N1.

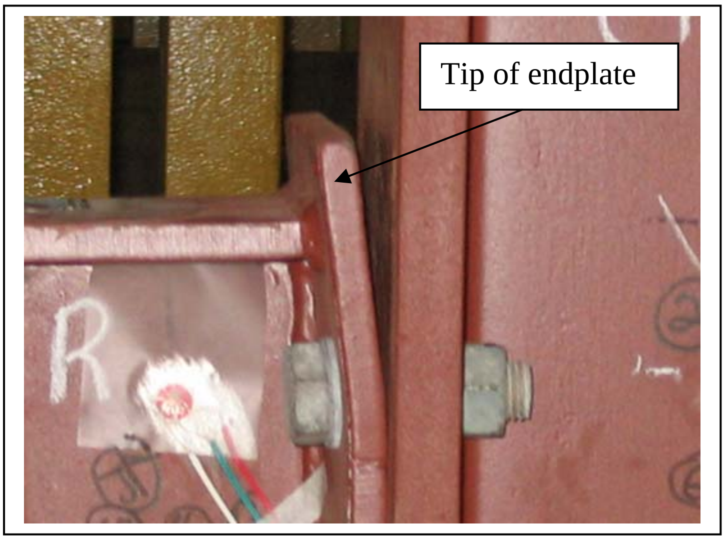
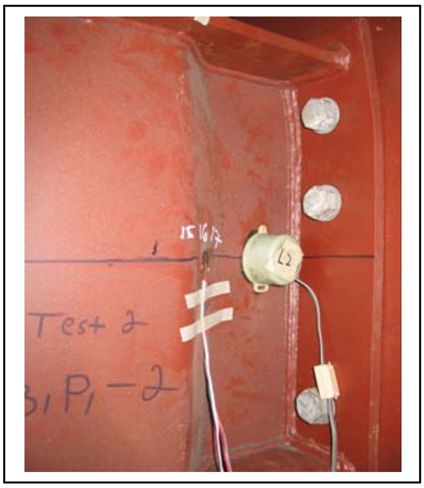


Figure 6.1: Deformation of flush endplate connection of specimen N1

(a)



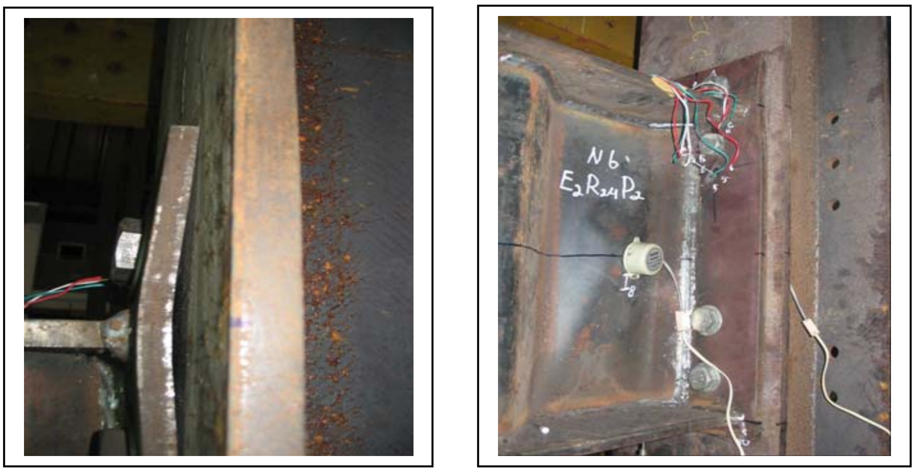
(b)

Figure 6.2: Deformation of flush endplate connection of specimen N2: (a) yielding of endplate plus (b) buckling of web

There was hardly any deformation on the columns throughout the experimental programme. This was also to be expected since the columns for all specimens (UC 254 x 254 x 107 for Phase 1 and UC 305 x 305 x 118 for Phase 2) were designed to adequately sustain the panel shear and the compression action along the bottom flange of the beam.

As for the Phase 2 tests, which consisted of four extended end plate connections and one flush endplate connection, the observation on the four extended endplate specimens would be described. The deformation and failure mode of this flush endplate connection was as described for specimens in Phase 1.

At the initial stage of loading, as in the flush endplate specimens, there was apparently no visible deformation even up to the one third of the predicted load. Higher capacity was expected for the extended endplate connections due to the addition of one row of bolts at the extended top portion of the endplate. Gradually, it was only after the unloading load that the tension region of the connection began to show some deformation. Unlike the flush endplate, since there existed one row of bolts at the extended top portion of the endplate, the deformation of the connection translated the endplate away from the face of the column in a ‘Y-shape’ form. Further increase of the load applied has not just deformed the endplate more but also has started to deform the rows of bolts above and below the top flange of the beam. The deformation of the endplate and the elongation of the bolts were then followed by some buckling on the web of the beam. Figure 6.3(a) shows the deformation of the extended endplate connection of specimen N5 at failure in the form of a ‘Y-shape’ deformation of the endplate and the slight elongation of the rows of bolts above and below the flange of the beam. Figure 6.3(b), on the other hand, shows the slight buckling of the web of the beam in addition to the deformation of the endplate and the elongation of the bolts of specimen N6.



(a) **(b)**

Figure 6.3: (a) Deformation of extended endplate connection of specimen N5, and (b) Buckling of web of specimen N6

Though there was hardly any deformation of the columns throughout the experimental programme, bigger beams especially in the Phase 2 experiments tend to exert more compression force along the bottom flange towards the face of the column. This was evidence through the noticeable lines of ‘skin tearing’ on the web of the column straight along the bottom flange of the beam.

6.2.1 Test Results

Results of all nine joint tests are best shown by the plots of load versus deflection (maximum) and moment versus rotation (of the joint). In addition other plots were also obtained even though most of them were not dealt with specifically. These plots are load versus deflection at other locations along the beam, load versus deflection of column, moment versus rotation of column, moment versus ratio of beam-to-column rotation, and the plots of load versus strain at all points that deemed significant of each specimen.

6.2.1.1 Load-Deflection Curves

Figure 6.4 to 6.12 show the load versus deflection at the location of the maximum deflection on the beam (DT4 for Phase 1 tests and DT1 for Phase 2 tests). The maximum load of each plot clearly represents the ultimate load that can be sustained by the respective joint. Table 6.1 summarises the results based on the plots of load versus deflection for all specimens.

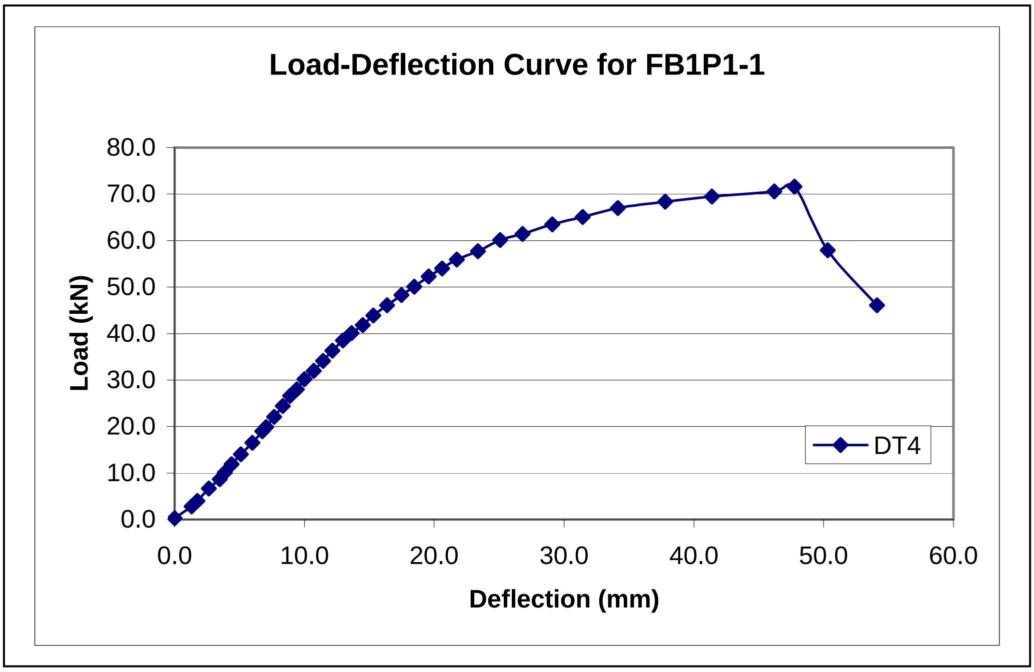


Figure 6.4: Load versus deflection for specimen N1 (FB1P1-1)

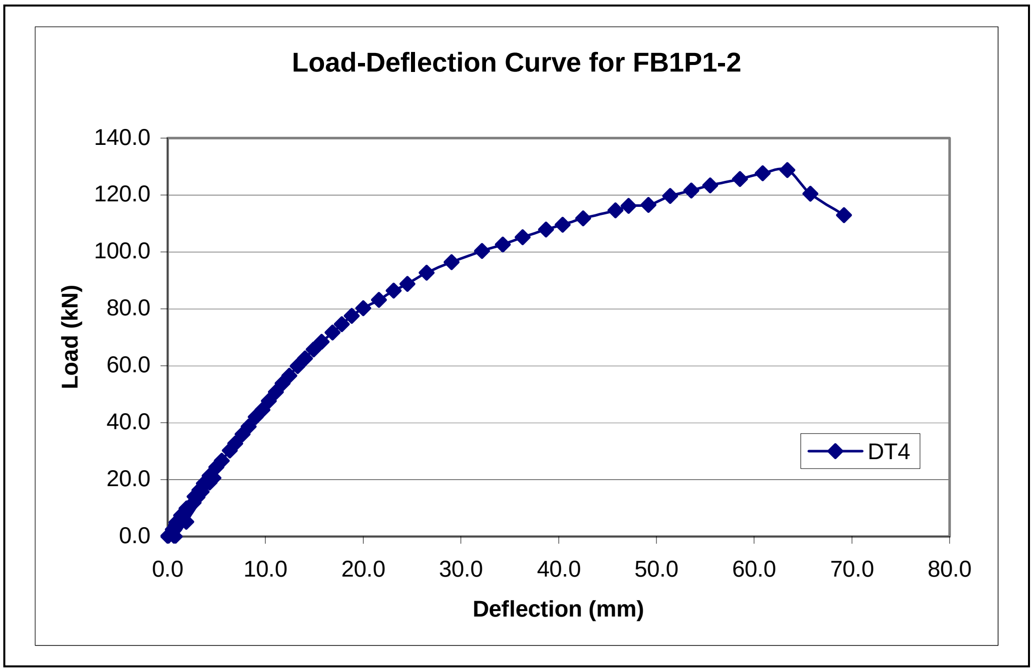


Figure 6.5: Load versus deflection for specimen N2 (FB1P1-2)

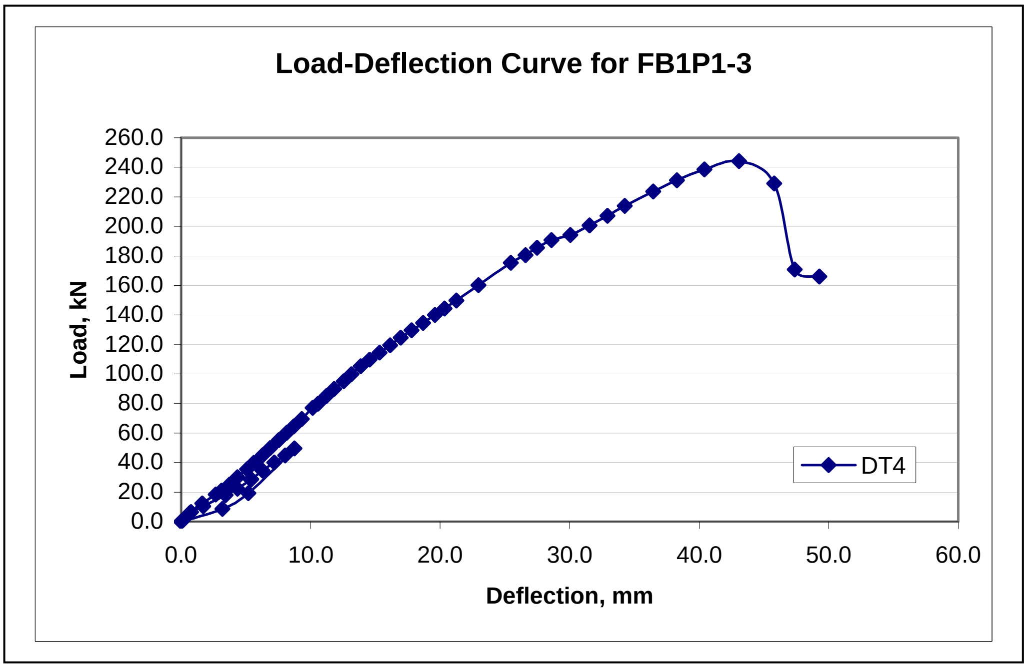


Figure 6.6: Load versus deflection for specimen N3 (FB1P1-3)

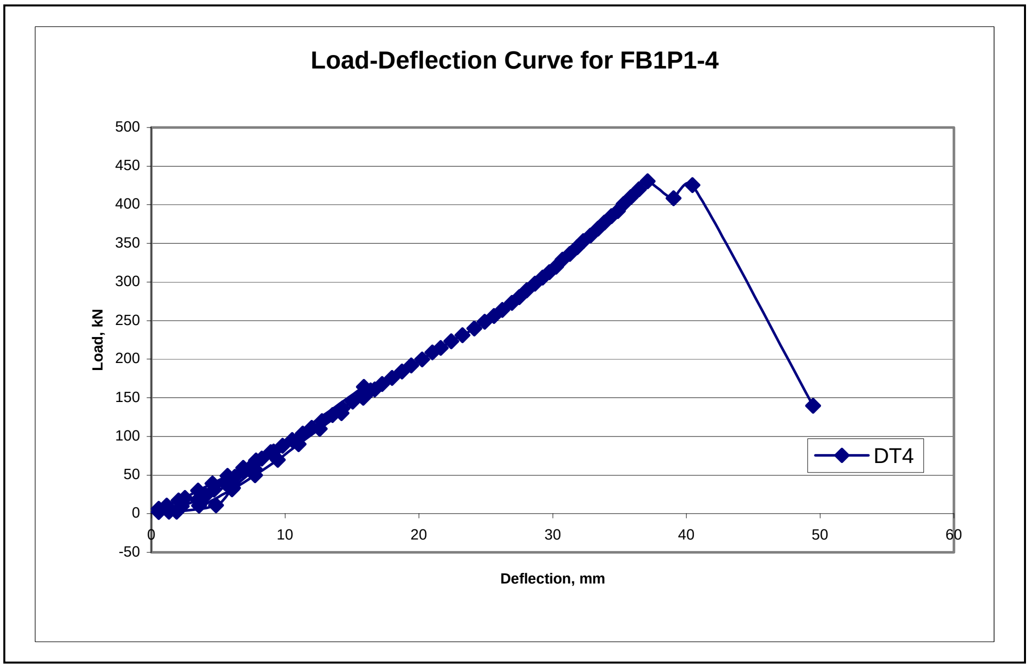


Figure 6.7: Load versus deflection for specimen N4 (FB1P1-4)

