ANALYSIS AND DESIGN OF MULTI-STOREYED BUILDING IN STEEL USING IS 800 DRAFT CODE

Major Project

Submitted in partial fulfillment of the requirements

For the degree of

Master of Technology in Civil Engineering
(Computer Aided Structural Analysis & Design)

By

Hari K. Desai
(04MCL003)

Guide
Prof. G. N. Patel

DEPARTMENT OF CIVIL ENGINEERING
Ahmedabad 382481
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CERTIFICATE

This is to certify that the Major Project entitled "**Analysis and Design of Multi-storeyed building in steel using IS 800 Draft code**" submitted by **Mr. Hari K. Desai (04MCL003)**, towards the partial fulfillment of the requirements for the degree of **Master of Technology in Civil Engineering** (**Computer Aided Structural Analysis and Design**) of Nirma University of Science and Technology, 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.

Prof. G. N. Patel
Guide,
Assistant Professor,
Department of Civil Engineering,
Institute of Technology,
Nirma University,
Ahmedabad

Dr. G. N. Gandhi
Head,
Department of Civil Engineering,
Institute of Technology,
Nirma University,
Ahmedabad

Dr. H.V. Trivedi
Director,
Institute of Technology,
Nirma University,
Ahmedabad

Examiner

Examiner

Date of Examination

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Hari K. Desai
Roll No.04MCL003

ABSTRACT

Institute of steel development & Growth and Ministry of Steel, the bureau of Indian Standards, has jointly circulated a new draft IS: 800, based on Limit State Method. In Introduction to Limit State, the design philosophy is based in probabilistic approach. The design is based on characteristic values for material strengths and applied loads, which take into account the probability of variations in the material strengths and in the loads to be supported. In Limit State Method of design partial safety factors for both material and load variability is considered. In Limit State Method of IS: 800 – Draft, there are separate partial factors for material strengths and applied loads & load combinations.

In India, most of multi-storey buildings are made in concrete. With recent development in steel, it can be possible to construct it in steel. An attempt is made to design 10 storey building in steel using IS: 800 – Draft having RC voided slab. RC voided slab is used to minimise the weight of the slab and to economize section of beam & column which ultimately reduces the total weight of the structure.

The shear connector is designed for full shear connection using eurocde 4 as a composite beam element. The flow chart of the design of flexural member including bearing stiffener is prepared and the built up I section is designed as a laterally restrained beam with all necessary checks such as check for section classification, check for section modulus, check for shear, check for bearing stiffener, check for outstand and check for buckling as per IS 800 Draft. A computer application for the flexural member is prepared in c++ for ISMB, ISWB and ISLB sections. It also provides the percentage strength of the Indian rolled steel I sections. The beam flange and web splices having bolted connection are designed with the use of HSFG bolts as per IS 800 Draft considering reduction in strength due to bolt holes.

The box column with perforated plates and 4-ISA at corners is designed as a beam-column for the governing load condition with all necessary checks i.e. check for section strength, check for slenderness ratio, check for overall member

strength, check for shear and check for clear distance between perforations. Also design moment capacity about major axis is reduced due to lateral torsional buckling about minor axis as mentioned in the IS 800 Draft code. This phenomenon is similar to beam column designed by IS 800-1984 where permissible stress about major axis is reduced as compared to permissible stress about minor axis i.e. 0.66fy. Also as an economical section, the comparision of the designed column with the section 2ISHB with cover plates is provided.

Moment resisting welded connections are designed for the tensile force (caused by hogging moments) at the top flange of the beam which is transferred to the top flange plate by fillet welds and from the plates to the column by groove welds. Also stiffening seat at the beam bottom is designed for shear considering its safe bearing length.

The column splices (lap joints) are designed for minimum required strength 0.6 times $f_y A_f$ for each flange splice and 0.6 times $f_y A_w$ for each web splice with the partial penetration groove welds considering design strength of the joints of at least equal to 200% of the required strength as mentioned in IS 800 Draft code. The moment resisting column bases are designed with the welded stiffening plates which is provided at all four sides of column. Also HSFG bolts are provided to resist the tension in the base.

Also the effect on drift due to change in the orientation of column is studied.
In the analysis of 30 storeyed steel building, the tube in tube structural system is used to resist the lateral loads and the behavior of building as vertical cantilever and shear lag effect is studied.

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NOTATIONS

Abbreviations:

ISMB	Indian Standard Medium Beam
ISWB	Indian Standard Wide Flange Beam
ISHB	Indian Standard Heavy Beam
DL	Dead Load
LL	Live Load
WL	Wind Load
EL	Earthquake Load

Nomenclature:

A	Area of cross section
A_e	Effective cross sectional area
A_g	Gross cross sectional area
b_f	Width of the flange
c_m	Moment reduction factor for lateral torsional buckling strength calculation
D	Overall depth/diameter of the cross section
d	Depth of web, Nominal diameter
d_h	Diameter of the hole
E	Modulus of elasticity for steel
F_d	Factored design load
f_{bd}	Design bending compresssive stress corresponding to lateral buckling
f_{cc}	Elastic buckling stress of a column, Euler's buckling stress
f_{cd}	Design compressive stress
f_o	Proof stress
f_u	Characteristic ultimate tensile stress
f_y	Characteristic yield stress
h	Depth of the section
h_c	Height of the column
h_s	Storey height

I	Moment of inertia of the member about an axis perpendicular to the plane of the frame
I_y	Moment of inertia about the minor axis
I_z	Moment of inertia about the major axis
KL	Effective length of the member
KL/r	Appropriate effective slenderness ratio of the section
KL/r_y	Effective slenderness ratio of the section about the minor axis
KL/r_z	Effective slenderness ratio of the section about the major axis
L	Actual length, unsupported length, Length centre to centre distance of the intersecting members
L_w	Length of weld
M	Bending moment
M_{cr}	Elastic critical moment corresponding to lateral torsional buckling
M_d	Design flexural strength
M_p	Plastic moment capacity of the section
M_y	Factored applied moments about the minor axis of the cross section
M_z	Factored applied moments about the major axis of the cross section
P	Factored applied axial force
P_{cc}	Elastic buckling strength under axial compression
P_d	Design axial compressive strength
P_e	Elastic euler buckling load
r_y	Radius of gyration about the minor axis
r_z	Radius of gyration about the major axis
R_v	Rivet value
t_f	Thickness of flange
t_p	Thickness of plate
t_w	Thickness of web
V	Factored applied shear force
V_{cr}	Critical shear strength corresponding to web buckling
V_d	Design shear strength
V_{nb}	Nominal shear strength of bolt
V_n	Nominal shear strength
Z_e	Elastic section modulus
Z_p	Plastic section modulus

Notations:

α Imperfection factor

β_{MLT} Equivalent uniform moment factor for lateral torsional buckling

χ Stress reduction factor due to buckling under compression

χ_{LT} Strength reduction factor for lateral torsion buckling of a beam

δ Storey deflection

γ_f Partial safety factor for load

γ_m Partial safety factor for material

γ_{m0} Partial safety factor against yield stress and buckling

γ_{m1} Partial safety factor against ultimate stress

γ_{mf} Partial safety factor for bolted connection with HSFG bolts

ε Yield stress ratio, $(250/f_y)^{1/2}$

λ Non dimensional slenderness ratio

μ Poisson's ratio

τ_b Buckling shear stress

τ_{cr} Elastic critical shear stress

1. INTRODUCTION

1.1 GENERAL

A multi-storey building may be defined as "the building because of its height is affected by lateral forces due to wind or earthquake actions to an extent that they play a governing role in structural design".

Population of India is increasing at very fast rate and now it is more than 1000 million people. By the turn of the century, land will become scarce and there will be an urgent need to built high-rise structures in greater number in middle cities also. At present these cities are expanding horizontally or in a mixed manner but with the scarcity of land there is need for vertical expansion.

The advantages of multi-storey buildings are as follows:

- Easy availability of community facilities
- Closer network of services and transport
- Better planning of city
- Availability of more open space
- Fresh air at upper height of building

The construction of multi-storeyed buildings is dependent on available materials, the level of construction technology and the availability of services such as elevators necessary for the use in the building. In ancient Rome, people used to build multi-storeyed structures with wood. For those buildings built after the Great Fire of Rome, Nero used brick and a form of concrete material for construction. Wood lacked strength for buildings of more than five stories and was more susceptible to fire hazard and the buildings constructed with brick masonry occupied a large space for their walls. So due to these drawbacks, development of construction materials took place with the high strength and structurally more efficient materials like wrought iron and then subsequently steel. It permits the lightweight skeletal structures with greater height and larger interior open spaces.

Advantages for using steel frames in the construction of multi-storey buildings are listed below:

- Possibility for the creation of large column-free internal spaces for open-plan offices and large auditoria and concert halls.
- Steel structure occupies lesser percentage of floor area in multi-storey buildings.
- Faster erection compared with reinforced concrete frames & shorter period of time results in economic advantages to the owner due to shorter period of deployment of capital.
- Lighter in comparison with concrete construction, which results in very much reduced loads on foundations.
- Effective quality control due to prefabricated in the factory.
- The use of steel frame results in sufficient extra space to accommodate all service conduits without significant loss in head room.
- Subsequent alterations or strengthening of floors are relatively easy in steel frames compared with concrete frames.

The 10-tall multi-storey (skyscrapers) buildings constructed are listed as follows:

TABLE 1.1 WORLD'S TEN-TALL SKYSCRAPERS

Name	City / Country	Ht. (m)	Floors	Status	Year of completion	Use
Burj Dubai	Dubai, UAE	705	160	Const.	2008	Mixed
Taipei 101	Taipei, Taiwan	508	101	Built	2004	Mixed
Petronas Tower	Kuala Lumpur, Malaysia	452	88	Built	1998	Office
Sears Tower	Chicago IL, US	442	108	Built	1974	Office
Jin Mao Tower	Shanghai SH, China	420.5	93	Built	1998	Mixed
2 International finance Center	Hong Kong HK, China	415.8	90	Built	2003	Office
CITIC plaza	Guangzhou GD, China	391.1	80	Built	1997	Office
Shun Hing Square	Shenzhen GD, China	384	69	Built	1996	Office
Empire State Building	New York city NY, US	381	102	Built	1931	Office
Central Plaza	Hong Kong HK, China	374	78	Built	1992	Office

1.2 DESIGN PHILOSOPHY OF IS – 800: DRAFT

Structural engineers design structures that are safe, serviceable & economical with the use of art, skill, science, experience, judgments and integrity. Structural design features are:

- Safety: It should not collapse or experience excessive damage during its intended lifetime.
- Serviceability: It should perform satisfactorily during its intended lifetime.
- Economy: It should be economical to build & maintain.

1.2.1 RELIABILITY

- It is defined as a quantitive measure for accounting structural safety.
- Reliability = 1- P_f , where P_f = Probability of failure.
- It is to be calculated based on uncertainties in the structures. i. e. calculated from scientific method for handling varying quantities (from experiments & research).

1.2.2 UNCERTAINTIES

TABLE 1.2 UNCERTAINTIES IN LOADS & RESISTANCE

Loads	Resistance
1)Dead Load Less uncertainties(Density) 2)Live Load High uncertainties(Magnitude & Position) 3)Environmental Loads Very High uncertainties(Magnitude, Location, Duration, Repetition, Occurrence)	1)Geometry Dimensions & Sizes 2)Material Variation in material properties Assumptions of isotropic, homogeneity 3)Analysis Method Approximations Linearity assumption Simplifications

1.2.3 EVOLUTION OF DESIGN CODES

After considering all uncertainties in the structure, decide the safety factors from probability such that following criteria must be satisfied:

Design Loads ≤ Design Strength (Design Resistance)

It can be easily understood from the fig.1.1, i.e. load verses resistance curve.

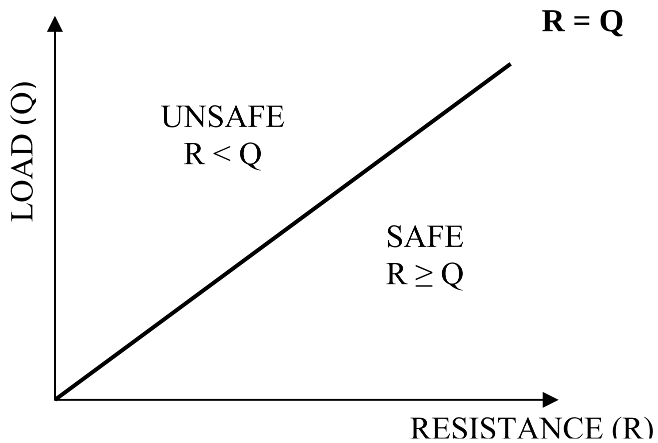


FIGURE 1.1 LOAD (Q) VERSES RESISTANCE (R)

From the variation of material property and change in loads based on experiments and research, partial safety factors is to be calculated in both resistance & loads curve (a) should be enough away from curve(b) as shown in fig.1.2 , considering minimum percentage of failure criteria. Characteristic load is therefore that load which will not be exceeded in 95% of the cases and Characteristic resistance of a material is defined as that value of resistance below which not more than a 5% of test results may be expected to fall. Finally any structure should be designed for the following criteria:

$$(F.O.S.)_R * \text{Resistance} \geq (F.O.S.)_L * \text{Load}$$

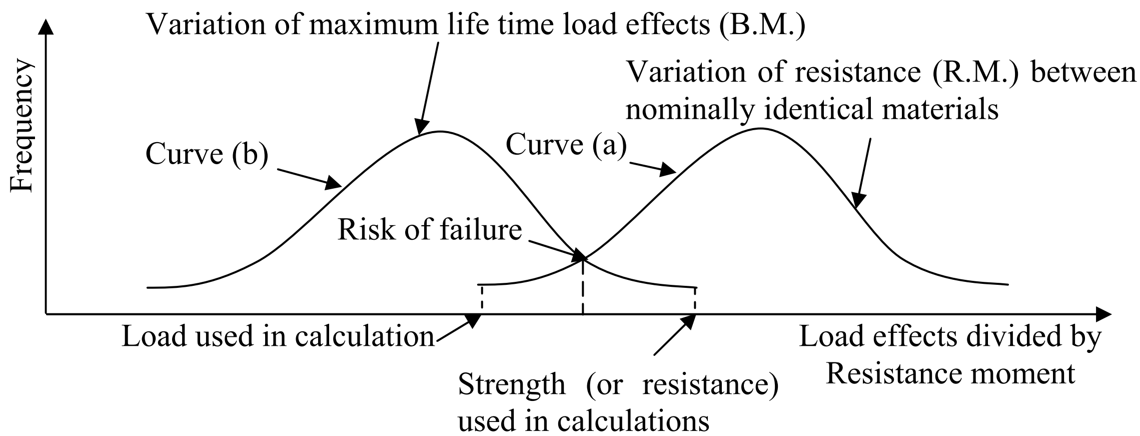


FIGURE 1.2 STATISTICAL MEANING OF SAFETY

1.3 OBJECTIVE OF WORK

The main objectives of the project are:

- To analysis and design ten storeyed building in steel using IS-800: Draft code (Limit state method of design) considering dead load, live load, wind load, earthquake load and its combinations.
- To analysis thirty storeyed building in steel using IS-800: Draft code (Limit state method of design) considering dead load, live load, wind load, earthquake load and its combinations.
- It is also planned to carry out parametric study of effect on drift due to change in orientation of column.

1.3.1 USE OF IS: 800 DRAFT CODE

The basic economical design consideration has to be compared in limit state method and working stress method and also how it is different based on rationality approach.

1.3.2 USE OF RC VOIDED (RIBBED) SLAB

RC voided (ribbed) slab reduces the weight of the slab compared to hollow block and hence chosen for the project.

1.4 BASIC CONCEPT FOR LATERAL LOAD ANALYSIS

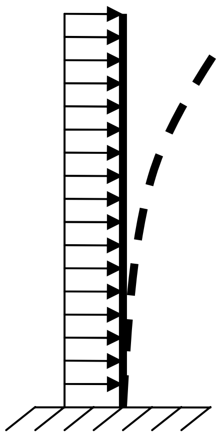


FIGURE 1.3 MULTI-STOREY BUILDING AS VERTICAL CANTILEVER

The structural systems for multi-storey buildings should be considered as a "Vertical Cantilever Beam" for the lateral force design as shown in fig.1.3.

The laterally directed force generated either due to wind or earthquake tends to shear and to bend it.

Systems resisting shear forces:

- Should not break by shearing off.
- Should not strain beyond the limit of elastic recovery.

Systems resisting bending:

- Should be safe against overturning due to gravity and lateral load.
- Should not break columns by crushing or tensile forces.
- Deflection should not exceed the limit of elastic recovery i.e. H/350 to H/650, where H is the total height of the structure.

2.1 LITERATURE SURVEY

Literature survey is carried out to study the behaviour in Limit State Method (LSM) of design and applications of it in multi-storey building in the field of structural engineering. Such applications include analysis and design in most economic way. In literature survey, main emphasis is given codal provisions of IS: 800 Draft, INSDAG teaching resources, books and published papers on steel structures.

INSDAG teaching resources includes introduction of multi-storey building including its historical development considering availability of material, level of construction technology and available services. It also includes advantages of steel frames in multi storey buildings by considering its material property, durability, time rate of construction, transportation, installation, labour, its load resistance capacity, etc. It include introduction to limit state method which considers the characteristic strength and characteristic load with appropriate partial safety factors considering ultimate strength and serviceability criteria based on uncertainties in the structure by probabilistic approach. It includes different types of gravity (floor) and lateral load resisting structural system used in steel multi-storey buildings depending upon its behaviour and strength for lateral load resistance. It also includes different type of connections such as simple, rigid and semi-rigid connection. It also includes flexural member behavior considering its full capacity condition of flange and web and its effect on design, solved examples based on BS code and its applications. It includes the design criteria of continuous composite beam & shear connector design as per eurocode 4. The bolted connection design criteria for the HSFG bolts are also included in these resources which states that all lateral load resisting frames are to be designed with HSFG bolts only.

Tall building structures by Coull & Smith**,** includes various serviceability criteria for the multistory building design such as drift limitations ranging from H/350 to H/650,where H is the total height of the structure, depending upon the inter-storey drift value of 0.001 to 0.005 times the storey height. It also includes the behaviour of frame structure for the lateral and gravity load analysis. It also includes check for stability buckling and secondary analysis.

Steel structures by Salmon and Johnson, includes design philosophy of LRFD and design criteria of flexure member based on LRFD method. The design of laterally supported flexural member considers on the bases of its behaviour considering the full capacity of flange & web i. e. compression & tension flange braced perpendicular to its plane by web act as laterally braced in the direction perpendicular to the plane of web so full participation to develop the moment strength of the section without allowing overall buckling of compression flange as column.

Design of Steel Structures by Arya and Ajmani, includes the recommendations for the design of perforated plates which includes 'Guide to design criteria for metal compression members' published by Column Research Council- U.S.A. It also includes the design criteria for the welded moment resisting connections with stiffened seat and moment resisting column bases as per working stress method.

Steel plate shear wall by Astaneh –Asl**,** this paper is intended to provide a summary of the past experimental research on steel plate shear walls with emphasis on research done in North America for the derivation of R value, different type of steel plate shear wall stiffened and unstiffened, a brief summary of state of practice in using steel plate shear walls in highly seismic areas and its seismic design recommendations.

Dr. S. R. Satishkumar**,** this paper includes earthquake resistant configuration and systems, inelastic behaviour of steel sections (members) and frames, seismic behaviour of bracing members and different connections. In this paper it is suggested that for the connection design reference should be taken from British or New Zealand standards.

New Zealand Standards and Euro code, this includes foreign design standards for steel. It also includes the connection design considering the strong column-weak beam concept which suggests that the joint should be capable enough to transfer the maximum beam moment without local failure of column. These codes have the standardization based on LRFD approach.

2.2 SCOPE OF WORK

- Analysis & Design of 10-storey steel building as per IS-800 draft code considering DL, LL, WL, EL & load combination.
- Analysis of 30-storey steel building as per IS-800 draft code considering DL, LL, WL, EL & load combination.
- Orientation of columns – effect on drift
- Computer Program for the design of flexural member (Laterally Restrained Beam)

3. LATERAL LOAD RESISTING STRUCTURAL SYSTEMS

As stated earlier, Lateral forces due to wind or seismic loading must be considered for multi-storey buildings along with gravity forces because it governs the design criteria. The total deflection (drift) at top of the lateral load resisting structure should be less than the permissible deflection. The drift is measured by drift index, Δ/h, where, Δ is the horizontal deflection at top of the building and h is the height of the building. Lateral drift of a typical moment resisting frame is shown in Fig. 3.1.

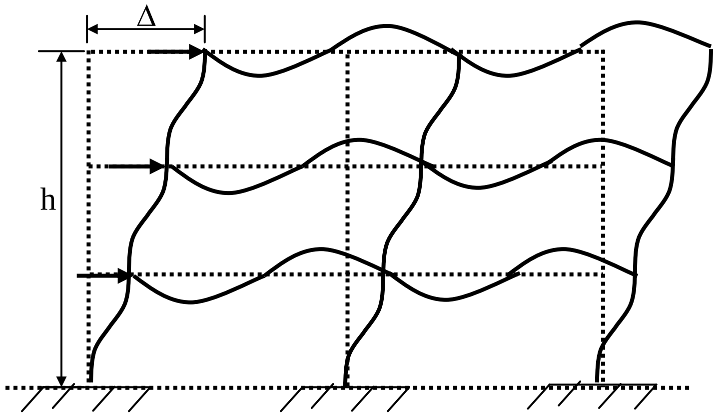


FIGURE 3.1 LATERAL DRIFT

The usual practice in the design of multi-storey steel buildings is to provide a structure with sufficient lateral stiffness to keep the drift index between approximately 0.0015 and 0.0030 of the total height (H/650 to H/350). The IS code require drift index to be not more than 0.002 of total height (H/500).

The control of the dynamic response (i.e. behavior after application of lateral load) of the multi-storey building can be achieved in the following ways:

- By increasing stiffness through the use of an efficient structural systems.
- By selecting an efficient building shape.
- By generating additional forces in the building to counteract the lateral action.
- By increasing weight of the structure (not feasible).
- By increasing building density through the addition of more structural and fill-in material (not feasible).

Here main importance is given for the use of efficient structural system. Provision of efficient building shape is possible and attempted in buildings effectively. But it requires the different architectural aspects which can be possible by the combined efforts of structural engineer and architecture. Use of base isolation is also an uneconomical solution but can be adopted in very high seismic zone in important structures.

The following structural systems are used to resist the lateral loads and limit the total deflection at top within acceptable range as mentioned above:

3.1 Moment resisting frames
- 3.1.1 Frame with semi-rigid connections
- 3.1.2 Rigid Frames

3.2 Steel plate shear walls
3.3 Braced frames
3.4 Tube structures

3.1 MOMENT RESISTING FRAMES

A steel frame can be classified depending upon the beam-column connection. There are three types of connections simple, rigid and semi-rigid. A multi-storey building with no lateral bracing, i.e. with simple beam connection is shown in Fig. 3.2(a). Simple connection is detailed to allow the beam end to rotate freely and the beam behaves as a simply supported beam. Such a connection transfers shear and axial forces between the connecting members but does not transfer bending moment. Generally in these types of connections, the frame would have practically no resistance to the lateral forces and become geometrically unstable. The frame would laterally deflect as shown in Fig. 3.2(b) even under a small lateral load. So it is necessary to provide the beam-column connection as rigid or semi-rigid. The moment resisting frames are as shown in fig 3.3

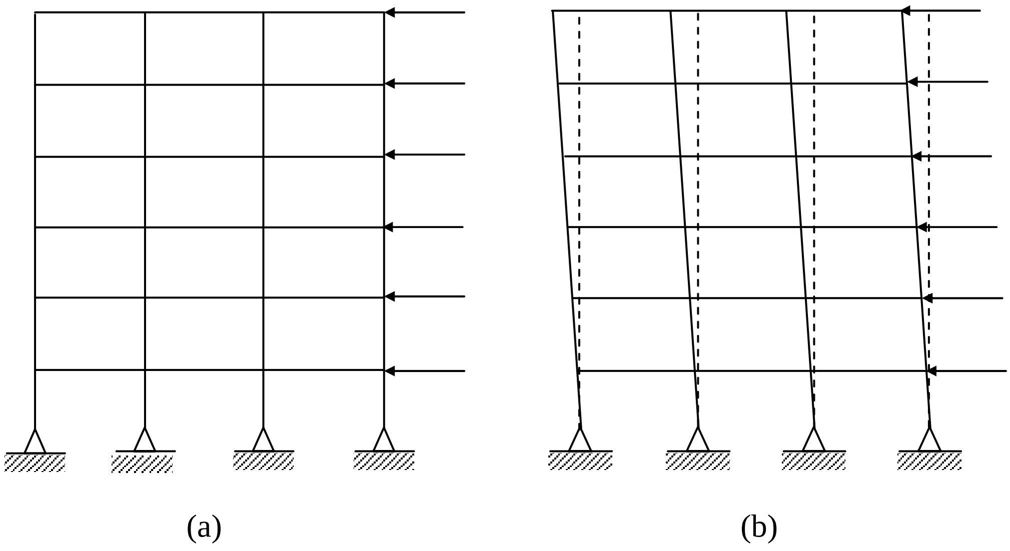


FIGURE 3.2 MULTI-STOREY FRAME WITHOUT LATERAL BRACING

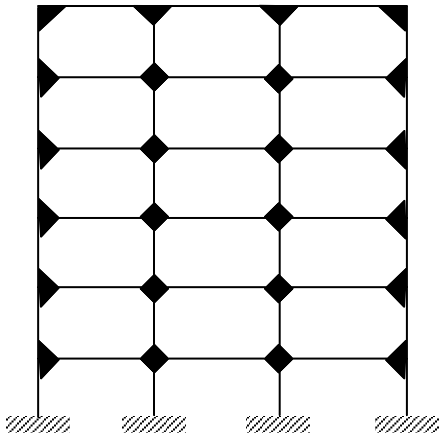


FIGURE 3.3 MOMENT RESISTING FRAMES

3.1.1 FRAME WITH SEMI-RIGID CONNECTIONS

In these types of connections, Due to flexibility of the joint some relative rotation between the beam and column occurs. When this is substantial, the joints are designed as semi-rigid. These connections, as shown in fig.3.4, are designed to transmit the full shear force and a fraction of the bending moment across the joint. The analysis of frames with such joints is complex. It shows following characteristics:

- Semi rigid connections show very high ductility without sudden loss of strength.
- Dissipation of energy is more compared to other due to high yielding capacity.
- It gives cost effective construction.

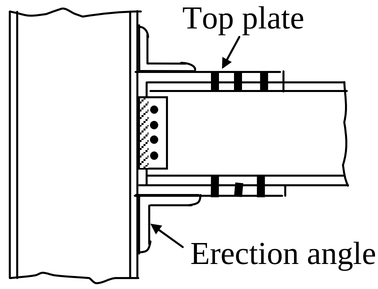


FIGURE 3.4 SEMI- RIGID CONNECTION

3.1.2 RIGID FRAMES

Rigidly jointed frames or sway-frames are those with moment resisting connections between beams and columns. This connection is detailed to ensure a monolithic joint such that the angle between beam and column before deformation remains the same even after deformation. Such a connection transfers shear, axial force and bending moment from the beam to the column. Resistance to lateral loading is provided by the bending resistance of the columns, girders and joints. However, moment resisting connections may be necessary in locations where loads are applied eccentrically with respect to centre line of the columns. Three types of commonly employed moment resisting connections are shown in Fig. 3.5. The connection shown in Fig. 3.5(*a*) and 3.5(*c*) are more economical. However, the moment-rotation performance of the connection shown in Fig. 3.5(*b*) is likely to be superior to that of either Fig. 3.5(*a*) or Fig. 3.5(*c*).

