DESIGN AND ANALYSIS OF A MULTI-STOREY STRUCTURE

Project Report submitted in partial fulfillment of the Degree of

Bachelor of Technology

In

Civil Engineering

Under the Supervision of

Mr. Chandra Pal Gautam

By

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To

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CERTIFICATE

This is to certify that the project report entitled "**DESIGN AND ANALYSIS OF A MULTI-STOREY STRUCUTRE**" submitted by **Varun Dhanotia(101610) and Mehul Bansal(101670)** in partial fulfilment for the award of degree of B.Tech Civil Engineering of Jaypee University of Information Technology, Waknaghat has been carried out under my supervision.

This work has not been submitted partially or wholly to any other University or Institute for the award of this or any other degree or diploma.

Signature of Supervisor ……………………..

Name of Supervisor	Mr. Chandra Pal Gautam
Designation	Astt. Professor, JUITW
Date	……………………..

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ABSTRACT

The principle objective of this project is to analyze and design a multi-storeyed building (G+9) with Steel Frame using MS excel and Staad Pro. The design involves application of various loads (Dead, Live, Earthquake, Wind etc.) on the structure and analyzing the structure in the Staad Pro. The design of the structure is done using both the manual calculations using MS Excel and Staad Pro. The structure is supposed to be located in Delhi which falls under the Indian seismic zone IV.

We considered a 3-D steel frame with dimensions of 90.75ft in X-direction and 68.17ft in Z-direction. The Y-axis consisted of 10 floors of height 10ft each. The plan consisted of 64 beams and 48 columns on each floor level. Modeling in Staad Pro is done by assigning proper material properties and sections. The supports were taken as fixed. The frame was subjected to self-weight, dead loads, live load, seismic load and wind load as per Indian Standard Codes.

Steel Design is done with limit state method conforming to IS-800:2007 and design of concrete slabs is according to IS:456-2000. The design results given by Staad Pro are compared with the results obtained manually. Ultimately, the various steel sections for beams and columns are obtained. After designing the building, the post processing mode in Staad Pro can be used to study the bending moment and shear force values with the generated diagrams. We can also check the deflections of the members under the given load.

Design and analysis of complicated and high-rise structures need very time taking and cumbersome calculations using conventional manual methods. STAAD.Pro provides us a fast, efficient, easy to use and accurate platform for analyzing and designing structures.

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CHAPTER 1

INTRODUCTION

1.1 INTRODUCTION

A multi-storey building is a tall building or structure used as a residential or office building. The materials used for the structural system of multi-storey buildings are reinforced concrete and steel. Tall structures pose following design challenges for structural and geotechnical engineers:

Particularly if situated in a seismically active region or if the underlying soils have geotechnical risk factors such as high compressibility or bay mud. They also pose serious challenges to firefighters during emergencies in high-rise structures. New and old building design, building systems like the building standpipe system, HVAC systems (Heating, Ventilation and Air conditioning), fire sprinkler system and other things like stairwell and elevator evacuations pose significant problems.

Apartment blocks have technical and economic advantage in areas with high population density. They have become a distinguished form of housing accommodation in virtually all densely populated urban areas around the world. In contrast with low-rise and single-family houses, apartment blocks accommodate more inhabitants per unit of area of land they occupy and also decrease the cost of municipal infrastructure.

In developed countries a very large percentage of multi-storeyed buildings are built with steel whereas steel is not so commonly used in construction of multi-storeyed frames in India even though it is a better material than reinforced concrete. The use of steel in multi-storey building construction results in many advantages during the construction and the use. The advantages of using steel frames in the construction of multi-storey buildings are listed below:

- Steel has high strength to weight ratio which enables large spans and light weight construction. Steel structures can have a variety of structural forms like braced frames and moment resistant frames suitable to meet the specific requirements.
- Steel frames are faster to erect compared with reinforced concrete frames resulting in economy.
- The steel frame construction is more suitable to withstand lateral loads caused by wind or earthquake.

- Subsequent alterations and strengthening of floors are relatively easy in steel frames compared with concrete frames.
- Prefabricated members of the framework manufactured in the factory ensure better quality control.

1.2 OBJECTIVE

The aim of the project is to design and analyze a frame of a multi-storey building which has ten floors. The design will be such that the structure performs satisfactorily during its intended life. The frame will be designed such that it can sustain wind and seismic loads. Structural members are designed according to the Limit State Method. The design of the structural elements will be according to IS:456-2000 for concrete and IS:800-2007 for steel. The codes used for obtaining load values will be IS:875 for dead load, live load,wind load and IS:1893 for seismic load.

1.3 PROCEDURE

1. Determining the plan and elevation of the structure.
2. Determining the position of the beams and columns.
3. Design of one-way and two way slabs.
4. Modelling the structure in Staad Pro.
5. Calculation of loads.
6. Assigning the loads and member properties to the members.
7. Analyzing the structure to get the maximum bending moments and axial forces.
8. Designing the beams and columns using MS Excel.
9. Designing of beams and columns using Staad Pro.
10. Analyzing the structure for Seismic Loading.
11. Analyzing the structure for wind load.

After analyzing for all the possible loading conditions, the results obtained by analyzing the frame will be compared. The loading condition in which resulting moments and axial forces are largest will be considered as the building should be able to sustain the worst possible situation. Finally, after obtaining the sections the amount of the steel required will be calculated and the cost estimation will be performed.

CHAPTER 2

LOADS CONSIDERED

2.1 DEAD LOADS:

All permanent constructions of the structure form the dead loads. The dead load comprises of the weights of walls, partitions floor finishes, false ceilings, false floors and the other permanent constructions in the buildings. The dead load loads may be calculated from the dimensions of various members and their unit weights. the unit weights of plain concrete and reinforced concrete made with sand and gravel or crushed natural stone aggregate may be taken as 24 kN/m” and 25 kN/m” respectively. The dead load of steel sections are obtained according to SP-6.

2.2 IMPOSED LOADS:

Imposed load is produced by the intended use or occupancy of a building including the weight of movable partitions, distributed and concentrated loads, load due to impact and vibration and dust loads. Imposed loads do not include loads due to wind, seismic activity, snow, and loads imposed due to temperature changes to which the structure will be subjected to, creep and shrinkage of the structure, the differential settlements to which the structure may undergo.

2.3 WIND LOAD:

Wind is air in motion relative to the surface of the earth. The primary cause of wind is traced to earth’s rotation and differences in terrestrial radiation. The radiation effects are primarily responsible for convection either upwards or downwards. The wind generally blows horizontal to the ground at high wind speeds. Since vertical components of atmospheric motion are relatively small, the term ‘wind’ denotes almost exclusively the horizontal wind, vertical winds are always identified as such. The wind speeds are assessed with the aid of anemometers or anemographs which are installed at meteorological observatories at heights generally varying from 10 to 30 metres above ground.

2.3.1 Design Wind Speed (V)

The basic wind speed (V) for any site shall be obtained from and shall be modified to include the following effects to get design wind velocity at any height (V,) for the chosen structure:

a) Risk level;

b) Terrain roughness, height and size of structure; and

c) Local topography.