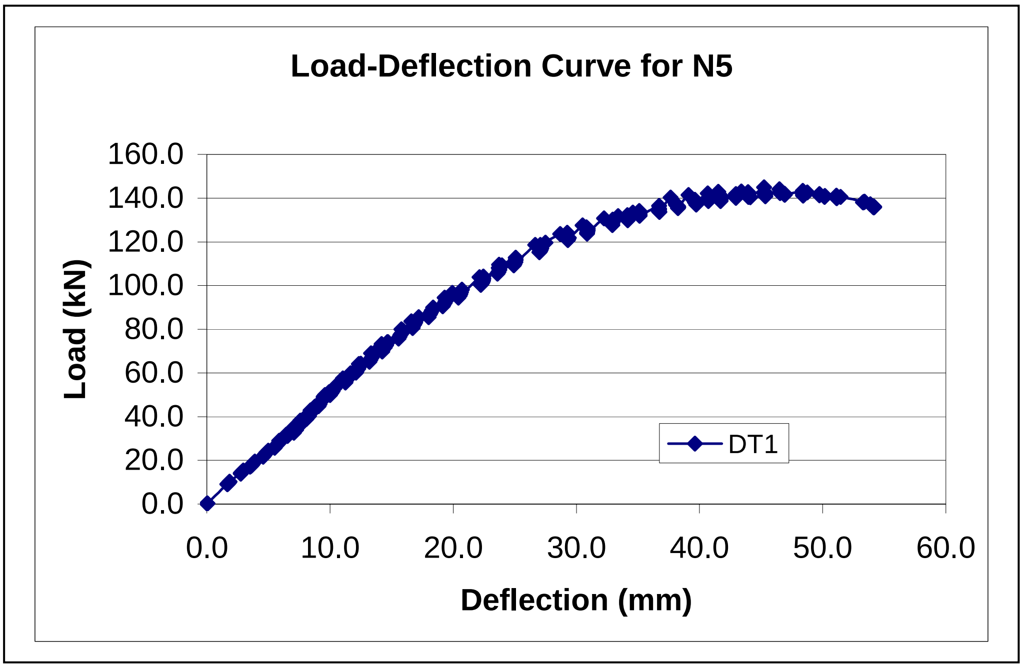


Figure 6.8: Load versus deflection for specimen N5 (E2R20P1)

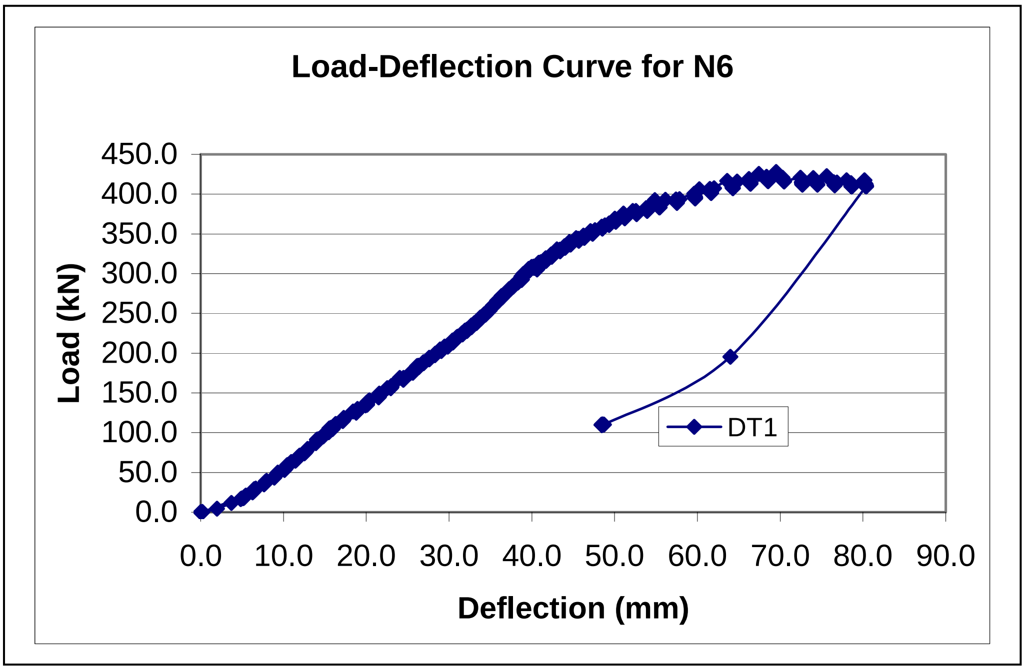


Figure 6.9: Load versus deflection for specimen N6 (E2R24P2)

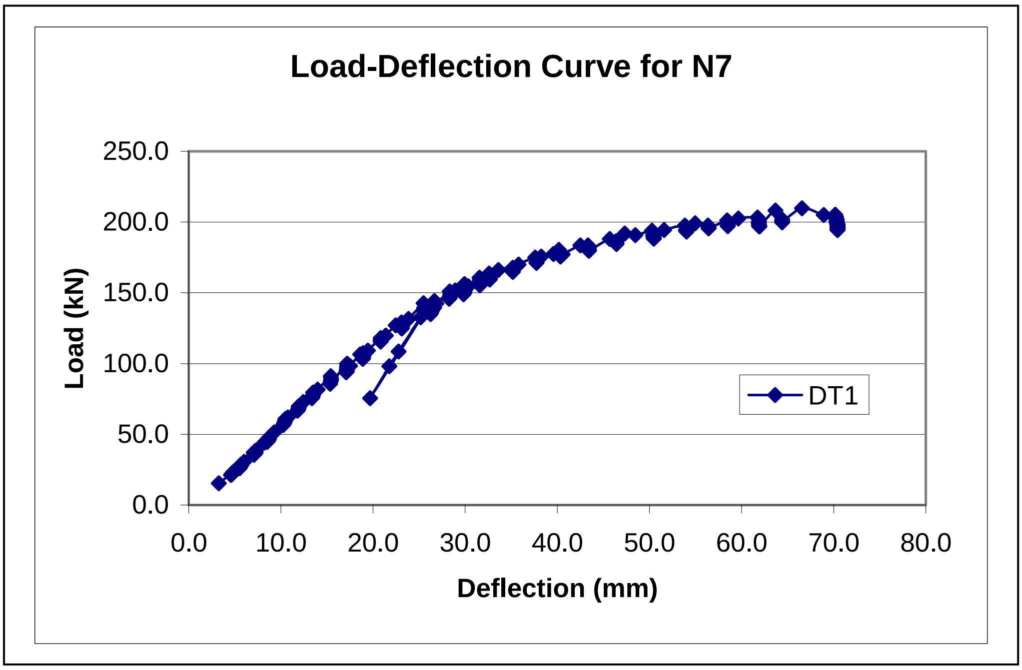


Figure 6.10: Load versus deflection for specimen N7 (3R20P1)

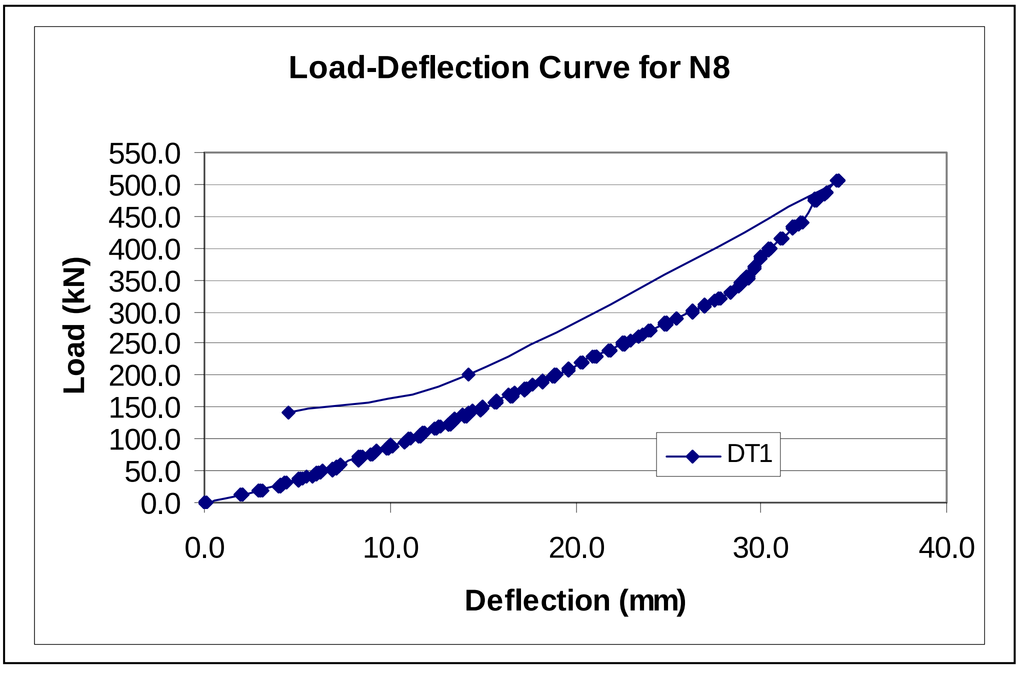


Figure 6.11: Load versus deflection for specimen N8 (3R24P2)

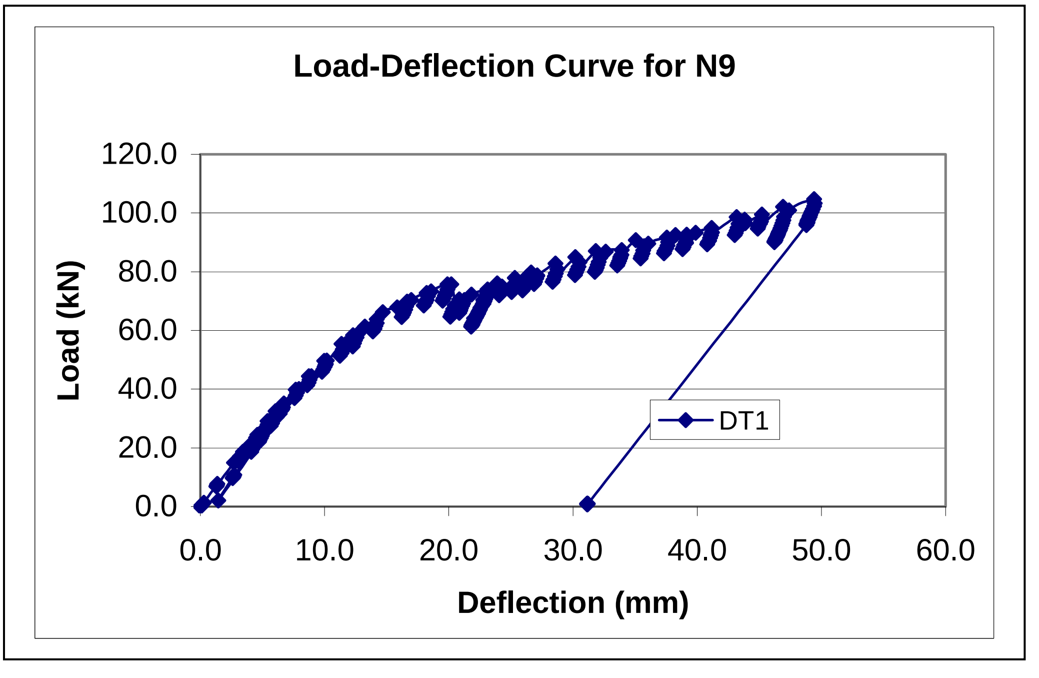


Figure 6.12: Load versus deflection for specimen N9 (F2R20P1)

Table 6.1: Test result based on the load versus deflection plots

REFERENCE	NAME	ULTIMATE LOAD	MAXIMUM DEFLECTION
N1	FB1P1-1	71.6kN	47.76mm
N2	FB1P1-2	128.8kN	63.40mm
N3	FB1P1-3	244.2kN	43.08mm
N4	FB1P1-4	425.4kN	40.44mm
N5	E2R20P1	144.3kN	54.10mm
N6	E2R24P2	425.0kN	80.2mm
N7	E3R20P1	207.3kN	69.56mm
N8	E3R24P2	506.2kN	34.16mm
N9	F3R20P1	105.5kN	49.38mm

6.2.1.2 Moment – Rotation Curves

Among all of the results obtained, the most important one is the moment versus rotation plot of a joint. From this plot, the behavioural characteristics of a particular joint can be determined based on the three significant parameters, which are the moment resistance (moment capacity), the stiffness (flexibility) and the rotational rigidity (ductility). Subsequently, the joint can be further classified according to these values into rigid, semi-rigid or pinned, and full strength or partial strength joint.

Figure 6.13 to 6.21 show the plots of moment versus rotation of all the nine joints in the full-scale isolated joint tests. Table 6.2, on the other hand, summarises the important values obtained from each plot of the tests.

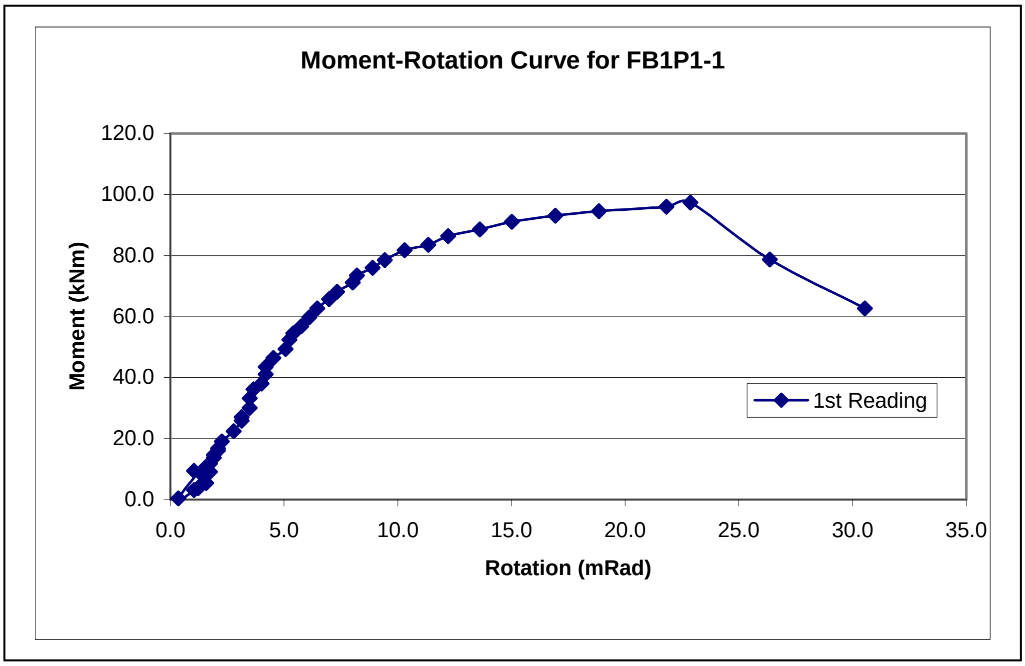


Figure 6.13: Moment versus rotation for specimen N1 (FB1P1-1)

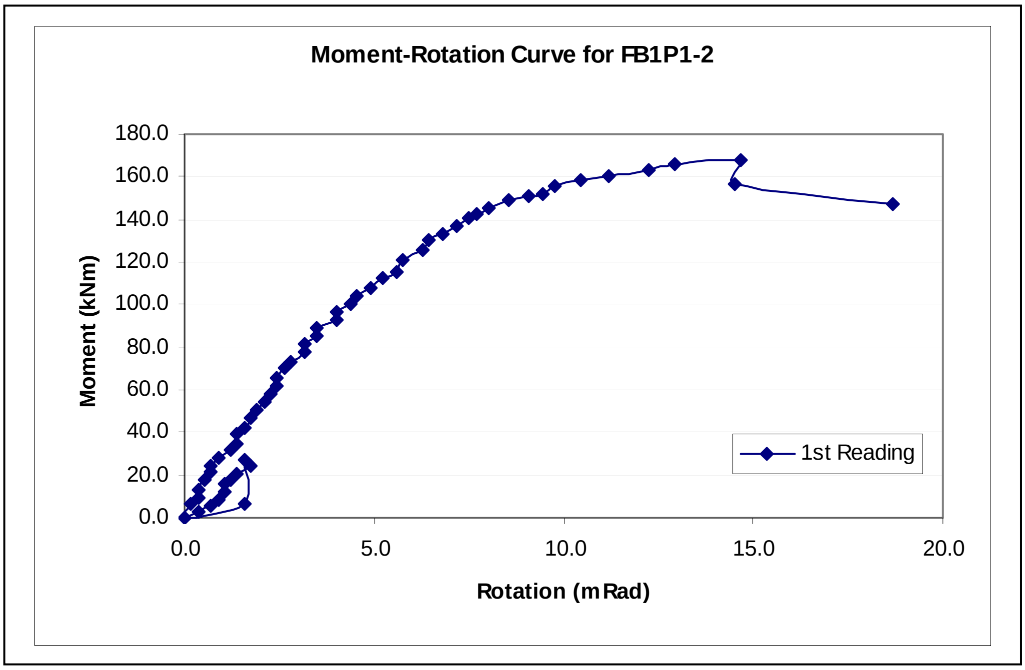


Figure 6.14: Moment versus rotation for specimen N2 (FB1P1-2)

