DESIGN OF MOMENT RESISTING FRAME AND ITS CONNECTIONS

BY

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10MCLC07

DEPARTMENT OF CIVIL ENGINEERING

INSTITUTE OF TECHNOLOGY

NIRMA UNIVERSITY

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Major Project

Submitted in partial fulfillment of the requirements

For the degree of

Master of Technology in Civil Engineering

(Computer Aided Structural Analysis and Design)

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Declaration

This is to certify that

a. The major project comprises my original work towards the Degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) at Nirma University and has not been submitted elsewhere for a degree.

b. Due acknowledgement has been made in the text to all other material used.

Bharat P. Makhijani

Certificate

This is to certify that the Major Project entitled "Design of Moment Resisting Steel Frame and its Connections" submitted by Makhijani Bharat Purshotam (10MCLC07), towards the partial fulfillment of the requirement for the degree of Master of Technology in civil Engineering (Computer Aided Structural Analysis and Design) of Nirma University, Ahmedabad, is the record of work carried out by him under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

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Abstract

Conventional analysis and design of steel frames are usually carried out under the assumption that the beams to column connections are either fully rigid or ideally pinned. Although the adoption of such idealized joint behavior greatly simplifies the analysis and design processes, the predicted response of the idealized structure may be quite unrealistic compared to the response of the actual structure. Any structural connections deform to some extent and will resist a certain amount of bending moment. The relationship between beam end moment, M, and the relative change in angle, θ_r, can be described by means of a typical moment-rotation diagram.

Moment-resisting steel frames are used frequently in low-rise and mid-rise buildings located in high seismic areas due to their high ductility and economic solutions. In these type of structures, strong column weak-beam design requirements result in larger column sections and over design in low-rise long-span buildings. To mitigate this problem, moment-resisting steel frames with energy-dissipative semi-rigid/partial strength connections can be used as an alternative to perimeter frames. By using energy-dissipative semi-rigid connections, the strong-column weak-beam requirement is eliminated and more economical column sections are used. In this study, a three-span three-bay frame with 5m ,7m and 9m span lengths is designed with Eight types of semi-rigid connections. Load combinations are taken as per the IS 800 -2007. The design with reduced connection capacity resulted in an increase in the beam weights, a decrease in the column weights and an overall decrease in the structural weight. Analysis of the frames is carried out in staad pro software ,Moment rotation curve is found out using Frye and Morris model and stiffness of the connections is found then these connections are modelled as as rotational spring element. The overdesign problem in low-rise long span-buildings is eliminated to some extent without using the perimeter frame approach. Furthermore, the top displacements in semi-rigid frames becomes lower than those of their rigid counterparts

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Abbreviation,Notation and Nomenclature

A_g .... Gross cross sectional area

A_n .... Net area of the total cross section

A_{nb} .... Net tensile cross sectional area of bolt

A_{sb} .... Gross sectional area of a bBlt

A_{tn} . Net sectional area in tension from the centre of the hole to the toe of the angle perpendicular to the angle

b_1 .... Stiff bearing length ,

b_f .... Width of the flange

C_{my}, C_{mz} .... Moment amplification factor about the respective axis

c_m .... Moment reduction factor for lateral torsional buckling

D .... Overall depth

d .... Depth of web

d_b .... diameter of bolt

d_p .... Panel zone depth in the beam column junction

E .... Modulus of Elasticity of steel

f_d .... Factored design load

f_n .... Normal force

f_{bd} .... Design bending compressive stress corresponding to lateral buckling.

f_{cc} .... Elastic buckling stress of a column

f_{cd} .... Design compressive stress

f_o .... proof stress

f_u .... Characteristic ultimate tensile stress

f_{ub} .... Characteristic ultimate tensile stress of the bolt

G .... Modulus of rigidity of steel.

g .... gauge length

h .... depth of the section

I .... Moment of inertia of the member about an axis perpendicular to the frame

K_b .... Effective stiffness of the beam and column

L .... Actual length

L_{LT} .... Effective length for lateral torsional buckling.

l_e .... Distance between prying force and Bolt centre line

l_w .... Length of weld

M .... Bending moment

M_{cr} Elastic critical moment corresponding to lateral torsional Buckling of the beam.

M_d .... Design flexural strength

M_{dz}, M_{dy} .... Design bending strength about the minor axis and major axis

M_{ndz}, M_{ndy} ...... Design bending strength under combined Axial force and Uniaxial moment

N_d .... Design strength in tension or in compression

n .... Number of Bolts

P .... Factored applied axial force

P_d .... Design axial compressive strenght

Q .... Prying force

T_{dn} .... Rupture strenght of net section under axial tension

T_{db} .... Design strenght of bolt under axial tension

t_f .... Thickness of flange

t_p .... thickness of plate

t_w .... Thickness of web

V_{cr} .... Critical shear strenght corresponding to web buckling

V_d .... Design shear strenght

V_{npb} .... Nominal bearing strenght of bolt

V_{nsb} .... Nominal shear capacity of bolt

Z_e .... Elastic section modulus

α .... Imperfection factor

χ .... Strenght reduction factor to account for buckling under compression

χ_{LT} ..... Strenght reduction factor to account for lateral torsional buckling of beams

δ .... Storey deflection

γ_m .... Partial safety factor for load

γ_{mo} .... Partial safety factor against yield stress and buckling

ε .... yield stress ratio $(250/f_y)^{0.5}$

λ_{LT} .... Non dimensional slenderness ratio in lateral bending

μ .... Poisson ratio

ψ . Ratio of the moments at the ends of the laterally unsupported length of a beam

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Chapter 1

Introduction

1.1 Behaviour of Moment Resisting Frame

Moment-resisting steel frames are used frequently in low-rise and mid-rise buildings located in high seismic areas due to their high ductility and economic solution.In these structures, the strong-column weak-beam design approach is used to allow for plastic hinges to develop in the beams prior to the columns and increase the ductility of structure to prevent collapse. One of the major shortcomings of strong column weak-beam provisions is that they result in larger column sections and over design in low-rise long-span buildings In order to overcome these shortcomings, a typical practice is to utilize lateral load-resistant frames only in the perimeter frames. but low redundancy and lack of redistribution capacity are the main disadvantages of using such an approach. the main disadvantages of using such an approach. Minor local damages in the perimeter frames could increase the eccentricity of the structure and even result in total collapse of the building. The most famous example of this is the California State University Parking Structure. By designing low-rise long-span structures with energy dissipative zones in semi-rigid/partial strength connections, most of the aforementioned shortcomings can be eliminated. Field bolted connections shorten the field erection process, require less skilled labour and provide more reliable construction quality. In addition, the need for strong-column weak-beam provision can be eliminated by only designing the columns to be stronger than the connections. As a result, smaller column sections can be used and an alternative to perimeter frames can be obtained.

In this study, three-story three-bay buildings with three different span lengths 5m,7m and 9m is studied with eight different types of connections and capacity of connection is found out using Frye and Morris model . By using these semi-rigid connections, the strong-column weak-beam requirement is eliminated and alternative economic systems to perimeter frames is investigated.

1.1.1 Beam Behaviour

It is expected that beams will undergo large inelastic rotations at targeted plastic hinge locations, which might be at the ends of beams, at deliberately weakened portions of the beams with reduced beam section designs, or within the beam span if large gravity moments are present. Failure modes can include excessive local buckling and lateral torsional buckling. Each mode by itself, or the combination of both, leads to a continuous decrease in strength and stiffness.

1.1.2 Column Behaviour

The intention is to keep inelastic deformations out of most columns to minimize detrimental effects of high axial loads on bending behaviour and potential formation of single-story mechanisms. Regardless, many columns designed in accordance with the strong-column/weak beam requirement so it might experience significant inelastic rotations in a major seismic event. Therefore, excessive local buckling and lateral-torsional buckling are potential failure modes, in addition to basic flexural buckling of columns.

1.1.3 Beam-to-column Connections

The connections must be capable of transferring the moment and shear forces that can be developed in the beam to the column. As a result of material over strength, and strain hardening effects, these moment and shear forces can be substantially larger than the design forces obtained from analysis, using code-specified loads. Depending on the type of connection used, this might trigger any of the following failure modes:

- Fracture in or around welds
- Fracture in highly strained base material
- Fractures at weld access holes
- Net section fracture at bolt holes, shearing and tensile failure of bolts, bolt bearing and block shear failures

1.1.4 Joint Panel Zone Behaviour

The joint panel zone resists significant shear forces from the beams framing into a column. Acting as part of the column, it can also be subjected to significant compressive stresses. Potential failure modes include shear buckling and, if doubler plates are used to reinforce the panel zone, fracture at welds. Failure modes associated with the direct transfer of forces from the beam flange to the column can include column flange bending, web crippling, and web buckling.

1.2 Behaviour of Moment Resisting Connection

The type of connections used in the structure relates to the type of frame used. Moment frames refer to frames which have rigid or moment connections between structural elements. These types of frames are often used when lateral bracing is unfeasible or architecturally undesirable. Moment frames are also generally more flexible under lateral loads. For global elastic analysis of a moment frame, rigidity of the connections is assumed so that elastic beam theory can apply. On the other hand, for plastic global analysis of a moment frame, full strength of the connections is assumed, so that the plastic hinge occurs in the beam before it occurs in the connection. Moment connections cost more to fabricate and erect. Semi-rigid frames have some continuity of rotation in the connections, but do not achieve the full bending rigidity of the members. Semi-rigid frames tend to be used for shorter buildings when lateral forces are low, or in beams where some amount of rigidity at the ends helps to reduce deflections.

1.3 Objective of Study

The objective of the study is as follows:

- The main objective of study to mitigate the problem of strong column weak mechanism for long span structure.
- To study in detail the behaviour of the Semi rigid connections for low rise long span structures..
- To see the effect of modelling of connection for bending moment, shear force, end span moment, time period and base shear.
- To find the type of semi rigid connections which reduces the total weight of the structure.

1.4 Scope of Work

For the project planning of the work is done as follows:

- To study the behaviour of semi rigid connection.
- To study the design specification of the various eight types of connections as per Is 800-207 .
- To design various types of connections also for section design.
- To obtain Moment rotation curve for various types of connections using Frye and Morris Model.
- To Study the model of connection and to use it for design of sections for building.

- To carry out the analysis and design for G+3 storey Building with 5m, 7m and 9m span lengths.
- To find the connection which gives optimum total weight of the structure for various span lengths.

1.5 Organization of Report

Project entitled "Design of Moment resisting Frame and its Connection" is divided into 7 Chapters.The Description of the chapters is as Follows:

Chapter 1, Introduction, Gives the overview of the major project it also includes the basic information of the moment resisting frame , connections,objective of study, scope of work.

Chapter 2, Literature Review, This chapter covers various papers related to modelling of frames and its connection, and design of many moment resisting steel frames.

Chapter 3,Design of Connections, This chapter covers basic concepts of connections according to Is 800 -2007 and method to obtain moment rotation curve according to frye and morris model.

Chapter 4, Analysis of g+3 Building, Load calculation, member forces ,comparison of analytical and software results is covered in this chapter.

Chapter 5, Design of g+3 Building,Design of members beams columns ,connection obtaining stiffness for various connections, comparison of results for moments shears,time period,displacement.etc. is included in this chapter.

Chapter 6, Conclusion and Future scope,comparison of results for various span lengths and effective ness of semi rigid connections is discussed in this chapter.

Chapter 2

Literature Review

2.1 General

In this chapter papers related to the design of moment resisting frames ,concept of moment rotation curve ,modelling of connections , failure of connections in North ridge earthquake and Semi rigid connections and its use in frame design are discussed below.

2.2 Research papers

2.2.1 Design of Frame

Aksoylar et al.[1] Moment-resisting steel frames are used frequently in low-rise located and in these type of structures, strong column weak-beam design requirements result in larger column sections and over design in low-rise long-span buildings. For this problem, moment-resisting steel frames with energy-dissipative semi-rigid/partial strength connections can be used as an alternative to perimeter frames.When the frames are designed with reduced connection capacities, the column steel weight is only marginally decreased. Limitations of the existing compact section sizes are responsible for the less than significant decrease in the weight of the structure.The over design problem in low-rise long span buildings are eliminated to some extent without using the perimeter frame approach. Furthermore, the average top displacements of all semi-rigid frames with SMTR-type connection modelling are smaller than those of rigid frames under the design earthquake level.

2.2.2 Semi-rigid Connections

Hadianfard and Razani[2] The actual behaviour of beam to column connections in steel frames is seldom fully rigid or fully pinned. The true behaviour of the connections is usually semi-rigid. Neglecting the real behaviour of the connection in the analysis may lead to unrealistic predictions of the response and reliability of steel frames. More realistic semi-rigid behaviour modelling of connections should be considered in the reliability analysis of steel-framed structures if more reliable results are desired.

Manson[3] Semi rigid frames offer a economical choice of framing system than the other alternatives.partial fixidity of the connections shifts the moment diagram. The total moment is shared between the positive moment at mid span and the negative moments at the supports thereby decreasing the design moments.The physical joints for semi rigid connections are more complicated to design than simple pinned or moment supports. pinned are those which provide connection to the web of the beam only and moment connections are those which connect to both the web and flange of the beam. the semi rigid connection fall some where between two.True semi rigid connections provide a small amount of fixidity between the beam flange and the column.

Kameshki and Saka[4] It is noticed that overall gravity loading is much larger compared to lateral loading and is dominant in the design of the frame, linear semi-rigid frames are lighter than linear rigid frames. Comparison of optimum linear frames with various types of semi-rigid connections shows that when the connection becomes more flexible, the frame becomes lighter. It is also noticed that the frame with top and seat angle without web cleat is 7.8 % lighter than the frame with rigid connections. On the other hand, if the overall gravity loading is not that large compared to lateral loading, i.e. both loadings are active in the linear frame design, semi-rigid

connections produce heavier frames. The increase in the weight of the linear frame with an end plate connection was 9.3 % compared to the rigid frame in three-storey, two-bay frames.
Consideration of geometric nonlinearity in the frame design yields lighter frames compared to linear frames in the case of rigid connections. However, this is not the case in the semi-rigid connections. It is observed that nonlinear semi-rigid frames are lighter in some cases and heavier in some others, compared to linear semi-rigid frames, depending on the magnitude of loading and frame configuration. This result indicates the importance of realistic connection modeling in the optimum design of steel frames. Failure of accurate modeling of connections may yield unsafe designs.

2.2.3 Modelling

Daz et al.[5] In the analysis of the semi-rigid frame, one assumes that the connections can transfer the entire vertical shear and also have the capacity to transfer some moment. When a moment M is applied to a beam-column connection, the connected beam and column rotate relative to each other by an amount of Θr. The relationship between M and Θr, can be shown by the use of a MΘr diagram. This diagram is usually derived by fitting suitable curves to the experimental data.Several types of models can be used to obtain the moment rotation curve, these are: analytical, empirical, experimental, informational, mechanical and numerical. The most popular of these are the mechanical models, of which the most used is the component method. With this method it is possible to evaluate the rotational stiffness and moment capacity of semi-rigid joints when subjected to only pure bending.

Hayalioglu and Degertekin[6] A connection rotates through angle θ_r caused by applied moment M. This is the angle between beam and column from their original position. Several moment rotation relationships have been derived from experimental studies for modelling semi-rigid connections of steel frames. These relationships vary from linear model to exponential models and are non-linear in nature. Relative moment-rotation curves of extensively used semirigid connections are shown by a polynomial model offered by Frye and Morris [14] is used because of its easy application. Semi-rigid end connections of a beam can be represented by rotational springs θ_A and θ_B are the relative spring rotations of both ends and k_A and k_B are the corresponding spring stiffness

Subramanium[7] Behaviour of various types of connection simple connection and moment connection. Design for various types of connection, beams and columns using the limit state design philosophy. According to the Is 800 2007

2.2.4 Case Study

Youssef et al.[8] A Survey of Steel Moment-Resisting Frame Buildings Affected by the 1994 Northridge Earthquake. Data comprises 1290 inspected floor-frames from 51 steel MRF buildings. In low-rise buildings (3 to 5 stories), lower floor levels were more damaged than upper floor levels. No similar patterns were apparent for mid-rise or high-rise buildings.
About 40%of all reported floor-flames have some cracking in the bottom weld; about15% have some cracking in the top weld. The most serious damage types, column web cracking and shear connection damage, each occurred in about 4% of reported flcmr frames, and always in combination with weld or column flange fracture. Column web fracture was observed in a variety of building locations, sizes, flame configurations, diaphragm types, and framing details
Frame configuration (bay length and number of bays per tie) did not come late with damage ratios and/or darnage scores.Structural redundancy (number of frames and bays in a given direction) did not correlate with damage ratios and/or damage scores.Structural regularity (principally building line setbacks and reentrant comers) did not correlate with damage ratios and/or damage scores.

Chapter 3

Design of Connections

3.1 General

In this chapter definition of connections its classification according to IS 800 -2007 ,design steps for bolted shear connections , moment connections,Welded connections importance of semi rigid connection in frames and modelling of connections using frye and morris model is discussed.

3.2 Connections

Steel structure is the assembly of of different members such as beam , column etc.which have different components such as plates, angles,I beams, or channels.These different components have to be connected properly by means of fasteners, so they will act together as a single composite unit.The behaviour of connections is very complex due to the various factors which influence them , such as geometric imperfection, lack of fit, residual stresses,connection flexibility ,slipping and non linear load deformation characteristics.Fig3.11 shows the different types of connections according to Is 800 - 2007

3.2.1 Simple Connection

In simple connection no moment transfer is assumed between the connected parts .The rotational movement of the joint is large in this case . Actually a small amount of moment is developed but is normally ignored in design joint eccentricity of about 60 mm or less is ignored Typically used in frames upto five storey height where strength governs the design rather than stiffness where separate load resisting system is to be provided.

3.2.2 Rigid Connection

Rigid connection develop full moment capacity of connecting members and retain the original angle between the members under any joint rotation.Rigid connections are used in High rise and slender structures where stiffness governs the design this type of connection is necessary in sway frames and for lateral load resisting frames.

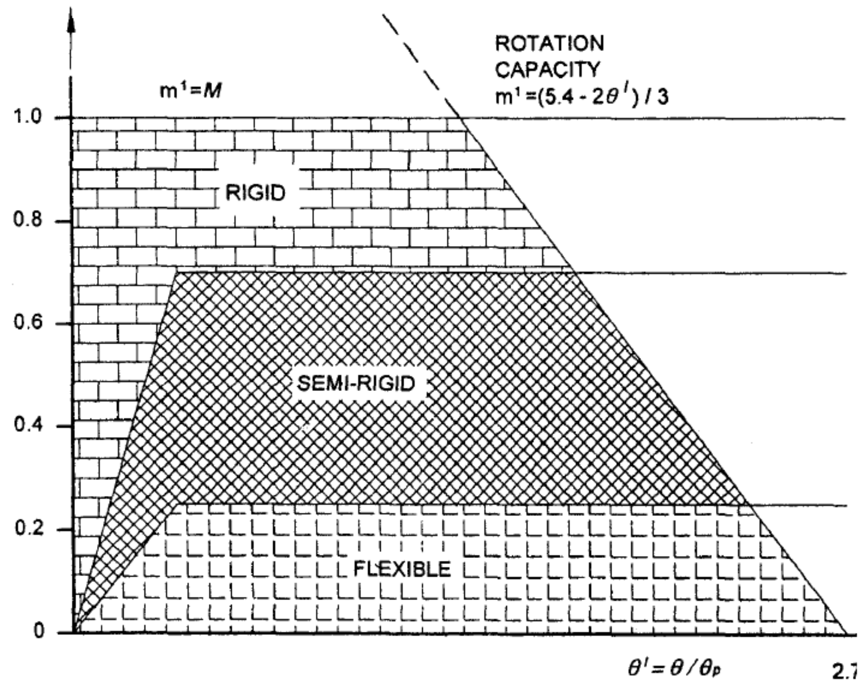


Figure 3.1: Classification of Connections

3.2.3 Semi Rigid Connection

This type of connection falls between two types of connections Semi rigid connection do not have sufficient rigidity to hold the original angles between the members and develop less than the full moment capacity of the connected members. These connection requires the moment rotation relationship of the connection or the test results to determine the moment capacity of connection .But simple method to analyze is to idealize it as equivalent rotational spring by linear or by linear moment rotation characteristics Actually all connections are semi rigid.

Table 3.1: Classification of Connections

Sr no.	Nature of connection	Strength	Stiffness
1	Rigid	$m^{'} \geq 0.7$	$m^{'} \geq 2.5\theta^{'}$
2	Semi rigid	$0.7\, m^{'} > 2$	$2.5\theta^{'} > m^{'} > 0.5\theta^{'}$
3	Flexible	$m^{'} < 0.2$	$m^{'} < 0.5\theta^{'}$

3.3 Bolted Connection

3.3.1 Advantages of Bolted Connection

- Use of unskilled labour and simple tools.
- Noiseless and quick fabrication.
- No special equipments.
- Fast Progress of work.
- Connection supports loads as soon as the bolts are tightened.

3.3.2 Shear Connection

This type of connection only transfers the force only.two types of loads occur in these connections Fig3.2 shows this.First, the force acts in the connection plane and the fasteners between these plates act in shear.Second, type the force acts out of the plane of the connection and the fasteners acts in tension .Simple connections are used in the non sway frame where lateral load resistance is provided by bracings or shear walls.

The different types of simple connections are classified as below

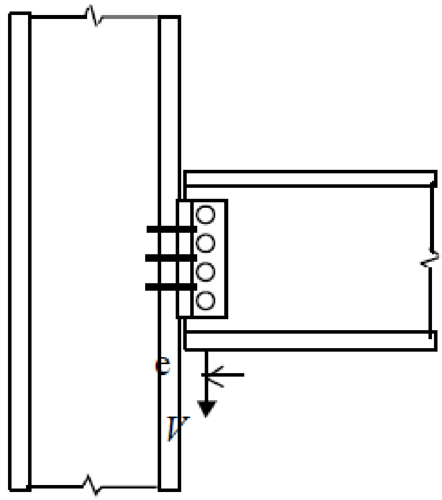


Figure 3.2: Simple Connections

- Seat angle connection.
- Web angle connection
- Stiffened and Un-Stiffened seat angle connection.
- Header plate connection.

3.3.2.1 Clip and Seating Angle Connection

Clip and seating connection also called angle seat connection or seat angle connection transfers reaction from the beam to column through the angle seat.the top cleat is provided for the lateral or torsional restraint to the top flange of the beam. The beam reaction is transferred by bearing,shear and bending of the horizontal leg of the bottom angle, by vertical shear through the fasteners and the horizontal force in the fasteners between the vertical leg and the column. The outstanding leg of the

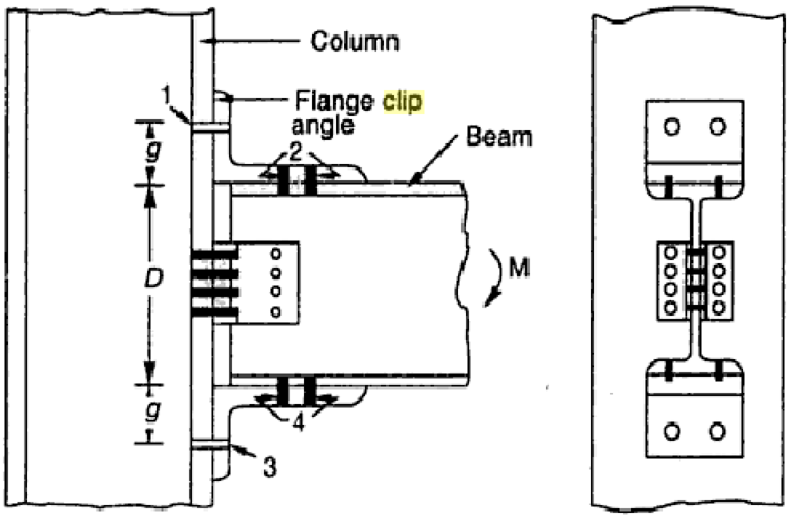


Figure 3.3: Clip and Seating Angle Connection

seat angle must be stiffened when the reaction from the beam is too large or when the seating leg is not able to provide the bearing area.for this reason additional angle called stiffener angle is provided.Fig3.3 and Fig3.4 shows the arrangement for clip angles and seat angles. **Design Steps for Un-stiffened Seat Connection**

A. Seating angle have length equal to width of beam.

B. The length of the outstanding leg

$$b = [R/t_w(f_{yw}/\gamma_{mo})] \tag{3.1}$$

where R= reaction of beam t_w = thickness of the web f_{yw}= yield strength γ_{mo}= partial safety factor

C. A dispersion of 45 degree is taken.so length of bearing on the cleat

$$b_1 = b - (T_f + r_b) \tag{3.2}$$

T_f= thickness of the web of beam r_b= root radius of beam flange. The distance of end bearing on cleat to root angle.

$$b2 = b1 + g - (t_a + r_a) \tag{3.3}$$

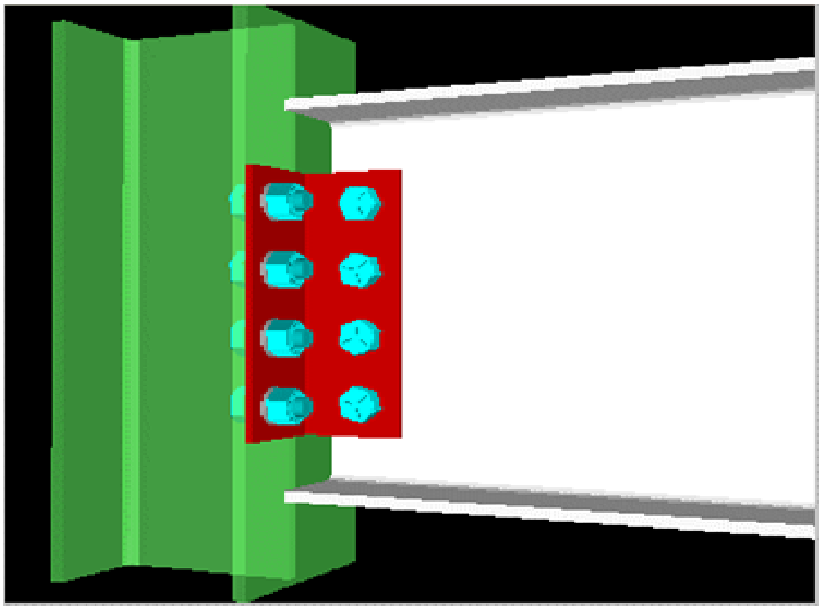


Figure 3.4: Clip and Seating Angle Connection

t_a=thickness of angler_a= root radius of angle g=clearance and tolerance

D. Connected leg is so chosen that at least two horizontal rows of bolts can be accommodated

E. Bending moment of seat angle outstanding leg

$$M_u = R(b_2/b_1) \times (b_2/2) \tag{3.4}$$

Moment capacity is calculated as

$$M_d = 1.2 \times Z(f_y \times \gamma_{mo}) \tag{3.5}$$

If $M_d < M_u$ then section is revised

F. Shear capacity of outstanding leg

$$V_{dp} = w \times t \times f_y/(\sqrt{3} \times \gamma_{mo}) \tag{3.6}$$

Value should not be more the reaction of the beam.

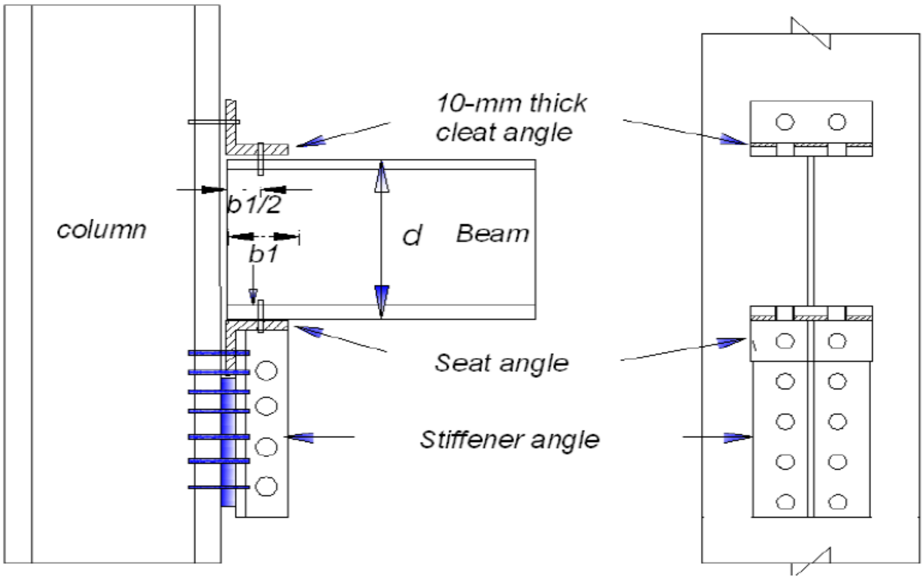


Figure 3.5: Stiffened Seat Connection

G. The number of bolt is calculated as

$$n = Reaction/Strenghtof Bolt \tag{3.7}$$

H. A cleat angle of nominal size is provided on the top flange of the beam connected by two bolts on each of its legs.

Design Steps for Stiffened Seat Connection Fig3.5 and Fig3.6

A. Assume the size of seat angle on the basis of bearing length.

B. Provide bearing area for stiffener of the out standing leg outstanding leg must not exceed 14*ϵ where $\epsilon = \sqrt{250/f_y}$

$$A_{br} = R/(f_y/\gamma_{mo}) \tag{3.8}$$

where R= Reaction of the Beam f_y= yield strength γ_{mo} = Partial safety factor The thickness of the stiffener angle should not be more than web of the angle

C. Due to the stiffener , the seat angle is not flexible and the Bolts in the connecting leg are subjected to moments in addition to shear

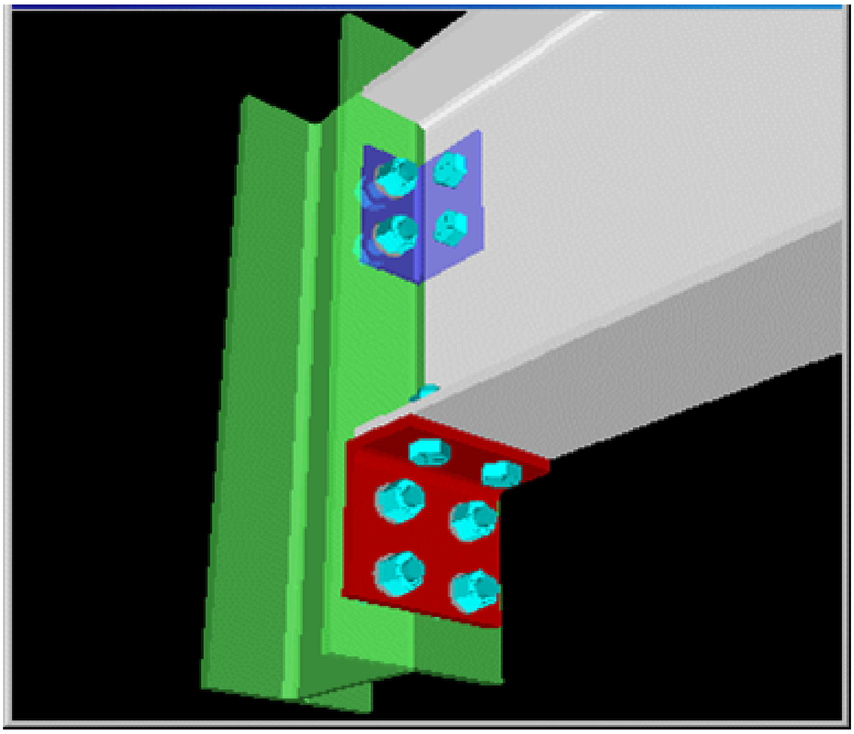


Figure 3.6: Stiffened Seat Connection

D. Eccentricity , the bending moment , and the tension acting in the critical bolts are computed.

E. A cleat angle of nominal size is provided on the top flange of the beam connected by two bolts on each of its legs.

3.3.2.2 Web Angle Connection

A web cleat connection also known as web cleat or angle cleat connection is used to transfer Beam reaction through Web angles either to the the flange or to the web of the supporting member.One or more angles cleats can be used but usually Double web cleat connection is preferred.This connection has very little moment connection.Fig3.7 and Fig3.8 shows arrangement for single web angle connection.

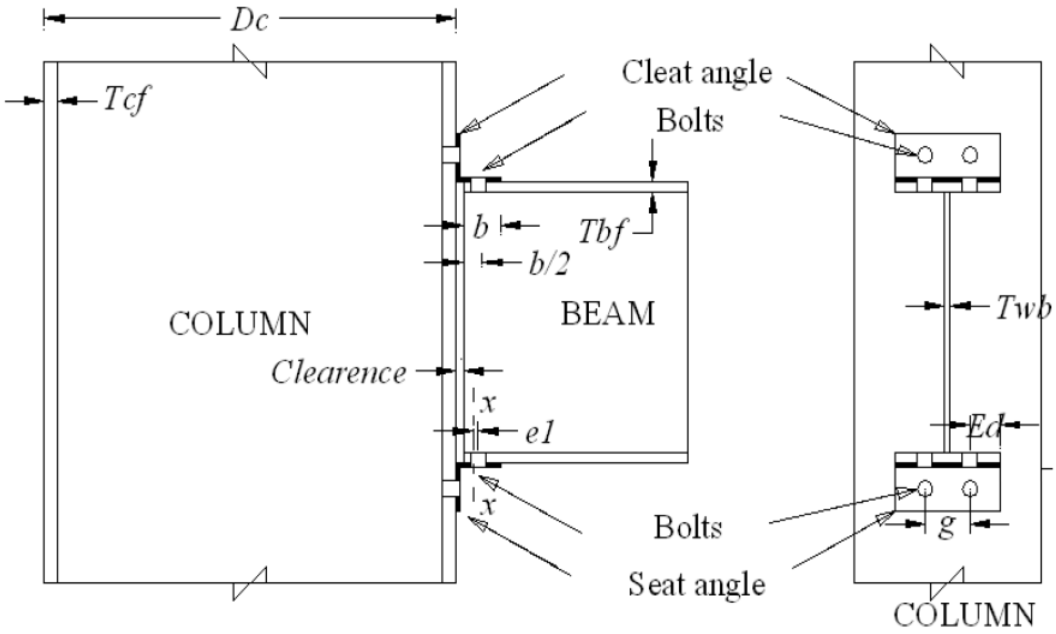


Figure 3.7: Web Angle Connection

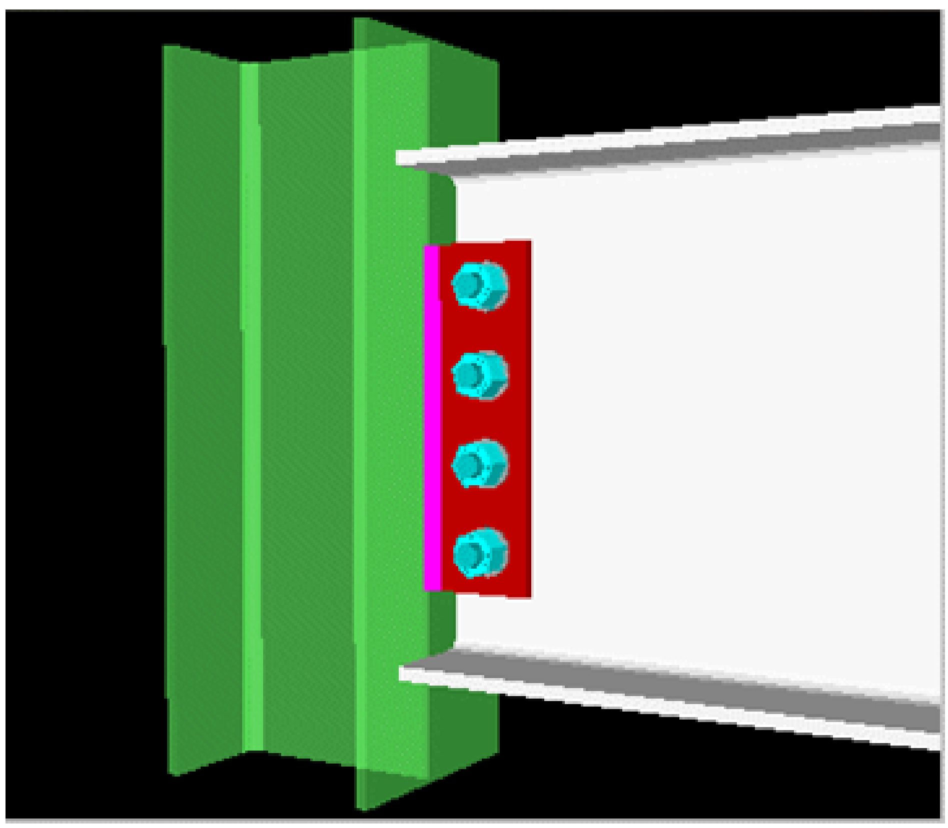


Figure 3.8: Web Angle Connection

Design Steps for Web Angle Connection

a. The beam is designed as a simply supported beam and the supporting member is designed for the eccentric beam reaction.

b. The length of the web angle is decided based on the number of the bolts and pitch of bolts. In general length is kept 0.6 to 0.75 times the depth of beam.

c. Bolts are designed for shear force only

d. A minimum thickness of 8 mm is kept Thicker angle decreases the flexibility of joint.

e. Joint is kept flexible keeping gauge distance in range of 100 to 140 mm.

3.3.2.3 Header Plate Connection

Flexible End plate connection is also called as header plate connection consists of an end plate connected to the web of the beam, at the beam ends by fillet welds and this plate is connected to column flanges or web by means of bolts.as shown in Fig 3.9

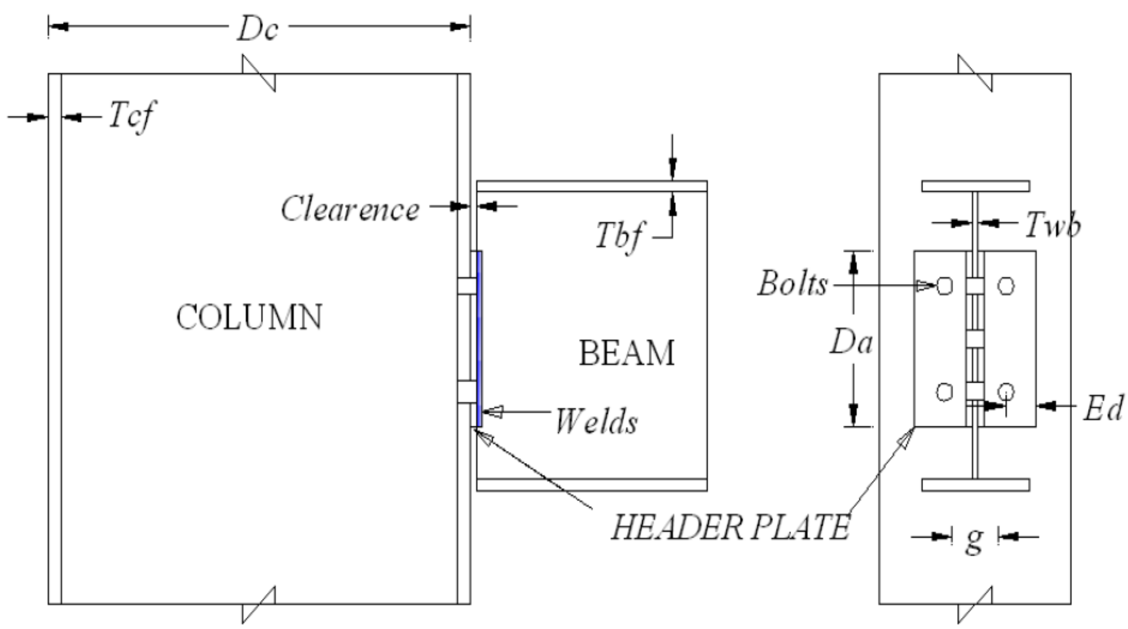


Figure 3.9: Header Plate Connection

Design Steps of Header Plate Connection

a. An empirical thickness of 8 mm is used for beams having a depth of 450mm and 10mm for beams having height more than 450mm.

b. End plate thickness is kept minimum to reduce end moments.

c. Length of portion a = 30t,
where, t = thickness of end plate
Length of cut portion = 20tw,
t_w = thickness of web

d. Beam is designed for zero end moments. Column is designed for eccentric beam reaction

e. Reaction from beam is transferred by weld shear to end plates, by shear and bearing to bolts. Weld is referred by its leg size. Design strength of a shop site welded

$$s = V/(Lwfwd) > 6mm \tag{3.9}$$

Where, s=size of welds.
V= Reaction.
L_w= Length of welds.
f_{wd} = design strength in weld.

3.3.3 Moment Connection

Moment resistant connection is capable of transferring moment,axial force and shear from one member to another as shown in Fig3.10.These kinds of connections are used in Framed structures, where the joints are considered rigid. Another kind of situation is one in which the eccentricity of load from the centroid of the fasteners group may cause forces and moments on the joint.

Different types of moment connections are as follows

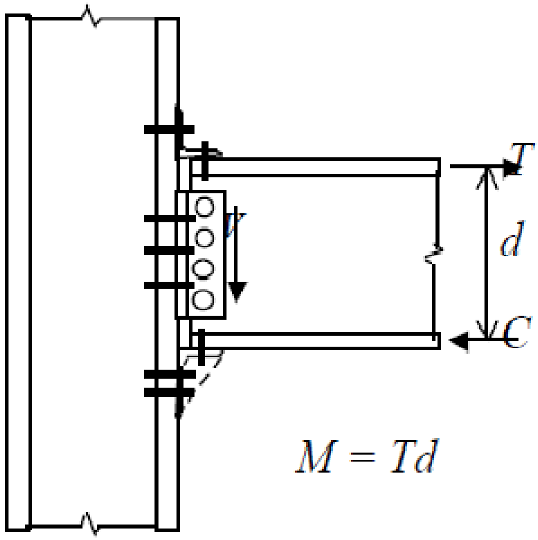


Figure 3.10: Moment Connection

- End plate connections.
- T-stub connections.
- Flange angle connections.

3.3.3.1 Bolted End Plate Connection

Bolted Bolted end plate connection is used to transfer moment, axial force and shear force from one member to another. End plate is fillet welded to web and flange of beam, and bolted to column flange or web as shown in Fig3.11and Fig3.12. Due to concentration of forces in beam, web of column may require additional strengthening in form of web stiffener, diagonal stiffeners and web plate. End plates may be extended beyond the column webs to carry more moments This type of connection can transfer about 80 of the yield moment capacity of the beam.Tn these connections the bending moment, axial force and shear are transferred by tension and compression or shear through the flange welds to the end plate. then they are transferred from the end plates to the bolts by bending and shear.

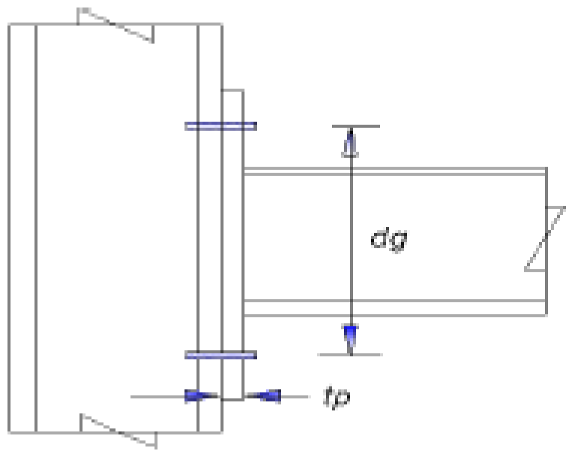


Figure 3.11: Bolted End Plate Connection

Design steps for Bolted End Plate Connection

a. Assume the dimensions of end plate based on dimensions of the Beam and column Flange.Layout of bolts is also fixed by providing nominal minimum distance of 50 - 60 mm from flanges and web of beam.

b. Connection is assumed to pivot about hot spot and loads in bolts are assumed to be proportional to their distances from centers of bottom flange. It is assumed that bolts in cantilever end of plate and bolts near top of beam flange carry equal loads.

c. Tension and shear capacity of bolt is computed. An approximate value of prying force is assumed Q= (tension capacitybolt load)

$$Q = (l_v/2l_e)(T_e - \beta\gamma b_e t^4/(27l_e(l_v)^2)) \tag{3.10}$$

d. Moment at toe of weld is calculated $M = T \times l_v - Q \times l_e$ where, T = Tension Capacity of Bolt,
l_v = distance from bolt centre line to toe of fillet weld or to half root radius for a rolled section;
l_e = distance between prying force and bolt centre line and is minimum of either end dist

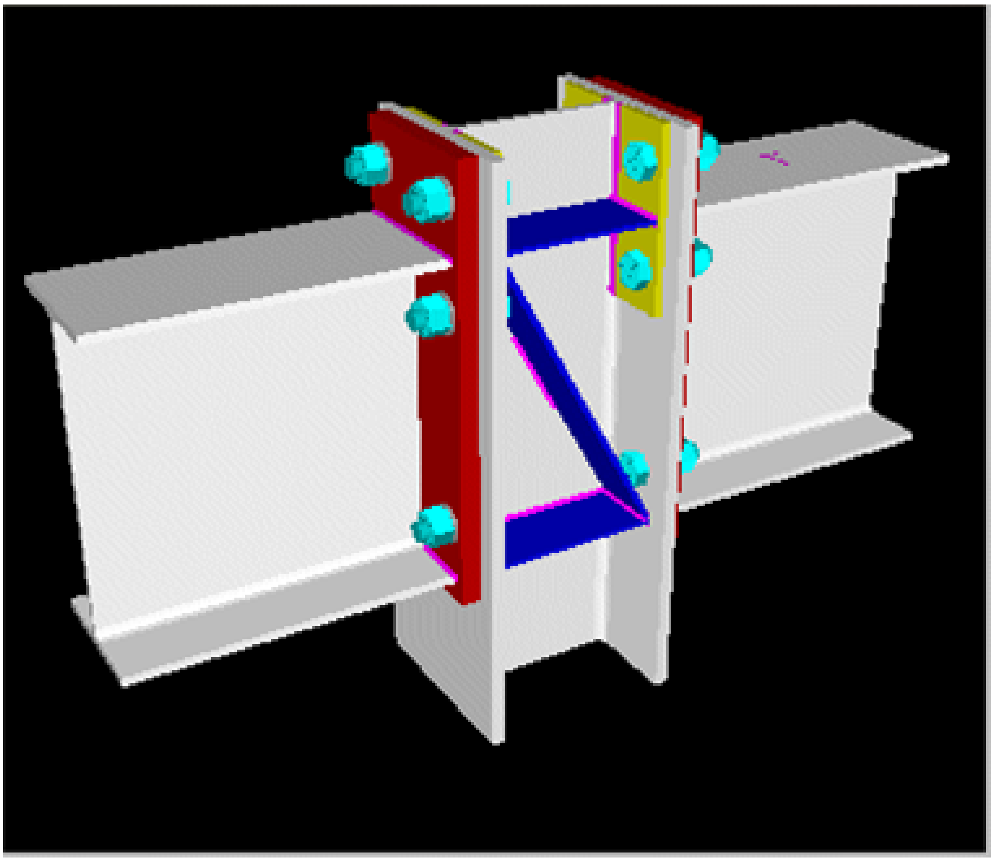


Figure 3.12: Bolted End Plate Connection

e. Moment capacity of plate thickness of end plate is calculated as

$$M = (f_y/1.10) \times ((wT^2)/4) \tag{3.11}$$

Where, f_y is yield strength.

T = thickness of end plate.

w = Effective length of end plate per bolt.

f. Prying force is computed using this thickness and checked with assumed prying force. If it is found more than the assumed prying force, Thickness has to be changed.

g. Check for combined shear and tension in the bolt is required.

h. Check for the capacity of the flange plate to carry the reaction due to bolt forces.

i. Welds connecting the beam to end plate is checked.

j. capacity of column web to support the reaction is checked.If column web is not sufficient then web has to be strengthened by means of web plates. Column web stiffener at beam compression (Bottom flange of beam)

(1) Local web yielding

$$P_{bf} = (f_{wc}t_{wc}(5k + t_{fb}))/(\gamma_{mo}) \tag{3.12}$$

$k = t_{fb} + r_1, f_{wc} = 250N/mm2.$ t_{wc} = thickness of column web.
t_{fb} =thickness of beam flange.