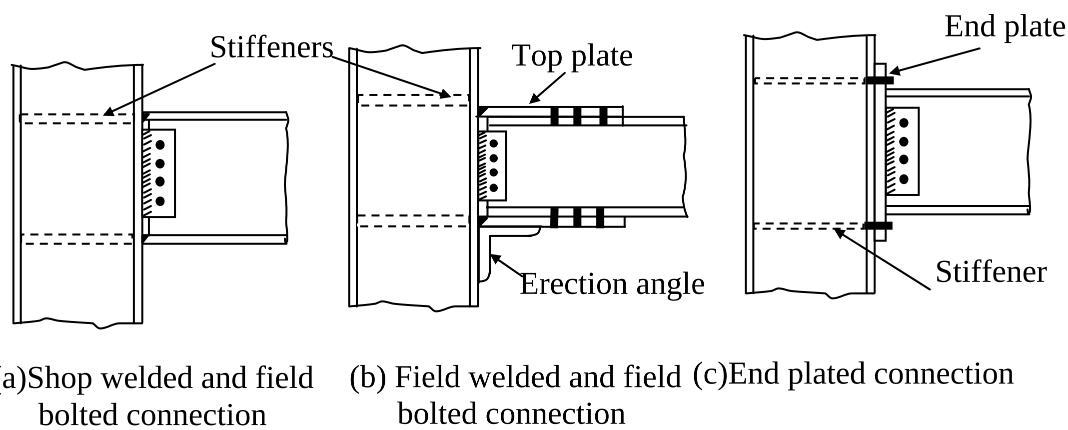
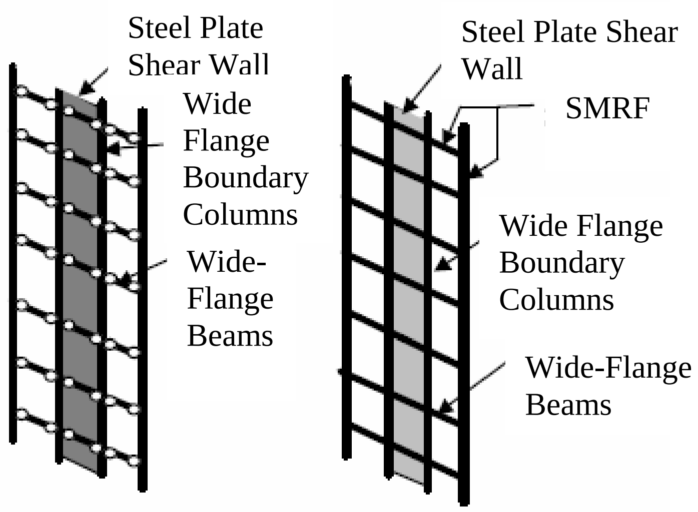


FIGURE 3.5 MOMENT RESISTANT CONNECTIONS

Forces caused by external shear are resisted by bending of columns and beams of the frame in double curvature with point of contra-flexure at approximately mid-height or length causing racking of the frame and horizontal deflection in each story. So overall deflected shape will be concavity upwind, i.e. a maximum inclination near the base and minimum inclination at top which shows shear mode behaviour. The overall moment of the external horizontal load is resisted in each story level by the axial tensile and compressive forces in the columns on opposite sides of the structure which shows flexural mode behaviour. It is economical for building of only up to about 25 storeys, for more storeys its drift resistance is costly to control.

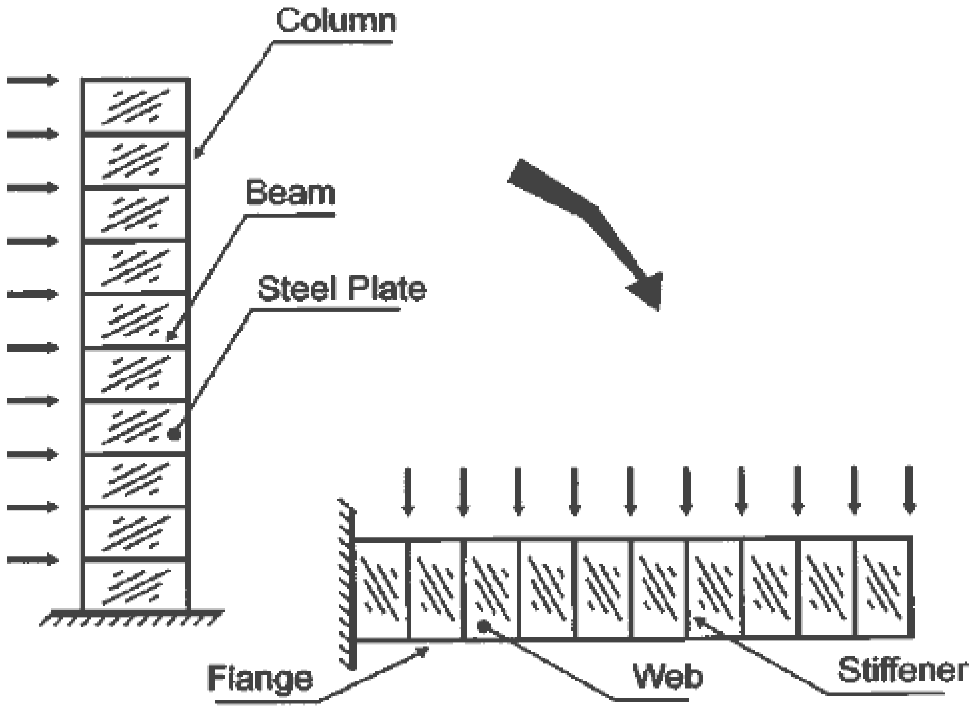
3.2 STEEL PLATE SHEAR WALLS



a) Steel Plate Shear Wall b) Dual Steel Plate Shear Wall system

FIGURE 3.6 STEEL PLATE SHEAR WALL

The lateral loads are assumed to be concentrated at the floor levels. The rigid floors spread these forces to the columns or walls in the building. Specially designed steel plate walls parallel to the directions of load are used to resist a large part of the lateral loads caused by wind or earthquakes by acting as vertical plate girder provided between beams fixed at foundation. These elements are called as shear walls. Frequently buildings have interior steel core walls around the elevator and service wells. Such walls may be considered as shear walls. As shown in fig.3.6 (a), steel plate shear walls can be provided as individual lateral load resisting system or also as a dual system as shown in fig. 3.6 (b).



Steel plate wall **Cantilever plate girder**

FIGURE 3.7 STEEL PLATE SHEAR WALL IDEALISED AS VERTICAL CANTILEVER PLATE GIRDER

Shear walls are provided in the buildings to serve the following purposes:

- To avoid column shear failures due to inter-story distortion.
- To reduce lateral drift.
- To reduce joint detailing problem.
- To avoid column failure due to P-Δ secondary effect.

Shear wall have considerably high stiffness in its own plane (along its length) but usually very less stiffness in the perpendicular direction (along its thickness). Steel plate shear wall have two types i.e. stiffened and unstiffened. Stiffened shear wall can be provided depending upon the openings requirement and size of wall. Steel plate shear walls contribute significant lateral stiffness, strength, ductility and energy dissipation capacity. It is much lighter and faster to construct than R.C.C. shear wall. It can be used for retrofitting of existing building. They are provided between column lines. So it can be provided in the wide flange beam-column with pinned joints and moment resisting joints. In Earlier case, the lateral load moment is resisted by shear wall only while in the later case it is resisted by both shear wall and moment resisting frame. This type of structural system can be adopted up to 35 storeys.

3.2.1 FAILURE MODES

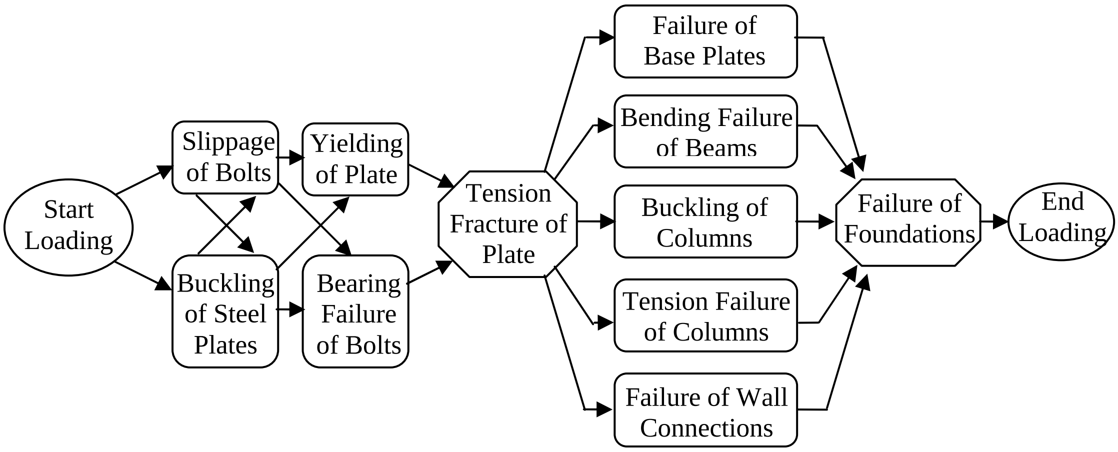


FIGURE 3.8 FAILURE MODES OF STEEL PLATE SHEAR WALL

The failure modes can be listed to obtain a desirable ductile performance as shown in fig. 3.8.

- The slippage of the boundary bolts should not be considered a consequential failure mode because such slippage provides a mechanism of energy dissipation through friction and introduces some beneficial semi-rigidity to the structure.
- The buckling of plate in unstiffened steel plate shear walls is not considered detrimental in performance and has no significant effect on the ultimate shear strength and overall performance of the wall. If buckling of the wall, which will result in out-of-plane deformations, is creating serviceability problems, then stiffened shear walls should be used to delay buckling of the wall and to reduce out-of-plane deformations due to buckling. But such stiffened walls are expected to be more expensive to fabricate.
- The yielding of diagonal tension is the best mechanism of failure and should be established as the governing failure mode in seismic or wind design.
- The fracture in tension or buckling in compression of boundary columns should be avoided in design since such failures can have serious stability problems as well as very high cost of post earthquake repairs.

3.3 BRACED FRAMES

To resist the lateral deflections, the simplest method from a theoretical standpoint is the intersection of full diagonal bracing or *X*-bracing as shown in Fig. 3.9. The *X*-bracing system works well for 20 to 60 storey height, but it does not give room for openings such as doors and windows. To provide more flexibility for the placing of windows and doors, the K-bracing system shown in Fig. 3.10(a) is preferred instead of *X*- bracing system. If, we need to provide larger openings, it is not possible with *K*-bracing system; we can use the full-storey knee bracing system shown in Fig. 3.10(b). Knee bracing is an eccentric bracing that is found to be efficient in energy dissipation during earthquake loads by forming plastic hinge in beam at the point of their intersection of the bracings with the beam.

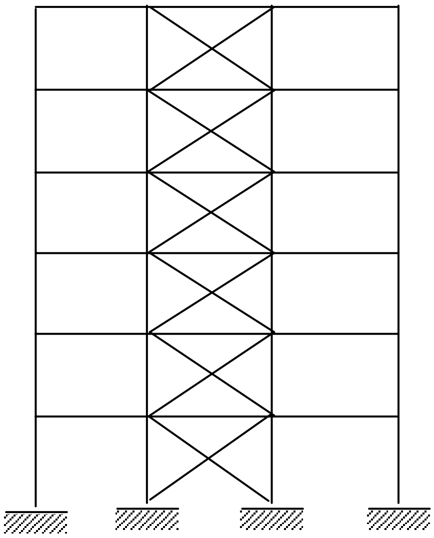
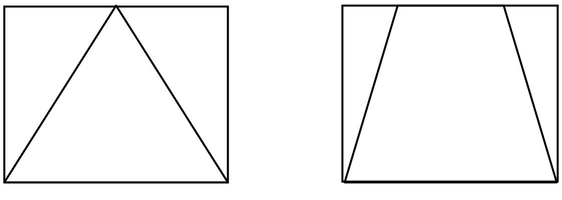


FIGURE 3.9 BRACED FRAMES WITH X-BRACING



(a) K- brace (b) Knee brace

FIGURE 3.10 ALTERNATIVE BRACING SYSTEMS

The braces and girders act as the web members of the truss, while the column act as the chords. Bracing is efficient because the diagonals work in axial stress and therefore call for minimum member sizes in providing stiffness and strength against horizontal shear. Diagonal bracing is inherently obstructive to the architectural plan and can pose problems in the organization of internal space and traffic as well as in locating window and door opening. Because lateral loading on a building is reversible, braces will be subjected to both tension and compression; consequently, they are usually designed for the more significant case of compression and checked for tension. The behaviour of the structure due to flexural and shear deflection is same that of rigid-frame behaviour. Sometimes eccentric braced system can also be used to reduce concentration of forces at joint. In such systems all axial forces are transferred at some eccentricity to the joint which indirectly cause the yielding of beam first and then joint.

3.4 TUBE STRUCTURES

A tube structure may be defined as a space structure composed of frames, braced frames or shear walls joined at their edges to form a vertical tube like structure capable of resisting lateral load in any direction by cantilevering action. The bracing systems discussed so far are not efficient for buildings taller than 60 storeys. This section introduces more advanced types of structural forms that are adopted in steel-framed multi-storeyed buildings larger than 60 storeys high. Common types of tube forms are:

3.4.1 FRAMED -TUBE STRUCTURES

The lateral resistance of framed-tube structures is provided by very stiff moment-resisting frames; consist of closely spaced columns joined by deep spandrel beams that form a “tube” around the perimeter of the building. The tube carries all the lateral loads while the gravity load is shared between the tube and interior columns or walls. The framed tube is one of the most significant modern developments in high-rise structural form. The frames consist of closely spaced columns, 2 - 4 m between centers, joined by deep girders. The tube will act like a continuous perforated chimney or stack. This structural form offers an efficient, easily constructed structure appropriate for buildings having 40 to100 storeys. The close spacing of columns at the entrance levels is unacceptable so they are terminated on a transfer beam a few stories above the base, so that only a few larger more widely spaced columns continue to base. When lateral load acts, the perimeter frames aligned in the direction of loading act as the "webs" and those normal to the direction of the loading act as the "flanges".

Even though framed tube is a structurally efficient form, flange frames tend to suffer from shear lag. This results in the mid face flange columns being less stressed than the corner columns and therefore not contributing to their full potential lateral strength. The tube looks like the grid-like façade as small windowed and is repetitious and hence use of prefabrication in steel makes the construction faster. A typical framed tube is shown in Fig. 3.11

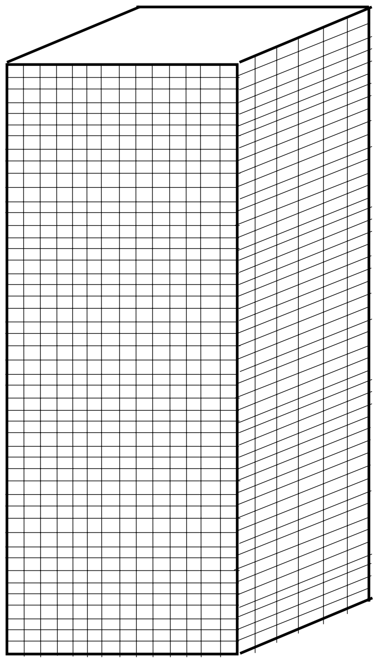
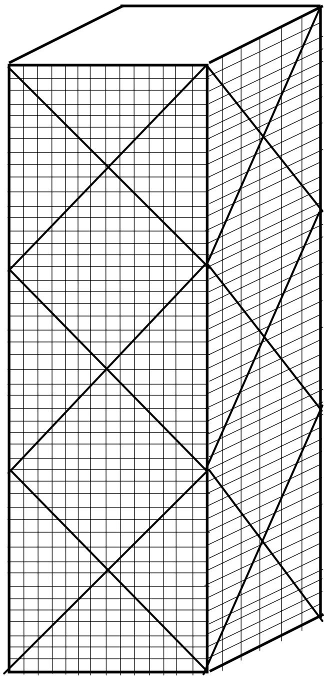


FIGURE 3.11 FRAMED TUBE STRUCTURE

3.4.2 BRACED TUBE STRUCTURES

Further improvements of the tubular system can be made by cross bracing the frame with *X*-bracing over many stories, as illustrated in Fig. 3.12. This arrangement was first used in a steel structure, in Chicago's John Hancock Building, in 1969. As the diagonals of a braced tube are connected to the columns at each intersection, they virtually eliminate the effects of shear lag in both the flange and web frames. So the structure behaves under lateral loads more like a braced frame reducing bending in the members of the frames. Hence, the spacing of the columns can be increased and the depth of the girders will be less, thereby allowing large size windows than in the conventional framed tube structures. In the braced tube structure, the braces transfer axial load from the more highly stressed columns to the less highly stressed columns and eliminates differences between load stresses in the columns.



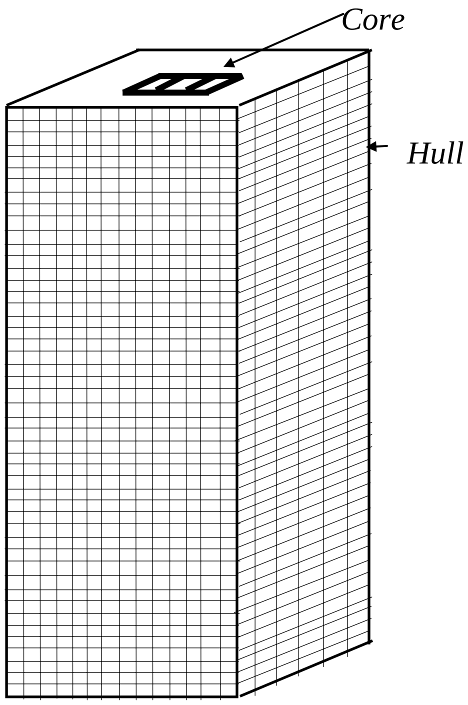


FIGURE 3.12 BRACED FRAMED TUBE **FIGURE 3.13 TUBE-IN-TUBE FRAME**

3.4.3 TUBE-IN-TUBE STRUCTURES

This is a type of framed tube consisting of an outer-framed tube together with an internal elevator and service core. The inner tube may consist of braced frames or shear walls. The outer and inner tubes act jointly in resisting both gravity and lateral loading in steel-framed buildings. However, the outer tube usually plays a dominant role because of its much greater structural depth. This type of structures is also called as Hull (Outer tube) and Core (Inner tube) structures. A typical Tube-in-Tube structure is shown in Fig. 3.13.

3.4.4 BUNDLED TUBE

The bundled tube system can be visualised as an assemblage of individual tubes resulting in multiple cell tube. The increase in stiffness is apparent. The system allows for the greatest height and the most floor area. This structural form was used in the Sears Tower in Chicago. In this system, introduction of the internal webs greatly reduces the shear lag in the flanges as shown in fig 3.14. Hence, their columns are more evenly stressed than in the single tube structure and their contribution to the lateral stiffness is greater.

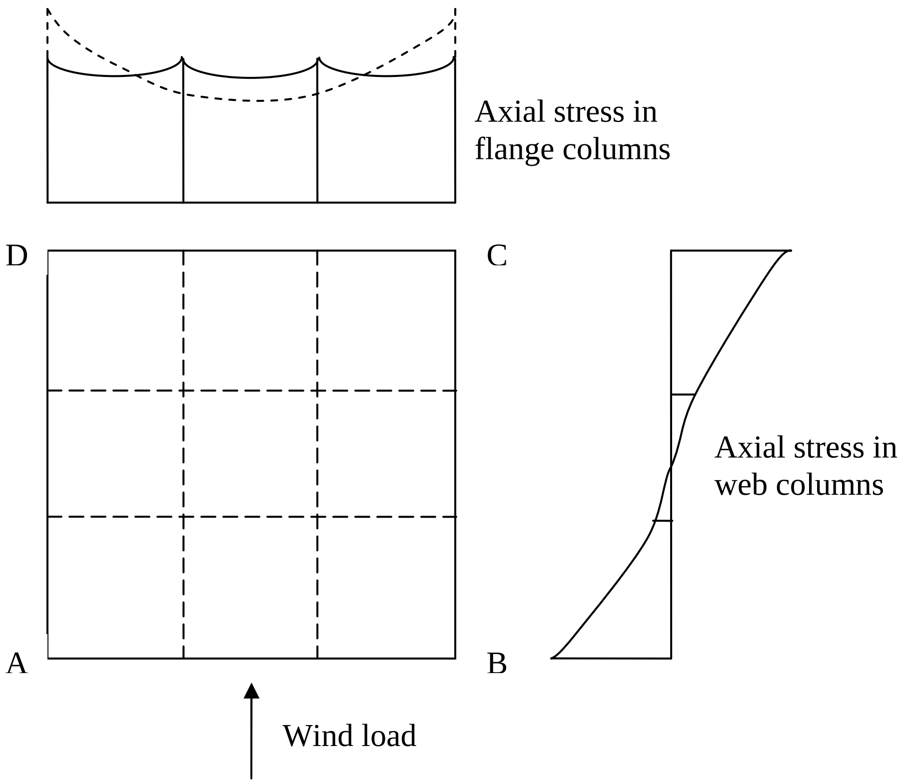


FIGURE 3.14 SHEAR LAG IN BUNDLED-TUBE STRUCTURES

4. ANALYSIS AND DESIGN OF 10 STOREYED STEEL BUILDING

4.1 GENERAL DATA

The plan of the building is as shown in fig. 4.1, the layout has following details:
Plan area: 43.6 m x 23 m [6 grids (1 to 6) in z–dir and 4 grids (A to D) in x–dir]
Storey height: 5 m at ground floor and 3.5 m at typical floor
Building Type: Office building
S.B.C. - 250 kN/m^2
Concrete Grade – M_{20} for Slab & M_{30} for Footing

4.2 MODELING

The structure is modeled as beam-column space frame structure with fixed support condition in the STAAD PRO-2004 as shown in fig.4.2, with steel beam as ISMB600 and ISMB450 and the steel column as box section 300 x 400 x 12 mm of weight 131.9 kg/m. Here the rigid diaphragm property is provided at each storey for the equal deflection of all nodes in a storey. The beam and column number of frame-1 in z direction is as shown in fig.4.3.

4.3 LOADING

4.3.1 DEAD LOAD

R.C.C. (slab) – 25 kN/m^3
Floor Finish – 1.0 kN/m^2
Steel Density (M.S.) – 7850 kg/ m^3 and E – 2 x 10^5 N/mm^2

4.3.2 LIVE LOAD

As per IS: 875 (Part- 2) -1987, Live Load – 4.0 kN/m^2
Reduction in LL is made for the design load calculation of column at different storey.

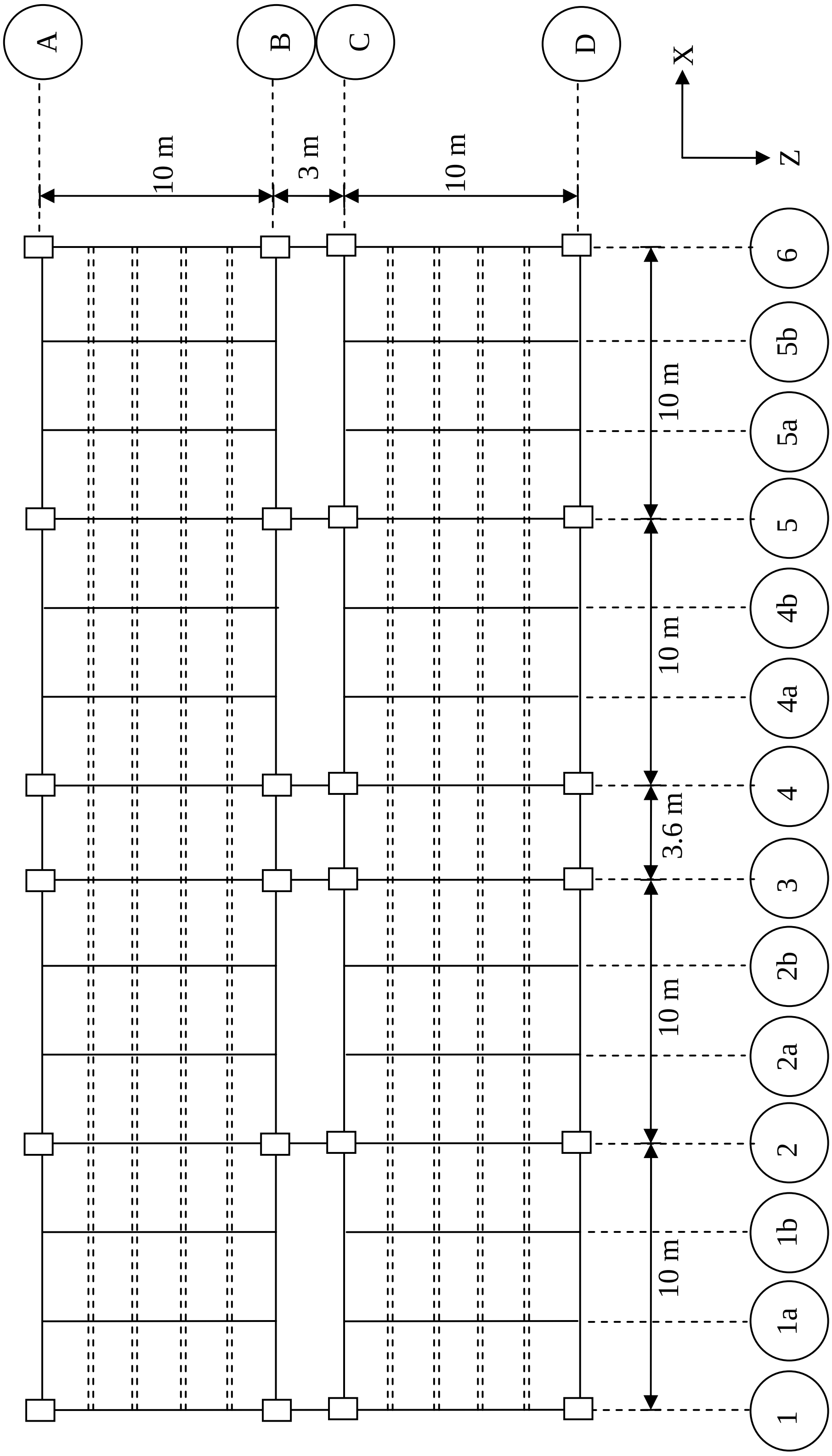


FIGURE 4.1 PLAN OF 10 STOREYED STEEL BUILDING

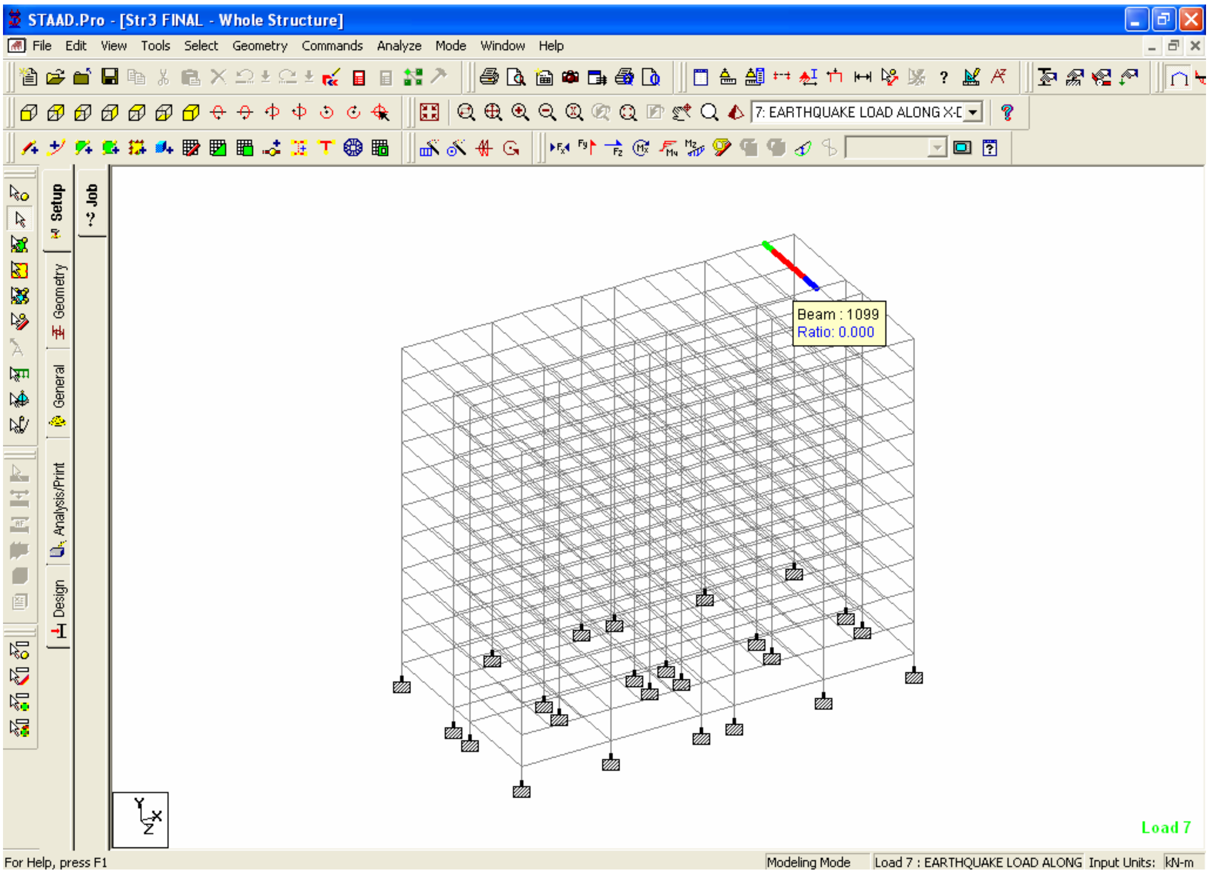


FIGURE 4.2 STRUCTURE MODELED IN STAAD PRO 2004

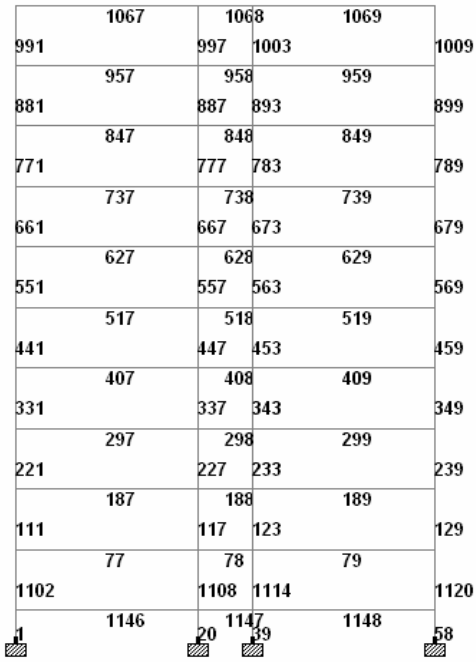


FIGURE 4.3 BEAM AND COLUMN NUMBERS OF FRAME-1 IN Z DIRECTION

4.3.3 WIND LOAD

As per IS: 875 (Part 3) -1987,

Basic wind speed, V_b = 39 m/sec

Risk Coefficient, K_1 = 1.0

Terrain, height and structure factor, K_2 = 0.88, for height up to 10 m
= 0.94, for height 10-15 m
= 0.98, for height 15-20 m
= 1.03, for height 20-30 m
= 1.09, for height 30-50 m

Topography factor, K_3 = 1.0

Design wind speed, $V_z = V_b * K_1 * K_2 * K_3$

Wind Intensity, $P_z = 0.6 * V_z^2$

Wind pressure coefficient (for both direction), C_f = 1.20

Wind force $F = C_f * A_e * P_z$

4.3.4 EARTHQUAKE LOAD

As per IS: 1893 (Part 1) – 2002,

Zone – IV

Soil Condition – Medium soil

Importance Factor, I = 1.0

Response Reduction Factor for Steel moment resisting frame, R = 5.0

Time Period for steel frame building, $T = 0.085h^{0.75}$ (without brick infill panels)

$\therefore$T = 1.262 sec, where h = 36.5 m

4.4 LOAD COMBINATION

4.4.1 LOAD COMBINATION FOR ULTIMATE LOAD

1 1.5*DL+1.5*LL

Wind load combination

2 $1.2*(DL + LL \pm WL_X)$

3 $1.2*(DL + LL \pm WL_Z)$

4 $1.2*(DL + LL) \pm 0.6*WL_X$

5 $1.2*(DL + LL) \pm 0.6*WL_Z$

6 1.5*(DL ± WL_X)

7 1.5*(DL ± WL_Z)

8 0.9*DL ± 1.5*WL_X

9 0.9*DL ± 1.5*WL_Z

Seismic load combination

10 1.2*(DL + LL ± EQ_X)

11 1.2*(DL + LL ± EQ_Z)

12 1.2*(DL + LL) ± 0.6*EQ_X

13 1.2*(DL + LL) ± 0.6*EQ_Z

14 1.5*(DL ± EQ_X)

15 1.5*(DL ± EQ_Z)

16 0.9*DL ± 1.5*EQ_X

17 0.9*DL ± 1.5*EQ_Z

4.4.2 LOAD COMBINATION FOR SERVICEABILITY CONDITIONS

18 1.0*DL+1.0*LL

Wind load combination

19 1.0*DL + 0.8*LL ± 0.8*WL_X

20 1.0*DL + 0.8*LL ± 0.8*WL_Z

21 1.0*DL ± 1.0*WL_X

22 1.0*DL ± 1.0*WL_Z

Seismic load combination

23 1.0*DL + 0.8*LL ± 0.8*EQ_X

24 1.0*DL + 0.8*LL ± 0.8*EQ_Z

25 1.0*DL ± 1.0*EQ_X

26 1.0*DL ± 1.0*EQ_Z

4.5 PRELIMINARY ANALYSIS

In preliminary analysis, the structure is checked for permissible deflection and inter-storey drift for the governing serviceability load combination i.e. 1.0*DL + 1.0*EQ_Z and the column section is revised as shown in table 4.1. So the revised box column section is 400 x 500 x 12 mm.