It can be mathematically expressed as follows:

Where:

$$\boldsymbol{V = V_b \times k_1 \times k_2 \times k_3}$$

V_b = design wind speed at any height z in m/s;

k_l = probability factor (risk coefficient)

k = terrain, height and structure size factor and

k_3 = topography factor

Risk Coefficient (k_I Factor) gives basic wind speeds for terrain Category 2 as applicable at 10 m above ground level based on 50 years mean return period. In the design of all buildings and structures, a regional basic wind speed having a mean return period of 50 years shall be used.

Terrain, Height and Structure Size Factor (k_2 Factor)

Terrain - Selection of terrain categories shall be made with due regard to the effect of obstructions which constitute the ground surface roughness. The terrain category used in the design of a structure may vary depending on the direction of wind under consideration. Wherever sufficient meteorological information is available about the nature of wind direction, the orientation of any building or structure may be suitably planned.

Topography (k_s Factor) - The basic wind speed V_b takes account of the general level of site above sea level. This does not allow for local topographic features such as hills, valleys, cliffs, escarpments, or ridges which can significantly affect wind speed in their vicinity. The effect of topography is to accelerate wind near the summits of hills or crests of cliffs, escarpments or ridges and decelerate the wind in valleys or near the foot of cliff, steep escarpments, or ridges.

2.4 SEISMIC LOAD:

2.4.1 Design Lateral Force

The design lateral force shall first be computed for the building as a whole. This design lateral force shall then be distributed to the various floor levels. The overall design seismic force thus obtained at each floor level shall then be distributed to individual lateral load resisting elements depending on the floor diaphragm action.

2.4.2 Design Seismic Base Shear

The total design lateral force or design seismic base shear (Vb) along any principal directionshall be determined by the following expression:

$$\boldsymbol{V_b = A_h \times W}$$

Where,

A_h = horizontal acceleration spectrum

W = seismic weight of all the floors

2.4.3 Fundamental Natural Period

The approximate fundamental natural period of vibration (T,), in seconds, of a moment resisting frame building without brick in the panels may be estimated

by the empirical expression:

$T_a = 0.075 \times h^{0.75}$ for RC frame building

$T_a = 0.085 \times h^{0.75}$ for steel frame building

Where,

h = Height of building, in m. This excludes the basement storeys, where basement walls are connected with the ground floor deck or fitted between the building columns. But it includes the basement storeys, when they are not so connected. The approximate fundamental natural period of vibration (T,), in seconds, of all other buildings, including moment-resisting frame buildings with brick lintel panels, may be estimated by the empirical Expression:

$$\boldsymbol{T = \frac{0.09H}{\sqrt{D}}}$$

Where,

h= Height of building

d= Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force.

2.4.4 Distribution of Design Force

Vertical Distribution of Base Shear to Different Floor Level

The design base shear (V) shall be distributed along the height of the building as per the following expression:

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$$

Q_i=Design lateral force at floor i,

W_i=Seismic weight of floor i,

h_i=Height of floor i measured from base, and

n=Number of storeys in the building is the number of levels at which the masses are located.

Distribution of Horizontal Design Lateral Force to Different Lateral Force Resisting

Elements in case of buildings whose floors are capable of providing rigid horizontal diaphragm action, the total shear in any horizontal plane shall be distributed to the various vertical elements of lateral force resisting system, assuming the floors to be infinitely rigid in the horizontal plane. In case of building whose floor diaphragms cannot be treated as infinitely rigid in their own plane, the lateral shear at each floor shall be distributed to the vertical elements resisting the lateral forces, considering the in-plane flexibility of the diagram.

2.5 Dynamic Analysis

Dynamic analysis shall be performed to obtain the design seismic force, and its distribution to different levels along the height of the building and to the various lateral load resisting elements, for the following Buildings:

a) *Regular buildings* -Those greater than 40 m in height in Zones IV and V and those Greater than 90 m in height in Zones II and 111.

b) *Irregular buildings* – All framed buildings higher than 12m in Zones IV and V and those greater than 40m in height in Zones 11 and III.

The analytical model for dynamic analysis of buildings with unusual configuration should be such that it adequately models the types of irregularities present in the building configuration. Buildings with plan irregularities cannot be modeled for dynamic analysis.

For irregular buildings, lesser than 40 m in height in Zones 11and III, dynamic analysis, even though not mandatory, is recommended. Dynamic analysis may be performed either by the

Time History Method or by the Response Spectrum Method. However, in either method, the design base shear (VB) shall be compared with a base shear (VB).

2.5.1 Response Spectrum Method-

Response spectrum analysis is a procedure for computing the statistical maximum response of a structure to a base excitation. Each of the vibration modes that are considered may be assumed to respond independently as a single-degree-of-freedom system. Design codes specify response spectra which determine the base acceleration applied to each mode according to its period (the number of seconds required for a cycle of vibration).

Having determined the response of each vibration mode to the excitation, it is necessary to obtain the response of the structure by combining the effects of each vibration mode because the maximum response of each mode will not necessarily occur at the same instant, the statistica maximum response, where damping is zero, is taken as the square root of the sum of the squares (SRSS) of the individual responses.

Response spectrum analysis produces a set of results for each earthquake load case which is really in the nature of an envelope. It is apparent from the calculation, that all results will be absolute values - they are all positive. Each value represents the maximum absolute value of displacement, moment, shear, etc. that is likely to occur during the event which corresponds to the input response spectrum.

To explain the response spectrum concept, we consider a SDOF system in which an external action is applied like an applied force or support displacement. For the response spectrum, it is necessary to evaluate the value of the maximum response, which may be determined once the equation q(t) is fully known.The equation of displacement q(t) for a SDOF system with damping § and natural frequency w

$$q(t)\ddot{+} 2w\S\dot{q}\ (t) + w^2 q(t) = \frac{p(t)}{m}$$

CHAPTER 3
PROBLEM FORMULATION

3.1 General

In this chapter, the buildings data have been described. The loads taken from various parts of IS-875 have been imposed on the frame and ultimate loads coming on the beams and columns are calculated. Seismic coefficient method will be applied to calculate base-shear due to earthquakes, as per IS-1893:2002. Wind loads are also applied separately as per IS-875-part-III.