(2) compression buckling of web

$$P_{bf} = 101710(((t_{wc})^3/h))(\sqrt{f_{wc}1.1}) \tag{3.13}$$

t_{wc} = thickness of column web.
$h = D - 2 \times t_{fb}$
if $P_{bf} \leq$ beam flange force .then it is min of above two
Provide stiffener to resist excessive force in beam flange
Required $A_{st} = (P_f - P_{bf})/(f_y/1.1)$
Local buckling limit =b/t =9.4
b=width of stiffener.
t=thickness of stiffener.
Same stiffener is provided for tension flange

3.3.3.2 Flange Angle Connection

An additional pair of angles in the seat angle connection is used to connect the web of the beam to the flange of the column as shown in fig it is known as clip angle connection or light moment connection. This connection consists of four angles(two pairs) .one pair of angle is used to connect the web of the beam with the column flange (one angle on each side of the web) and the other pair of angles (called clip angles) is placed one on top of the beam flange and other below the bottom flange of the beam.
fig 3.10

3.3.3.3 T-stub connection

Instead of an angle, a T-Stub cut from an I-section can be used to clip top flange of an I-beam as shown in Fig, deformation of T is symmetrical about point B. so that tangent at B remains vertical. P is the tensile force in the bolt, the moments in the T will be Fig3.13 $M_A = M_B = 0.5pg$

Design steps for T-stub connection

a. Split beam to beam top flange connection Flange force, P = Moment in beam/Depth of beam
   Thickness of T-web,$t = P/(bxf_y/(\gamma_{mo})$
   where, $b = B_f - 2\phi_{br}$
   f_y=yield stress,
   $\gamma_{mo} = 1.1$

b. Moment capacity of T-Stub flange

$$M = 1.2fybftf2/(1.1 * 6000) \tag{3.14}$$

   b_{fb}=width of beam flange, t_f= thickness of beam flange

c. Check for Prying Force

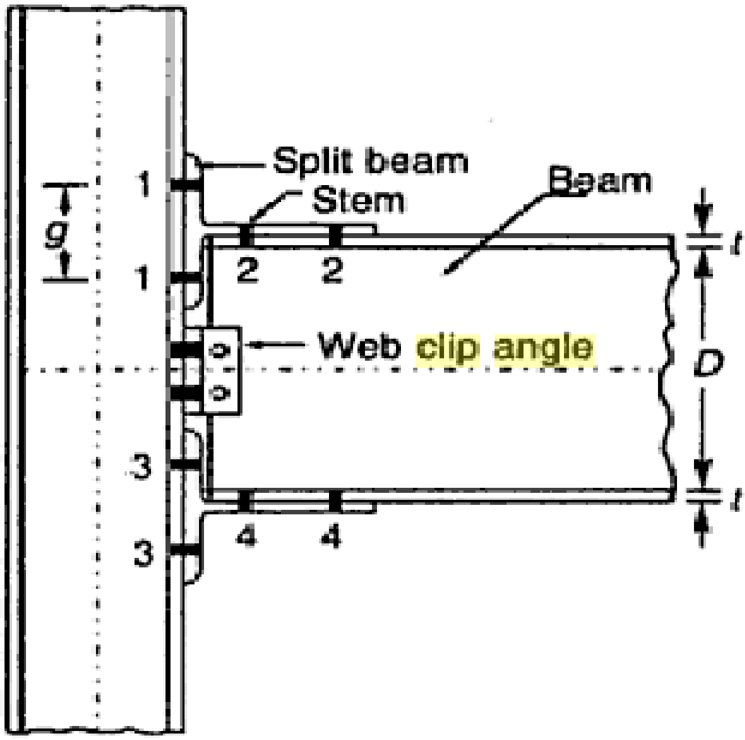


Figure 3.13: T stub Connection

d. Check for thickness of column flange

$$\sqrt{\frac{\frac{4nF_tc}{af_y}}{4\sqrt{2}+(n-1)\times s_b/a}} \tag{3.15}$$

where, n=No of bolts to connect,

$S_b = b_{fb} - 2E_d, E_d$ is Edge distance.

F_t= tensile force in bolts, $a = (b_{fc} - t_w)/2$

3.4 Welded Connection

3.4.1 Advantages Welded Connection

a. Welded connection eliminates the need for making holes in the members.since the holes at the ends governs the design of bolted connections (edge distance) , a welded connection results in a member with a smaller gross section.

b. It offers airtight and watertight jointing of plates so used for water/oil storage tanks,ships.

c. Welded joints are economical since they enable direct transfer of stresses between the members.Approximately 15 percentage saving is achieved then bolted connections

d. Welded structures are more rigid structures.

e. Aesthetic in appearance and appear less cluttered in contrast to bolted connection.

f. Welding is practicable even for complicated shapes of joints.

3.4.2 Shear Connection

shear type of connections are as follows:

a. Angle set connection

b. Web angle

c. end seat connections

d. end plate connections

3.4.2.1 Un-stiffened Seat Angle Connection

In this connection angle is designed to carry the entire reaction. The thickness of seat angle is determined by the flexural strength at a critical section of the angle as shown in Fig3.14. When the beam is attached to the seat with bolts,rotation of the beam at the end creates a force that tends to restrain the pull away from the column so critical section for flexure is near the base of the fillet on the out standing leg.For practical purpose,the distance c may be taken at 9.5 mm from the face of the angle.the bending moments on the critical section of the angle and on the connection to the column is determined by multiplying the reaction with a critical distance.

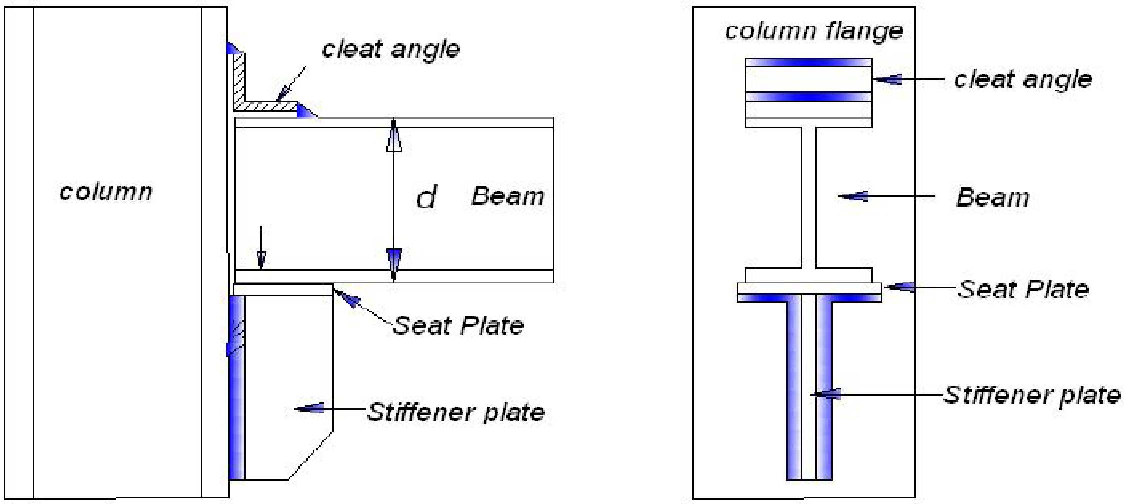


Figure 3.14: Un-stiffened Seat Angle Connection

Unstiffened Seat Angle Connection

a. Selection of a seat angle having a length equal to width of beam.

b. Length of outstanding leg of a seat angle is calculated on basis of web crippling of the beam.

$$b = (R/(t_w(f_{yw}/\gamma_{m0}))) \tag{3.16}$$

where, R is reaction from beam.
t_w is thickness of web of beam.

f_{yw} is yield strength of web of beam.

γ_{m0} is partial safety factor of material

Determine length of bearing on cleat.

$b_1 = b - (T_f + r_b)$

where,T_f is thickness of beam flange.

r_b^4 is root radius of beam flange.

c. Calculate distance of end bearing on cleat to root angle

$$b_2 = b_1 + g - (t_a + r_a) \tag{3.17}$$

wheret_a is thickness of angle.

r_a is radius of angle.

g is clearance and tolerance

d. Reaction is assumed to be uniformly distributed over bearing length b2. Bending Moment,

$$M_u = R(b_2/b_1)(b_2/2) \tag{3.18}$$

By equating bending moment to strength of solid rectangular section bent about its weak axis, thickness of weak axis is determined

e. Determine required weld size.

(1) Without taking eccentricity, length of weld on each side can be evaluated by using,

$$L_W = R/(2R_W) \tag{3.19}$$

$R_W = strength of weld per mm$

(2) Considering eccentricity, resultant force in weld due to shear and bending

is given by

$$R_W = (R/(2(L_W)^2))(\sqrt{(((L_W)^2 + 20.25(b_2/2)^2))}) \tag{3.20}$$

b_2= Distance of end bearing on cleat to root angle

3.4.2.2 Web Angle Connection

The field-Welded shear connection using web angle is shown in fig.Angles are kept flexible so that beams are capable of rotating at the ends to provide simply supported end condition and designed to transmit shear only. The pair of angles are shop welded to the beam and field welded or connected to the column by means of HSFG bolts.The angles projet out of the beam web by a distance of about 12 mm so that beam can be fitted with the acceptable tolerances.

Erection bolts are used to erect beams and then the angles are welded at site.Bolts are often provided at the bottom of the angles.Length of angle is kept equal to distance between fillets of beam, so that sufficient length is available for welding. Usually a weld size 2-3 mm smaller than web angle thickness is chosen.Connection though assumed to transfer only shear forces due to eccentricity is also subjected to a bending moment. Due to rotation effect, field welds motive web angles to press against beam web at top and tear apart from bottom and thus indicating horizontal shear in fillet weld. It is assumed that neutral axis is at a distance of L/6 from top of angle. Horizontal shear is taken as zero at this point and maximum at bottom of angle Applied moment from load = resisting moment of weld

$$(P/2)e_2 = (2/3)RL \tag{3.21}$$

Where L is length of weld e_2= eccentricity of weld from reaction acting P = reaction on beam

$$Hence, R = 0.75Pe_2/L \tag{3.22}$$

From force triangle,

$$R = 0.5R_x(5/6)L \tag{3.23}$$

From above equations

$$R_x = 9Pe_2/(5L^2) \tag{3.24}$$

Vertical force on weld,

$$R_v = P/(2L) \tag{3.25}$$

Resultant Force on weld,

$$Rres = \sqrt{(9Pe_2/(5L^2))^2 + (P/(2L))^2} \tag{3.26}$$

Equation neglects eccentricity e1, which tends to cause tension at top of weld lines. Flexural tension components Rx at top of weld B is

$$R_x = My/I = Pe_1(L/2)(2L^3/12) = 3Pe_1/L^2 \tag{3.27}$$

thus

$$R_{res} = \sqrt{(P/(2L^2) + (3Pe_1/L^2)^2)} \tag{3.28}$$

Considering returns to be equal to L/12,

$$R_{res} = P/(2L^2)\sqrt{(L2 + 20.25e_1^2)}N/mm \tag{3.29}$$

3.4.3 Moment Connection

3.4.3.1 Stiffened Beam Seat Connection

A welded stiffened seat connection consists of two plates forming T or a split I section used as seat as shown in Fig3.15.The thickness of the stem of the T should not be less than the web thickness of the beam it supports.The thickness of seat plate should not be less than the thickness of the flange of the beam.The depth of the stem should be short enough to avoid local buckling and it depends upon the length of the vertical weld required.Yhe under side of the flange of the T is also welded to increase the torsional stiffness of the connection.The seat plate is kept wider than the flange of the beam by at least twice the size of the weld on each side of beam flange to facilitate welding.

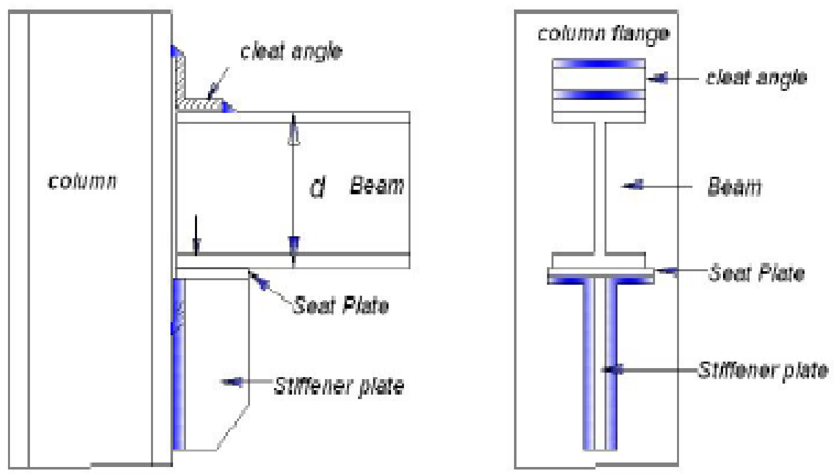


Figure 3.15: Stiffened Beam Seat Connection

Design Steps for Stiffened Welded Seat Connection

a. The width of the seat angle is calculated.

b. The thickness of the seat plate is chosen as equal to the thickness of the flange plate.

c. The thickness of stiffening plate chosen as equal to the thickness of the web of the beam.

d. The eccentricity of the load and the bending moment due to it are calculated.

e. Vertical and horizontal shear stresses due to the shear force and bending moment are calculated.The resultant shear of the weld is computed as the vector sum.

f. And finally weld size is decided.

3.5 Semi-Rigid Connections and Frames

Semi-rigid frames can be, in some instances, a more economical choice of framing system than the other alternatives. In some sense they offer a solution to the problem of choosing between a moment frame and a braced frame. With braced frames, the connections are cheap, but the structural members need to be larger than they would be in a moment or semi-rigid frame. The reason is that, for typical loadings of a distributed load along the beam, fixity of the connections shifts the moment diagram upward from the simply supported case. This means that the total moment is shared between the positive moment at mid span and the negative moment at the supports, thereby decreasing the design moment. For example, the moment diagram for a simply supported beam with a factored distributed load of 14.59 KN/m and ISMB 450 section is shown in Figure For this simply supported beam of 7.44 m , the design moment which the member must be able to resist is 100.95 kNm., In comparison, the

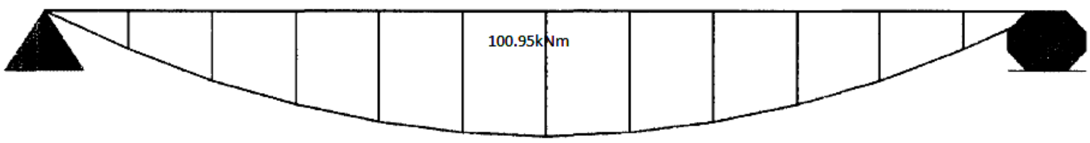


Figure 3.16: Simply Supported Beam

moment diagram for a fixed-fixed beam with rigid connections is shown in Figure . The fixity at the supports shifts the moment diagram upward, and now the design moment for the beam is only 48.59 kNm., as compared to 100.95 kNm. for the simply supported case.

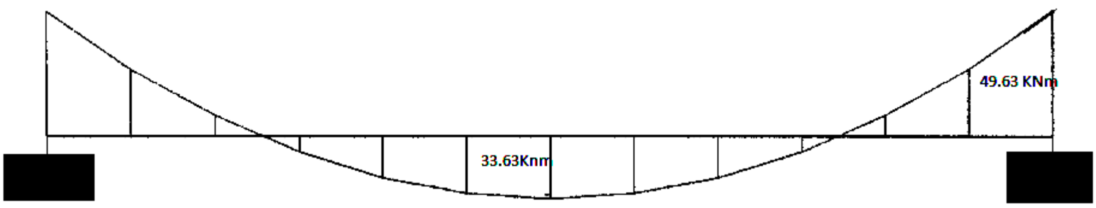


Figure 3.17: Fixed Beam

By comparing these diagrams. the fixed-fixed beam has a smaller maximum I moment, which is approximately 100.95 kNm., compared to the maximum moment of the simply supported span. Therefore, the fixed-fixed beam has a smaller requirement for the flexural strength of the section. However, this design is not as economical as it could be, because the difference between the negative moment and the positive moment is not minimized. To reach the optimal design moment condition, the moments at the support and at mid span should be equal or close to equal. A semi-rigid beam with a partial fixity factor, which is the relationship between moment and rotation, of 1000 kNm/degree gives favorable results, as shown in the moment diagram in Figure . In this design, the positive moment at the support and the negative moment at mid span are very similar. Therefore, the semi-rigid beam need only be designed for a flexural strength of 52.59 kNm. as opposed to 100.95 kNm. for the pinned-pinned case and 33.35 kNm for the fixed-fixed case.

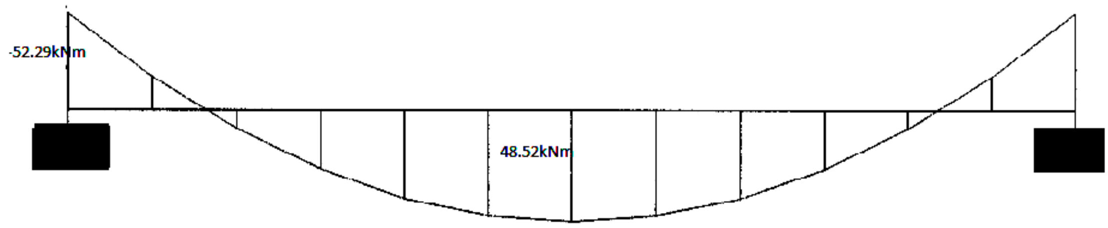


Figure 3.18: Beam With Rotational Stiffness

An analysis program, Staad pro, can model structures with semi-rigid connections. When choosing joint releases, the user can either choose to completely release the moment transfer between connected elements or to specify a partial fixity factor. The partial fixity factor is the relationship between the moment and the rotation at the connection, or the equivalent rotational spring constant. For example, the moment diagram shown above in Figure 3 shows the moment diagram for a beam with partial fixities at both ends of the beam of 1000 kNm/degree. In order to check that the results are accurate, it is possible to determine the equation for the moment diagram, integrate it to find the rotation equation, and check that the moment and the rotation at the ends of the beam are related by the specified partial fixity factor. For the beam as shown above in Figure , this check was performed. With the loading of 14.59kN/m plus the dead load o so dead wt of section ISMB 450 so total wt = 14.59 +.71 =15.3

kN/m

Shear force $=\frac{15.3*7.44}{2}= 56.9616$

V $= 15.3x - 56.916$

M $=\int V(x)dx$

$= \frac{15.3x^2}{2} + 56.916x + C_1$

$Mx = EI\frac{\partial^2 v}{\partial x^2}$

$Mx = EI\frac{d\theta}{dx}$

$\frac{M}{EI} = \frac{1}{R} = \frac{d\Theta}{dx} = \frac{\partial^2 y}{\partial x^2}$

$k = \frac{\partial \theta}{\partial x}$

$\theta_x = \int \frac{M}{EI}\partial x$

$=\frac{1}{EI}(-\frac{15.3x^3}{6} + \frac{56.916x^2}{2} + C_1 x + C_2)$

$\theta = \frac{dv}{dx}$

V $= \int \theta_x$

$=\frac{1}{EI}(-\frac{15.3x^4}{24} + \frac{56.916x^3}{6} + C_1\frac{x^2}{2} + C_2 x C_3)$

For ISMB 450 $I_{xx} = 303908000mm^4$

using the boundary condition

$k(0) = \frac{M(0)}{1000}$

at x = 0 , $\theta = 0$,v = 0

at x = L , $\theta = 0$,v = 0

we get$C_1 = 46.90$

$C_2 = 655.58$

$C_3 = 0$

M_x =47.17 kNm

$k_0 = \frac{M_0}{EI*Partial fixidity}$

$= \frac{47.18}{2*10^5*303908000*1000} = 0.0165 radians$

3.5.1 Connections as per I.S 800-2007

According Is 800-2007 code there are eight types of connections.

3.5.1.1 Simple Connection

Simple connections are assumed to transfer only shear at some nominal eccentricity.Types of simples connection are

a. Single web angle.

b. Double web angle.

c. Top and seat Angle with Double web angle.

d. Top and seat Angle without Double web angle.

e. Header plate.

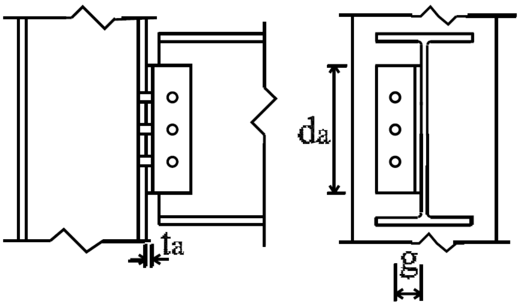
3.5.1.2 Rigid Connection

Rigid connections transfer significant moments to the columns and are assumed to undergo negligible deformation.Types of Rigid connection are

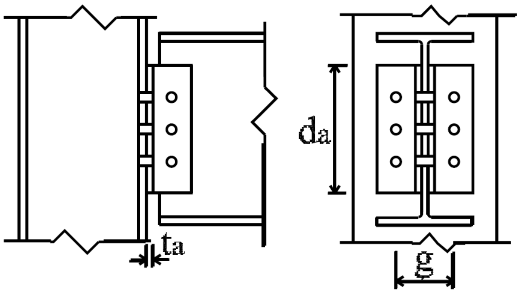
a. End plate without column stiffeners.

b. End plate with column stiffeners.

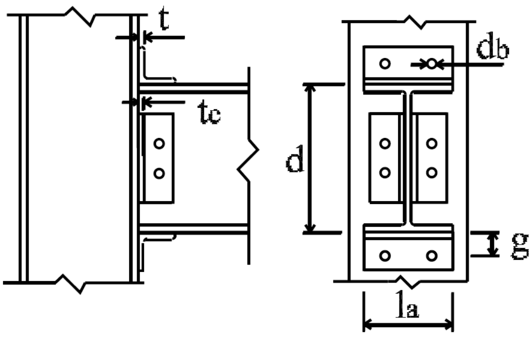
c. T-stub connection.



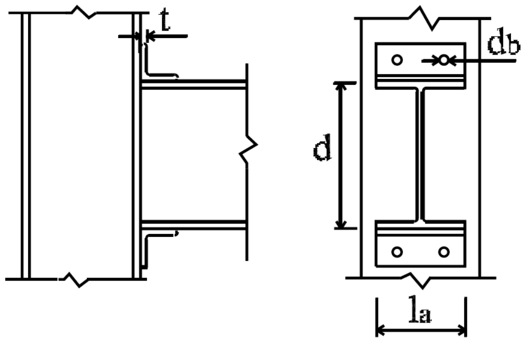
(a) Single web angle



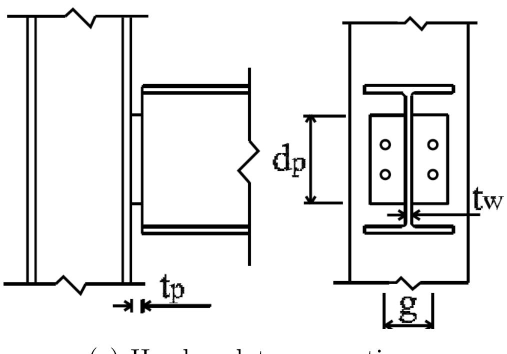
(b) Double web angle.



(c) Top and Seat Angle with Double web angle.

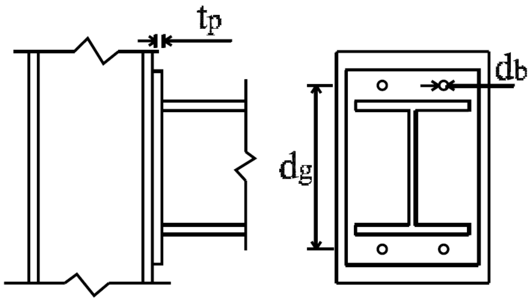


(d) Top and Seat Angle without Double web angle.

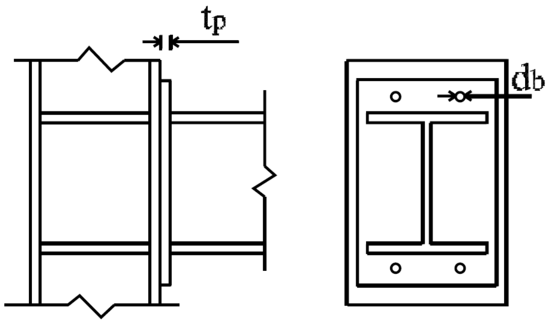


(e) Header plate connection

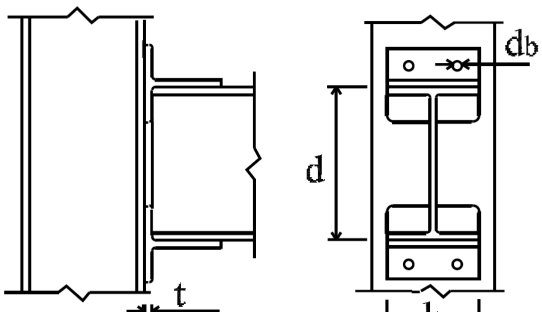
Figure 3.19: Shear Connections as Per Is 800- 2007



(a) End Plate Without Column Stiffeners.



(b) End Plate Without Column Stiffeners.



(c) T-stub Connection

Figure 3.20: Moment Connections as Per Is 800- 2007

3.6 Moment Rotation Curves

The concept of semi-rigidity has been around for a long time,but one of the reasons why this concept is not more often used in practical design of frames is that it is difficult to model.one of the key elements in semi rigid connections is the moment rotation curve. For example , in a beam column connection moment connections are modelled with the assumption that no relative rotation between the beam and column occurs, where as for pinned connections, it is assumed that the beam is free to rotate from the column. for semi rigid frames, the response is somewhere between these two cases and that is why the moment-rotation curve is so crucial.Moment rotation curves for various connections is shown in Fig3.21

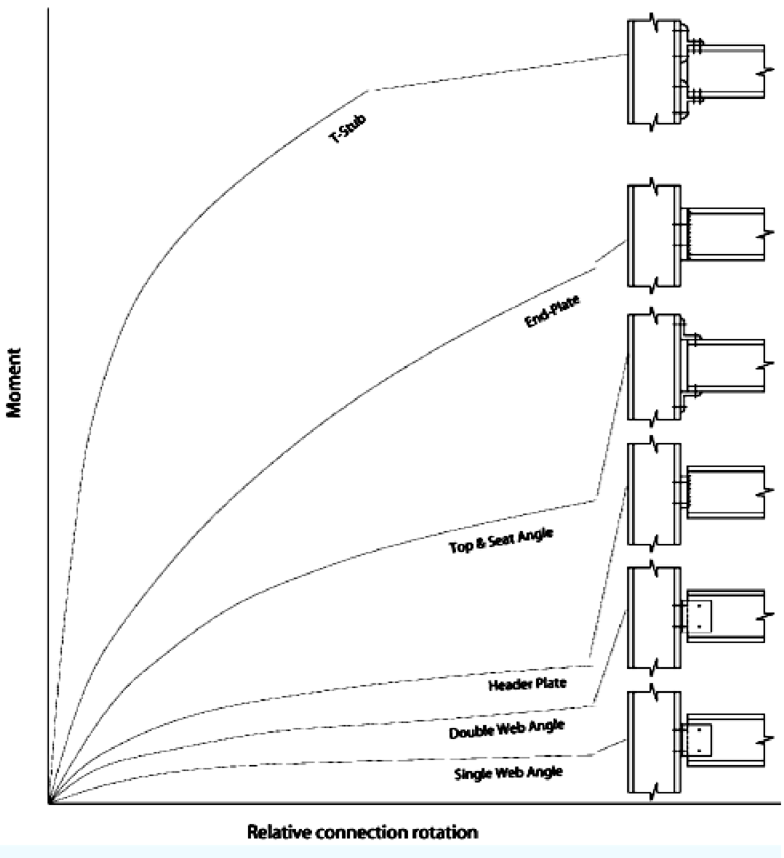


Figure 3.21: Moment Rotation Curves

Moment rotation curves are non linear.For smaller values of moments the rotation increases more quickly than it does for higher values. This non linear behaviour is due mostly to local yielding of parts of the connection, and also to material discontinuity,Stress concentrations , local buckling in the vicinity of the connection, and changes in the geometry under the loading.The nonlinear behaviour can be exacerbated in certain types of connections,especially bolted connections, where slip can cause a higher rotation than that caused the moment.

3.7 Modelling of Connection

The important parameters of a moment-rotation curve are the initial stiffness, R_k, which is the initial slope of the curve, the secant stiffness,R_b, which is the effective stiffness of the connection, the tangent stiffness,R_{kt}, which is the instantaneous stiffness, which decreases with increasing moment, and the unloading stiffness, R_k, which is the result of load removals or reversals. Fig3.22 shows these stiffness parameters.

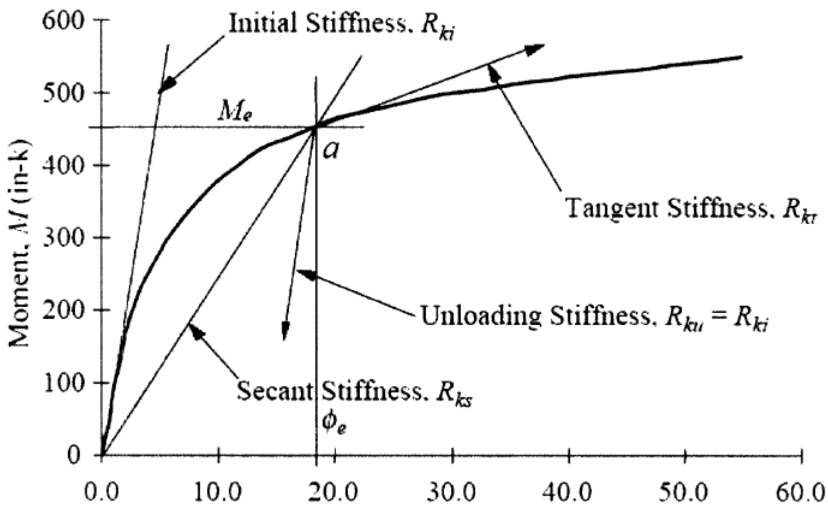


Figure 3.22: Modelling of Connection

This curve can be obtained by a number of different methods. One method is by experiment. In this case, care must be taken to ensure that the test conditions are an accurate representation of the design problem. The moment-rotation curve of the actual connection being designed is not known exactly, and the amount of variability in connection behavior makes this approach uncertain. In order to convert the moment rotational empirical data into a format that is useful for design, curve-fitting techniques are necessary which approximate the actual curve are necessary.

3.7.1 Frye And Morris Model

The Frye and Morris model is based on an odd-power polynomial representation of the momentrotation curve.

$$\Theta = C_1(KM) + C_2(KM)^3 + C_3(KM)^5(1 \tag{3.30}$$

where,

K is a parameter depending on the geometrical and mechanical properties of the structural detail.

C1, C2 and C3 are curve-fitting constants.

Secant stiffness values have been evaluated based on secant stiffness at a rotation of 0.01radian.

Table 3.2: Connection Constants in Frye -Morris Model

Sr. No.	Connections type	Curve fitting constants	Standardization constants
1	Single web angle	$C_1 = 1.91 \times 10^4$ $C_2 = 1.30 \times 10^{11}$ $C_3 = 2.70 \times 10^{17}$	$K = d_a^{-2.4} t_c^{-1.81} g^{0.15}$
2	Double web angle	$C_1 = 1.64 \times 10^3$ $C_2 = 1.03 \times 10^{14}$ $C_3 = 8.18 \times 10^{25}$	$K = d_a^{-2.4} t_c^{-1.81} g^{0.15}$
3	Top and seat angle connection with double web angle	$C_1 = 2.24 \times 10^{-1}$ $C_2 = 1.86 \times 10^4$ $C_3 = 3.23 \times 10^8$	$K = d^{-1.287} t_a^{-1.128} t_c^{-0.415}$ $l_a^{-0.694} (g - 0.5 d_b)^{1.35}$
4	Top and seat angle connection without double web angle	$C_1 = 1.63 \times 10^3$ $C_2 = 7.25 \times 10^{14}$ $C_3 = 3.31 \times 10^{23}$	$K = d^{-1.5} t_a^{-0.5} l_a^{-0.7} d_b^{-1.10}$
5	End plate connection without column stiffener	$C_1 = 1.78 \times 10^4$ $C_2 = -9.55 \times 10^{16}$ $C_3 = 5.54 \times 10^{29}$	$K = d_g^{-2.4} t_p^{-0.4} t_f^{-1.5}$
6	End plate connection with column stiffener	$C_1 = 2.60 \times 10^2$ $C_2 = 5.37 \times 10^{22}$ $C_3 = 1.31 \times 10^{17}$	$K = d_g^{-2.4} t_p^{-0.6}$
7	T-stub	$C_1 = 4.05 \times 10^2$ $C_2 = 4.45 \times 10^{13}$ $C_3 = -2.03 \times 10^{23}$	$K = d^{-1.5} t_f^{-0.5} l_t^{-0.7} d_b^{-1.1}$
8	Header plate	$C_1 = 3.87$ $C_2 = 2.71 \times 10^5$ $C_3 = 6.06 \times 10^{11}$	$K = t_p^{-1.6} g^{1.6} d_b^{-2.30} t_w^{-0.5}$

3.8 Summary

Basic concepts of connections , its classification, design steps of connections , understanding of drawings of various connections , method to obtain moment rotation curve . A connection has some rotational stiffness so every connection is a semi rigid connection because it has some stiffness so its in importance in frames is presented in this chapter.

Chapter 4

Analysis of G+ 3 storey building

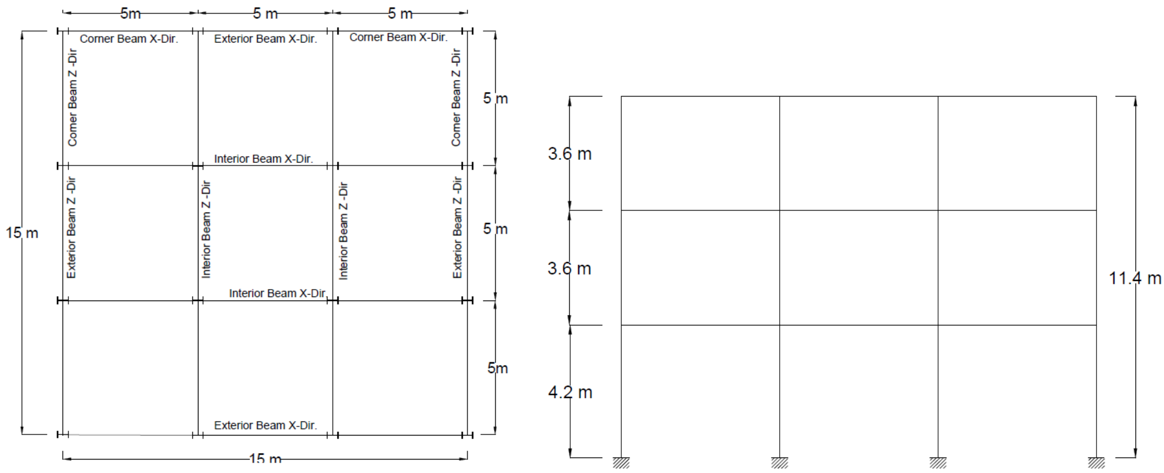
4.1 General

In this chapter, analysis of 3 storey moment resisting frame is presented. Modeling, and analysis of frame frame is carried out using STAAD PRO software. All load combinations are taken according to the IS 800:2007 [?] . The static analysis of wind loads as per IS:875(III)-1987 and earthquake loads IS:1893-2002. carried out using staad pro software.

4.2 Building Configuration

The 3 storey office building is having 9m x 9m square plan as shown in Fig.4.1(a). The plan and side view of the building are shown in Fig.4.1(b).Typical height is 4.2 m and storey height is 3.6 m. Total 3 models have been taken with varying span length of 5m ,7m and 9m.It is a Smrf (special moment resisting frame) building.

Design data:



(a) Plan of Building (b) Elevation of Building

Figure 4.1: Plan and Elevation of Building

No. Of stories	=3
No. Of bays in x direction	=4
No. Of bays in y direction	=4
Distance of bay in x direction	=5m
Distance of bay in y direction	=5m
Storey height	=3.6 m
Typical storey height	=4.2m

4.3 Loading Data

Following loads are considered for the analysis and design of structure;

Dead Load :

Dead load of slab =3.80 kN/m^2

Including =0.50 kNm^2 for partition walls

Self-weight of the structural members.

Live Load :

Live load =1.50 kN/m^2 for roof

Live load =3.00 kN/m^2

Wind Load:

Static wind loading is calculated as per IS: 875(III)-1987[?]

Location	:Ahmedabad
Basic wind speed	:39 m/sec
Terrain category	:II
Class	:A

Earthquake Load:

Earthquake Loading is calculated as per IS: 1893-2002[?]

For earthquake load	1893 - 2002
Soil type	= II (Medium soil)
Importance factor	=1
Zone	=V
Response factor	=5 (Smrf)

Initially the sizes of beam was taken as ISMB 250

and size of column was taken as ISHB 250

Load Combinations:

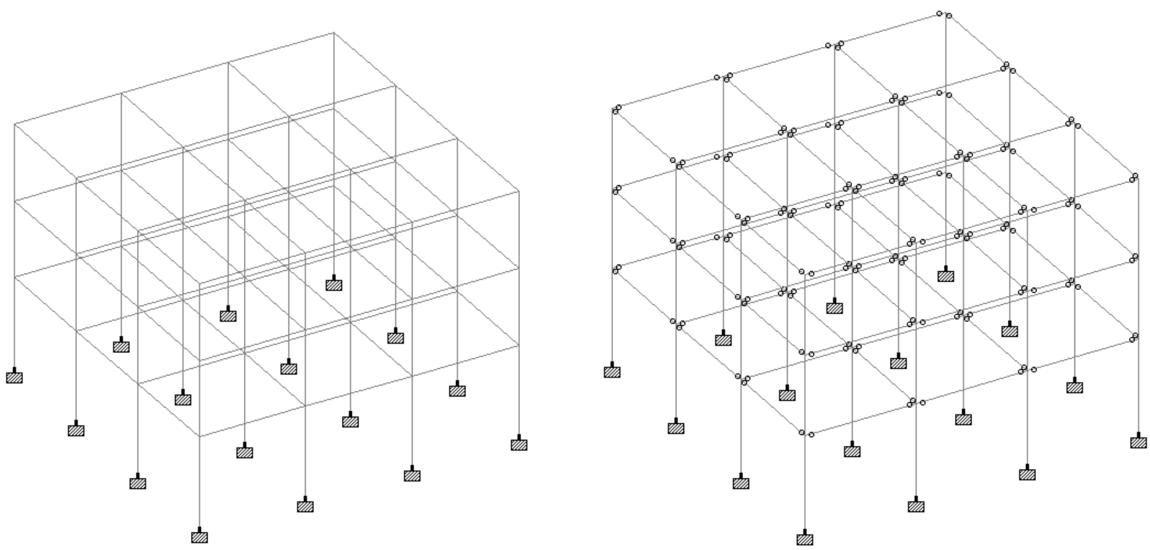
Load combinations are consider as per IS:800-2007 [?] for design of structure the load combinations for the analysis and design of structure as per IS:800-2007 [?].

1. 1.5(DL+LL)
2. 1.2(DEAD+LIVE+EQ_X)
3. 1.2(DEAD+LIVE-EQ_X)
4. 1.2(DEAD+LIVE+EQ_Z)
5. 1.2(DEAD+LIVE-EQ_Z)
6. 1.5(DEAD+EQ_X)
7. 1.5(DEAD-EQ_X)
8. 1.5(DEAD+EQ_Z)
9. 1.5(DEAD-EQ_Z)

10. 0.9DEAD+1.5EQ_X
11. 0.9DEAD-1.5EQ_X
12. 0.9DEAD+1.5EQ_Z
13. 0.9DEAD-1.54EQ_Z
14. 1.2(DEAD+LIVE+WL_X)
15. 1.2(DEAD+LIVE-WL_X)
16. 1.2(DEAD+LIVE+WL_Z)
17. 1.2(DEAD+LIVE-WL_Z)
18. 1.5(DEAD+WL_X)
19. 1.5(DEAD-WL_X)
20. 1.5(DEAD+WL_Z)
21. 1.5(DEAD-WL_Z)
22. 0.9DEAD+1.5WL_X
23. 0.9DEAD-1.5WL_X
24. 0.9DEAD+1.5WL_Z
25. 0.9DEAD-1.5WL_Z

4.4 Modelling and Analysis

In this section, modelling and analysis of Moment resisting frame and its connection are discussed. For analysis the beams and columns are modeled by beam elements and connection is modelled as rotational spring element having some specific value of stiffness. Analysis results of 3 storey Building are presented.



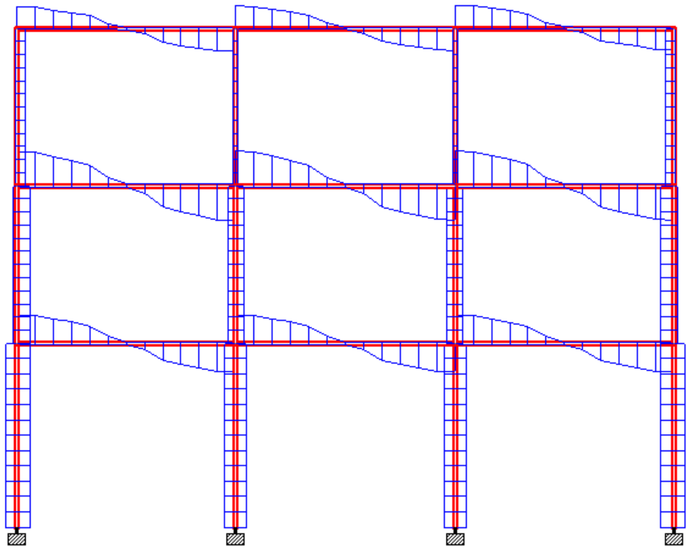
(a) Modelling (b) Modelling

Figure 4.2: Model of Building

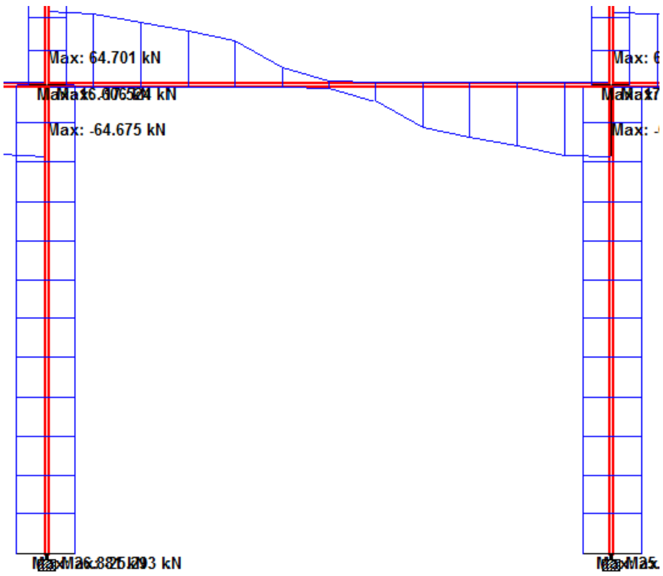
Following steps considered for modelling and analysis are as follows;

- Prepared the model of building as shown in Fig4.4 in staad-pro software.
- Assigned the fixed support at base .
- For Pin condition (Shear connection)?? release the the moments at the start and ends of the beam of beams.
- Applied the loads on the structure as mentioned in the section 4.3
- Assigned the section properties to the elements.
- Analyze the building.

4.4.1 Analysis Results of 3 Storey Building

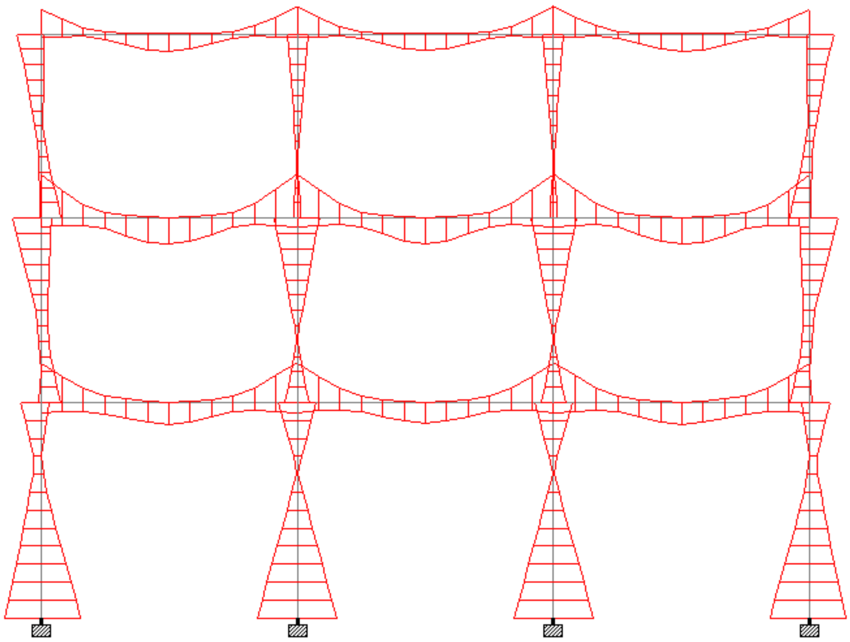


(a)

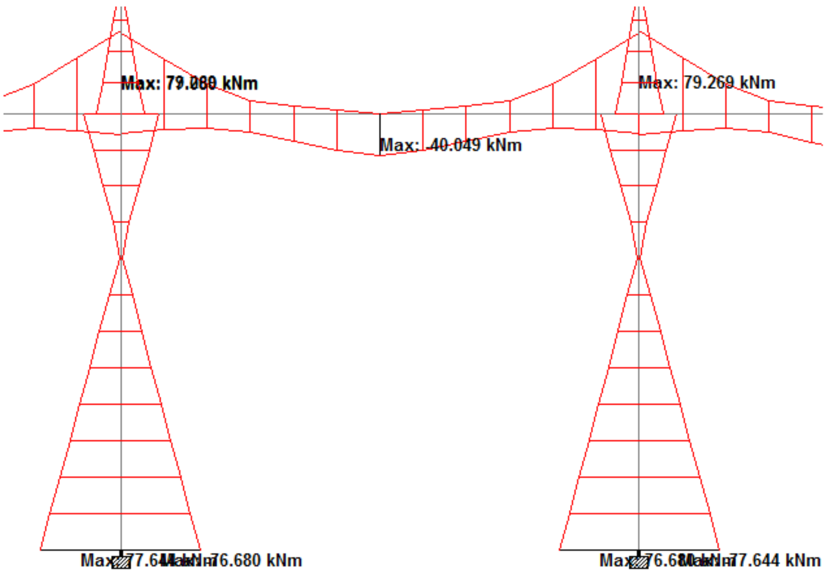


(b)

Figure 4.3: Shear Force for Interior Frame



(a) Bending Moment for Interior Frame



(b) Shows value of Bending Moment for Interior Frame

Figure 4.4: Bending Moment for Interior Frame

Table 4.1: Analysis Results of 3 Storey Building for 5 m Span

	3rd storey						**2nd storey**						**1st storey**					
	Interior		Exterior		Corner		Interior		Exterior		Corner		Interior		Exterior		Corner	
	Pin	Fix	Pin	Fix	Pin	Fix	Pin	Fix	Pin	Fix	Pin	Fix	Pin	Fix	Pin	Fix	Pin	Fix
X direction																		
BM kNm	**84.6**	31.5	84.6	34.5	43.2	17.6	131	49.2	131	51.2	66.6	25.9	108	40.2	54.9	42.5	54.9	21.6
End kNm	0	**53**	0	53.7	0	24.5	0	53.5	0	81.6	0	44.4	0	80.5	0	83.1	0	44.3
S.F **kN**	**52**	51.1	52	52.6	27	25.4	79.2	79.2	79.2	78.4	40.3	40.8	65.1	65.1	33.3	66.5	33.3	34
Z direction																		
B.M kNm	81.6	29.3	84.6	40.9	43.2	20.8	131	47.5	131	58.1	66.6	29.4	108	38.2	54.9	49	54.9	21.2
End kNm	0	**55.3**	0	58.9	0	30.1	0	83.9	0	90.3	0	49.1	0	83.2	0	95.7	0	54.3
S.F kN	51.1	**51.1**	52	57.2	27	29.4	79.2	79.2	79.2	86	40.3	43.7	65.1	65.1	33.2	73.2	33.3	39.8
Column																		
Axial F.(kN)	208	**215**	107	108	55.7	51.7	528	543	270	274	139	132	792	815	410	411	210	198
M_y **Top**	0	**13.7**	0	8.02	0	16.4	18.2	33.2	9.11	21.7	9.11	23.3	73	49.9	36.4	33.7	36.4	30.6
M_y **Btm**	18.2	9.87	9.11	7.77	9.11	14.5	73	30.3	36.5	19.5	36.25	19.5	**183**	56.25	91.5	37.4	91.5	35.9
M_z **Top**	0	**19.3**	0	46.3	0	24.5	18.3	40.1	18.3	51.7	9.14	27.7	**73**	41.5	36.1	41.7	36.1	22.6
M_z **Btm**	18.3	**5.23**	18.3	40.6	9.11	20.6	73.2	27	73.4	38.2	36.7	20.4	183	74.2	92	73.8	92	41.7
S.F **Top**	7.17	**7.26**	1.42	1.37	0.98	0.98	18.4	18.4	9.93	6.66	8.64	3.99	24.1	27.6	23.2	20.2	15.3	11.5
Btm	7	**6.85**	4.13	24.2	12.2	12.2	18.6	18.6	14.3	25	9.76	13.4	23.9	27	23.5	27.5	15.5	15.3

4.5 Summary

Here the general configuration of building , various load cases and load combination as per Is 800-2007 ,preparation of model for connection Steps of building model and the analysis result for fix case and pin case for all the three storey is given in this chapter.

Chapter 5

Design of G+3 Storey Building

5.1 General

In this chapter steps of design of beam, column ,various sizes of connection , moment rotation curve ,steps to find rotational stiffness of connections , moment rotation curve of connections for all the stories and Results of for bending moment,shear force of beams ,columns,axial force of column,time period ,displacement beam weight ,column weight and total weight. Results and discussion of for various span lengths.

5.2 Beam Design

5.2.1 Steps of Design of Beam

1. The design loads are obtained by summing up the loads multiplied by the appropriate partial safety factors.

2. A trial section is selected assumed and the Maximum Bending moment and shear force is calculated . It is going to be plastic section.

3. Required sectional modulus is found out.

4. A trial section is chosen Which has a section modulus greater than it.

5. Then it is checked for the class it belongs.

6. Check for bending strength.

7. Check for shear strength.

8. Check for web crippling and Buckling.

9. Check for deflection. If any check fails then section is revised.

5.2.1.1 Calculation of Beam Design

1. Select Section ISMB200
   Section

Properties:	ISMB200
D	=300mm
B	=100mm
t_f	= 10.8mm
t_w	= 5.7 mm
Z_{ezz}	= 223500 mm^3
Z_{pzz}	= 253860 mm^3
$M.I_{zz}$	=22354000mm^4
r	=13mm

Factored Load

2.

Span of Beam	= 5m
B.M	= 41KN.m
S.F	= 59KN
$Z_{(reqd)}$	= 372240 mm^3

$$M_d = \beta_b \times Z_p \times f_y \times \frac{1}{\gamma_{mo}} \leq 1.2 \times Z_e \times f_y \times \frac{1}{\gamma_{mo}} for simply supported beam. \quad (5.1)$$

where,

β_b=1.0 for plastic and compact section.

γ_{mo} = partial safety factor.

Z_e/Z_p for semi compact sections.

Z_p,Z_e = plastic ans elastic section modulii of the cross section respectively.

3. Section classification:

   Flange criterion= 4.6<9.4
   $(B/2 * t_f)$

   Web criterion= 44.6<84
   $(D - 2 * (t_f + r))/t_w$
   Plastic section

4. Moment of resistance of the cross section:

   Md= Z_pXf_y /γ_{mo} (Cl:8.2.1.2)

   Where Z_p is the plastic modulus

   Md= 57.69 KN.m > 41 KN.m

   Hence ISMB 200 is adequate in flexure.