Permissible Deflection = H/650 = 56 mm

Permissible inter-storey drift = 0.004*h = 14 mm (for 3.5 m ht.) and 20 mm (for 5 m ht.)

TABLE 4.1 CHECK FOR DEFLECTION FOR DIFFERENT COLUMN SECTION

No.	Checked Column Section	Area (cm^2)	Weight (kg/m)	X-Trans (mm)	Z-Trans (mm)
1	Box Section 300 x 400 x 12 mm	168.00	131.88	66.31	61.50
2	Box Section 400 x 400 x 12 mm	192.00	150.72	48.65	56.90
3	Box Section 400 x 500 x 12 mm	216.00	169.56	45.52	48.69
4	Box Section 400 x 550 x 12 mm	228.00	178.98	44.29	46.08

The permissible deflection and inter-storey drift of chosen box column section 400*500*12 mm is as shown in table 4.2. The deflected shape of the structure in x and z direction is as shown in fig.4.4 (a) and (b) respectively. It shows that it is more flexible than rigid frame having maximum inclination at top and minimum at bottom.

TABLE 4.2 DEFLECTION FOR BOX COLUMN SECTION 400 X 500 X 12 mm

Storey	Height (m)	X-Trans (mm)	Z-Trans (mm)	Inter-storey Drift (mm)	
				X - dir	Z - dir
Base	0	0.00	0.00	0.00	0.00
1	5	5.43	5.17	5.43	5.17
2	8.5	11.00	11.07	5.57	5.90
3	12	16.65	17.20	5.65	6.13
4	15.5	22.21	23.25	5.56	6.06
5	19	27.55	29.08	5.34	5.82
6	22.5	32.54	34.51	4.99	5.43
7	26	37.02	39.38	4.48	4.87
8	29.5	40.81	43.49	3.78	4.11
9	33	43.69	46.64	2.89	3.15
10	36.5	45.52	48.69	1.82	2.05

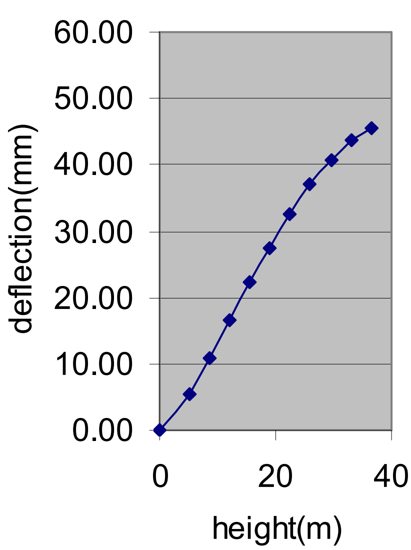


FIG. 4.4(A) DEFLECTION IN X – DIR

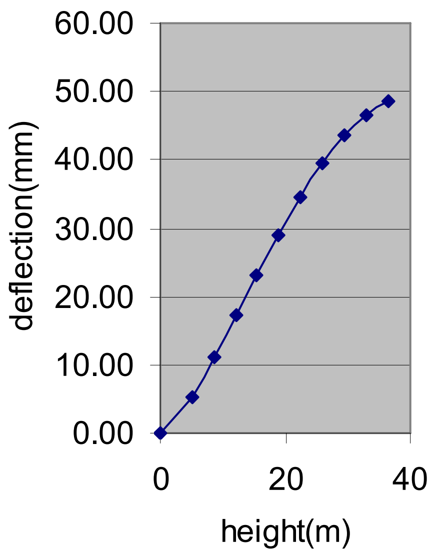


FIG. 4.4(B) DEFLECTION IN Z – DIR

4.6 ANALYSIS RESULT

4.6.1 SHEAR DISTRIBUTION

The shear distribution for the earthquake wind load is as shown in table 4.3 which is equal in all the frames. Also the base shear distribution due to earthquake is shown in fig.4.5

TABLE 4.3 SHEAR DISTRIBUTION IN Z-DIRECTION (kN)

Column	A1	B1	A2	B2	A3	B3	Storey	Base
Storey	Frame-1 Shear		Frame-2 Shear		Frame-3 Shear		Shear	Shear
1	38.1	76.9	38.1	76.9	38.1	76.9	1379.8	8.2
2	44.5	69.8	44.6	69.8	44.6	69.8	1371.6	21.8
3	43.4	69.1	43.4	69.1	43.4	69.1	1349.9	41.3
4	42.6	66.5	42.6	66.5	42.6	66.5	1308.6	65.5
5	40.9	62.7	40.9	62.7	40.9	62.7	1243.1	93.3
6	38.3	57.6	38.3	57.6	38.3	57.6	1149.8	130.8
7	34.4	50.5	34.4	50.5	34.4	50.5	1018.9	174.7
8	29.1	41.2	29.1	41.2	29.1	41.2	844.2	224.9
9	22.1	29.5	22.1	29.5	22.1	29.5	619.3	281.4
10	13.7	14.5	13.7	14.5	13.7	14.5	337.9	337.9

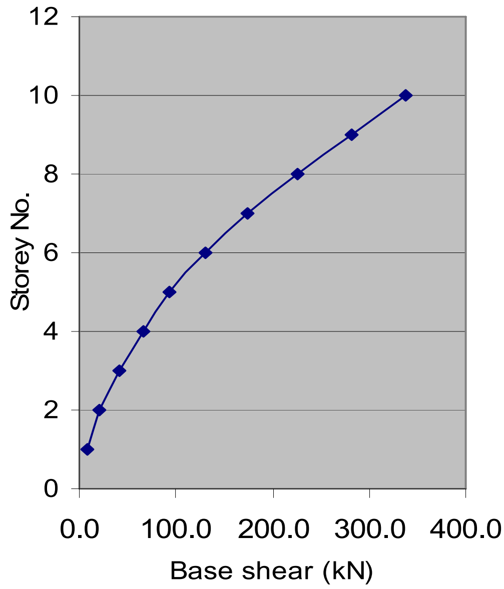


FIG 4.5 BASE SHEAR DISTRIBUTION IN Z-DIRECTION DUE TO EQ (kN)

4.6.2 BEAM FORCES AND COLUMN FORCES COMPARISION

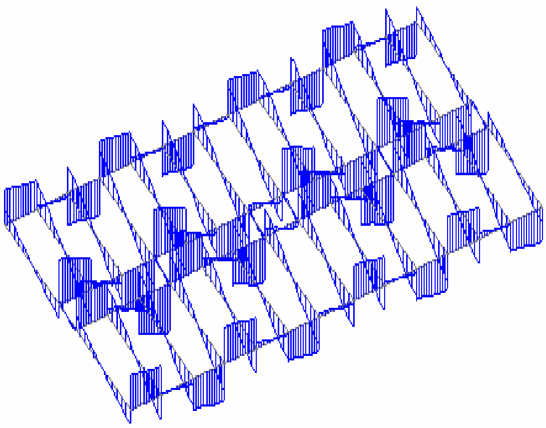
The design end shears and design end moments of beams in z and x direction are shown in table 4.4(a) and 4.4(b) respectively.

TABLE 4.4(a) DESIGN END SHEARS AND END MOMENTS OF BEAMS IN Z-DIR

Storey	Exterior Frame	Shear kN	Moment kNm	Interior Frame	Shear kN	Moment kNm
1	Beam AB	108.1	324.1	Beam AB	174.5	435.0
2		106.5	331.2		169.7	435.6
3		101.1	316.3		161.0	412.9
4		96.2	304.3		152.7	393.2
5		91.5	294.4		144.7	375.0
6		89.2	278.2		142.4	363.1
7		85.7	255.0		141.8	345.8
8		80.9	223.5		142.0	320.5
9		75.8	195.5		142.9	297.2
10		73.8	146.0		139.7	237.8
1	Beam BC	124.0	184.2	Beam BC	129.9	193.3
2		120.7	180.0		126.6	188.8
3		107.2	159.4		113.1	168.2
4		94.0	139.5		99.9	148.2
5		85.4	126.4		91.3	135.1
6		74.8	110.3		80.7	119.0
7		61.3	90.1		67.2	98.7
8		44.6	65.1		50.5	73.9
9		24.3	33.3		30.6	40.9
10		16.5	29.1		24.4	47.9

TABLE 4.4(b) DESIGN END SHEARS AND END MOMENTS FOR BEAMS IN X-DIR

Storey	Exterior Frame	Shear kN	Moment kNm	Interior Frame	Shear kN	Moment kNm
1	Beams 12 & 23	229.8	512.6	Beams 12 & 23	304.0	624.8
2		213.0	478.9		281.8	580.8
3		199.9	463.3		264.3	553.6
4		192.0	452.5		252.8	540.3
5		183.9	437.5		241.5	522.6
6		185.4	431.2		243.4	517.6
7		186.8	420.4		245.1	508.2
8		187.4	418.0		245.8	507.8
9		190.4	417.3		249.6	505.0
10		179.2	352.5		235.8	451.9
1	Beam 34	134.5	280.0	Beam 34	142.8	305.1
2		130.9	272.1		139.2	290.6
3		120.6	255.2		128.9	274.1
4		109.6	236.6		117.9	255.7
5		97.7	216.1		105.9	235.4
6		83.8	191.9		92.0	211.4
7		66.7	161.9		75.0	185.4
8		45.6	129.6		53.8	156.5
9		19.5	94.4		31.5	126.3
10		27.1	152.9		37.5	200.0



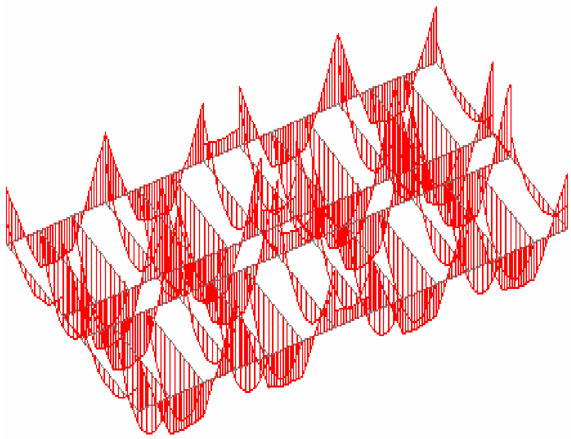


FIG-4.6 SHEARS (kN) AND MOMENTS (kNm) IN BEAMS

The shears and moments in beams for first storey are shown in fig. 4.6.

The design column forces are shown in table 4.5 (A), (B) and (C) for the frame-1, frame-2 and frame-3 respectively. The comparison of design column forces in frame A & B are shown in fig 4.7(A) & 4.7(B) respectively.

TABLE 4.5 (A) DESIGN COLUMN FORCES FOR FRAME -1

Column	ID No.	AXIAL FORCE(kN)	SHEAR-Y kN	SHEAR-Z kN	MOMENT-Y kNm	MOMENT-Z kNm
A1	1	2941.56	109.94	110.85	206.29	265.20
	111	2479.84	89.39	141.48	255.72	164.86
	221	2178.78	89.81	133.01	233.38	157.85
	331	1889.01	88.66	131.71	231.06	157.99
	441	1610.62	86.68	127.33	224.21	156.82
	551	1344.00	83.17	124.52	222.52	153.30
	661	1075.44	77.58	122.35	219.53	146.43
	771	805.11	70.52	119.53	217.47	138.01
	881	534.00	57.25	106.84	194.38	114.72
	991	259.36	66.15	160.64	324.54	146.21
B1	20	3643.41	166.58	131.12	241.69	335.60
	117	3058.66	121.94	169.58	306.42	220.97
	227	2680.88	121.57	158.89	278.70	214.56
	337	2319.64	116.85	157.24	275.46	206.01
	447	1974.79	110.82	151.91	266.84	197.60
	557	1646.61	102.71	148.93	265.48	185.81
	667	1316.99	91.79	147.19	263.05	169.34
	777	986.00	77.95	147.01	262.50	148.35
	887	654.45	58.46	136.39	248.15	115.27
	997	319.73	45.25	205.05	414.28	99.14

TABLE 4.5 (B) DESIGN COLUMN FORCES FOR FRAME -2

Column	ID No.	AXIAL FORCE(kN)	SHEAR-Y kN	SHEAR-Z kN	MOMENT-Y kNm	MOMENT-Z kNm
A2	2	5562.48	157.59	90.86	177.70	323.20
	112	4635.23	122.10	95.30	168.90	222.71
	222	4040.41	118.32	96.36	169.14	207.88
	332	3479.35	115.72	95.68	169.02	203.92
	442	2952.24	111.20	92.84	165.05	197.55
	552	2458.72	107.49	87.68	157.22	194.84
	662	1967.01	103.75	79.71	144.72	190.05
	772	1476.98	99.40	68.48	126.58	186.55
	882	987.75	84.43	53.84	102.75	155.47
	992	501.89	117.59	37.79	76.70	243.89
B2	21	6935.98	213.42	92.48	177.40	376.60
	118	5754.72	138.06	95.35	168.90	251.10
	228	5003.85	136.18	96.62	169.65	240.51
	338	4300.01	130.72	96.09	169.78	230.18
	448	3642.87	124.33	93.37	166.03	221.12
	558	3031.63	115.88	88.33	158.38	208.77
	668	2424.42	104.63	80.45	146.06	191.69
	778	1820.87	90.85	69.24	127.90	171.09
	888	1219.06	69.83	54.82	104.63	133.93
	998	624.02	104.38	41.80	73.96	192.92

TABLE 4.5 (C) DESIGN COLUMN FORCES FOR FRAME -3

Column	ID No.	AXIAL FORCE(kN)	SHEAR-Y kN	SHEAR-Z kN	MOMENT-Y kNm	MOMENT-Z kNm
A3	3	3869.07	135.57	149.49	256.10	299.30
	113	3251.15	124.72	160.09	286.60	229.36
	223	2850.72	118.17	151.77	266.46	207.46
	333	2467.42	115.45	148.02	259.61	203.20
	443	2101.46	110.35	142.39	250.92	195.93
	553	1753.17	106.30	134.75	238.77	192.68
	663	1403.21	102.33	124.16	221.58	187.49
	773	1051.77	97.78	112.53	201.78	183.65
	883	699.57	82.75	95.82	171.71	152.57
	993	344.94	114.61	103.60	199.59	238.03
B3	22	4818.48	189.45	166.59	274.54	361.60
	119	4032.32	142.30	179.59	321.49	259.54
	229	3527.20	138.22	169.81	298.36	243.73
	339	3046.74	133.37	163.92	287.57	234.85
	449	2590.95	127.17	155.58	272.28	226.18
	559	2159.98	118.94	147.97	261.72	214.19
	669	1728.22	107.86	140.28	249.03	197.38
	779	1295.74	94.25	129.70	231.93	177.11
	889	862.81	73.08	112.49	200.43	139.46
	999	428.88	76.26	130.72	251.83	163.01

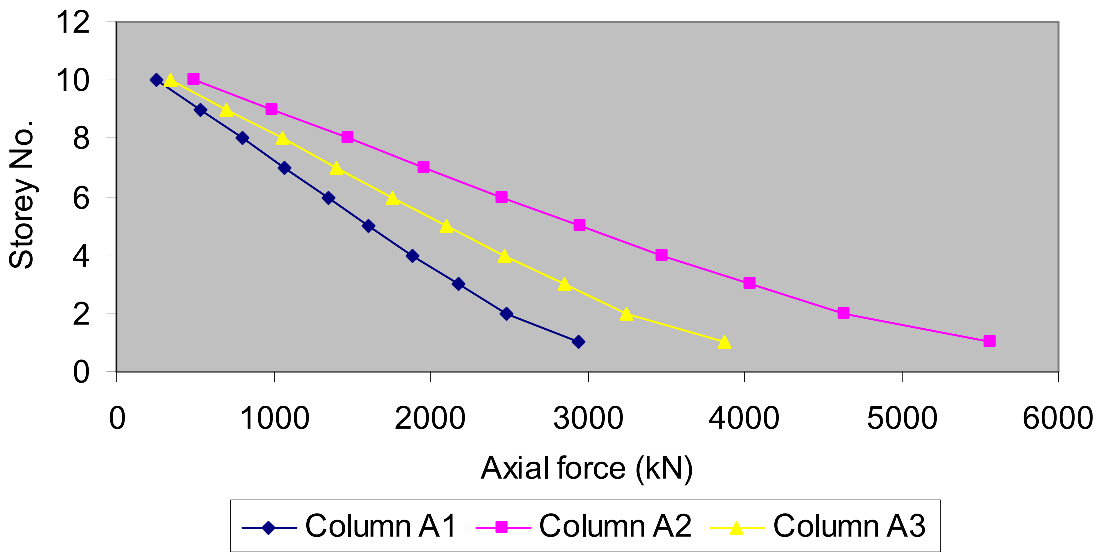


FIG. 4.7(A) COLUMN AXIAL FORCES IN FRAME A

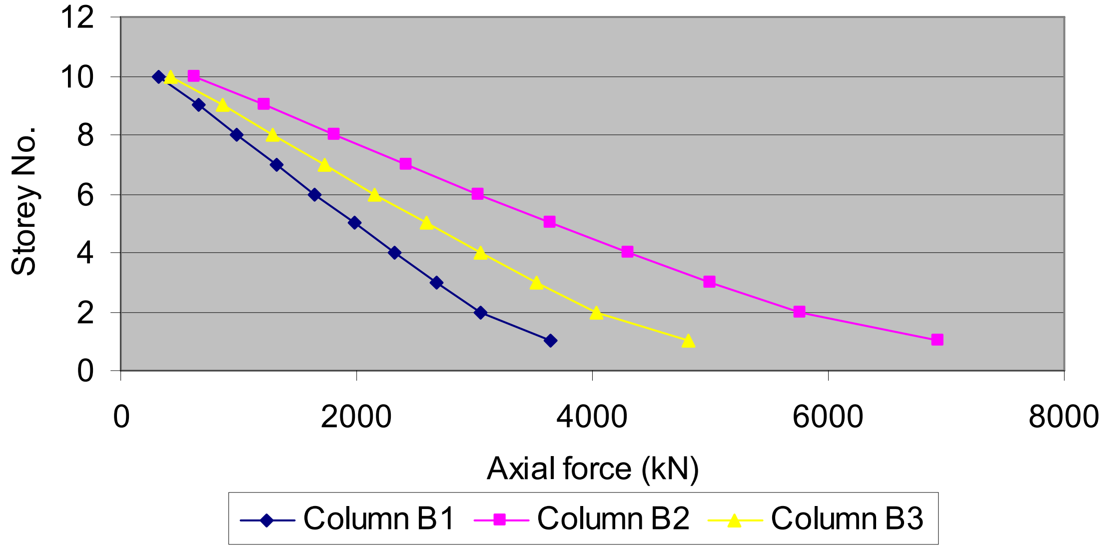


FIGURE 4.7(B) COLUMN AXIAL FORCES IN FRAME B

4.7 DESIGN

4.7.1 RC VOIDED (RIBBED) SLAB -WITHOUT HOLLOW BLOCK

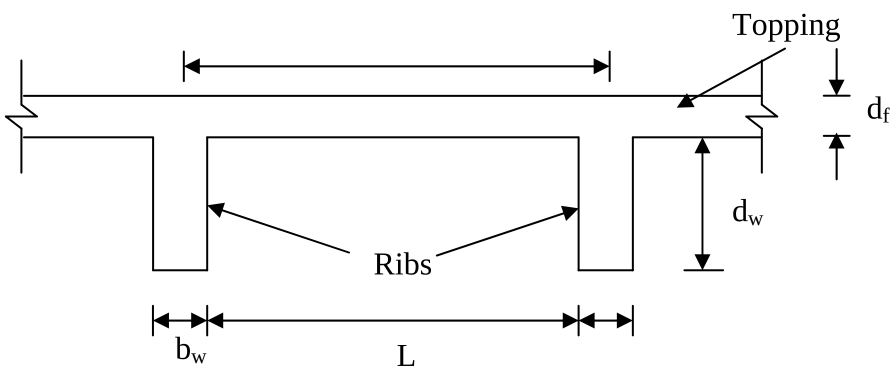


FIGURE 4.8 RC VOIDED (RIBBED) SLAB WITHOUT HOLLOW BLOCK

- The principal advantage of these types of slab is the reductions in weight as much as 15% to 30% achieved by introduction of voids in the slab. They also reduce loads on the foundations allowing more storeys to be built on the same foundations.
- For slabs without hollow blocks the minimum thickness of topping is generally recommended by BS 8110 to be 50 mm or one tenth distance between ribs, whichever is greater.

Size & Position of ribs: (As per IS- 456:2000/Cl:30.5)

1) $b_w > 65$ mm
2) $b < 1.5$ m
3) $d_w < 4 * b_w$

Let the sectional properties of slab are as follows:

1) b_w = 100 mm Check O.K.
2) b = 600 mm Check O.K.
3) d_w = 150 mm Check O.K.
4) d_f = 60 mm

$\therefore$ D = 210 mm

Load Calculation:

Design rib as T- beam

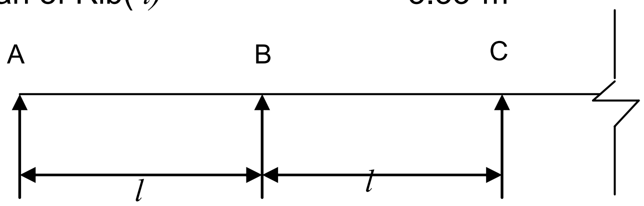
Effective flange width = c/c distance of ribs

Density of Concrete = 25 kN/m^3

		DL (kN/m^2)		LL (kN/m^2)	
1.Load on topping:					
a) Self Weight	=	1.50	+	0.00	
b) Floor Finish	=	1.00	+	0.00	
c) Live Load	=	0.00	+	4.00	
$\therefore$ Total Load	=	2.50	+	4.00	kN/m^2
		DL (kN/m)		LL (kN/m)	
2. Load on rib:					
a) topping load (b* total load)	=	1.50	+	2.40	
b) Self Weight (bw*dw*25)	=	0.38	+	0.00	
$\therefore$ Total Load	=	1.88	+	2.40	kN/m
$\therefore$ Factored Load Wu = (1.5*T.L.)	=	2.81	+	3.60	kN/m

Taking continuty advantage for rib:(From IS-456:2000/Table 36)

Span of Rib(*l*) = 3.33 m



Shear Force Coefficient:

End Support	Next to End Support		Inter. Sup.	Load	S.F.
AB	BA	BC	CB		
0.40	0.60	0.55	0.50	DL	S.F.C.
0.45	0.60	0.60	0.60	LL	*Wu*l

∴ Maximum S.F. = 12.83 kN

Bending Moment Coefficient:

Span Moments		Support Moment		Load	B.M.
Mid End Span(+ve)	Mid Interior Span(+ve)	Next to End Support(-ve)	Interior Support(-ve)		
0.083	0.063	0.100	0.083	DL	B.M.C.
0.100	0.083	0.111	0.111	LL	*Wu*l^2

∴ Max. mid span B.M. $Mud_{(+)}$ = 6.60 kNm (for internal & external span)

∴ Max. support B.M. $Mud_{(-)}$ = 7.57 kNm (for internal & external span)

∴ Eff. Depth d = 180 mm (Using 10 mm# dia bars)

Design for flexure:

∴ b_f / b_w = 6.00 &

d_f / d = 0.33

f_{ck} (N/mm^2) = 20

f_y (N/mm^2) = 415

∴ $x_{u,max} / d$ = 0.48

∴ $M_{u,lim} / f_{ck} b_w d^2$ = 0.76

∴ $M_{u,lim}$ = 49.08 kNm Check O.K. ($M_{u,lim} > M_{ud}$)

Calculation of Mid span steel, $A_{st(+)} = M_{ud(+)} / [0.87*f_y*(d-0.5*D_f)]$

∴ $A_{st(+)\,r}$ = 121.94 mm^2 Check for min.steel O.K.

∴ Provide 2 no. 10 mm# throughout at bottom

∴ $A_{st(+)\,p}$ = 157.00 mm^2 Check O.K. ($A_{st(+)\,p} > A_{st(+)\,r}$)

Calculation of Support Steel, $A_{st(-)} = p_{t(-)} * b_w * d/100$

$M_{ud(-)}/[b_w * d^2]$ = 2.34 ∴ $p_{t(-)}$ = 0.77 %

∴ $A_{st(-)\,r}$ = 138.71 mm^2 Check for min.steel O.K.

∴ Provide 2 no. 10 mm# throughout at bottom

∴ $A_{st(-)\,p}$ = 157 mm^2 Check O.K. ($A_{st(-)\,p} > A_{st(-)\,r}$)

Design for shear :

∴ Nominal Shear Stress $\tau_v = V_u/bd$

∴ τ_v (N/mm^2) = 0.71

∴ $p_{t(-)p}$ = 0.87 %

∴ β = 2.66

Design Shear Stress:

∴ τ_c (N/mm^2) = 0.59 Check $\tau_v < k^* \tau_c$ (?)

∴ k = 1.18 Result : Shear Design is necessary

Shear resisted by shear reinforcement:

$V_{us} = V_u - \tau_c * b^* d$

∴ V_{us} = 2.16 kN

∴ Vus / d = 12.01 N/mm

Req. spacing of 2 legged 6 mm Ø stirrups,

∴ Sv = 135.00 mm

∴ Provide 2 legged 6 mm Ø stirrups @ 130 mm

Check for Deflection:

∴ Basic L/D for simply-supported beam = 20

∴ Steel stress of service loads (fs) = 212.7 N/mm^2

∴ Modification factor F_1,for tension steel = 1.19

∴ Modification factor F_2,for comp. steel = 1

∴ Modification factor F_3,for T- beam = 0.8

∴ Allowable :(L/D) = $(L/D)_b*F_1*F_2*F_3$ = 19.07

& Actual (L/D) = 15.87

∴ Actual (L/D)<Allowable (L/D) ,Hence O.K.