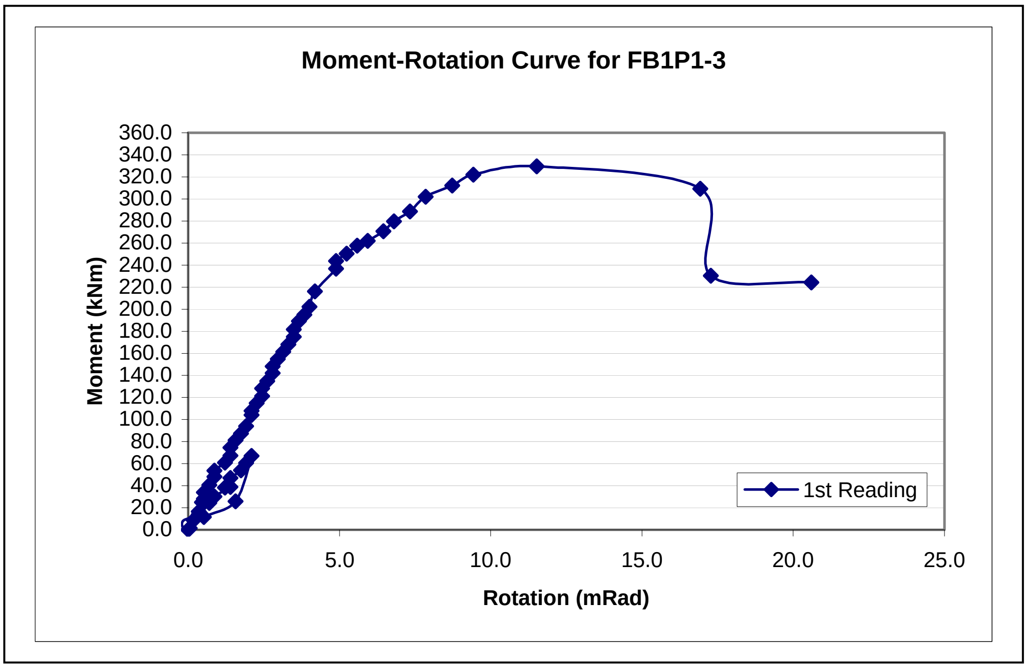


Figure 6.15: Moment versus rotation for specimen N3 (FB1P1-3)

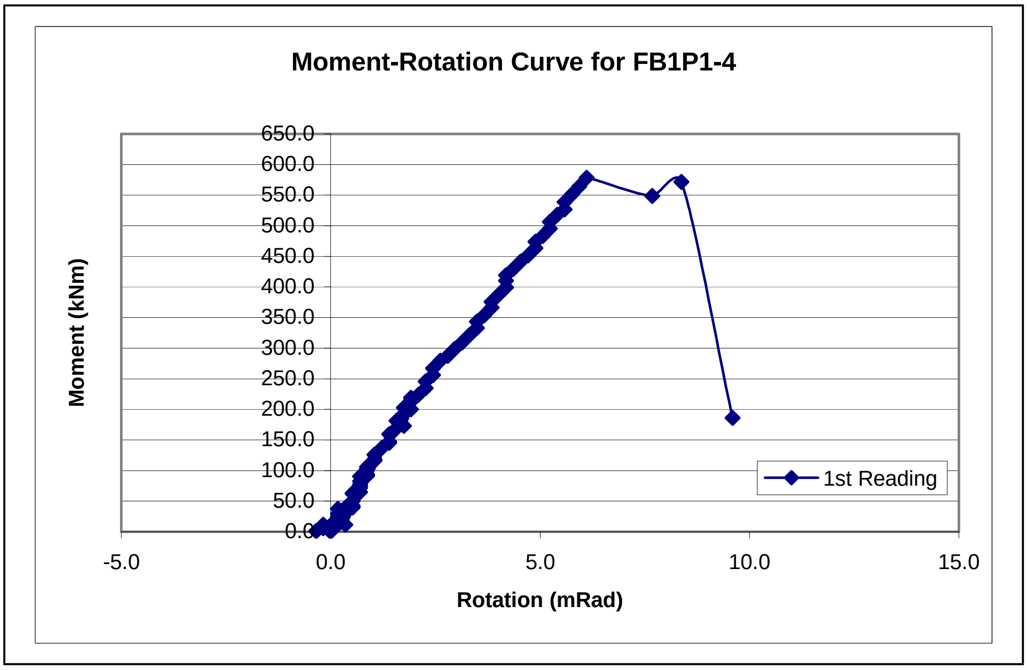


Figure 6.16: Moment versus rotation for specimen N4 (FB1P1-4)

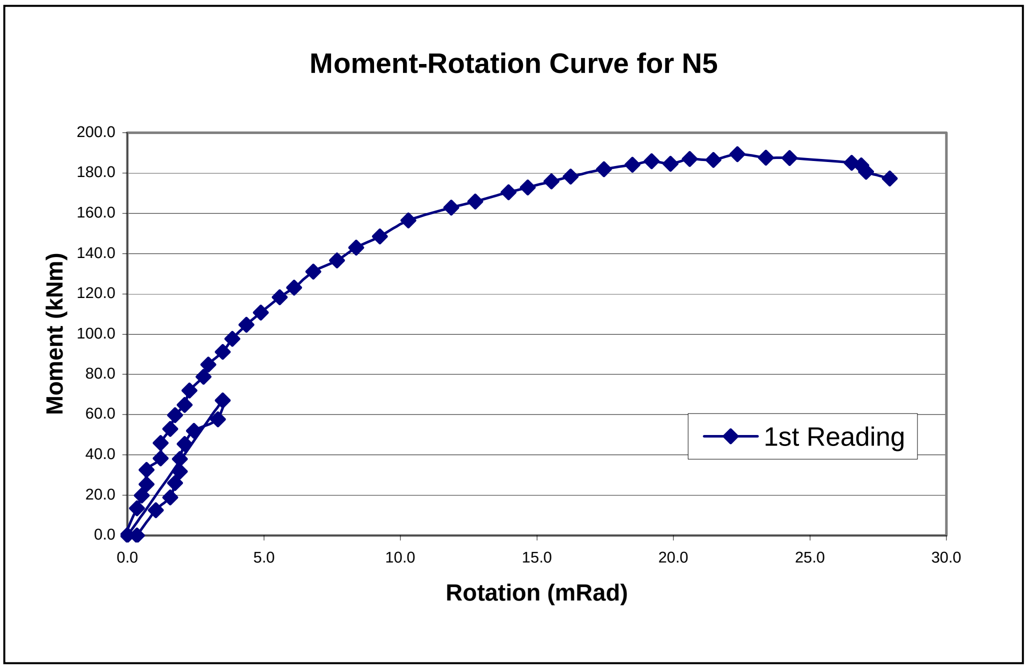


Figure 6.17: Moment versus rotation for specimen N5 (E2R20P1)

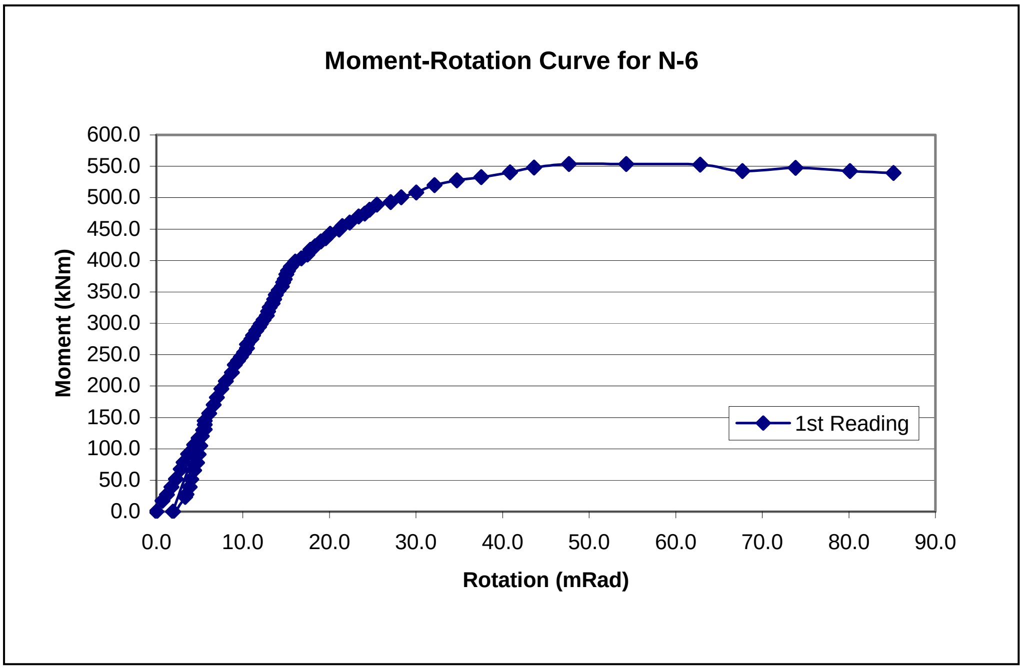


Figure 6.18: Moment versus rotation for specimen N6 (E2R24P2)

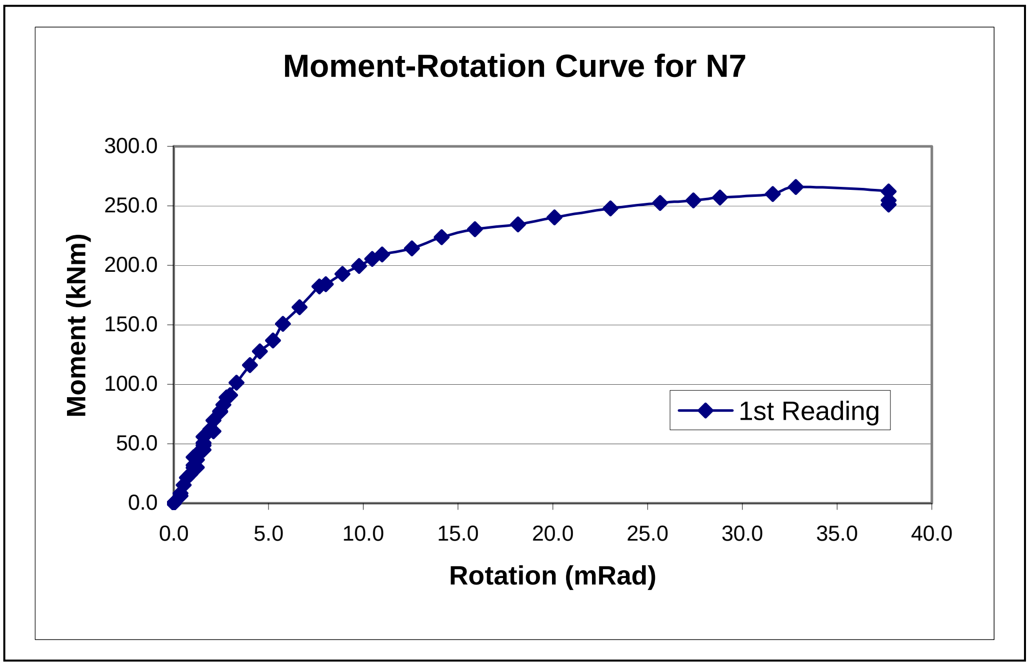


Figure 6.19: Moment versus rotation for specimen N7 (E3R20P1)

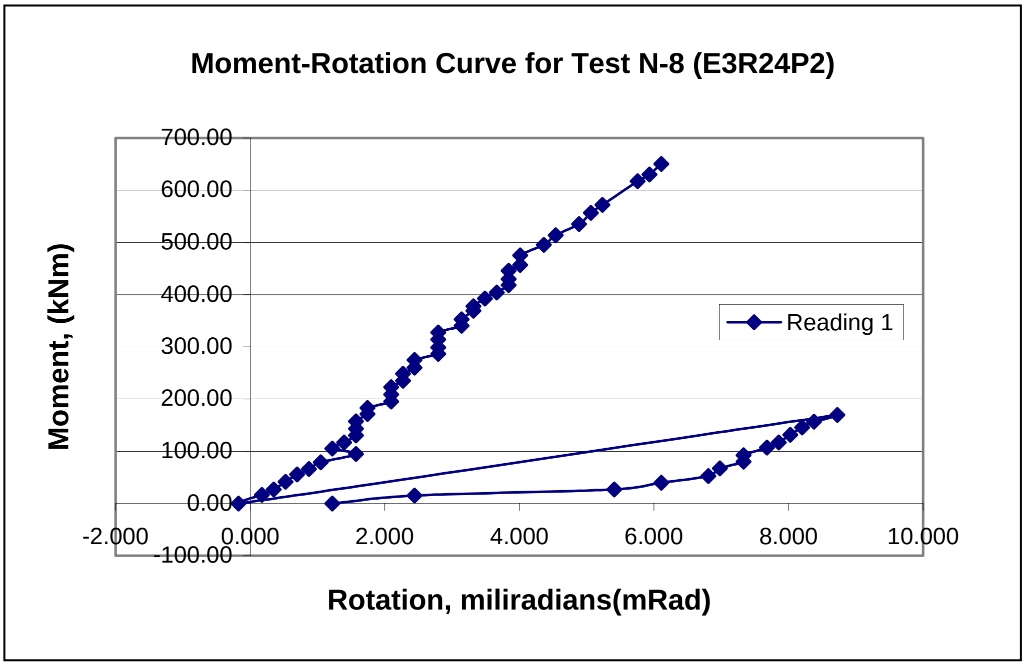


Figure 6.20: Moment versus rotation for specimen N8 (3R24P2)

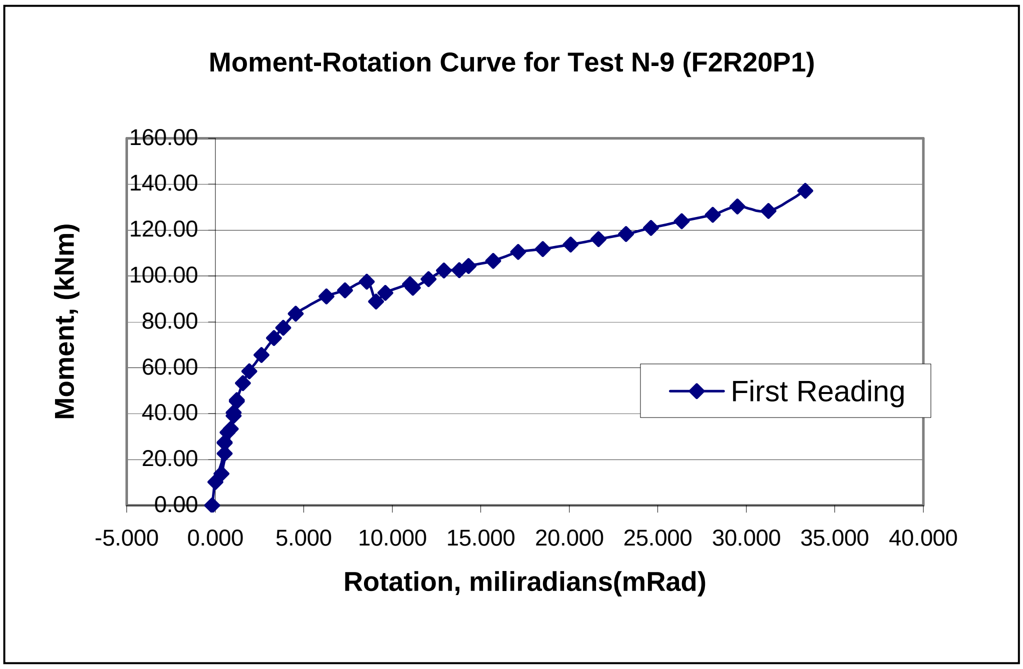


Figure 6.21: Moment versus rotation for specimen N9 (F3R20P1)

Table 6.2: Test result based on the moment versus rotation plots

REF	NAME	MOMENT ULTIMATE	MOMENT CAPACITY	ROTATION AT ULTIMATE	FAILURE MODE
N1	FB1P1-1	97.4	72	22.87	Yielding of end plate
N2	FB1P1-2	167.3	110	16.57	Yielding of end plate, buckling of beam web
N3	FB1P1-3	309.2	225	17.10	Yielding of end plate
N4	FB1P1-4	571.6	370	8.38	Yielding of end plate
N5	E2R20P1	189.4	132	27.93	Deformation of end plate
N6	E2R24P2	552.5	386	85.17	Deformation of end plate, buckling of beam web
N7	E3R20P1	263.5	187	37.41	Deformation of end plate
N8	E3R24P2	658.1	461	6.46	Deformation of end plate, buckling of beam web
N9	F3R20P1	137.2	96	33.34	Yielding of end plate

Shown in Figure 6.22 are the plots of the moment versus rotation for all tests on flush endplate connections Phase 1. Whereas Figure 6.23 shows the plots of moment versus rotation for all tests on extended endplate connections Phase 2. Moreover, Figure 6.24 shows the plots of moment versus rotation for specimen N7 (E2R20P1) and specimen N9 (F2R20P1). Furthermore, all of the components that made up the both connections were identical except that the endplate was flush for N9 but was extended for N7. Therefore, a comparison between the two types of connection could be made.