5. Shear resistance of the cross section:

Shear capacity(V_c)= 0.6*$f_y * A_v/\gamma_m$ (Cl:8.4)

V_c= 224.37 KN

V/v_c= 0.262 < 0.6

V/V_c < 0.6 then $\therefore$Safe.

6. Check for Web buckling:

Assumed, b_1= 80 mm

n= 150 mm

h/b_f= 3>1.2 (Table -10)

t_f= 10.8 <40

Class b

$\lambda = 112.45$

F_{cd}133.01 N/mm^2 Table - 9(b)

F= 174.37 KN

Section Safe in web buckling.

7. Check for Web crippling at support:

b_1= 80 mm

n_2= 54.55 mm

F_w= 166.66 KN

Section is safe in web crippling.

8. Check for Serviceability - Deflection

$\delta_{(permissible)}$= L/300= 16.67 mm
Section is Safe in deflection.

5.2.2 Design of Column

5.2.2.1 Steps of Design of Column

1. Determine the factored loads and moments acting on the beam -column using first order elastic analysis.
2. Choose a initial section and calculate the sectional properties.
3. Classify the cross section (Plastic Semi compact or compact).
4. Calculate Bending strength of the cross section about the major and minor axis.
5. a. Determine the shear resistance of the cross section .When the design shear force exceeds 0.6 V_d.
   b. Check whether Buckling is taken into account or not.
6. Calculate the reduced Plastic flexural strength , if the section is plastic or compact.
7. Check the interaction equation for cross - section resistance for Biaxial bending.
8. Calculate the design compressive strength P_{dz} and P_{dy}.
9. Calculate the Moment amplification factors.
10. Check with the interaction equation for Buckling strength.

5.2.2.2 Calculation of Column Design

Factored load: 215 KN

Factored moments:

1.

	M_z	M_y
Bottom	5.23 KN.m	9.87 KN.m
Top	19.3 KN.m	13.5 KN.m

fy= 250 N/mm^2

E= 200000 N/mm^3

length of column= 3.6 m

Effective length of the column as 2.34 m along both the axe

2.

SECTION	
PROPERTIES	ISHB225
AREA	=5966 mm^2
DEPTH	=225 mm
h_f	=206.8
WIDHT	=225 mm
t_f	=9.1 mm
t_w	=8.6 mm
I_{zz}	=54788000 mm^4
I_{yy}	=13966000 mm^4
r_{xx}	=95.8 mm
r_{yy}	=48.4 mm
I_{xx}	=487000 mm^3
I_{yy}	=123000 mm^3
Z_{pz}	=542220 mm^3
Z_{py}	= 234167.5 mm^3
r	=11 mm

3. Section classification:

Flange criterion= $12.36 < 42\epsilon$

$(B/2 * t_f)$ Semi compact section

Web criterion= $21.49 < 42\epsilon$

$(D - 2 * (t_f + r))/t_w$ Semi compact section

Hence cross section is Semi compact section

4. Check for resistance of cross-section to the combined effects for yielding:

$$f_{yd} = f_y/\gamma_a = 250/1.15 = 217.3913043\ N/mm^2$$

$A_g = 5966\ mm^2$

$Z_{zz} = 487000\ mm^3$

$Z_{yy} = 123000 mm^3$

P = 215 KN

M_z = 9.87 KN.m

M_y = 19.3 KN.m

5. The interaction equation is:

$$\frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1 \tag{5.2}$$

where ,

N = Factored applied axial force (tension T or Compression P)

N_d = Design strength in tension or compression.

M_y, M_z = Factored applied moment about minor and major axis.of cross - section respectively.

M_{dy}, M_{dz} = Design strength of section in bending when acting alone about y - axis and z - axis respectively.

=0.98 < 1 safe

6. Check for resistance of cross-section to the combined effects for buckling:

   a. A)Determination of P_{dz},P_{dy} and P_d (Clause 7.1.2)

      $KL_z = 2340$ mm

      $KL_y = 2340$ mm

      $\lambda_z = KL_z/r_z$ 24.25

      $\lambda_y = KL_y/r_y$ 48.34

      $\lambda_1 = \prod(E/f_y)^{1/2}$ 88.81

      Therefore, non-dimensional slenderness ratios

      $\lambda_z = \lambda_z/\lambda_1$ 0.257

      $\lambda_y = \lambda_y/\lambda_1$ 0.544

$$\chi = \frac{1}{\phi + [(\phi) - (\lambda)]^0.5} \leq 1 \tag{5.3}$$

      Calculation ofχ

      Imperfection factors: $\alpha_z = 0.21$ $\alpha_y = 0.34$

$$\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda] \tag{5.4}$$

      ϕ- values:

      $\phi_z = 0.54$ $\phi_y = 0.71$

      χ - values:

      $\chi_z = 0.91$ $\chi_y = 0.70$

      $P_{dz} = 1242.36$ KN $P_{dy} = 959.31$ KN

      There fore $= P_d = P_{dy} = 959.31$ KN> 215 KN Safe

b. Determination of M_{dz}

M_{cr}= 575902906 N.mm

λ_{LT} =$\sqrt{Z_{ez} f_y / M_{cr}}$

λ_{LT}= 0.48

$$\chi_{LT} = \frac{1}{\phi_{LT} + [(\phi_{LT}^2) - (\lambda_{LT}^2)]^0.5} \leq 1 \tag{5.5}$$

$$\phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2 \tag{5.6}$$

ϕ_{LT}= 0.65

λ_{LT}= 0.21

α_{LT} = 0.21 for rolled section

α_{LT} =0.21 $\leq$ 1

$$f_{bd} = \chi_{LT} \times f_Y / \gamma_{mo} \tag{5.7}$$

χ_{LT}=0.93 f_{bd} =211.10 N/mm^2

$$M_{dz} = Z_e \times f_{bd} \tag{5.8}$$

M_{dz} = 114.46 >9.87 kNm

c. Determination ofM_{dy}

=53.21 >19.3 kNm

d. Determination of C_{my}

Y_z = M_2/M_1

Y_z = 0.27

C_{my} = 0.6 + 0.4ψ0 ¿ 0.4 = 0.71 >0.4

e. Determination ofC_{mz}

$Y_z = M_1/M_2$

$Y_z = 0.73$

C_{mLT} = 0.6 + 0.4ψ0 ¿ 0.4 = 0.90 >0.4

f. Determination ofC_{mLT}

$Y_z = M_2/M_1$

$Y_z = 0.270$

C_{mLT} = 0.6 + 0.4ψ0 ¿ 0.4 = 0.708 >0.4

g. Determination of k_z , k_y and k_{LT}

K_y , K_z,K_{LT} = Moment amplification factors.

$$K_y = 1 + (\lambda_y - 0.2) \times n_y \leq 1 + 0.8n_y \tag{5.9}$$

$$K_z = 1 + (\lambda_z - 0.2) \times n_z \leq 1 + 0.8n_z \tag{5.10}$$

n_y = P/P_{dy} = 0.22

n_z = P/P_{dz} = 0.17

λ_y= 0.544

λ_z= 0.275

λ_{LT}= 0.485

k_y= 1.07$\leq$ 1.17

k_z= 1.01 $\leq$ 1.13

k_{LT} = 0.97 $\geq$ 0.94

$\therefore$Safe

h. Interaction checks

$$(\frac{P}{P_{dy}}) + (\frac{K_y \times C_{my} \times M_Y}{M_{dz}}) + (\frac{K_{LT} \times M_z}{M_{dz}}) \leq 1.0 \tag{5.11}$$

0.85$\leq$ 1

$\therefore$ safe

$$(\frac{P}{P_{dz}}) + (\frac{0.6 \times K_y \times C_{my} \times M_Y}{M_{dz}}) + (\frac{K_z \times C_{mz} \times M_z}{M_{dz}}) \leq 1.0 \tag{5.12}$$

0.41$\leq$ 1

$\therefore$ safe

5.3 Results for G+3 With 5m Span

5.3.1 Design of Connections

Single Web Angle Connection

Table 5.1: Single Web Angle connection for 5 m Span

Storey	3^{rd}	2^{nd}	1^{st}
Beam section	**ISMB 250**	ISMB 300	ISMB 300
Column section	**ISHB 400**	ISHB 450	ISHB 450
Gauge distance g	**28.45**	28.45	28.45
Angle size	**70 X 70 X 6**	70 X 70 X 6	70 X 70 X 6
Edge distance	**40 mm**	40 mm	40 mm
Length	**200**	200	200
NO of Bolts	**3**	3	3
Dia of Bolts	**16**	16	16

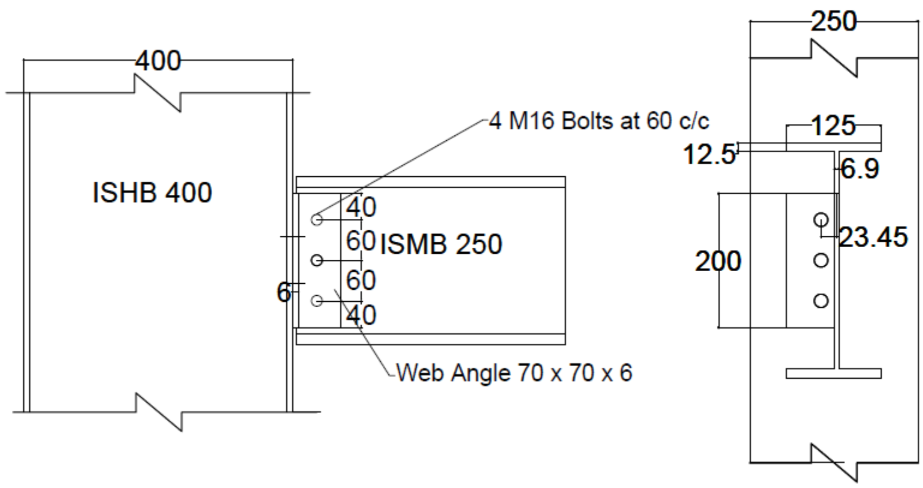


Figure 5.1: Single Web Angle Connection

Double Web Angle Connection

Table 5.2: Double Web Angle connection for Storey 5 m Span

Storey	3^{rd}	2^{nd}	1^{st}
Beam Section	**ISMB=250**	ISMB=300	ISMB=300
Column Section	**ISHB=400**	ISHB=450	ISWB=600
Gauge Distance	**106.9**	112.4	112.4
Angle Size	**90X90X6**	70X70X6	70X70X6
Edge Distance	**53.45**	56.1	56.1
Pitch	**70**	60	60
Bolt Dia	**16**	16	16
No. of Bolts	**2**	3	2
Length	**150**	220	150

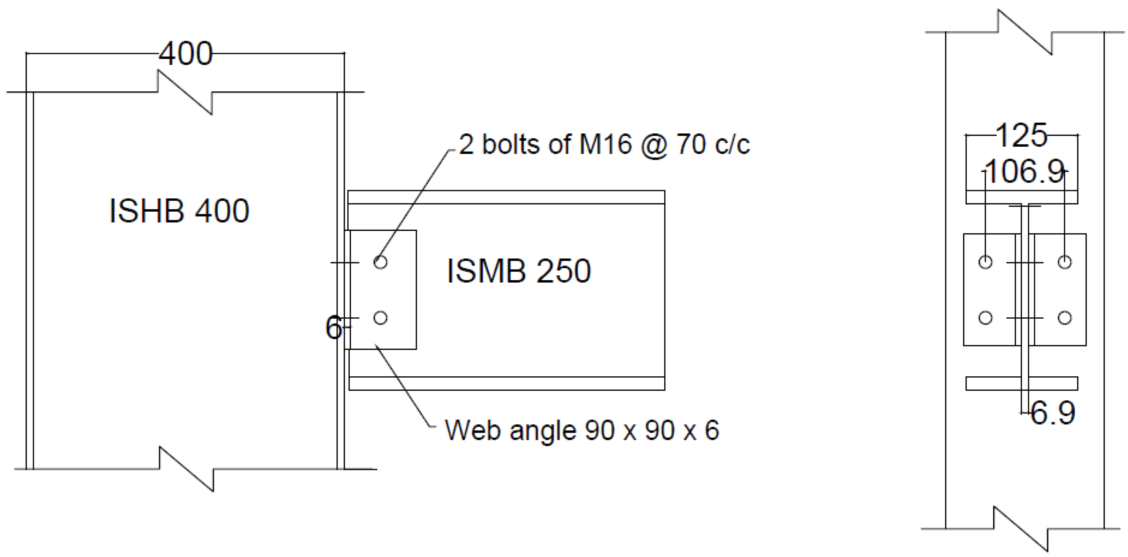


Figure 5.2: Double Web Angle Connection

Header Plate Connection

Table 5.3: Header Plate connection for Storey 5 m Span

Storey	3^{rd}	2^{nd}	1^{st}
Beam Section	**ISMB=250**	ISMB=300	ISMB=300
Column Section	**ISHB=400**	ISHB=450	ISWB=600
End Plate Size	**140X110X6**	140X140X6	140X140X6
Edge Distance	**40**	40	40
Pitch	**60**	60	60
Bolt Dia	**16**	20	20
No. of Bolts	**2**	2	2
Welding Length	**128**	128	128

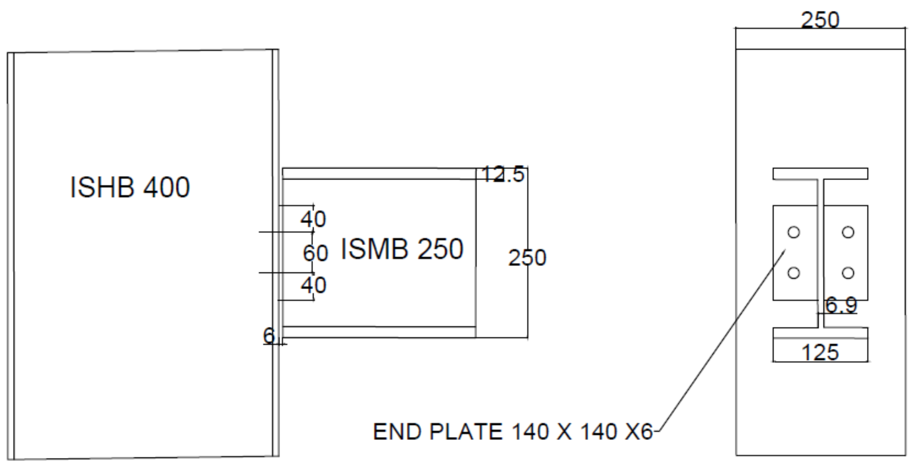


Figure 5.3: Header Plate connection

Top and Bottom Seat Angle with Web Angle

Table 5.4: Top and Bottom Seat Angle with Web Angle for Storey 5 m Span

Storey	3^{rd}	2^{nd}	1^{st}
Beam Section	**ISMB=250**	ISMB=300	ISMB=300
Column Section	**ISHB=400**	ISHB=450	ISWB=600
Angle Size	**50X50X6**	60X60X6	60X60X6
Stiffener Angle	**2 Nos. 40X40X6**	2 Nos. 50X50X6	2 Nos. 50X50X6
Edge Distance	**45**	45	45
Pitch	**70**	70	70
Bolt Dia	**16**	16	16
No. of Bolts	**2**	4	2
No. of Rows	**2**	2	2
Length	**160**	300	160

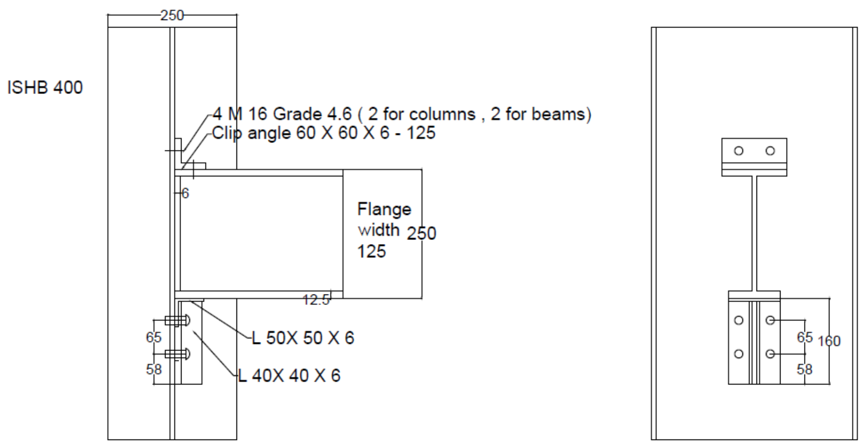


Figure 5.4: Top and Bottom Seat Angle with Web Angle

Top and Bottom Seat Angle without Web Angle

Table 5.5: Top and Bottom Seat Angle without Web Angle for Storey 5 m Span

Storey	3^{rd}	2^{nd}	1^{st}
Beam Section	**ISMB=250**	ISMB=300	ISMB=300
Column Section	**ISHB=400**	ISHB=450	ISWB=600
Seat Angle	**110X110X10**	110X110X10	110X110X10
Edge Distance	**40**	40	40
Pitch	**60**	60	60
Bolt Dia	**16**	16	16
No. of Bolts	**2**	2	2
Length	**125**	140	140

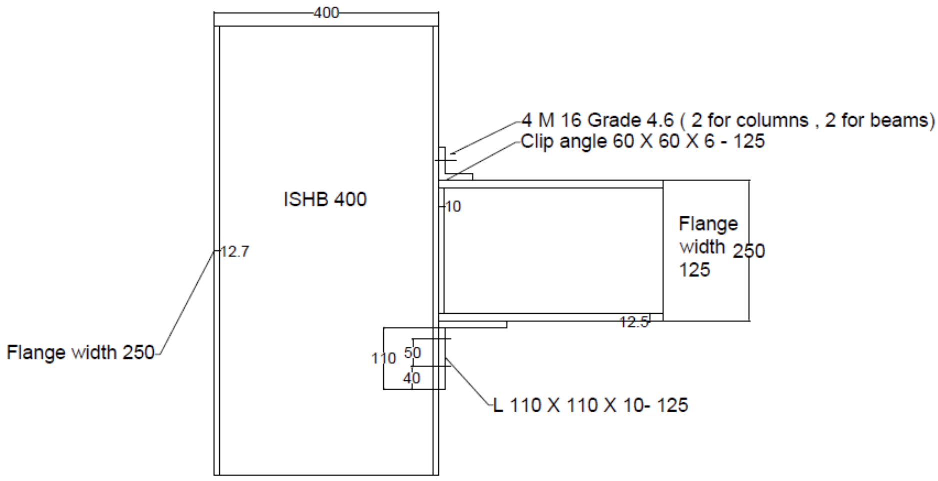


Figure 5.5: Top and Bottom Seat Angle without Web Angle

End Plate with Stiffener Connections

Table 5.6: End Plate with Stiffener Connections for Storey 5 m Span

Storey	3^{rd}	2^{nd}	1^{st}
Beam Section	**ISMB=200**	ISMB=225	ISMB=200
Column Section	**ISHB=400**	ISHB=400	ISWB=600
Plate Size	**275X150**	250X150	250X150
Thickness	**8**	12	12
Edge Distance			
Pitch			
Bolt Dia	**20**	20	22
No. of Bolts	**6**	6	6
Weld Size	**10**	14	16
Length	**150**	175	175
Web Stiffner	**30X4**	90X10	100X10
Weld Size	**6**	6	10

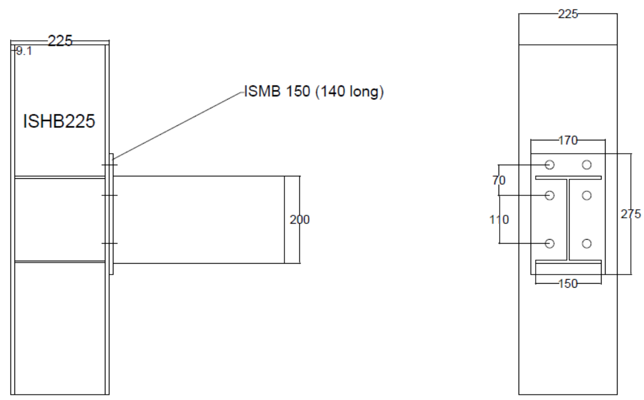


Figure 5.6: End Plate with Stiffener Connections

T-Stub Connection

Table 5.7: T-Stub Connection for Storey 5 m Span

Storey	3^{rd}	2^{nd}	1^{st}
Beam Section	**ISMB=200**	ISMB=225	ISMB=200
Column Section	**ISHB=400**	ISHB=400	ISWB=600
Web Angle	**90X90X8**	90X90X8	90X90X8
T-Stab Cut From	**ISMB=150**	ISMB=250	ISMB=250
Length	**150**	150	150
Edge Distance	**25**	40	40
Pitch	**50**	50	50
Bolt Dia	**20**	20	20
No. of Bolts	**4**	4	4
No. of Rows	**2**	2	2

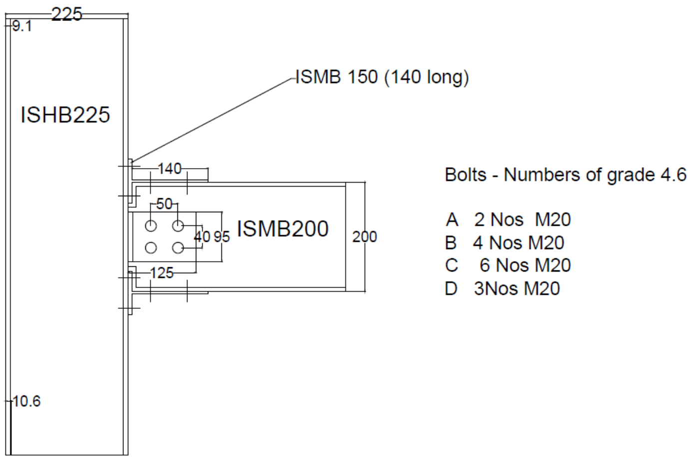


Figure 5.7: T-Stub Connection

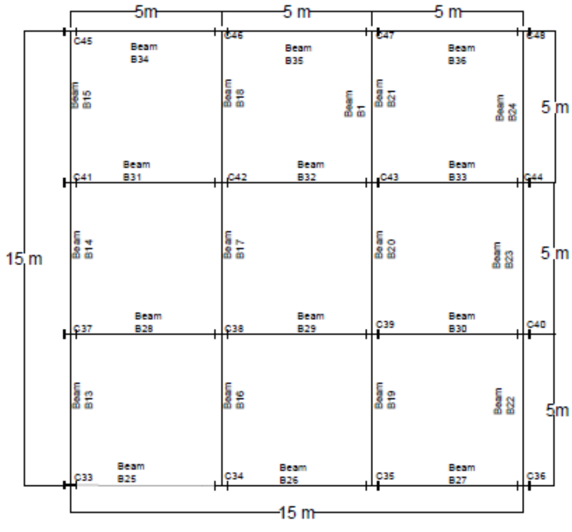
Table 5.8: Member Sizes for 5 m Span for Various Connections

K Values 3^{rd} Storey	0	9.05	22.75	61.51	83.89	493.95	876.96	10000
Beam Section	**ISMB 250**	ISMB 225	ISMB 225	ISMB 225	ISMB 225	ISMB 200	ISMB 200	ISMB 200
Column Section	**ISHB 400**	ISHB 400	ISHB 300	ISHB 350	ISHB 350	ISHB 225	ISHB 225	ISHB 225
K Values 2^{nd} Storey	0	9.05	22.75	61.51	83.89	493.95	876.96	10000
Beam Section	ISMB 300	ISMB 300	ISMB 300	ISMB 250	ISMB 250	ISMB 225	ISMB 225	ISMB 225
Column Section	ISHB 450	ISWB 550	ISHB 400	ISHB 400	ISHB 450	ISHB 450	ISHB 450	ISHB 400
K Values 1^{st} Storey	0	9.05	22.75	61.51	83.89	493.95	876.96	10000
Beam Section	ISMB 300	ISMB 250	ISMB 250	ISMB 250	ISMB 225	ISMB 200	ISMB 200	ISMB 200
Column Section	ISMC 250	ISMC 200	ISMC 300	ISMC 200	ISMC 200	ISWB 600	ISWB 600	ISWB 600
C/C between 2 ISMC	300	300	300	300	250	-	-	-
Plate Widht	300	300	300	300	250	-	-	-
Thickness	16	16	16	16	16	-	-	-

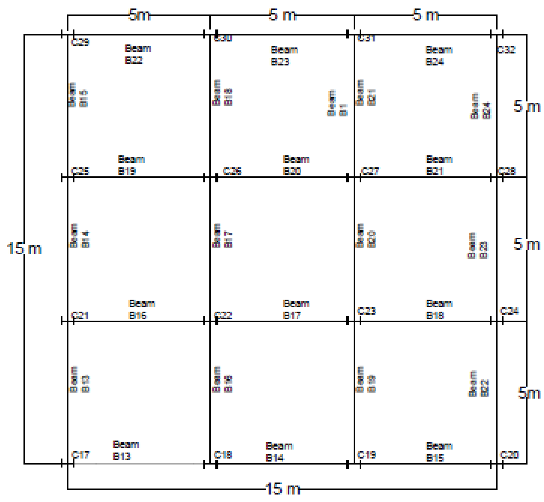
Table 5.9: Member Sizes for 5 m Span for Various Connections

K Values 3^{rd} Storey	0	4.7	8.5	16.8	27.7	13.3	44.6	46.4	10000
Beam Section	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB
	250	225	225	225	225	200	200	200	175
Column Section	ISHB	ISHB	ISHB	ISHB	ISHB	ISHB	ISHB	ISHB	ISHB
	400	4000	225	250	250	225	250	225	225
2^{nd} Storey	0	4.7	21.8	16.8	40.2	18.7	79.1	127	10000
Beam Section	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB
	300	300	300	250	300	250	250	225	200
Column Section	ISHB	ISHB	ISHB	ISHB	ISHB	ISHB	ISHB	ISHB	ISHB
	450	450	450	450	450	450	450	450	400
1^{st} Storey	0	4.7	8.5	16.8	40.2	44.6	58.1	127	10000
Beam Section	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB
	300	250	250	250	250	250	250	200	200
Column Section	ISMC	ISMC	ISWB	ISMC	ISMC	ISMC	ISWB	ISWB	ISWB
	200	175	600	175	175	175	600	600	600
C/C between	450	450	-	450	450	450	-		-
Plate Width	450	450	-	450	450	450	-		-
Thickness	16	16	-	16	16	-	-		-

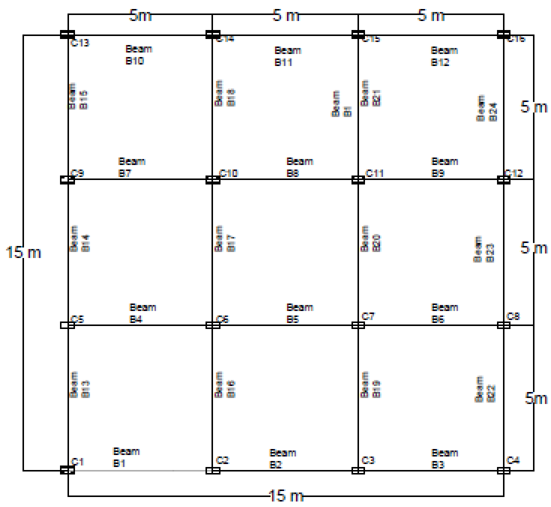
As per the suggestions of the external examiner the sections have been revised.

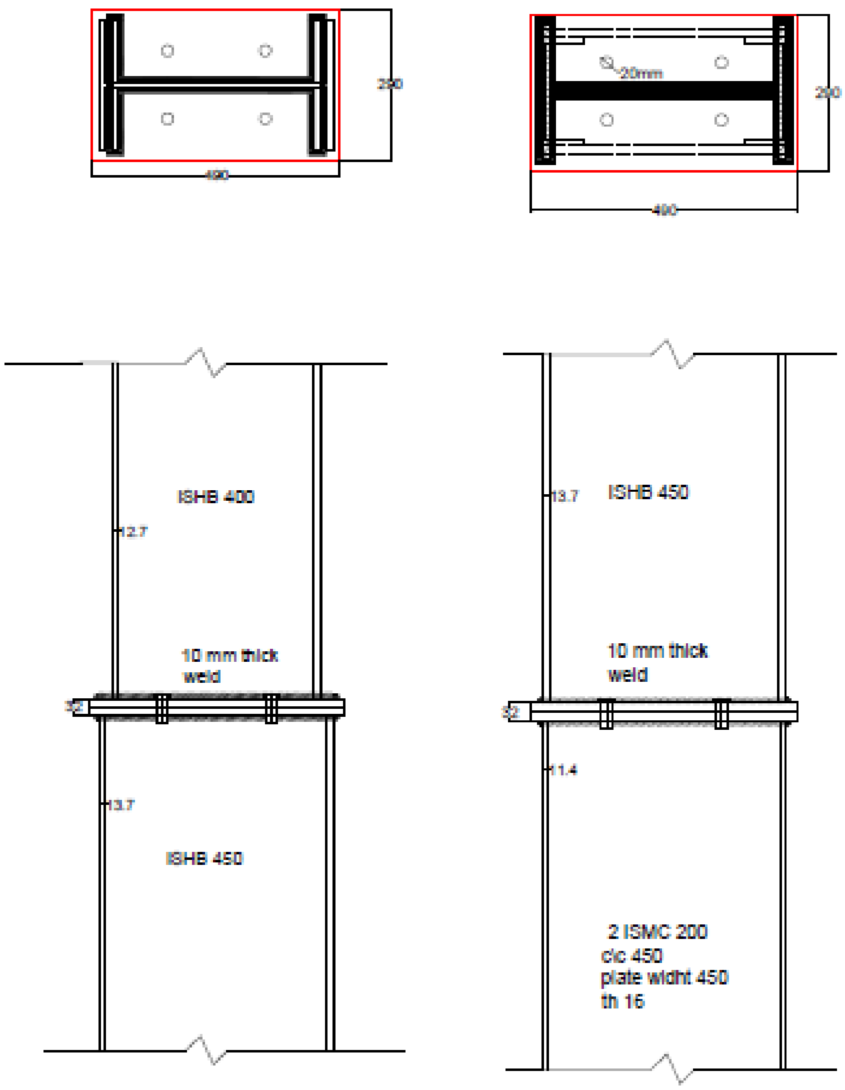


(a) Column Orientation for3^{rd}Storey



(b) Column Orientation for2^{nd}Storey





(a) Column Connection for 2^{nd} storey column (b) Column Connection for 1^{st} storey column

Figure 5.9: Column Connection Details

Here the design of connections is done for 5 m span G+3 building for all the 3 stories. Value of curvature is found out at 0.01 radians that is 620.08 kN/radians 1 radians =57.29 degree so 620.08 kN/radians = 22.75 kN/degree

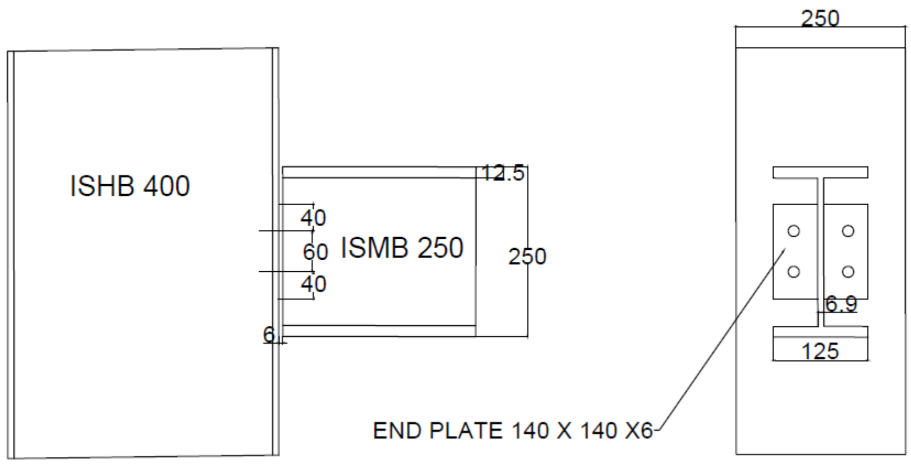


Figure 5.10: Header Plate connection

C_1 =3.87
C_2 =2.71×10^5
C_3 =6.06×10^{11}
K =$t_p^{-1.6} g^{1.6} d_b^{-2.30} t_w^{-0.5}$

where

t_p	=thickness of end plate	= 6mm
g	= gauge distance	= 60 mm
d_b	=depth of beam	= 140mm
t_w	=thickness of the web of the beam	= 9.1 mm

Table 5.10: Moment Rotation Values for Header Plate Connection

Moment	Rotation	Curvature k(M/θ)
0	0	0
5	0.00323	1544.28
10	0.01194	836.93
15	0.05059	296.49
20	0.18159	110.13
25	0.52434	47.67
30	1.27417	23.54
35	2.72134	12.86
40	5.27003	7.59

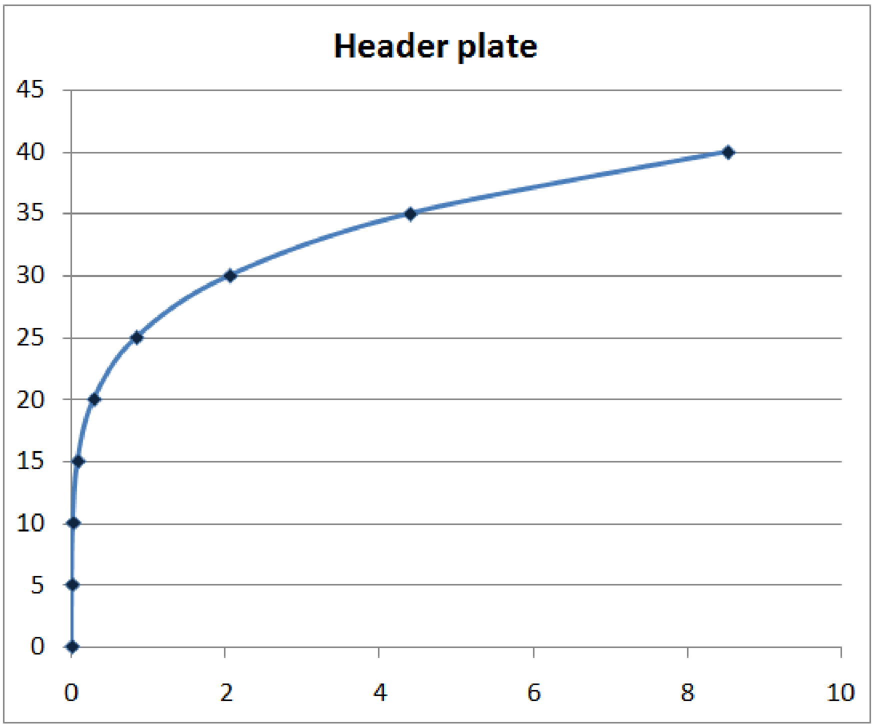


Figure 5.11: Moment Rotation Curve for Header plate

Table 5.11: Values of k for Connections for 3^{rd} Storey 5 m Span

		3^{rd} Storey	
Sr No	Connection	kN/radian	kN/degree
1	Single web angle	270.18	4.72
2	Double web angle	485.58	8.48
3	Header plate	812.71	14.19
4	Top and bottom without web angle	1586.88	27.70
5	Top and bottom with web angle	764.21	13.34
6	End plate connection	2554.73	44.59
7	T stub connection	2657.61	46.39

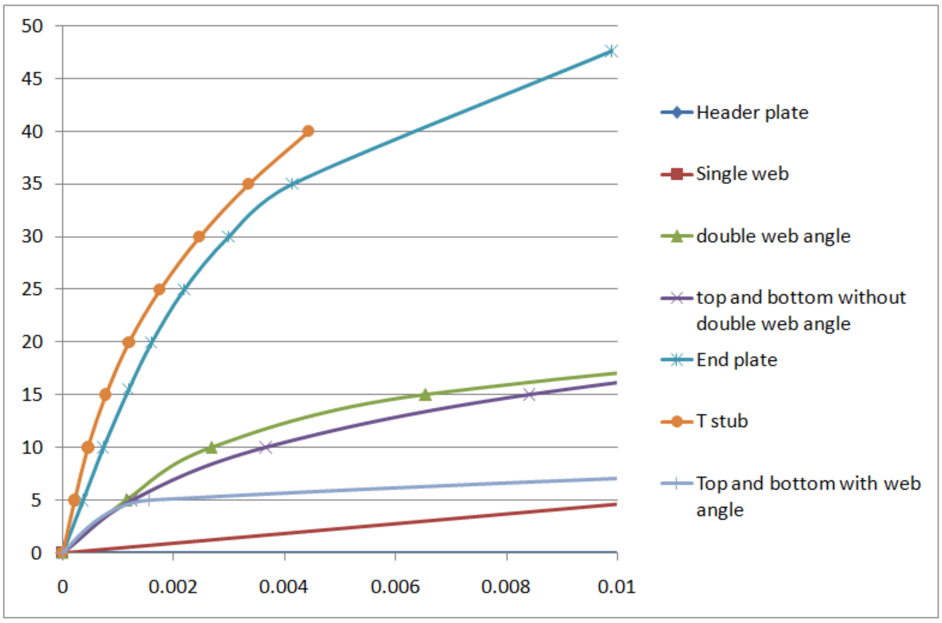


Figure 5.12: Moment Rotation Curve for Different connections of 3^{rd} Storey 5 m Span

Table 5.12: Values of k for Connections for 2^{nd} Storey 5 m Span

		2^{nd} Storey	
Sr No	Connection	kN/radian	kN/degree
1	Single web angle	270.18	4.72
2	Double web angle	1250.39	21.83
3	Header plate	960.23	16.76
4	Top and bottom without web angle	2302.46	40.19
5	Top and bottom with web angle	1070.51	18.69
6	End plate connection	4529.89	79.07
7	T stub connection	7273.75	126.96

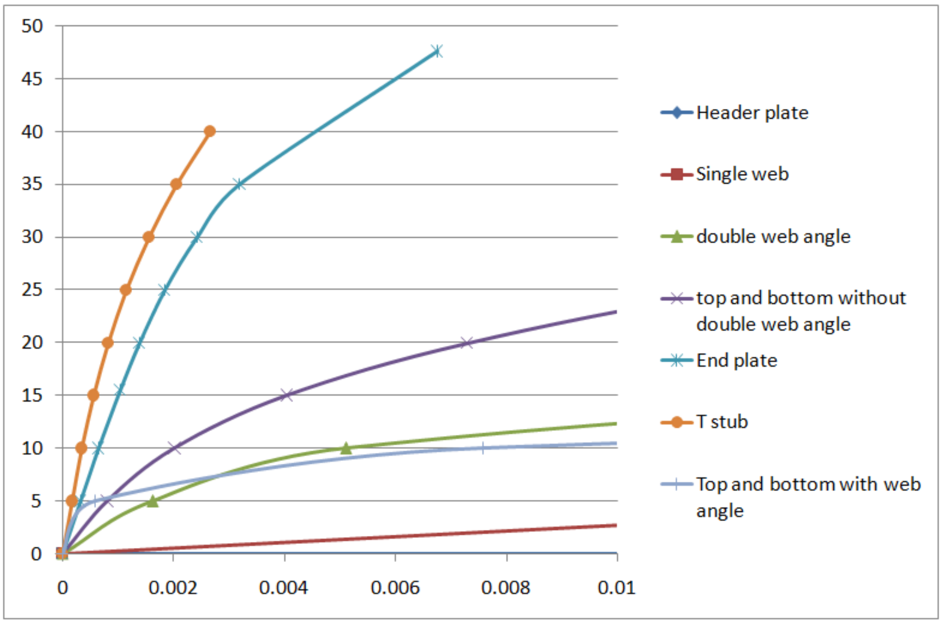


Figure 5.13: Moment Rotation Curve for Different connections of 2^{nd} Storey 5 m Span

Table 5.13: Values of k for Connections for 1st Storey 5 m Span

		1st Storey	
Sr No	Connection	kN/radian	kN/degree
1	Single web angle	270.18	4.72
2	Double web angle	485.58	8.48
3	Header plate	960.23	16.76
4	Top and bottom without web angle	2302.46	40.19
5	Top and bottom with web angle	2554.73	44.59
6	End plate connection	3328.43	58.10
7	T stub connection	7273.75	126.96

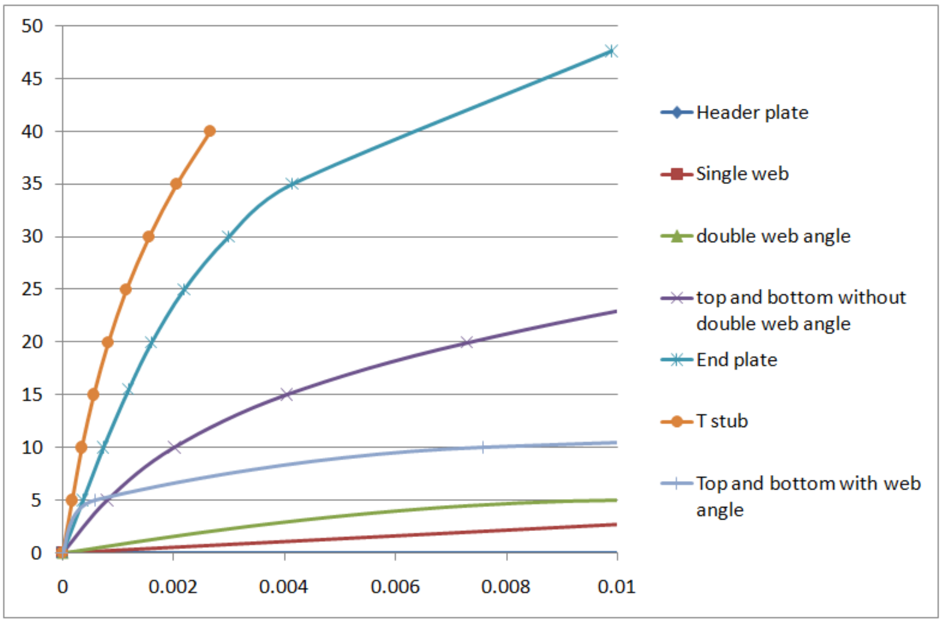
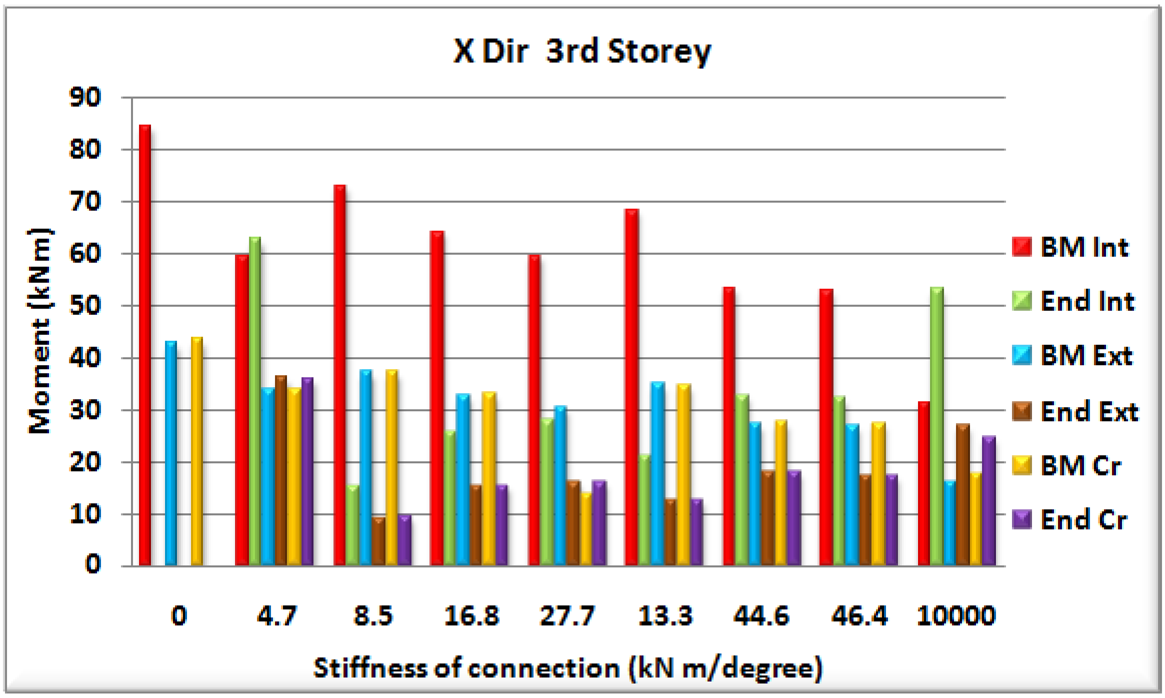
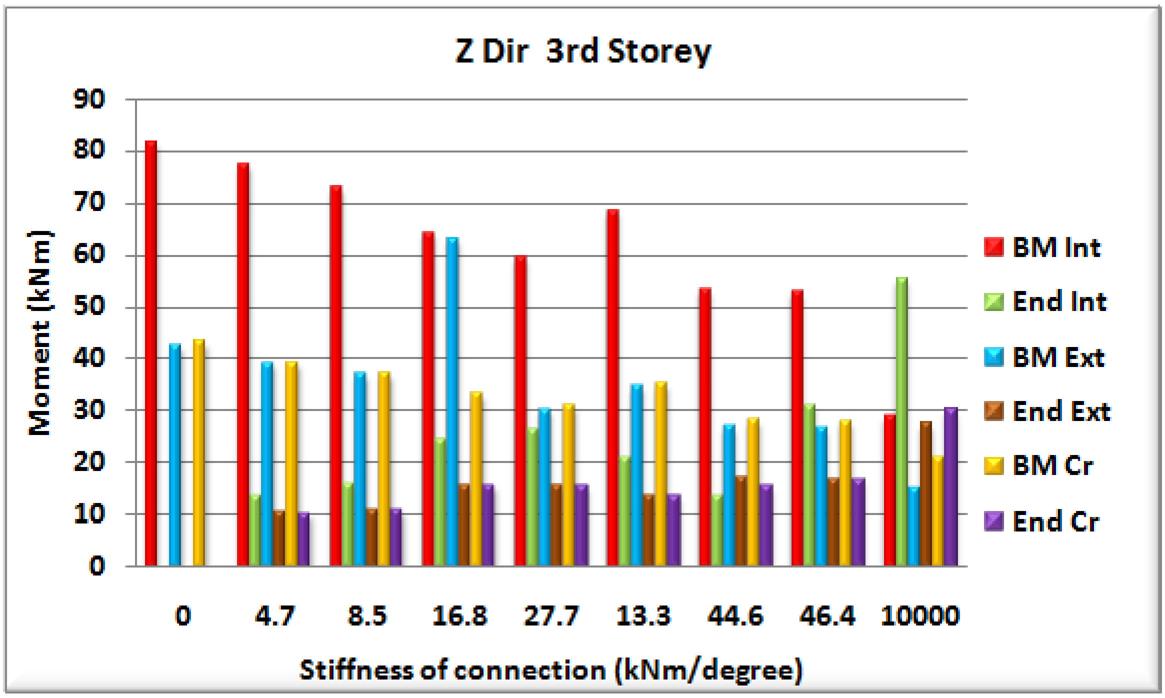


Figure 5.14: Moment Rotation Curve for Different connections of 1st Storey 5 m Span

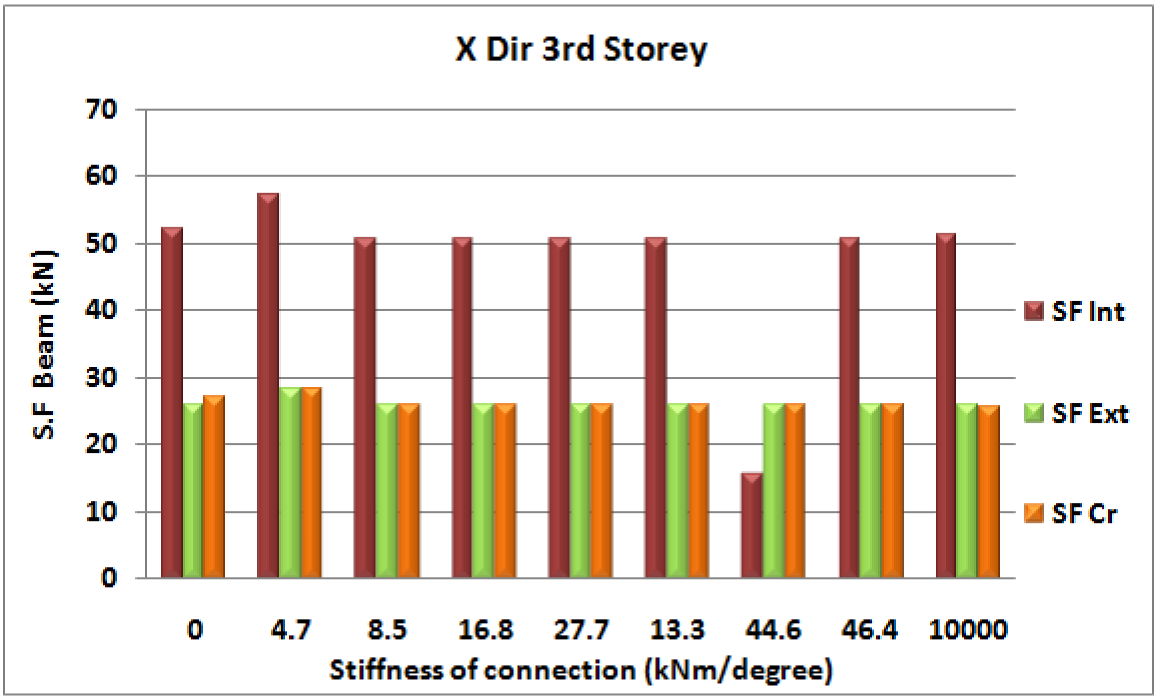


(a) Reduction in Moment of X Dir Beams

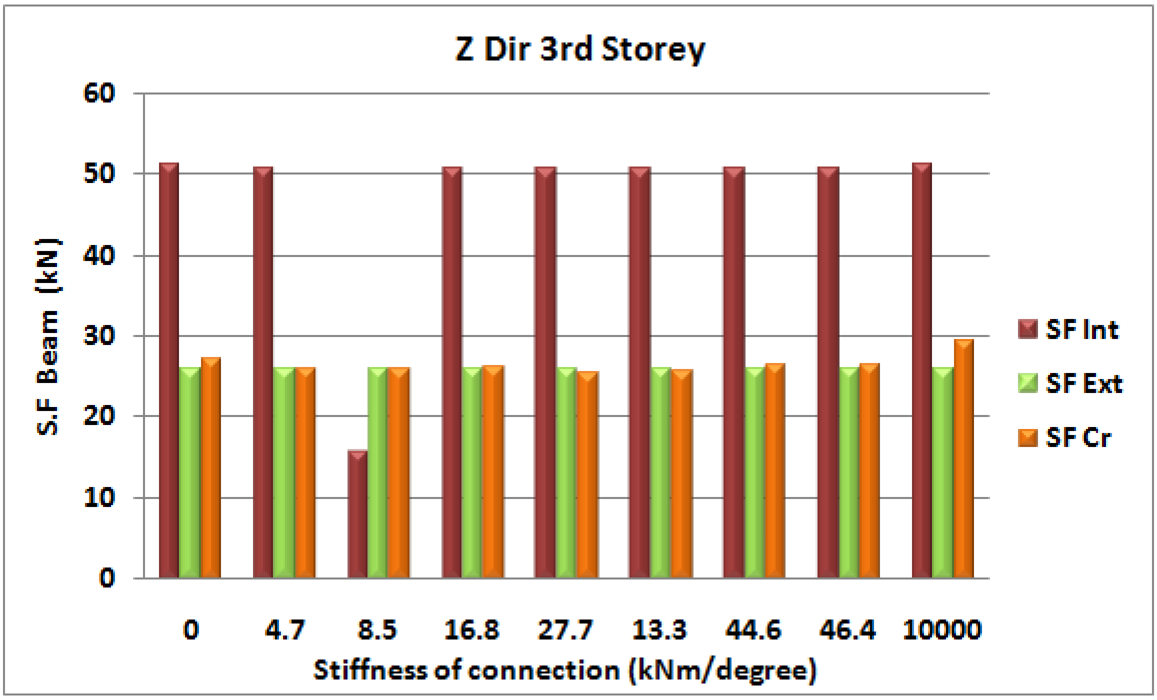


(b) Reduction in Moment of Z Dir Beams

Figure 5.15: Reduction in Moment of Beams Due to Connections of 3^{rd} Storey 5 m Span