Quantity Estimation:

∴ Rib area/m length = 0.051 m^2

∴ No. of rib for one span = 18

∴ Quantity of Concrete = 3.06 m^3

∴ Quantity of steel = 190.9 kg

Comparision of RC Voided (Ribbed) slab with & without Hollow Block:

Slab Type	bw (mm)	dw (mm)	b (mm)	df (mm)	Quantity of Concrete (m3)	Quantity of steel (kg)
without H.B.	100	150	600	60	3.06	191
with H.B.	100	160	600	60	3.12	255

Hence RC ribbed slab without hollow block is more economical and have less weight as compared with hollow block.

4.7.2 SHEAR CONNECTOR

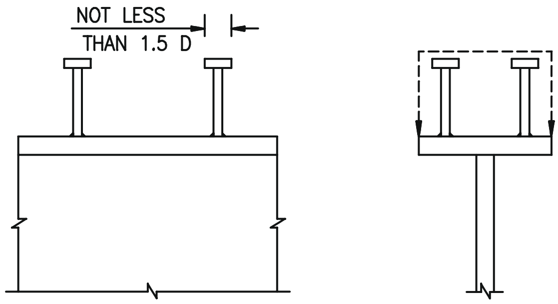


FIGURE 4.9 SHEAR (STUD) CONNECTOR

The shear connectors (as shown in fig 4.9) are designed,

- To transmit longitudinal shear along the interface between a concrete slab and steel beam
- To prevent separation of steel beam and concrete slab at the interface.

The shear connectors deform and transfer the load to concrete through bearing. In the case of continuous beam, there is a possibility of yielding in the negative moment region. So provide tension steel at support and to take account of this the negative moments is further reduced as per moment resistance design method. Also Shear connectors transfer the interfacial shear to concrete slab by thrust. Therefore to avoid splitting in concrete transverse reinforcement is provided.

Basic Design Considerations (Eurocode 4):

1) Design Method: The analysis of composite section is made using Limit State of collapse method. *IS:11384 -1985* Code deals with the design and construction of only simply supported composite beams. Therefore, the method of design for continuous beam follows *EC4.* The ultimate strength of composite section is determined from its plastic capacity, provided the elements of the steel cross section should not be semi-compact or slender as defined in the section on plate buckling.

2) Effective breadth of flange: A composite beam acts as a T-beam with the concrete slab as its flange. The ratio of the effective breadth of slab to actual breadth (b_{eff}/B) is a function of the type of loading, support condition and the section under consideration. For continuous beams ℓ_o is obtained from Fig 4.10.

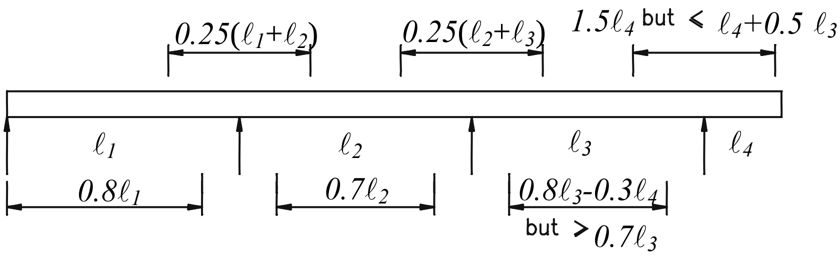


FIGURE 4.10 VALUE OF ℓ_0 FOR CONTINUOUS BEAM AS PER EC4

3) Shear Connection: At the interface of concrete and steel in a composite beam under uniform load, the elastic shear flow increases linearly from zero at the centre to maximum at the end. Once the elastic limit of connectors is reached, redistribution of forces occurs towards the less stressed connectors i.e. in the case of flexible shear connectors (i.e. stud connectors) as shown in Fig 4.11. Therefore at collapse load level it is assumed that all the connectors carry equal force, provided they have adequate shear capacity and ductility.

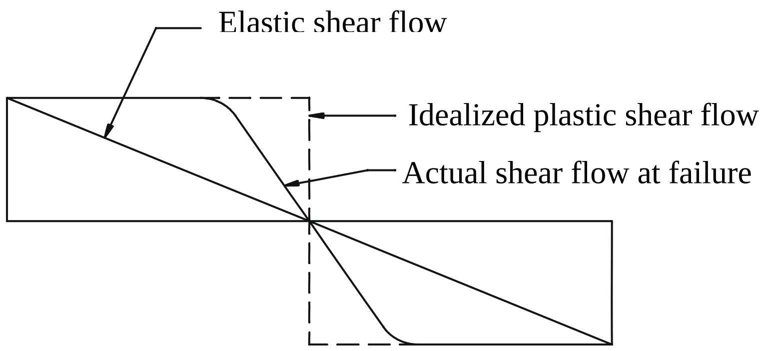


FIGURE 4.11 SHEAR FLOW AT INTERFACE

Design Calculation: (Beam A3B3)

Moment at support, $\mathbf{Mu_{(-)}}$ = **435.0** kNm

Shear at support, **Vu** = **174.5** kN

At support:

Zp req. = Mu(-) / (fy /γ_{m0}) where γ_{m0} = 1.1

∴ Zp req. = 1.91E+06 mm^3

Negative Bending Moment :

Let the Beam dimensions at support,	Hence provided beam dimension,
∴ b_f = 200 mm	∴ b_f = 200 mm
∴ t_f = 18 mm	∴ t_f = 20 mm
∴ d_w = 350 mm	∴ d_w = 350 mm
∴ t_w = 10 mm	∴ t_w = 10 mm
	∴ A_{a1} = 11500 mm^2

(a) effective width of the concrete flange, b_{eff} = $\ell_0/4 = 0.25(\ell_1 + \ell_2)/4$

(At interior support)

∴ b_{eff} = 810 mm where ℓ_1 = 10 m & ℓ_2 = 3 m

Let, provide 12 mm Ø bar @ 120 mm c/c

∴ As = 763.02 mm^2

(b) Location of neutral axis

$\gamma_a = 1.1$ & $\gamma_s = 1.15$

∴ $F_a = A_{a1} * fy/\gamma_a$ = 2613.6 kN

∴ $F_s = A_s * fsk/\gamma_s$ = 275.4 kN < Fa

Therefore NA lies in the web.

∴ a = 0.87*fy /(0.36 *$(fck)_{cu}$) = 50.15 mm

∴ Reduced Negative M.R. for I-section at support, Mu(As per EC4)

Mu >= Muc - As*fsk*(D/2 + a)/γs

∴ Redused moment at support, Mur(-)= **335.9** kNm

Design of Shear Connector:

∴ Longitudinal shear force, V_λ = Fa + Fs = 2889.0 kN

∴ Length = 5000 mm

Provide 25 mm dia. studs 100 mm high

From IS:961-1975 & for M20 concrete, P = 86 kN

Assuming full shear connection,

∴ No. of shear connectors, n_f = 18

∴ Spacing = 277.8 mm

Provide 25 mm dia. Shear Studs @ 200 mm c/c throughout as shown in fig 4.12

Transverse reinforcement :

Assuming a 0.20 % reinforcement (perpendicular to the beam)

∴ Ae = 0.002Acv = 400 mm^2/m & Acv = 200000 mm^2

Where,

η = 1 for normal weight concrete

Basic shear strength (0.25 f_{ctk}/γ_c),ζ = 0.3

f_{ctk} - the characteristic tensile strength of concrete

∴ Longitudinal shear force in the slab,

$v_r = 2.5\ A_{cv*}\eta*\zeta + A_e*f_{sk}/\gamma_s$ or $v_r = 0.2\ A_{cv*}\eta*(f_{ck})_{cy}/\gamma_c$ whichever is less

∴ Vr = 294.35 kN/m

∴ The longitudinal design shear force,

V_ℓ = 430.00 kN/m

∴ For each shear plane, V_ℓ = 215 kN/m < Vr, Hence Safe

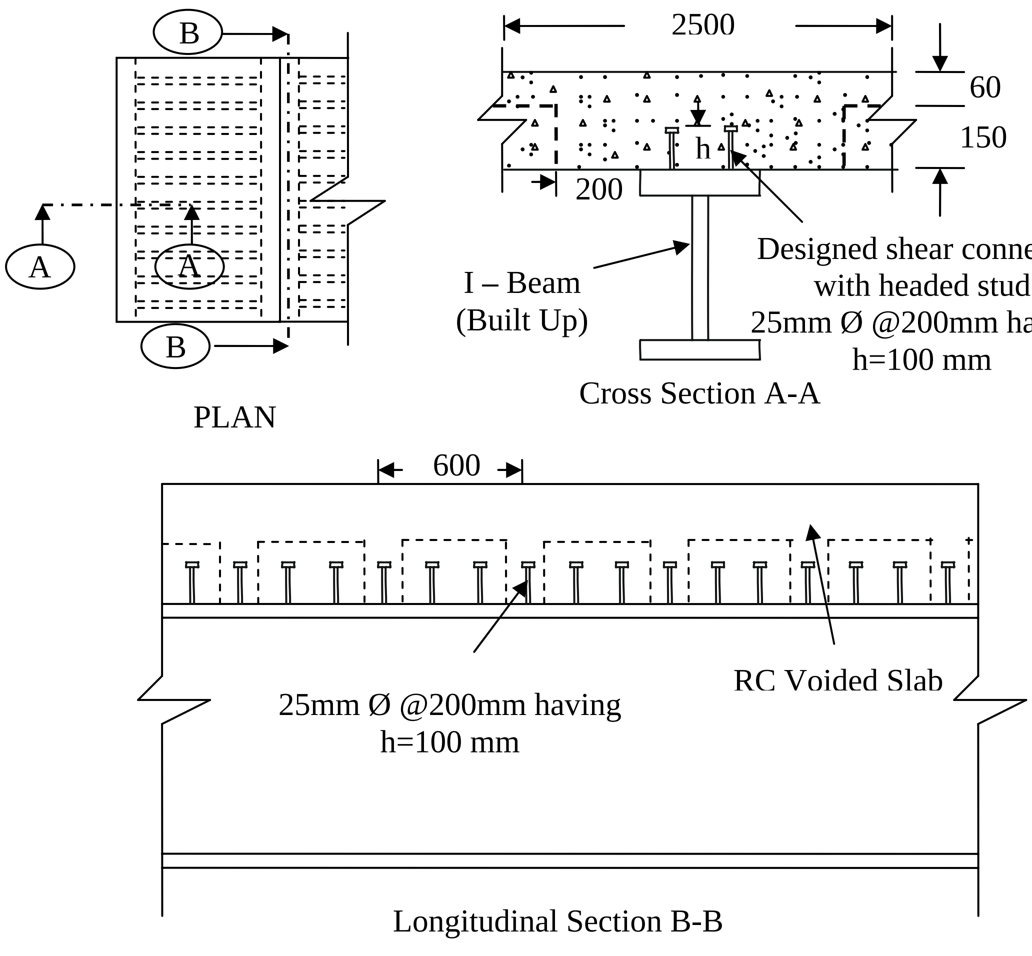


FIGURE 4.12 SHEAR CONNECTOR DESIGN

All dimensions are in mm unless specified

4.7.3 FLEXURAL MEMBER (LATERALLY RESTRAINED BEAM)

Design Consideration:

For a beam (loaded by flexure) two essential requirements must be met to develop its full moment capacity:

- The elements of the beam (i.e. flange and web) should not buckle locally
- The beam as a whole should not buckle laterally

Concept of Section Classification: As we know, for a given material and boundary conditions, the critical local buckling stress is inversely proportional to its breadth to thickness ratio (i.e. slenderness) of the plate element of a beam. Hence depending upon the slenderness of the constituent plate element of the beam, they are classified as slender, semi-compact, compact and plastic. And by reducing the slenderness of the plate elements, its resistance to local buckling could be enhanced. Once the local buckling is prevented, the beam can develop its full flexural moment capacity or the limit state in flexure. The section classifications are specified in Table 3.1/IS 800 Draft and for the I-section refer flow chart -fig. 4.16, followed later.

To avoid the lateral buckling referred to under the second condition, restraints are provided to the beam in the plane of the compression flange. In buildings, floor decks (provided on top of beams) provide restraint to the compression flange. In the absence of any such restraints, provide adequate lateral supports to the compression flange to avoid lateral buckling of beams.

Design Strength in Bending (Flexure): From IS-800 Draft/ Cl: 8.2, The factored design moment, M at any section, in a beam due to external actions shall satisfy

$$M \le M_d$$

Where, M_d =design bending strength of the section, calculated as given below.

As per IS 800 Draft/ Cl: 8.2.1.2, When the factored design shear force does not exceeds 0.6 V_d, where V_d is the design shear strength of the cross section (Cl: 8.4), the design bending strength as governed by plastic strength, M_d, shall be taken as

$$M_d = \beta_b Z_p f_y / \gamma_{m0} \leq 1.2\, Z_e f_y / \gamma_{m0}$$

Where,

β_b =1.0 for plastic and compact section

$\beta_b = Z_e / Z_p$ for semi-compact section

Z_p, Z_e = plastic and elastic section modulli of the cross section respectively

f_y = yield stress of the material

γ_{m0} = partial safety factor =1.1 (Table 5.2/IS 800 Draft)

Combined Shear and Bending:

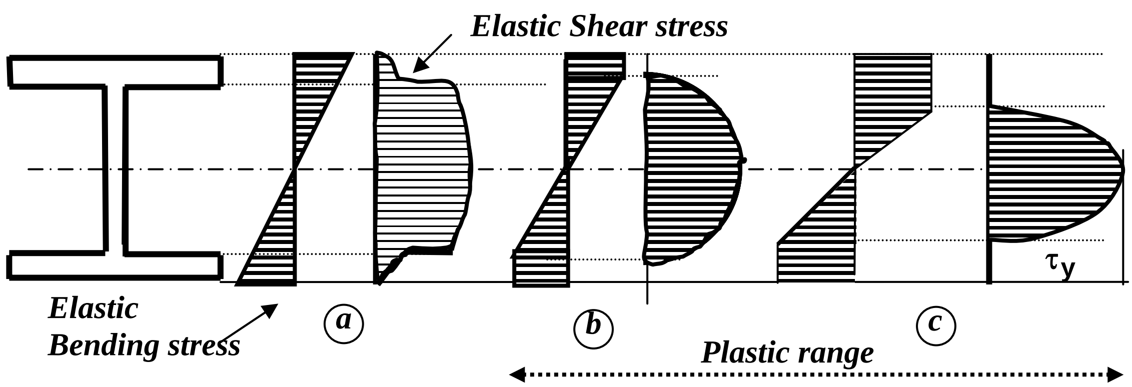
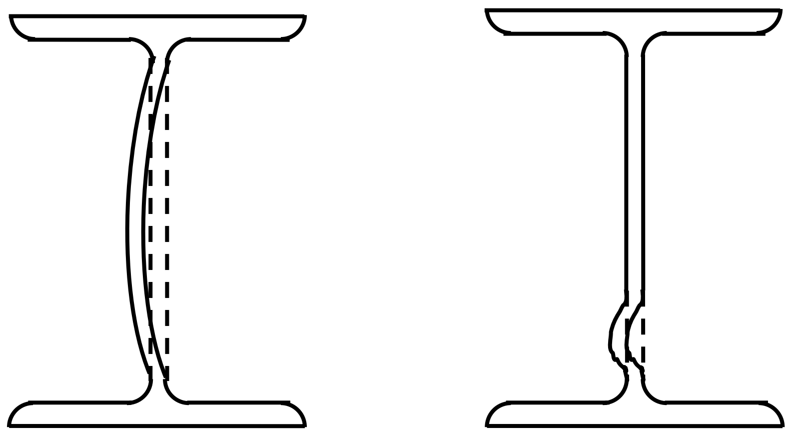


FIGURE 4.13 COMBINED BENDING AND SHEAR IN BEAMS

In I-sections, the flanges predominantly resist the moment and the webs predominantly resist the shear as shown in Fig.4.13 (a) and due to plastic redistribution of stress over the cross section, the web also contribute to the flexural action as shown in Fig.4.13 (b) & (c). Hence the shear capacity of the web gets reduced when the web has to carry a relatively high shear and also a high bending moment at the same cross section as in the case of supports of continuous beams. As larger part of the web yields in flexure, the maximum shear stress in the remaining web reaches the yield stress in shear. The code specify (IS 800:Draft/Cl:9.2.2), that if the external shear load is greater than 0.6 times the shear capacity of the web, then the effect of shear should be considered and the plastic moment capacity of the cross section should be reduced. For the value less than 0.6, no reduction is required and provides the beam with full moment capacity (IS 800 Draft/Cl:8.2.1.3) depending upon section classification.

Web Buckling and web crippling: The application of heavy concentrated loads produces a region of high compressive stresses in the web either at the support or under the load which may cause either the web to buckle (fig. 4.14a) or to cripple(fig. 4.14b). In the case of web buckling, the web may be considered as a strut restrained by the beam flanges. Such idealised strut should be considered at the points of application of concentrated load or reactions at the supports. In both the cases the load is spread out over a finite length of the web which is known as the dispersion length. The dispersion length for web buckling is taken as $(b_1 + n_1)$ where b_1 is the stiff bearing length and n_1 is the dispersion of 45° line at the mid depth of the section. Similarly in web crippling case, the dispersion length is taken as b_1+n_2, where n_2 is the length obtained by dispersion through the flange to web connection (i.e. root radius) at a slope of 1:2.5 to the plane of the flange.



(a) Web buckling *(b) Web crippling*

FIGURE 4.14 LOCAL BUCKLING OF THE WEB

As per IS 800 Draft/Cl:8.7.4, Bearing stiffeners should be provided for webs where forces applied through a flange by loads or reactions exceed the local capacity of the web at its connection to the flange as given below:

$$F_w = (b_1 + n_2) t_w f_{yw} / \gamma_{m0}$$

Where,

b_1= stiff bearing length (As per Cl: 8.7.1.3)

n_2= length obtained by dispersion through the flange to the web junction at a slope of 1:2.5 to the plane of the flange.

t_w = thickness of the web

f_{yw} = yield stress of the web

The flow chart for the flexural member design is as shown in fig. 4.15

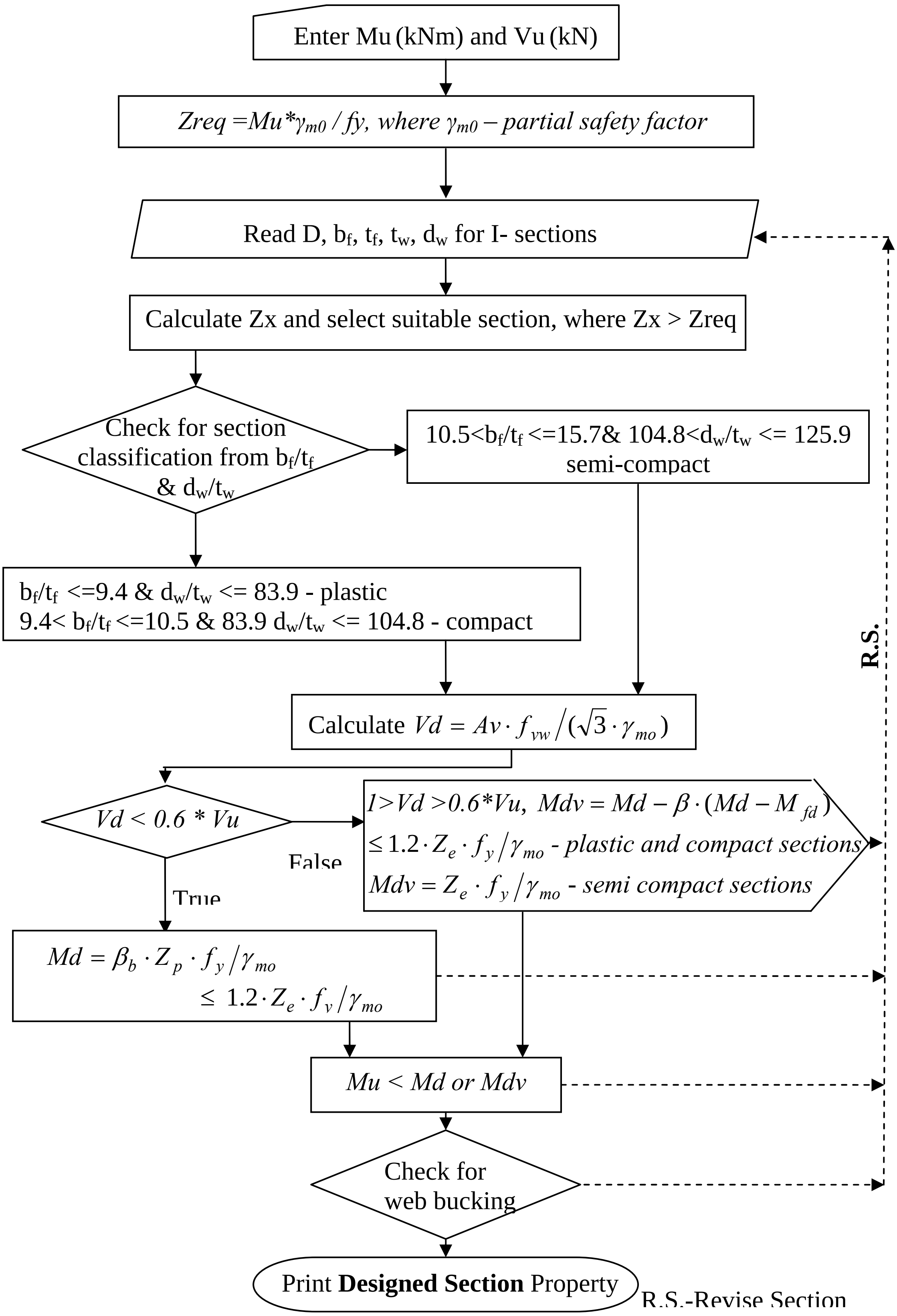
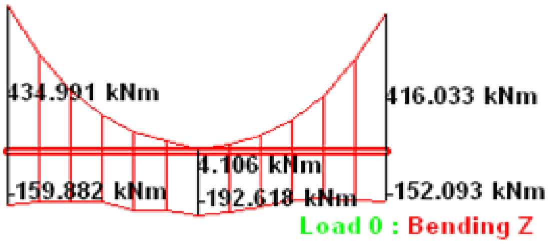
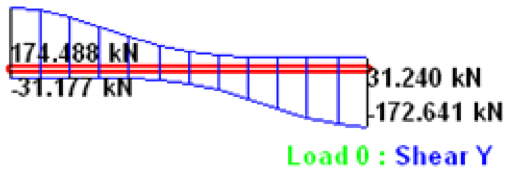


FIGURE 4.15 FLEXURAL MEMBER (LATERALLY RESTRAINED) DESIGN

Design of Flexural Member (beam) - A3B3

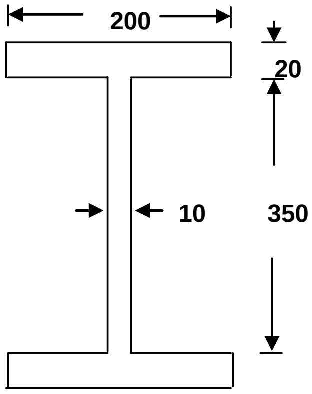
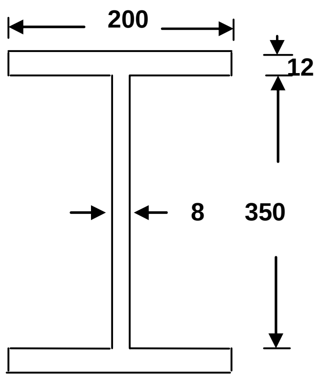


B.M. Envelope



S.F. Envelope

Moment at support, **$Mu_{(-)}$** =	**435.0** kNm
Shear at support, **Vu** =	**174.5** kN
Moment at mid span, **$Mu_{(+)}$** =	**192.6** kNm
∴ Redused moment at support, Mur(-)=	**335.9** kNm

At support:	**At mid span:**
Zp req. = Mur(-) / (fy /Υm0)	Zp req. = Mu(+) / (fy /Υm0)
∴ Zp req. = 1.48E+06 mm^3	∴ Zp req. = 8.48E+05 mm^3
	
∴ Zp pro. = 1.79E+06 mm^3	∴ Zp pro. = 1.11E+06 mm^3
> Zp req. Hence O.K.	> Zp req. Hence O.K.

Check for section classification(Table 3.1):

∴ $SC_1 = b_f/(2t_f)$ = 5.0	∴ $SC_1 = b_f/(2t_f)$ = 8.3
< 8.4, Hence O.K.	<8.4, Hence O.K.
∴ $SC_2 = d/t_w$ = 35.0	∴ $SC_2 = d/t_w$ = 43.8
< 83.9, Hence O.K.	< 83.9, Hence O.K.

Check for serviceability(Cl:8.6.1):

Minimum web th., t_{min} = d/200	
∴ t_{min} = 1.8 mm	∴ t_{min} = 1.8 mm
< tp, Hence O.K.	< tp, Hence O.K.

All dimensions are in mm unless specified

Check for Shear(Check for web buckling - Cl:8.4.2):

d/tw = 35 < 67, Hence provide bearing stiffner at supports only.

Using Simple Post-Critical Method (Cl:8.4.2.2/IS 800 Draft),

The transverse stiffeners are provided only at supports,

$\therefore K_v$ = 5.35

& The yield strength of the web, fyw = 227.27 N/mm^2

The elastic critical shear stress of the web,

$$\tau_{cr} = \frac{k_v \pi^2 E}{12(1-\mu^2)(d/t_w)^2} = 788.65 \text{ N/mm}^2$$

The non -dimensional web slenderness ratio for shear buckling stress,

$$\therefore \lambda_w = \sqrt{f_{yw}/(\sqrt{3}\,\tau_{cr,e})} = 0.41 < 0.8$$

Shear stress corresponding to buckling,

$\therefore \zeta_b$ = 131.22 N/mm^2

$\therefore$ Design Shear, Vd =Vcr = d* tw*ζ_b = 459.26 kN

$\therefore$ Vu / Vd = 0.38 < 0.6, Hence O.K.

Design of End Bearing Stiffner(Cl:8.7.5.2):

Design as a Column i.e. for compression force due to bearing and moment.

$\therefore F_b$ = 174.49 kN & c = 10 m

$\therefore F_m$= M/c = 43.50 kN, where c = spacing between the stiffner

$\therefore$ Total, Fc = 217.99 kN

Effective sectional area of stiffner in contact with flange,Ae > Aq i.e.0.8Fc/f_{yq}

$\therefore$ Aq = 767.32 mm^2

Try stiffner of 2 flats of size 90 x **8** mm thick

Check for outstand(Cl:8.7.1.2):

Outstand= 90 mm < 20*thickness of the stiffner, Hence O.K.

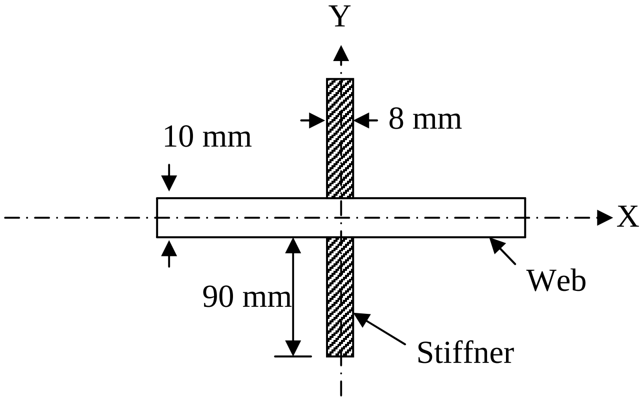


FIGURE 4.16 END BEARING STIFFNER

Where, f_{yq} = yield stress of the stiffener = fy/γ_{m0}

Check Stiffner for Buckling(Cl:7.1):

∴ I_x = 4.57E+06 mm^4 Ae = 1440 mm^2 >Aq, Hence safe

& $r_x = \sqrt{(I_x/Ae)}$ = 56.35 mm & Le = 0.7*ℓ

∴ Le/rx = 4.35

∴ From Cl:7.1/IS800 Draft, Design Compressive Strength *Pd = Ae * fcd*

∴ $\lambda = \sqrt{(fy*(Le/r_x)^2/(3.14^2E))}$ = 0.05 & α = 0.49

∴ $Ø = 0.5*[1+\alpha(\lambda-0.2)+\lambda^2]$ = 0.46

∴ $f_{cd} = fy/\gamma_{m0}/[Ø+(Ø^2 - \lambda^2)]^{0.5}$ = 245.49 N/mm^2

∴ *Pd = Ae * fcd =* 353.50 kN > Fc, Hence safe

Where,

λ = non-dimensional effective slenderness ratio

Le/r_x = effective slenderness ratio, ratio of effective length Le, to appropriate radius of gyration r_x

α = imperfection factor

f_{cd} = design stress in compression

Check as a Bearing Stiffner:

∴ n_2 = 50 mm

∴ Local capacity of web, $F_w = (b_1 + n_2)t_w f_{yw} / \gamma_{m0}$ = 113.64 kN

∴ Design bearing stiffner force,Fa = Fc - Fw = 104.35 kN

∴ Bearing capacity of stiffner, Pa =Ae* f_{yw} = 327.27 kN

> Fa, Hence O.K.