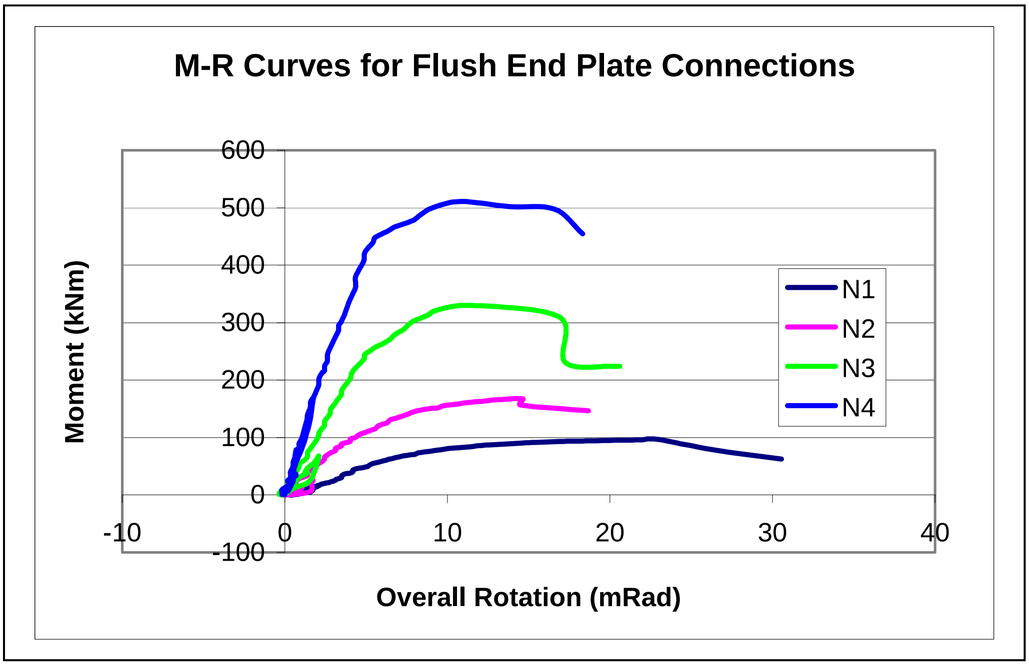


Figure 6.22: Moment versus rotation for all tests on FEP Phase 1

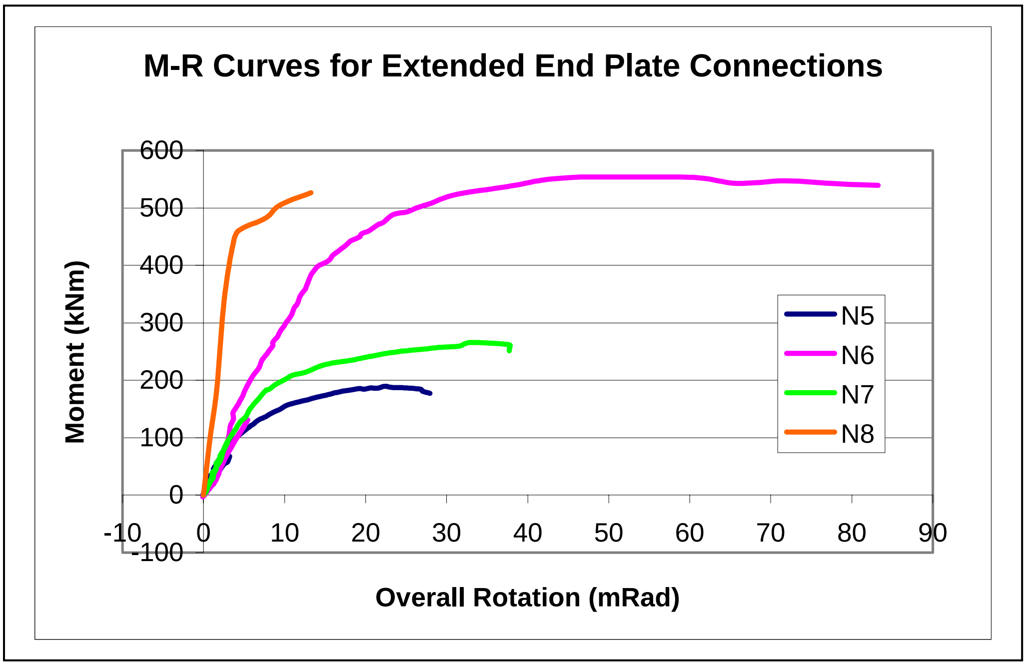


Figure 6.23: Moment versus rotation for all tests on EEP Phase 2

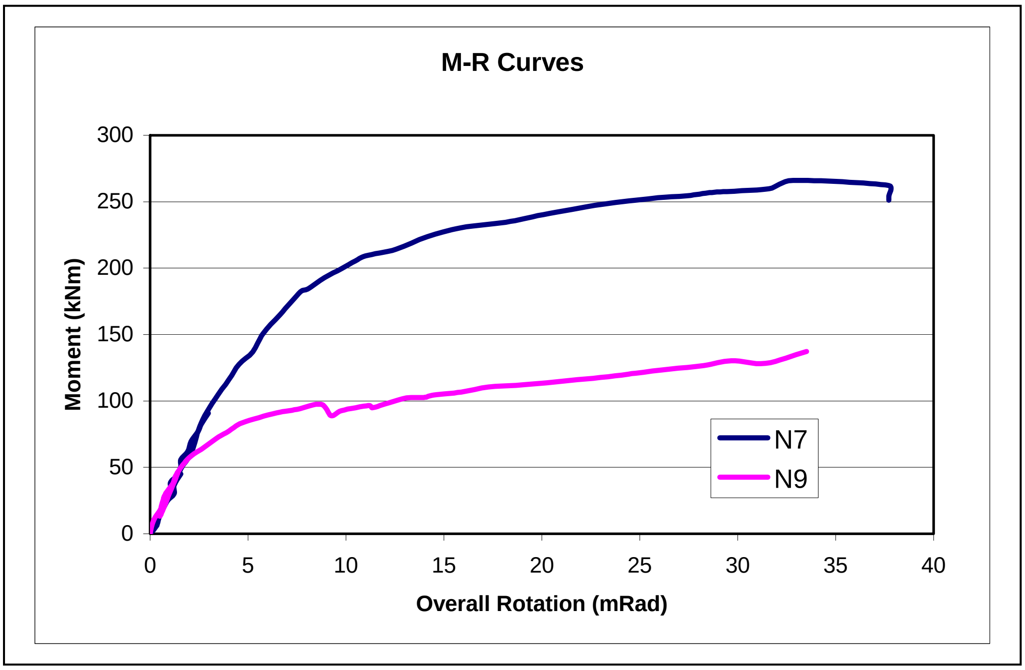


Figure 6.24: Moment versus rotation for specimen N7 (E2R20P1) and N9 (F2R20P1)

6.2.1.3 Strains

In order to gather as much information as possible from the full scale isolated tests, strain gauges of several types (rosette, bi-linear and linear) were placed at all possible locations that were deemed significant around the vicinity of the connections, the column and the beam. The data from these strain gauges could show the areas that were greatly affected by the applied load. The data could also show the location where yielding occurred first. Furthermore, from these data, loads and moments at any particular instances could be determined.

Details discussion as well as the analysis of results on the strain data would not be touched in depth in the scope of this research. The work will only be included as one of the recommendations for future work.

6.3 Standardised Partial Strength Connection Capacity Tables

The analytical approach adopted in generating the standardized partial strength connection capacity tables with TWP sections was based on the procedures outlined in the SCI publication (1995). The description of the component method used in the publication was presented in Annex J of EC 3 (DD ENV 1993-1-1: 1992 and BS EN 1993-1-8: 2005). In the process of checking the details of strength on bolts, welds, and steel sections, some of the requirements in the BS 5950-1:2000 have been employed. As indicated in Section 3.4 of Chapter 3, the checking on the capacity of the connections was categorised into three zones namely tension zone, compression zone, and shear zone as shown in Figure 3.3 in Chapter 3. The basic principles of the distribution of bolt forces need to be addressed first before details of the checking on all possible modes of failures can be discussed.

6.3.1 Distribution of Bolt Forces

The moment resistance of a connection transmitted by an end plate connection is through the coupling action between the tension forces in bolts and compression force at the centre of the bottom flange. Each bolt above the neutral axis of the beam produced tension force whereas the bolts below the neutral axis are dedicated to shear resistance only. EC 3 suggests that the bolt forces distribution should be based on the plastic distribution instead of the traditional triangular distribution. The forces of the bolt based on the plastic distribution are the actual value calculated from the critical zones mentioned above. Forces from the bolt rows at the top transmitted a series of tension forces to the endplate connection and in turn, resulted in a balanced compression force exerted at the bottom flange of the beam to the column. The endplate was connected to the web and both flanges by an all-around fillet welding. The formation of tension at the top and compression at the bottom contributes to the development of moment resistance of the connection. Tests on the connections have shown that the centre of compression flange, which bears against the column, was found to be the centre of rotation of the connection.

The force permitted in any bolt row was based on its potential resistance and not just the length of the lever arm.

6.3.2 Tension zone

The resistance at each bolt row in the tension zone may be limited due to the bending of column flange, endplate, column web, beam web, and the bolt strength. Column flange or endplate bending was checked by using the EC 3's procedures, which converts the complex pattern of yield lines around the bolts into a simple 'equivalent tee-stub'. Details of the procedures were described extensively in Chapter 3.

6.3.3 Compression zone

The procedures of checking in the compression zone were the same as the ones mentioned in BS 5950-1: 2000 which required checks on web bearing and web buckling. The compression failure modes can be either on the column side or on the beam side. The column side should be checked for web buckling and web bearing due to the compression force exerted on the column. The use of stiffeners or the effect of having other beams connected to the web of the column was not included so as to reduce the cost of fabrication and simplified the calculation. The compression on the beam side can usually be regarded as being carried entirely by the beam flange, however, when large moments combine with axial load, the compression zone will spread to the web of the beam which will affect the centre of compression. As a result, stiffening of the beam web needs to be done. However, in this study the moment resistance of the connection was not considering the use of stiffeners in order to reduce the cost of fabrication.

6.3.4 Shear zone

The column web can also fail due to the shearing effect of the tension and compression force applied to the web of the column. The failure caused by the shearing of the web is most likely to occur first before the failure due to bearing or buckling. This is due to the fact that the thickness of the column flange is much greater than the thickness of the column web. As in the compression zone, the use of stiffeners in the shear zone was also not considered so as to reduce the cost of fabrication.

6.3.5 Welding

Fillet weld was preferred than the butt weld in connecting the endplate to the beam. The position of the endplate, which was perpendicular to the cross section of the beam, made it suitable for an all-around fillet weld to be used. The end plate was fillet welded to the web of the beam on both sides using a suggested size of 8mm, whilst a 10mm fillet weld was suggested for connecting the endplate to the flanges. The weld was designed in such a way that the failure mode of the connection was not on the welding. This was to ensure that adequate ductility, which is necessary for a partial strength connection, could be achieved.

6.3.6 Validation of the standardised capacity tables

The validation of the standardised capacity tables with TWP sections is best presented by comparing the predicted values in the tables with full scale testing of the particular connections. In lieu with that, full scale isolated joint tests comprising of nine specimens altogether were conducted as described in detail in Chapter 5. Although the tests did not cover the whole range of the proposed connections, the comparison of the tests and the predicted values could still be established. Table 6.3 shows the comparison of moment capacities between the analytical and experimental results for four specimens of Phase 1. It was found out that the test results showed good agreement with the predicted values.

Table 6.3: Comparison of moment capacities for validation of standardised tables

TWP	Equivalent UB	Analytical (kNm)	Experimental (kNm)	% Difference
300 x 130 (FB1P1-1)	305 x 127	59	72	22.0
400 x 170 (FB1P1-2)	406 x 178	84	110	31.0
530 x 210 (FB1P1-3)	533 x 210	192	225	17.2
680 x 250 (FB1P1-4)	686 x 254	270	370	37.0

6.3.7 Explanation on the Notations Used in the Proposed Capacity Tables for Flush Endplate Connections

Six standardised capacity tables for flush endplate connections have been generated as shown in Table 6.4. A spreadsheet computer program using Excel software was developed to calculate and predict the moment capacity and shear capacity of the proposed standardised connections (listed as in Table 6.4). The values obtained were based on the critical zones' checks and the method as described in Chapter 3. Details capacities and other relevant information of the standardised capacity tables are tabulated in Table 6.5 to 6.10. The value of the moment capacity was calculated by summing up the multiplication of the force in each bolt row with the corresponding lever arm. The lever arm for the lowest row of tension bolts, which is labelled as 'Dimension A', was measured from the centreline of the row to the action line of the compressive force. The lever arm for the next row of tension bolts was measured as 'Dimension A' plus the distance between the two rows of which in this case was 90mm.

A tick in the tension zone of the 'Column Side' indicates that the column flange and web have a larger capacity than the respective bolt force as indicated in the 'Beam Side'. Nevertheless, if happens that the column carries a smaller capacity, a reduced bolt force was stated instead. The moment capacity, then, has to be re-calculated using the reduced bolt force or forces. Similarly, a tick in the compression zone indicates that the column web has a larger compressive capacity than the sum of the bolt row forces. A vertical shear capacity is the shear resistance of the bolts due to shearing of bolts, bearing of bolts, and bearing of plate from both the shear and tension zones.