3.2 Building Parameters

Height of the building = 110 feet
No. of storeys =10
Height of each floor =10 feet
Grade of concrete= M25
Grade of Steel= Fe 415
Unit weight of Concrete = 25 KN/m^3
Live Load on Floors = 2 KN/m^2
Load due to floor finish =1 KN/m^2

LAYOUT

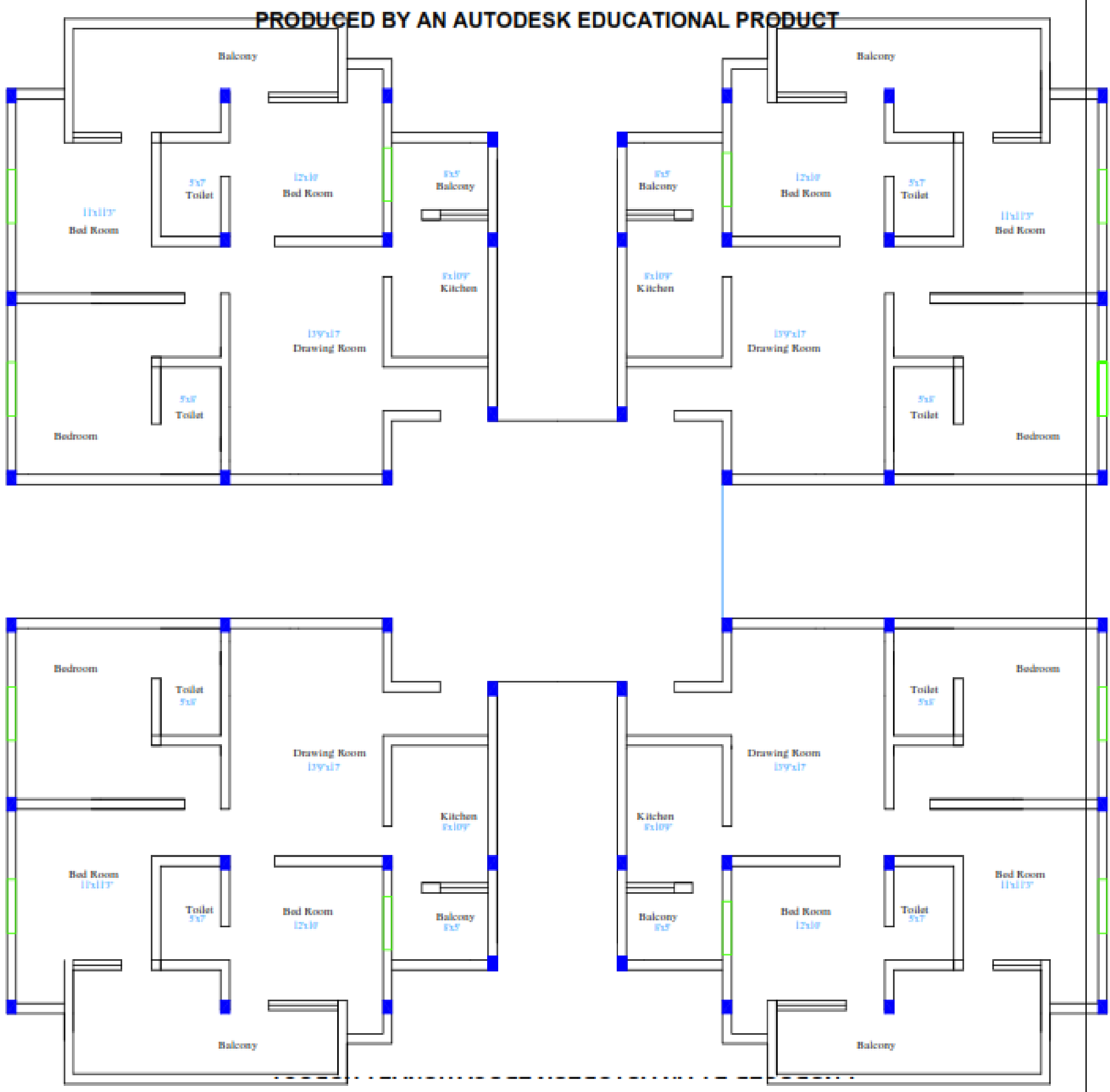


Fig 3.1 Plan of the Structure

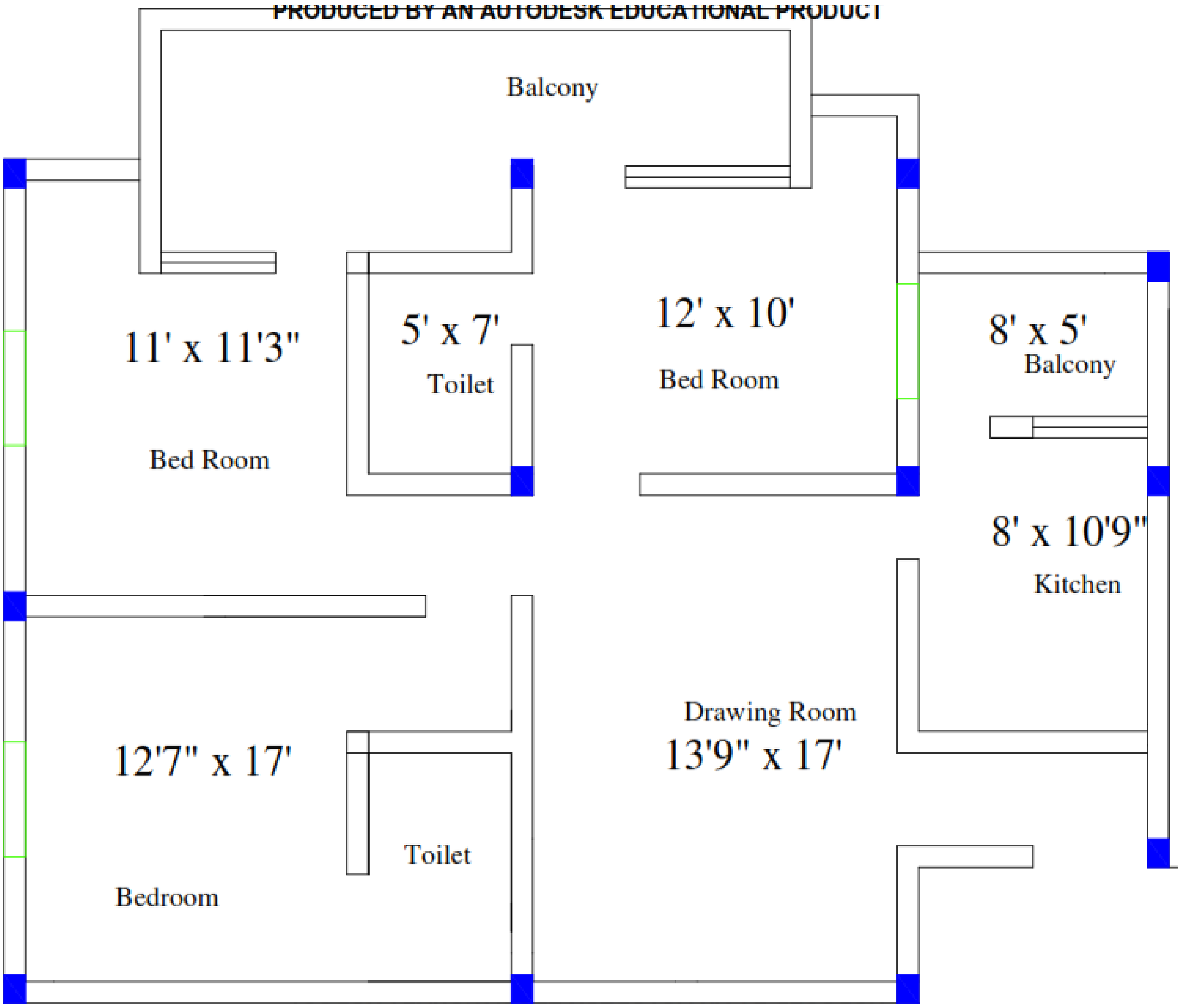


Fig. 3.2 Enlarged View of a Typical Section

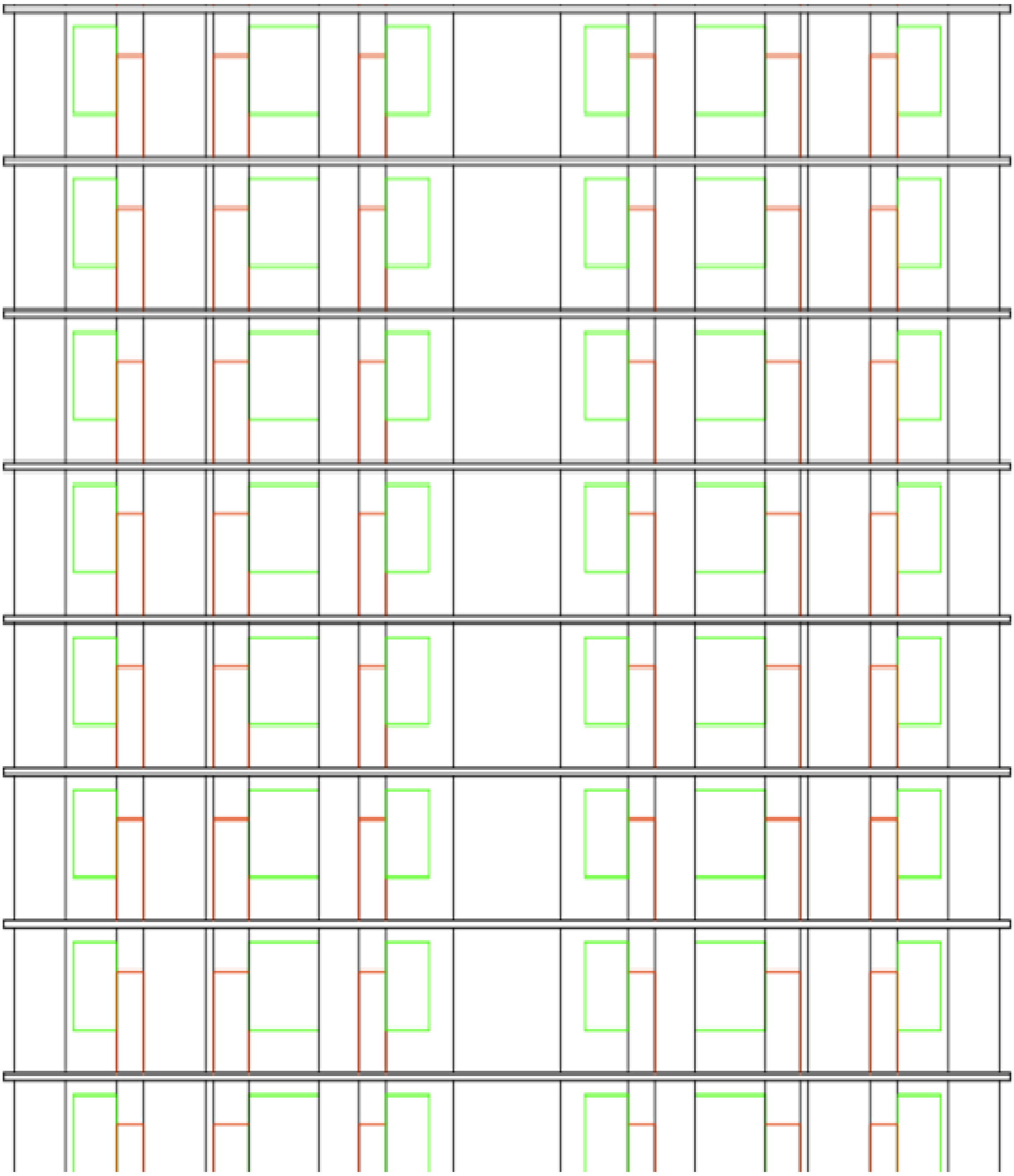


Fig. 3.3 Elevation

3.3 DESIGN OF SLABS(IS:456-2000)

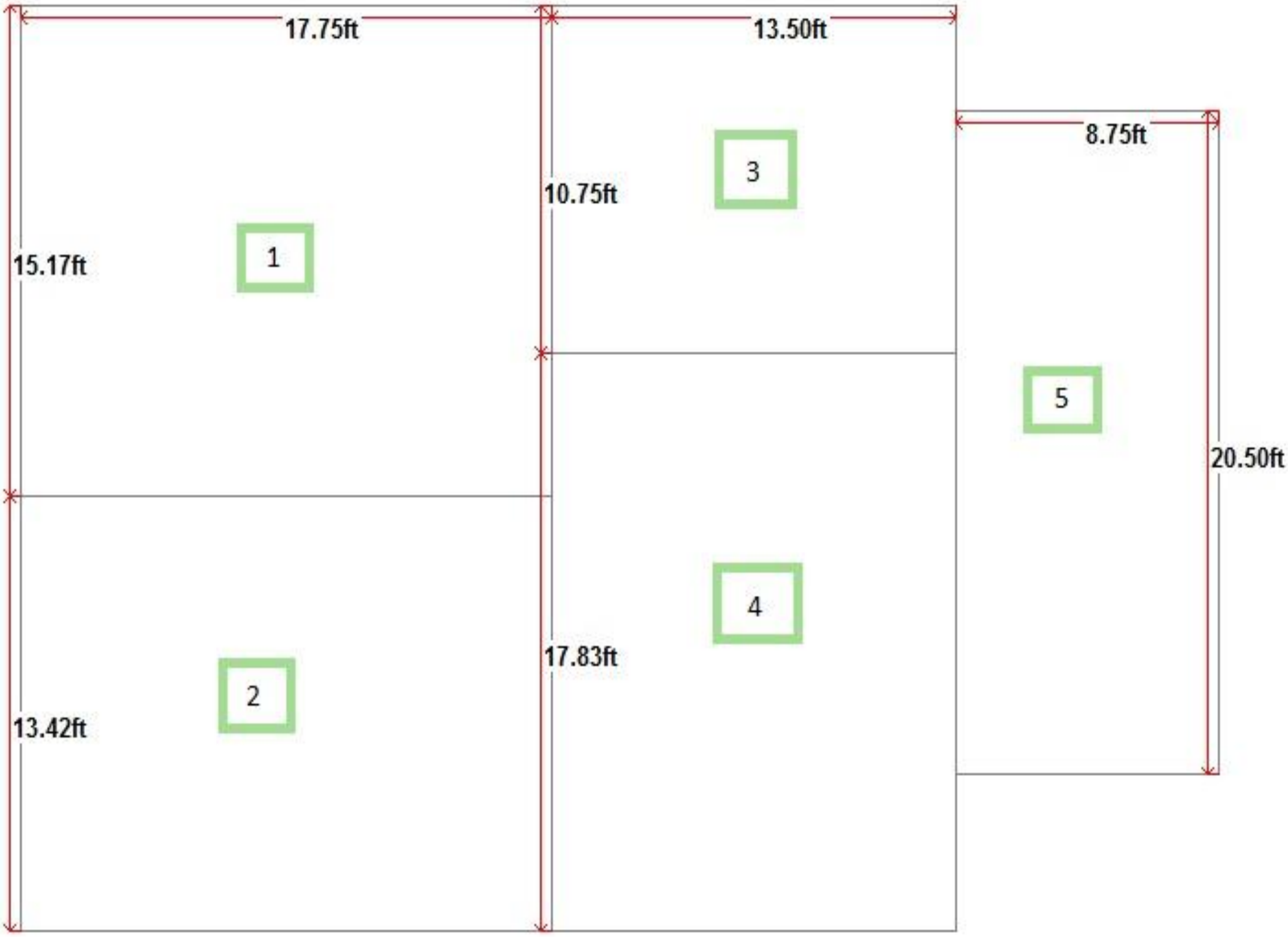


Fig. 3.4 Slab Panels

Panel 1

Ly	5410.2	mm		
Lx	4623.8	mm		
Depth	100	mm		
Effective Cover(Cl 26.4.2, Table 16)	20	mm		
Total depth (D)	120	mm		
Factored loads				
Dead Load of Slab	3	kn/m		
Load of floor finish	1	kn/m		
Live Load	2	kn/m		
Total load	6	kn/m		
Factored Load	**9**	kn/m		
MAXIMUM Bending Moments				
	Short Span		**Long Span**	
	coeff	Moment (kn-m)	coeff	Moment (kn-m)
Negative Moment At continuous edge(Annex D, D-2.1)	0.0579	11.14	0.047	9.04
Positive Moment At Mid Span(Annex D, D-2.1)	0.0435	8.37	0.035	8.37
MAXIMUM Positive Bending Moment	8.37			
MAXIMMUM Negative bending Moment	11.14			
MAXIMUM Bending Moments	11.14	KN-m		
Check for depth	56.83	mm	OK	
Check for Shear Force				
Shear Force	20.81	kn		
τv(Cl. 40.1)	0.21	kn/mm2		
τc(Table 19)	0.364	kn/mm2	OK	
Area of Steel required				
Short Span				
Positive Steel Area**(Annex G, G-1.1(b))**	**241.51**	**mm2**	OK	

Negative Steel Area	**326.24**	**mm2**	OK	
Long Span				
Positive area of steel	**241.51**			