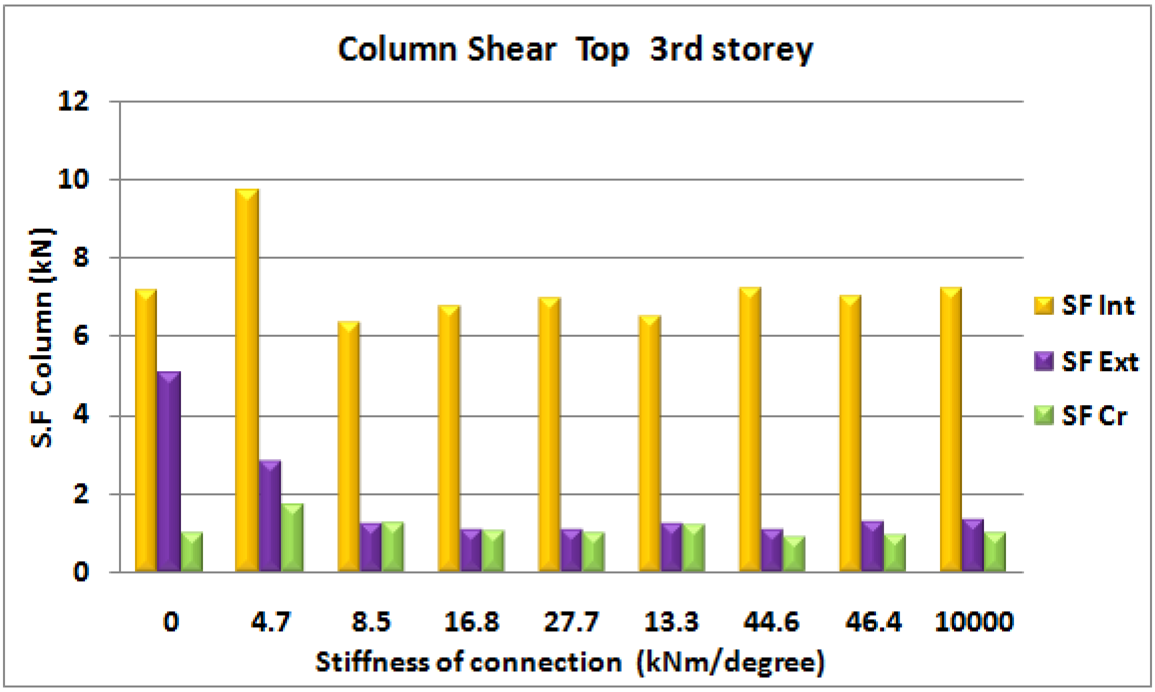


(a) Effect on Shear Force of X Dir Beams

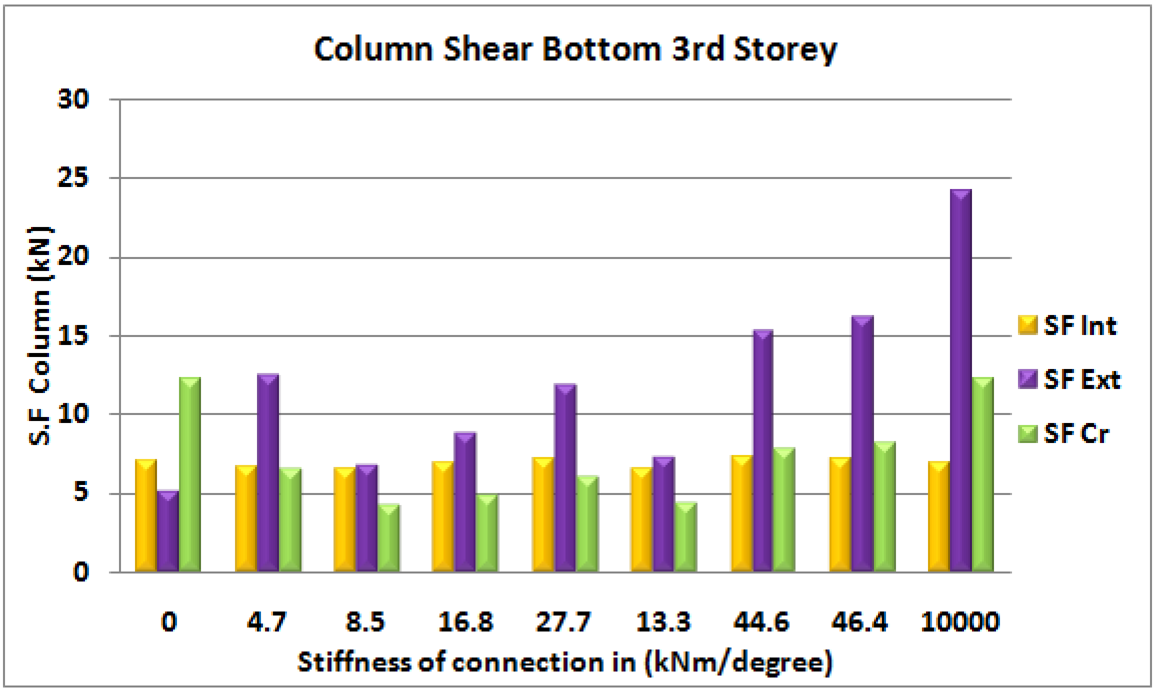


(b) Effect on Shear Force of Z Dir Beams

Figure 5.16: Effect og Shear Force on Beams Due to Connections on 3^{rd} Storey 5 m Span



(a) Effect of Shear Force on Top of Column



(b) Effect of Shear Force on Bottom of Column

Figure 5.17: Effect of Shear Force on Column Due to Connections of 3^{rd} Storey 5 m Span

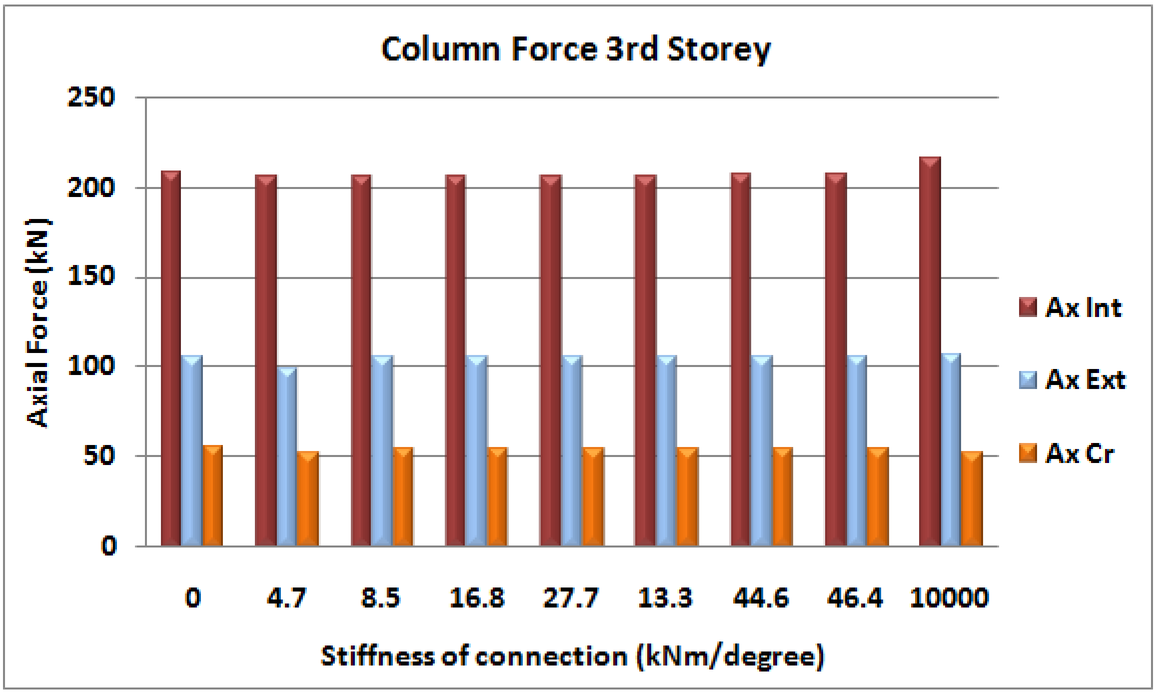
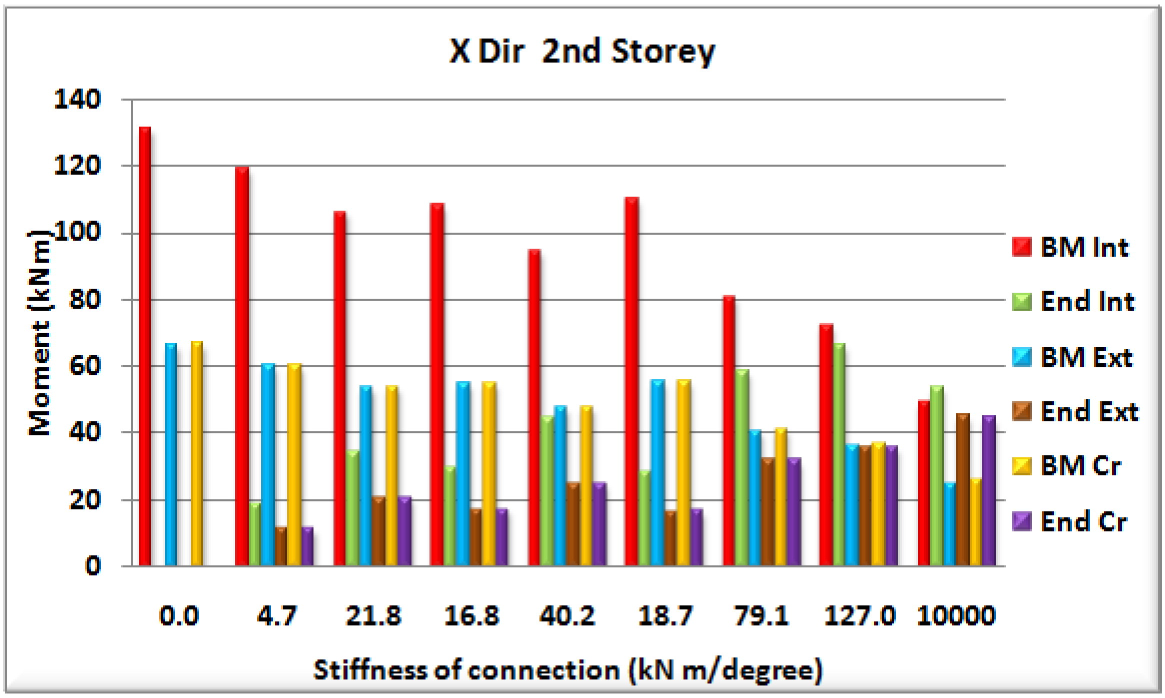
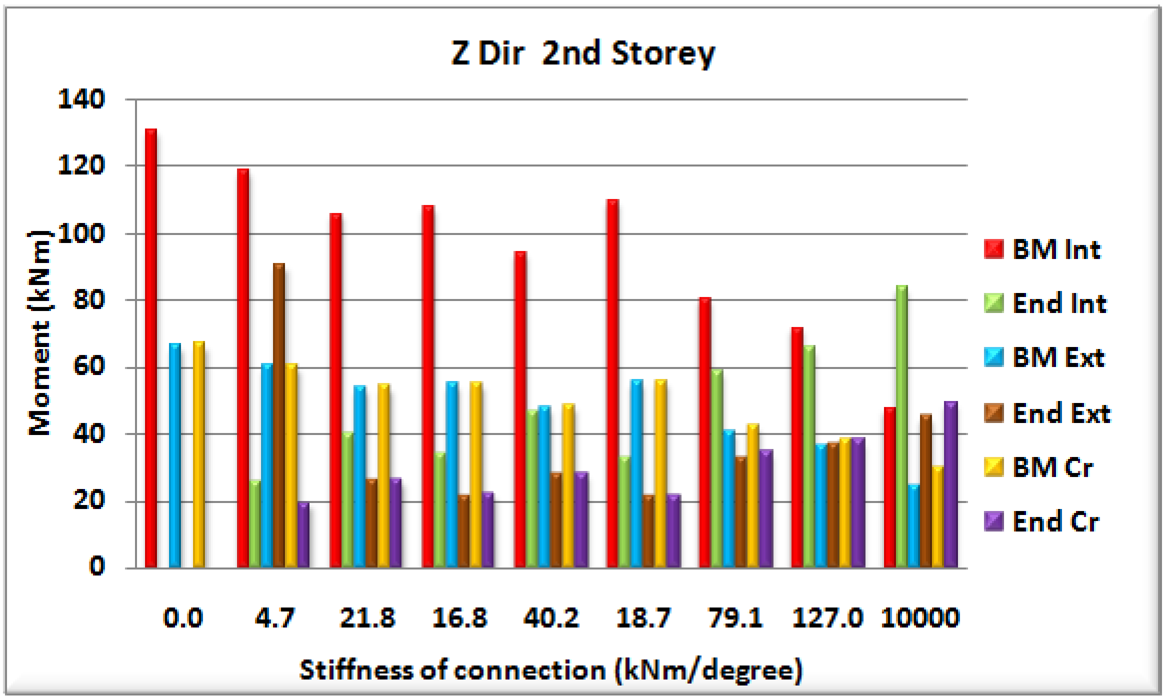


Figure 5.18: fig:Effect on Axial Force of Column Due to Connections of 3^{rd} Storey 5 m Span

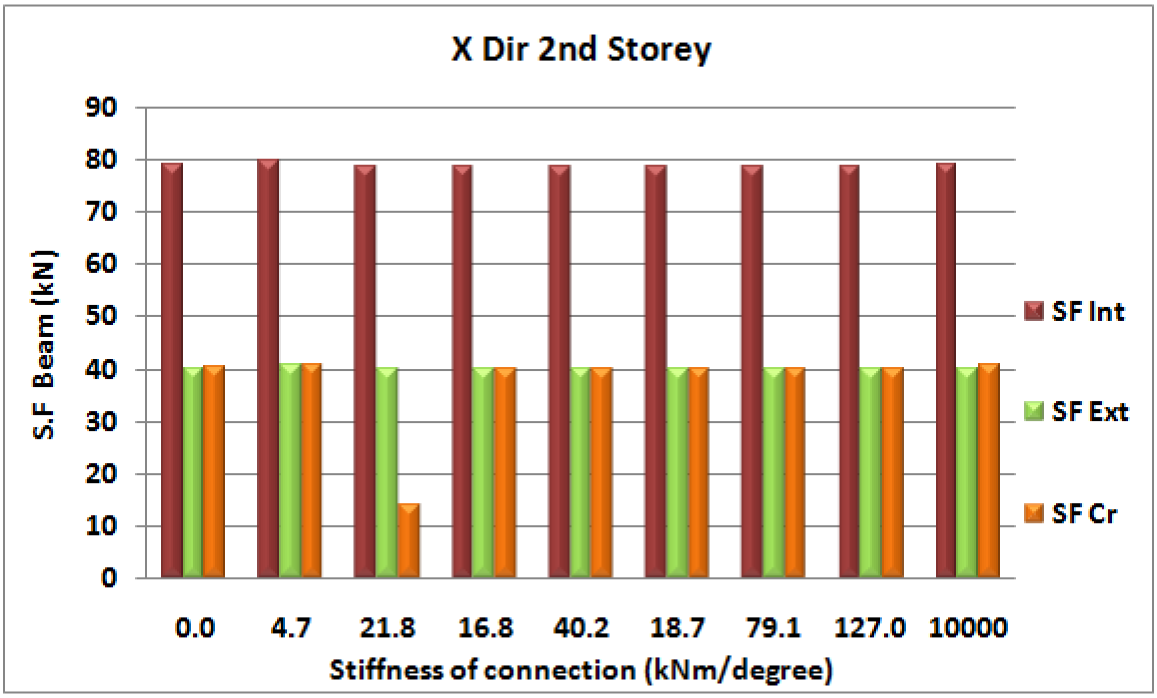


(a) Reduction in Moment of X Dir Beams

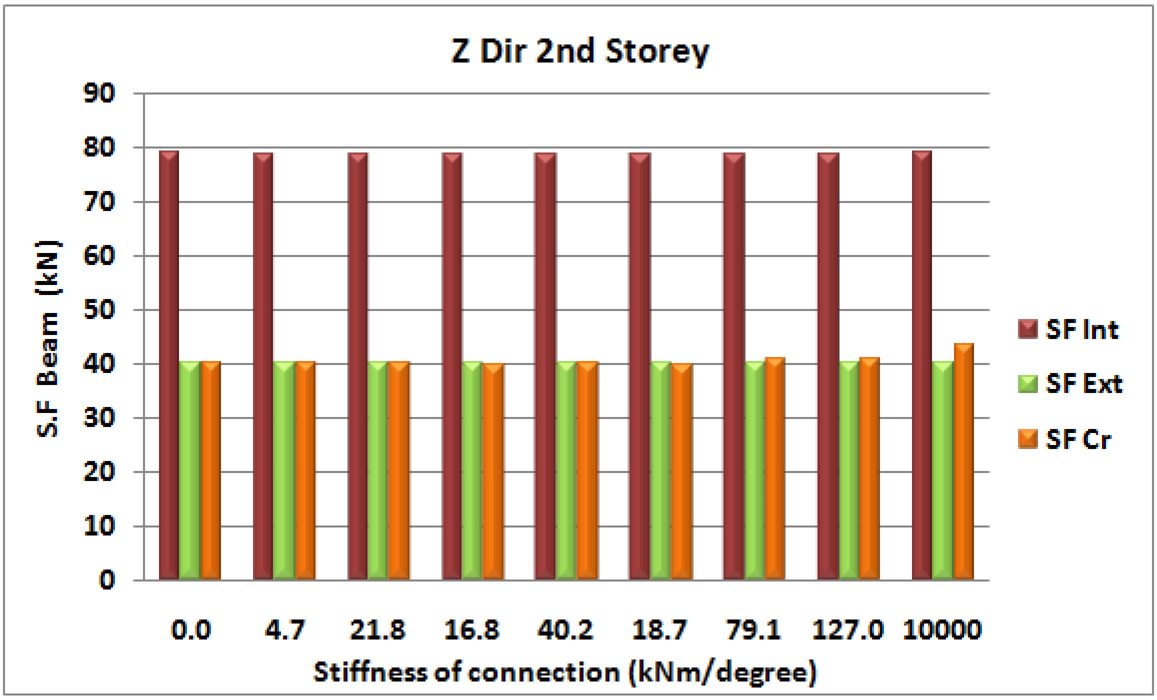


(b) Reduction in Moment of Z Dir Beams

Figure 5.19: Reduction in Moment of Beams Due to Connections of 3^{rd} Storey 5 m Span

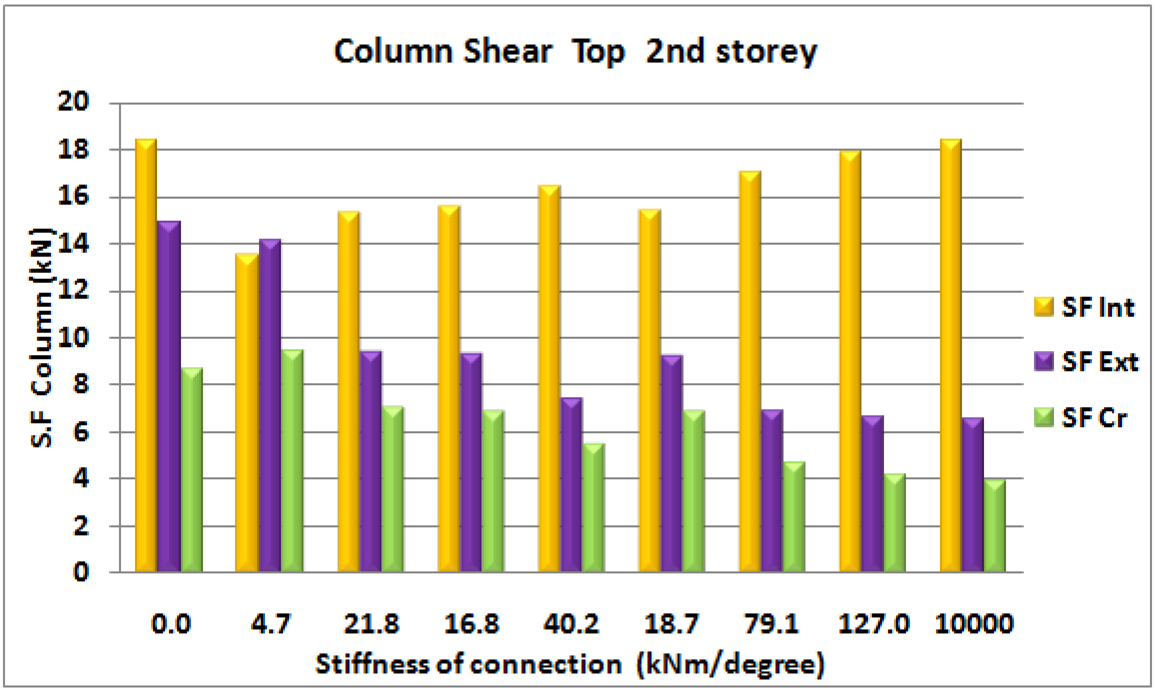


(a) Effect on Shear Force of X Dir Beams

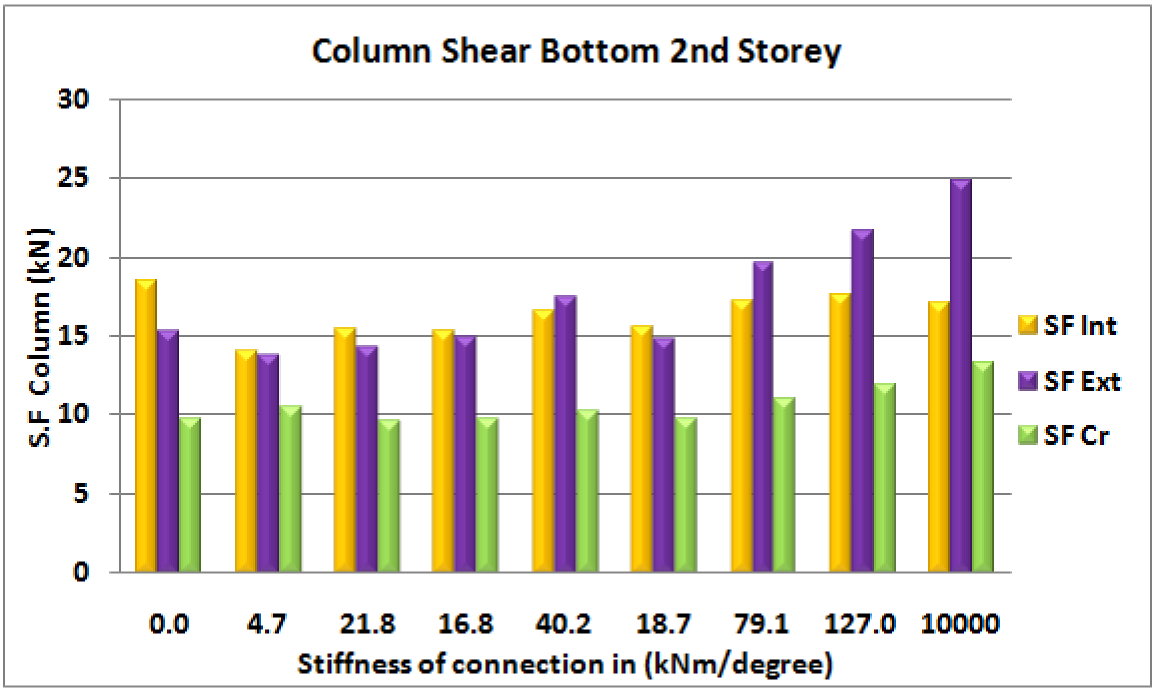


(b) Effect on Shear Force of Z Dir Beams

Figure 5.20: Effect og Shear Force on Beams Due to Connections on 2^{nd} Storey 5 m Span



(a) Effect of Shear Force on Top of Column



(b) Effect of Shear Force on Bottom of Column

Figure 5.21: Effect of Shear Force on Column Due to Connections of 2^{nd} Storey 5 m Span

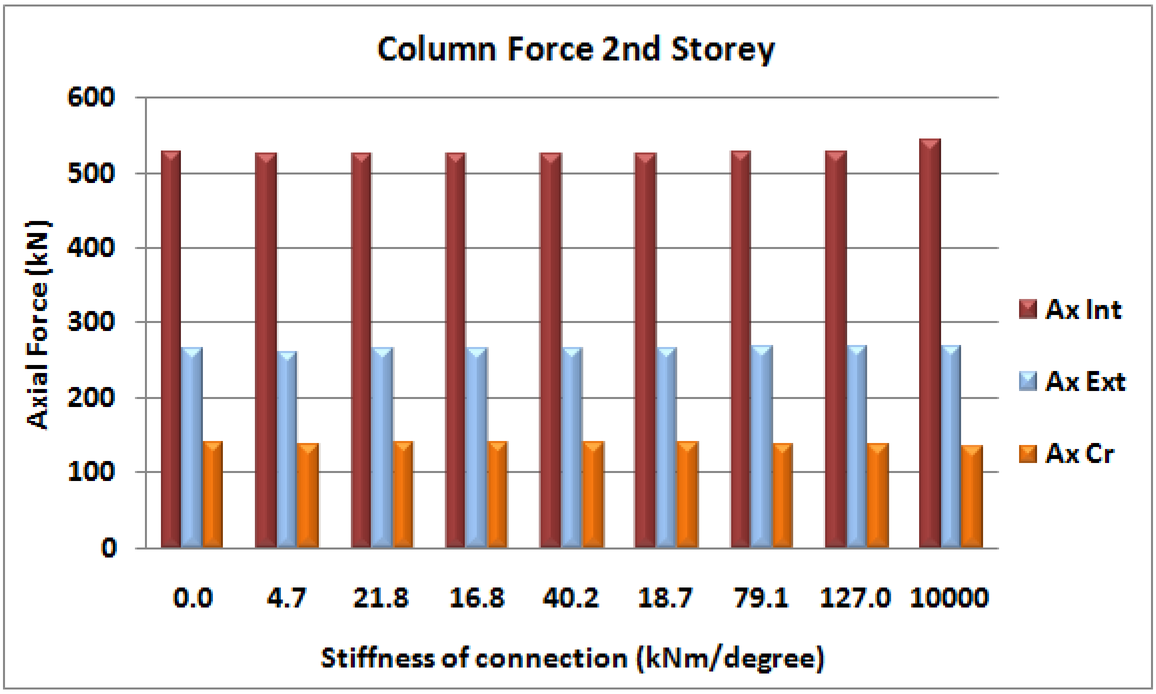
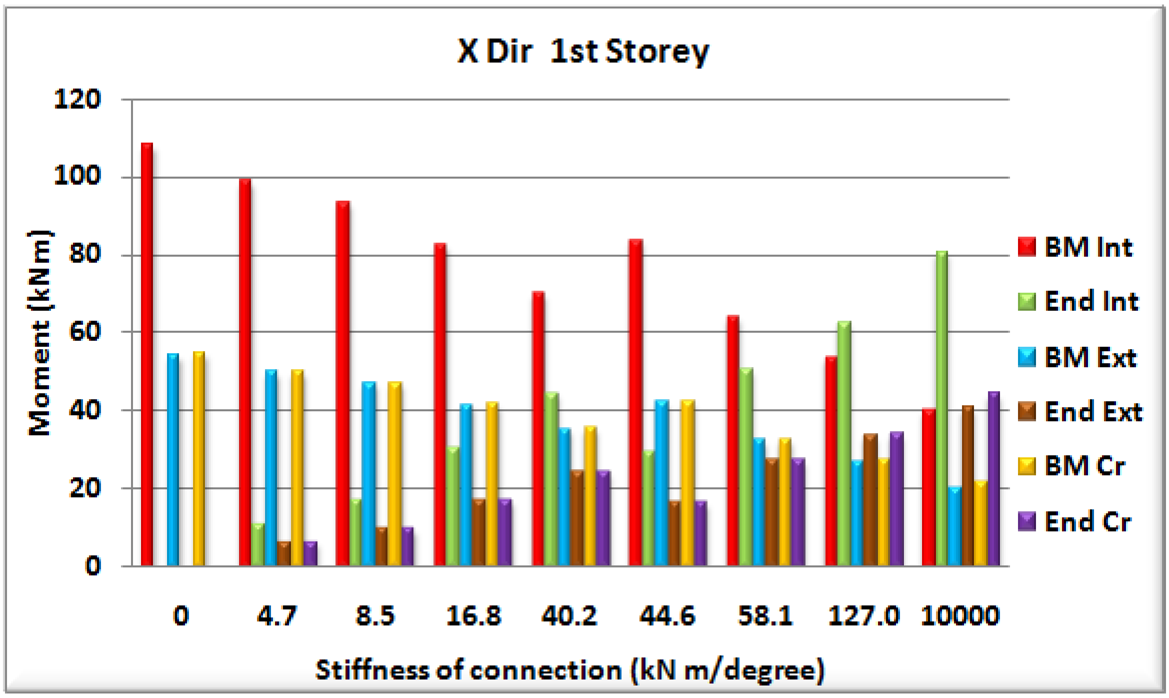
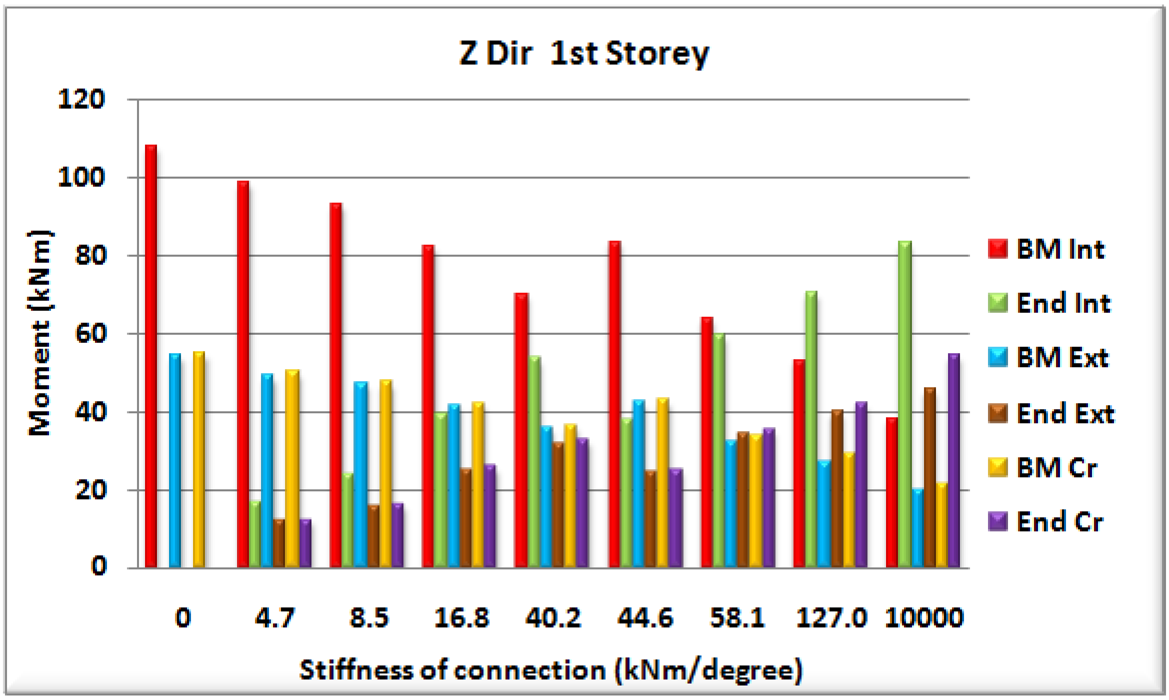


Figure 5.22: fig:Effect on Axial Force of Column Due to Connections of 2^{nd} Storey 5 m Span

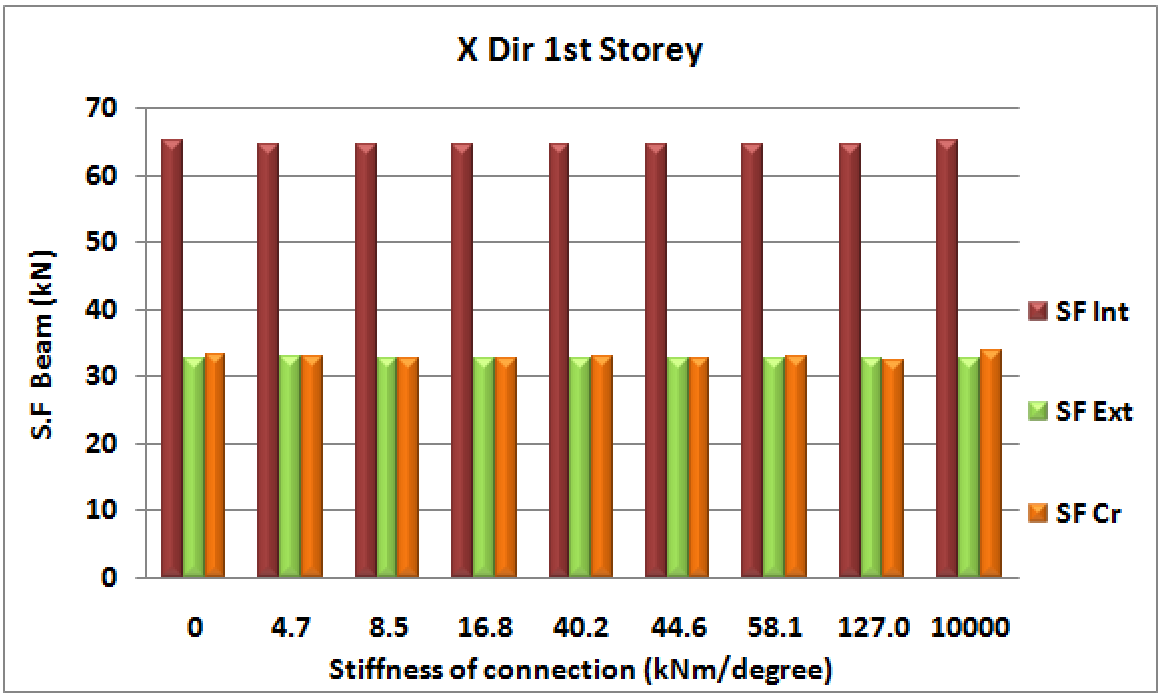


(a) Reduction in Moment of X Dir Beams

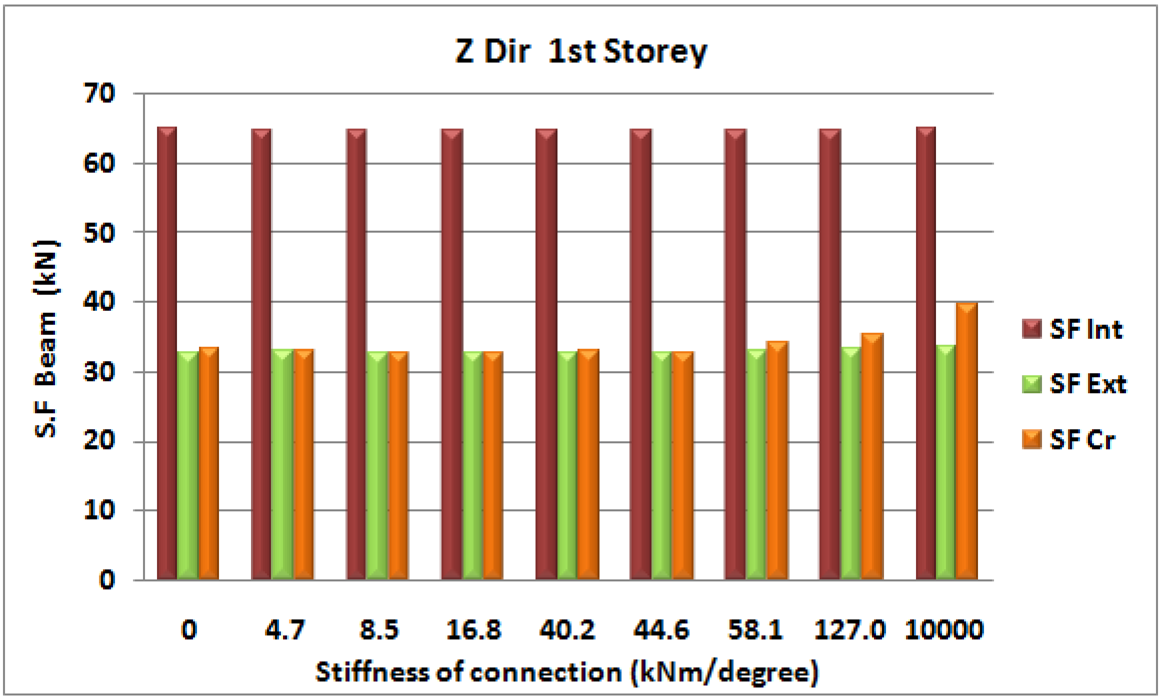


(b) Reduction in Moment of Z Dir Beams

Figure 5.23: Reduction in Moment of Beams Due to Connections of 1^{st} Storey 5 m Span

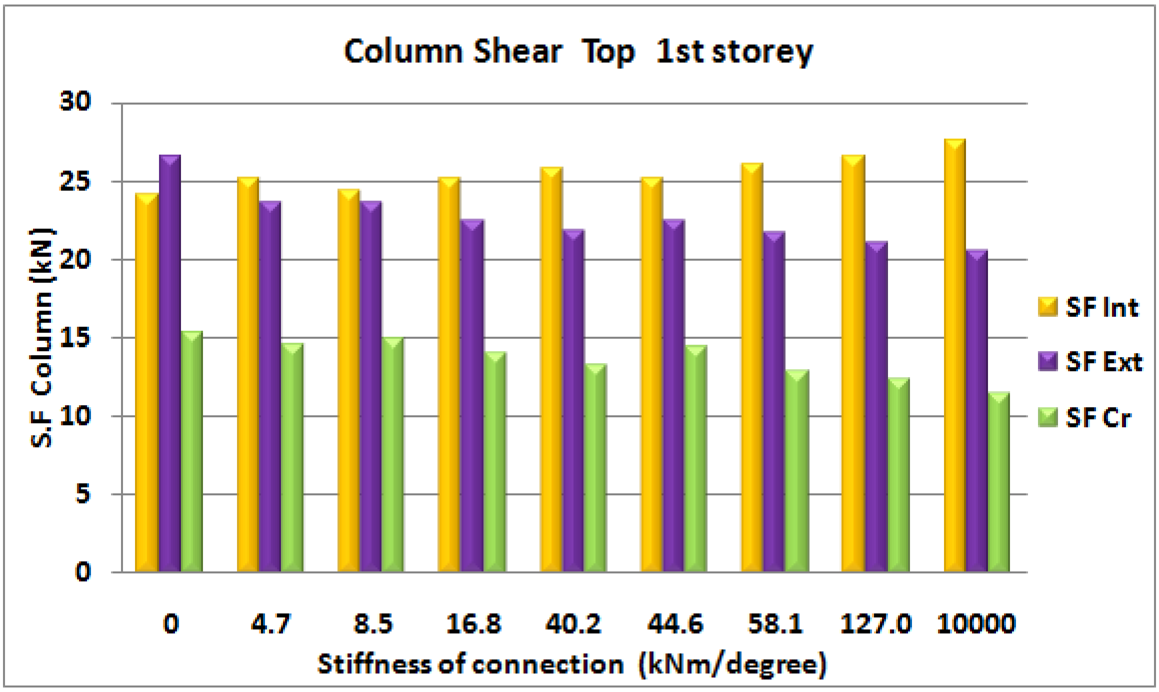


(a) Effect on Shear Force of X Dir Beams

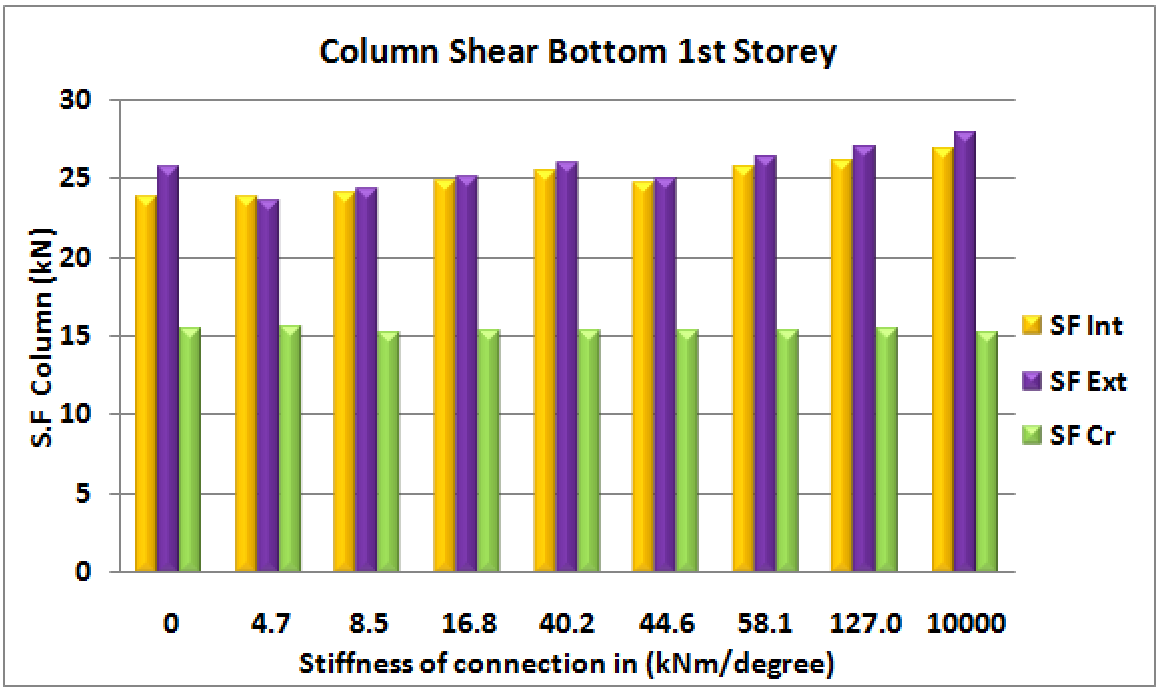


(b) Effect on Shear Force of Z Dir Beams

Figure 5.24: Effect of Shear Force on Beams Due to Connections on 1^{st} Storey 5 m Span



(a) Effect of Shear Force on Top of Column



(b) Effect of Shear Force on Bottom of Column

Figure 5.25: Effect of Shear Force on Column Due to Connections of 1^{st} Storey 5 m Span

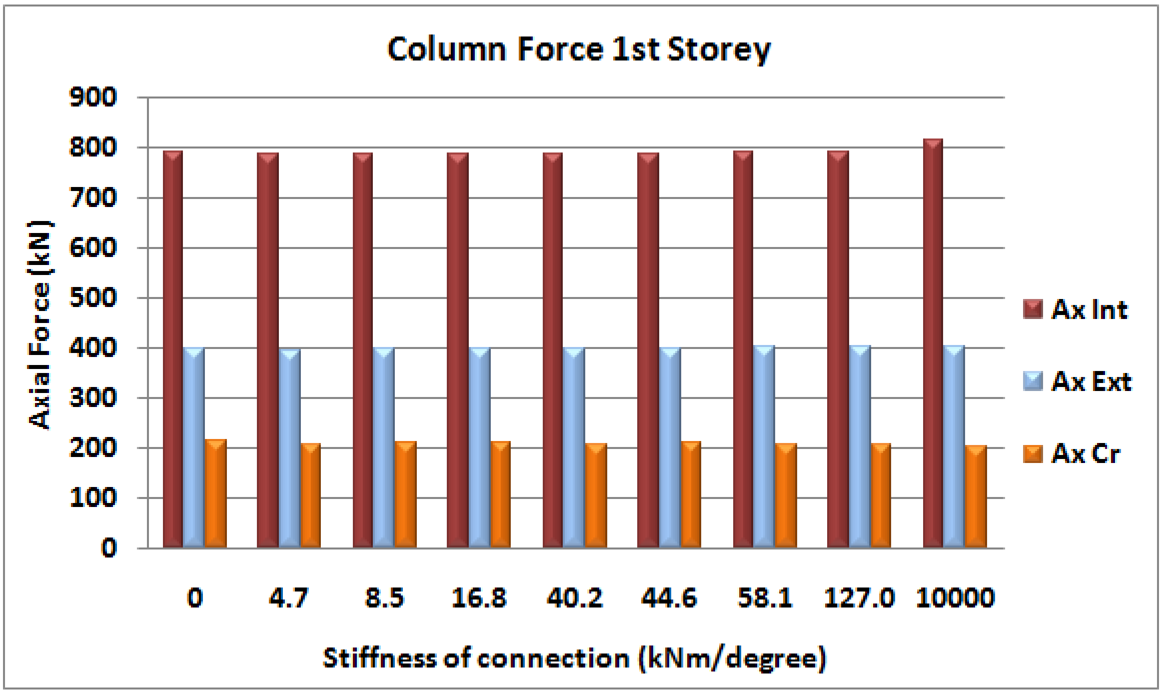
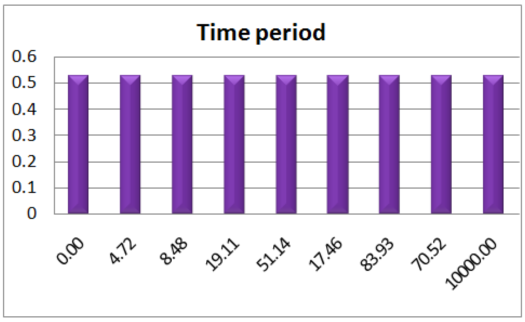
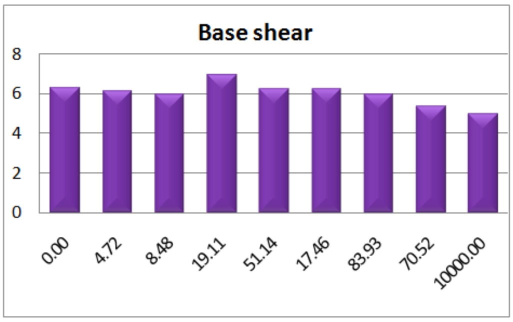


Figure 5.26: fig:Effect on Axial Force of Column Due to Connections of 1^{st} Storey 5 m Span

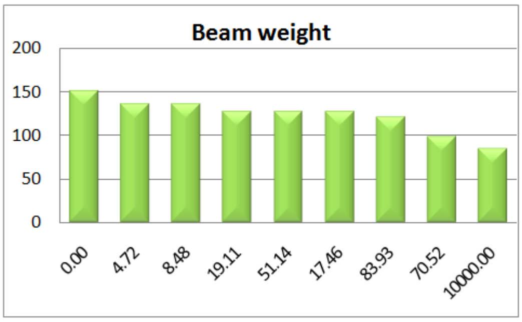


(a) Effect on Time Period

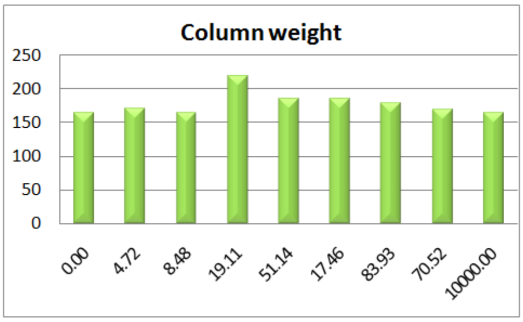


(b) Effect on Base Shear 5m Span

Figure 5.27: Effect on Time Period and Base Shear Due to Connections on 3^{rd} storey 5 m Span

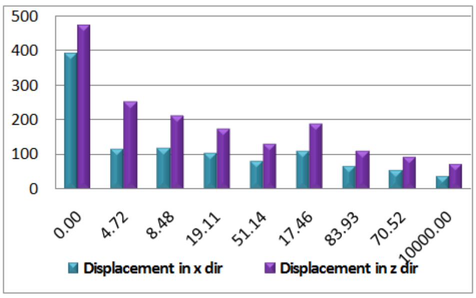


(a) Reduction in Beam Weight 5m Span

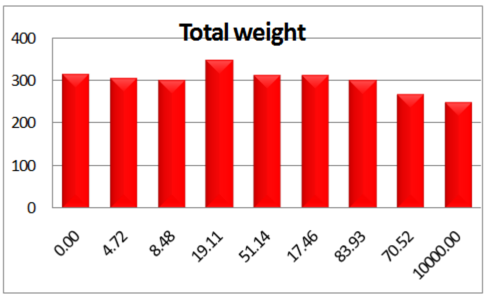


(b) Reduction in Column Weight 5m Span

Figure 5.28: Reduction in Beam Weight and Column Weight Due to Connections on 3^{rd} storey 5 m Span



(a) fig:Effect on Displacement5m Span



(b) Reduction in Total Weight

Figure 5.29: Effect on Displacement and Total Weight Due to connections of 3^{rd} Storey 5 m Span

5.4 Results and Discussions

These all are the plots for 3rd storey similarly the same plots are shown for different stories also.