Table 6.4. Configuration of the flush endplate connections for generating standardised tables

Designation	Row of Tension Bolts	Type of Bolts	Size of End Plates	Design Grade
FEP,1RM20,200W12T	1	M20 8.8	200 x 12	43
FEP,1RM24,200W15T	1	M24 8.8	200 x 15	43
FEP,2RM20,200W12T	2	M20 8.8	200 x 12	43
FEP,2RM20,250W12T	2	M20 8.8	250 x 12	43
FEP,2RM24,200W15T	2	M24 8.8	200 x 15	43
FEP,2RM24,250W12T	2	M24 8.8	250 x 12	43

All of the standardised tables (Table 7.5 to 7.10) depicted the geometrical configuration of the standard connection and provide the relevant capacities associated with the connection. Inside each table, the suggested sizes of columns and beams that can be used with the connection are listed out. The moment capacity was calculated in accordance to the size of beams. The smallest suggested size was taken as 300 x 120 due to the fact that it is non-economical to produce a TWP section that is smaller than 300 x 120. On the other hand, the largest suggested size was taken as 650 x 250 although a TWP section can be fabricated up to 1600mm deep. This is to ensure that the ductility of the connection, which is important for partial strength connections, is maintained. As mentioned in the previous section, the shear capacity of the connection is determined from the shear capacity of the tension bolt rows and bottom bolt rows. However, the bottom bolt rows were designed to carry most of the vertical shear force. The value of moment capacities depends on the size of bolts, number of bolts, size of endplate, and thickness of endplate. For easy identifying, special notation was used for each designated connection. For example, the connection in Table 7.5 is designated as FEP, 1RM20, 200W12T, which reads that it is a flush endplate connection with one row of M20 grade 8.8 bolt, and an endplate size of 200mm wide and 12mm thick. Comparisons of moment capacities based on the different geometrical configuration of the connections are discussed in the subsequent sections.

Table 6.5: Standardised table for flush endplate connection (FEP, 1RM20, 200W12T)

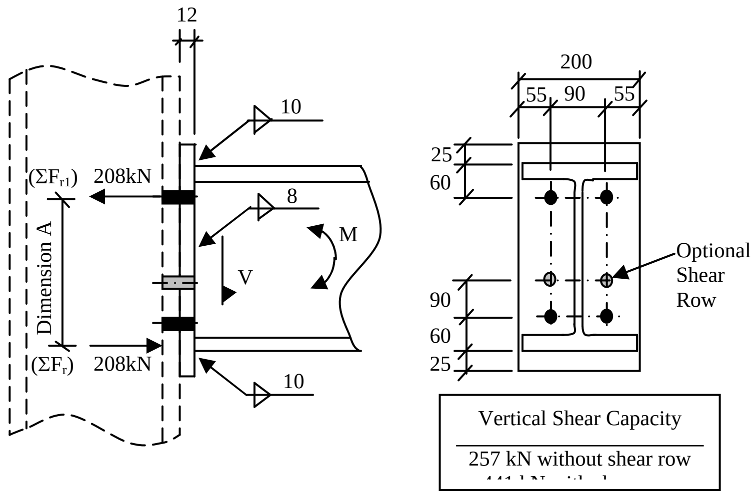
1 ROW M20 8.8 B0LTS

200 x 12 DESIGN GRADE 43 FLUSH END PLATE

BEAM – FLANGE S355
WEB S275

Beam Side

Beam Serial Size DxBxkg/m (T/t)	Dimension 'A' (mm)	Moment Capacity (kNm)
300 x 120 x 26.3 (10/3)	235	48
300 x 140 x 33.7 (12/3)	234	49
350 x 120 x 27.6 (10/3)	285	59
350 x 140 x 35.1 (12/3)	284	59
400 x 140 x 39.7 (12/4)	334	69
400 x 160 x 48.4 (14/4)	333	69
450 x 160 x 50.2 (14/4)	383	79
450 x 180 x 60.1 (16/4)	382	79
500 x 160 x 52.0 (14/4)	433	90
500 x 180 x 61.9 (16/4)	432	90
550 x 200 x 73.3 (16/5)	482	100
600 x 200 x 80.5 (16/6)	532	110



Column Side

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

Tension Zone:

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

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

Compression Zone:

√ Column capacity exceeds ΣF_r.

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

Table 6.6 Standardised table for flush endplate connection (FEP, 1RM24, 200W15T)

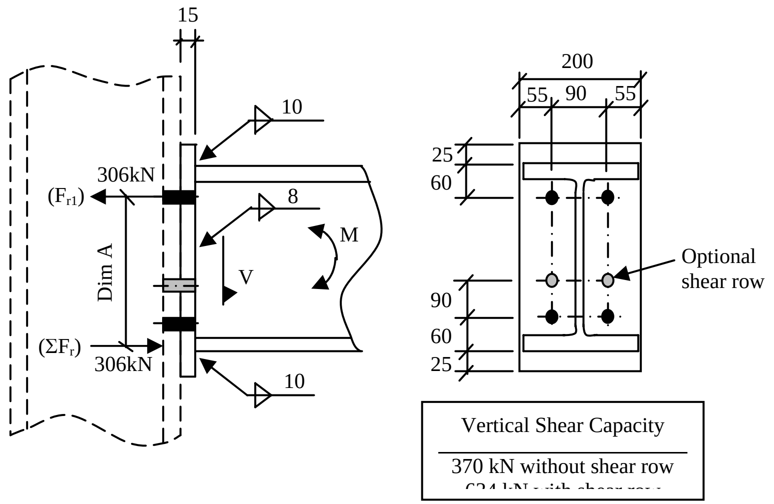
1 ROW M24 8.8 B0LTS

200 x 15 DESIGN GRADE 43 FLUSH END PLATE

BEAM – FLANGE S355
WEB S275

Beam Side

Beam Serial Size DxBxkg/m (T/t)	Dimension 'A' (mm)	Moment Capacity (kNm)
300 x 120 x 26.3 (10/3)	235	71
300 x 140 x 33.7 (12/3)	234	72
350 x 120 x 27.6 (10/3)	285	86
350 x 140 x 35.1 (12/3)	284	87
400 x 140 x 39.7 (12/4)	334	102
400 x 160 x 48.4 (14/4)	333	102
450 x 160 x 50.2 (14/4)	383	117
450 x 180 x 60.1 (16/4)	382	117
500 x 160 x 52.0 (14/4)	433	133
500 x 180 x 61.9 (16/4)	432	132
550 x 200 x 73.3 (16/5)	482	148
600 x 200 x 80.5 (16/6)	532	163



Column Side	DESIGN GRADE S275			COLUMN	DESIGN GRADE S355		
	Panel Shear Capacity (kN)	Tension Zone F_{R1} (kN)	Compn. Zone	Serial Size	Compn. Zone	Tension Zone F_{R1} (kN)	Panel Shear Capacity (kN)
	1000	√	√	356 x 368 x 202	√	√	1302
	849	√	√	177	√	√	1105
	725	√	√	153	√	√	944
	605	√	√	129	√	√	787
	1037	√	√	305 x 305 x 198	√	√	1350
	816	√	√	158	√	√	1062
	703	√	√	305 x 305 x 137	√	√	915
	595	√	√	118	√	√	774
	503	√	√	97	√	√	649
	882	√	√	254 x 254 x 167	√	√	1149
	685	√	√	132	√	√	892
	551	√	√	107	√	√	717
	434	√	√	89	√	√	566
	360	297	√	73	√	√	465
	459	√	√	203 x 203 x 86	√	√	598
	353	√	√	71	√	√	460
	322	297	√	60	√	√	415
	272	265	√	52	√	296	351
	245	204	√	46	√	263	316

Tension Zone:

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

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

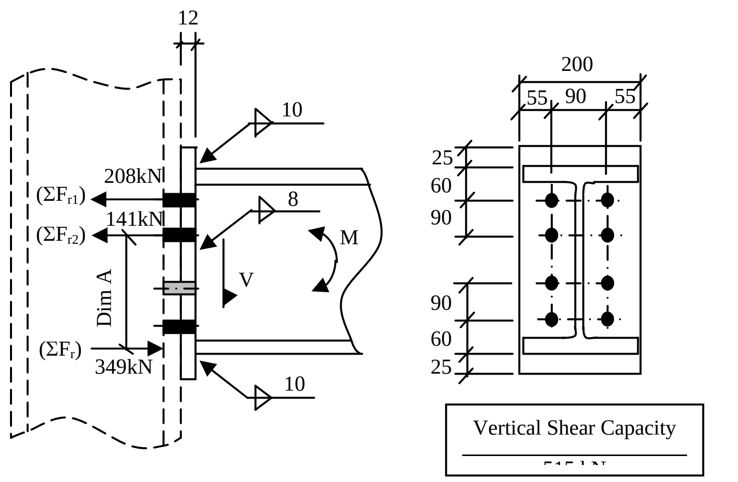
Compression Zone:

√ Column capacity exceeds ΣF_r.

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

Table 6.7 Standardised table for flush endplate connection (FEP, 2RM20, 200W12T)

2 ROWS M20 8.8 B0LTS			
200 x 12 DESIGN GRADE 43 FLUSH END PLATE			
	BEAM – FLANGE S355 WEB S275		
	Beam Serial Size DxBxkg/m (T/t)	Dimension 'A' (mm)	Moment Capacity (kNm)
Beam Side	400 x 140 x 39.7 (12/4)	244	103
	400 x 160 x 48.4 (14/4)	243	103
	450 x 160 x 50.2 (14/4)	293	120
	450 x 180 x 60.1 (16/4)	292	120
	500 x 160 x 52.0 (14/4)	343	137
	500 x 180 x 61.9 (16/4)	342	137
	550 x 200 x 73.3 (16/5)	392	155
	600 x 200 x 80.5 (16/6)	442	173
	650 x 250 x 103.4 (18/6)	491	190



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

Tension Zone:

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

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

Compression Zone:

√ Column capacity exceeds ΣF_r.

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

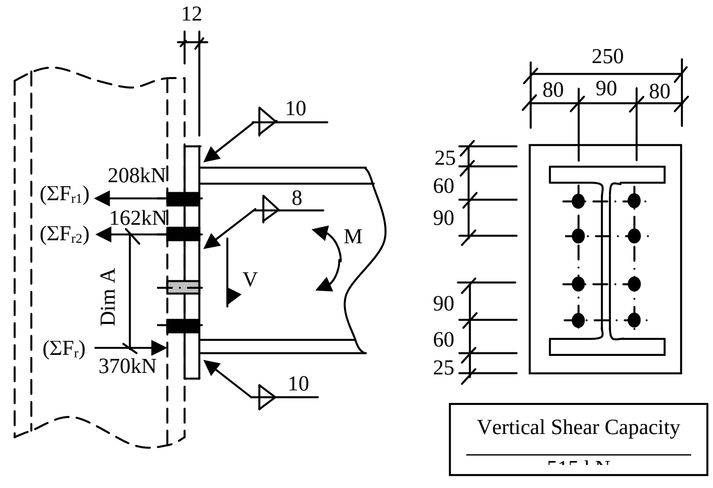
Table 6.8: Standardised table for flush endplate connection (FEP, 2RM20, 250W12T)

2 ROWS M20 8.8 B0LTS

250 x 12 DESIGN GRADE 43 FLUSH END PLATE

BEAM – FLANGE S355 WEB S275

Beam Side	Beam Serial Size DxBxkg/m (T/t)	Dimension 'A' (mm)	Moment Capacity (kNm)
	400 x 140 x 39.7 (12/4)	244	109
	400 x 160 x 48.4 (14/4)	243	109
	450 x 160 x 50.2 (14/4)	293	125
	500 x 160 x 52.0 (14/4)	343	144
	500 x 180 x 61.9 (16/4)	342	143
	550 x 200 x 73.3 (16/5)	392	162
	600 x 200 x 80.5 (16/6)	442	182
	650 x 250 x 103.4 (18/6)	491	200



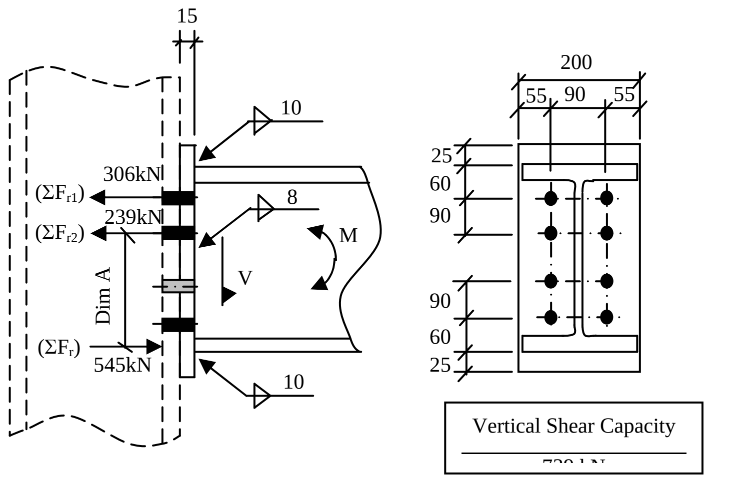
Column Side

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

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

Table 6.9: Standardised table for flush endplate connection (FEP, 2RM24, 200W12T)

	2 ROWS M24 8.8 B0LTS 200 x 15 DESIGN GRADE 43 FLUSH END PLATE		
	BEAM – FLANGE S355, WEB S275		
	Beam Serial Size DxBxkg/m (T/t)	Dimension 'A' (mm)	Moment Capacity (kNm)
Beam Side	350 x 120 x 27.6 (10/3)	195	134
	350 x 140 x 35.1 (12/3)	195	134
	400 x 140 x 39.7 (12/4)	244	160
	400 x 160 x 48.4 (14/4)	243	160
	450 x 160 x 50.2 (14/4)	293	187
	450 x 180 x 60.1 (16/4)	292	187
	500 x 160 x 52.0 (14/4)	343	214
	500 x 180 x 61.9 (16/4)	342	214
	550 x 200 x 73.3 (16/5)	392	241
	600 x 200 x 80.5 (16/6)	442	269
	650 x 250 x 103.4 (18/6)	491	295



Column Side	DESIGN GRADE S275				COLUMN	DESIGN GRADE S355			
	Panel Shear Capacity (kN)	Tension Zone		Compn. Zone	Serial Size	Compn. Zone	Tension Zone		Panel Shear Capacity (kN)
		F_{R1} (kN)	F_{R2} (kN)				F_{R1} (kN)	F_{R2} (kN)	
	1000	√	√	√	356 x 368 x 202	√	√	√	1302
	849	√	√	√	177	√	√	√	1105
	725	√	√	√	153	√	√	√	944
	605	√	√	√	129	√	√	√	787
	1037	√	√	√	305 x 305 x 198	√	√	√	1350
	816	√	√	√	158	√	√	√	1062
	703	√	√	√	137	√	√	√	915
	595	√	√	√	305 x 305 x 118	√	√	√	774
	503	√	√	√	97	√	√	√	649
	882	√	√	√	254 x 254 x 167	√	√	√	1149
	685	√	√	√	132	√	√	√	892
	551	√	√	√	107	√	√	√	717
	434	√	√	√	89	√	√	√	566
	360	297	√	S(484)	73	√	√	√	465
	459	√	√	√	203 x 203 x 86	√	√	√	598
	353	√	√	√	71	√	√	√	460
	322	297	204	S(491)	60	√	√	√	415
	272	265	118	√	52	√	296	198	351
	245	204	90	√	46	√	263	116	316

Tension Zone:

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

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

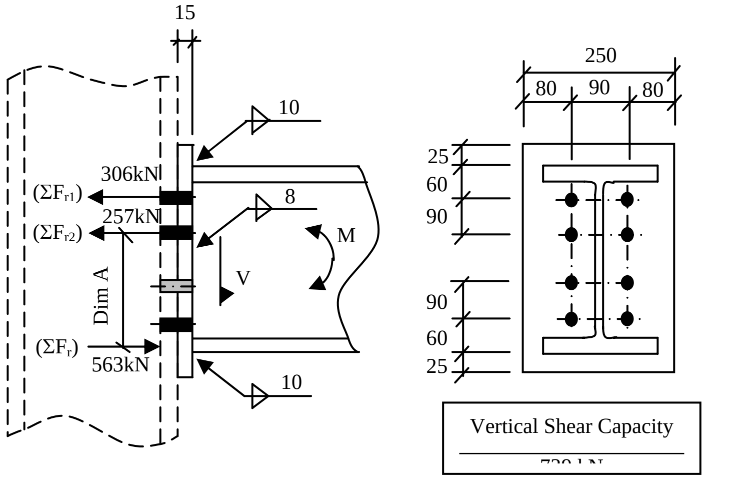
Compression Zone:

√ Column capacity exceeds ΣF_r.