Negative Steel Area	**261.86**			
Minimum Reinforcement Required	144.00	mm2		
TORSION				
Distance from the centre of support upto which torsional steel must be provided	924.76	mm		
Ast of Torsional Steel	181.13	mm2		

OUTPUT					
Short Span	**Dia. of bars**	**No. of bars**	**Ast Provided**	**Spacing**	**% steel**
Positive Ast	8	5	251.2	200	0.2512
Negative Ast	8	8	401.92	125	0.40192
Long Span					0
Positive Ast	8	6	301.44	165	0.30144
Negative Ast	8	6	301.44	165	0.30144

Panel 2

Ly	5410.2	mm		
Lx	4090.416	mm		
Depth	100	mm		
Effective Cover(Cl 26.4.2, Table 16)	20	mm		
Total depth (D)	120	mm		
Factored loads				
Dead Load of Slab	3	kn/m		
Load of floor finish	1	kn/m		
Live Load	2	kn/m		
Total load	6	kn/m		
Factored Load	**9**	kn/m		
MAXIMUM Bending Moments				
	Short Span		**Long Span**	
	coeff	Moment (kn-m)	coeff	Moment (kn-m)
Negative Moment At continuous edge(Annex D, D-2.1)	0.0662	9.97	0.047	7.08
Positive Moment At Mid Span(Annex D, D-2.1)	0.0498	7.50	0.035	7.50
MAXIMUM Positive Bending Moment	7.50			
MAXIMMUM Negative bending Moment	9.97			
MAXIMUM Bending Moments	9.97	KN-m		
Check for depth	53.75	mm	OK	
Check for Shear Force				
Shear Force	18.41	kn		
τv(Cl. 40.1)	0.18	kn/mm2		
τc(Table 19)	0.364	kn/mm2	OK	
Area of Steel required				
Short Span				
Positive Steel Area(Annex G, G-1.1(b))	**215.40**	**mm2**	OK	

Negative Steel Area	**290.07**	**mm2**	OK	
Long Span				
Positive area of steel	**215.40**			
Negative Steel Area	**202.85**			
Minimum Reinforcement Required	144.00	mm2		
TORSION				