4.7.4 BEAM FLANGE & WEB SPLICES – BOLTED CONNECTION

The bolts used in steel structures are of three types:

- Black Bolts
- Turned and Fitted Bolts
- High Strength Friction Grip (HSFG) Bolts

For the bolts grade *x.y* (as per the I.S.O.), *x* indicates one-tenth of the minimum ultimate tensile strength of the bolt in kgf/mm^2 and *y* indicates one-tenth of the ratio of the yield stress to ultimate stress, expressed as a percentage. e.g., 4.6 grade bolts will have a minimum ultimate strength 40 kgf/mm^2 (392 Mpa) and minimum yield strength of 0.6 times 40 i.e. 24 kgf/mm^2 (235 Mpa).

The HSFG bolts provide efficient connections which perform well under fluctuating/fatigue load conditions. These bolts should be tightened to their proof loads and require hardened washers to distribute the load under the bolt heads. The HSFG connections are designed such that under service load the force does not exceed the frictional resistance to avoid the relative slip during service. When the external force exceeds the frictional resistance the plates slip until the bolts come into contact with the plate and start bearing against the hole. Beyond this point the external force is resisted by the combined action of the frictional resistance and the bearing resistance. As per Cl:12.4.1, All bolts used in frames resisting earthquake loads shall be fully tensioned, High Strength Friction Grip (HSFG) bolts, in standard holes.

Codal provisions:

As per Cl:10.4.3, *Slip Resistance* –Design for friction type bolting in which slip is required to be limited, a bolt subjected only to a factored design shear force, *Vsf,* in the interface of connections at which slip cannot be tolerated, shall satisfy the following:

$$V_{sf} \leq V_{nsf} / \gamma_{mf}$$

Where,

Vnsf = nominal shear capacity of a bolt as governed by slip for friction type connection, calculated as follows:

$$Vnsf = \mu f \; ne \; Kh \; Fo$$

Where,

μf = coefficient of friction (slip factor) as specified in Table 10.2 ($\mu f \leq 0.55$)

= 0.2 (for surfaces not treated)

ne = number of effective interfaces offering frictional resistance to slip

Kh = 1.0 for fasteners in clearance holes

γmf = 1.25 (designed at ultimate load)

Fo = minimum bolt tension (proof load) at installation - 0.8 *Asb fo*

Asb = shank area of the bolt

fo = proof stress (576 MPa for $\phi \leq 24$ and 510 MPa for $\phi > 24$ for 8.8 grade)

As per Cl:10.4.4, *Bearing Resistance* – Design for friction type bolting, in which bearing stress in the ultimate limit state is required to be limited, (*Vbf* = factored load bearing force) shall satisfy

$$Vbf \leq Vnbf / \gamma mf$$

Where,

Vnbf = bearing capacity of a bolt, for friction type connection, calculated as follows:

$$Vnbf = 2.2 \; d \; t \; fup \leq 3 \; d \; t \; fyp$$

Where,

fup = ultimate tensile stress of the plate

fyp = tensile yield stress of the plate

d = nominal diameter of the bolt

t = summation of thicknesses of all the connected plates experiencing bearing stress in the same direction

Let flange splice carries the moment and web splice carries the shear.

Design of Flange Splice:

∴ Flange Force, F = M /(d-t_f) = 406.5 kN

For Gr.8.8 HSFG bolts in double shear with **22** mm dia. **2** rows

Fo = 0.8 Asb fo = 175.08 kN (As per Cl:10.4.3)

Nominal shear capacity of a bolt as governed by slip(i.e. Slip Resistance)

for friction type connection, Vnsf = μf ne Kh Fo

∴ V_{nsf}/γ_{mf} = 56.0 kN

& Bearing Resistance per bolt(As per Cl:10.4.4), Vnbf = 2.2 d t fup,

∴ V_{nbf}/γ_{mf} = 317.5 kN

∴ Bolt Value = 56.0 kN

Provide 4 bolts of 2 rows.

∴ Net area of flange, $A_{net} = (b- 2*d_h)*t_f$ = 3060 mm^2

where, d_h = hole diameter = bolt dia + 1.5

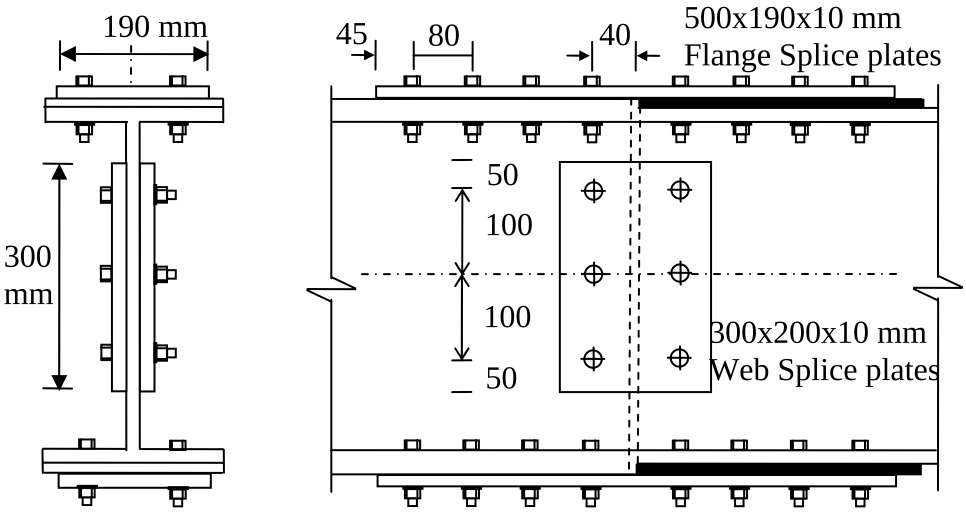
∴ Flange capacity, Fp = A_{net} * fy /γ_{m0} = 695.5 kN **> F, Hence safe**

∴ Thickness required, $t_{req} = F * \gamma_{m0}/\{(b_c - 2* d_h)*fy\}$

where, b_c = width of flange plate = 190 mm

∴ t_{req} = 14.2 mm

Use Flange splice plate of size 650 x 190 x 16 mm



(a) Cross Section (b) Longitudinal Section

FIGURE 4.17 BEAM FLANGE AND WEB SPLICES

Design of Web Splice:

For Gr.8.8 HSFG bolts in double shear with **22** mm dia. **2** rows

∴ V_{nsf} / γ_{mf} = 56.0 kN

Try 10 mm thick web splice plates on both sides of the web.

∴ V_{nsf} / γ_{mf} = 158.8 kN

∴ Bolt Value = 56.0 kN

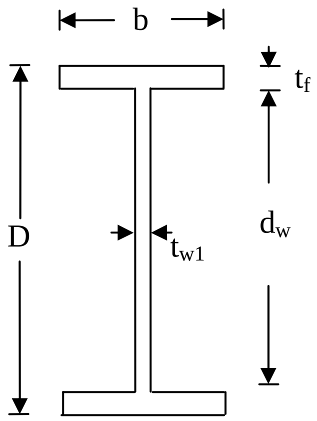
Try 3 bolts at 100 mm vertical pitch and 50 mm from the center of joint.

∴ Horizontal S.F. on bolt due to
moment due to eccentricity = 30.0 kN

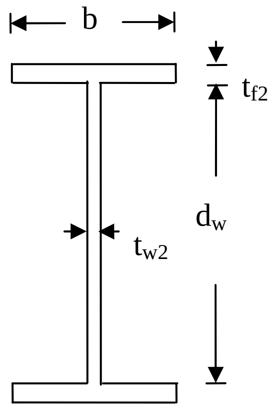
∴ Vertical S.F. per bolt = 40.0 kN

∴ R = 50.0 kN < Fb, Hence O.K.

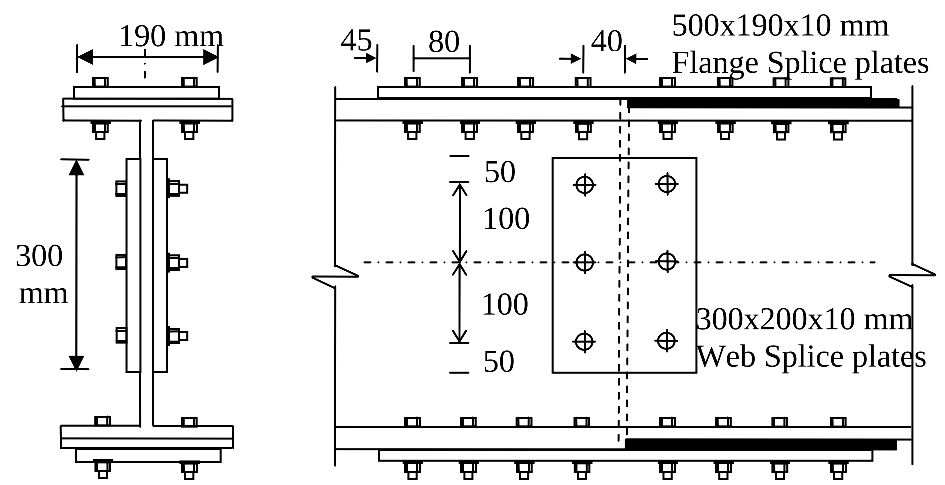
Use web splice plate of size 300 x 200 x 10 mm



BEAM C/S AT SUPPORT



BEAM C/S AT MID SPAN



(a) Cross Section (b) Longitudinal Section

FLANGE AND WEB SPLICES OF BEAM A3B3

No.	Beam Members	**Beam Details**						**Section Modulus**				**Bearing Stiffner**		**Flange Plate Splices**						**Web Plate Splices**						**Moment of Inertia**
		b_f	t_f	t_{f2}	d_w	t_{w1}	t_{w2}	$Z_{R(-)}$	$Z_{P(-)}$	$Z_{R(+)}$	$Z_{P(+)}$	Size(mm)		Bolt Detail			Size mm			Bolt Detail			Size mm			
		mm						$*10^6$ mm^3				bs	ts	Rows	Dia.(mm)	No.	Lf	Bf	Tf	No.	Dia.(mm)	Rows	Dw	Bw	Tw	I (mm^4)
1	$A2B2_{(1-4)}$,$A3B3_{(1-4)}$,$C2D2_{(1-4)}$,$C3D3_{(1-4)}$	200	20	12	350	10	8	1.5	1.8	0.8	1.1	90	8	2	22	4	650	190	16	3	22	2	300	200	10	3.75E+07
2	$A2B2_{(5-7)}$,$A3B3_{(5-7)}$,$C2D2_{(5-10)}$,$C3D3_{(5-10)}$ $A2B2_{(8-10)}$,$A3B3_{(8-10)}$,$A1B1_{(1-7)}$,$C1D1_{(1-7)}$,$34_{(1-10)}$	175	20	12	300	8	8	1.2	1.3	0.7	0.8	80	8	2	22	4	650	165	18	3	22	2	250	170	8	1.94E+07
3	$A1B1_{(8-10)}$,$C1D1_{(8-10)}$	150	16	10	250	8	8	0.6	0.8	0.4	0.5	70	12	2	20	4	650	140	18	3	20	2	210	150	8	1.12E+07
4	BC	150	12	-	250	6	-	0.5	0.6	-	-	70	8	-	-	-	-	-	-	-	-	-	-	-	-	8.33E+06
5	$B1B2_{(1-2)}$,$B2B3_{(1-2)}$	225	20	16	450	10	10	2.3	2.6	1.5	2.2	100	10	2	30	4	650	215	25	4	30	2	400	200	10	7.84E+07
6	$B1B2_{(3-10)}$,$B2B3_{(3-10)}$,$A1A2_{(1-7)}$,$A2A3_{(1-7)}$	200	20	12	450	10	8	1.9	2.4	1.3	1.5	90	10	2	30	4	650	190	30	4	30	2	325	170	10	7.81E+07
7	$A1A2_{(8-10)}$,$A2A3_{(8-10)}$	175	16	12	400	10	8	1.4	1.6	0.9	1.2	80	10	2	30	4	650	165	30	4	30	2	325	170	10	5.46E+07

Note:

1. All checks i.e. check for section classification, check for section modulus, check for shear, check for bearing stiffener, check for outstand, check for buckling and check for flange capacity are satisfied for all the beams mentioned in the table.
2. The beam c/s at support section is attached to beam c/s at mid span with the beam flange and web splices i.e. 0.25L from support.
3. The bearing stiffeners are provided at support and at the application of point loads.
4. Notations:
   Lf, Bf, Tf, Lw, Bw and Tw is the length, width and thickness of beam flange and web splices respectively.
   $Z_{R\,(-)}$, $Z_{P\,(-)}$, $Z_{R\,(+)}$ and $Z_{R\,(+)}$ is the beam section modulus required and provided for the c/s at support and mid span respectively.
5. All the steel properties are confirming to Cl:2.2.4/IS 800 Draft and all the bolts are 8.8 grade HSFG bolts.

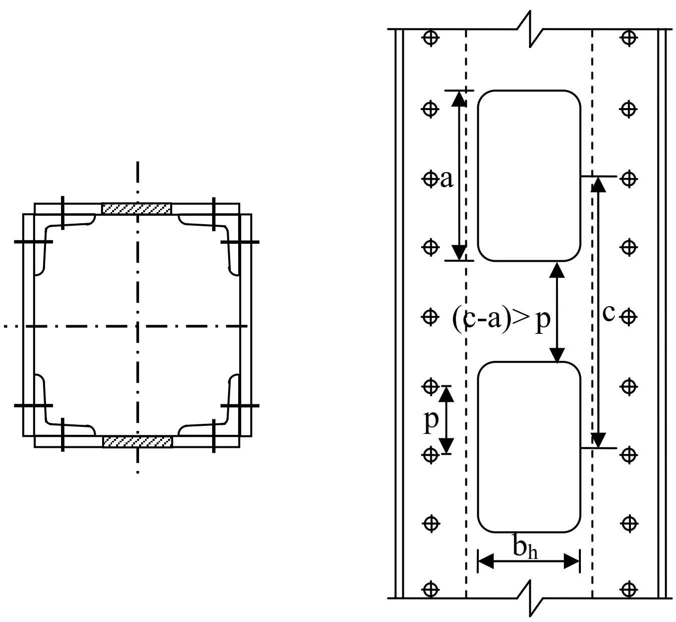
TITLE	**BEAM DETAILS**			
PROJECT	DESIGN OF 10 STOREYED STEEL BUILDING			
NAME:	HARI K. DESAI		GUIDE:	Prof. G.N.PATEL
ROLL NO.	04MCL003	DATE:	12/6/2005	SIGN:

4.7.5 BEAM- COLUMN

The member subjected to axial compression and bending is referred to as a beam-column. When a multi-storey multi-bay un-braced frame is subjected to gravity loads and lateral loads due to wind or earthquake, the columns are subjected to sway deflection and bending. In such cases, the columns experience axial compression as well as bending moments.

In the design of beam column different sections are used such as I-section ,I-section with angle sections, 2-ISHB with top & bottom cover plates, Welded box section using plates and Box section with 4-ISA having perforated cover plates. The box column with perforated plates (as shown in fig. 4.18) gives more economical and practicable solution. 'Guide to design criteria for metal compression members' published by Column Research Council- U.S.A., has given the recommendations for the design of perforated plates as follows:

- The clear distance (c-a) between perforations should not be less than the unsupported distance between the nearest lines of connecting bolts.
- The net section of the column (i.e. gross cross section less the section of the perforations) should be used in computing the axial rigidity and the sectional moments of inertia about the X and Y axes.
- At the point of perforation, each flange should be designed to resist the total transverse shear force.



(a) Plan (b) Elevation

FIGURE 4.18 BOX COLUMN WITH PERFORATED PLATES

Here the perforations afford access for bolting and painting. Also the portions of the perforated cover plate outside the perforations are included in the member area. So it has superior stiffness and straightness as compared to laced or battened columns. It also reduces the fabrication and maintenance cost.

Design Codal Provisions:

As per Cl: 9.3.1.1, For Section Strength (for Plastic and Compact Sections) — In the design of members subjected to combined axial force (tension or compression) and bending moment, the following interaction relationship shall be satisfied.

$$\left(\frac{M_y}{M_{ndy}}\right)^{\alpha_1} + \left(\frac{M_z}{M_{ndz}}\right)^{\alpha_2} \leq 1.0$$

Where,

M_y, M_z = factored applied moments about the minor and major axis of the cross section, respectively

M_{ndy}, M_{ndz} = design reduced flexural strength under combined axial force and the respective uniaxial moment acting alone, (For Rectangular Hollow sections and Welded Box sections, symmetric about both axis)

$$M_{ndz} = M_{dz}\,(1-n) / (1-0.5a_w) \leq M_{dz}$$

$$M_{ndy} = M_{dy}\,(1-n) / (1-0.5a_f) \leq M_{dy}$$

N = factored applied axial force (Tension T, or Compression F)

N_d=design strength in compression

$$N_d = A_g f_y / \gamma_{m0}$$

M_{dy}, M_{dz} = design strength under corresponding moment acting alone (as state earlier in flexural member design)

A_g = gross area of the cross section (net area if perforations are provided)

α_1, α_2 = constants as given in Table 9.1 = 1.66/ (1-1.13n^2) ≤ 6

As per Cl: 9.3.2, Overall Member Strength (for overall buckling failure) Bending and Axial Compression – Members subjected to combined axial compression and biaxial bending shall satisfy the following interaction relationship.

$$\frac{P}{P_d}+\frac{K_y M_y}{M_{dy}}+\frac{K_z M_z}{M_{dz}}\le 1.0$$

Where,

K_y, K_z = moment amplification factor about minor and major axis respectively

P = factored applied axial compression

M_y, M_z = maximum factored applied bending moments about y and z axis of the member, respectively.

P_d, M_{dy}, M_{dz} = design strength under axial compression, bending about y and z axis respectively, as governed by overall buckling as given below:

a) The design compression strength is the smallest of the minor axis (P_{dy}) and major axis (P_{dz}) buckling strength

b) Design bending Strength M_{dz} about major axis as given below

$$M_{dz}=M_d$$

Where,

M_d = design flexural strength about z axis given by Cl: 8.2.1, when lateral torsional buckling is prevented and by Cl: 8.2.2, where lateral torsional buckling governs

For flexural buckling failure

$$K_z = 1-\frac{\mu_z P}{P_{dz}} \le 1.5$$

Where,

μ_z is larger of μ_{LT} and μ_{fz} as given below:

$$\mu_{LT}=0.15\lambda_y \beta_{MLT}-0.15 \quad \le 0.90$$

$$\mu_{fz} = \lambda_z \left(2\beta_{Mz} - 4\right)+\frac{Z_{pz}-Z_{ez}}{Z_{ez}} \quad \le 0.90$$

$$K_y = 1-\frac{\mu_y P}{P_{dy}}$$

$$\mu_y = \lambda_y\left(2\beta_{My}-4\right)+\left[\frac{Z_{py}-Z_{ey}}{Z_{ey}}\right]\le 0.9$$

β_{My}, β_{Mz}, β_{MLT} = equivalent uniform moment factor obtained from Table 9.2, according to the shape of the bending moment diagram between lateral bracing points in the appropriate plane of bending

$= 1.8\text{-}0.7\psi$

P_{dy}, P_{dz} = design compressive strength as governed by flexural buckling about the respective axis

$$P_d = A_e f_{cd}$$

Where,

A_e = effective sectional area

f_{cd} = design stress in compression

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}} \quad = \quad \chi f_y / \gamma_{m0} \quad \leq f_y / \gamma_{m0}$$

Where,

$\phi = 0.5[1+\alpha(\lambda - 0.2) + \lambda^2]$

λ_y, λ_z = non-dimensional slenderness ratio about the respective axis

$= \sqrt{f_y / f_{cc}} = \sqrt{f_y \left(KL/r\right)^2 / \pi^2 E}$

f_{cc} = euler buckling stress $= \pi^2 E/(KL/r)^2$

KL/r = effective slenderness ratio, ratio of effective length KL, to appropriate radius of gyration r

α = imperfection factor = 0.34

χ = stress reduction factor

c) *Design Bending Strength about Minor axis*

$$M_{dy} = M_d$$

Where,

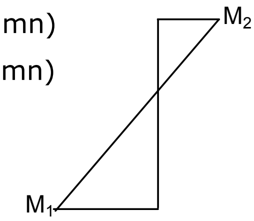
M_d = design flexural strength about y-axis calculated using plastic section modulus for plastic and compact sections and elastic section modulus for semi-compact sections.

Design of Beam Column:(Alterate-1)

	Pu kN	Mz_1 kNm	Mz_2 kNm	My_1 kNm	My_2 kNm	**Result:**
						Check- I
						0.045
Case-I (Max. comp.)	**6936.0**	23.8	109.6	5.9	5.9	**<1, Safe**
Case-II (Max. Mz)	3600.5	**376.6**	251.0	0.4	0.3	**Check- II**
Case-III (Max.My)	3877.2	88.9	94.2	**177.4**	141.6	**1.01**
						O.K.

Preliminary Design:

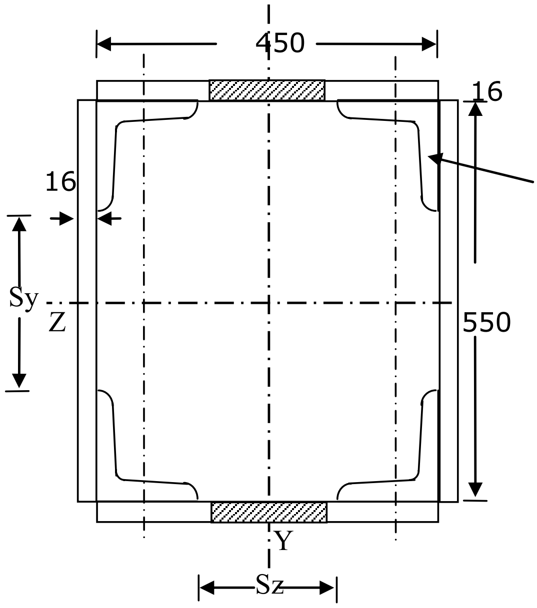
BMD



Assume the section considering P_{EQ} = 1.5P (For Uniaxial Column)
& 1.75P (For Biaxial Column)

$\therefore$ P_{EQ} = 9710.4 kN

$\therefore$ From Cl:7.1.2, Ae = P_{EQ}/fcd, Ae = 42726 mm^2



ISA110x110x15

Column B2

Sectional Properties:

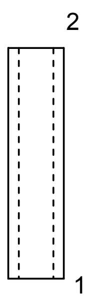
A =	39524	mm^2
I_z =	1.95E+09	mm^4
I_Y =	1.50E+09	mm^4
Ze_z =	7.09E+06	mm^3
Ze_Y =	6.65E+06	mm^3
Zp_z =	8.12E+06	mm^3
Zp_Y =	8.09E+06	mm^3
r_z =	222.1	mm
r_y =	**194.6**	mm
S_Z =	230	mm
S_Y =	330	mm
b_h =	150	mm
a =	300	mm
c =	450	mm

Check for section classification(Table 3.1):

$\therefore$ $b_f / (t_f)$ = 28.9 < 29.3

$\therefore$ d / t_w = 34.4 < 40

Hence cross-section is 'Plastic'



Effective Length of Column(Appendix E):

The effective length factor K of column in sway frames:

$$K = \left[\frac{1 - 0.2(\beta_1 + \beta_2) - 0.12\beta_1\beta_2}{1 - 0.8(\beta_1 + \beta_2) + 0.6\beta_1\beta_2} \right]^{0.5} \quad \text{Where} \quad ,\beta = \frac{\Sigma K_c}{\Sigma K_c + \Sigma K_b}$$

Stiffness K_c, K_b = C (I / L)

Where, C= connection factor(Table 5.1)for far end condition

= 0.67(1-0.4*n) (for far end fixed),where n = P/Pe

For Ground Floor Column

P = 6935.98 kN

L_{C1} = 5000 mm& L_{C2} = 3500 mm

At 2 @ z- axis: K_{C1} = 2.57E+05 mm^3 & K_{C2} = 3.70E+05 mm^3

L_{b1}= 10 m, I_{b1} = 3.75E+07 mm^4, K_{b1} = 3.74E+03 mm^3

L_{b2}= 3 m, I_{b2} = 8.33E+06 mm^4, K_{b2} = 2.77E+03 mm^3

∴ β_2 = 0.99

& β1 = 0

At 2 @ y- axis: K_{C1} = 1.96E+05 mm^3 & K_{C2} = 2.83E+05 mm^3

L_{b3}= 10 m, I_{b3} = 7.84E+07 mm^4, K_{b3} = 7.83E+03 mm^3

L_{b4}= 10 m, I_{b4} = 7.84E+07 mm^4, K_{b4} = 7.83E+03 mm^3

∴ β_2 = 0.97

& β1 = 0

∴ K_z = 1.96 & K_y = 1.89

∴ Le_z = 9813 mm & Le_y = 9459 mm

Check For Slenderness Ratio(Table 3.2):

∴ $\lambda_z = Le_z / r_z$ = 44.18 **<180, Hence O.K.**

∴ $\lambda_y = Le_y / r_y$ = 48.60 **<180, Hence O.K.**

Check for section strength (Cl: 9.3.1):

$$R_1 = \left(\frac{M_y}{M_{ndy}}\right)^{\alpha_1} + \left(\frac{M_z}{M_{ndz}}\right)^{\alpha_2} \leq 1.0$$

The elastic lateral buckling moment(Cl;8.2.2),

$$M_{cr} = \frac{\beta_{LT}\pi^2 EI_y h}{2(KL)^2}\left[1+\frac{1}{20}\left[\frac{KL/r_y}{h/t_f}\right]^2\right]^{0.5}$$

∴ Mcrz = 12037.89 kNm

The non-dimensional slenderness ratio, $\lambda_{LT} = \sqrt{\beta_b Z_p f_y / M_{cr}}$

∴ λ_{LTZ} = 0.41

& $\phi_{LT} = 0.5\left[1+\alpha_{LT}(\lambda_{LT}-0.2)+\lambda_{LT}^2\right]$

∴ $Ø_{LTZ}$ = 0.62

The reduction factor, $\chi_{LT} = \frac{1}{[\phi_{LT} + [\phi_{LT}^2 - \lambda_{LT}^2]^{0.5}]} \leq 1.0$

∴ X_{LTZ} = 0.92

The design bending compressive stress, $f_{bd} = \chi_{LTZ}\ f_y / \gamma_{m0}$

$\therefore f_{bdz} =$ 209.50 MPa

The design bending strength, $M_d = \beta_b Z_p f_{bd}$

$\therefore M_{dz} =$ 1701.7 kNm & $M_{dy} =$ 1838.8 kNm

$$N_d = A_g f_y / \gamma_{m0}$$

$\therefore N_d =$ 8982.73 kN

$\therefore n = N/N_d =$ 0.77 & $a_w = (2*A_{ISA} + 2*d*t_w)/A =$ 0.5 ≤ 0.5

$a_f = (2*A_{ISA} + 2*b*t_f)/A =$ 0.4 ≤ 0.5

$\therefore M_{ndz} = M_{dz}(1-n) / (1-0.5a_w) =$ 517.00 kNm ≤ Mdz

$\therefore M_{ndy} = M_{dy}(1-n) / (1-0.5a_f) =$ 523.33 kNm ≤ Mdy

$\therefore M_z =$ 109.6 kNm & $M_y =$ 5.9 kNm,

$\alpha_1 =$ 6 & $\alpha_2 =$ 6

$\therefore$ **R_1 = 0.04 <1, Hence Safe**

Check for Overall Member Strength(Cl:9.3.2):

$$R_2 = \frac{P}{P_d} + \frac{K_y M_y}{M_{dy}} + \frac{K_z M_z}{M_{dz}} \leq 1.0$$

λ' = non-dimensional effective slenderness ratio(Cl:7.1.2.1)

$\therefore \lambda_z' =$ **0.50** & $P_{dz} =$ 7586.341 kN

& $\lambda_y' =$ **0.55** & $P_{dy} =$ 7333.213 kN

$\therefore P_d =$ **7333.2** kN

$\therefore Mz_2 =$ 23.8 kNm & $M_{z1} =$ 109.6 kNm

& $My_2 =$ 5.9 kNm & $M_{y1} =$ 5.9 kNm

$$K_z = 1 - \frac{\mu_z P}{P_{dz}} \qquad K_y = 1 - \frac{\mu_y P}{P_{dy}}$$

& $\mu_z = \min(\mu_{LT}, \mu_{fz})$ $\quad \boldsymbol{\mu_{LT}} = 0.15\boldsymbol{\lambda_y \beta_{MLT}} - 0.15$, $\mu_{fz} = \lambda_z(2\beta_{Mz} - 4) + \dfrac{Z_{pz} - Z_{ez}}{Z_{ez}}$

& $\mu_y = \lambda_y(2\beta_{My} - 4) + \left[\dfrac{Z_{py} - Z_{ey}}{Z_{ey}}\right] \leq 0.9$

$\therefore \psi_z =$ -0.22 & $\psi_y =$ -0.99

$\therefore \beta_{MLT} =$ 1.95 & $\beta_{Mz} =$ 1.95 & $\beta_{My} =$ 2.49

$\therefore \mu_z =$ 0.01 & $\mu_y =$ 0.12

$\therefore K_z =$ 0.99 & $K_y =$ 0.89

$\therefore P/P_d =$ **0.95** & $K_y M_y / M_{dy} =$ 0.003 & $K_z M_z / M_{dz} =$ 0.064

$\therefore$ **R_2 = 1.01 ≈ 1.0, Hence O.K.**

Check for Shear : Fz = 92.5 kN & Fy = 213.4 kN

About z:

For Gr.8.8 HSFG bolts in single shear with **22** mm dia.(As per Cl:10.4.3)

∴ Bolt Value = 28.0 kN

∴ No. of bolts required = 3.30 <4, Hence O.K.