A. Fig5.15(a) shows the plot of connection stiffness Vs Bending moment and End moment for beams in X direction for interior beams exterior beams and corner beams.

B. Fig5.15(b) shows the plot of connection stiffness Vs Bending moment and End moment for beams in Z direction for interior beams exterior beams and corner beams.

C. Fig5.16(a) and Fig5.16(b) shows the plot of connection stiffness Vs Shear force of beams for beams in X direction and Z direction for interior beams exterior beams and corner beams.

- There is no effect of connection on the Shear force of the Beam.

D. Fig5.17(a) and Fig5.17(b) shows the plot of connection stiffness Vs Shear force on Top of columns in X direction and Z direction for interior columns exterior columns and corner columns.

E. Fig5.3.1 shows the plot of connection stiffness Vs Axial force of columns in Z direction for interior columns exterior columns and corner columns.

- There is no effect of connection on the Axial force of the Column.

F. Fig5.27(a) and Fig5.27(b) shows the plot of connection stiffness Vs time period and connection stiffness Vs base shear.

- There is no effect on the time period of the structure.

G. Fig5.28(a) and Fig5.28(b) shows the plot of connection stiffness vs Beam weight and connection stiffness Vs Column weight.

H. Fig5.29(a) and Fig5.29(b) shows the plot of connection stiffness Vs Displacement in X direction and Z direction and connection stiffness vs total weight.

I. These all are the plots for 3rd storey similarly the same plots are shown for different stories also.
The same results are shown in Appendix C for different span length 7m and 9m.

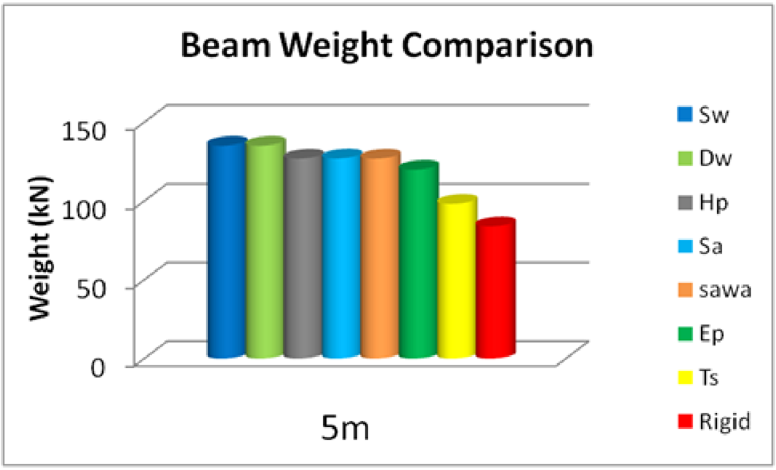
J. Table5.14 shows the values of connection stiffness for all the three stories for spans 7m and 9m Span.

Table 5.14: Values of k for Connections for Different Span

	Value of k kNm/degree						
Connection type	**Single web Angle** (Sw)	**Double web Angle** (Dw)	**Header Plate** (Hp)	**Top and Bottom Seat with Web angle** (Sa)	**Top and Bottom Seat without Web angle** (SawW)	**End plate With Web Stiffener** (Ep)	**T stub** (Ts)
				3^{rd} **Storey**			
5m	4.7	8.5	16.8	27.7	13.3	44.6	46.4
7m	4.7	21.8	16.1	77.4	28.4	190.2	371.2
9m	8.8	42.3	65.1	173.0	54.1	441.8	1185.5
				2^{nd} **Storey**			
5m	4.7	21.8	16.8	40.2	18.7	79.1	127.0
7m	8.8	41.9	61.5	130.5	47.8	270.3	580.9
9m	21.3	106.7	215.8	211.4	64.1	718.6	1659.6
				1^{st} **Storey**			
5m	4.7	8.5	16.8	40.2	44.6	58.1	127.0
7m	4.7	41.9	59.2	97.2	35.5	270.3	580.9
9m	21.3	106.7	215.8	211.4	64.1	539.6	1185.5

Table 5.15: Comparison of Connections

Connection type	Single web Angle	Double web Angle	Header Plate	Top and Bottom Seat with Web angle	Top and Bottom Seat without Web angle	End plate With Web Stiffener	T stub	Rigid
Time Period (sec)								
5m	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53
7m	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53
9m	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53
Base Shear (kN)								
5m	6.12	6.00	6.94	6.26	6.26	5.99	5.36	4.98
7m	12.00	10.82	11.12	10.53	10.53	10.25	9.87	8.63
9m	20.36	20.68	19.10	19.27	19.67	17.32	16.37	13.26
Beam weight (kN)								
5m	135.17	135.17	126.88	127	126.88	119.91	98.24	84.02
7m	371.23	342.24	346.97	313.2	313.2	299.48	283.92	211.13
9m	753.68	753.68	753.68	715.1	715.51	617.53	550.04	402.69
Column Weight (kN)								
5m	135.17	135.17	126.88	127.00	126.88	119.91	98.24	84.02
7m	229.01	198.83	208.88	213.05	213.05	213.05	209.68	220.48
9m	264.55	280.51	239.95	247.93	366.08	315.82	415.85	662.75
Displacement in x dir(mm)								
5m	115.49	117.99	102.93	79.54	110.16	64.99	55.59	37.33
7m	62.30	45.17	41.93	29.71	43.87	20.33	14.98	13.48
9m	41.51	28.71	22.03	19.96	31.32	12.31	8.93	5.60
Displacement in z dir(mm)								
5m	251.12	210.35	174.70	129.54	186.80	108.36	92.66	70.34
7m	183.33	129.87	113.00	80.19	128.87	59.12	50.63	63.99
9m	223.50	92.17	69.05	67.19	111.02	51.14	45.59	40.81
Total Weight (kN)								
5m	306.24	300.12	346.97	313.20	313.20	299.48	267.76	248.97
7m	600.24	541.07	555.85	526.25	526.25	512.53	493.60	431.61
9m	1018.23	1034.19	955.05	963.44	983.61	865.86	818.54	662.75



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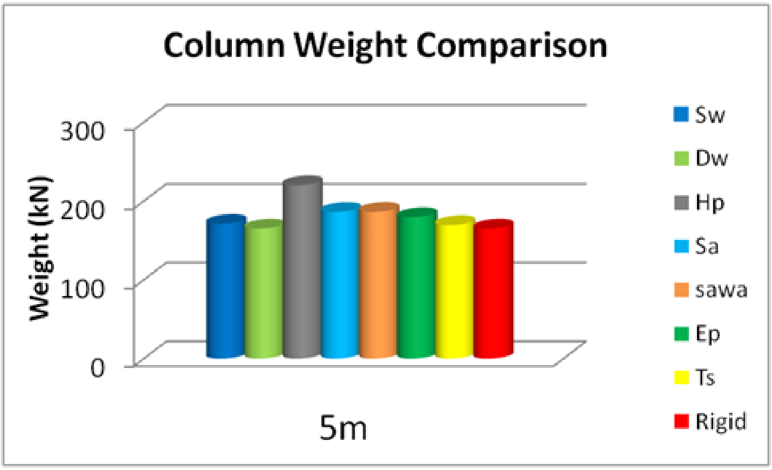

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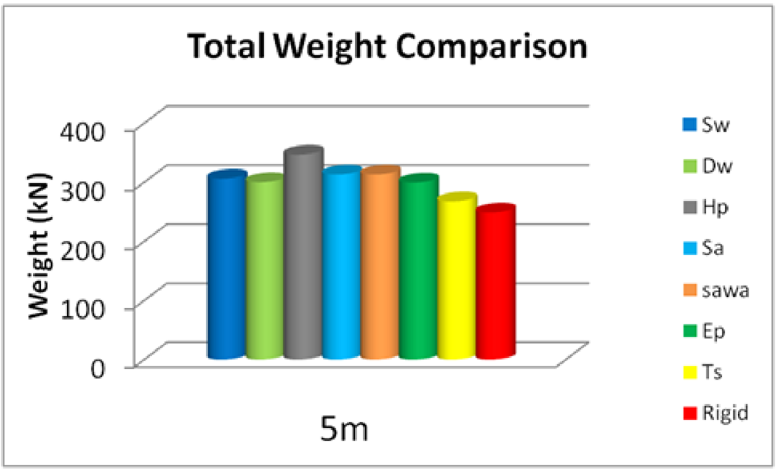

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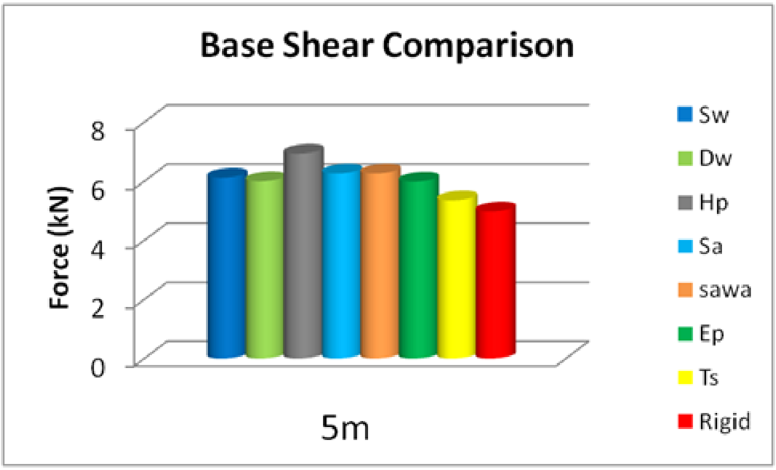
Figure 5.30: Weight Comparison for 5m Span



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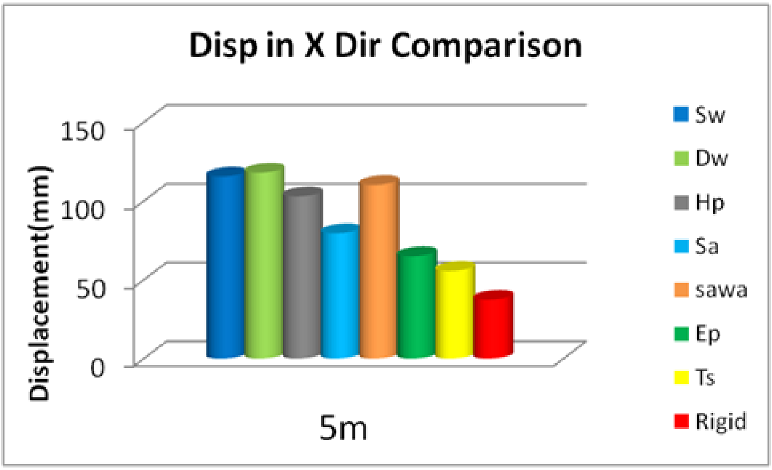

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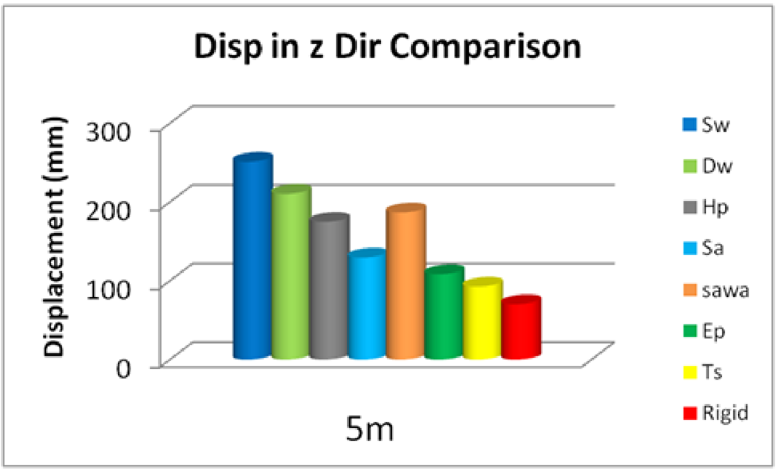

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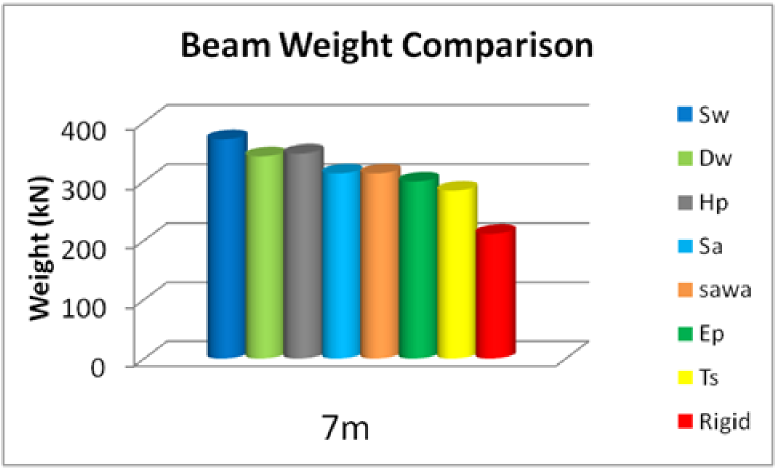
Figure 5.31: 5m Span

Values of T stub Connection For 5 m Span

1. Fig5.4 shows comparison for Beam, column and total weight of buildings.
2. There is no effect on the time period of the structure.
3. For 5m span Beam weight increases by 16.92 %.
4. For 5m span Column weight increases by 2.77 %.
5. For 5m span Total Weight weight increases by 7.55 %.
6. Fig5.4 shows the value of base shear displacement in X direction and displacement in Z directions for various connection stiffness.
7. Base shear increases by 7.55%.
8. Displacement in X direction increases by 48.92% but it is within the permissible limit of H/500.
9. Displacement in Z direction increases by 31.73% but it is within the permissible limit of H/500.



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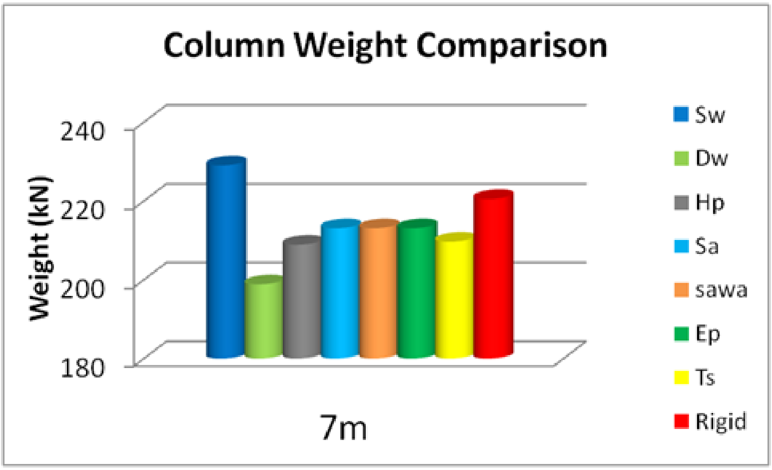

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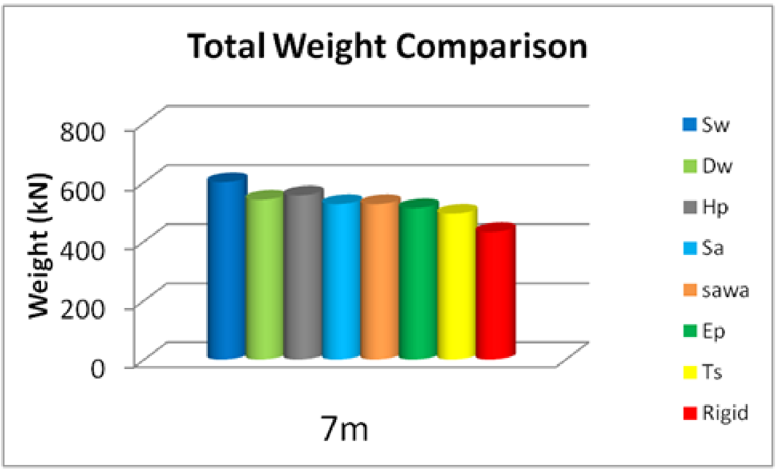
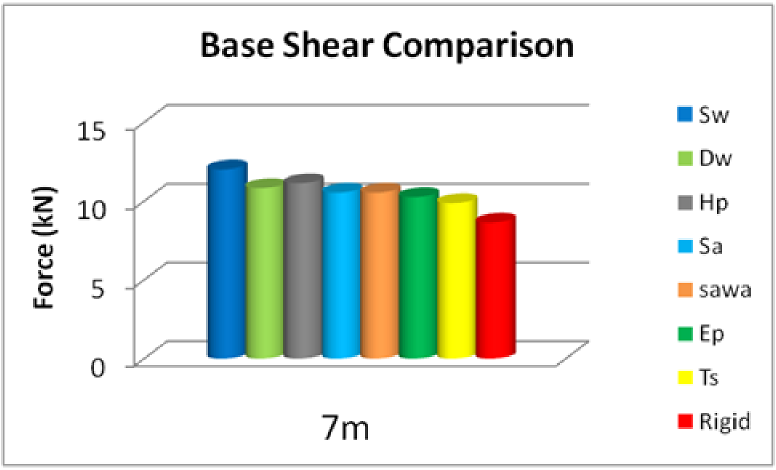


(c)

Figure 5.32: Weight Comparison for 7m Span



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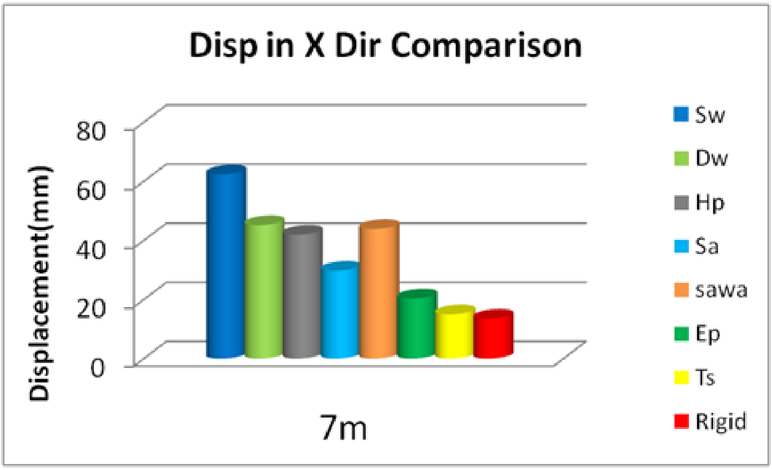

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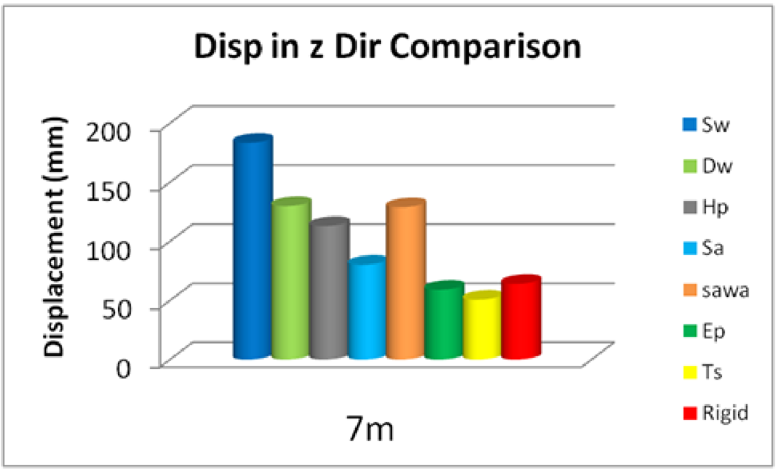

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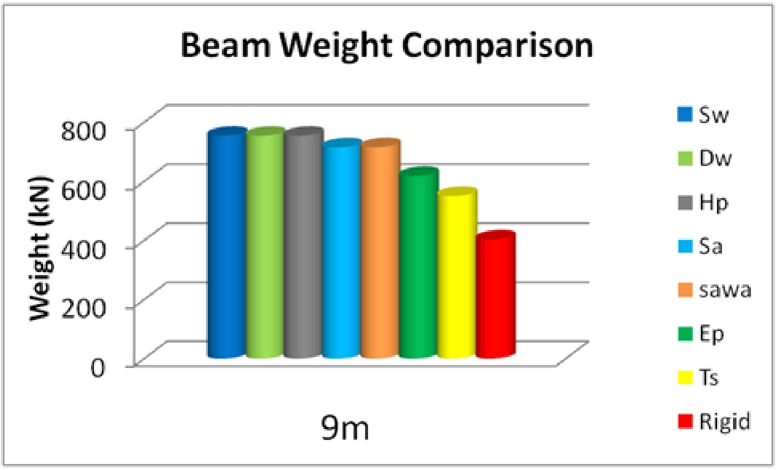
Figure 5.33: 7 m Span

Values of T stub Connection For 7 m Span

1. Fig5.4 shows comparison for Beam, column and total weight of buildings.
2. There is no effect on the time period of the structure.
3. For 7m span Beam weight increases by 34.48 %.
4. For 7m span Column weight decreases by 4.90 %.
5. For 7m span Total weight increases by 14.36%.
6. Fig5.4 shows the comparison for the value of base shear displacement in X direction and displacement in Z directions for various connection stiffness.
7. Base shear increase by 14.36%.
8. Displacement in X direction increases by 11.13% but it is within the permissible limit of H/500.
9. Displacement in Z direction decreases by 20.88% but it is within the permissible limit of H/500.



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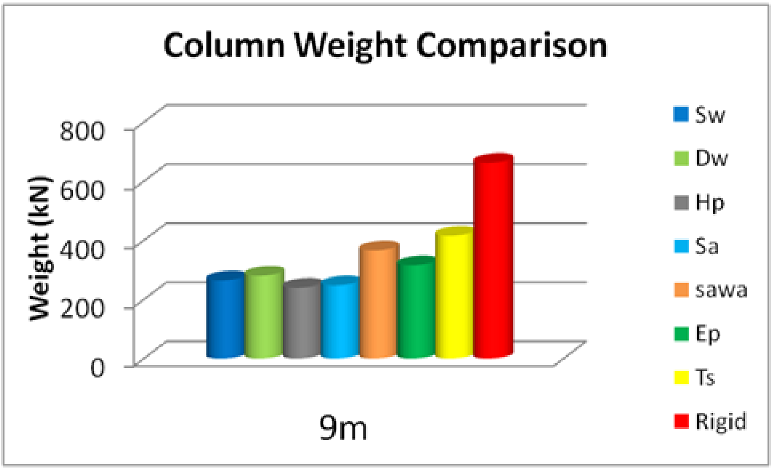

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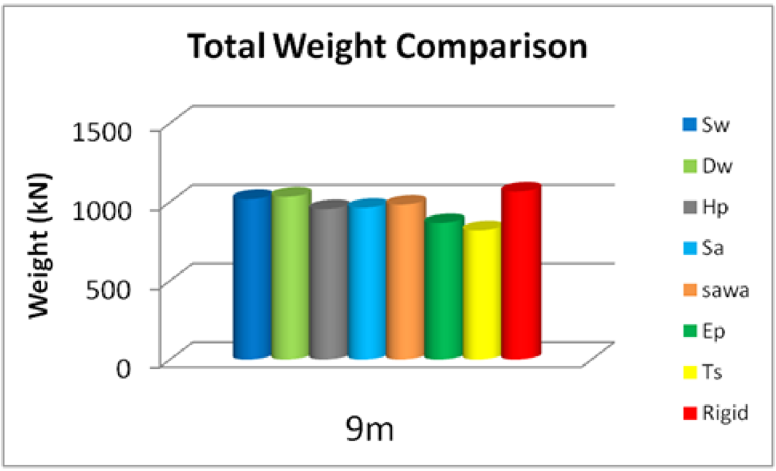

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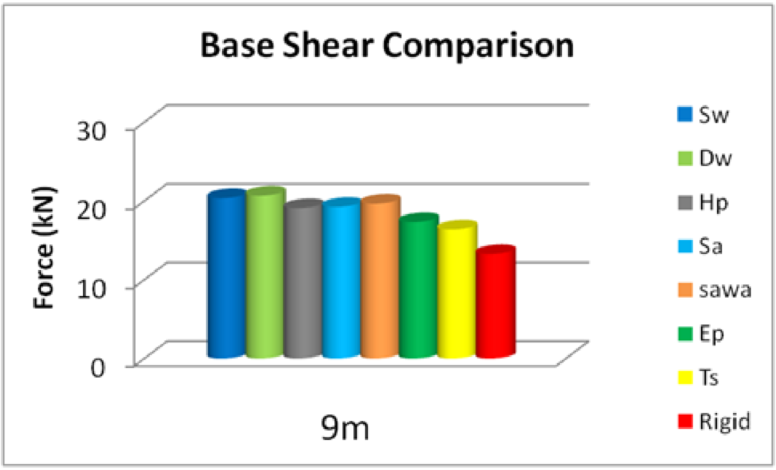
Figure 5.34: Weight Comparison for 9m Span



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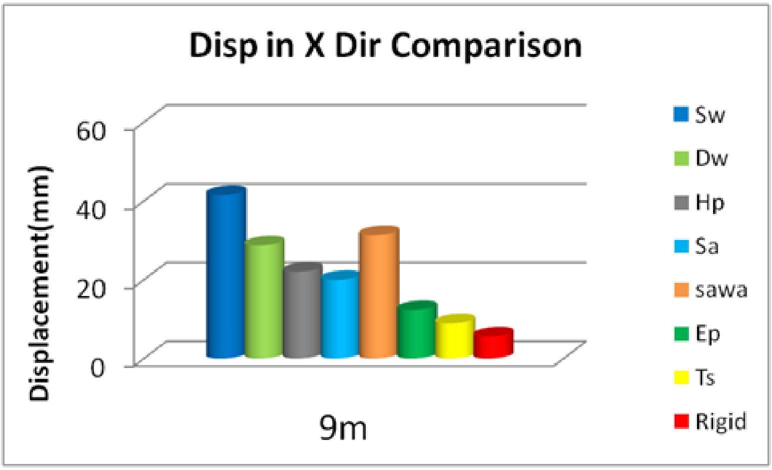

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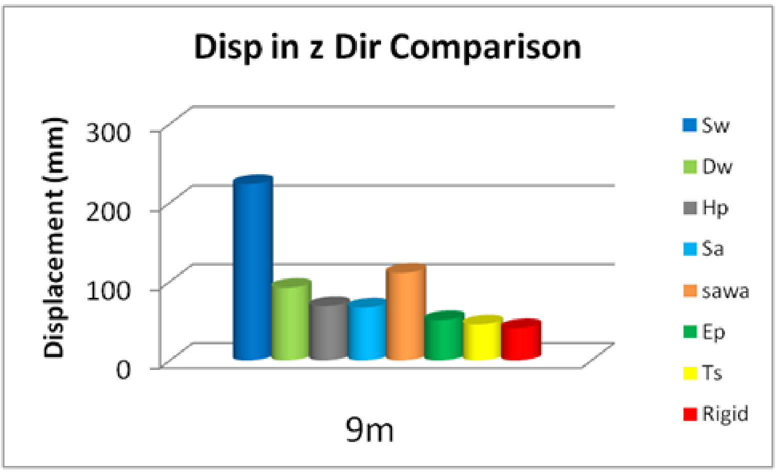

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Figure 5.35: 9 m Span

Values of T stub Connection For 7 m Span

1. Fig5.4 shows comparison for Beam, column and total weight of buildings.
2. There is no effect on the time period of the structure.
3. For 9m span Beam weight increases by 36.59 %.
4. For 9m span Column weight decreases by 37.25 %.
5. For 9m span Total weight decreases by 9.34 %.
6. Fig5.4shows the comparison for the value of base shear displacement in X direction and displacement in Z directions for various connection stiffness.
7. Base shear decreases by 9.34%.
8. Displacement in X direction increases by 59.46% but it is within the permissible limit of H/500.
9. Displacement in Z direction increases by 11.71% but it is within the permissible limit of H/500.

Values of End Plate Connection For 7 m Span

1. Fig5.4 shows comparison for Beam, column and total weight of buildings.
2. There is no effect on the time period of the structure.
3. For 9m span Beam weight increases by 53.35 %.
4. For 9m span Column weight decreases by 52.35 %.
5. For 9m span Total weight decreases by 12.40 %.
6. Fig5.4shows the comparison for the value of base shear displacement in X direction and displacement in Z directions for various connection stiffness.

7. Base shear decreases by 12.40%.
8. Displacement in X direction increases by 119.82% but it is within the permissible limit of H/500.
9. Displacement in Z direction increases by 25.31% but it is within the permissible limit of H/500.

5.4.1 Discussions

For End plate connection for all the spans

1. Beam weight increase by 41%. to 54%.
2. Column weight decreases by 3%. to 52%.
3. total weight of building decreases upto 13%
4. Base shear increase by upto 13%.
5. Displacement in X direction increases by 4%. to 119%.
6. Displacement in Z direction increases upto 54%.

For T-stub connection for all the span

1. Beam weight increase by 16% to 37%.
2. Column weight decreases by 4% to 38%.
3. Total weight of building decreases upto 9.5%
4. Base shear increase by upto 9.5%.
5. Displacement in X direction increases by 11%. to 60%.
6. Displacement in Z direction increases upto 32%.

5.5 Summary

In this chapter the design of beams,columns and is connection is carried out.Stiffness of the connection is calculate as per Frye and Morris model,effect of the semi rigid connections on the span length for low rise building is given through the graphs showing the effect on various parameters such as Bending moment,End moment,Shear force of Beam,Shear force of column,Axial force of column,Time period of structure ,Total displacement ,Beam weight,Column weight and Total weight is studied,Results show that that by considering the stiffness of the connection in the design of beams reduces total weight of the structure considerably which reduces the overall cost of the structure.

Chapter 6

Conclusion and Future Scope

6.1 Summary

In this study, alternative, economical and reliable systems for perimeter frames is investigated by designing low-rise long span frames with energy-dissipative semi rigid/ partial strength connections. In the design process, three different span lengths and seven different connection capacities are taken into account The connections are modelled with Frye and Morris model and rotational stiffness of the connections is found out. When the frames are designed with reduced connection capacities, the column steel weight is only marginally decreased. In order to be able to optimize the steel weight in low-rise long span Semi-rigid frames, more hot-rolled sections in this range should be researched. On the other hand, the beam sections of the Semi-rigid frames are larger than those in the rigid frames. This is due to the fact that the stiffness of the semi-rigid connection is neglected during the design of beams under gravity loads. As a result, the total steel weight is reduced as the connection capacity Reduced. Furthermore, this reduction can be even higher if the connection stiffness is taken into account in the design of beams under gravity loads. According to the analysis, all of the sample frames satisfied the acceptance criteria and showed a reliable performance under the earthquake load. The over design problem in low-rise long span Buildings is eliminated to some extent without using the perimeter frames

6.2 General Conclusion

1. Semi rigid connections are better connections.

2. In reality every connection has some rotational stiffness so there is no rigid connection or simple connection.

3. As per IS 800-2007 Eight types of connections are studied and modelled as per frye and Morris model.

4. There is no effect of connection on shear force of beam.

5. There is no effect on the Axial force of the column.

6. Time period of the building remains constant for all the connections.

7. T-stub connection is better upto the span length of 7m.

8. After 7m span End plate connection is better for span 9 m.

6.3 Conclusion

Comparison of T stub connection with End Plate connection Upto Span of 7 m

1. Beam weight decreases by 5% to 23%.

2. Column weight increases by 1 to 6%.

3. Total weight of building decreases by 3 to 12%

4. Base shear decreases by 3% to 12%.

5. Displacement in X direction decreases by 16% to 35%.

6. Displacement in Z direction decreases by 15% to 17%.

Values for End Plate connection for 9m Span

1. Beam weight of building of 9 m span increases by 53.35 %.
2. Column weight of building of 9 m span decreases by 52.35 %.
3. Total weight of building of 9 m span decreases by 12.40 %.
4. Base shear decreases by 12.40%.
5. Displacement in X direction increases by 119.82% which is within permissible limit H/500.
6. Displacement in Z direction increases by 25.31% which is within permissible limit H/500.

Importance of Semi rigid connections.

1. Reduction in the weight of the column is observed marginally about for spans upto 7m. Average reduction in the weight of column is about from 3 % to 54%.
2. While the weight of the beams increases by form 47 % to 73 % for different spans and various types of connections.
3. Average reduction in the total weight of the building is around 7 %.
4. Base shear decreases from 7%.because the time period of the structure remains same.
5. Displacement in X direction increases by 147% to 320%. which is within the safe limit H/500.
6. Displacement in Z direction increases upto 99% to 145%.which is within the safe limit H/500.

6.4 Future Scope

1. Parametric study of Three bay frame with G+5, G +7, and G+10 can be done further to see the effect of semi rigid connection with varying height.

2. Non linear analysis of the building G+3, G+5, G+7 and G+10 to compare the results with linear analysis is needed to be done further.

3. Semi rigid column base modeling is to be considered to study behavior and economical aspects of structure design.

4. Moment rotation curve can be obtained using various models such as Experimental,Empirical ,Mechanical Models etc. Then comparative study on various types of Models

5. Moment rotation curvature is to be evaluated experimentally to find stiffness of different types of connections.

6. Modeling of connection can be checked by using this moment rotation curves for user defined hinges sat the face of the column to compare the results with spring model.

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Appendix A

Shear Connection

A.1 Single Web Angle Connection.

Input Data Reactions from beam = 140 kN

Section for Beam = ISMB 350

Section for column = ISWB 600

Grade of steel =250M_{pa}

Dia of Bolts =20mm

Grade of Bolts = 4.6

fy = 250 MPa

Design steps involved in design of web angle connection are as follows.

1. Shear capacity of Bolt in single shear =45.3 kN

   Bearing capacity of Bolt =56.58 kN

   Tf =6.9 mm

2. Adopt a gauge distance g =20(assumed)+.5*6.9=23.45 mm

3. Select a angle 90x90x8

4. No of Bolts= 140/45.3 $\approx$ 3

Adopting vertical pitch =60mm

Edge distance = 40 mm

5. Shear is assumed to be acting on face of column.

   Due to eccentricity, some horizontal shear forces is acting on bolt group in addition to shear due to reaction.

   Eccentricity e = 50 mm

   $H = \frac{V_x e_x r_i}{\sum r_i^2} = 14.07kN$

   Vertical shear force per bolt = 140/6 = 23.33 kN

   Resultant shear force $= \sqrt[2]{14.07^2 + 23.33^2} = 27.47 < 45.3$ kN

   Hence connection is safe

   Assuming μ=.48, Slip resistance per bolt = 52.71 kN

6. Determine length of web angle based on number of bolts, pitch and normally not less than 0.6 to 0.75 times depth of beam.

   Cleat angle Take two angles of ISA90X90X8 of length 200 mm.

   Check Bending at bolt line of connection to column flange.

   Bending Moment = $\frac{140}{2}x50$ = 1641.5 kNmm

   . Moment Capacity $=\frac{1.2 f_y Z}{\gamma_{mo}}$=18050 > 1641.5 kNmm

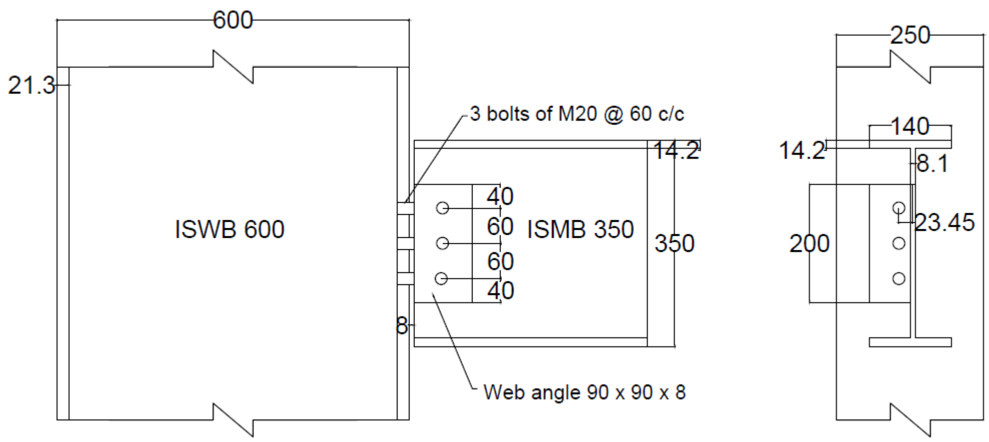


Figure A.1: Single Web Angle

A.1.1 Double Web Angle Connection.

Input Data Reactions from beam = 140 kN

Section for Beam = ISMB 350

Section for column = ISHB 200

Grade of steel =250M_{pa}

Dia of Bolts =22mm

Grade of Bolts = 4.6

fy = 250 MPa

Design steps involved in design of web angle connection are as follows.

1. Shear capacity of Bolt in single shear =52.6 kN

   Bearing capacity of Bolt =72.98 kN

   Tf =8.9 mm

2. Adopt a gauge distance g =2*50(assumed)+8.9=108.9mm

3. Select a two angle 90x90x8

4. No of Bolts= 140/52.6 $\approx$ 4

   Adopting vertical pitch =70mm

Edge distance = 40 mm

5. Shear is assumed to be acting on face of column.

   Due to eccentricity, some horizontal shear forces is acting on bolt group in addition to shear due to reaction.

   Eccentricity e = 50 mm

   $H = \frac{V_x e_x r_i}{\sum r_i^2} = 30kN$

   Vertical shear force per bolt = 140/4 = 35 kN

   Resultant shear force =$\sqrt[2]{30^2 + 35^2}$ = 46.09 < 52.6 kN

   Hence connection is safe

   Assuming μ=.48, Slip resistance per bolt = 52.71 kN

6. Determine length of web angle based on number of bolts, pitch and normally not less than 0.6 to 0.75 times depth of beam.

   Cleat angle Take two angles of ISA90X90X8 of length 290 mm.

   Check Bending at bolt line of connection to column flange.

   Bending Moment = $\frac{140}{2}x40$ = 3500 kNmm

   . Moment Capacity =$\frac{1.2 f_y Z}{\gamma_{mo}}$=33640 > 3500 kNmm

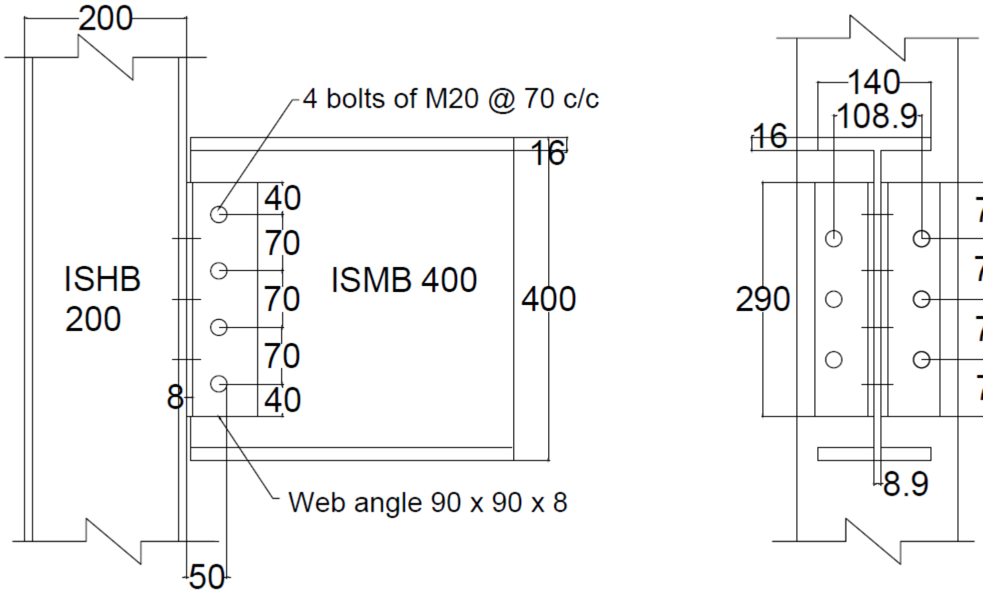


Figure A.2: Double Web Angle

A.1.2 Header Plate Connection.

Input Data

Reactions from beam = 140 kN

Section for Beam = ISMB 400

Section for column = ISHB 200

Grade of steel =250M_{pa}

Dia of Bolts =20mm

Grade of Bolts = 4.6

fy = 250 MPa

1. Shear capacity of Bolt in single shear =45.3 kN

   Bearing capacity of Bolt =49.20 kN

   Assume thickness of end plate = 6mm

2. No of Bolts= 140/45.3 $\approx$ 4

   Adopting vertical pitch =60mm

   Edge distance = 50 mm

Length of end plate= 60+2*50 =160 mm< $30 * t = 30 * 6 = 180$ mm

3. Provide the dimensions of end plate l = 150 mm

   b= 150mm

   t= 6 mm

4. length of fillet weld connecting end plate to beam web = 150-2*6=138 mm

   size of weld=$(\frac{140*10^3}{2*138*158}) = 3.21$mm

   Provide minimum size of weld = 6mm

   Shear stress on the web of the beam =$(\frac{140*10^3}{8.9*150}) = 104.9Mpa < (\frac{250}{\sqrt{3}x1.10}) = 131.2Mpa$ Hence connection is safe..

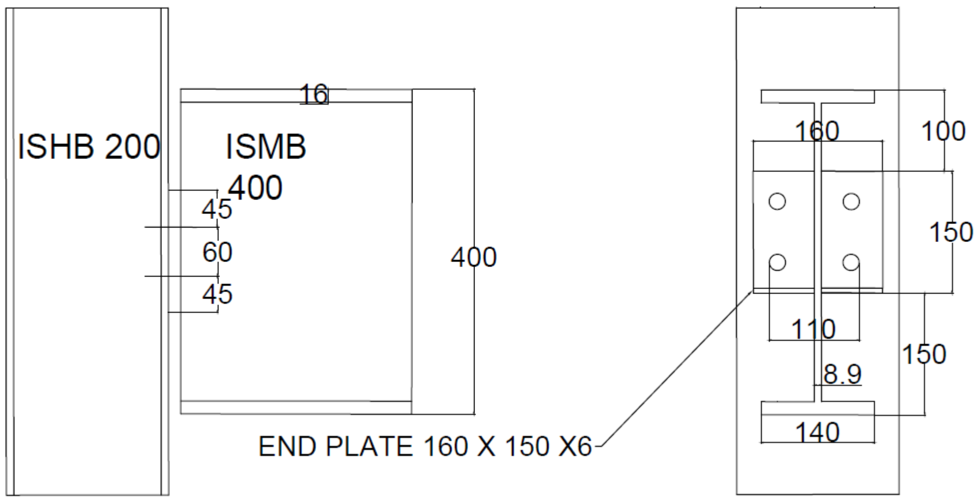


Figure A.3: Header Plate Connection.

A.1.3 Top and seat Without Web Angle Connection.

Input Data

Reactions from beam = 140 kN

Section for Beam = ISMB 400

Section for column = ISHB 200

Grade of steel =250M_{pa}

Dia of Bolts =20mm

Grade of Bolts = 4.6

fy = 250 MPa

1. Shear capacity of Bolt in single shear =45.3 kN

   Strength in bearing $= 2.5 * 0.5 * \emptyset * t * f_u/\gamma_{mo}$ = 123 kN.

   Hence, Strength of Bolts = 45.3 kN.

   Required No of Bolts= $140/45.3 \approx 4$

   assume angle l = 150

   b = 75

   t = 12

2. width of beam = 140 mm

   Length of angle= 140 mm

   Length of bearing required at root line of beam from Equation

   l= $\frac{R}{t_w}(\frac{f_{yw}}{)}\gamma_{mo}$=57.14 mm

3. A dispersion of 45 is taken from bearing on cleat to root line

   Required Length of out standing leg=57.14+10=67.14 <150

4. Length of bearing on cleat,b_1 = 57.15 -(T+r)=30.05mm

   Distance from the end of bearing on cleat to root angle

   $b_2 = b_1 + 5 + 5 - (t + r_a)$ =30.05+10 - (12+10)=18.05mm

5. Moment at root of angle $=(\frac{100*18.05}{30.05})(\frac{18.05}{2})$ = 542 Nm

   Moment capacity $=(\frac{1.2Zf_y}{\gamma_{mo}}) = 916Nm > 542Nm$

Hence connection is safe

6. Shear capacity of the outstanding leg of cleat=$\frac{R*t*f_y}{\sqrt{3}*1.10} = 220kN > 100kN$

   Shear strength of beam $=V_d = (\frac{A_v f_{yw}}{\sqrt{3}*1.10}) = 303KN > 100kN$

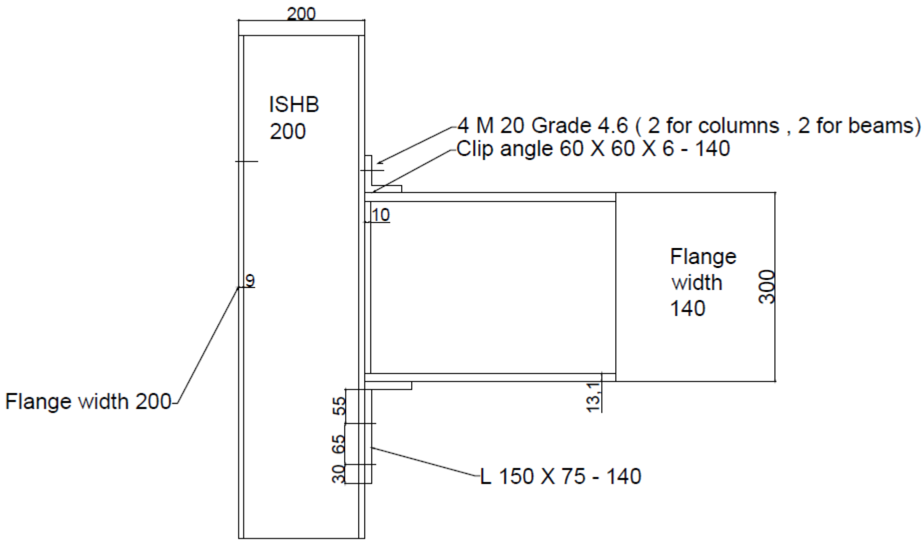


Figure A.4: Top and seat Without Web Angle Connection

Appendix B

Moment Connection

B.1 End Plate Connection.

Input Data factored shear force = 120 kN

factored Bending moment = 120kN

H_z = 20 kN

Section for Beam = ISMB 300

Section for column = ISHB 200

Grade of steel =250Mpa

Dia of Bolts =20mm

Grade of Bolts = 4.6

fy = 250 MPa

1. Dimensions of end plates are fixed based on dimension of beam and column flange

   $t_f = 13.1$

   Adopted end distance le= 70 mm.

   $d_f = 300$ mm

   $120 * 10^3 + 20 * (300/2 - 13.1/2) = (2F_1 + 2F_2) * (300 - 13.1) + 2F_3 * (70 - 13.1 * .5)$

   assuming $F_1 = F_2$

   so we get $F_1 = F_2 = 104.51$ kN

   and by similarity of trianglesF_3=$(70 - 0.5 * 13.1)/(300 - 13.1) * F_1 = 23.11$ kN

Reaction at bottom flange F_c = 2(104.51 + 104.51 + 23.11) - 20

=444.26 kN

Capacity of beam flange = $(f_y/\gamma_{mo})A = 416.8kN$

2. Width of end plate = 180 mm.

   Distance from center line of bolt to toe of fillet weld,

   l_v= 40-10 = 30 mm.

   Adopted end distancel_e= 50 mm.

   Effective length of end plate per bolt = 180/2 = 90 mm

3. Tension Capacity of M-20 Bolt 141 = kN

   Allowable pry load Q = 141 - 104.5 = 36.49 kN

   Moment at toe of weld =$Tl_e - Ql_e$ = 104.51*30-36.49*50 = 1310.8 Nm.

   Moment capacity of the plate=$(f_y/1.10)(wT^2/4)$

   $\therefore T = \sqrt{1310.8 * 10^3 * 1.10 * 4/(250 * 90)} = 16mm$ Adopt T = 20 mm

4. proof stress =$0.7f_{ub} = 0.7 * 800/1000 = 0.56kN/mm^2$

$$Q = (l_v/2l_e)(T_e - \beta\gamma b_e t^4/(27l_e(l_v)^2)) \tag{B.1}$$

   β = 2 for non pre loaded and, γ= 1.5 for factored load

   =25.38 < 36.49 kN .

5. check for combine shear and tension

   shear capacity of M 20 Bolt = 52.6 kN

   Tensile capacity of the Bolt = 120/6 = 2kN

   $(20/52.6)^2 + ((104.51 + 25.38)/141)^2 = 0.993 < 1$

   hence connection is safe

B.1.1 T stub Connection.

Input Data factored shear force = 120 kN

factored Bending moment = 120kN

H_z = 20 kN

Section for Beam = ISMB 300

Section for column = ISHB 200

Grade of steel =250Mpa

Dia of Bolts =20mm

Grade of Bolts = 4.6

fy = 250 MPa

1. Strength in single shear = 29 kN. Strength in double shear = 58 kN. Bearing strength on web of thickness 8.1 mm (ISMB 350) = 103.68 kN Tensile strength of bolts = 42.7 kN.