Distance from the centre of support upto which torsional steel must be provided	818.08	mm		
Ast of Torsional Steel	161.55	mm2		

OUTPUT					
Short Span	**Dia. of bars**	**No. of bars**	**Ast Provided**	**Spacing**	**% steel**
Positive Ast	8	5	251.2	200	0.2512
Negative Ast	8	6	301.44	165	0.30144
Long Span					0
Positive Ast	8	5	251.2	200	0.2512
Negative Ast	8	5	251.2	200	0.2512

Panel 3

Ly	4114.8	mm		
Lx	3276.6	mm		
Depth	100	mm		
Effective Cover(Cl 26.4.2, Table 16)	20	mm		
Total depth (D)	120	mm		
Factored loads				
Dead Load of Slab	3	KN/m		
Load of floor finish	1	KN/m		
Live Load	2	KN/m		
Total load	6	KN/m		
Factored Load	**9**	KN/m		
MAXIMUM Bending Moments				
	Short Span		**Long Span**	
	coeff	Moment (kn-m)	coeff	Moment (kn-m)
Negative Moment At continuous edge(Annex D, D-2.1)	0.055	5.31	0.037	3.58
Positive Moment At Mid Span(Annex D, D-2.1)	0.042	4.06	0.028	4.06
MAXIMUM Positive Bending Moment	4.06			
MAXIMMUM Negative bending Moment	5.31			
MAXIMUM Bending Moments	5.31	KN-m		
Check for depth	39.25	mm	OK	
Check for Shear Force				
Shear Force	14.74	KN		
τv(Cl. 40.1)	0.15	KN/mm2		
τc(Table 19)	0.364	KN/mm2	OK	
Area of Steel required				
Short Span				
Positive Steel Area(Annex G, G-1.1(b))	144.00	mm2	OK	
Negative Steel Area	150.98	mm2	OK	
Long Span				

Positive area of steel	144.00			
Negative Steel Area	144.00			
Minimum Reinforcement Required	144.00	mm2		
TORSION				
Distance from the centre of support upto which torsional steel must be provided	655.32	mm		
Ast of Torsional Steel	113.23	mm2		