About Y:

For Gr.8.8 HSFG bolts in single shear with **30** mm dia.(As per Cl:10.4.3)

∴ Bolt Value = 54.9 kN

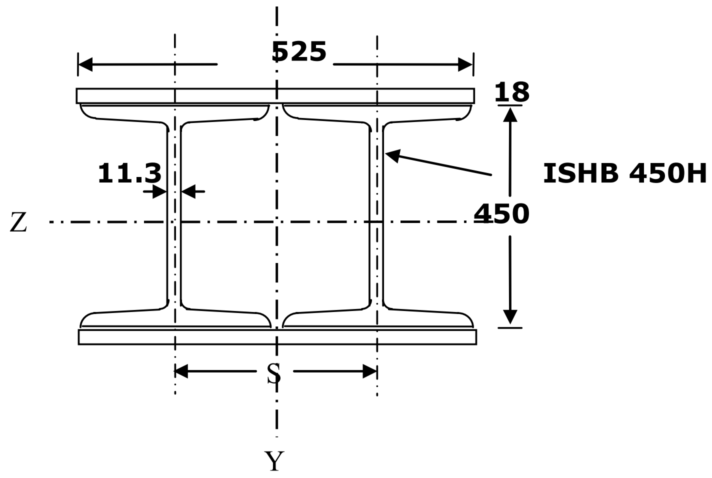
∴ No. of bolts required = 3.89 <4,Hence O.K.

Check for clear distance :

∴ Clear Distance beween perforation (450-300)= 150 mm

> Pitch Of Rivet = 120 mm

Design of Beam Column:(Alterate-2)



Sectional Properties:

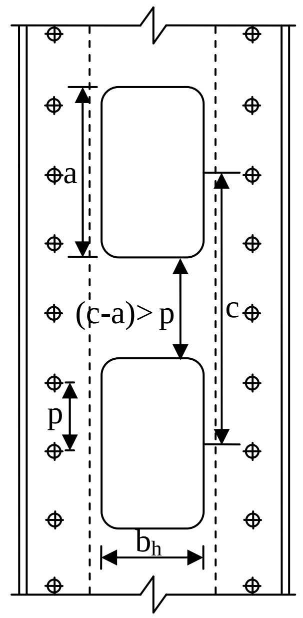
A =	42478	mm^2
I_z =	1.84E+09	mm^4
I_Y =	9.41E+08	mm^4
Ze_z =	7.58E+06	mm^3
Ze_Y =	3.58E+06	mm^3
Zp_z =	1.27E+07	mm^3
Zp_Y =	4.54E+06	mm^3
r_z =	208.3	mm
r_y =	**148.8**	mm
c =	25	mm
s =	**275**	mm
$t_{f(ISHB)}$=	13.7	mm
$b_{(ISHB)}$=	250	mm

TABLE 4.6 COMPARISION OF BEAM COLUMN DESIGN ALT-I & II

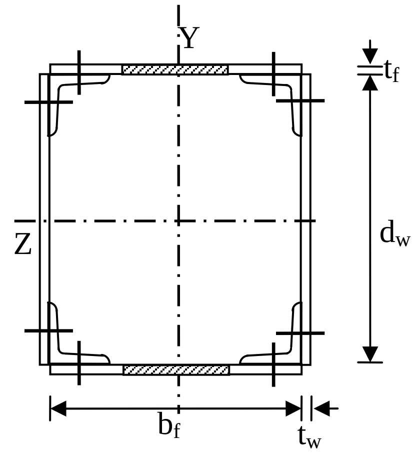
Comparision	**ALT-I**	**ALT-II**
A (mm^2) =	**39524**	**42478**
r_y (mm) =	**194.6**	**148.8**
λ_z =	44.2	45.9
λ_y =	**48.6**	**60.8**
P_{dz} (kN) =	7586.3	8048.5
P_{dy} (kN) =	**7333.2**	**7088.1**
P/P_d =	**0.95**	**0.98**
K_yM_y/M_{dy} =	0.003	0.005
K_zM_z/M_{dz} =	0.06	0.04
Check- II (R_2)	**1.01**	**1.03**

Hence, due to high radius of gyration about y, alternate-I gives economical design section than alternate- II. So such type of section is preferable as a beam column.

COLUMN	**A_1**		**A_2**		**B_2**		**A_3 & B_1**		**B_3**	
b_f (mm)	350		400		450		350		400	
t_f (mm)	12		16		16		16		16	
d_w (mm)	400		500		550		400		450	
t_w (mm)	12		16		16		16		16	
Angle	ISA100x100x12		ISA110x110x12		ISA110x110x15		ISA100x100x12		ISA100x100x12	
b_h (mm)	100		120		150		100		150	
A (mm^2)	24636		34968		39524		29836		31436	
r_z (mm)	163.0		201.4		222.1		162.6		181.2	
r_Y (mm)	148.9		173.0		194.6		150.0		172.6	
$SC_1 = b_f/t_f$	29.2	O.K.	28.1	O.K.	28.9	O.K.	24.6	O.K.	28.1	O.K.
$SC_2 = d/t_w$	33.3	O.K.	31.3	O.K.	34.4	O.K.	25.0	O.K.	28.1	O.K.
Effective Length										
At 2 @ z-axis										
I_{b1} (mm^4)	1.90E+07		3.75E+07		3.75E+07		3.75E+07		3.75E+07	
I_{b2} (mm^4)	0.00E+00		0.00E+00		8.33E+06		0.00E+00		8.33E+06	
At 2 @ y-axis										
I_{b3} (mm^4)	7.81E+07		7.81E+07		7.84E+07		7.81E+07		7.84E+07	
I_{b4} (mm^4)	0.00E+00		7.81E+07		7.84E+07		1.94E+07		1.94E+07	
K_z & K_Y	1.97	1.86	1.97	1.85	1.96	1.89	1.95	1.86	1.93	1.89
L_{ez}&L_{ey}(m)	9.84	9.29	9.85	9.26	9.81	9.46	9.74	9.28	9.66	9.46
Check for Overall Member Strength										
P/P_d	0.689		0.893		0.946		0.785		0.870	
K_zM_z/M_{dz}	0.105		0.101		0.066		0.111		0.075	
K_yM_y/M_{dy}	0.192		0.003		0.003		0.040		0.049	
R_2	**0.98 O.K.**		**1.00 O.K.**		**1.01 O.K.**		**0.94 O.K.**		**0.99 O.K.**	



ELEVATION



PLAN

BOX COLUMN WITH PERFORATED PLATES

Note:

1) All checks i.e. check for section strength, check for slenderness ratio, check for overall member strength, check for shear and check for clear distance between perforations are satisfied for all the columns mentioned in the table.
2) Notations:
   SC_1 and SC_2 is the section classification of flange and web of column.
   I_{b1}, I_{b2}, I_{b3} and I_{b4} is the moment of inertia of connecting beam @ z and y axis at top of column (i.e. at 2) respectively.
   K_Z and K_Y is the effective length factor @ z and y axis respectively.
   a = 300 mm, c = 450 mm and p =120 mm such that (c-a) > p
3) All the steel properties are confirming to Cl:2.2.4/IS 800 Draft and all the bolts are 8.8 grade HSFG bolts.

TITLE	**COLUMN DETAILS**		
PROJECT	DESIGN OF 10 STOREYED STEEL BUILDING		
NAME:	HARI K. DESAI	GUIDE:	Prof. G.N.PATEL
ROLL NO.	04MCL003	DATE:	12/6/2005	SIGN:	

4.7.6 BEAM- COLUMN CONNECTIONS

The Moment-resistant connections, as shown in fig. 4.19, are used for fully continuous structures where the connections are designed to resist full moments. The efficiency of a welded connection is fully utilized in this type of connection.

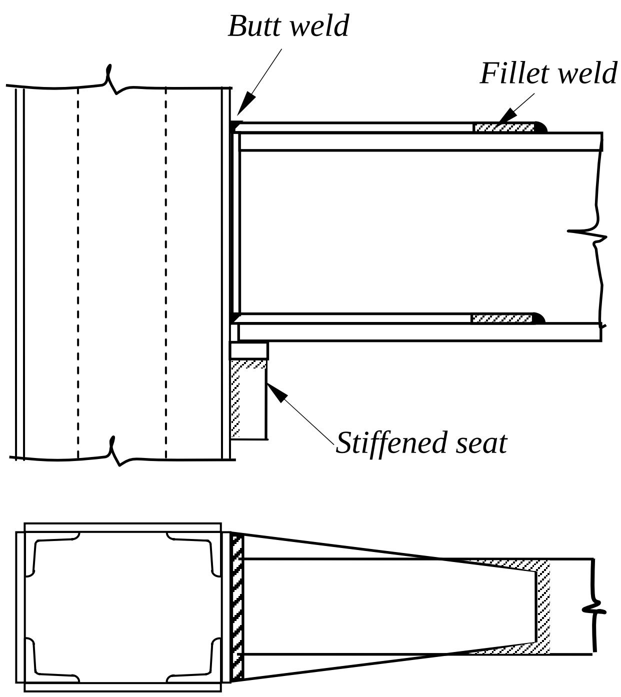


FIGURE 4.19 MOMENT RESISTING CONNECTION

For these connections, the tensile force (caused by hogging moments) at the top flange of the beam is transferred to the top flange plate by fillet welds and from the plates to the column by groove welds. Full penetration butt welds are formed when the parts are connected together within the thickness of the parent metal. Groove welds have high strength, high resistance to impact and cyclic stress. As per Cl:12.4.2, All welds used in frame resisting earthquake loads shall be complete penetration butt welds, except in column splice.

The top plate is often provided a taper for ease in welding. When the lateral forces are considered, both of the top and bottom plates are to be designed for tension considering the reversal of stresses and the welds must be designed to resist both tension and compression. While providing moment-resisting welded connections, it is usual to provide butt weld that beam flanges remain flush with the column at one end and connect the beam at the other end as per connection details as described above. Permissible stresses for butt welds are assumed same as for the parent metal with a thickness equal to the throat thickness.

Design Codal Provisions:

As per Cl: 10.5.7.1.1, the design strength of a fillet weld, *fwd*, shall be based on its throat area.

$$fwd = fwn / \gamma mw \text{ in which } \quad fwn = f_u / \sqrt{3}$$

Where,

fu = smaller of the ultimate stress of the weld and the parent metal

γmw = partial safety factor (Table 5.2)

= 1.25 (For Shop Fabrications)

= 1.50 (For Field Fabrications)

As per Cl:10.5.9, For Stresses Due to Individual forces – When subjected to either compressive or tensile or shear force alone, the stress in the weld is given by:

$$fa \text{ or } q = \frac{P}{t_t \, l_w}$$

Where,

fa = calculated normal stress due to axial force in N/mm^2

q = shear stress in N/mm^2

P = force transmitted (axial force *N* or the shear force *Q*)

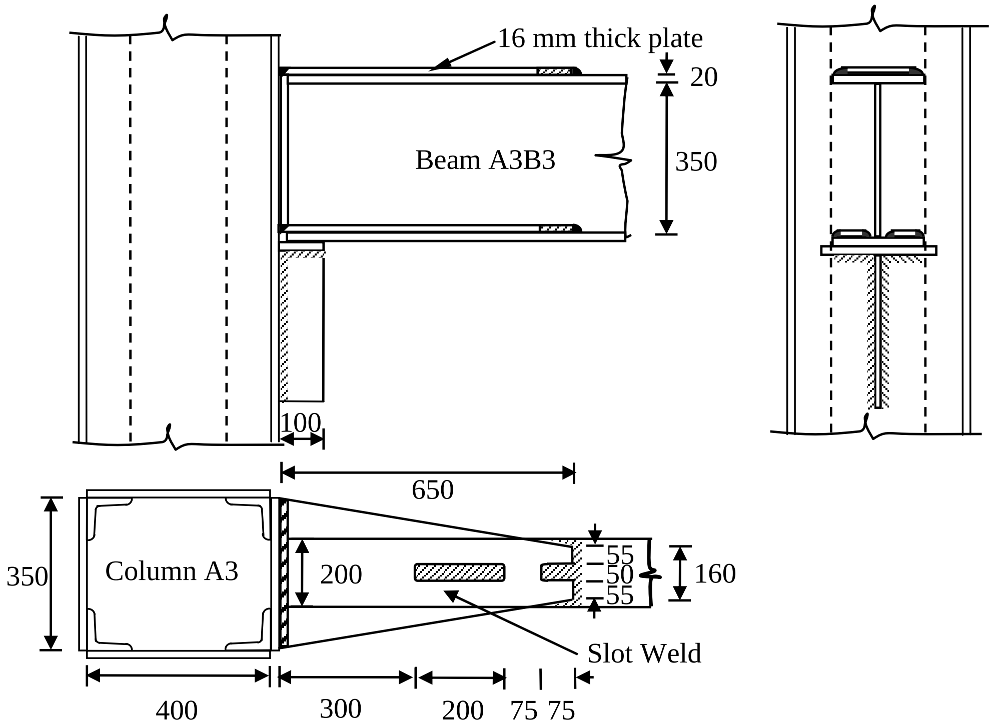
t_t = effective throat thickness of weld in mm

l_w= effective length of weld in mm

As per Cl: 10.5.2.3, the size of fillet welds shall not be less than 3 mm. The minimum size of the first run or of a single run fillet weld shall be as given in Table10.3, to avoid the risk of cracking in the absence of preheating and also as per Cl: 10.5.3.1, the effective throat thickness of a fillet weld shall generally not exceed 0.7*t,* and 1.0*t* under special circumstances, where *t* is the thickness of the thinner plate of elements being welded. As per Cl: 10.5.3.3, the effective throat thickness of a complete penetration butt weld shall be taken as the thickness of the thinner part joined.

Design of Beam – Column Connection:

∴ Mu = **335.90** kNm & Vu = **174.49** kN



BEAM (A3B3) - COLUMN (A3) JOINT

All dimensions are in mm unless specified.

For the connection as shown in fig., the tensile force (caused by hogging moments) at the top flange of the beam is transferred to the top flange plate by fillet welds and from the plates to the column by full penetration butt welds.

∴ Force in Cover Plate F = M/d_b = 861.3 kN

& Required plate area, A = $(F * \gamma_{mw} / fy)$= 5168 mm^2

γ_{mw} = partial safety factor = 1.5 (for field fabrication)

The effective throat thickness of a complete penetration butt weld shall be taken as the thickness of the thinner part joined (Cl: 10.5.3.3)

Width of beam flange = 200 mm

∴ Width of cover plate at column joint, b_j = 350 mm (equal to width of column)

∴ Thickness of cover plate, t = A/b = 14.8 mm

∴ Provide cover plate of size 350 x 16 mm connected by butt weld to column flange.

Minimum size of fillet weld specified in (IS:800 Draft/Table 10.3) for

16 mm thick plate 5 mm and shall not exceed 0.7t

where t is the thickness of the thinner plate of elements being welded (Cl:10.5.3.1)

∴ The fillet weld thickness = 16 mm - equal to top flange plate thickness of beam as it is subjected to wind & eq loads not decrease by 1.5 mm (Cl:10.5.8.4)

∴ Design strength of fillet weld, $f_{wd} = fu/(\sqrt{3} * \gamma_{mw}) =$ 96.2 N/mm^2

where ,

fu = smaller of the ultimate stress of the weld and the parent metal

∴ Total Length of weld required = $F/0.7*t*f_{wd}$ = 799 mm

∴ Width of cover plate at end, b_e = 160 mm

∴ Provide the slot weld of width 50 mm (≥3T or 25 mm) as shown in fig . such that total weld length provided is more than length required, having rounded corners with radius 25 mm (≥1.5T or 12 mm) and with clear distance between two consecutive holes 75 mm (≥ 2T or 25 mm)

∴ Use 200 mm length interior slot and 75 mm length end slot

∴ Total Length of weld provided Lw_p = 810 mm >Lwr, Hence O.K.

The unwelded length equal to width of plate is necessary to impart ductility to the connection so provide unwelded length = $(b_j + b_e)/2$ = 300 mm

∴ Total Length of cover plate = 650 mm

Design of Stiffened Seat:

Beam web thickness, tw = 10 mm

As per Cl:8.7.4/IS 800 Draft, Safe Crripling length, $B = V_u * \gamma_{m0} /(f_{yw} * t_w)$

∴ $B = b_1 + n_2$ = 76.8 mm

∴ Safe bearing length $b_1 = B - n_2 = B - 2.5\ t_f$ = 26.776 mm

< B/2 Hence adopt minimum B/2

∴ b_1 = 38.4 mm

Assuming a clearance of 12 mm between the beam end and the column,

∴ Width of seat plate = 50.4 mm

Use b = 100 mm

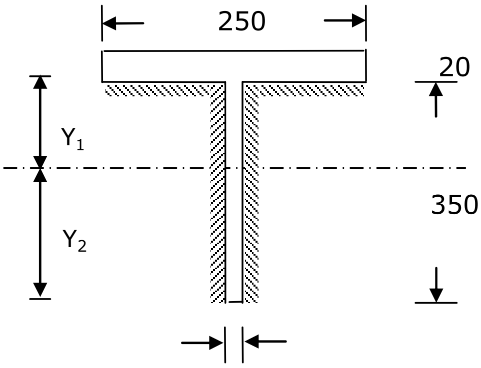
Provide thickness of seat plate (equal to beam flange thickness) = 20 mm

& thickness of stiffening plate (equal to beam web thickness) = 10 mm

Width of seat plate = 250 mm

Depth of seat plate = 350 mm

Design of fillet weld:



All dimensions are in mm unless specified.

$\therefore$ e = b - b_1/2 = 80.8 mm

$\therefore$ M = Vu * e = 14.1 kNm

$\therefore$ Y_1 = 130.3 mm & Y_2 = 219.7 mm

$\therefore$ I_{xx} = 5.49E+06 mm^4

Horizontal shear per mm length, $q_h = M*Y_2 /I_{xx}$ = 563.81 N/mm

Vertical shear per mm length, q_v = Vu/ Total length of weld = 185.63 N/mm

$\therefore$ Resultant shear per mm length of weld, $q = \sqrt{(q_h^2 + q_v^2)}$ = 593.58 N/mm

$\therefore$ Size of weld,s = q /(0.7*f_{wd}) = 8.81 mm

$\therefore$ Provide fillet weld size = 10 mm

4.7.7 COLUMN SPLICE

As per Cl:12.5.2.1/IS 800 Draft, a Partial-joint penetration groove weld may be provided in column splice, such that the design strength of the joints shall be at least equal to 200% of the required strength.

As per Cl:12.5.2.2/IS 800 Draft, The minimum required strength for each flange splice shall be 0.6 times f_yA_f as shown Fig. 4.20.

Where,
A_f = total flange area of the smaller connected column

$P_{min} = 0.6\ f_yA_f$ $P_{min} = 0.6\ f_yA_f$

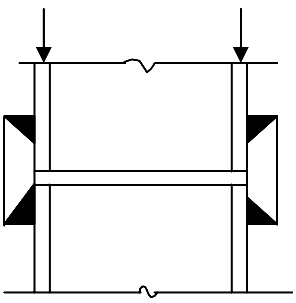


FIGURE 4.20 PARTIAL PENETRATION GROOVE WELD IN COLUMN SPLICE

Design Calculation:

Design forces for column B2 at top of second storey, considering maximum axial compression, maximum Mz & maximum My, are as follows:

	Pu kN	Mz kNm	My kNm	Fy kN	Fz kN
Case-I (Max. comp.)	5003.85	115.40	4.26	67.93	2.62
Case-II (Max. Mz)	2898.99	240.51	2.50	136.18	1.56
Case-III (Max.My)	3088.92	57.72	169.65	32.53	96.62

Area of Box Section = 39524 mm^2

Area of @ z-axis (flange) = 15762 mm^2

Area of @ y-axis (web) = 23762 mm^2

Splice @ Z-Axis (Flange Splice):

P_{min} = 0.6*fy*Af = 1182.2 kN

Portion of load carried by flange = 0.5*P *Af /A+ Mz/d+0.5* Fz

∴ P_Z = 999.3 kN < 0.6*fy*Af (i.e. Pmin)

Hence design for P_{min}(Cl:12.5.2.2)

∴ Design Force, F_{dz}= P_Z*2/1.5= 1576.2 kN (Cl:12.5.2.1)

& Required plate area, A = (F * γ_{mw} / fy)= 9457.2 mm^2

Provide the plate width, bz = 450 mm

∴ thickness of flange plate, t_f = A/bz = 25 mm

∴ Providing partial groove weld of size 20 mm (i.e. 3/4* tf)

∴ Total Length of weld required, L_{wr} = F/0.7*t*f_{wd} = 1170 mm

∴ depth of plate required = (L_{wr} -2 *b_z) = 270.1 mm

∴ Provide the plate depth = 300.0 mm

∴ Use plate size of 450 x 300 x25 mm as shown in fig

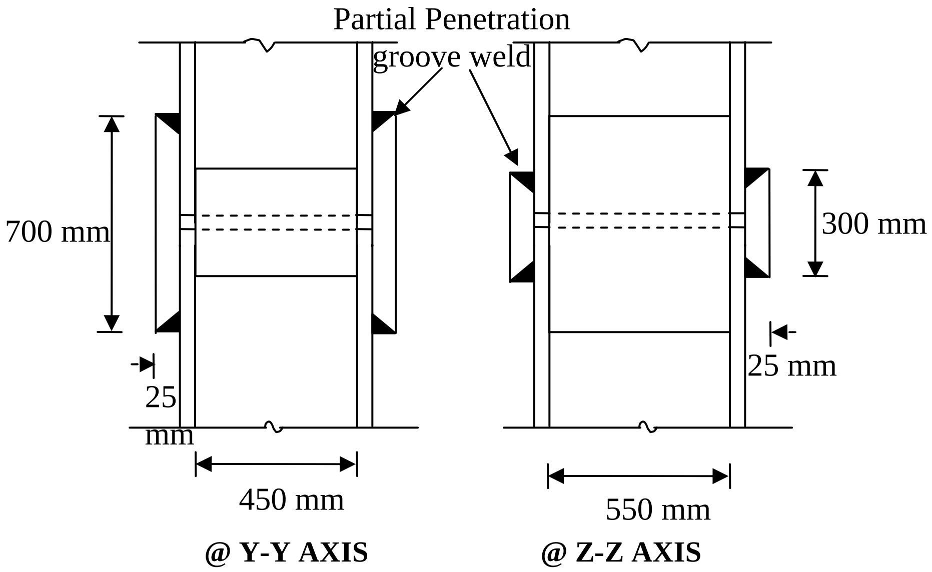


FIGURE 4.21 COLUMN SPLICES

Splice @ Y-Axis(Web Splice):

P_{min} = 0.6*fy*Aw = 1782.2 kN

Portion of load carried by web = 0.5*P*A_w /A + M_y/b+0.5*F_y

∴ P_Y = 1505.5 kN < 0.6*fy*Aw(i.e. Pmin)

∴ Design Force, F_{dy}= P_Y*2/1.5= 2376.2 kN (Cl:12.5.2.1)

& Required plate area, A = (F * γ_{mw} / fy)= 14257.2 mm^2

Provide the plate width, by = 550.0 mm

∴ thickness of web plate, t_w = A/by = 25 mm

∴ Providing partial groove weld of size 20 mm (i.e. 3/4* tw)

∴ Total Length of weld required, L_{wr} = F/0.7*t*f_{wd} = 1764 mm

∴ depth of plate required = (L_{wr} -2 *b_y) = 664.0 mm

∴ Provide the plate depth = 700.0 mm

∴ Use plate size of 550 x 700 x25 mm as shown in fig

4.7.8 MOMENT RESISTANT COLUMN BASES

In the moment resistant column bases, the bearing pressure between the base plate and the footing is assumed to vary linearly. The tension bolts are required to take up tension and to prevent the column from overturning. When the column is subjected to heavy axial compression with high bending moment in both directions, the unattached base (as shown in fig.4.22) with flange & web stiffening plates is more suitable.

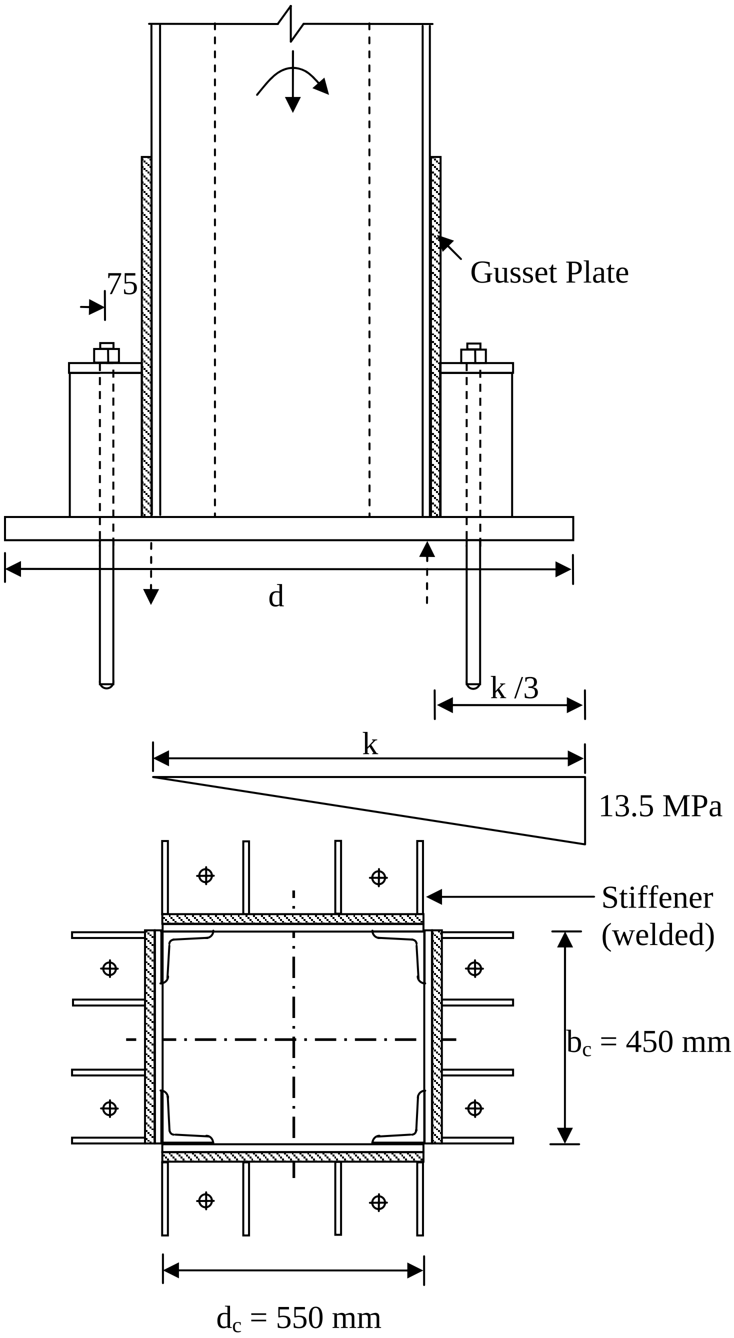


FIGURE 4.22 MOMENT RESISTANT COLUMN BASES

Design of Column Base – B2

For Pu = 6936.0 kN

Let the size of base plate is b x d and the area in contact with footing is b x k as shown in fig.4.22

∴ pk x b /2 = Pu = 6935979.0 N

& $(d/2 - k/3) = d_c/2 - t_f/2 =$ 267 mm

Let d = 1200 mm & b = 1100 mm (assuming equal projections)

Solving above two equations,

∴ k = 999.0 mm

& p = 12.6 Mpa

< 0.45fck- Bearing stress (i.e. 13.5 Mpa- For M30 concrete) Hence O.K.

Sagging moment at column face, M

$=\{1/2*(4*(k-dc/2-tf)/k +1/3*4(1-(k-dc/2-tf)/k)*(dc/2+tf)^2\}*b$

∴ M = 92.0 kNm

Now as we know that, f x Z = M

∴ $bt^2/6 \times fy/\gamma_{m0} = M$

∴ t = 47.0 mm

Provide the base plate of size 1100 x 1200 x 50 mm

For Pu = 3600.5 kN

Check for Bearing and Overturning:

∴ $f = Pu/A \pm (M/Ze)_z \pm (M/Ze)_y$

∴ f_{max} = 135.5 MPa < 227.27 Mpa i.e. fy/γm0, Hence O.K.(Cl:7.4.2.1)

& f_{min} = 46.7 MPa > 0, Hence O.K.

∴ $e_z = M_z/P_u$ = 16 mm

∴ $e_y = M_y/P_u$ = 0.8 mm

Provide 8-24 mm Ø, 8.8 grade HSFG bolts (minimum) of depth 600 mm with embeded length of 300 mm as shown in fig. 4.22 (Due to heavy DL, there is no tension in the flange and web)

Bearing Compression by each flange = $0.5*(P/A_f)$ = 264.03 Mpa

∴ Provide 20 mm gusset plate to all four sides of column as shown in fig.4.22

∴ Bearing Compression with bearing plate = 191.23 Mpa

< 227.27 Mpa i.e. fy/γ_{m0}, Hence O.K.