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

Table 6.10: Standardised table for flush endplate connection (FEP, 2RM24, 250W15T)

Beam Side	2 ROWS M24 8.8 B0LTS 250 x 15 DESIGN GRADE 43 FLUSH END PLATE		
	BEAM – FLANGE S355, WEB S275		
	Beam Serial Size DxBxkg/m (T/t)	Dimension ‘A’ (mm)	Moment Capacity (kNm)
	400 x 140 x 39.7 (12/4)	244	165
	400 x 160 x 48.4 (14/4)	243	165
	450 x 160 x 50.2 (14/4)	293	192
	450 x 180 x 60.1 (16/4)	292	192
	500 x 180 x 61.9 (16/4)	342	220
	500 x 180 x 61.9 (16/4)	342	220
	550 x 200 x 73.3 (16/5)	392	248
	600 x 200 x 80.5 (16/6)	442	276
	650 x 250 x 103.4 (18/6)	491	304



Column Side

DESIGN GRADE S275				COLUMN		DESIGN GRADE S355			
Panel Shear Capacity (kN)	Tension Zone		Compn. Zone	Serial Size		Compn. Zone	Tension Zone		Panel Shear Capacity (kN)
	F_{R1} (kN)	F_{R2} (kN)					F_{R1} (kN)	F_{R2} (kN)	
1000	√	√	√	356 x 368 x	202	√	√	√	1302
849	√	√	√		177	√	√	√	1105
725	√	√	√		153	√	√	√	944
605	√	√	√		129	√	√	√	787
1037	√	√	√	305 x 305 x	198	√	√	√	1350
816	√	√	√		158	√	√	√	1062
703	√	√	√		137	√	√	√	915
595	√	√	√	305 x 305 x	118	√	√	√	774
503	√	√	√		97	√	√	√	649
882	√	√	√	254 x 254 x	167	√	√	√	1149
685	√	√	√		132	√	√	√	892
551	√	√	√		107	√	√	√	717
434	√	√	√		89	√	√	√	566
360	297	√	S(479)		73	√	√	√	465
459	√	√	√	203 x 203 x	86	√	√	√	598
353	√	√	S(561)		71	√	√	√	460
322	297	204	S(486)		60	√	√	√	415
272	265	118	√		52	√	296	198	351
245	204	90	√		46	√	263	116	316

Tension Zone:

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

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

Compression Zone:

√ Column capacity exceeds ΣF_r.

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

6.3.7.1 Effect of increasing the number of bolt rows from one row to two rows [(FEP,1RM20,200W12T) versus (FEP,2RM20,200W12T) and (FEP,1RM24,200W15T) versus (FEP,2RM24,200W15T)]

Table 6.5 and 6.6 show the moment capacity of the connection for single bolt row whereas Table 6.7, 6.8, 6.9 and 6.10 show the moment capacity of the connection for double bolt rows. The results of percentage increase in moment capacity for one and two bolt rows are shown in Table 6.11. The results indicate that by increasing the number of bolt row from one to two, the moment capacity of the connection is increased by an average of 51.2% for M20 bolt with 12mm thick endplate and an average of 59.3% for M24 bolt with 15mm thick endplate. The combination of M24 with 15mm thick endplate has contributed to the increase in the moment capacity of the connection. The increase in moment capacity is also very much linear to the depth of the beam. Hence, the moment capacity of a connection depends on the depth of the beam, the number and size of bolts, and the thickness of the endplate.

The vertical shear capacity of the connection in Table 6.5 increased from 258kN without the optional shear bolt row to 442kN with the optional shear bolt row. The increment of the shear capacity was not exactly double since the determination of the shear capacity depends also on the number of rows of the tension bolts. As for the connection in Table 6.7, the vertical shear capacity of the connection is 515kN with shear bolt row. This value is twice the vertical shear capacity of the connection in Table 6.5 without optional shear bolt row. This is because the number of bolt rows at the tension zone of the connection in Table 6.7 is two.

Panel shear capacities for connection in Table 6.5 and Table 6.7 are the same since the sizes of columns are the same. In addition, the tension and compression forces exerted on the column web were not large enough to change the calculated values.

6.3.7.2 Effect of increasing the size of endplate from 200mm to 250mm

[(FEP,2RM20,200W12T) versus (FEP,2RM20,250W12T) and
(FEP,2RM24,200W15T) versus (FEP,2RM24,200W15T)]

Table 6.7 and 6.9 show the moment capacities of a connection with an endplate width of 200mm. Table 6.8 and 6.10, on the other hand, show the moment capacities of a connection with an endplate width of 250mm. The idea of comparison is to know the percentage increase due to the increment of the endplate width. The results of percentage increase in moment capacity for 200mm and 250mm wide endplates are tabulated in Table 6.12. The results show that by increasing the size of endplate width from 200mm to 250mm, the moment capacity of the connection is increased by an average of 5.1% for M20 bolt with 12mm thick endplate and an average of 2.7% for M24 bolt with 15mm thick end-plate. The results show that the increment of the plate size from 200 to 250mm has contributed to a marginal amount of moment capacity to the connection. For M24 bolt, the increment in moment capacity is reduced by almost half of M20 bolt. This shows that the moment capacity of the connection depends on the strength of the bolt more than the strength of the endplate.

6.3.7.4 Effect of increasing the size of bolt from M20 with 12mm thick endplate to M24 with 15mm thick endplate

[(FEP,1RM20,200W12T) versus (FEP,1RM24,200W15T) and
(FEP,2RM20,200W12T) versus (FEP,2RM24,200W15T)]

The need to compare the result is to know the percentage increase due to increment of the size of bolt and thickness of the endplate. The results of percentage increase in moment capacity for M20 with 12mm thick endplate and M24 with 15mm thick endplate are tabulated in Table 6.13. The results show that by increasing the size of bolt from M20 with 12mm thick end-plate to M24 with 15mm thick end-plate, the moment capacity of the connection is increased by an average of 47.8% for one bolt row and 55.6% for two bolt rows. This indicates that the moment capacity of the connection depends on the strength of the bolt more than the strength of the endplate.

Table 6.11. Percentage increase in moment capacity due to the increasing number of bolt rows (FEP)

	(FEP,1RM20,200W12T) versus (FEP,2RM20,200W12T)			(FEP,1RM24,200W15T) versus (FEP,2RM24,200W15T)		
Size of TWP beam	One bolt row	Two bolt rows	% increase	One bolt row	Two bolt rows	% increase
350 x 120 x 27.6 (10/3)	59	86	45.8	86	134	55.8
350 x 140 x 35.1 (12/3)	59	87	47.5	87	134	54.0
400 x 140 x 39.7 (12/4)	69	103	49.3	102	160	56.9
400 x 160 x 48.4 (14/4)	69	103	49.3	102	160	56.9
450 x 160 x 50.2 (14/4)	79	120	51.9	117	187	59.8
450 x 180 x 60.1 (16/4)	79	120	51.9	117	187	59.8
500 x 160 x 52.0 (14/4)	90	137	52.2	133	214	60.9
500 x 180 x 61.9 (16/4)	90	137	52.2	132	214	62.1
550 x 200 x 73.3 (16/5)	100	155	55.0	148	241	62.3
600 x 200 x 80.5 (16/6)	110	173	57.3	163	268	64.4
	Average		**51.2**	Average		**59.3**

Table 6.12. Percentage increase in moment capacity due to the increasing size of endplate (FEP)

	(FEP,2RM20,200W12T) versus (FEP,2RM20,250W12T)			(FEP,2RM24,200W15T) versus (FEP,2RM24,250W15T)		
Size of TWP beam	250mm	200mm	% increase	250mm	200mm	% increase
400 x 140 x 39.7 (12/4)	109	103	5.8	164	160	2.5
400 x 160 x 48.4 (14/4)	109	103	5.8	164	160	2.5
450 x 160 x 50.2 (14/4)	125	120	4.2	192	187	2.7
450 x 180 x 60.1 (16/4)	125	120	4.2	192	187	2.7
500 x 160 x 52.0 (14/4)	144	137	5.1	220	214	2.8
500 x 180 x 61.9 (16/4)	143	137	5.8	220	214	2.8
550 x 200 x 73.3 (16/5)	162	155	4.5	248	241	2.9
600 x 200 x 80.5 (16/6)	182	173	5.2	276	268	2.9
		Average	**5.1**		Average	**2.7**

Table 6.13. Percentage increase in moment capacity due to the increasing size of bolts/endplate thickness (FEP)

	(FEP,1RM20,200W12T) versus (FEP,1RM24,200W15T)			(FEP,2RM20,200W12T) versus (FEP,2RM24,200W15T)		
Size of TWP beam	M20/EP12 mm	M24/EP15 mm	% increase	M20/EP12 mm	M24/EP15 mm	% increase
400 x 140 x 39.7 (12/4)	69	102	47.8	103	160	55.3
400 x 160 x 48.4 (14/4)	69	102	47.8	103	160	55.3
450 x 160 x 50.2 (14/4)	79	117	48.1	120	187	55.8
450 x 180 x 60.1 (16/4)	79	117	48.1	120	187	55.8
500 x 160 x 52.0 (14/4)	90	133	47.7	137	214	56.2
500 x 180 x 61.9 (16/4)	90	132	46.6	137	214	56.2
550 x 200 x 73.3 (16/5)	100	148	48.0	155	241	55.5
600 x 200 x 80.5 (16/6)	110	163	48.2	173	268	54.9
		Average	**47.8**		Average	**55.6**

6.3.8 Explanation on the Notations Used in the Proposed Capacity Tables for Extended Endplate Connections.

For the extended endplate connections, eight standardised capacity tables have been generated as listed in Table 6.14. A spreadsheet computer program using Excel was also developed to calculate and predict the moment capacity and shear capacity of the proposed standardised connections. Almost similar to the procedures carried out for the flush endplate connections, the values obtained were also based on the critical zones' checks and the method as described in Chapter 3. Details capacities and other relevant information of the standardised capacity tables for the connections are tabulated in Table 6.15 to 6.22. The determination of the moment capacity is carried out the same as for the flush end plate connections. However, the geometrical configuration and the positions of the bolt rows have to be taken into account. In the case of extended endplate connections, an additional row of tension bolts was placed at 40 mm above the top flange of the beam.

The rest of the notations in the standardised capacity tables for the extended endplate connections are the same as for the flush endplate connections.

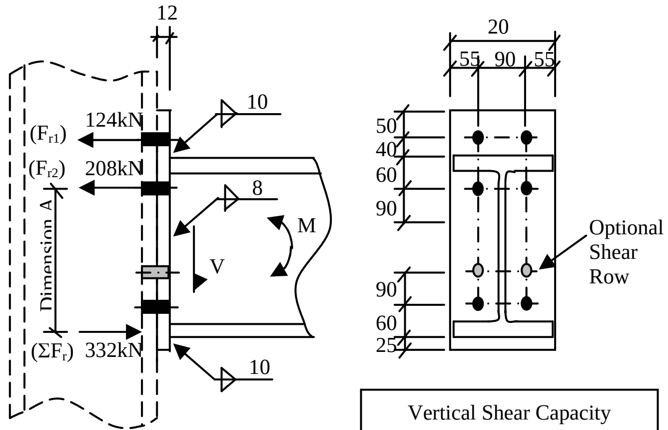
Table 6.14. Configurations of the endplate connections for generating standardised tables

Designation	Row of Tension Bolts	Type of Bolts	Size of Endplates	Design Grade
EEP,2RM20,200W12T	2	M20 8.8	200 x 12	43
EEP,2RM20,250W12T	2	M20 8.8	250 x 12	43
EEP,2RM24,200W15T	2	M24 8.8	200 x 15	43
EEP,2RM24,250W15T	2	M24 8.8	250 x 15	43
EEP,3RM20,200W12T	3	M20 8.8	200 x 12	43
EEP,3RM20,250W12T	3	M20 8.8	250 x 12	43
EEP,3RM24,200W15T	3	M24 8.8	200 x 15	43
EEP,3RM24,250W15T	3	M24 8.8	250 x 15	43

Maintaining the same format, all of the standardised tables mentioned above depicted the geometrical configuration of the standard connection and provide the relevant capacities pertaining to the connection. Inside each table, the suggested sizes of columns and beams that can be used with the connection are also listed out as before. The smallest suggested size of beam (from the low-capacity table) was taken as 350 x 140 whilst the largest suggested size of beam (from the high-capacity table) was taken as 750x250 although a TWP section can be produced up to 1600mm deep. This is to ensure that the ductility of the connection, which is important for partial strength connections, is maintained. The shear capacity of the connection is determined, as for the flush endplate connections, from the shear capacity of the tension bolt rows and bottom bolt rows. Here, the bottom bolt rows were also designed to carry most of the vertical shear force. The value of moment capacities for this type of connections also depends on the size of bolts, number of bolts, size of endplate, and thickness of endplate. For easy identifying, special notation was used for each designated connection. The notation used for the designated connection such as EEP,2RM20,200W12T meaning that the connection is extended endplate with two bolt rows of M20 grade 8.8 (one row in the extended part of endplate and one row beneath the flange), and an endplate size of 200mm wide and 12mm thick. Comparisons of moment capacities based on the different geometrical configuration of the connections are discussed in the subsequent sections.

Table 6.15. Standardised capacity table for extended endplate (EEP, 2RM20, 200W12T)

Beam Side	2 ROW M20 8.8 B0LTS — 200 x 12 DESIGN GRADE 43 EXTENDED END PLATE		
	BEAM – FLANGE S355, WEB S275		
	Beam Serial Size DxBxkg/m (T/t)	Dimension 'A' (mm)	Moment Capacity (kNm)
	350 x 140 x 35.1 (12/3)	284	106
	400 x 140 x 39.7 (12/4)	334	123
	400 x 160 x 48.4 (14/4)	333	123
	450 x 160 x 50.2 (14/4)	383	139
	450 x 180 x 60.1 (16/4)	382	139
	500 x 160 x 52.0 (14/4)	433	156
	500 x 180 x 61.9 (16/4)	432	156
	550 x 200 x 73.3 (16/5)	482	172
	600 x 200 x 80.5 (16/6)	532	189



Vertical Shear Capacity

331 kN without shear row
515 kN with shear row

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

Tension Zone:

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

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

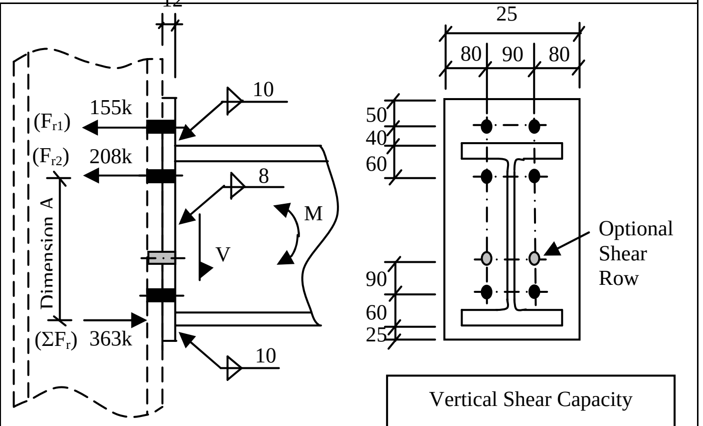
Compression Zone:

√ Column capacity exceeds ΣF_r.