OUTPUT					
Short Span	**Dia. of bars**	**No. of bars**	**Ast Provided**	**Spacing**	**% steel**
Positive Ast	8	4	200.96	250	0.20
Negative Ast	8	4	200.96	250	0.20
Long Span					
Positive Ast	8	4	200.96	250	0.20
Negative Ast	8	4	200.96	250	0.20

Panel 4

Ly	5434.6	mm		
Lx	4114.8	mm		
Depth	100	mm		
Effective Cover(Cl 26.4.2, Table 16)	20	mm		
Total depth (D)	120	mm		
Factored loads				
Dead Load of Slab	3	KN/m		
Load of floor finish	1	KN/m		
Live Load	2	KN/m		
Total load	6	KN/m		
Factored Load	**9**	KN/m		
MAXIMUM Bending Moments				
	Short Span		**Long Span**	
	coeff	Moment(kn-m)	coeff	Moment(kn-m)
Negative Moment At continuous edge(Annex D, D-2.1)	0.0478	7.28	0.032	4.88
Positive Moment At Mid Span(Annex D, D-2.1)	0.0366	5.58	0.24	5.58
MAXIMUM Positive Bending Moment	5.58			
MAXIMMUM Negative bending Moment	7.28			
MAXIMUM Bending Moments	7.28	KN-m		
Check for depth	45.95	mm	OK	
Check for Shear Force				
Shear Force	18.52	KN		
τv(Cl. 40.1)	0.19	KN/mm2		
τc(Table 19)	0.364	KN/mm2	OK	
Area of Steel required				
Short Span				
Positive Steel Area(Annex G, G-1.1(b))	158.65	mm2	OK	
Negative Steel Area	208.99	mm2	OK	
Long Span				

Positive area of steel	158.65			
Negative Steel Area	144.00			
Minimum Reinforcement Required	144.00	mm2		
TORSION				
Distance from the centre of support upto which torsional steel must be provided	822.96	mm		
Ast of Torsional Steel	156.75	mm2		

OUTPUT					
Short Span	**Dia. of bars**	**No. of bars**	**Ast Provided**	**Spacing**	**% steel**
Positive Ast	8	4	200.96	250	0.20
Negative Ast	8	5	251.2	200	0.25
Long Span					
Positive Ast	8	4	200.96	250	0.20
Negative Ast	8	4	200.96	250	0.20

3.4 Design of Beams

The beams are designed as per IS800-2007. In this section the calculations involved in the design of beams are shown. The beams designed are considered to be laterally supported. The sections taken for the design of beams are ISMB. The sections obtained will be later compared with the sections obtained through steel design in Staad Pro.

Steps:

- Calculate Ultimate Bending Moment and Ultimate Shear force values.
- Assuming type of section, calculate Plastic section Modulus (8.2.1) and assume a trial section as ISMB.
- Classify section as plastic or elastic by checking width to thickness ratios (Table 2).
- Design for Moment (section 8.2.1 or 8.2.2)

  $$M_d=\beta_b Z_p f_y / \gamma_{m0}$$

- Check for Shear (section 8.4 or 9.2)

  $$Vd=Avfyw/\sqrt{3}\ \gamma m0$$

- Check for Deflection (Table 6)