2. Provide angle 75x75x 8mm

   6No. M16 bolts connected to flanges of the beam

   3No. M16 bolts connected to web of the beam

   Use a t-stub cut from ISMB450

   b_f=150mm

   t_f= 17.4 mm

   t_w = 9.4 mm

   r =15mm

3. Flange force = $50 * 10^3/d_f$ = 142.86 kN. $\because$ d_f= 350

   Required no of bolts =142.86/29 =4.93

   Provide 6 Bolts in 2 rows

4. Thickness of T web (adopting a width of 140mm)

t=$(p/(bxf_y/\gamma_{mo})$= 5.99 mm < 9.4 mm
hence thickness is adequate

5. Provide a edge distance 30 mm
distance from fillet line to bolt =a= ((150/2)-(9.4/2)-15-30) = 25.3 mm
Moment in T stub = 0.5pxa = 0.5*142.86*25.3=1807.2 kNmm
Moment capacity of T-Stub flange from eq
$= 1.2X(250/1.10)X140X17.4^2/6X10^{-3}$
1926.65> 1807.2 kNmm
Hence thickness provided is adequate
The forces in a row of bolts connecting the T stub to column
=p/2 = 71.43kN
Provide 4 20mm dia Bolts in two rows witha pitch of 60 mm and a edge distance of 40mm
Tension capacity of20 mm Bolts = 4x66.6=266.4> 142.86 kN
Tensile force in Bolt due to applied moment = 142.86/4=35.72 kN

6. Check for prying force
l_e = 30 mm (provided) l_v = ((150/2)-(9.4/2)-(15/2)-30) = 32.8 mm
γ = 1.5 (factored load)
β = 2 (non preloaded)
$f_o = 0.7 f_{ub}$ = 0.7x400 = 280 Mpa

$$Q = (l_v/2l_e)(T_e - \beta\gamma b_e t^4/(27l_e(l_v)^2)) \tag{B.2}$$

Q = 12.76 kN
Total tensile load in bolt = 35.72 + 12.76 = 48.48¡66.6 kN

7. check for thickness of column flange

Assuming an edge distance of 40 mm on the column flanges s_b = 150 -(2x30) = 90 mm

a = (250-7.6)/2 = 121.2

Required thickness

$t_b = \sqrt{(4nF_t c)/(af_y)/(4\sqrt{2} + (n-1)s_b/a)}$

= 8.94 < 10.6 mm

Hence the thickness is adequate

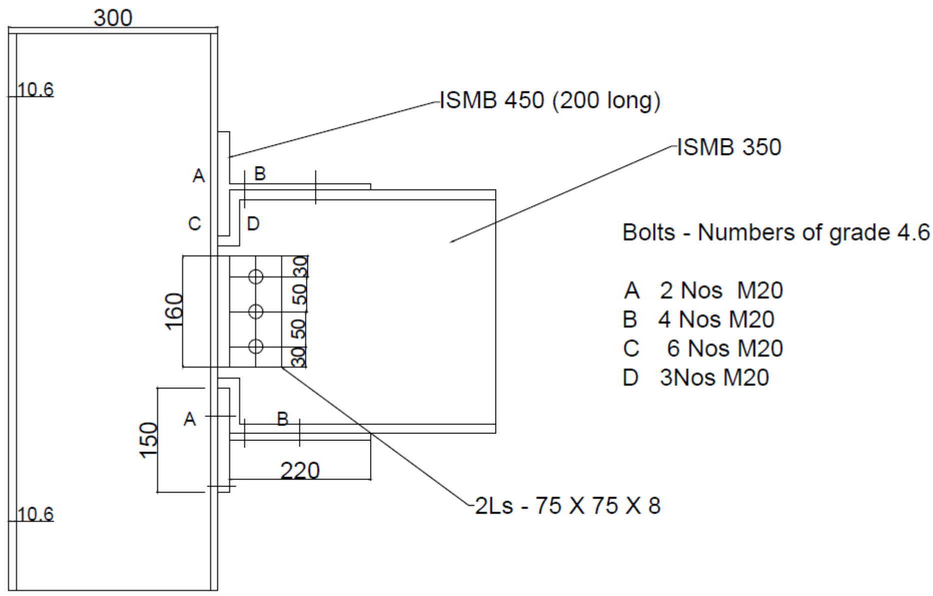


Figure B.1: T stub Connection

Appendix C

Results for G+3

C.1 Results for G+3 With 7m Span

C.1.1 Design of Connections

Single Web Angle Connection

Table C.1: Single Web Angle Connection for 7 m Span

Storey	3^{rd}	2^{nd}	1^{st}
Beam Section	ISMB=400	ISMB=500	ISMB=450
Column Section	ISHB=450	ISWB=600	2ISMC=350 c/c=400 Plate width=400 TK=16
Guage Distance	29.45	30.1	29.7
Angle Size	65X65X6	80X80X6	70X70X6
Edge Distance	40	40	40
Pitch	60	60	60
Bolt Dia	20	20	20
No. of Bolts	3	3	3
Length	200	200	200

Double Web Angle Connection

Table C.2: Double web Angle Connection for Storey 7 m Span

Storey	3^{rd}	2^{nd}	1^{st}
Beam Section	ISMB=400	ISMB=500	ISMB=450
Column Section	ISHB=450	ISWB=600	2ISMC=350 c/c=400 Plate width=400 TK=16
Guage Distance	108.9	110.2	109.4
Angle Size	90X90X6	90X90X6	90X90X6
Edge Distance	54.45	55.1	54.7
Pitch	70	70	70
Bolt Dia	20	20	20
No. of Bolts	3	4	4
Length	220	290	290

Header Plate Connection

Table C.3: Header Plate Connection for Storey 7 m Span

Storey	3^{rd}	2^{nd}	1^{st}
Beam Section	ISMB=400	ISMB=500	ISMB=450
Column Section	ISHB=450	ISWB=600	2ISMC=350 c/c=400 Plate width=400 TK=16
End Plate Size	140X140X6	200X200X8	200X200X8
Edge Distance	40	40	40
Pitch	60	60	60
Bolt Dia	20	20	20
No. of Bolts	2	3	3
Welding Length	128	184	184

Top and Bottom Seat Angle with Web Angle

Table C.4: Top and Bottom Seat Angle with Web Angle for Storey 7 m Span

Storey	3^{rd}	2^{nd}	1^{st}
Beam Section	ISMB=400	ISMB=500	ISMB=450
Column Section	ISHB=450	ISWB=600	2ISMC=350 c/c=400 Plate width=400 TK=16
Angle Size	80X80X6	110X110X6	90X90X6
Stiffner Angle	2 Nos.60X60X6	2 Nos. 90X90X6	2 Nos. 80X80X6
Edge Distance	45	45	45
Pitch	70	70	70
Bolt Dia	20	20	20
No. of Bolts	2	4	4
No. of Rows	2	4	4
Welding Length	160	300	300

Top and Bottom Seat Angle without Web Angle

Table C.5: Top and Bottom Seat Angle without Web Angle for Storey 7 m Span

Storey	3^{rd}	2^{nd}	1^{st}
Beam Section	ISMB=400	ISMB=500	ISMB=450
Column Section	ISHB=450	ISWB=600	2ISMC=350 c/c=400 Plate width=400 TK=16
Seat Angle	110X110X10	110X110X10	110X110X10
Edge Distance	40	40	40
Pitch	60	60	60
Bolt Dia	20	20	20
No. of Bolts	2	4	4
Length	125	140	140

End Plate with Stiffener Connection

Table C.6: End Plate with Stiffener Connections for Storey 7 m Span

Storey	3^{rd}	2^{nd}	1^{st}
Beam Section	ISMB=300	ISMB=400	ISMB=400
Column Section	ISHB=400	IWMB=600	2ISMC=225,c/c =350 Plate Width=350,Tk=16
Plate Size	300X190	300X190	300X190
Thickness	20	14	14
Bolt Dia	24	27	24
No. of Bolts	6	6	6
Weld Size	14	14	14
Length	250	250	250
Web Stiffener	110X12	80X6	80X6
Weld Size	12	6	6

T-Stub Connection

Table C.7: T-Stub Connection for Storey 7 m Span

Storey	3^{rd}	2^{nd}	1^{st}
Beam Section	ISMB=300	ISMB=400	ISMB=400
Column Section	ISHB=400	IWMB=600	2ISMC=225, c/c =350 Plate Width=350,Tk=16
Web Angle	90X90X8	90X90X8	90X90X8
T-Stab Cut From	ISMB=350	ISMB=400	ISMB=400
Length	240	300	300
Edge Distance	50	50	50
Pitch	50	50	50
Bolt Dia	22	24	24
No. of Bolts	4	6	6
No. of Rows	2	2	2

Table C.8: Member Sizes for 7 m Span for Various Connections

3^{rd} Storey	0	4.7	21.8	16.1	77.4	28.4	190.2	371.2	10000
Beam	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB
Section	400	400	350	400	350	350	300	300	250
Column	ISHB	ISHB	ISHB	ISHB	ISHB	ISHB	ISHB	ISHB	ISHB
Section	225	350	300	250	300	300	300	300	400
2^{nd} Storey	0	8.8	41.9	61.5	130.5	47.8	270.3	580.9	10000
Beam	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB
Section	500	500	450	450	450	450	450	400	300
Column	ISWB	ISWB	ISWB	ISWB	ISWB	ISWB	ISWB	ISWB	ISWB
	600	600	600	600	600	600	600	600	600
1^{st} Storey	0	4.7	41.9	59.2	97.2	35.5	270.3	580.9	10000
Beam Section	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB
Section	450	450	450	450	400	450	400	350	300
Column	ISMC	ISMC	ISMC	ISMC	ISMC	ISMC	ISMC	ISMC	ISMC
Section	350	350	250	250	250	250	250	250	200
C/C between	600	600	600	600	600	600	600	600	600
Plate Widht	600	600	600	600	600	600	600	600	600
Thickness	16	16	16	16	16	16	16	16	16

Table C.9: Values of k for Connections for 3^{rd} Storey 7m Span

	K valuein kNm/rad	kNm/degree	
		3^{rd} Storey	
Sr No	Connection	kN/radian	kN/degree
1	Single web angle	268.86	4.69
2	Double web angle	1250.27	21.82
3	Header plate	921.42	16.08
4	Top and bottom without web angle	4434.75	77.41
5	Top and bottom with web angle	1659.41	28.97
6	End plate connection	10897.21	190.21
7	T stub connection	21267.39	371.22

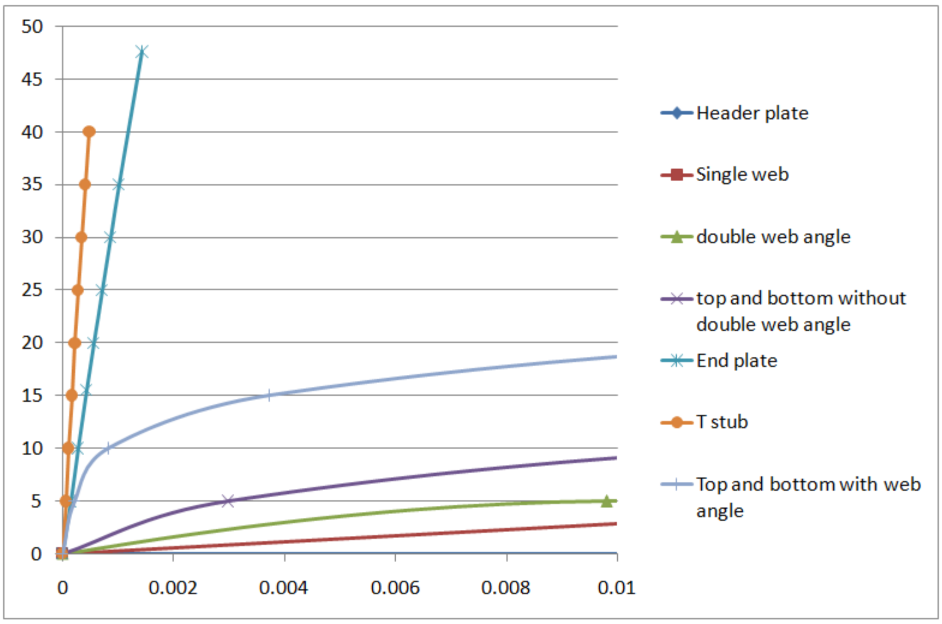


Figure C.1: Moment Rotation Curve for Various Connections of3^{rd} Storey 7 m Span

Table C.10: Values of k for Connections for2^{nd} Storey 7 m span

		2^{nd} Storey	
		2^{nd} Storey	
Sr No	Connection	kN/radian	kN/degree
1	Single web angle	504.65	8.81
2	Double web angle	2400.27	41.90
3	Header plate	3522.821	61.49
4	Top and bottom without web angle	7474.48	130.47
5	Top and bottom with web angle	2736.54	47.77
6	End plate connection	15486.65	270.32
7	T stub connection	33790.52	589.82

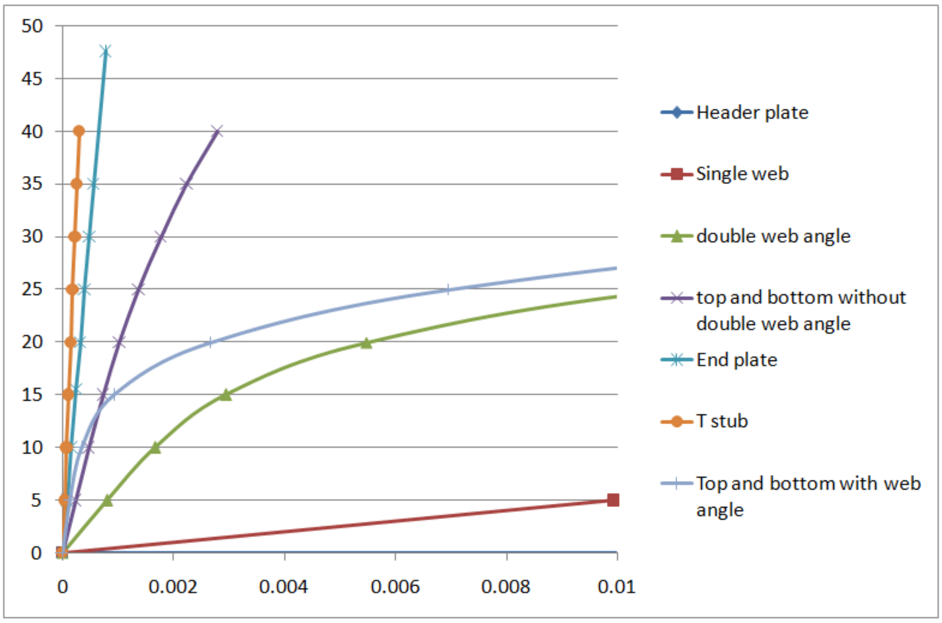


Figure C.2: Moment Rotation Curve for Various Connections of2^{nd} Storey 7 m Span

Table C.11: Values of k for Connections for1st Storey 7 m Span

Sr No	Connection	kN/radian	kN/degree
1	Single web angle	268.86	4.69
2	Double web angle	2400.27	41.90
3	Header plate	3391.78	59.20
4	Top and bottom without web angle	5567.14	97.17
5	Top and bottom with web angle	2024.66	35.34
6	End plate connection	15486.65	270.32
7	T stub connection	33790.52	589.82

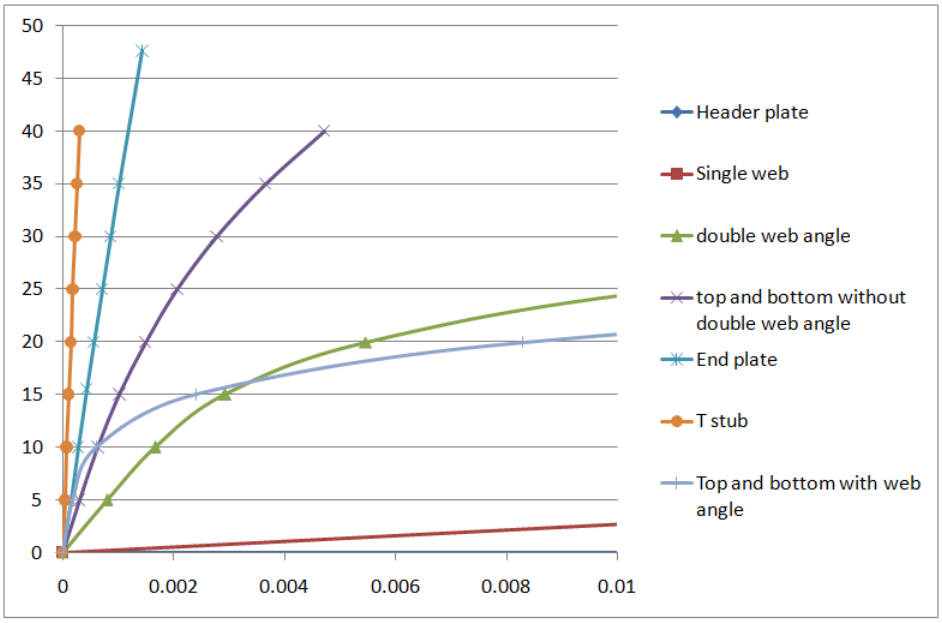
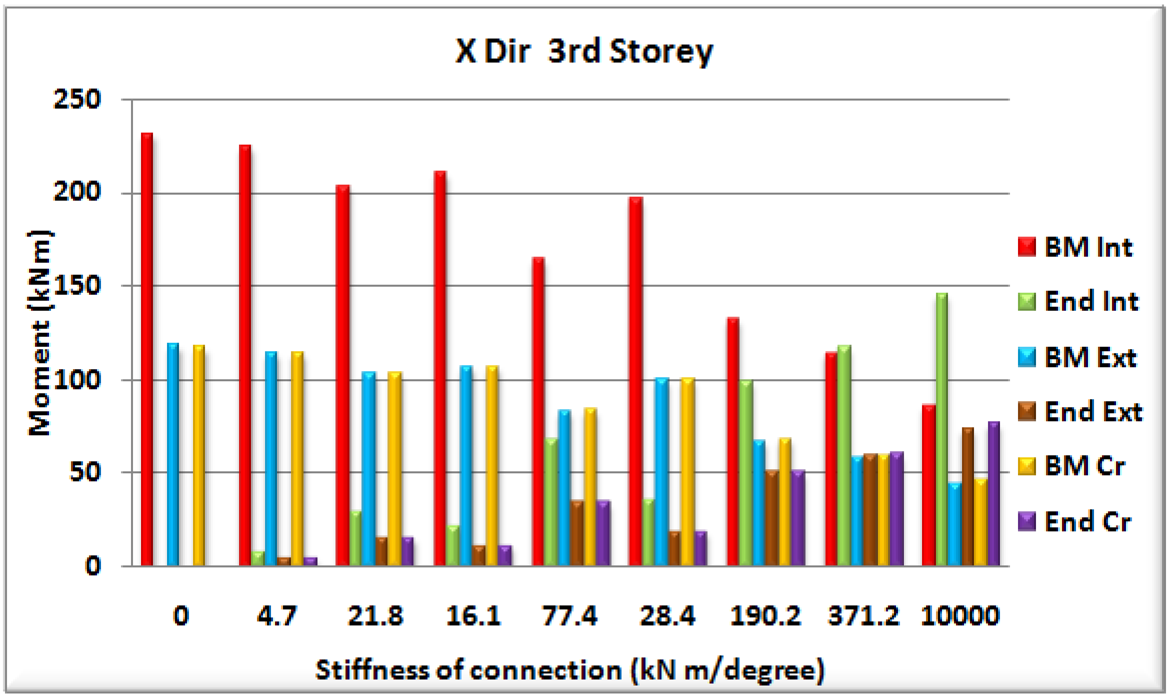
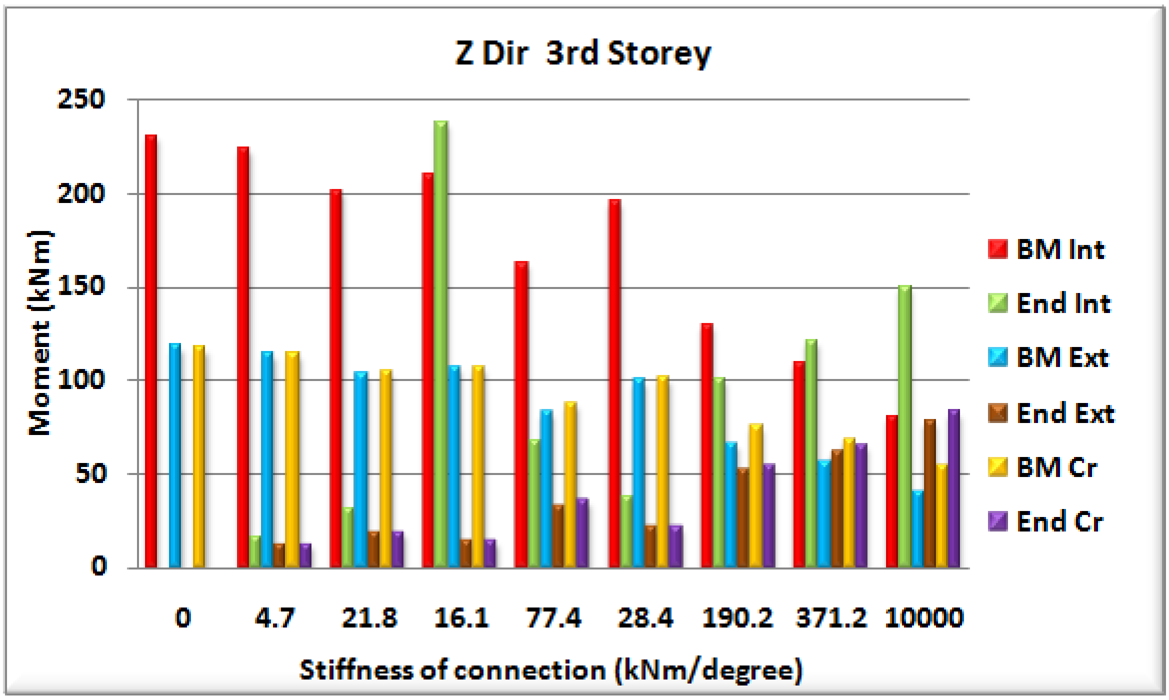


Figure C.3: Moment Rotation Curve for Various Connections of1st Storey 7 m Span

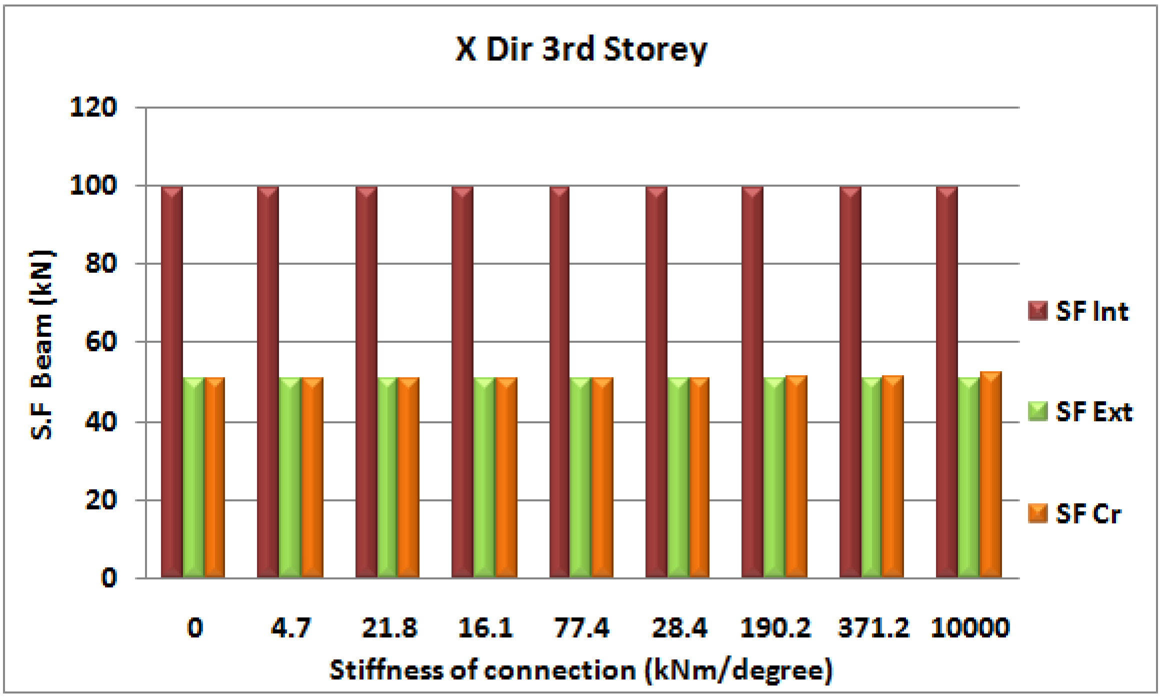


(a) Reduction in Moment of X Dir Beams

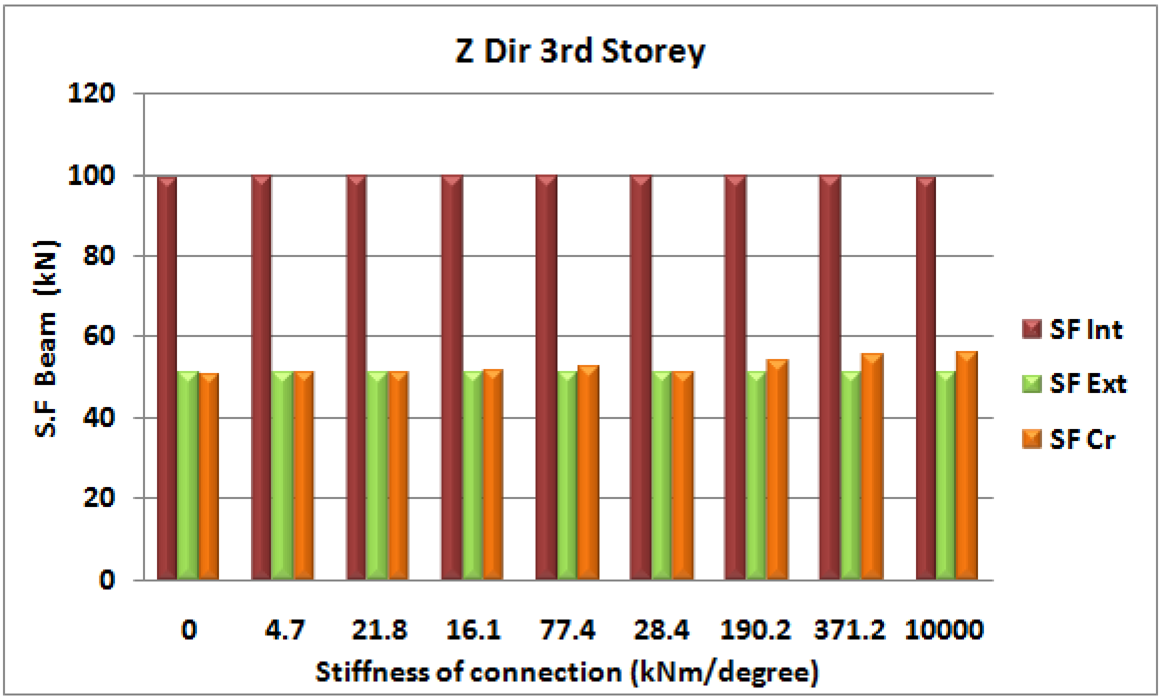


(b) Reduction in Moment of Z Dir Beams

Figure C.4: Reduction in Moment of Beams Due to Connections of 3^{rd} Storey 7 m Span

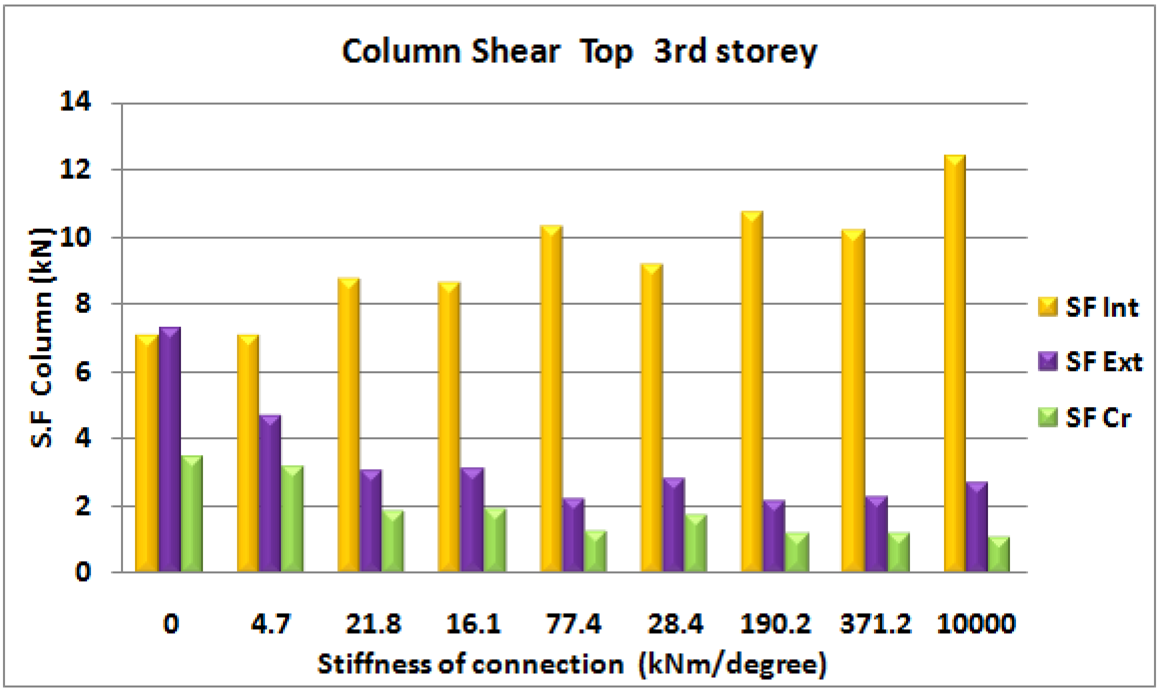


(a) Effect on Shear Force of X Dir Beams

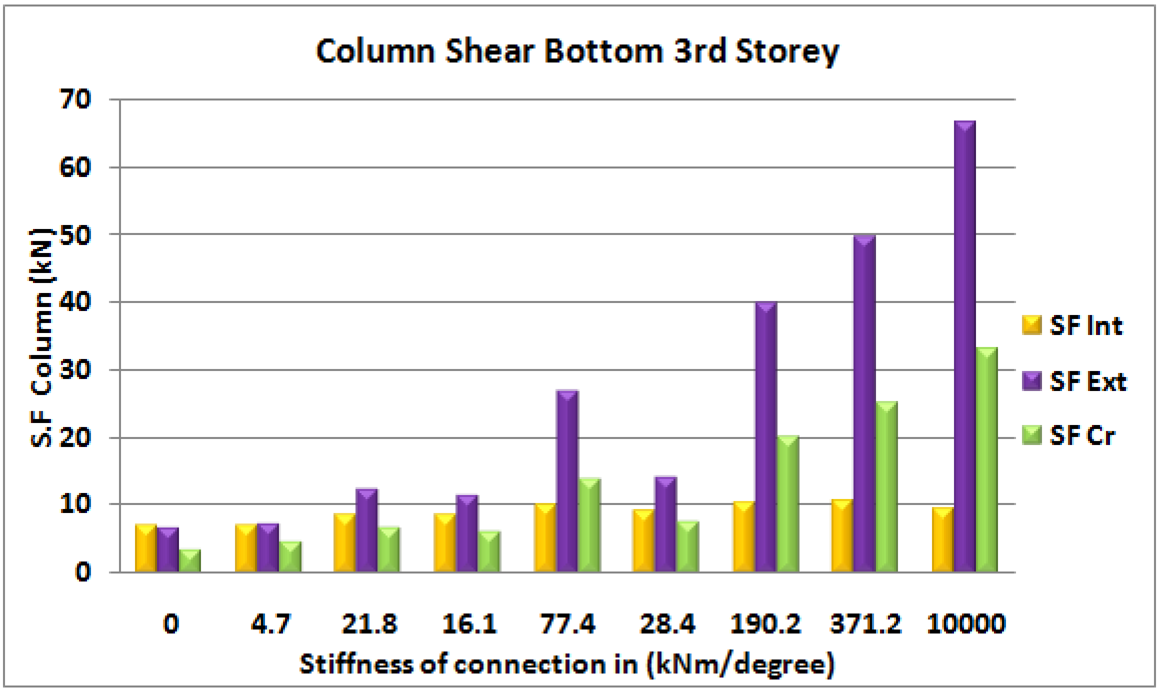


(b) Effect on Shear Force of Z Dir Beams

Figure C.5: Effect og Shear Force on Beams Due to Connections on 3^{rd} Storey 7 m Span



(a) Effect of Shear Force on Top of Column



(b) Effect of Shear Force on Bottom of Column

Figure C.6: Effect of Shear Force on Column Due to Connections of 3^{rd} Storey 7 m Span

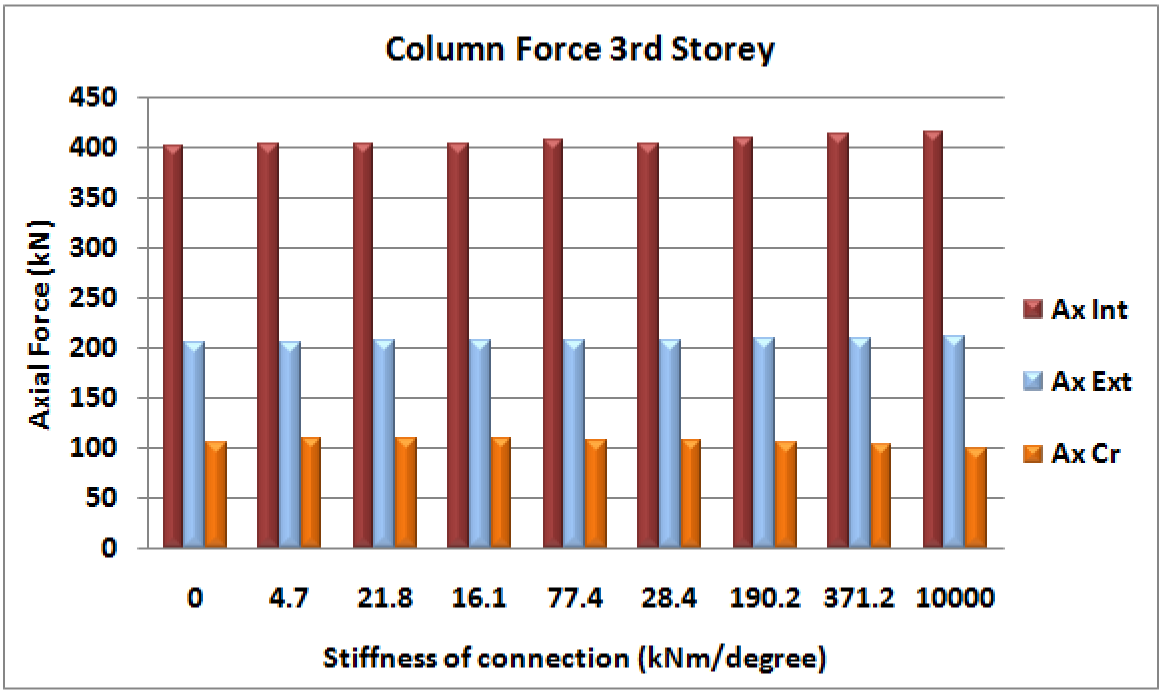
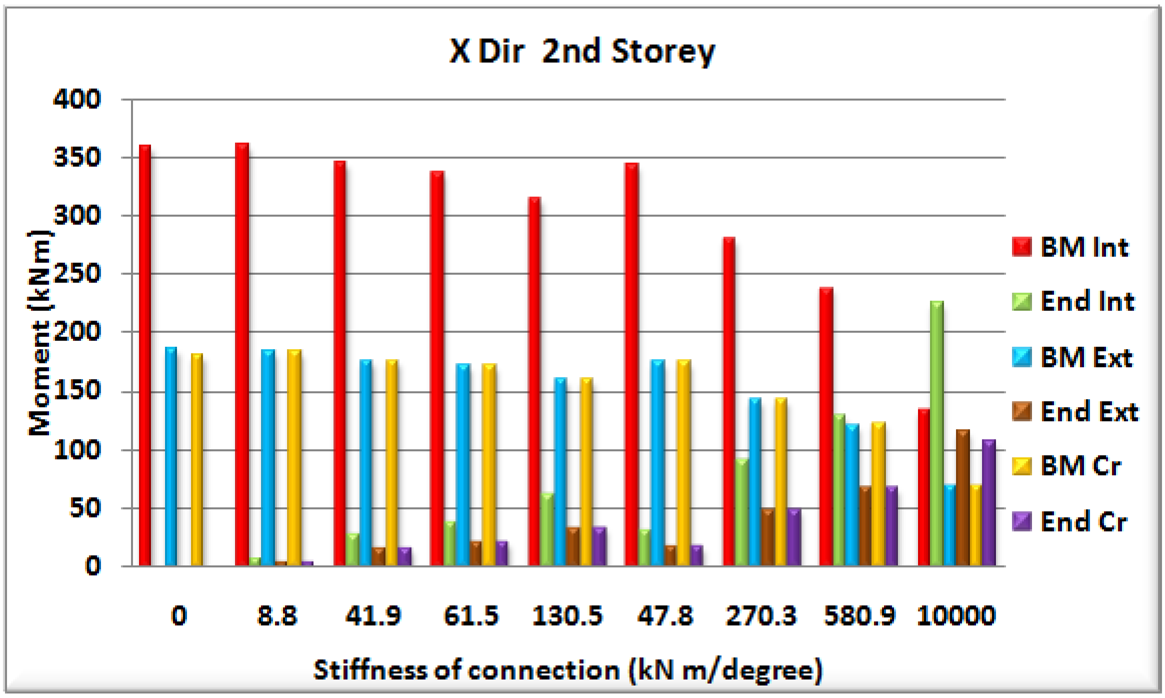
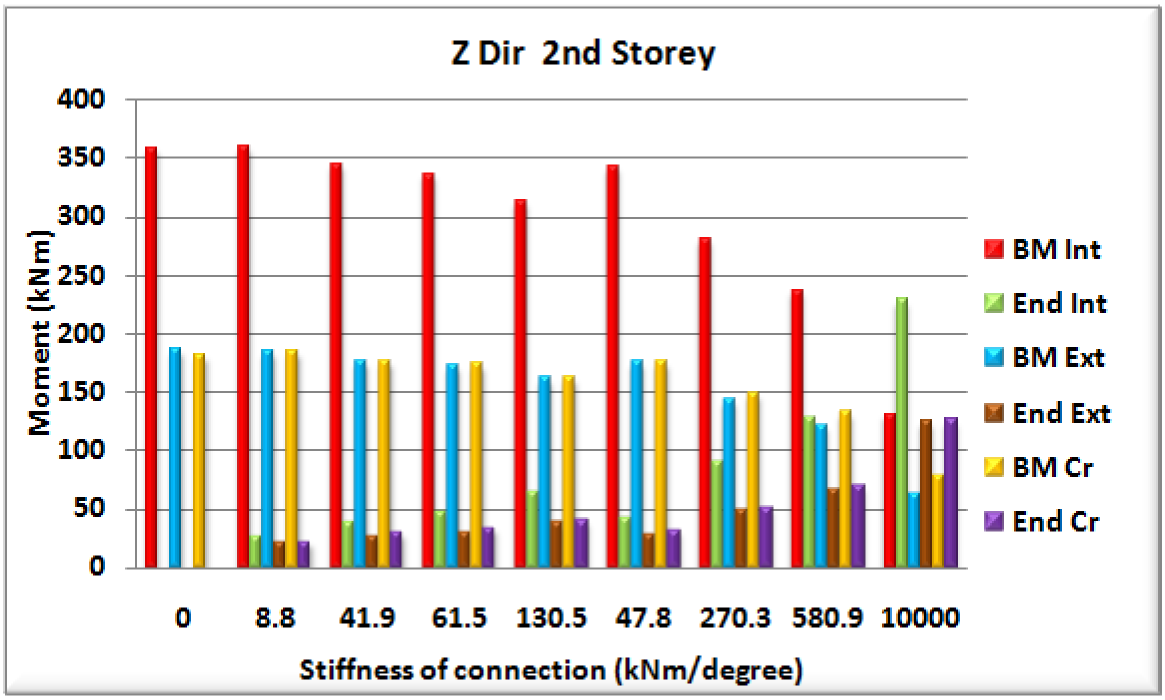


Figure C.7: fig:Effect on Axial Force of Column Due to Connections of 3^{rd} Storey 7 m Span

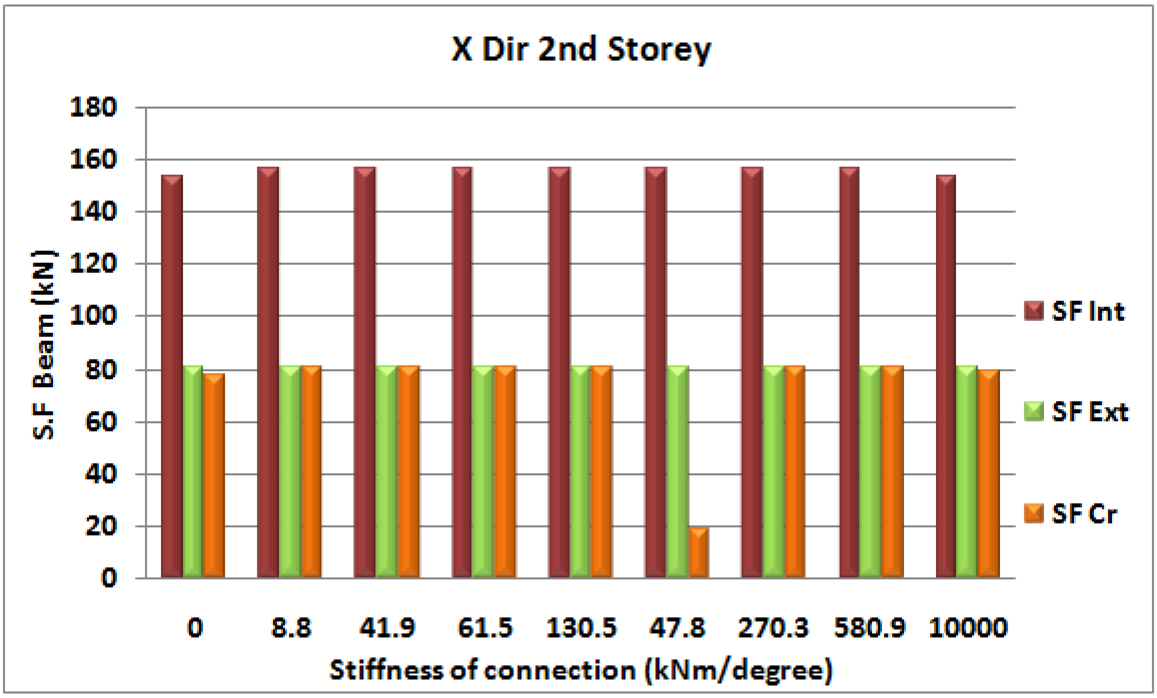


(a) Reduction in Moment of X Dir Beams

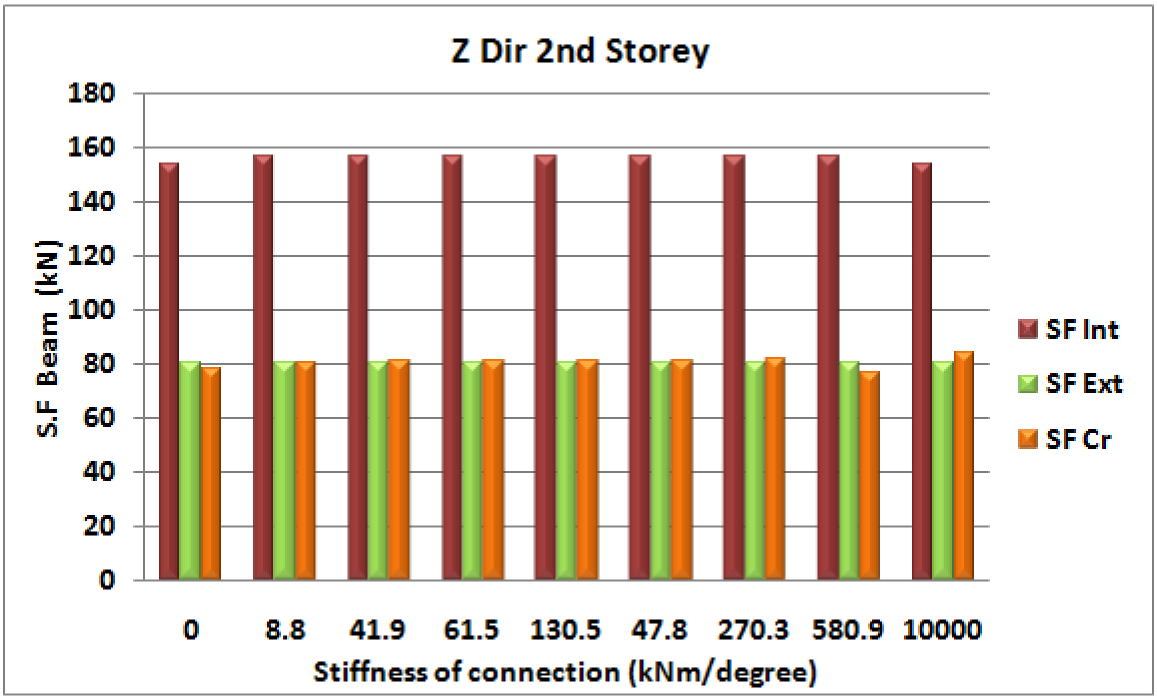


(b) Reduction in Moment of Z Dir Beams

Figure C.8: Reduction in Moment of Beams Due to Connections of 3^{rd} Storey 7 m Span

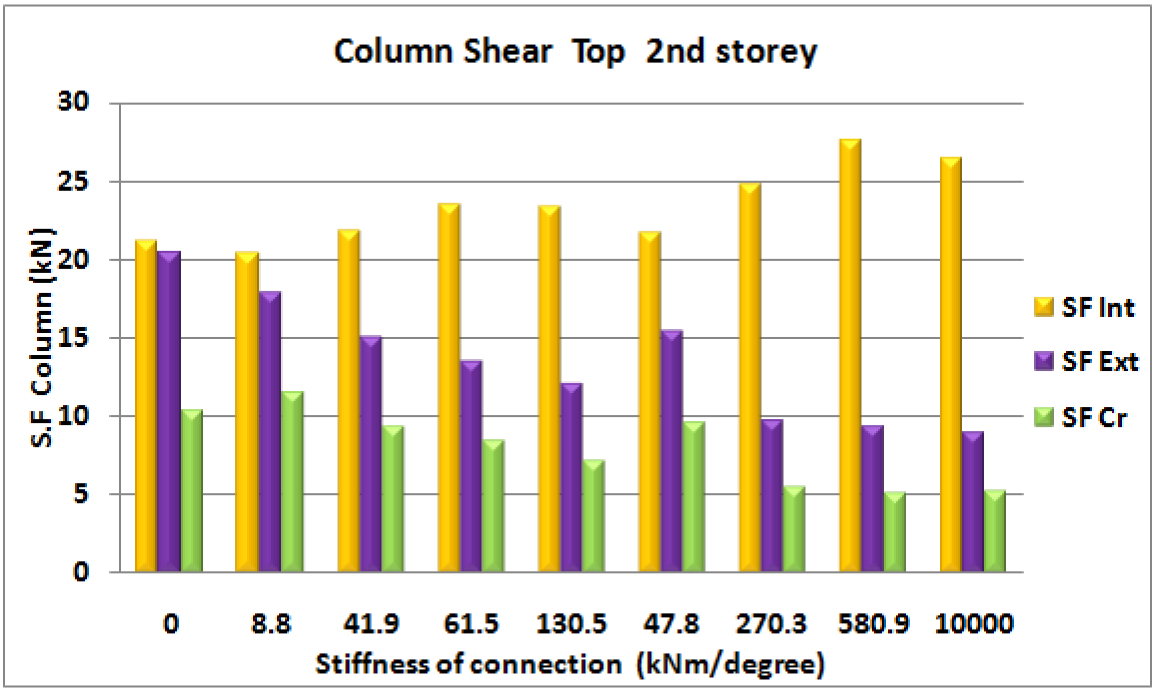


(a) Effect on Shear Force of X Dir Beams

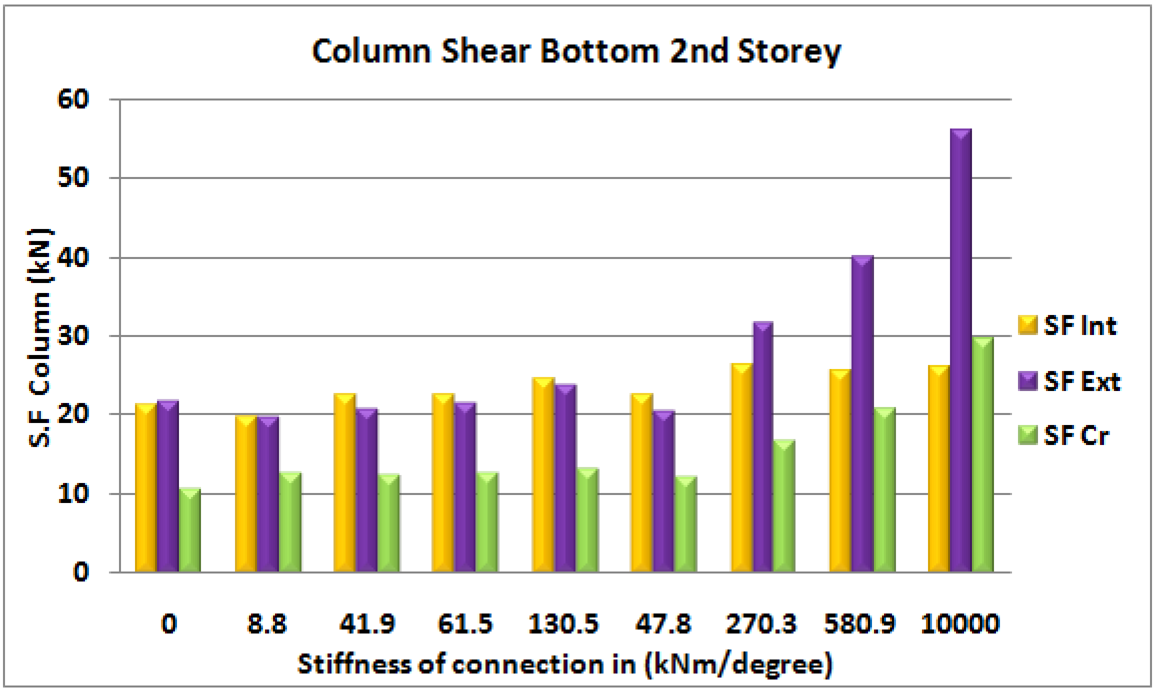


(b) Effect on Shear Force of Z Dir Beams

Figure C.9: Effect og Shear Force on Beams Due to Connections on 2^{nd} Storey 7 m Span