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

Table 6.16: Standardised table for extended endplate connection (EEP, 2RM20, 250W12T)

Beam Side	2 ROW M20 8.8 B0LTS 250 x 12 DESIGN GRADE 43 EXTENDED END PLATE			
	BEAM – FLANGE S355 WEB S275			
	Beam Serial Size DxBxkg/m (T/t)	Dimension 'A' (mm)	Moment Capacity (kNm)	
	400 x 160 x 48.4 (14/4)	333	136	
	450 x 160 x 50.2 (14/4)	383	154	
	450 x 180 x 60.1 (16/4)	382	154	
	500 x 180 x 61.9 (16/4)	432	172	
	550 x 200 x 73.3 (16/5)	482	190	
	600 x 200 x 80.5 (16/6)	532	208	
	650 x 250 x 103.4 (18/6)	581	226	



Vertical Shear Capacity

331 kN without shear row
515 kN with shear row

Column Side

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

Tension Zone:

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

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

Compression Zone:

√ Column capacity exceeds ΣF_r.

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

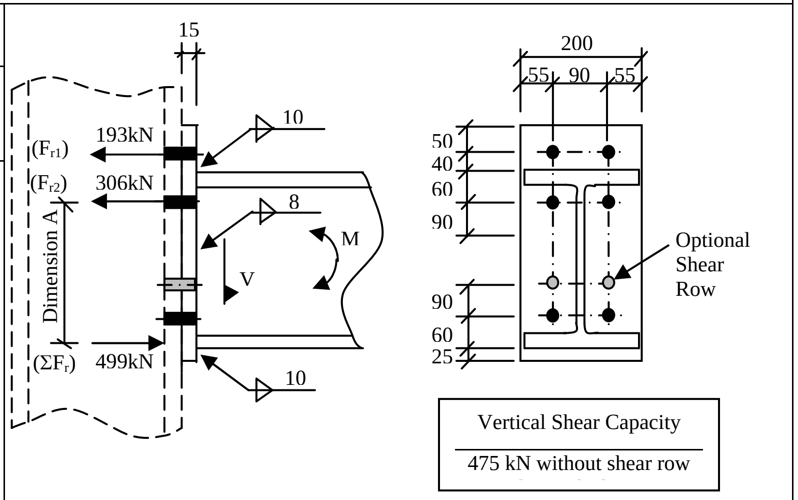
Table 6.17: Standardised table for extended endplate connection (EEP, 2RM24, 200W15T)

2 ROW M24 8.8 B0LTS

200 x 15 DESIGN GRADE 43 EXTENDED END PLATE

BEAM – FLANGE S355
WEB S275

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



Column Side

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

Tension Zone:

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

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

Compression Zone:

√ Column capacity exceeds ΣF_r.

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

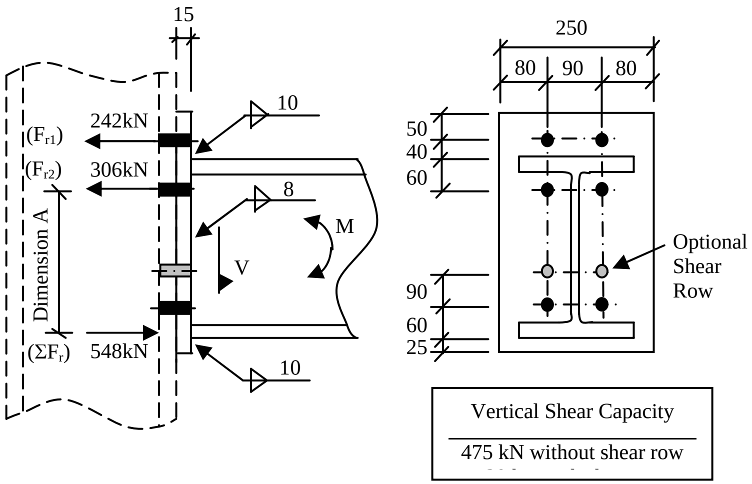
Table 6.18: Standardised table for extended endplate connection (EEP, 2RM24, 250W15T)

2 ROW M24 8.8 B0LTS

250 x 15 DESIGN GRADE 43 EXTENDED END PLATE

BEAM – FLANGE S355
WEB S275

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



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

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

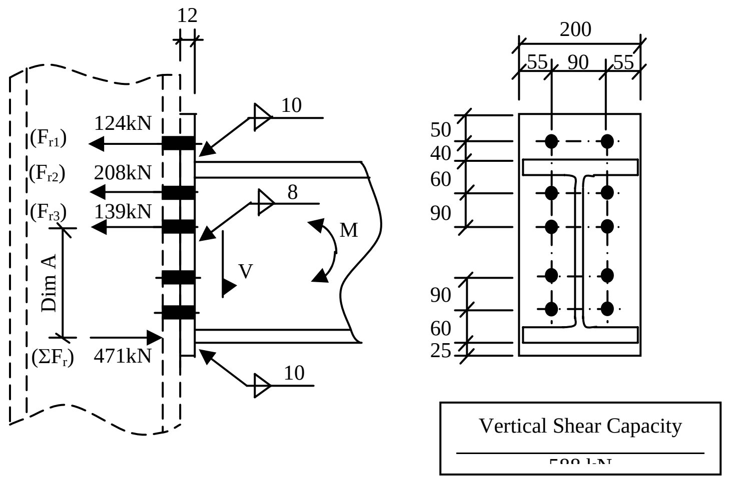
Table 6.19: Standardised table for flush endplate connection (EEP, 3RM20, 200W12T)

3 ROW M20 8.8 B0LTS

200 x 12 DESIGN GRADE 43 EXTENDED END PLATE

BEAM – FLANGE S355
WEB S275

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



Column Side

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

Tension Zone:

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

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

Compression Zone:

√ Column capacity exceeds ΣF_r.

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

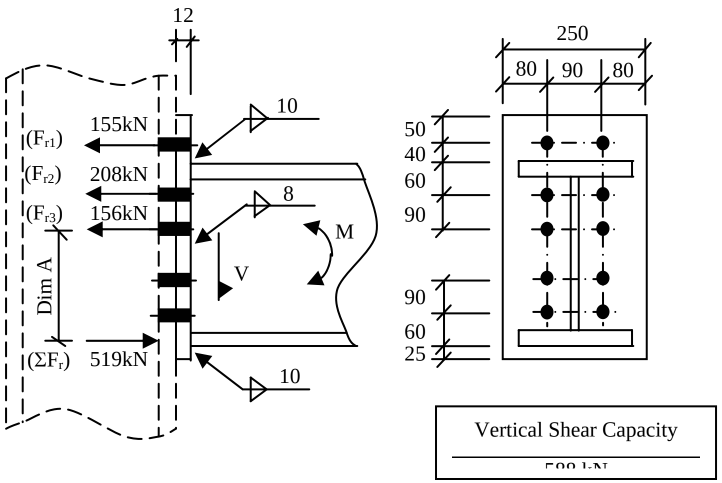
Table 6.20: Standardised table for extended endplate connection (EEP, 3RM20, 250W12T)

3 ROW M20 8.8 B0LTS

250 x 12 DESIGN GRADE 43 EXTENDED END PLATE

BEAM – FLANGE S355
WEB S275

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



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

Tension Zone:

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

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

Compression Zone:

√ Column capacity exceeds ΣF_r.

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

Table 6.21. Standardised table for extended endplate connection (EEP, 3RM20, 200W15T)

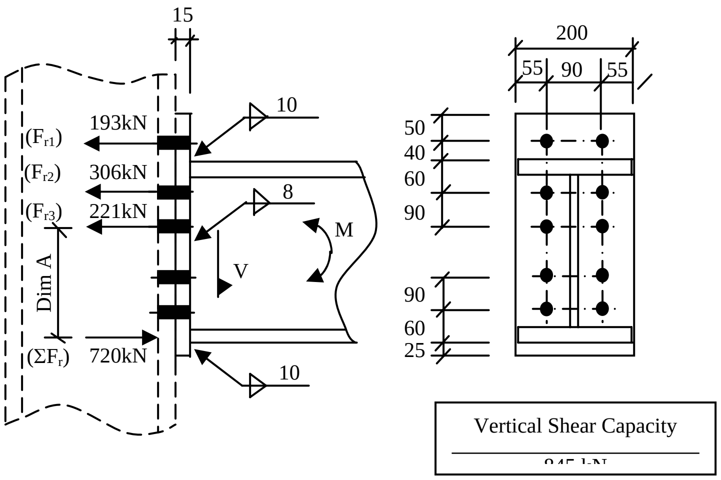
3 ROW M24 8.8 B0LTS

200 x 15 DESIGN GRADE 43 EXTENDED END PLATE

Beam Side

BEAM – FLANGE S355 WEB S275

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



Column Side

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

Tension Zone:

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

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

Compression Zone:

√ Column capacity exceeds ΣF_r.

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

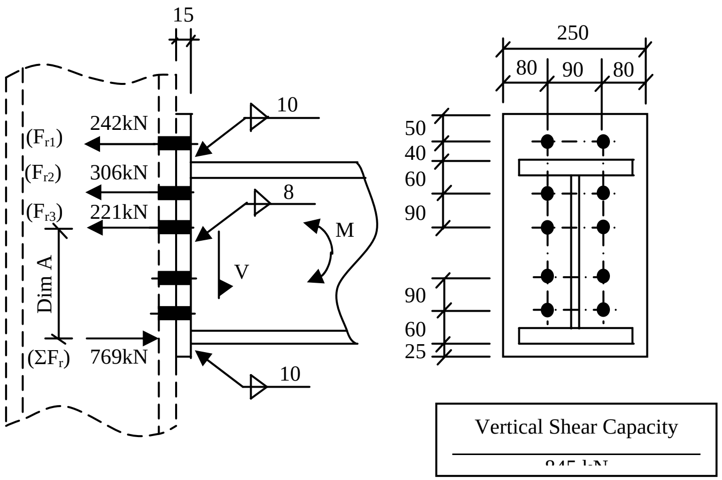
Table 6.22: Standardised table for extended endplate connection (EEP, 3RM24, 250W15T)

3 ROW M24 8.8 B0LTS

250 x 15 DESIGN GRADE 43 EXTENDED END PLATE

BEAM – FLANGE S355
WEB S275

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



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

Tension Zone:

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

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

Compression Zone:

√ Column capacity exceeds ΣF_r.

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

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

[(EEP,2RM20,200W12T) versus (EEP,3RM20,200W12T),
(EEP,2RM20,250W12T) versus (EEP,3RM20,250W12T),
(EEP,2RM24,200W15T) versus (EEP,3RM24,200W15T,and
(EEP,2RM24,250W15T) versus (EEP,3RM24,250W15T)]

Table 6.15 to 6.18 show the moment capacity of the connection for double bolt rows, whilst Table 6.19 to 6.22 show the moment capacity of the connection for triple bolt rows. The results of percentage increase in moment capacity for two and three bolt rows are shown in Table 6.23. The results show that by increasing the number of bolt row from two to three, moment capacity of the connection is increased by an average of 30.1% for M20 bolt with 12mm thick and 200mm wide end plate, 30.8% for M20 bolt with 12mm thick and 250mm wide end plate, 32.8% for M24 bolt with 15mm thick and 200mm wide end plate, 29.4% for M24 bolt with 15mm thick and 250mm wide end plate. The combination of M24 with 15mm thick end plate has contributed to the increase in the moment capacity of the connection. The increase in moment capacity is very much linear to the depth of the beam. This shows that the moment capacity of the connection depends on the depth of the beam, the number and size of bolt, and the thickness of the end plate.

The vertical shear capacity of connection in Table 6.15 and 6.16 is increased from 331kN without optional shear bolt row to 515kN with shear row. The vertical shear capacity of connection in Table 6.17 and 6.18 is increased from 475kN without optional shear bolt row to 739N with shear row. The increment of the vertical shear capacity is not exactly double as the determination of the shear capacity depends on the number of row of the tension bolt too. The vertical shear capacity of the connection in Table 6.19 and 6.20 is 588kN with optional shear bolt row. The vertical shear capacity of the connection in Table 6.21 and 6.22 is 845kN with optional shear bolt row. These values are about twice the vertical shear capacities of the connections in Table 6.15 and 6.16, and Table 4 and 5 respectively without optional shear bolt row. This is because the number of bolt row at the tension zone in Table 6.19, 6.20, 6.21 and 6.22 is three rows.

Panel shear capacity for all the connections is the same as the size of the columns is the same and the force of tension and compression that exert on the column web is not high enough to change the calculated values.

6.3.8.2 Effect of increasing the size of endplate from 200mm to 250mm

[(EEP,2RM20,200W12T) versus (EEP,2RM20,250W12T),
(EEP,2RM24,200W15T) versus (EEP,2RM24,250W15T),
(EEP,3RM20,200W12T) versus (EEP,3RM20,250W12T, and
(EEP,3RM24,200W15T) versus (EEP,3RM24,250W15T)]

Table 6.15, 6.17, 6.19 and 6.21 show the moment capacity of the connection for end-plate width of 200mm, whilst Table 6.16, 6.18, 6.20 and 6.22 show the moment capacity of the connection for end-plate width of 250mm. The idea of comparison is to know the percentage increase due to increment of the width of the endplate. The results of percentage increase in moment capacity for 200mm and 250mm wide of the endplate are tabulated in Table 6.24. The results show that by increasing the size of end-plate width from 200mm to 250mm, moment capacity of the connection is increased by an average of 10.5% for two rows of M20 bolt with 12mm thick endplate, 11.1% two rows of M24 bolt with 15mm thick endplate, 11.1% for three rows of M20 bolt with 12mm thick endplate, and 8.2% for three rows of M24 bolt with 15mm thick endplate. The results show that the increment of the plate size from 200 to 250mm has contributed to a marginal amount of moment capacity to the connection. The percentage of increase if M24 bolt was used is about the same in all cases, thus it can be said that that the moment capacity of the connection depends on the strength of the endplate more than the strength of the bolts.

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

[(EEP,2RM20,200W12T) versus (EEP,2RM24,200W15T),
(EEP,2RM20,250W12T) versus (EEP,2RM24,250W15T),
(EEP,3RM20,200W12T) versus (EEP,3RM24,200W15T), and
(EEP,3RM20,250W12T) versus (EEP,3RM24,250W15T)

The need to compare the result is to know the percentage increase due to increment of the size of bolt and thickness of the endplate. The results of percentage increase in moment capacity for M20 with 12mm thick endplate and M24 with 15mm thick end-plate are tabulated in Table 6.25. The results show that by increasing the size of bolt from M20 with 12mm thick end-plate to M24 with 15mm thick end-plate, the moment capacity of the connection is increased by an average of 51.1% for two bolt rows with 200mm thick endplate, 51.8% for two bolt rows with 250mm thick endplate, 52.9% for three bolt rows with 200mm endplate, and 49.1% for three bolt rows with 250mm thick endplate. It can be noticed that the increment in all cases are about the same. Hence, the results show that the moment capacity of the connection depends on the strength of the endplate more than the strength of the bolts.