  $$\delta=5WL4/384EI$$

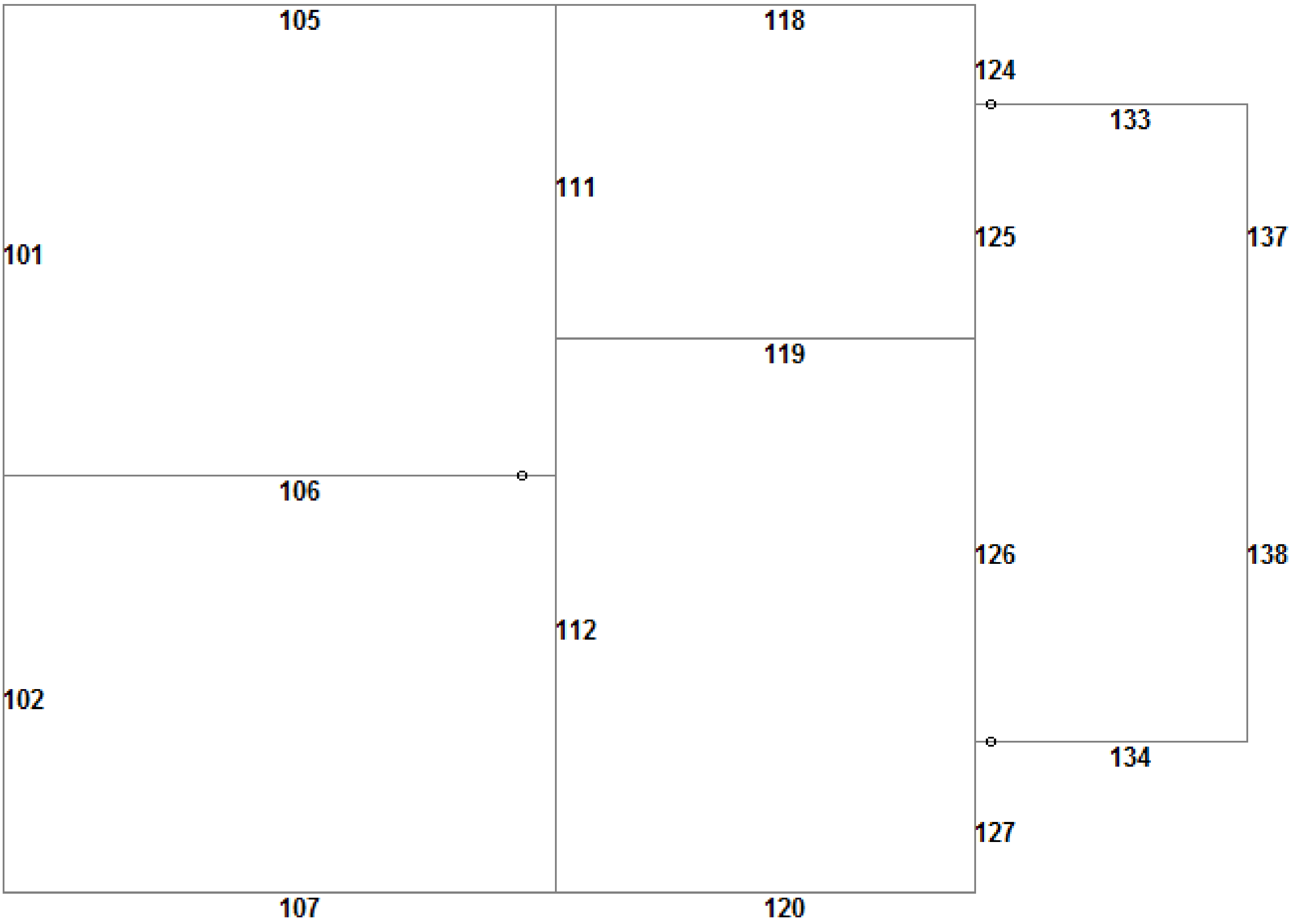


Fig 3.5 Typical Cross Section of the frame

Beam 101

Length of beam	4.60	m	
Maximum Bending Moment	29.074	KN-m	
Section Modulus Reqd.	127925.6	mm3	
Trial Section	**ISMB175**		
Section Classifiaction			
Yield Stress Ratio,ε(Table 2 Note 2)	1		
b/tf	5.23	Plastic	
d/tw	26.69	Plastic	
Class of Section(Acc. To Table 2)		Plastic	
β(Cl. 8.2.1.2)	1		
Check for Shear			
Design Shear Stength(Cl 8.4.1)	126.30	KN	
Actual Shear Force	39.00	KN	OK
0.6Vd(Cl. 8.2.1.3)	75.78	KN	OK
Check for Design Capacity			
d/tw(<67ε) (Cl. 8.4.2)	26.69		OK
Design Bending Strength(Md)(Cl. 8.2.1.2)	37.75	KN	
(1.2*Ze*fy)/(ϒy)(Cl. 8.2.1.2)	39.65	KN	OK
Maximum Bending Moment	29.07	KN-m	OK
Check For Deflection			
Deflection(δ)	6.11	mm	
Allowable Deflection(Table 6)	15.32	mm	OK

Beam 102

Length of beam	4.07	m	
Maximum Bending Moment	23.292	KN-m	
Section Modulus Reqd.	102484.8	mm3	
Trial Section	**ISMB175**		
Section Classifiaction			
Yield Stress Ratio,ε(Table 2 Note 2)	1		
b/tf	5.23	Plastic	
d/tw	26.69	Plastic	
Class of Section(Acc. To Table 2)		Plastic	
β(Cl. 8.2.1.2)	1		
Check for Shear			
Design Shear Stength(Cl 8.4.1)	126.30	KN	
Actual Shear Force	25.15	KN	OK
0.6Vd(Cl. 8.2.1.3)	75.78	KN	OK
Check for Design Capacity			
d/tw(<67ε) (Cl. 8.4.2)	26.69		OK
Design Bending Strength(Md)(Cl. 8.2.1.2)	37.75	KN	
(1.2*Ze*fy)/(ϒy)(Cl. 8.2.1.2)	39.65	KN	OK
Maximum Bending Moment	23.29	KN-m	OK
Check For Deflection			
Deflection(δ)	5.34	mm	
Allowable Deflection(Table 6)	13.56	mm	OK

Beam 105

Length of beam	5.38	m	17.75
Maximum Bending Moment	45.328	KN-m	
Section Modulus Reqd.	199443.2	mm3	
Trial Section	**ISMB200**		
Section Classifiaction			
Yield Stress Ratio,ε(Table 2 Note 2)	1		
b/tf	4.63	Plastic	1
d/tw	29.30	Plastic	1
Class of Section(Acc. To Table 2)		Plastic	1
β(Cl. 8.2.1.2)	1		
Check for Shear			
Design Shear Stength(Cl 8.4.1)	149.59	KN	
Actual Shear Force	39.44	KN	OK
0.6Vd(Cl. 8.2.1.3)	89.75	KN	OK
Check for Design Capacity			
d/tw(<67ε) (Cl. 8.4.2)	29.30		OK
Design Bending Strength(Md)(Cl. 8.2.1.2)	57.70	KN	
(1.2*Ze*fy)/(ϒy)(Cl. 8.2.1.2)	60.95	KN	OK
Maximum Bending Moment	45.33	KN-m	OK
Check For Deflection			
Deflection(δ)	6.44	mm	
Allowable Deflection(Table 6)	17.93	mm	OK

Beam 106

Length of beam	5.38	m	
Maximum Bending Moment	140.442	KN-m	
Section Modulus Reqd.	617944.8	mm3	
Trial Section	**ISMB300**		
Section Classifiaction			
Yield Stress Ratio,ε(Table 2 Note 2)	1		
b/tf	5.65	Plastic	
d/tw	34.69	Plastic	
Class of Section(Acc. To Table 2)		Plastic	
β(Cl. 8.2.1.2)	1		
Check for Shear			
Design Shear Stength(Cl 8.4.1)	295.24	KN	
Actual Shear Force	101.52	KN	OK
0.6Vd(Cl. 8.2.1.3)	177.14	KN	OK
Check for Design Capacity			
d/tw(<67ε) (Cl. 8.4.2)	34.69		OK
Design Bending Strength(Md)(Cl. 8.2.1.2)	148.12	KN	
(1.2*Ze*fy)/(ϒy)(Cl. 8.2.1.2)	156.44	KN	OK
Maximum Bending Moment	140.44	KN-m	OK
Check For Deflection			
Deflection(δ)	3.74	mm	
Allowable Deflection(Table 6)	17.93	mm	OK

Beam 107

Length of beam	5.38	m	17.75
Maximum Bending Moment	43.367	KN-m	
Section Modulus Reqd.	190814.8	mm3	
Trial Section	**ISMB200**		
Section Classifiaction			
Yield Stress Ratio,ε(Table 2 Note 2)	1		
b/tf	4.63	Plastic	1
d/tw	29.30	Plastic	1
Class of Section(Acc. To Table 2)		Plastic	1
β(Cl. 8.2.1.2)	1		
Check for Shear			
Design Shear Stength(Cl 8.4.1)	149.59	KN	
Actual Shear Force	37.9	KN	OK
0.6Vd(Cl. 8.2.1.3)	89.75	KN	OK
Check for Design Capacity			
d/tw(<67ε) (Cl. 8.4.2)	29.30		OK
Design Bending Strength(Md)(Cl. 8.2.1.2)	57.70	KN	
(1.2*Ze*fy)/(ϒy)(Cl. 8.2.1.2)	156.44	KN	OK
Maximum Bending Moment	43.37	KN-m	OK
Check For Deflection			
Deflection(δ)	6.11	mm	
Allowable Deflection(Table 6)	17.93	mm	OK

Beam 111

Length of beam	3.26	m	10.75
Maximum Bending Moment	76	KN-m	
Section Modulus Reqd.	334400	mm3	
Trial Section	**ISMB225**		
Section Classifiaction			
Yield Stress Ratio,ε(Table 2 Note 2)	1		
b/tf	4.66	Plastic	1
d/tw	28.98	Plastic	1
Class of Section(Acc. To Table 2)		Plastic	1
β(Cl. 8.2.1.2)	1		
Check for Shear			
Design Shear Stength(Cl 8.4.1)	191.90	KN	
Actual Shear Force	66.00	KN	OK
0.6Vd(Cl. 8.2.1.3)	115.14	KN	OK
Check for Design Capacity			
d/tw(<67ε) (Cl. 8.4.2)	28.98		OK
Design Bending Strength(Md)(Cl. 8.2.1.2)	79.15	KN	
(1.2*Ze*fy)/(ϒy)(Cl. 8.2.1.2)	83.43	KN	OK
Maximum Bending Moment	76.00	KN-m	OK
Check For Deflection			
Deflection(δ)	4.11	mm	
Allowable Deflection(Table 6)	10.86	mm	OK