(a) Effect of Shear Force on Top of Column



(b) Effect of Shear Force on Bottom of Column

Figure C.10: Effect of Shear Force on Column Due to Connections of 2^{nd} Storey 7 m Span

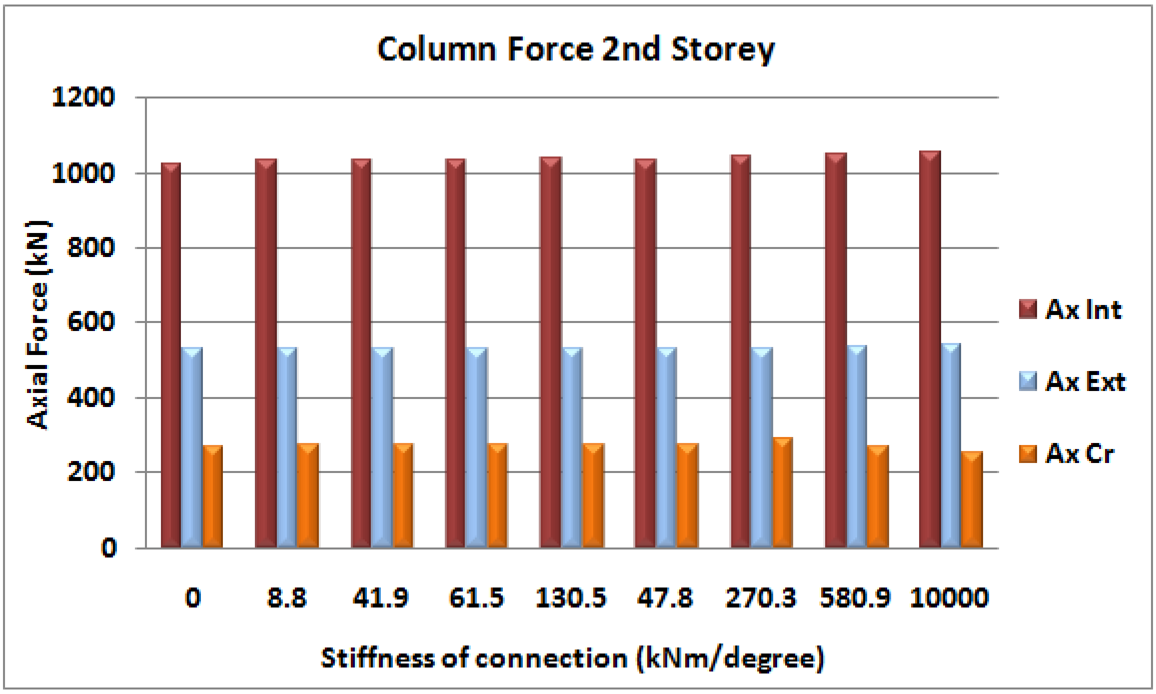
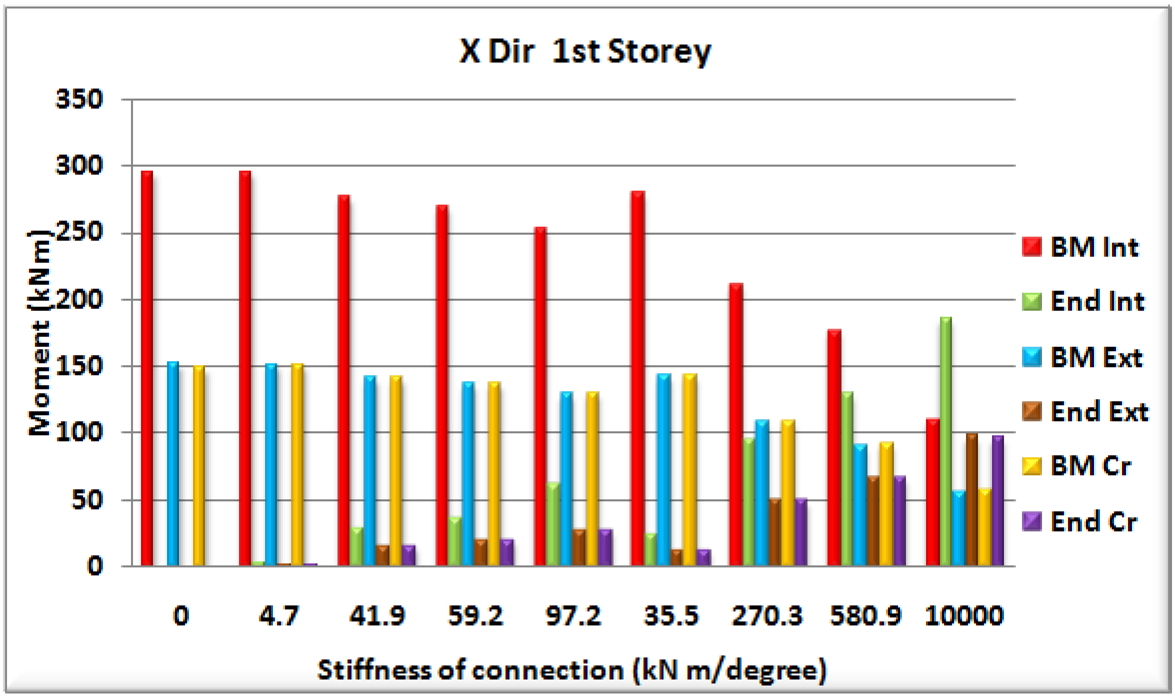
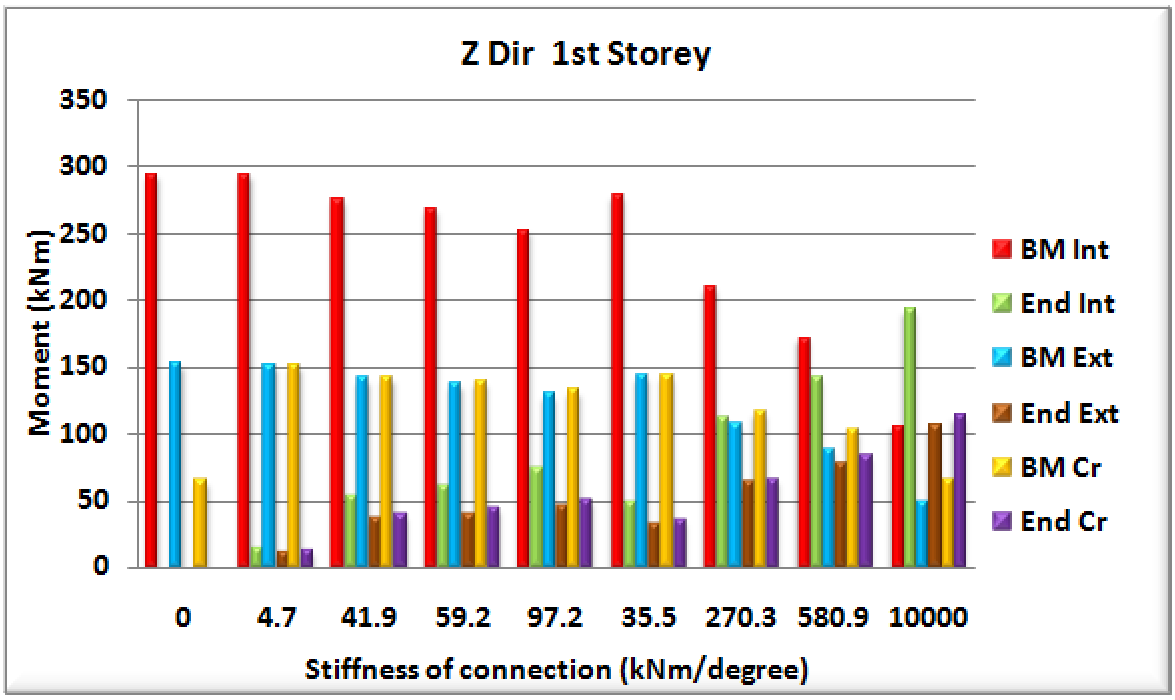


Figure C.11: fig:Effect on Axial Force of Column Due to Connections of 2^{nd} Storey 7 m Span

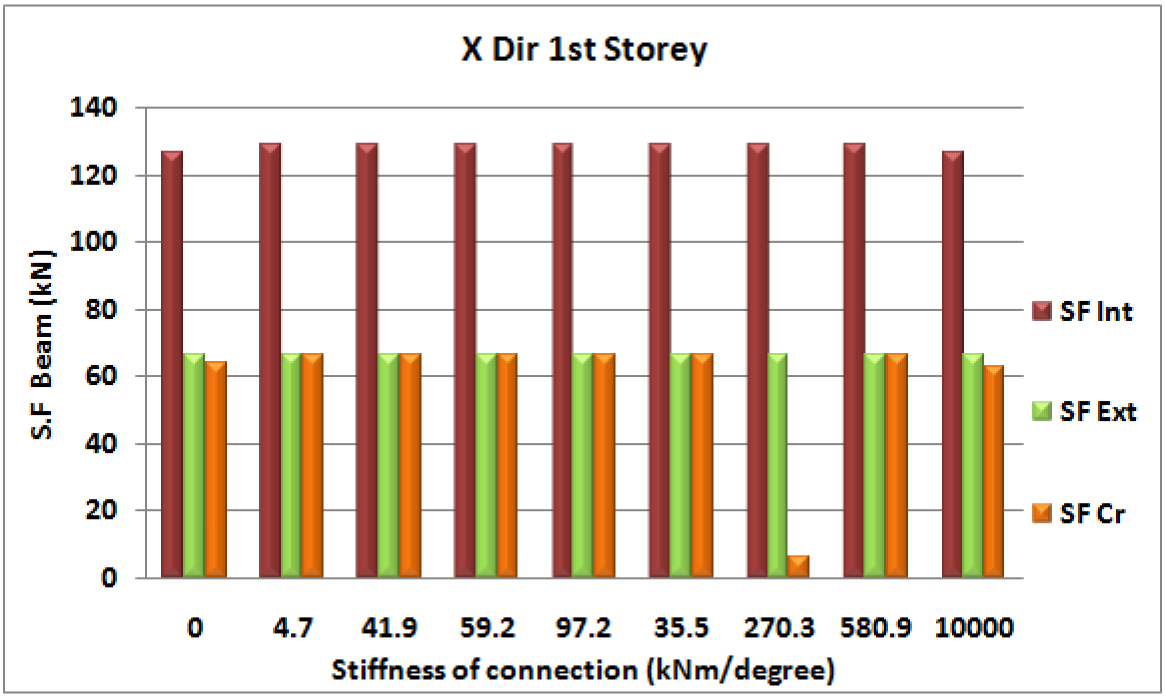


(a) Reduction in Moment of X Dir Beams

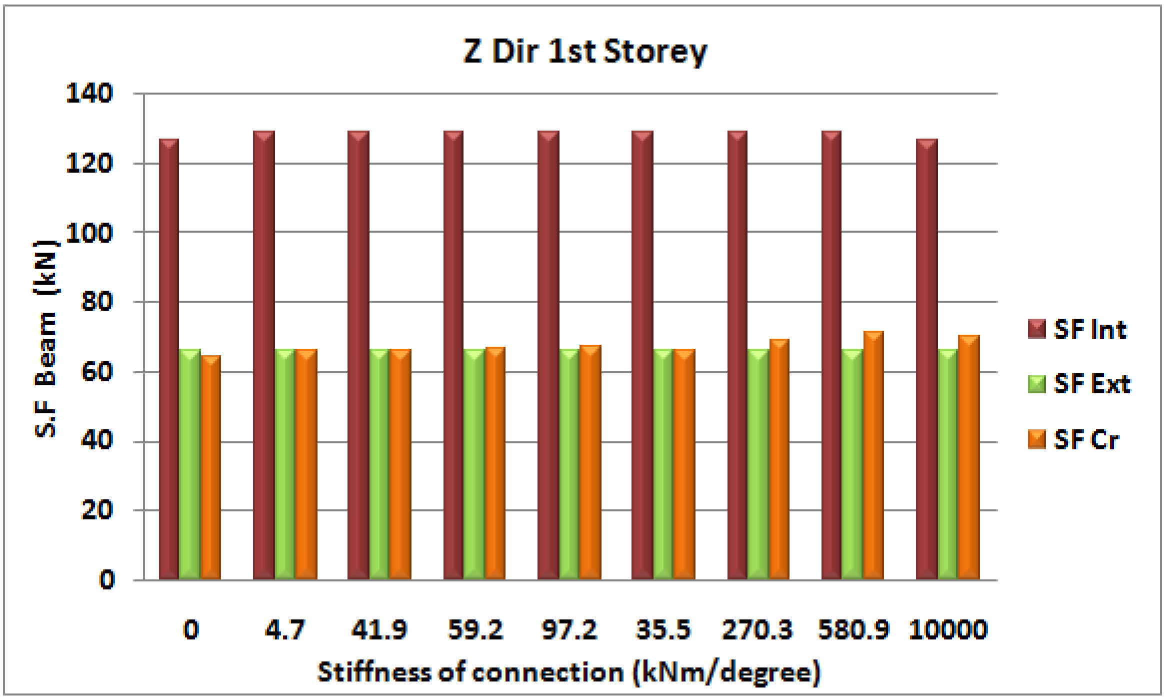


(b) Reduction in Moment of Z Dir Beams

Figure C.12: Reduction in Moment of Beams Due to Connections of 1st Storey 7 m Span

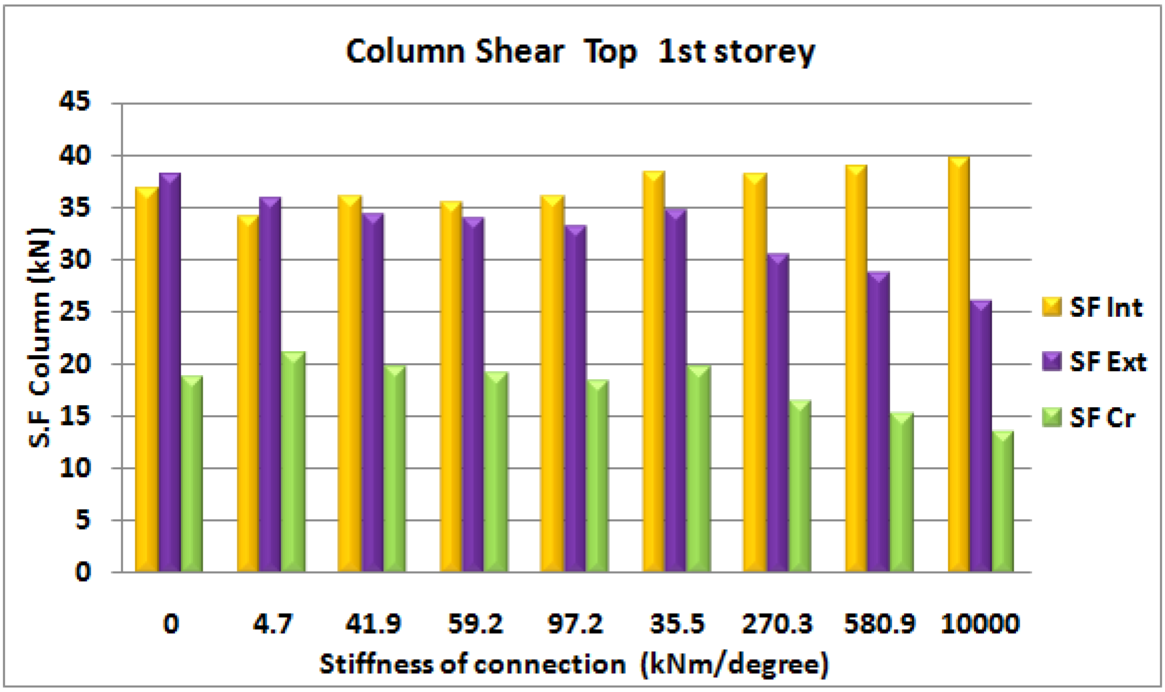


(a) Effect on Shear Force of X Dir Beams

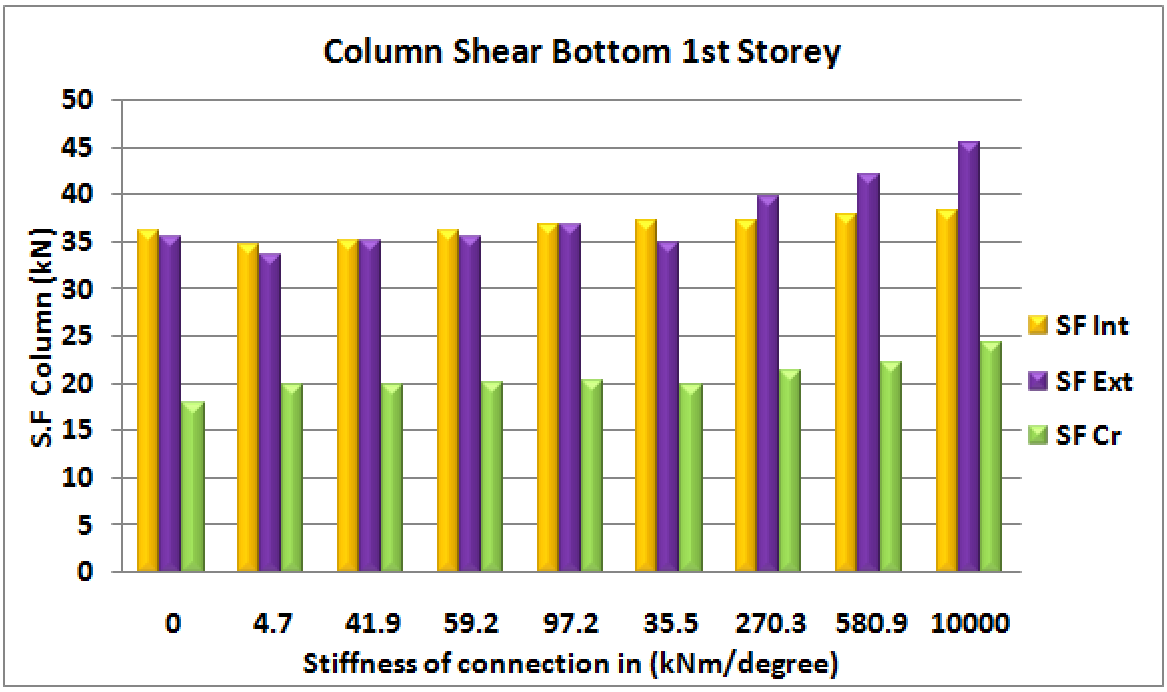


(b) Effect on Shear Force of Z Dir Beams

Figure C.13: Effect og Shear Force on Beams Due to Connections on 1^{st} Storey 7 m Span



(a) Effect of Shear Force on Top of Column



(b) Effect of Shear Force on Bottom of Column

Figure C.14: Effect of Shear Force on Column Due to Connections of 1st Storey 7 m Span

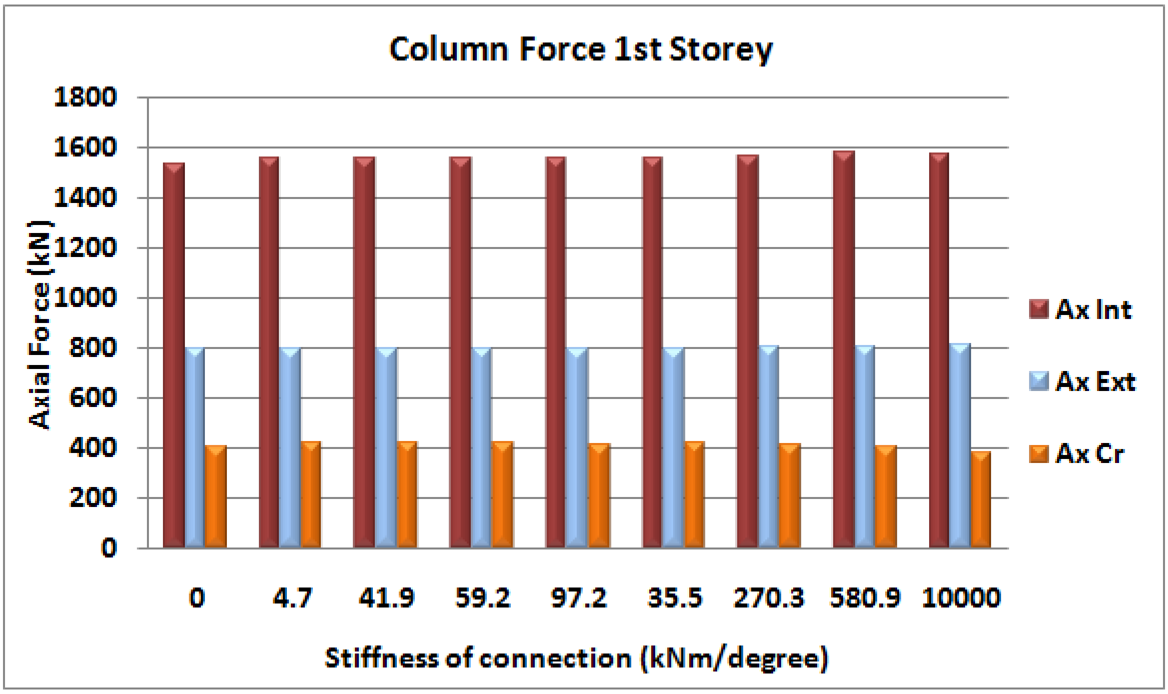
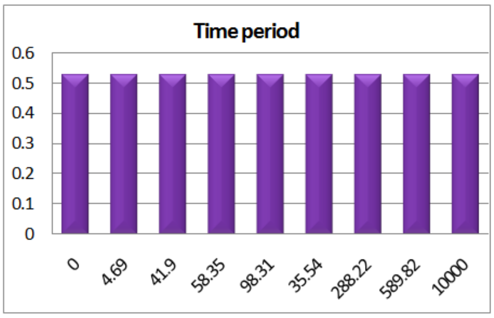
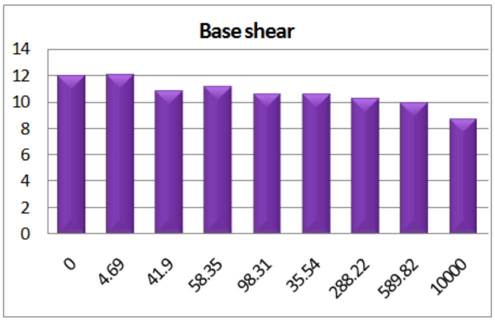


Figure C.15: fig:Effect on Axial Force of Column Due to Connections of 1^{st} Storey 7 m Span

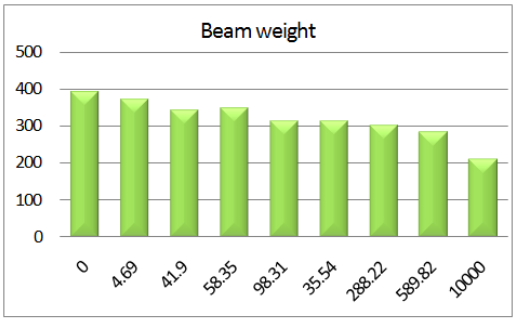


(a) Effect on Time Period

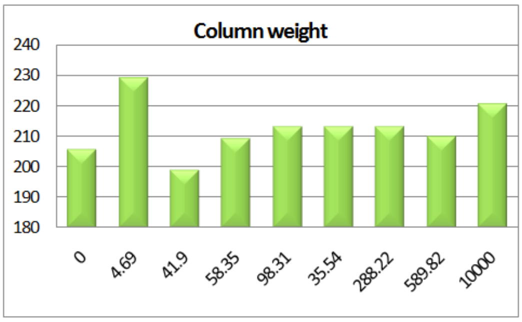


(b) Effect on Base Shear

Figure C.16: Effect on Time Period and Base Shear Due to Connections on 3^{rd} storey 7 m Span

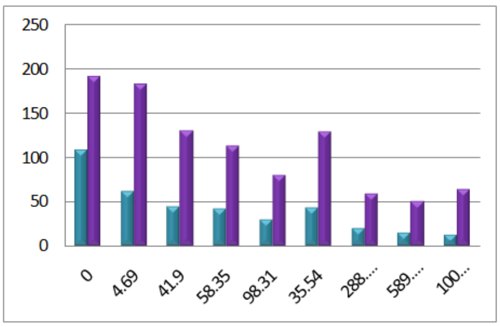


(a) Reduction in Beam Weight

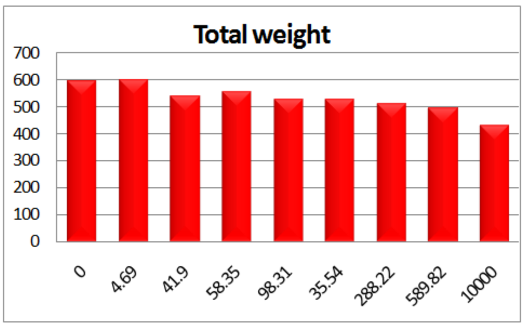


(b) Reduction in Column Weight

Figure C.17: Reduction in Beam Weight and Column Weight Due to Connections on 3^{rd} storey 7 m Span



(a) fig:Effect on Displacement



(b) Reduction in Total Weight

Figure C.18: fig:Reduction in Total Weight Due to connections of 3^{rd} Storey 7 m Span

C.2 Results for G+3 With 9m Span

C.2.1 Design of Connections

Single Web Angle Connection

Table C.12: Single Web Angle Connection for 9 m Span

Storey	3^{rd}	2^{nd}	1^{st}
Beam Section	ISMB=550	ISMB=600	ISMB=600
Column Section	ISWB=550	2ISMC=250 c/c=300 Plate width=400 TK=16	2ISMC=400 c/c=600 Plate width=600 TK=16
Guage Distance	30.6	35	35
Angle Size	80X80X6	80X80X6	80X80X6
Edge Distance	40	40	40
Pitch	60	60	60
Bolt Dia	20	20	20
No. of Bolts	4	6	6
Length	260	380	380

Double Web Angle Connection

Table C.13: Double web Angle Connection for Storey 9 m Span

Storey	3^{rd}	2^{nd}	1^{st}
Beam Section	ISMB=550	ISMB=600	ISMB=600
Column Section	ISWB=550	2ISMC=250 c/c=300 Plate width=400 TK=16	2ISMC=400 c/c=600 Plate width=600 TK=16
Guage Distance	111.2	130	130
Angle Size	90X90X6	90X90X6	90X90X6
Edge Distance	60	60	60
Pitch	70	70	70
Bolt Dia	20	20	20
No. of Bolts	4	6	6
Length	290	430	430

Header Plate Connection

Table C.14: Header Plate Connection for Storey 9 m Span

Storey	3^{rd}	2^{nd}	1^{st}
Beam Section	ISMB=550	ISMB=600	ISMB=600
Column Section	ISWB=550	2ISMC=250,c/c=300 Plate width=400,TK=16	2ISMC=400,c/c=600 Plate width=600,TK=16
End Plate Size	200X200X8	260X260X10	260X260X10
Edge Distance	40	40	40
Pitch	60	60	60
Bolt Dia	22	22	22
No. of Bolts	3	4	4
Welding Length	184	240	240

Top and Bottom Seat Angle with Web Angle

Table C.15: Top and Bottom Seat Angle with Web Angle for Storey 9 m Span

Storey	3^{rd}	2^{nd}	1^{st}
Beam Section	ISMB=550	ISMB=600	ISMB=600
Column Section	ISWB=550	2ISMC=250,c/c=300 Plate width=400,TK=16	2ISMC=400,c/c=600 Plate width=600,TK=16
Angle Size	90X90X6	150X150X6	130X130X6
Stiffener Angle	2 Nos. 80X80X6	2 Nos. 150X150X6	2 Nos. 130X130X6
Edge Distance	45	45	45
Pitch	70	70	70
Bolt Dia	20	20	20
No. of Bolts	4	4	4
No. of Rows	2	4	4
Welding Length	300	300	300

Top and Bottom Seat Angle without Web Angle

Table C.16: Top and Bottom Seat Angle without Web Angle for Storey 9 m Span

Storey	3^{rd}	2^{nd}	1^{st}
Beam Section	ISMB=550	ISMB=600	ISMB=600
Column Section	ISWB=550	2ISMC=250,c/c=300 Plate width=400,TK=16	2ISMC=400,c/c=600 Plate width=600,TK=16
Seat Angle	150X150X12	150X150X12	150X150X12
Edge Distance	40	40	40
Pitch	60	60	60
Bolt Dia	20	20	20
No. of Bolts	4	6	6
Length	125	140	140

End Plate with Stiffener Connection

Table C.17: End Plate with Stiffener Connection for Storey 9 m Span

Storey	3^{rd}	2^{nd}	1^{st}
Beam Section	ISMB=500	ISMB=550	ISMB=450
Column Section	ISWB=600	2ISMC=200,c/c =300	2ISMC=300,c/c =400
		Plate Width=350,Tk=16	Plate Width=400,Tk=16
Plate Size	300X190	300X240	300X200
Thickness	14	20	24
Bolt Dia	30	30	30
No. of Bolts	6	6	6
Weld Size	14	14	20
Length	250	500	400
Web Stiffner	80X6	160X20	170X20
Weld Size	6	16	16

T-Stub Connection

Table C.18: T-Stub Connection for Storey 9 m Span

Storey	3^{rd}	2^{nd}	1^{st}
Beam Section	ISMB=500	ISMB=550	ISMB=450
Column Section	ISWB=600	2ISMC=200,c/c =300	2ISMC=300,c/c =400
		Plate Width=350,Tk=16	Plate Width=400 ,Tk=16
Web Angle	90X90X8	90X90X8	90X90X8
T-Stab Cut From	ISMB=500	ISMB=600	ISMB=600
Length	400	400	400
Edge Distance	60	60	60
Pitch	50	50	50
Bolt Dia	22	24	24
No. of Bolts	4	6	6
No. of Rows	2	2	2

Table C.19: Member Sizes for 9 m Span for Various Connections

3^{rd} Storey	0	8.8	42.3	65.1	173	54.1	441.8	1185.5	10000
Beam	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB
Section	550	550	550	500	500	500	500	450	350
Column	ISHB	ISHB	ISWB	ISHB	ISHB	ISWB	ISHB	ISWB	ISWB
Section	350	400	500	400	400	550	400	550	500
2^{nd} Storey	0	21.3	106.7	215.8	211.4	64.1	718.6	1659.6	10000
Beam	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB
Section	600	600	600	600	600	600	550	500	450
Column	ISMC	ISMC	ISMC	ISMC	ISMC	ISMC	ISMC	ISMC	ISMC
Section	300	300	300	250	250	250	250	250	250
C/C between	300	300	300	250	250	250	250	250	250
Plate Widht	300	300	300	250	250	250	250	250	250
Thickness	16	16	16	16	16	16	16	16	16
1^{st} Storey	0	21.3	106.7	215.8	211.4	64.1	539.6	1185.5	10000
Beam	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB	ISMB
Section	600	600	550	550	550	550	500	500	400
Column	ISMC	ISMC	ISMC	ISMC	ISMC	ISMC	ISMC	ISMC	ISMC
Section	400	400	350	350	300	300	300	300	300
C/C between	600	500	500	500	450	550	500	500	500
Plate Widht	600	500	500	500	450	500	500	500	450
Thickness	16	16	16	16	16	16	16	16	16

Table C.20: Values of k for Connections for 3^{rd} Storey for 9 m Span

		3^{rd} Storey	
Sr No	Connection	kN/radian	kN/degree
1	Single web angle	501.6	8.76
2	Double web angle	2425.4	42.34
3	Header plate	3727.645	65.07
4	Top and bottom without web angle	9909.792	172.98
5	Top and bottom with web angle	3101.1	54.13
6	End plate connection	25309.83	441.78
7	T stub connection	67914.35	1185.45

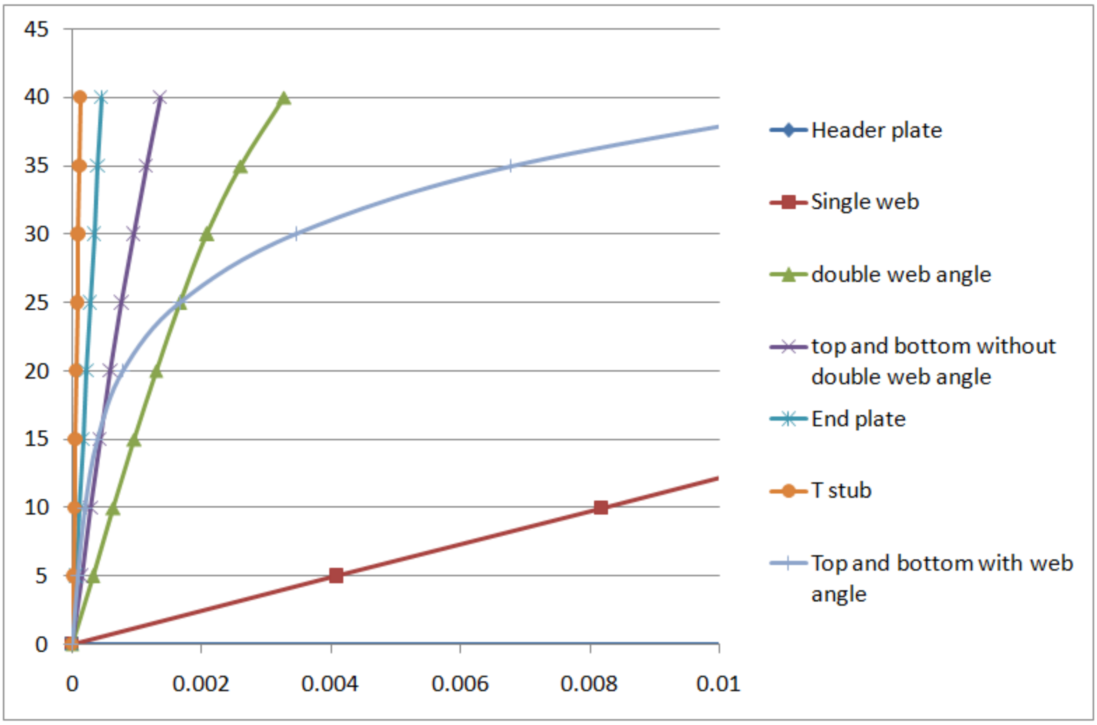


Table C.21: Values of k for Connections for 2^{nd} Storey for 9 m Span

		2^{nd} Storey	
Sr No	Connection	kN/radian	kN/degree
1	Single web angle	1222.61	21.34
2	Double web angle	6078.32	106.10
3	Header plate	12362.34	215.79
4	Top and bottom without web angle	12109.29	211.37
5	Top and bottom with web angle	3671.276	64.08
6	End plate connection	41168.9	718.61
7	T stub connection	95077.03	1659.57

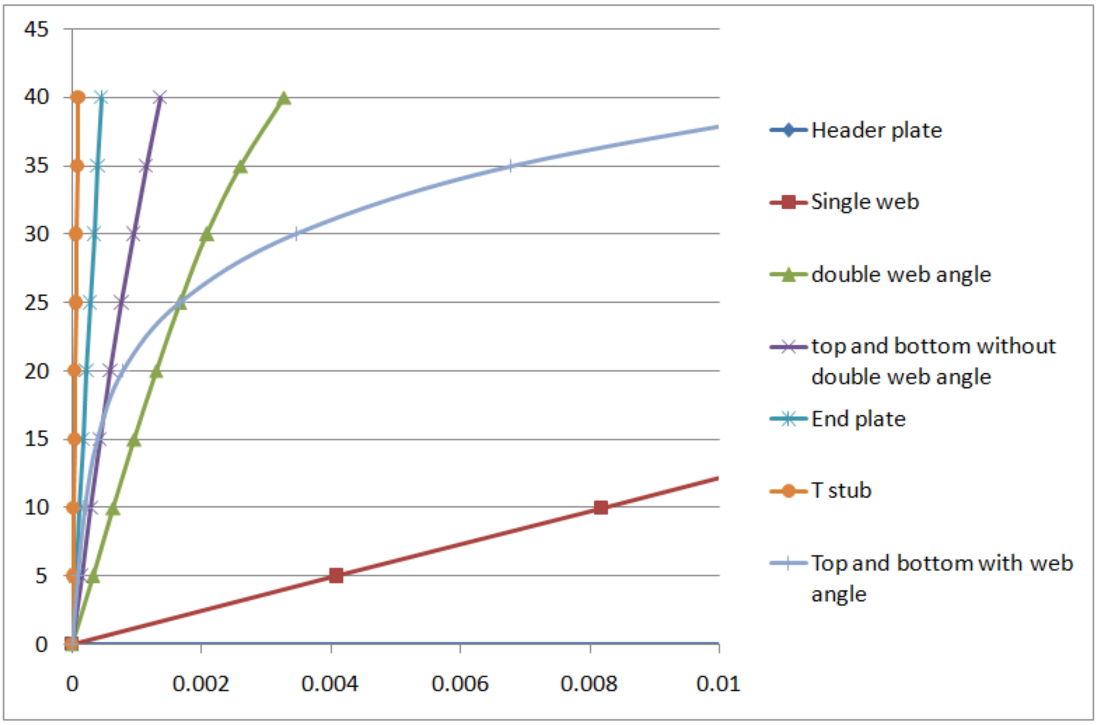
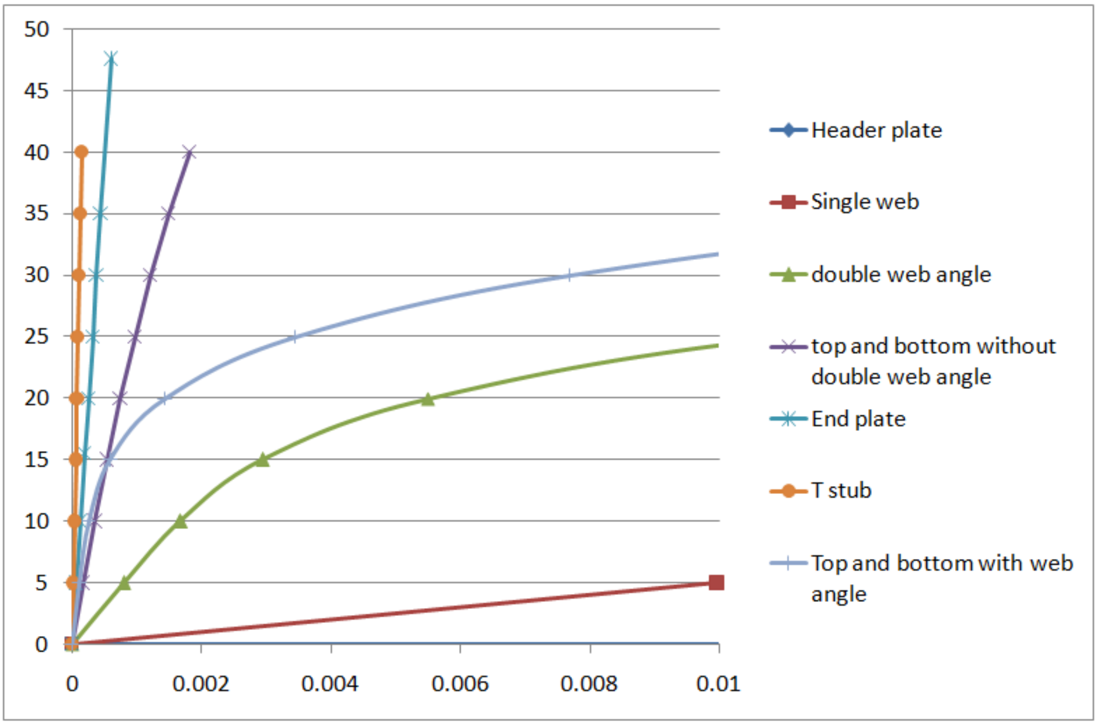
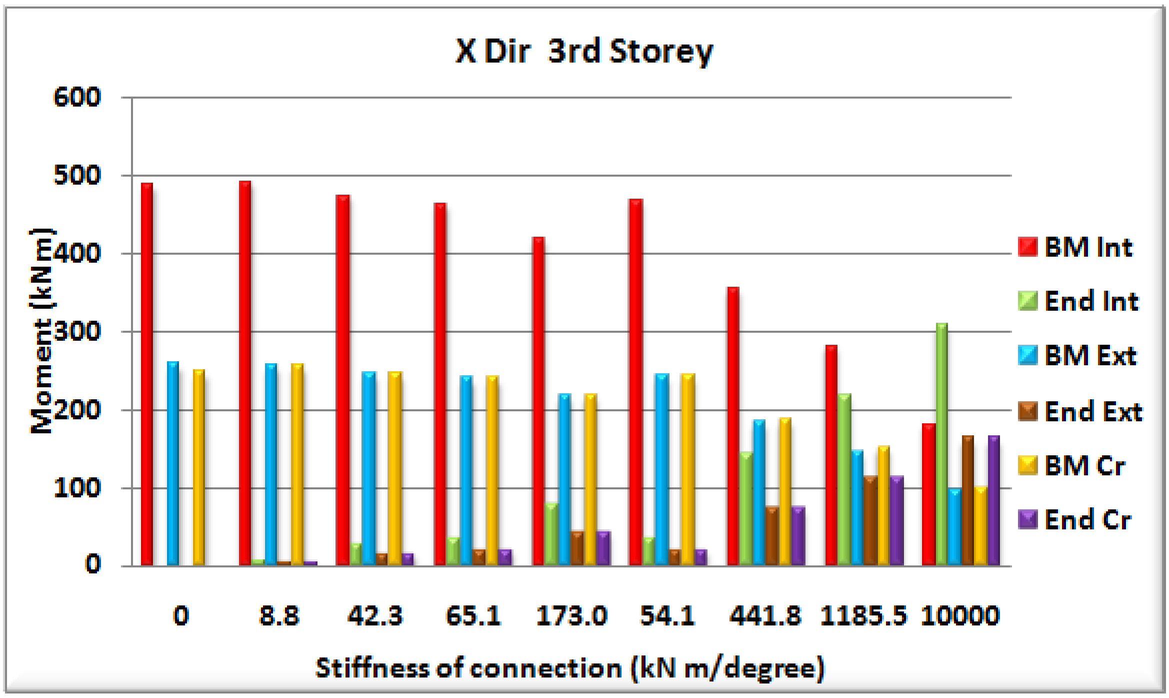


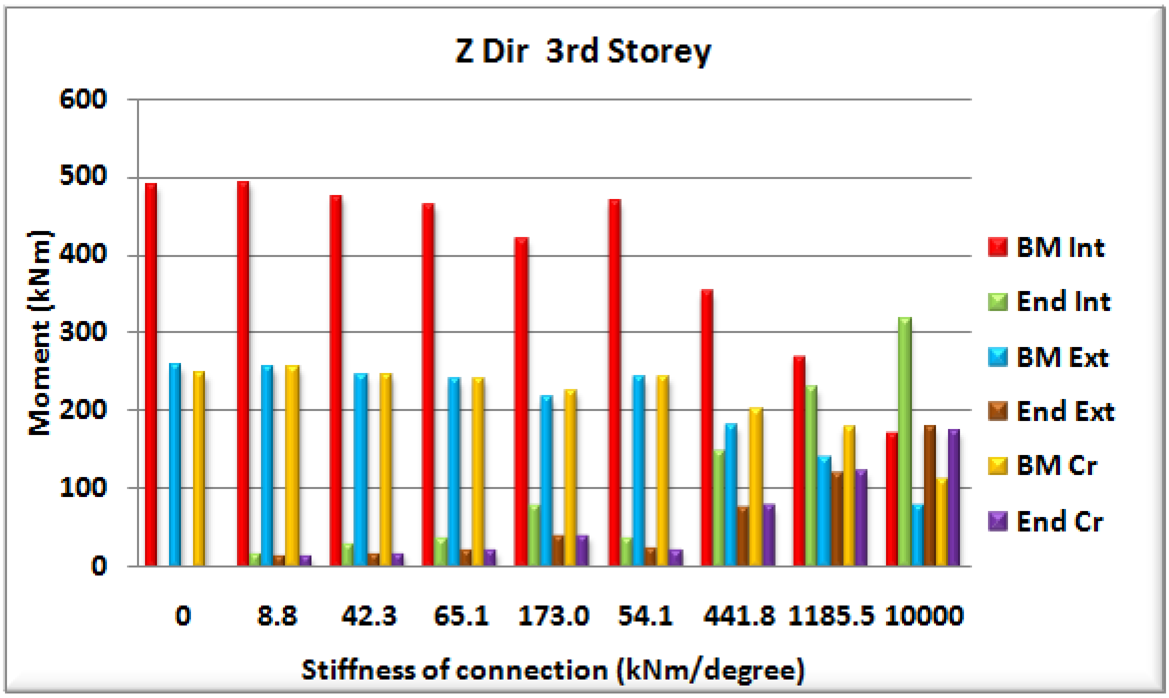
Table C.22: Values of k for Connections for 1^{st} Storey for 9 m Span

		1^{st} Storey	
Sr No	Connection	kN/radian	kN/degree
1	Single web angle	1222.618	21.34
2	Double web angle	6078.32	106.10
3	Header plate	12362.34	215.79
4	Top and bottom without web angle	12109.29	211.37
5	Top and bottom with web angle	3671.27	64.08
6	End plate connection	30915.25	539.63
7	T stub connection	67914.35	1185.45



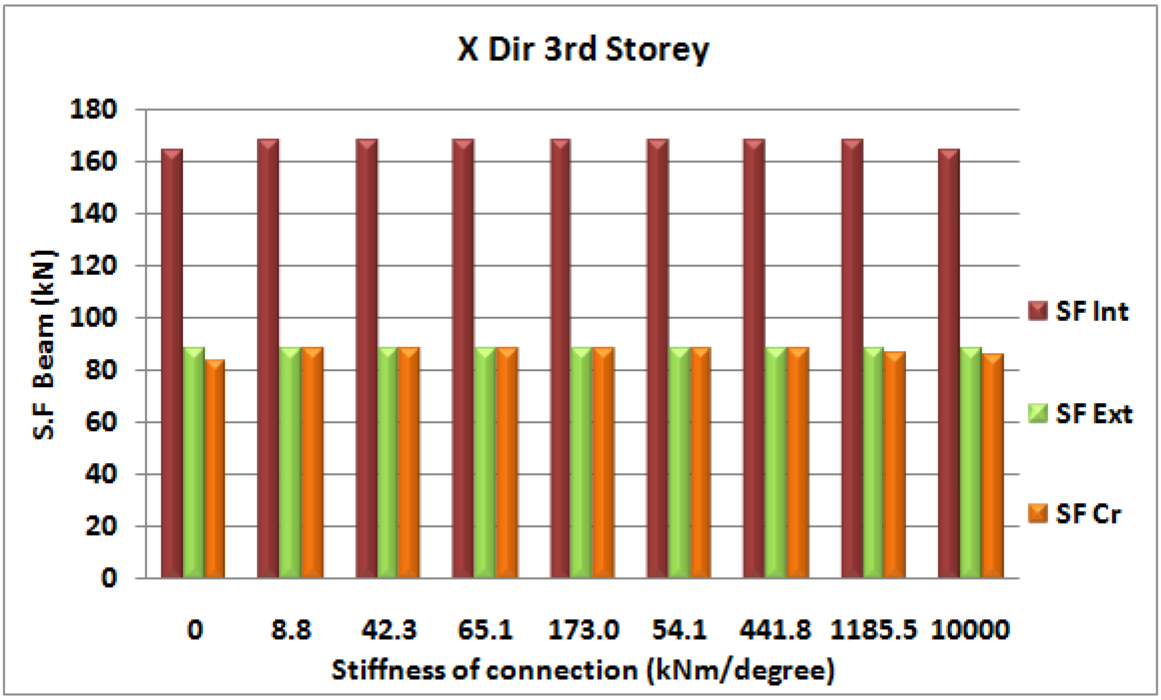


(a) Reduction in Moment of X Dir Beams

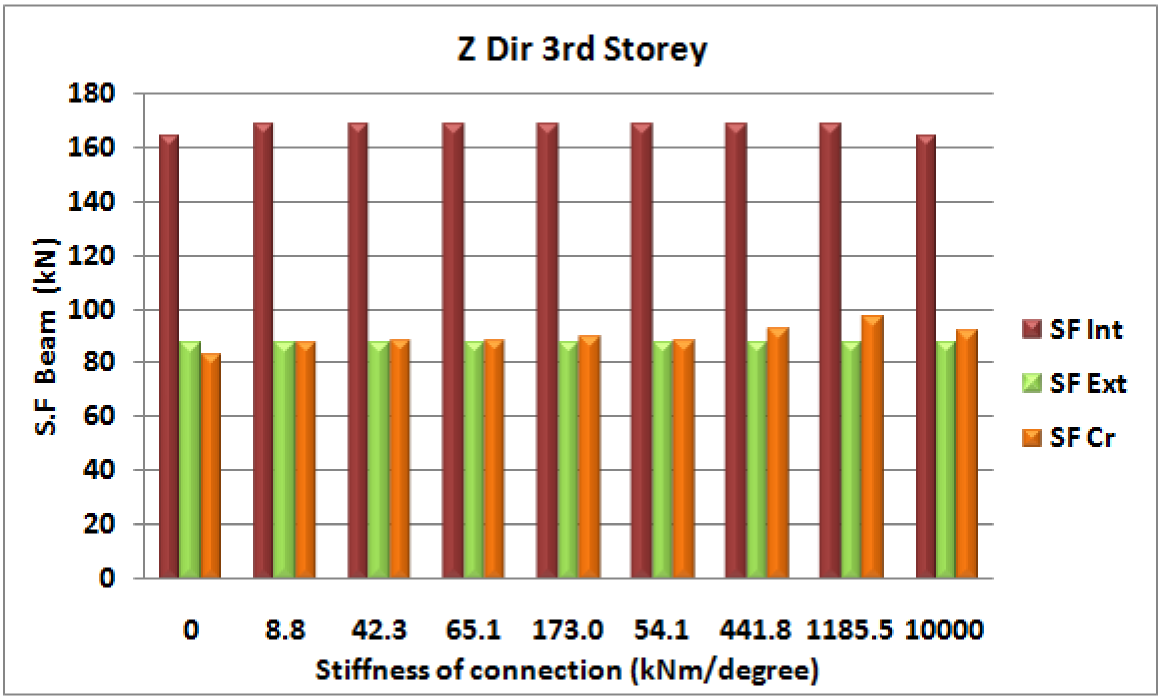


(b) Reduction in Moment of Z Dir Beams

Figure C.19: Reduction in Moment of Beams Due to Connections of 3^{rd} Storey 9 m Span