Table 6.23. Percentage increase in moment capacity due to the increasing number of bolt rows (EEP)

	EEP,2RM20,200W12T versus EEP,3RM20,200W12T			EEP,2RM20,250W12T versus EEP,3RM20,250W12T			EEP,2RM24,200W15T versus EEP,3RM24,200W15T			EEP,2RM24,250W15T versus EEP,3RM24,250W15T		
Size of TWP beam	Two bolt rows	Three bolt rows	% increase	Two bolt rows	Three bolt rows	% increase	Two bolt rows	Three bolt rows	% increase	Two bolt rows	Three bolt rows	% increase
400 x 160 x 48.4 (14/4)	123	156	26.8	136	174	27.9						
450 x 160 x 50.2 (14/4)	139	180	29.5	154	200	29.9						
450 x 180 x 60.1 (16/4)	139	180	29.5	154	200	29.9	210	275	31.0	234	298	27.4
500 x 180 x 61.9 (16/4)	156	203	30.1	172	225	30.8	235	311	32.3	261	337	29.1
550 x 200 x 73.3 (16/5)	172	227	32.0	190	252	32.6	260	347	33.5	288	375	30.2
600 x 200 x 80.5 (16/6)	189	251	32.8	208	278	33.7	285	383	34.4	316	413	30.7
		Average	**30.1**		Average	**30.8**		Average	**32.8**		Average	**29.4**

Table 6.24. Percentage increase in moment capacity due to the increasing size of endplate (EEP)

	EEP,2RM20,200W12T versus EEP,2RM20,250W12T			EEP,2RM24,200W15T versus EEP,2RM24,250W15T			EEP,3RM20,200W12T versus EEP,3RM20,250W12T			EEP,3RM24,200W15T versus EEP,3RM24,250W15T		
Size of TWP beam	200mm	250mm	% increase	200mm	250mm	% increase	200mm	250mm	% increase	200mm	250mm	% increase
400 x 160 x 48.4 (14/4)	123	136	10.6				156	174	11.5			
450 x 160 x 50.2 (14/4)	139	154	10.8				180	200	11.1			
450 x 180 x 60.1 (16/4)	139	154	10.8	210	234	11.4	180	200	11.1	275	298	8.4
500 x 180 x 61.9 (16/4)	156	172	10.3	235	261	11.1	203	225	10.8	311	337	8.4
550 x 200 x 73.3 (16/5)	172	190	10.5	260	288	10.8	227	252	11.0	347	375	8.1
600 x 200 x 80.5 (16/6)	189	208	10.1	285	316	10.9	251	278	10.8	383	413	7.8
		Average	**10.5**		Average	**11.1**		Average	**11.1**		Average	**8.2**

Table 6.25: Percentage increase in moment capacity due to the increasing size of bolts/endplate thickness (EEP)

	EEP,2RM20,200W12T versus EEP,2RM24,200W15T			EEP,2RM20,250W12T versus EEP,2RM24,250W15T			EEP,3RM20,200W12T versus EEP,3RM24,200W15T			EEP,3RM20,250W12T versus EEP,3RM24,250W15T		
Size of TWP beam	M20/EP 12mm	M24/EP 15mm	% increase	M20/EP 12mm	M24/EP 5mm	% increase	M20/EP 12mm	M24/EP 15mm	% increase	M20/EP 12mm	M24/EP 15mm	% increase
400 x 160 x 48.4 (14/4)	123	186	51.2									
450 x 160 x 50.2 (14/4)	139	211	51.8									
450 x 180 x 60.1 (16/4)	139	210	51.1	154	234	51.9	180	275	52.8	200	298	49.0
500 x 180 x 61.9 (16/4)	156	235	50.6	172	261	51.7	203	311	53.2	225	337	49.8
550 x 200 x 73.3 (16/5)	172	260	51.2	190	288	51.6	227	347	52.9	252	375	48.8
600 x 200 x 80.5 (16/6)	189	285	50.8	208	316	51.9	251	383	52.6	278	413	48.6
		Average	**51.1**		Average	**51.8**		Average	**52.9**		Average	**49.1**

6.4 Economic Aspects of Semi-Rigid Design

Parametric study for the design of multi-storey unbraced frames was carried out. The loadings and frame layout are listed in Table 4.1 and Table 4.2 of Chapter Four. Three load combinations of frame design were identified for the parametric study:

i. 1.4 dead load plus 1.6 imposed load plus factored notional horizontal force
ii. 1.2 dead load plus 1.2 imposed load plus 1.2 wind load
iii. 1.4 dead load plus 1.4 wind load

Manual calculation has been made to track the design steps. An excel design sheet was later established to support faster analysis and design. All design were based on BS5950-1:2000. Method of semi-continuous braced frames design was based on the worked example drawn by Couchman (1997). Design capacity of flush end-plate and extended end-plate connection were referred to the methods mentioned in Steel Construction Institute Publication "Joints in Steel Construction: Moment Connection" (SCI 1995).

6.4.1 Parametric Study Results

Parametric study on the design of unbraced steel frames using wind moment method was discussed in Chapter Four. The worked example of the calculation of wind load and analysis and design of unbraced frame were given in Section 4.4.1 and Section 4.5. The results of the parametric study were listed in Table 6.26 to Table 6.33.

Table 6.26: Total steel weight and sway deflection for the unbraced frames of 6 meter span major axis frames under minimum wind load in conjunction with maximum gravity load.

Unbraced Frame	Total Steel Weight (tonne)	Sway Deflection (mm)	Sway Limit hr/300 (mm)
2 Bay 2 Storey	3.156	14.2	30.00
2 Bay 4 Storey	7.077	31.4	56.67
2 Bay 6 Storey	11.691	54.57	83.33
2 Bay 8 Storey	17.529	78.59	110.00
4 Bay 2 Storey	5.772	8.39	30.00
4 Bay 4 Storey	12.925	16.03	56.67
4 Bay 6 Storey	21.677	28.88	83.33
4 Bay 8 Storey	31.327	43.56	110.00

Table 6.27: Total steel weight and sway deflection for the unbraced frames of 9 meter span major axis frames under minimum wind load in conjunction with maximum gravity load.

Unbraced Frame	Total Steel Weight (tonne)	Sway Deflection (mm)	Sway Limit hr/300 (mm)
2 Bay 2 Storey	5.859	7.63	30.00
2 Bay 4 Storey	13.427	15.85	56.67
2 Bay 6 Storey	21.827	28.05	83.33
2 Bay 8 Storey	30.765	43.00	110.00
4 Bay 2 Storey	11.061	4.28	30.00
4 Bay 4 Storey	25.741	9.09	56.67
4 Bay 6 Storey	41.573	15.23	83.33
4 Bay 8 Storey	59.211	22.57	110.00

Table 6.28: Total steel weight and sway deflection for the unbraced frames of 6 meter span major axis frames under maximum wind load in conjunction with minimum gravity load.

Unbraced Frame	Total Steel Weight (tonne)	Sway Deflection (mm)	Sway Limit hr/300 (mm)
2 Bay 2 Storey	3.195	19.90	30.00
2 Bay 4 Storey	8.373	41.10	56.67
2 Bay 6 Storey	15.591	61.14	83.33
2 Bay 8 Storey	24.484	86.81	110.00
4 Bay 2 Storey	4.860	17.03	30.00
4 Bay 4 Storey	11.485	41.12	56.67
4 Bay 6 Storey	21.047	66.62	83.33
4 Bay 8 Storey	32.035	82.34	110.00

Table 6.29: Total steel weight and sway deflection for the unbraced frames of 9 meter span major axis frames under maximum wind load in conjunction with minimum gravity load.

Unbraced Frame	Total Steel Weight (tonne)	Sway Deflection (mm)	Sway Limit hr/300 (mm)
2 Bay 2 Storey	5.211	19.38	30.00
2 Bay 4 Storey	11.836	39.85	56.67
2 Bay 6 Storey	21.169	63.51	83.33
2 Bay 8 Storey	32.211	87.26	110.00
4 Bay 2 Storey	9.585	12.48	30.00
4 Bay 4 Storey	20.161	30.42	56.67
4 Bay 6 Storey	32.595	56.74	83.33
4 Bay 8 Storey	47.629	81.39	110.00

Table 6.30: Total steel weight and sway deflection for the unbraced frames of 6 meter span minor axis frames under minimum wind load in conjunction with maximum gravity load.

Unbraced Frame	Total Steel Weight (tonne)	Sway Deflection (mm)	Sway Limit hr/300 (mm)
2 Bay 2 Storey	3.939	16.46	30.00
2 Bay 4 Storey	8.757	35.93	56.67
2 Bay 6 Storey	15.516	61.06	83.33
2 Bay 8 Storey	25.461	80.32	110.00
4 Bay 2 Storey	6.789	10.71	30.00
4 Bay 4 Storey	15.773	21.46	56.67
4 Bay 6 Storey	25.333	41.75	83.33
4 Bay 8 Storey	37.467	58.93	110.00

Table 6.31: Total steel weight and sway deflection for the unbraced frames of 9 meter span minor axis frames under minimum wind load in conjunction with maximum gravity load.

Unbraced Frame	Total Steel Weight (tonne)	Sway Deflection (mm)	Sway Limit hr/300 (mm)
2 Bay 2 Storey	6.696	10.13	30.00
2 Bay 4 Storey	14.971	25.59	56.67
2 Bay 6 Storey	24.617	42.39	83.33
2 Bay 8 Storey	35.290	63.26	110.00
4 Bay 2 Storey	12.708	5.74	30.00
4 Bay 4 Storey	28.421	13.26	56.67
4 Bay 6 Storey	46.887	22.08	83.33
4 Bay 8 Storey	66.936	33.36	110.00

Table 6.32: Total steel weight and sway deflection for the unbraced frames of 6 meter span minor axis frames under maximum wind load in conjunction with minimum gravity load.

Unbraced Frame	Total Steel Weight (tonne)	Sway Deflection (mm)	Sway Limit hr/300 (mm)
2 Bay 2 Storey	4.446	19.50	30.00
2 Bay 4 Storey	12.729	41.46	56.67
2 Bay 6 Storey	25.008	65.63	83.33
2 Bay 8 Storey	43.797	87.62	110.00
4 Bay 2 Storey	6.525	18.23	30.00
4 Bay 4 Storey	16.037	36.72	56.67
4 Bay 6 Storey	30.687	65.79	83.33
4 Bay 8 Storey	46.739	81.76	110.00

Table 6.33: Total steel weight and sway deflection for the unbraced frames of 9 meter span minor axis frames under maximum wind load in conjunction with minimum gravity load.

Unbraced Frame	Total Steel Weight (tonne)	Sway Deflection (mm)	Sway Limit hr/300 (mm)
2 Bay 2 Storey	6.480	16.48	30.00
2 Bay 4 Storey	15.216	41.15	56.67
2 Bay 6 Storey	31.601	65.55	83.33
2 Bay 8 Storey	51.499	86.77	110.00
4 Bay 2 Storey	10.539	19.29	30.00
4 Bay 4 Storey	23.035	38.09	56.67
4 Bay 6 Storey	40.409	59.95	83.33
4 Bay 8 Storey	60.735	86.62	110.00

6.5 Concluding Remarks

The results of the three main tasks of the study were presented and discussed. The behaviour of the FEP and EEP connections were studied from the nine isolated joint tests conducted. Altogether there were five tests on FEP and four tests on EEP. The important characteristics of both types of connections, which are the moment resistance, rotational capacity and rotational ductility, were obtained. In addition, strains around the vicinity of the connections were also observed.

As for the standardised connection tables using TWP sections, fourteen tables comprising of six tables for the FEP and eight tables for the EEP were developed and generated. The effects of bolt diameters, number of bolts and end plate sizes were discussed in details. From the parametric study of the design of multi-storey braced frames, the sizes of the beams obtained by utilizing universal beam sections and TWP sections for both simple and semi-continuous constructions were obtained. Subsequently, a set of computer programs for the design was written with a capacity to design using simple design method and semi-rigid design method.

These three tasks were then blended together to produce the overall picture of the behaviour of partial strength connections of the flush endplate and the extended endplate.

CHAPTER VII

CONCLUSION

7.1 Summary of Work and Conclusion

The results of the three main tasks of the study were presented and discussed in Chapter Six. For the standardised connection tables using TWP sections, fourteen tables comprising of six tables for the FEP and eight tables for the EEP were developed and generated. The effects of bolt diameters, number of bolts and end plate sizes were discussed in details.

From the parametric study of the design of multi-storey braced frames, the sizes of the beams obtained by utilizing universal beam sections and TWP sections for both simple and semi-continuous constructions were obtained. Subsequently, a set of computer programs for the design was written with a capacity to design using simple design method and semi-rigid design method.

The behaviour of the FEP and EEP connections were studied from the nine isolated joint tests conducted. Altogether there were five tests on FEP and four tests on EEP. The important characteristics of both types of connections, which are the moment resistance, rotational capacity and rotational ductility, were obtained. In addition, strains around the vicinity of the connections were also observed.

These three tasks were then blended together to produce the overall picture of the behaviour of partial strength connections of the flush endplate and the extended endplate.

Form the works carried out in this research, the following conclusion can be drawn:

1. The complete design of both semi-rigid and rigid multi-storey unbraced steel frames has been given in Chapter Four, Table 4.5 to Table 4.20. The comparison between semi-rigid construction and rigid construction for multi-storey unbraced steel frames has shown that the total steel weight savings for frame using full strength connections compared to partial strength connections is ranging from 5.56% to 21.80% for major axis frames. From the parametric study, the unbraced multi-storey steel frames can be designed up to eight floors at the column's major axis, provided the minor axis of the column is braced. For multi-storey steel frame with both major and minor axes unbraced, it would be needed to apply other types of column such as cruciform column.
2. The optimum design of unbraced steel frame structures can be achieved using wind moment method and partial strength joints. The step-by step calculation of wind load and the wind moment method design procedures for multi-storey unbraced steel frame has been depicted in Chapter Four, which is less complicated than the rigid frame design that involved massive mathematical iterations. The actual moment capacities of the partial strength joints using flush and plate and extended end plate has been studied, and shown 17.2% to 37.0% higher than the predicted capacities. The experimental study has proven of the extend application of partial strength joints to Trapezoidal Web Profiled Section, which is classified as semi-compact section.
3. The standardized table of partial strength connection has been established for limited flush end plate connection and extended end plate connection. The standard connection design table for flush end plate were given in Table 6.5 to Table 6.10; meanwhile the standard connection design table for extended end plate were given in Table 6.15 to Table 6.22. It was clearly seen that using the standardized connection design tables would be faster and easier then the detailed connection design and checking procedures, as discussed in Chapter Three,

Section 3.6. However, more works need to be done, such as the sub-assemblage frame tests, full scale 2-D tests and full-scale 3-D frame tests, to get complete scope of the overall standardized table for partial strength connections using trapezoidal web profiled section. These can be done in further research work by other research fund.

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

7.2 Suggestion for Future Works

The economic aspect of the use of partial strength connection for unbraced steel frame has been studied in this research. However, in order to obtain better understanding to the actual performance of connection method, full-scale experimental test is a good approach, especially to determine the possible application of the proposed design guide for steel frame design. There are several other suggestions in efforts of gathering standardized partial strength connections and TWP members' performance data:

- Gather experimental data on the similar topic in Malaysia, or even in other countries. Experimental testing can be carried out by universities, manufacturers or other institution such as Steel Construction Institute (SCI), ASCI, etc.
- Gather professional's comment and experience in using the semi-continuous steel frame design and TWP members in real-world constructions.

- Gather numerical studies, statistics etc. which related to the topic.
- As the data base for standardized partial strength connection being increased, researchers can be more confidence in promoting the new connection design tables.

Further research developments that can be carried out in order to increase the robustness of the unbraced steel frame and minimise the steel weight are the use of composite connections and cruciform column. Application of composite beam to the multi-storey steel frame has been proven to lead to the use of shallower steel beam size, compared to bare steel beam. The design of composite connections would further increase the steel weight saving, as well as enhance the stiffness of the frame structure towards lateral deflection.

The use of Universal Column (UC) will provide good stiffness and capacities at its major axis. However, it is weak at the minor axis, which would lead to over-design in order to strengthen the steel frame at its minor axis. The use of cruciform column can be one solution to minimize the over-design of steel column in unbraced frame. The studies of the actual capacities and a simplified method of design to carry out the application of composite connections and cruciform column would be needed, and thus proposed here as a suggestion for future development.

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