Design of weld:

Portion of load carried by flange, $F_z = 0.5*Pu*A_f/A + Mz/d_c + 0.5*Fy$

∴ F_z = 1427.7 kN

Portion of load carried by web, F_y = 0.5*Pu* A_w /A + M_y/b_c+0.5*F_z

∴ F_y = 2088.9 kN

∴ Provide 12 mm fillet weld all 4 sides.

∴ Weld Length required @ z, L_{wz} = Fz/(0.7*t*fwd)= 883.2 mm

∴ Weld Length required @ y, L_{wy} = Fy/(0.7*t*fwd)= 1292.3 mm

∴ Provide 4 - 300 x150 x 12 mm plates all 4 sides of column as shown in fig.4.22

4.8 ORIENTATION OF COLUMN

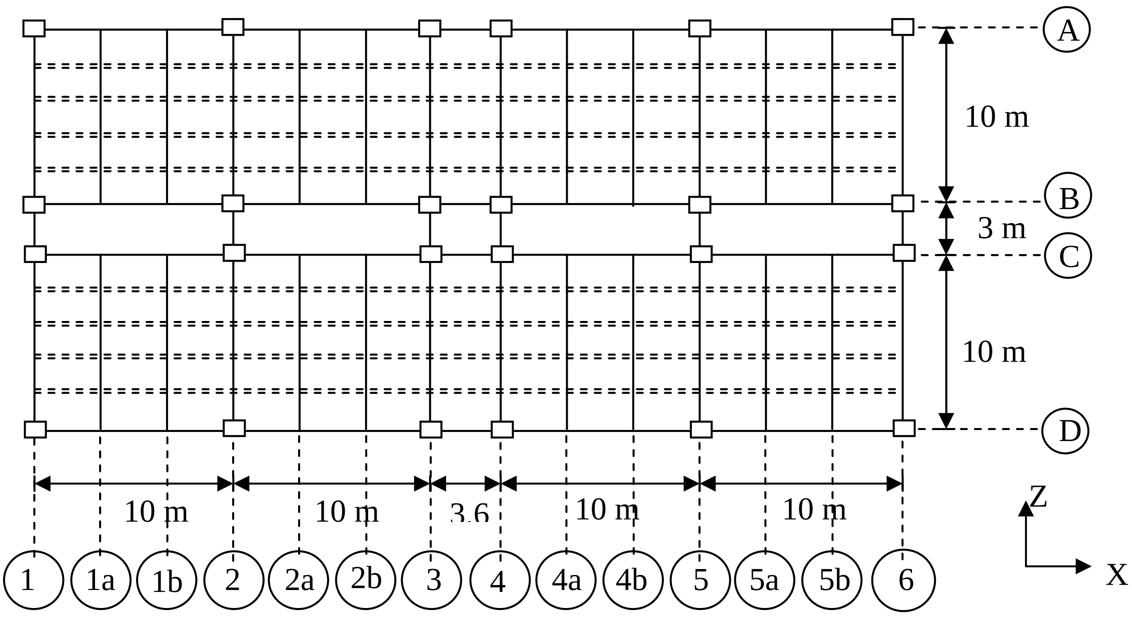


FIGURE 4.23 ORIENTATION OF COLUMN

Generally, the size and orientation of column is decided depending upon stiffness requirement of the frame. So the frame having smaller dimension require more stiffness and we provide the depth of column parallel to the smaller dimension of the building. Here the orientation of all the columns of the same 10 storeyed building is changed as shown in fig.4.23 and the table 4.7 and 4.8 shows the percentage increase in deflection, axial and shear stresses due to earthquake load in z-direction. The comparison of the deflection, axial and shear stresses due to earthquake load in z-direction are shown in fig 4.24.

TABLE 4.7 PERCENTAGE INCREASE IN DEFLECTION (mm) IN Z-DIRECTION

Height m	δz(mm) due to original column	δz(mm) due to oriented column	% increase in δz
0.0	0.0	0.0	0.0
5.0	5.2	6.0	16.1
8.5	11.1	12.6	13.8
12.0	17.2	19.4	12.5
15.5	23.3	26.0	11.8
19.0	29.1	32.4	11.3
22.5	34.5	38.3	11.0
26.0	39.4	43.6	10.8
29.5	43.5	48.1	10.7
33.0	46.6	51.5	10.5
36.5	48.7	53.7	10.3

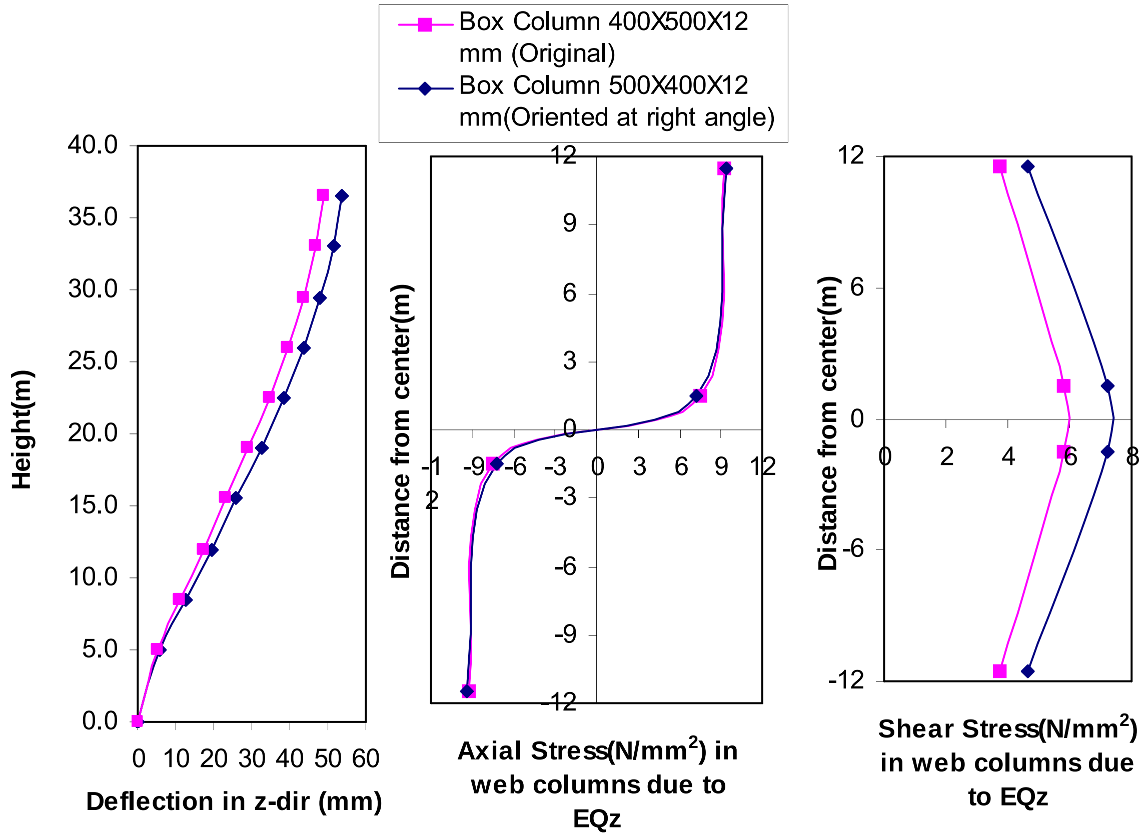


FIGURE 4.24 COMPARISONS OF THE DEFLECTION, AXIAL AND SHEAR STRESSES DUE TO EARTHQUAKE LOAD IN Z-DIRECTION

TABLE 4.8 AXIAL AND SHEAR STRESSES (N/mm^2) IN Z-DIRECTION

Distance From Center (m)	Axial Stress (N/mm^2) of original column	Axial Stress (N/mm^2) of oriented column	Shear Stress (N/mm^2) of original column	Shear Stress (N/mm^2) of oriented column
-11.5	-9.30	-9.35	3.71	4.67
-1.5	-7.46	-7.28	5.81	7.24
1.5	7.46	7.28	5.81	7.24
11.5	9.30	9.35	3.71	4.67

As shown in the fig.4.24 and table 4.7, the deflection of column is increased by 10.3%. The total deflection is 53.7 mm which is within permissible limit i.e. 56 mm. The increase in the values is very less due to provision of box column. The increase in the axial and shear stresses is negligible.

5. ANALYSIS OF 30 STOREYED STEEL BUILDING

5.1 GENERAL DATA

The plan of the building is as shown in fig. 5.1, the layout has following details:

Plan area: 36.9 m x 36.9 m [13 grids (1 to 13) in x–dir and 13 grids (A to M) in z–dir]

Storey height: 7.68 m at ground floor and 3.84 m at typical floor

Building Type: Office building

The building shown in fig. 5.1 represents the tube in tube structure. Here the exterior columns, placed 3 m c/c, resist all the lateral loads (i.e. wind & earthquake) while the interior column resist the vertical loads (i.e. DL, LL, FF) which is placed 6 m c/c for column free interior spaces.

5.2 MODELING

The structure is modeled as beam-column space frame structure with fixed support condition in the STAAD PRO-2004 as shown in fig.5.2, with steel beam as ISMB600 and the steel column as box section 400 x 400 x 12 mm of weight 150.72 kg/m. Here the rigid diaphragm property is provided at each storey for the equal deflection of all nodes in a storey.

5.3 LOADING

5.3.1 DEAD LOAD

R.C.C. (slab) – 25 kN/m^3

Floor Finish – 1.0 kN/m^2

Steel Density (M.S.) – 7850 kg/ m^3 and E – 2 x 10^5 N/mm^2

5.3.2 LIVE LOAD

As per IS: 875 (Part- 2) -1987, Live Load – 4.0 kN/m^2

Reduction in LL is made for the design load calculation of column at different storey.

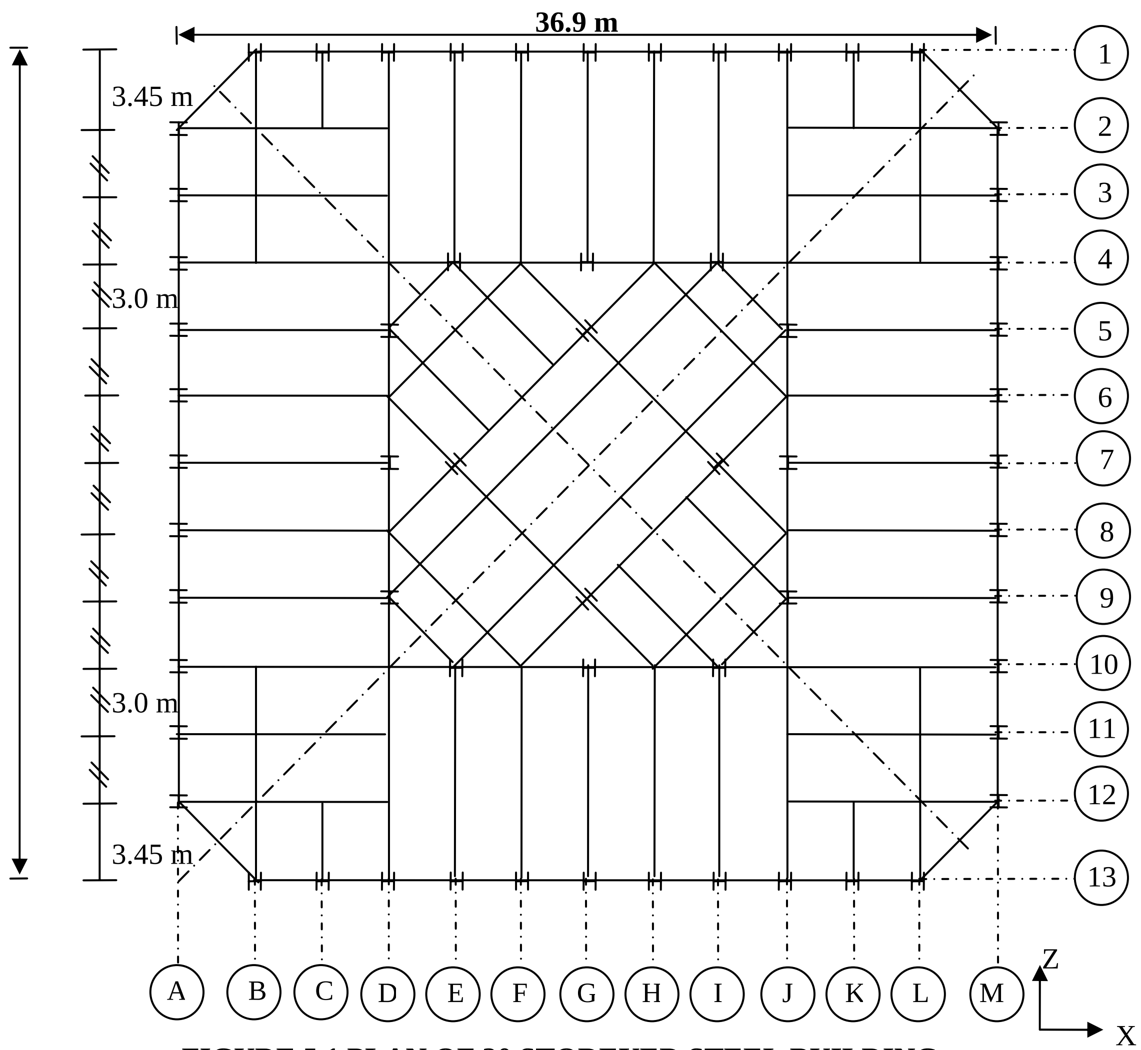


FIGURE 5.1 PLAN OF 30 STOREYED STEEL BUILDING

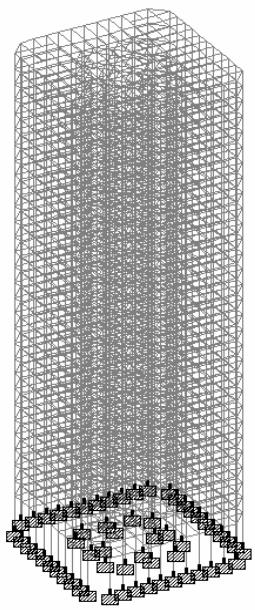


FIGURE 5.2 STRUCTURE MODELED IN STAAD PRO 2004

5.3.3 WIND LOAD

As per IS: 875 (Part 3) -1987,

Basic wind speed, V_b = 39 m/sec

Risk Coefficient, K_1 = 1.0

For Terrain Category-3 & Class of Structure – C,

Terrain, height and structure factor, K_2 = 0.82, for height up to 10 m
= 0.87, for height 10-15 m
= 0.91, for height 15-20 m
= 0.96, for height 20-30 m
= 1.02, for height 30-50 m
= 1.10, for height 50-100 m
= 1.15, for height 100-150 m

Topography factor, K_3 = 1.0

Design wind speed, $V_z = V_b * K_1 * K_2 * K_3$

Wind Intensity, $P_z = 0.6 * V_z^2$

Wind pressure coefficient (for both direction), C_f = 1.30

Wind force $F = C_f * A_e * P_z$

5.3.4 EARTHQUAKE LOAD

As per IS: 1893 (Part 1) – 2002,

Zone – III

Soil Condition – Medium soil

Importance Factor, I = 1.0

Response Reduction Factor for Steel moment resisting frame, R = 5.0

Time Period for steel frame building, $T = 0.085h^{0.75}$ (without brick infill panels)

$\therefore T$ = 3.063 sec, where h = 119.04 m

The load combinations are same as taken in 4.4.

5.4 PRELIMINARY ANALYSIS

In preliminary analysis, the structure is checked for permissible deflection and inter-storey drift for the governing serviceability load combination i.e. 1.0*DL + 1.0*WL_Z for column 400 x 400 x 12 mm. The deflection and inter-storey drift is as shown in table 5.1.

Permissible Deflection = H/500 = 238.1 mm

Permissible inter-storey drift = 0.004*h = 15 mm (for 3.84 m ht.) and 31 mm (for 7.84 m ht.)

TABLE 5.1 DEFLECTION FOR BOX COLUMN SECTION 400 X 400 X 12 mm

Storey	Height (m)	X-Trans (mm)	Z-Trans (mm)	Inter-storey Drift (mm)	
				X - dir	Z - dir
Base	0.0	0.0	0.0	0.0	0.0
1	7.7	22.1	22.1	22.1	22.1
2	11.5	33.2	33.4	11.2	11.2
3	15.4	44.2	44.4	11.0	11.0
4	19.2	55.0	55.3	10.8	10.9
5	23.0	65.7	66.1	10.7	10.8
6	26.9	76.2	76.6	10.5	10.6
7	30.7	86.5	87.0	10.3	10.4
8	34.6	96.6	97.1	10.1	10.1
9	38.4	106.4	107.0	9.8	9.9
10	42.2	115.9	116.5	9.5	9.6
11	46.1	125.1	125.8	9.2	9.3
12	49.9	134.1	134.9	9.0	9.0
13	53.8	142.7	143.5	8.6	8.7
14	57.6	151.0	151.9	8.3	8.3
15	61.4	158.9	159.8	7.9	8.0
16	65.3	166.5	167.4	7.5	7.6
17	69.1	173.6	174.6	7.2	7.2
18	73.0	180.4	181.4	6.8	6.8
19	76.8	186.8	187.9	6.4	6.4
20	80.6	192.9	193.9	6.0	6.0
21	84.5	198.5	199.6	5.6	5.7
22	88.3	203.7	204.8	5.2	5.3
23	92.2	208.6	209.7	4.8	4.9
24	96.0	213.0	214.2	4.4	4.5
25	99.8	217.0	218.2	4.0	4.0
26	103.7	220.6	221.8	3.6	3.6
27	107.5	223.8	225.0	3.2	3.2
28	111.4	226.6	227.8	2.7	2.7
29	115.2	228.9	230.1	2.3	2.3
30	119.0	230.8	232.0	1.9	1.9

The deflected shape of the structure in x and z direction is as shown in fig.5.3 (A) and (B) respectively.

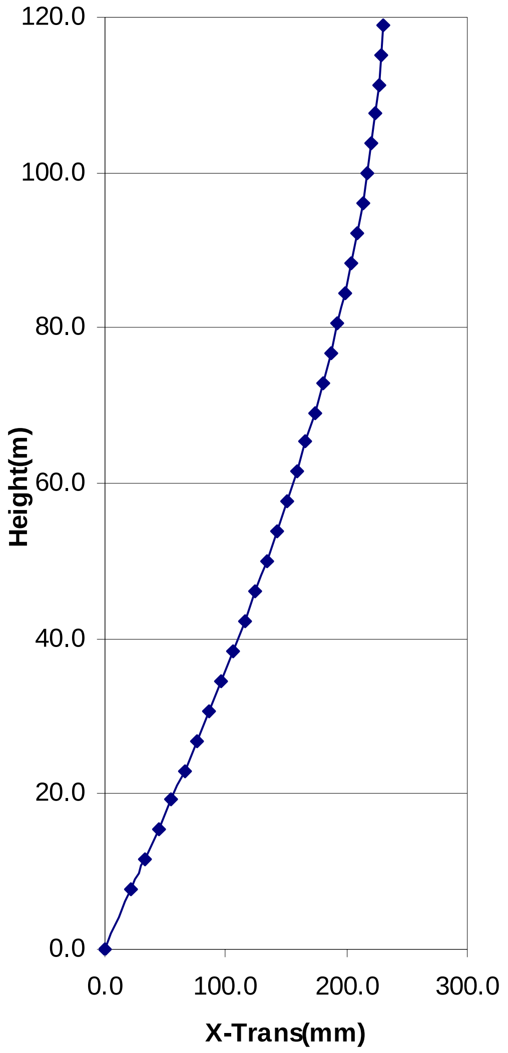


FIG. 5.3(A) DEFLECTION IN X – DIR

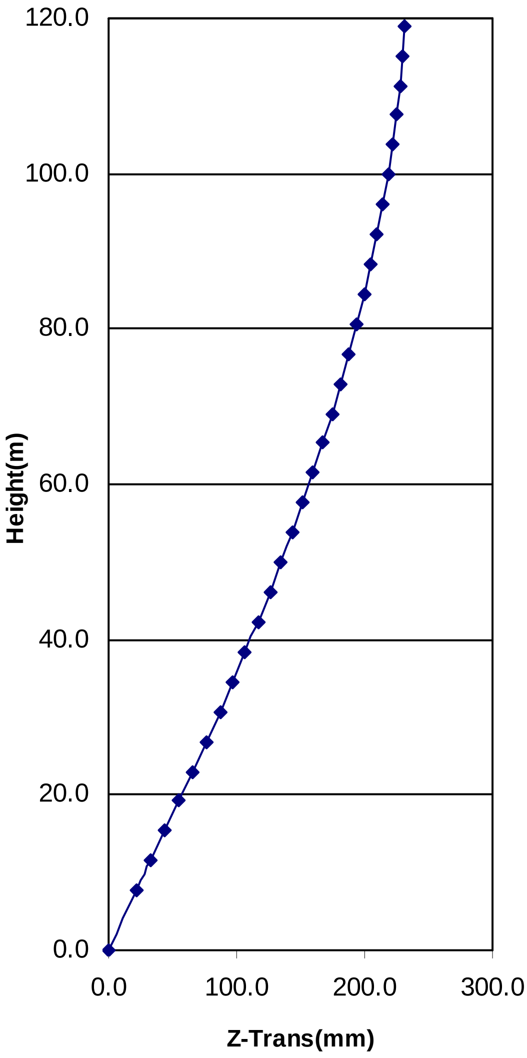


FIG. 5.3(B) DEFLECTION IN Z – DIR

5.5 ANALYSIS RESULT

5.5.1 COLUMN AXIAL FORCES COMPARISION

The design axial forces of columns are shown in table 5.2 for the frame-7, frame-A. The comparison of design column forces in frame 7 & A are shown in fig 5.4(A) & 5.4(B) respectively.

TABLE 5.2 DESIGN COLUMN FORCES FOR FRAME -7 & FRAME A

Column / Storey	A7	D7	E7	A4	A3	A2
1	5701.8	9753.2	10642.0	5817.8	5689.2	5542.2
2	5524.3	9325.5	10151.5	5583.5	5458.2	5269.3
3	5352.8	8918.7	9681.2	5367.0	5248.0	5086.4
4	5182.3	8527.4	9236.1	5161.8	5046.6	4907.5
5	5012.8	8152.9	8815.2	4966.1	4854.2	4731.7
6	4845.0	7795.8	8418.8	4779.5	4670.2	4559.8
7	4672.7	7441.2	8026.6	4592.1	4485.8	4385.5
8	4496.4	7089.3	7638.7	4403.5	4300.6	4209.3
9	4316.3	6740.5	7255.1	4213.8	4114.6	4031.2
10	4132.8	6394.8	6875.7	4022.8	3927.7	3851.4
11	3946.1	6052.5	6500.6	3830.5	3739.7	3670.3
12	3763.0	5729.1	6150.0	3646.7	3559.5	3494.1
13	3577.2	5409.1	5803.5	3461.6	3378.2	3316.8
14	3389.1	5092.3	5460.8	3275.3	3195.9	3138.2
15	3198.8	4778.5	5121.9	3087.8	3012.7	2958.6
16	3006.5	4467.6	4786.5	2899.3	2828.5	2777.9
17	2812.4	4159.4	4454.4	2709.9	2643.5	2596.2
18	2616.6	3853.8	4125.5	2519.6	2457.6	2413.6
19	2419.4	3550.6	3799.5	2328.4	2271.0	2230.2
20	2220.8	3249.6	3476.3	2136.5	2083.7	2045.9
21	2021.0	2950.7	3155.6	1943.9	1895.8	1860.9
22	1820.2	2653.6	2837.2	1750.7	1707.3	1675.1
23	1618.4	2358.3	2520.8	1557.0	1518.3	1488.7
24	1415.7	2064.5	2206.3	1362.8	1329.0	1301.7
25	1212.2	1772.1	1893.4	1168.2	1139.4	1114.1
26	1008.1	1481.0	1581.8	973.3	949.7	925.9
27	803.3	1190.9	1271.4	778.2	760.0	737.1
28	597.8	901.9	961.7	582.9	570.5	547.5
29	392.3	612.7	653.0	387.8	381.2	358.0
30	181.5	331.6	346.0	190.7	194.5	163.5

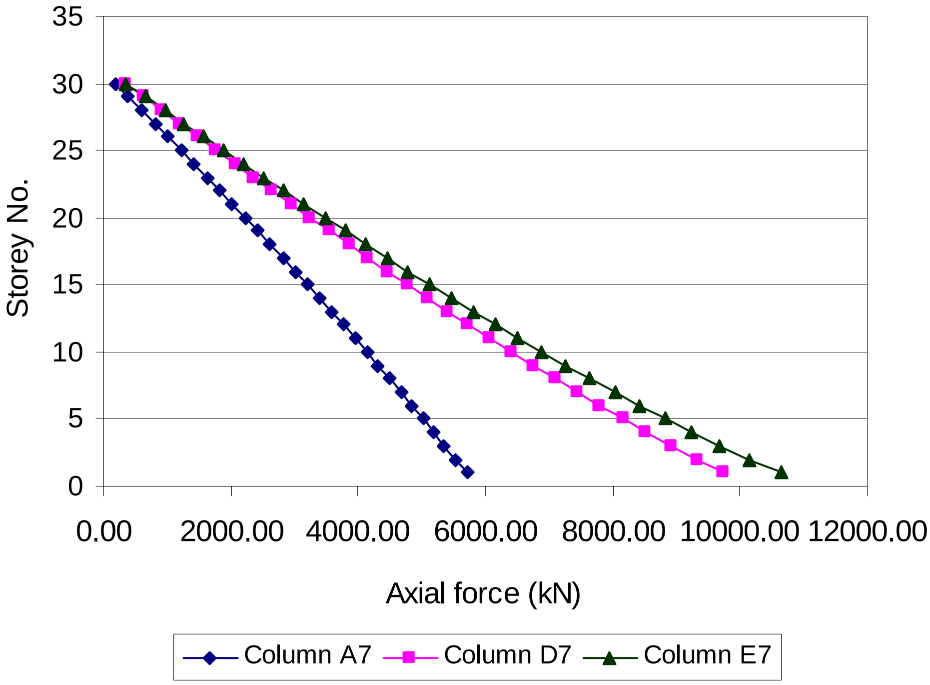


FIGURE 5.4(A) COLUMN AXIAL FORCES IN FRAME 7

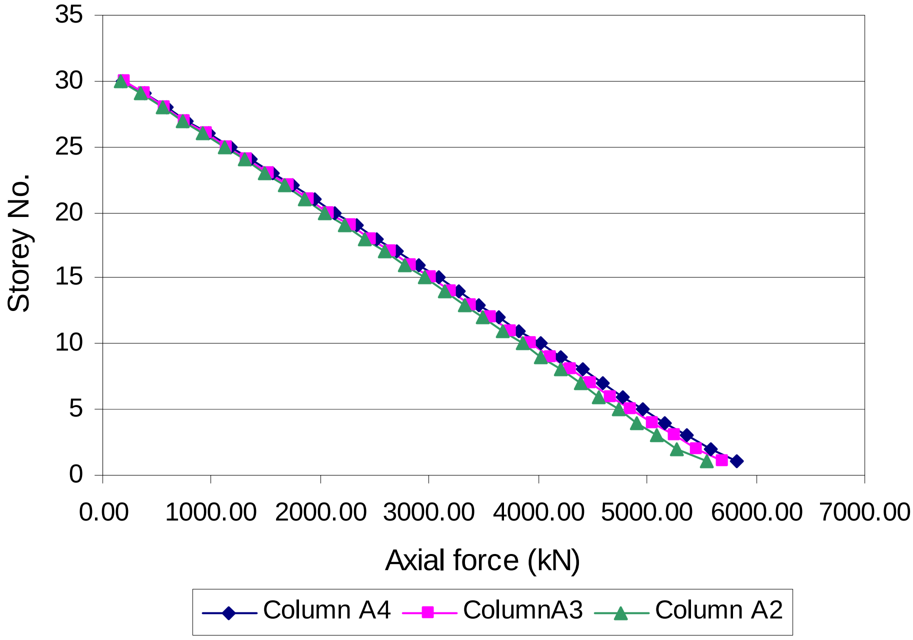


FIGURE 5.4(B) COLUMN AXIAL FORCES IN FRAME A

Here as shown in fig. 5.4(A) & 5.4(B), the interior columns are subjected to more axial force than exterior column due to vertical loads.