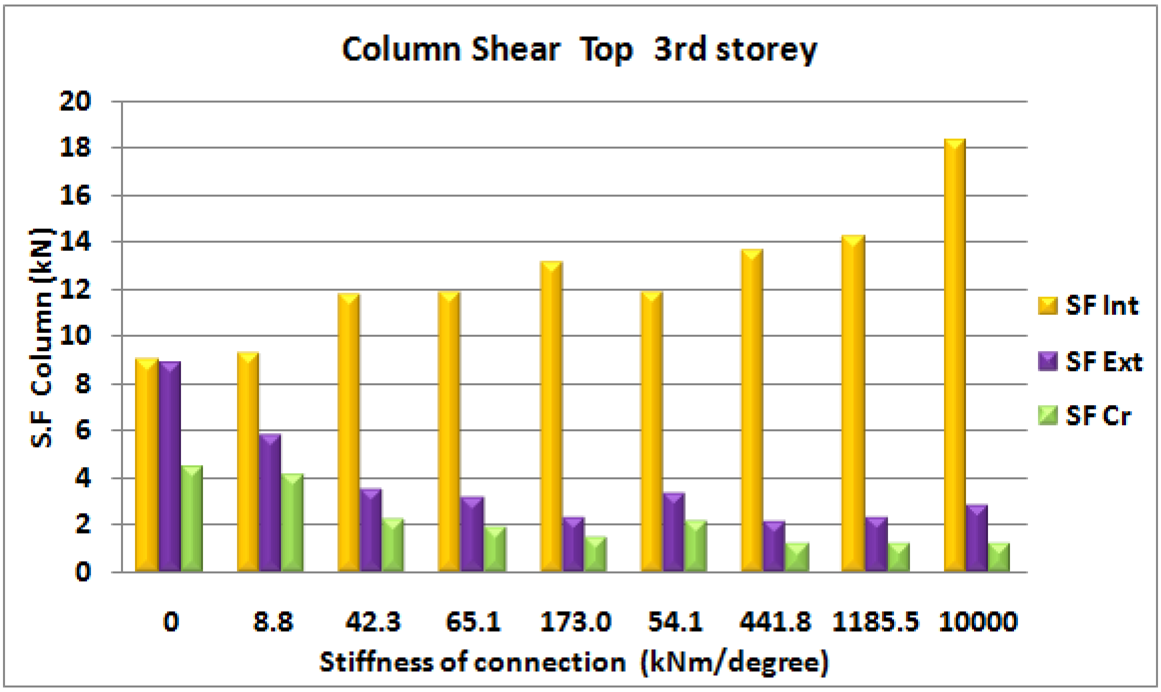


(a) Effect on Shear Force of X Dir Beams

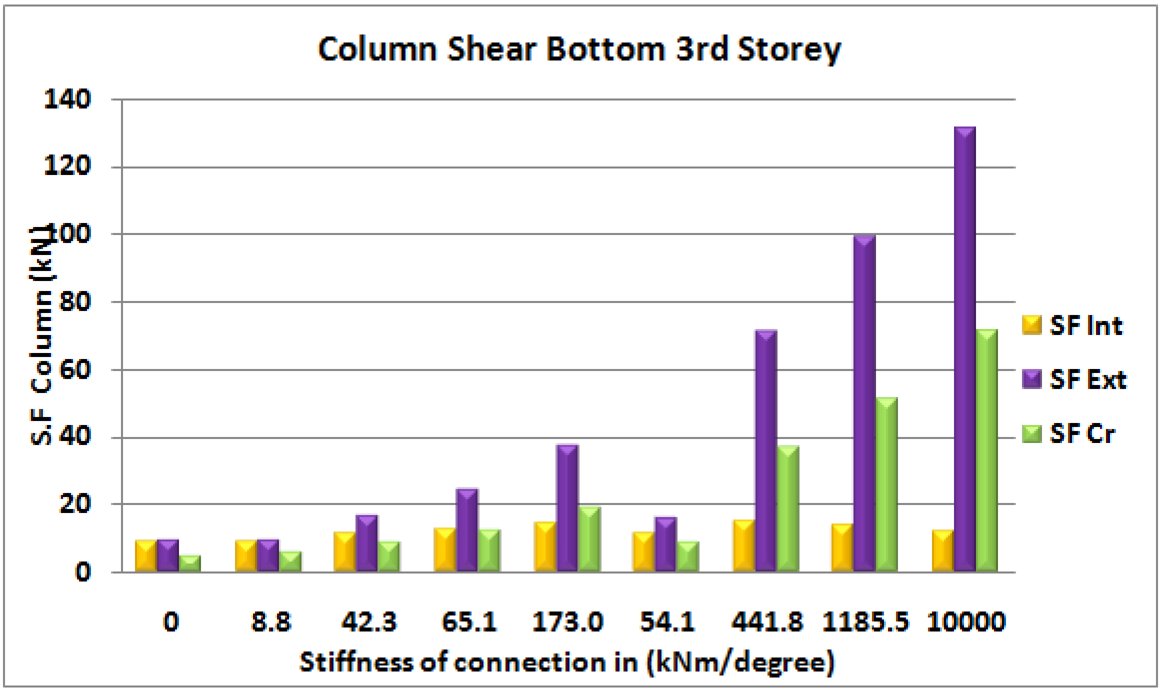


(b) Effect on Shear Force of Z Dir Beams

Figure C.20: Effect og Shear Force on Beams Due to Connections on 3^{rd} Storey 9 m Span



(a) Effect of Shear Force on Top of Column



(b) Effect of Shear Force on Bottom of Column

Figure C.21: Effect of Shear Force on Column Due to Connections of 3^{rd} Storey 9 m Span

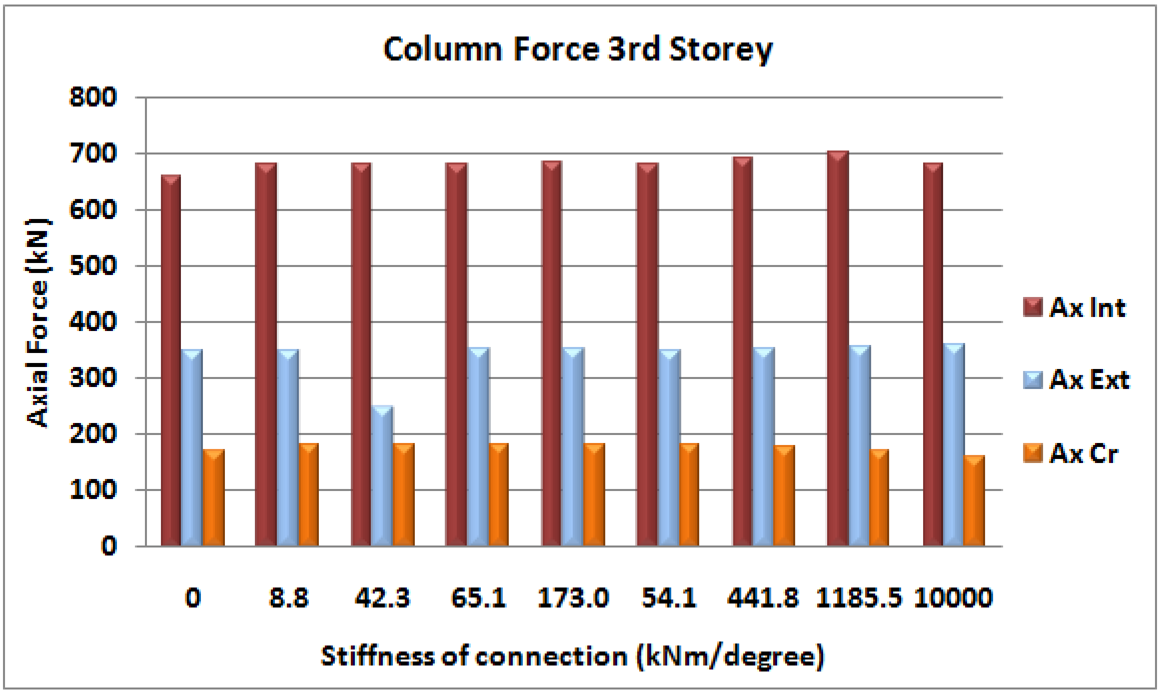
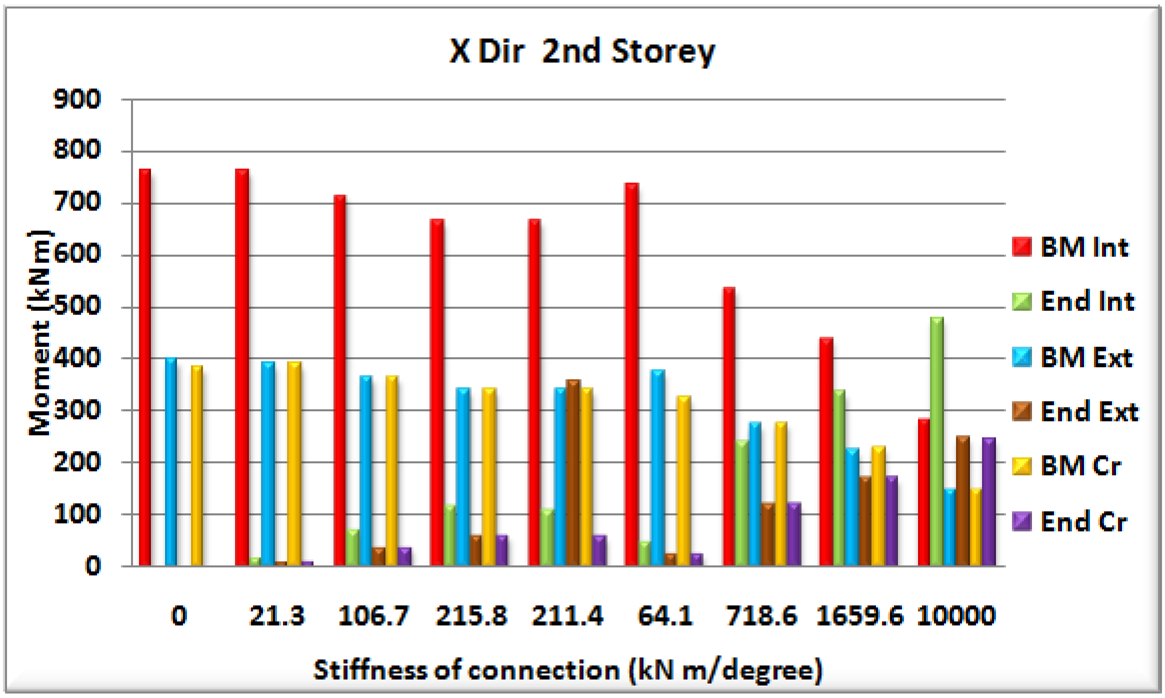
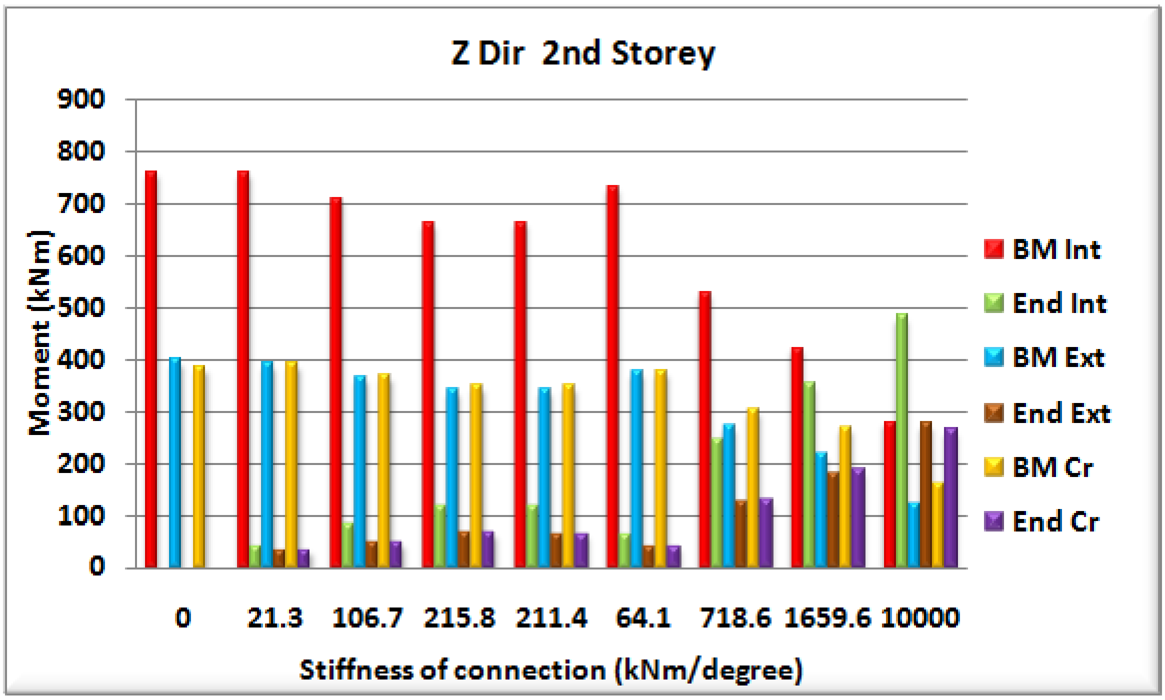


Figure C.22: fig:Effect on Axial Force of Column Due to Connections of 3^{rd} Storey 9 m Span

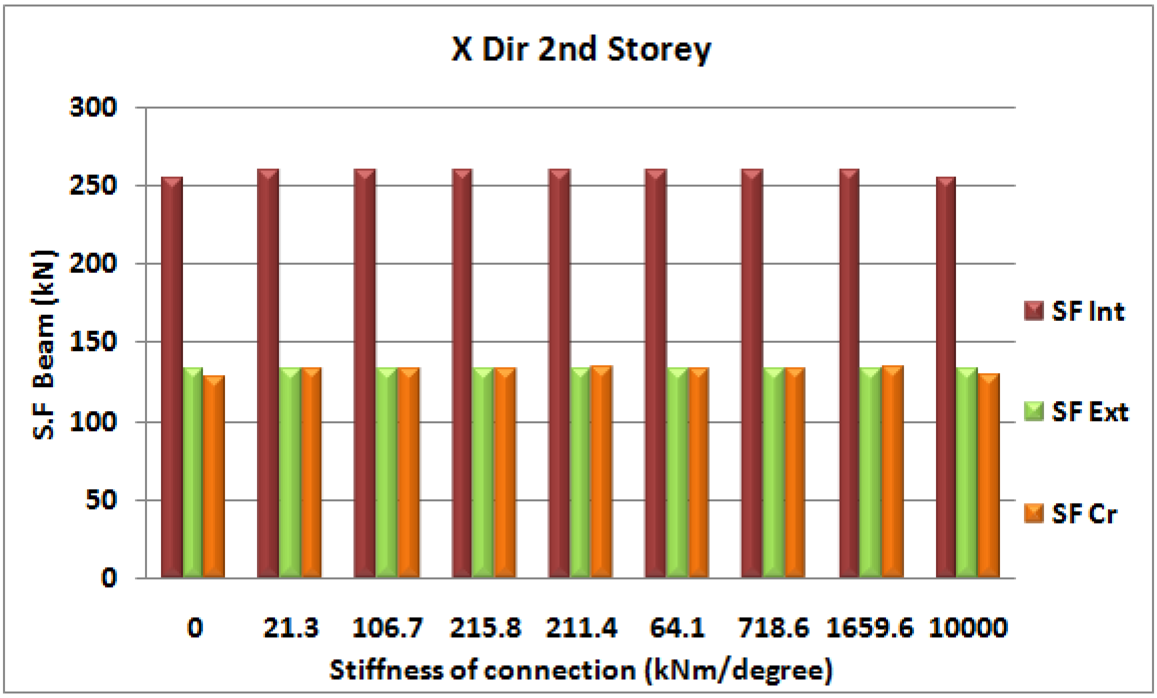


(a) Reduction in Moment of X Dir Beams

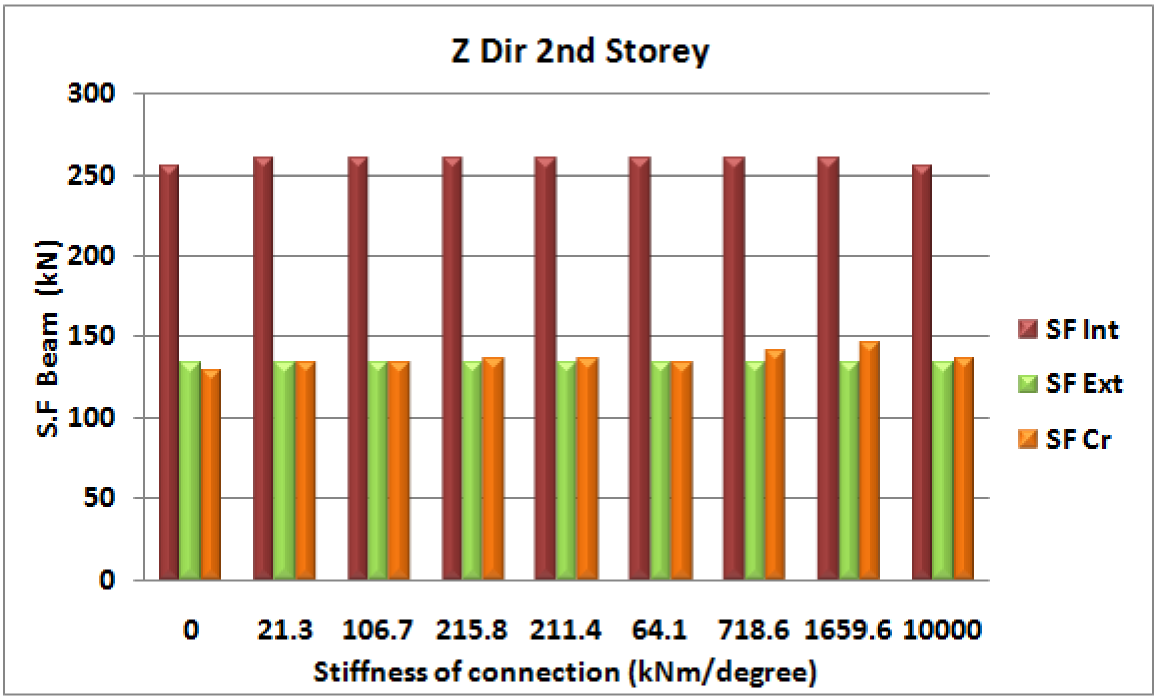


(b) Reduction in Moment of Z Dir Beams

Figure C.23: Reduction in Moment of Beams Due to Connections of 3^{rd} Storey 9 m Span

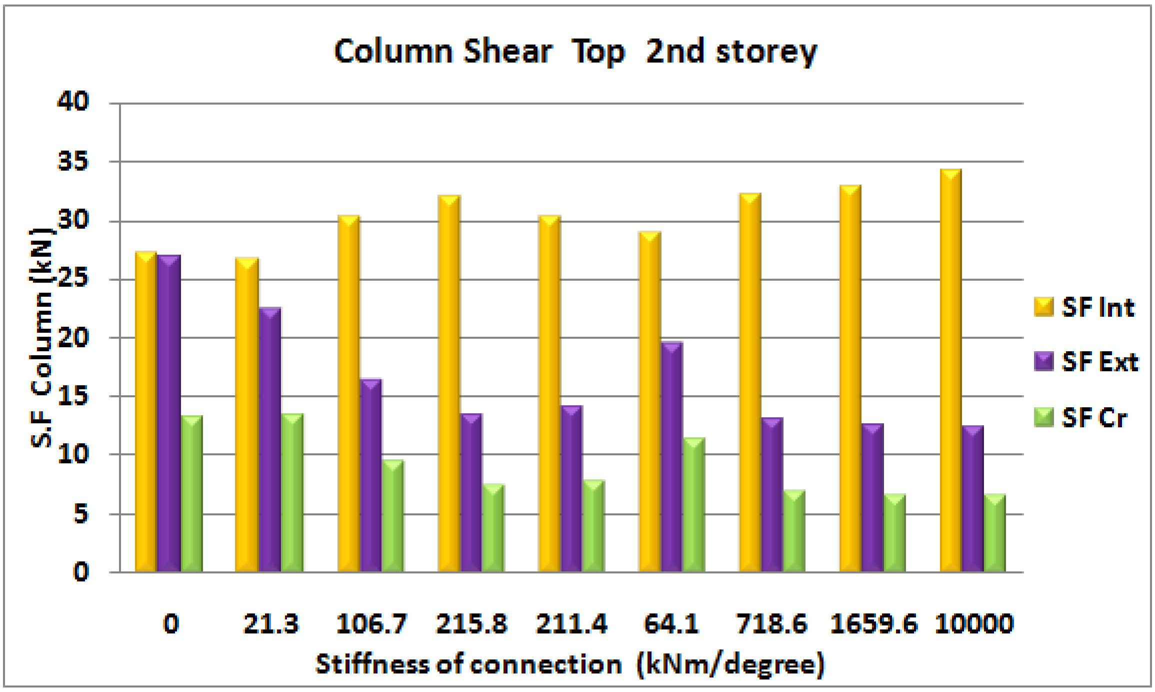


(a) Effect on Shear Force of X Dir Beams

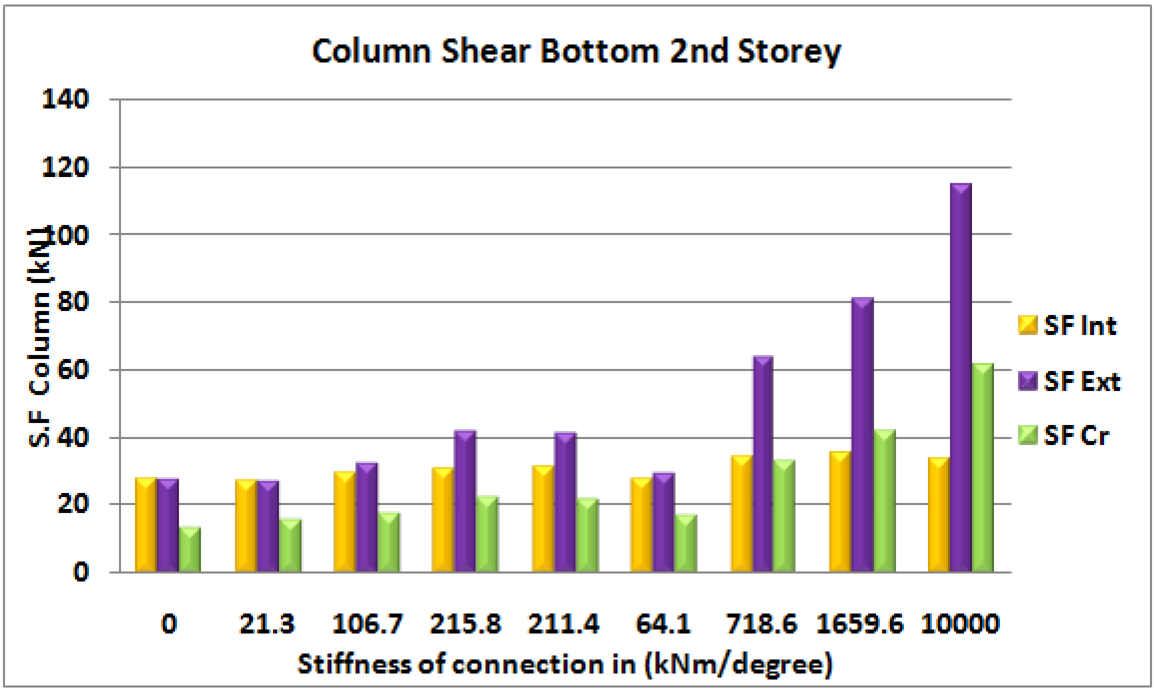


(b) Effect on Shear Force of Z Dir Beams

Figure C.24: Effect og Shear Force on Beams Due to Connections on 2^{nd} Storey 9 m Span



(a) Effect of Shear Force on Top of Column



(b) Effect of Shear Force on Bottom of Column

Figure C.25: Effect of Shear Force on Column Due to Connections of 2^{nd} Storey 9 m Span

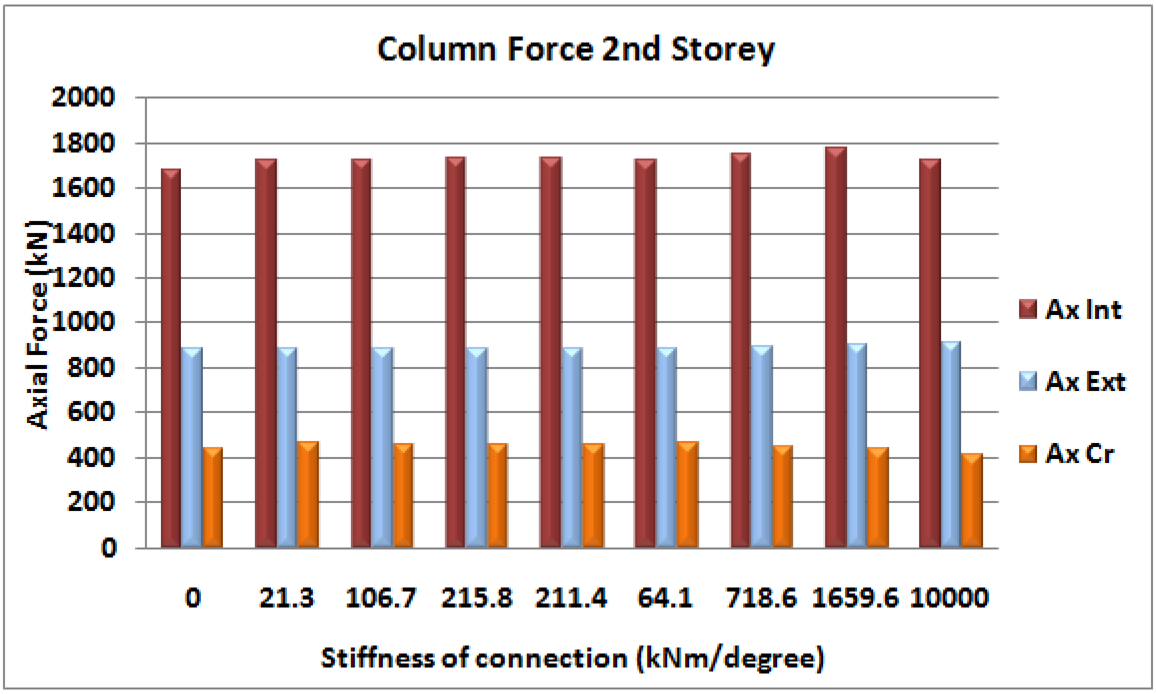
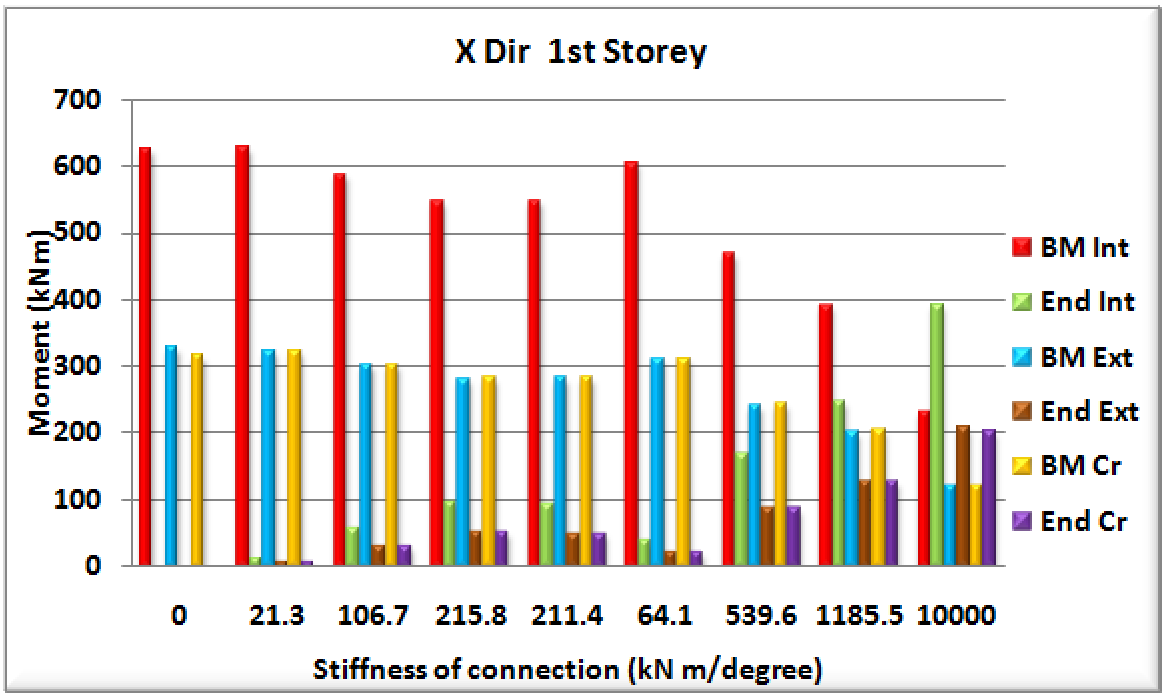
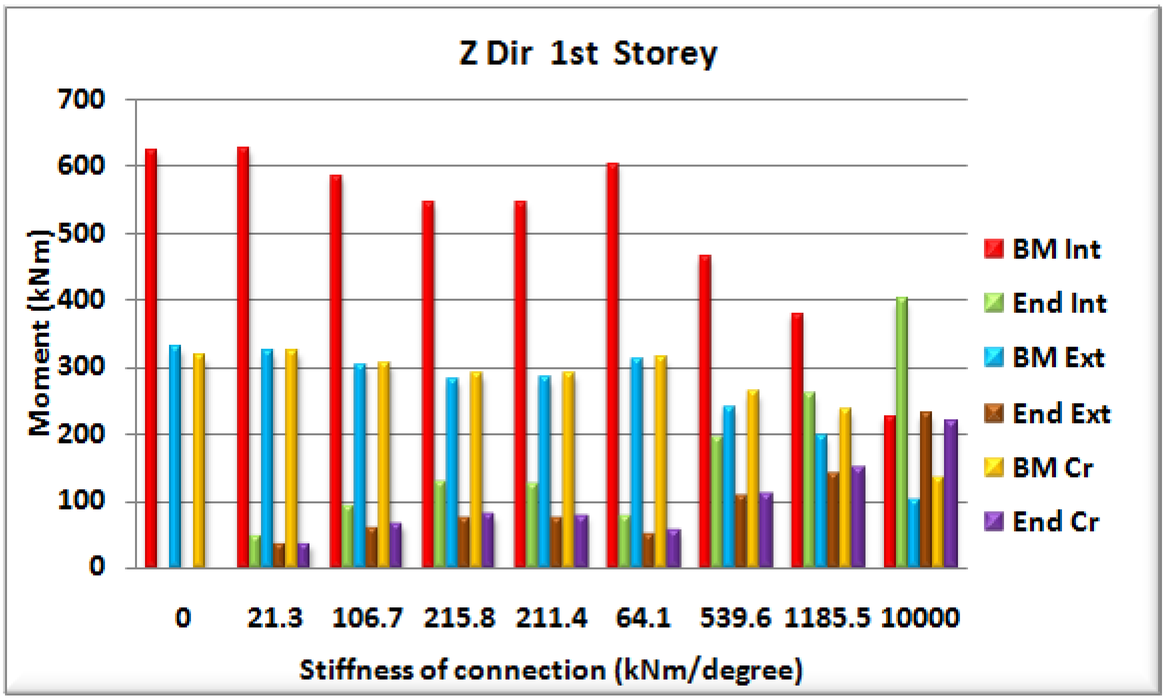


Figure C.26: fig:Effect on Axial Force of Column Due to Connections of 2^{nd} Storey 9 m Span

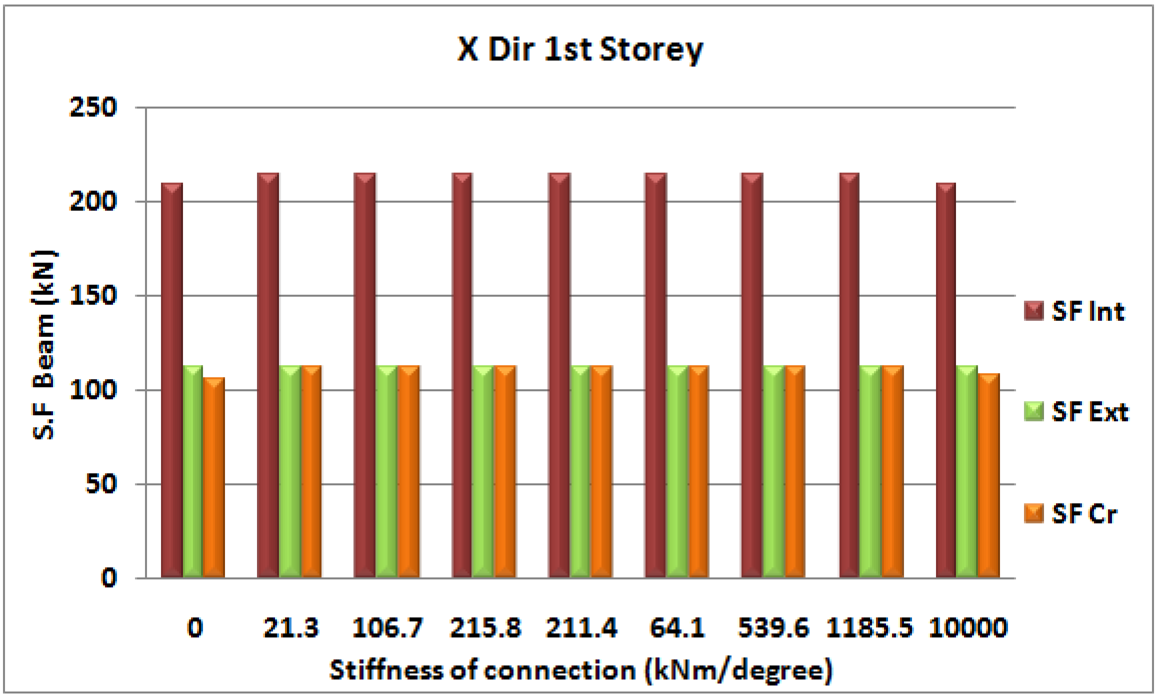


(a) Reduction in Moment of X Dir Beams

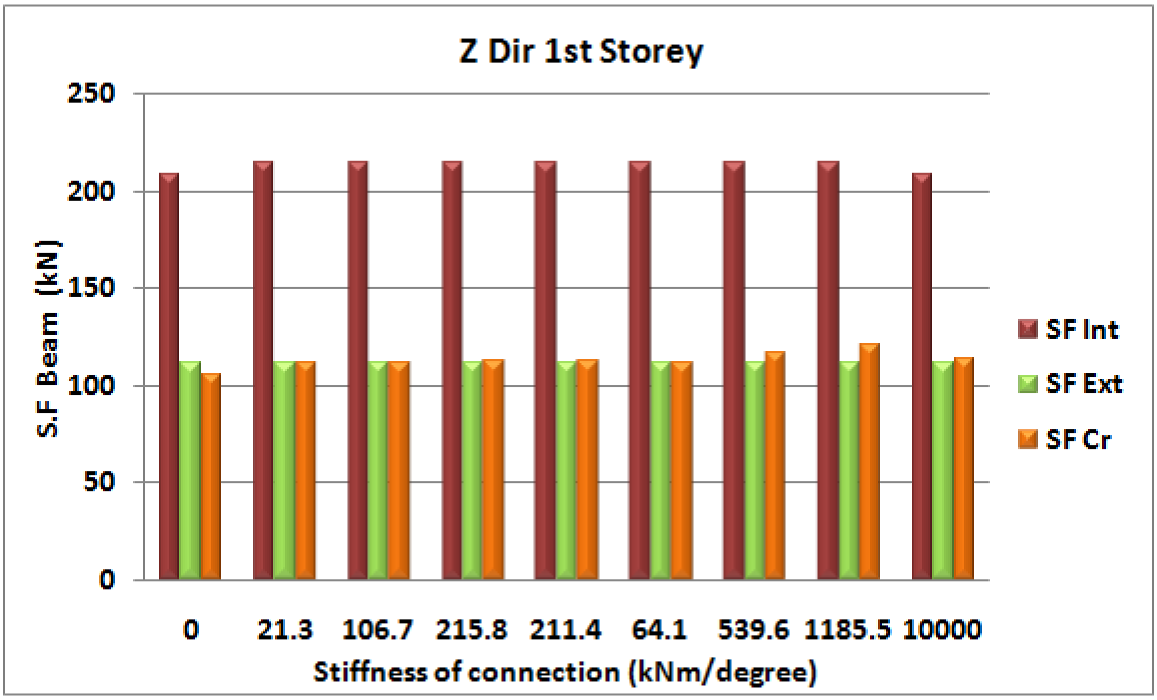


(b) Reduction in Moment of Z Dir Beams

Figure C.27: Reduction in Moment of Beams Due to Connections of 1st Storey 9 m Span

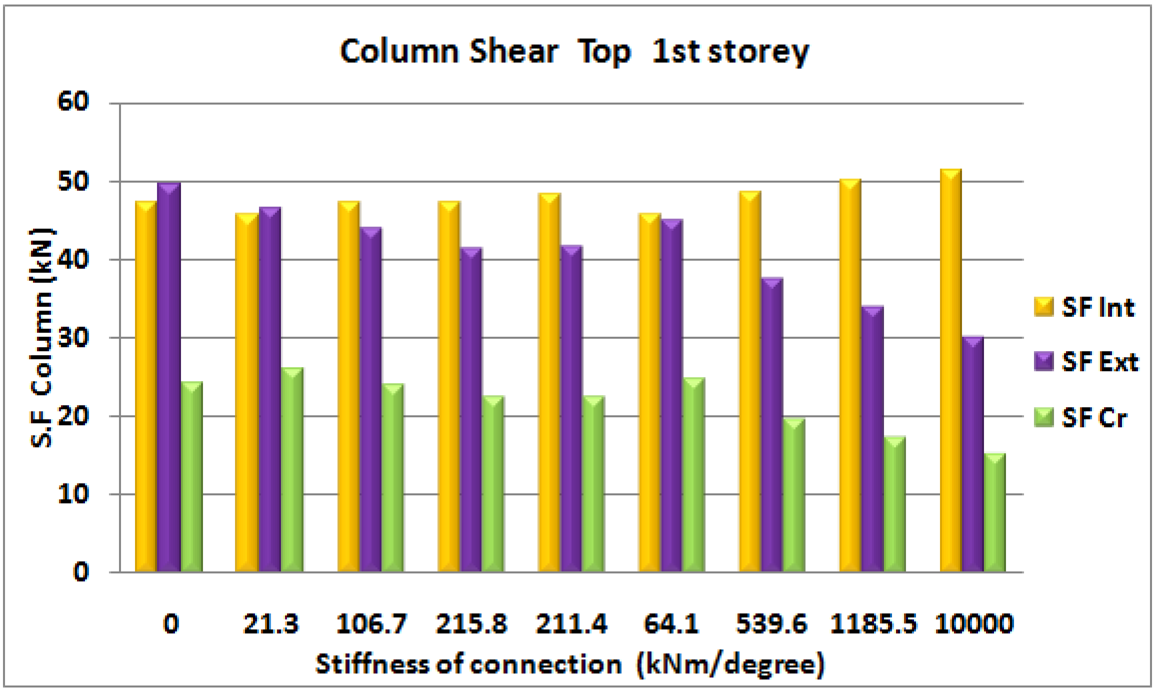


(a) Effect on Shear Force of X Dir Beams

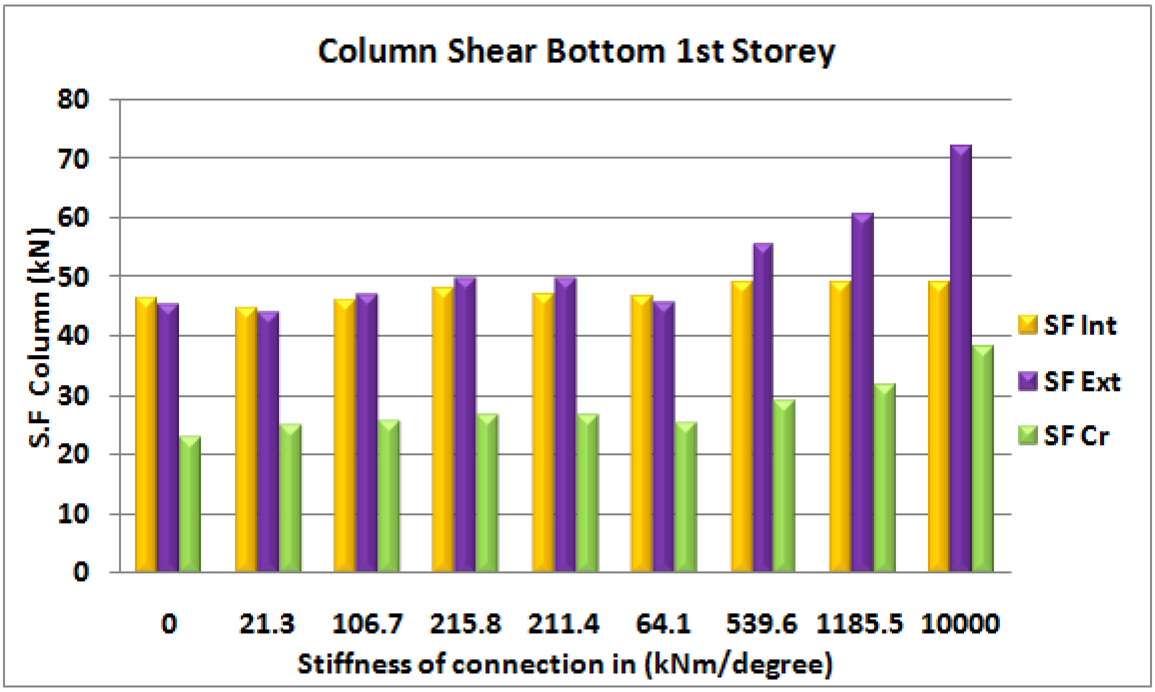


(b) Effect on Shear Force of Z Dir Beams

Figure C.28: Effect og Shear Force on Beams Due to Connections on 1^{st} Storey 9 m Span



(a) Effect of Shear Force on Top of Column



(b) Effect of Shear Force on Bottom of Column

Figure C.29: Effect of Shear Force on Column Due to Connections of 1st Storey 9 m Span

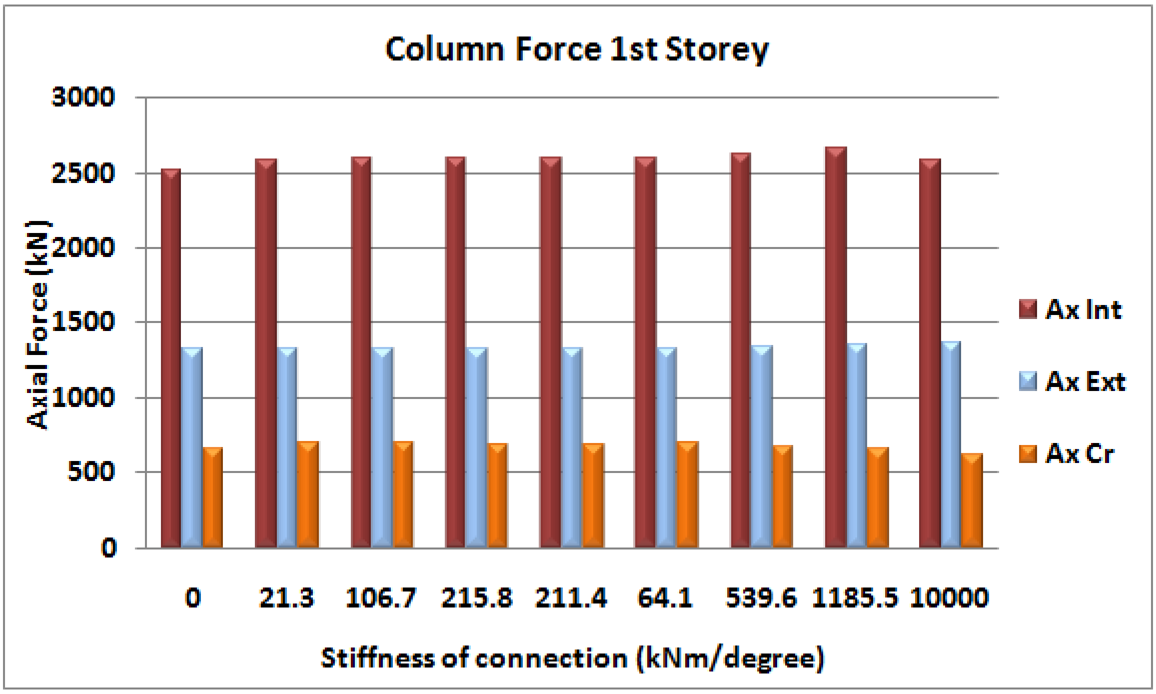
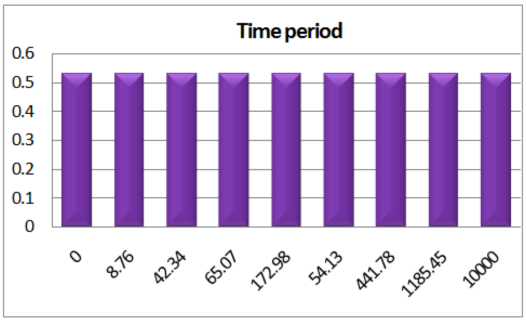
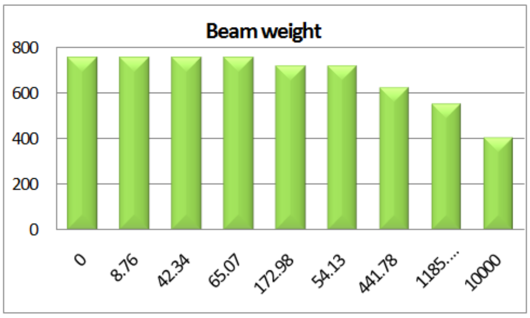


Figure C.30: fig:Effect on Axial Force of Column Due to Connections of 1^{st} Storey 9 m Span

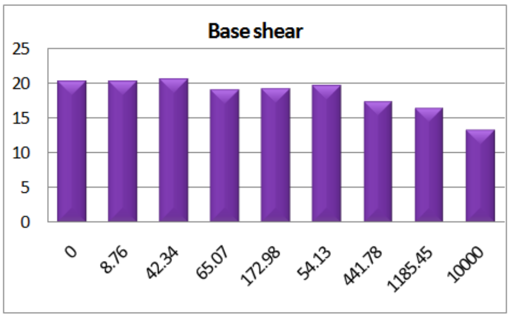


(a) Effect on Time Period

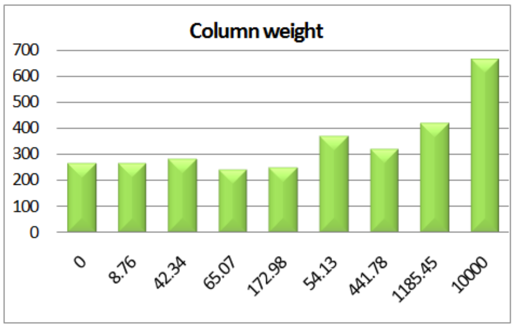


(b) Effect on Base Shear

Figure C.31: Effect on Time Period and Base Shear Due to Connections on 3^{rd} storey 9 m Span

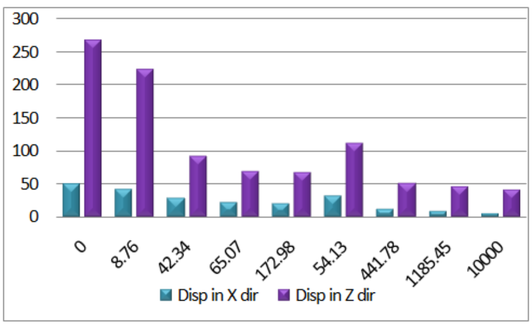


(a) Reduction in Beam Weight

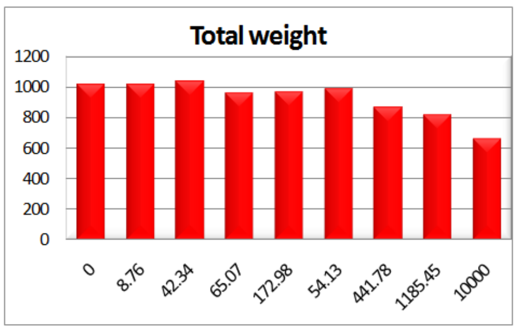


(b) Reduction in Column Weight

Figure C.32: Reduction in Beam Weight and Column Weight Due to Connections on 3^{rd} storey 9 m Span



(a) fig:Effect on Displacement



(b) Reduction in Total Weight

Figure C.33: fig:Reduction in Total Weight Due to connections of 3^{rd} Storey 9 m Span

Appendix D

List of Papers Communicated

Bharat Makhijani and Prof N.C vyas"Effect of Semi rigid Connection on Low Rise long Span Steel Structure":Submited to Nuicone 2012 Conference ; Nirma university