5.5.2 SHEAR LAG EFFECT OF COLUMN

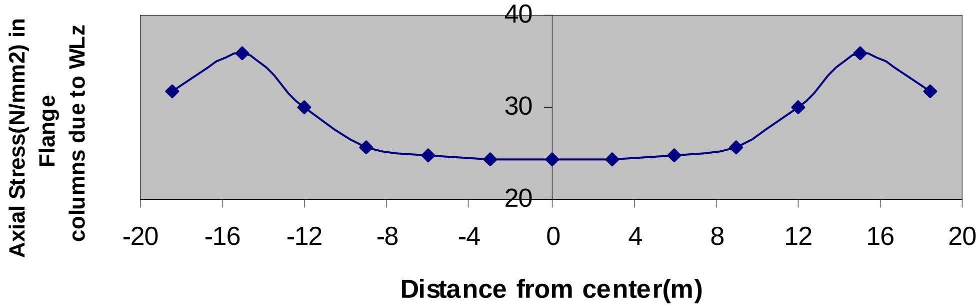


FIGURE 5.5(A) SHEAR LAG EFFECT IN FLANGE COLUMNS

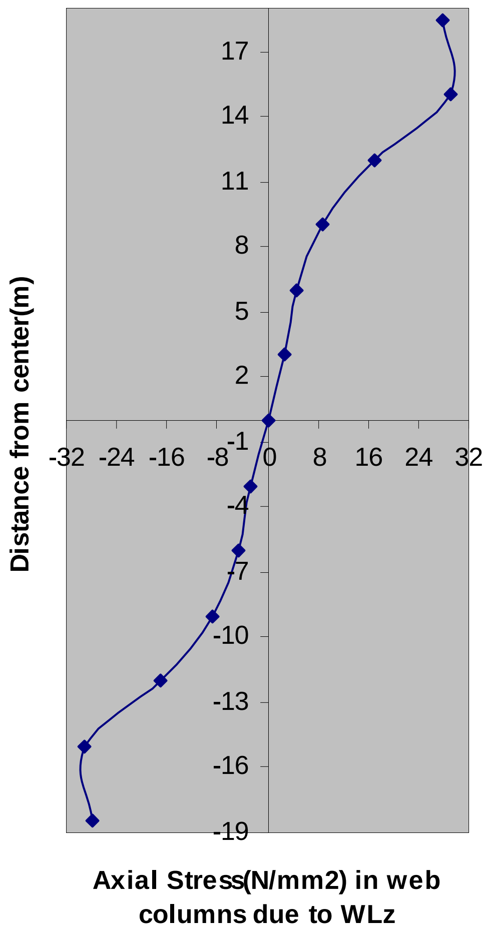


FIGURE 5.5(B) BUILDING AS VERTICAL CANTILEVER BEAM DUE TO WIND LOAD

Here flange columns of the tube in tube structure are subjected to shear lag effect, as shown in fig. 5.5(A), as there is no interior frames are provided. Also the axial stress behavior in web columns is same as the cantilever beam behavior of the building as shown in fig 5.5(B). The shape of the building (i.e. without end column) enhances the lateral load behavior of the building by reducing the axial stresses.

6. FLEXURAL MEMBER DESIGN PROGRAM (LATERALLY RESTRAINED BEAM)

6.1 GENERAL

The c++ program is prepared for the design of flexural member. For the factored moment (kNm) & shear (kN), it will give the safe and economical ISMB, ISWB & ISLB section. Hence we can take any one economical section as per our design requirement. This program satisfy all the checks i.e. check for section classification, check for shear(web buckling) and check for moment capacity. It also gives the percentage strength of the section. The input database of the program includes the number of ISMB, ISWB, ISLB sections and area (cm^2), depth(mm), width(mm) , flange thickness(mm), web thickness(mm), moment of inertia @ x-axis(cm^4), moment of inertia @ y-axis(cm^4) and distance between root fillet(mm) of all the sections.

6.2 C++ PROGRAM

```
//Program for design of laterally restrained flexural member(BEAM)
//combined for ISLB, MB & WB sections
#include<iostream.h>
#include<stdlib.h>
#include<iomanip.h>
#include<fstream.h>
#include<math.h>
#include<conio.h>
ifstream fin("finCombined.txt");
void main()
{
	int D[60],B[60],n1,n2,n3,i,x,y;
	float area[60],tf[60],tw[60],ix[60],iy[60],b[60],d[60],mu,vu,
		vd[60],md[60],sc1[60],sc2[60],zex[60],zpx[60],PerStr[60];
	cout<<"Enter the Factored moment Mu(kNm):";
	cin>>mu;
	cout<<"Enter the Factored shear Vu(kN):";
```

```
cin>>vu;
fin>>n1>>n2>>n3;
x=n1+n2;
y=n1+n2+n3;
for(i=0;i<y;i++)
{
if(i<n1)
{
        fin>> D[i]>>area[i]>> B[i]>>tf[i]>>tw[i]>>ix[i]>>iy[i]>>d[i];
        //Calculation of Elastic section modulus
        zex[i]=ix[i]*2.0*10000.0/D[i];
        //Calculation of Plastic section modulus
        zpx[i]=zex[i]*1.14;
        //Section classification
        b[i]=B[i]/2.0;
        sc1[i]=b[i]/tf[i];// Assuming Rolled Section
        sc2[i]=d[i]/tw[i];// Assuming N.A. at mid-depth
        vd[i]=D[i]*tw[i]*250.0/1.10/1.732/1000;
        {
                if(vu <=0.6*vd[i])
                {
                        if(sc1[i]<=10.5 && sc2[i]<=104.8)//i.e. for plastic and
                        compact section
                        {
                        //Check for moment capacity(i.e. mu <= md)
                        md[i]=zpx[i]*250.0/1.1/1000000;
                        if(mu<=md[i])
                        {
                        if(sc1[i]<=9.4 && sc2[i]<=83.9)
                        {
                        cout<<endl<<"Section ISMB"<<D[i]<<" is safe i.e.
                        Mu<=Md";
                        cout<<endl<<"The Design moment
                        Md(kNm):"<<setprecision(5)<<md[i];
                        PerStr[i]=mu/md[i]*100.0;
```

```
cout<<endl<<"The % strength achieved is
"<<setprecision(3)<<PerStr[i]<<"%";
cout<<endl<<"Check for Section Classification:
Section is Plastic";
cout<<endl<<"Check for Shear: S.F. doesn't govern
permissible moment capacity i.e.Vu<0.6*Vd";
{
if(sc2[i]<=67)
cout<<endl<<"Check for web buckling : Safe";
else
cout<<endl<<"Provide check for web buckling
criteria";
}
i=n1;
cout<<endl;
}
else
{
cout<<endl<<"Section ISMB"<<D[i]<<" is safe i.e.
Mu<=Md";
cout<<endl<<"The Design moment
Md(kNm):"<<setprecision(5)<<md[i];
PerStr[i]=mu/md[i]*100.0;
cout<<endl<<"The % strength achieved is
"<<setprecision(3)<<PerStr[i]<<"%";
cout<<endl<<"Check for Section Classification:
Section is Compact";
cout<<endl<<"Check for Shear: S.F. doesn't govern
permissible moment capacity i.e.Vu<0.6*Vd";
{
if(sc2[i]<=67)
cout<<endl<<"Check for web buckling : Safe";
else
cout<<endl<<"Provide check for web buckling
criteria";
```

```
}
i=n1;
cout<<endl;
}
}
else
{
cout<<endl<<"Section ISMB"<<D[i]<<" is not safe
because Mu > Md";
}
}
else if((sc1[i]>10.5 && sc1[i]<=15.7) || (sc2[i]>104.8
&& sc2[i]<=125.9))//i.e. for semi-compact section
{
md[i]=zex[i]*250.0/1.1/1000000;
if(mu<=md[i])
{
cout<<endl<<"Section ISMB"<<D[i]<<" is safe i.e.
Mu<=Md";
cout<<endl<<"The Design moment
Md(kNm):"<<setprecision(5)<<md[i];
PerStr[i]=mu/md[i]*100.0;
cout<<endl<<"The % strength achieved is
"<<setprecision(3)<<PerStr[i]<<"%";
cout<<endl<<"Check for Section Classification:
Section is Semi-Compact";
cout<<endl<<"Check for Shear: S.F. doesn't govern
permissible moment capacity i.e.Vu<0.6*Vd";
{
if(sc2[i]<=67)
cout<<endl<<"Check for web buckling : Safe";
else
cout<<endl<<"Provide check for web buckling
criteria";
}
```

```
        i=n1;
        cout<<endl;
        }
        else
        {
        cout<<endl<<"Section ISMB"<<D[i]<<" is not safe";
        }
        }
        else
        {
        cout<<endl<<"Section ISMB"<<D[i]<<" is slender";
        }
}
else if(vu >0.6*vd[i] && vu < 1.0*vd[i])
{
        if(sc1[i]<=10.5 && sc2[i]<=104.8)//i.e. for plastic and
        compact section
        {
        //Check for moment capacity(i.e. mu <= md)
        md[i]=(zpx[i]-(2*vu/vd[i]-1)*(2*vu/vd[i]-
        1)*tw[i]*D[i]*D[i]/6)*250.0/1.1/1000000;
        if(mu<=md[i])
        {
        if(sc1[i]<=9.4 && sc2[i]<=83.9)
        {
        cout<<endl<<"Shear Force doesn't govern permissible
        moment capacity";
        cout<<endl<<"Section is Plastic";
        cout<<endl<<"Section ISMB"<<D[i]<<" is safe i.e.
        Mu<=Md";
        if(sc2[i]<67)
        cout<<endl<<"Section is safe for web buckling";
        else
        cout<<endl<<"Provide check for web buckling
        criteria";
```

```
i=n1;
}
else
{
cout<<endl<<"Section is Compact";
cout<<endl<<"Section ISMB"<<D[i]<<" is safe i.e.
Mu <= Md";
i=n1;
}
}
else
{
cout<<endl<<"Section ISMB"<<D[i]<<" is not safe
because Mu > Md";
}
}
else if((sc1[i]>10.5 && sc1[i]<=15.7) &&
(sc2[i]>104.8 && sc2[i]<=125.9))//i.e. for semi-
compact section
{
md[i]=zex[i]*250.0/1.1/1000000;
if(mu<=md[i])
{
cout<<endl<<"Section is Semi-Compact";
cout<<endl<<"Shear Force doesn't govern permissible
moment capacity i.e.Vu < 0.6*Vd";
cout<<endl<<"Section ISMB"<<D[i]<<" is safe i.e.
Mu<=Md";
i=n1;
}
else
{
cout<<endl<<"Section ISMB"<<D[i]<<" is not safe";
}
}
```

```
                else
                {
                cout<<endl<<"Section ISMB"<<D[i]<<" is slender";
                }
            }
            else
                cout<<endl<<"Section ISMB"<<D[i]<<" is not safe
                because Vu > Vd";
        }

    }
    cout<<"ISWB SECTIONS";
    else if(i>=n1 && i<x)
    {
        fin>> D[i]>>area[i]>> B[i]>>tf[i]>>tw[i]>>ix[i]>>iy[i]>>d[i];
        //Calculation of Elastic section modulus
        zex[i]=ix[i]*2.0*10000.0/D[i];
        //zey[i]=ix[i]*2.0/B[i];
        //Calculation of Plastic section modulus
        zpx[i]=zex[i]*1.14;
        //Section classification
        b[i]=B[i]/2.0;
        sc1[i]=b[i]/tf[i];// Assuming Rolled Section
        sc2[i]=d[i]/tw[i];// Assuming N.A. at mid-depth
        vd[i]=D[i]*tw[i]*250.0/1.10/1.732/1000;
        {
            if(vu <=0.6*vd[i])
            {
                if(sc1[i]<=10.5 && sc2[i]<=104.8)//i.e. for plastic and
                compact section
                {
                //Check for moment capacity(i.e. mu <= md)
                md[i]=zpx[i]*250.0/1.1/1000000;
                if(mu<=md[i])
                {
```

```
if(sc1[i]<=9.4 && sc2[i]<=83.9)
{
cout<<endl<<"Section ISWB"<<D[i]<<" is safe i.e.
Mu<=Md";
cout<<endl<<"The Design moment
Md(kNm):"<<setprecision(5)<<md[i];
PerStr[i]=mu/md[i]*100.0;
cout<<endl<<"The % strength achieved is
"<<setprecision(3)<<PerStr[i]<<"%";
cout<<endl<<"Check for Section Classification:
Section is Plastic";
cout<<endl<<"Check for Shear: S.F. doesn't govern
permissible moment capacity i.e.Vu<0.6*Vd";
{
if(sc2[i]<=67)
cout<<endl<<"Check for web buckling : Safe";
else
cout<<endl<<"Provide check for web buckling
criteria";
}
i=x;
cout<<endl;
}
else
{
cout<<endl<<"Section is Compact";
cout<<endl<<"Section ISWB"<<D[i]<<" is safe i.e.
mu <= md";
cout<<endl<<"The Design moment
Md(kNm):"<<setprecision(5)<<md[i];
PerStr[i]=mu/md[i]*100.0;
cout<<endl<<"The % strength achieved is
"<<setprecision(3)<<PerStr[i]<<"%";
cout<<endl<<"Check for Shear: S.F. doesn't govern
permissible moment capacity i.e.Vu<0.6*Vd";
```

```
{
if(sc2[i]<=67)
cout<<endl<<"Check for web buckling : Safe";
else
cout<<endl<<"Provide check for web buckling
criteria";
}
i=x;
cout<<endl;
}
}
else
{
cout<<endl<<"Section ISWB"<<D[i]<<" is not safe
because Mu > Md";
}
}
else if((sc1[i]>10.5 && sc1[i]<=15.7) || (sc2[i]>104.8
&& sc2[i]<=125.9))//i.e. for semi-compact section
{
md[i]=zex[i]*250.0/1.1/1000000;
if(mu<=md[i])
{
cout<<endl<<"Section ISWB"<<D[i]<<" is safe i.e.
Mu<=Md";
cout<<endl<<"The Design moment
Md(kNm):"<<setprecision(5)<<md[i];
PerStr[i]=mu/md[i]*100.0;
cout<<endl<<"The % strength achieved is
"<<setprecision(3)<<PerStr[i]<<"%";
cout<<endl<<"Check for Section Classification:
Section is Semi-compact";
cout<<endl<<"Check for Shear: S.F. doesn't govern
permissible moment capacity i.e.Vu<0.6*Vd";
{
```

```
		if(sc2[i]<=67)
		cout<<endl<<"Check for web buckling : Safe";
		else
		cout<<endl<<"Provide check for web buckling
		criteria";
		}
		i=x;
		cout<<endl;
		}
		else
		{
		cout<<endl<<"Section ISWB"<<D[i]<<" is not safe
		because Mu > Md";
		}
		}
		else
		{
		cout<<endl<<"Section ISWB"<<D[i]<<" is slender";
		}
	}
	else if(vu >0.6*vd[i] && vu < 1.0*vd[i])
	{
		if(sc1[i]<=10.5 && sc2[i]<=104.8)//i.e. for plastic and
		compact section
		{
		md[i]=(zpx[i]-(2*vu/vd[i]-1)*(2*vu/vd[i]-
		1)*tw[i]*D[i]*D[i]/6)*250.0/1.1/1000000;
		if(mu<=md[i])
		{
		if(sc1[i]<=9.4 && sc2[i]<=83.9)
		{
		cout<<endl<<"Shear Force doesn't govern permissible
		moment capacity";
		cout<<endl<<"Section is Plastic";
```

```
cout<<endl<<"Section ISWB"<<D[i]<<" is safe i.e.
Mu<=Md";
if(sc2[i]<67)
cout<<endl<<"Section is safe for web buckling";
else
cout<<endl<<"Provide check for web buckling
criteria";
i=n2;
}
else
{
cout<<endl<<"Section is Compact";
cout<<endl<<"Section ISWB"<<D[i]<<" is safe i.e.
Mu <= Md";
i=x;
}
}
else
{
cout<<endl<<"Section ISWB"<<D[i]<<" is not safe
because Mu > Md";
}
}
else if((sc1[i]>10.5 && sc1[i]<=15.7) || (sc2[i]>104.8
&& sc2[i]<=125.9))//i.e. for semi-compact section
{
md[i]=zex[i]*250.0/1.1/1000000;
if(mu<=md[i])
{
cout<<endl<<"Section is Semi-Compact";
cout<<endl<<"Shear Force doesn't govern permissible
moment capacity i.e.Vu < 0.6*Vd";
cout<<endl<<"Section ISWB"<<D[i]<<" is safe i.e.
Mu<=Md";
i=x;
```

```
                        //goto START2;
                        }
                        else
                        {
                        cout<<endl<<"Section ISWB"<<D[i]<<" is not safe";
                        }
                        }
                        else
                        {
                        cout<<endl<<"Section ISWB"<<D[i]<<" is slender";
                        }
                }
                else
                        cout<<endl<<"Section ISWB"<<D[i]<<" is not safe
                        because Vu > Vd";
        }

}
cout<<"ISLB SECTIONS";
else
{
        fin>> D[i]>>area[i]>> B[i]>>tf[i]>>tw[i]>>ix[i]>>iy[i]>>d[i];
        //Calculation of Elastic section modulus
        zex[i]=ix[i]*2.0*10000.0/D[i];
        //Calculation of Plastic section modulus
        zpx[i]=zex[i]*1.14;
        //Section classification
        b[i]=B[i]/2.0;
        sc1[i]=b[i]/tf[i];// Assuming Rolled Section
        sc2[i]=d[i]/tw[i];// Assuming N.A. at mid-depth
        vd[i]=D[i]*tw[i]*250.0/1.10/1.732/1000;
        {
                if(vu <=0.6*vd[i])
                {
```

```
if(sc1[i]<=10.5 && sc2[i]<=104.8)//i.e. for plastic and
compact section
{
md[i]=zpx[i]*250.0/1.1/1000000;
if(mu<=md[i])
{
if(sc1[i]<=9.4 && sc2[i]<=83.9)
{
cout<<endl<<"Section ISLB"<<D[i]<<" is safe i.e.
Mu<=Md";
cout<<endl<<"The Design moment
Md(kNm):"<<setprecision(5)<<md[i];
PerStr[i]=mu/md[i]*100.0;
cout<<endl<<"The % strength achieved is
"<<setprecision(3)<<PerStr[i]<<"%";
cout<<endl<<"Check for Section Classification:
Section is Plastic";
cout<<endl<<"Check for Shear: S.F. doesn't govern
permissible moment capacity i.e.Vu<0.6*Vd";
{
if(sc2[i]<=67)
cout<<endl<<"Check for web buckling : Safe";
else
cout<<endl<<"Provide check for web buckling
criteria";
}
cout<<endl;
goto exit ;
}
else
{
cout<<endl<<"Section is Compact";
cout<<endl<<"Section ISLB"<<D[i]<<" is safe i.e.
mu <= md";
goto exit;
```

```
		}
		}
		else
		{
		cout<<endl<<"Section ISLB"<<D[i]<<" is not safe
		because Mu > Md";
		}
		}
		else if((sc1[i]>10.5 && sc1[i]<=15.7) || (sc2[i]>104.8
		&& sc2[i]<=125.9))//i.e. for semi-compact section
		{
		md[i]=zex[i]*250.0/1.1/1000000;
		if(mu<=md[i])
		{
		cout<<endl<<"Section is Semi-Compact";
		cout<<endl<<"Shear Force doesn't govern permissible
		moment capacity";
		cout<<endl<<"Section ISLB"<<D[i]<<" is safe i.e.
		Mu<=Md";
		goto exit;
		}
		else
		{
		cout<<endl<<"Section ISLB"<<D[i]<<" is not safe
		because Mu > Md";
		}
		}
		else
		{
		cout<<endl<<"Section ISLB"<<D[i]<<" is slender";
		}
	}
	else if(vu >0.6*vd[i] && vu < 1.0*vd[i])
	{
```

```
if(sc1[i]<=10.5 && sc2[i]<=104.8)//i.e. for plastic and
compact section
{
md[i]=(zpx[i]-(2*vu/vd[i]-1)*(2*vu/vd[i]-
1)*tw[i]*D[i]*D[i]/6)*250.0/1.1/1000000;
if(mu<=md[i])
{
if(sc1[i]<=9.4 && sc2[i]<=83.9)
{
cout<<endl<<"Shear Force doesn't govern permissible
moment capacity";
cout<<endl<<"Section is Plastic";
cout<<endl<<"Section ISLB"<<D[i]<<" is safe i.e.
Mu<=Md";
if(sc2[i]<67)
cout<<endl<<"Section is safe for web buckling";
else
cout<<endl<<"Provide check for web buckling
criteria";
goto exit;
}
else
{
cout<<endl<<"Section is Compact";
cout<<endl<<"Section ISLB"<<D[i]<<" is safe i.e. Mu
<= Md";
goto exit;
}
}
else
{
cout<<endl<<"Section ISLB"<<D[i]<<" is not safe
because Mu > Md";
}
}
```

```
}
                    else if((sc1[i]>10.5 && sc1[i]<=15.7) || (sc2[i]>104.8
                    && sc2[i]<=125.9))//i.e. for semi-compact section
                    {
                    md[i]=zex[i]*250.0/1.1/1000000;
                    if(mu<=md[i])
                    {
                    cout<<endl<<"Section is Semi-Compact";
                    cout<<endl<<"Shear Force doesn't govern permissible
                    moment capacity i.e.Vu < 0.6*Vd";
                    cout<<endl<<"Section ISLB"<<D[i]<<" is safe i.e.
                    Mu<=Md";
                    goto exit ;
                    }
                    else
                    {
                    cout<<endl<<"Section ISLB"<<D[i]<<" is not safe";
                    }
                    }
                    else
                    {
                    cout<<endl<<"Section ISLB"<<D[i]<<" is slender";
                    }
                }
                else
                    cout<<endl<<"Section ISLB"<<D[i]<<" is not safe
                    because Vu > Vd";
            }
        }
        }
        exit:
        getch();
}
```

6.3 INPUT DATABASE FILE

```
14      //No. of ISMB sections
12      //No. of ISWB sections
17      //No. of ISLB sections
//ISMB SECTIONS
//D     Area      B     tf     tw     Ix         Iy        h1
//mm    cm2       mm    mm     mm     cm4        cm4       mm
100     14.60     75    7.20   4.00   258.00     41.00     65.00
125     16.60     75    7.60   4.40   449.00     44.00     89.20
150     19.00     80    7.60   4.80   726.00     53.00     113.90
175     24.60     90    8.60   5.50   1272.00    85.00     134.50
200     32.30     100   10.80  5.70   2235.00    150.00    152.70
225     39.70     110   11.80  6.50   3442.00    218.00    173.30
250     47.50     125   12.50  6.90   5132.00    335.00    194.10
300     56.30     140   12.40  7.50   8604.00    454.00    241.50
350     66.70     140   14.20  8.10   13630.00   538.00    288.00
400     78.50     140   16.00  8.90   20458.00   622.00    334.40
450     92.30     150   17.40  9.40   30391.00   834.00    379.20
500     110.70    180   17.20  10.20  45218.00   1370.00   424.10
550     132.10    190   19.30  11.20  64894.00   1834.00   467.50
600     156.20    210   20.80  12.00  91813.00   2651.00   509.70
//ISWB SECTIONS
// D    Area      B     tf     tw     Ix         Iy        h1
150     21.70     100   7.00   5.40   839.00     95.00     116.60
175     28.10     125   7.40   5.80   1509.00    189.00    139.50
200     36.70     140   9.00   6.10   2625.00    329.00    158.80
225     43.20     150   9.90   6.40   3921.00    449.00    181.40
250     52.00     200   9.00   6.70   5943.00    858.00    203.80
300     61.30     200   10.00  7.40   9822.00    990.00    250.10
350     72.50     200   11.40  8.00   15522.00   1176.00   295.50
400     85.00     200   13.00  8.60   23427.00   1388.00   340.50
450     101.20    200   15.40  9.20   35058.00   1707.00   384.00
500     121.20    250   14.70  9.90   52291.00   2988.00   431.00
550     143.30    250   17.60  10.50  74906.00   3741.00   473.40
600     170.40    250   21.30  11.20  106199.00  4703.00   514.20
```

//ISLB SECTIONS

//D	Area	B	tf	tw	Ix	Iy	h_1
//mm	cm^2	mm	mm	mm	cm^4	cm^4	mm
75	7.70	50	5.00	3.70	73.00	10.00	51.70
100	10.20	50	6.40	4.00	168.00	13.00	73.00
125	15.10	75	6.50	4.40	407.00	43.00	95.40
150	18.10	80	6.80	4.80	688.00	55.00	116.90
175	21.30	90	6.90	5.10	1096.00	80.00	141.60
200	25.30	100	7.30	5.40	1697.00	115.00	165.70
225	29.90	100	8.60	5.80	2502.00	113.00	180.30
250	35.50	125	8.20	6.10	3718.00	193.00	202.60
275	42.00	140	8.80	6.40	5375.00	287.00	223.70
300	48.10	150	9.40	6.70	7333.00	376.00	245.10
325	54.90	165	9.80	7.00	9875.00	511.00	266.50
350	63.00	165	11.40	7.40	13158.00	632.00	288.30
400	72.40	165	12.50	8.00	19306.00	716.00	336.20
450	83.10	170	13.40	8.60	27536.00	853.00	384.00
500	95.50	180	14.10	9.20	38579.00	1064.00	430.20
550	110.00	190	15.00	9.90	53162.00	1335.00	476.10
600	126.70	210	15.50	10.50	72868.00	1822.00	520.20

6.4 INPUT

Enter the Factored moment Mu (kNm): 335.9

Enter the Factored shear Vu (kN): 174.5

6.5 OUTPUT

Section ISMB450 is safe i.e. Mu<=Md

The Design moment Md (kNm): 349.96

The % strength achieved is 96%

Section ISWB450 is safe i.e. Mu<=Md

The Design moment Md (kNm): 403.7

The % strength achieved is 83.2%

Section ISLB500 is safe i.e. Mu<=Md

The Design moment Md (kNm): 399.82

The % strength achieved is 84%

7. SUMMARY AND FUTURE SCOPE OF WORK

7.1 SUMMARY

The design of 10 storeyed steel building with various elements like RC voided slab, shear connector, flexural member (laterally restrained beam) with bearing stiffener , beam flange & web splices, beam-column, beam column welded connection, column splices(lap joint) and column bases have been discussed. The shear connector is designed for full shear connection using eurocde 4 as a composite beam element & so it reduces the factored moments of the flexural member.

The flow chart of the design of flexural member including bearing stiffener is shown and the built up I section is designed as a laterally restrained beam with all necessary checks such as check for section classification, check for section modulus, check for shear, check for bearing stiffener, check for outstand and check for buckling as per IS 800 Draft. A computer application for the flexural member is prepared in c++ for ISMB, ISWB and ISLB sections whose program code is shown in chapter 6. It also provides the percentage strength of the Indian rolled steel I sections.

The design of beam flange and web splices having bolted connection is shown with the use of 8.8 grade HSFG bolts. HSFG bolts provide efficient connections and perform well under fluctuating/fatigue load conditions so preferable for lateral load resisting frames as per IS 800 Draft. The important factor to be considered while evaluating the flange capacity is the reduction in strength due to bolt holes.

Design for biaxial bending of compression member i.e. beam-column is shown using design codal provisions. The box column with perforated plates and 4-ISA at corners is designed for the governing load condition from the considered three cases of the maximum axial compression, maximum moment about major axis and maximum moment about minor axis. Also design moment capacity about major axis is reduced due to lateral torsional buckling about minor axis as mentioned in the IS 800 Draft code. This phenomenon is similar to beam column designed by IS 800-1984 where permissible stress about major axis is reduced as compared to permissible stress about minor axis i.e. 0.66fy. Also all necessary checks i.e. check for section strength, check for slenderness ratio, check for overall member strength, check for shear and check for clear distance between perforations are also shown.

Moment resisting welded connections are designed for the tensile force (caused by hogging moments) at the top flange of the beam which is transferred to the top flange plate by fillet welds and from the plates to the column by groove welds. As mentioned in the draft code, all the lateral force resisting joints are designed with full penetration groove (butt) welds. Also stiffening seat at the beam bottom is designed for shear considering its safe bearing length.

The column splices (lap joints) are designed for minimum required strength 0.6 times $f_y A_f$ for each flange splice and 0.6 times $f_y A_w$ for each web splice as mentioned in IS 800 Draft code. Also all the joints with the partial penetration groove welds are designed considering design strength of the joints of at least equal to 200% of the required strength as mentioned in IS 800 Draft code. The moment resisting column bases are designed with the welded stiffening plates which is provided at all four sides of column. Also HSFG bolts are provided to resist the tension in the base.

Also the effect on drift due to change in the orientation of column is shown.

In the analysis of 30 storey steel building, different analysis results like the design column axial force, the behavior of building as vertical cantilever and shear lag effect of the used tube in tube structural building is shown.

7.2 CONCLUSION

- RC voided slab reduces the dead weight of structure by 15%. So gives economical beam & column section.

- Due to composite action between slab & beam by shear connector, the design negative moment at support reduces and leads to the economical section.

- Box Section with perforated plates and 4 ISA as a column having high radius of gyration @ minor axis, gives economical design section.

- Change in orientation of box column in shorter direction increases the drift of the structure by 10 % which is very less.

- The tube in tube structures shows the shear lag effect and the shape of the building (i.e. without end column) enhances the lateral load behavior of the building by reducing the axial stresses as shown in fig 5.5.

7.3 FUTURE SCOPE OF WORK

- Design of multistoreyed steel frames with diagonal braced members or vertical plate girder (shear wall) and its comparision using IS 800 Draft code.

- Design of different types of uniaxial & biaxial column and its comparision using IS 800 Draft code.

- Design of different types of welded & bolted connections and its comparision using IS 800 Draft code.

- Design of castellated beams and plate girders using IS 800 Draft code.

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