Beam 112

Length of beam	5.38	m	
Maximum Bending Moment	95.14	KN-m	
Section Modulus Reqd.	418616	mm3	
Trial Section	**ISMB250**		
Section Classifiaction			
Yield Stress Ratio,ε(Table 2 Note 2)	1		
b/tf	5.00	Plastic	
d/tw	30.61	Plastic	
Class of Section(Acc. To Table 2)		Plastic	
β(Cl. 8.2.1.2)	1		
Check for Shear			
Design Shear Stength(Cl 8.4.1)	226.35	KN	
Actual Shear Force	115.00	KN	OK
0.6Vd(Cl. 8.2.1.3)	135.81	KN	OK
Check for Design Capacity			
d/tw(<67ε) (Cl. 8.4.2)	30.61		OK
Design Bending Strength(Md)(Cl. 8.2.1.2)	105.84	KN	
(1.2*Ze*fy)/(ϒy)(Cl. 8.2.1.2)	111.95	KN	OK
Maximum Bending Moment	95.14	KN-m	OK
Check For Deflection			
Deflection(δ)	6.13	mm	
Allowable Deflection(Table 6)	17.93	mm	OK

Beam 118

Length of beam	4.09	m	13.5
Maximum Bending Moment	19.357	KN-m	
Section Modulus Reqd.	85170.8	mm3	
Trial Section	**ISMB150**		
Section Classifiaction			
Yield Stress Ratio,ε(Table 2 Note 2)	1		
b/tf	5.26	Plastic	1
d/tw	26.08	Plastic	1
Class of Section(Acc. To Table 2)		Plastic	1
β(Cl. 8.2.1.2)	1		
Check for Shear			
Design Shear Stength(Cl 8.4.1)	94.48	KN	
Actual Shear Force	22.29	KN	OK
0.6Vd(Cl. 8.2.1.3)	56.69	KN	OK
Check for Design Capacity			
d/tw(<67ε) (Cl. 8.4.2)	26.08		OK
Design Bending Strength(Md)(Cl. 8.2.1.2)	25.11	KN	
(1.2*Ze*fy)/(ϒy)(Cl. 8.2.1.2)	111.95	KN	OK
Maximum Bending Moment	19.36	KN-m	OK
Check For Deflection			
Deflection(δ)	4.30	mm	
Allowable Deflection(Table 6)	13.64	mm	OK

CHAPTER 4

MODELLING IN STAAD PRO

4.1 Details of the model

1. Supports were taken all fixed.

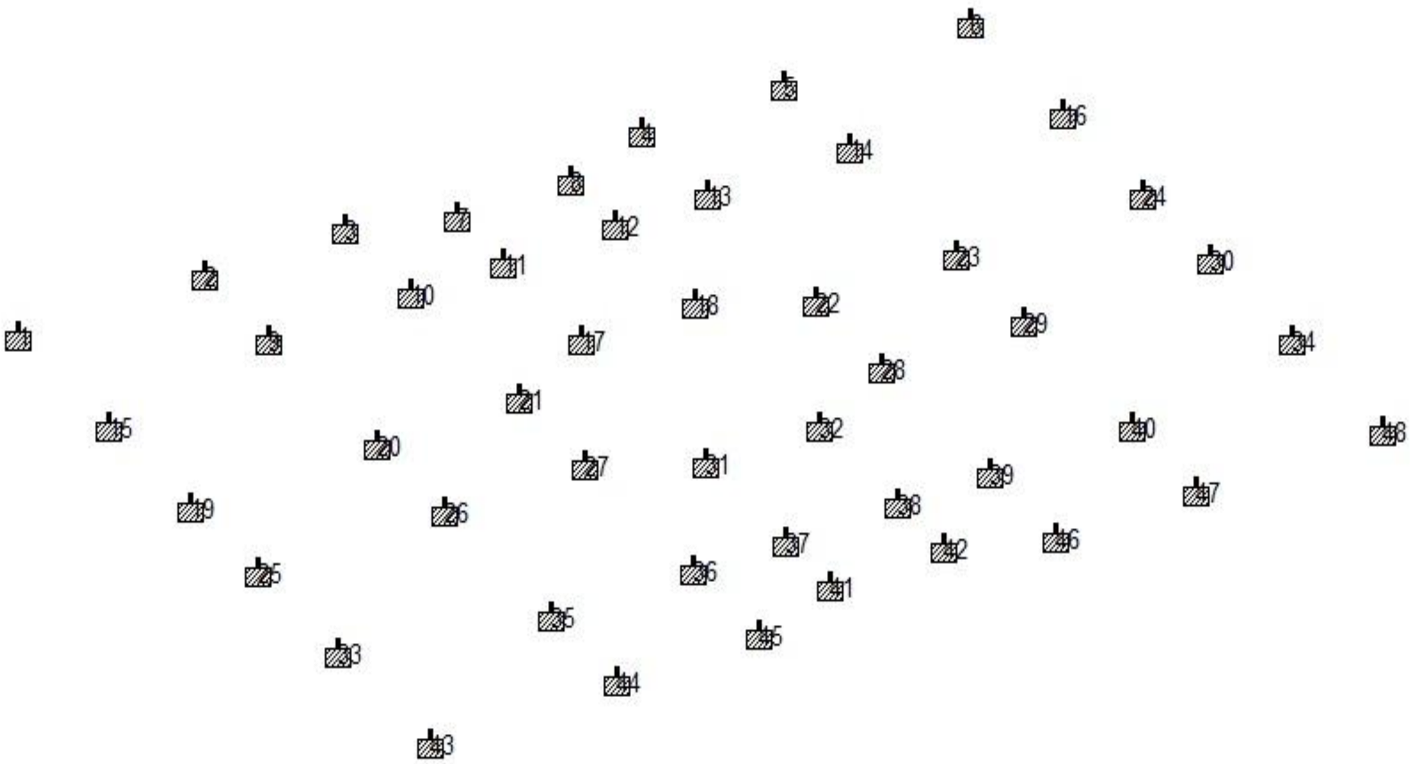


Fig 4.1 Supports in Staad Model

2. Member Properties were assigned to beams and columns as provided in the plan of the structure. For example :

DEFINE MATERIAL START

ISOTROPIC STEEL

E 1.90451e+007

POISSON 0.3

DENSITY 2.17529

ALPHA 1.2e-005

DAMP 0.03

END DEFINE MATERIAL

MEMBER PROPERTY INDIAN

19 24 25 103 104 113 TO 115 128 153 154 166 167 179 254 256 263 303 313 314 -
328 350 353 354 365 366 378 379 411 413 421 428 454 460 463 465 502 513

TABLE ST ISMB200

3.The secondary beams were released in X, Y and Z direction for M_x, M_y and M_z at appropriate points.

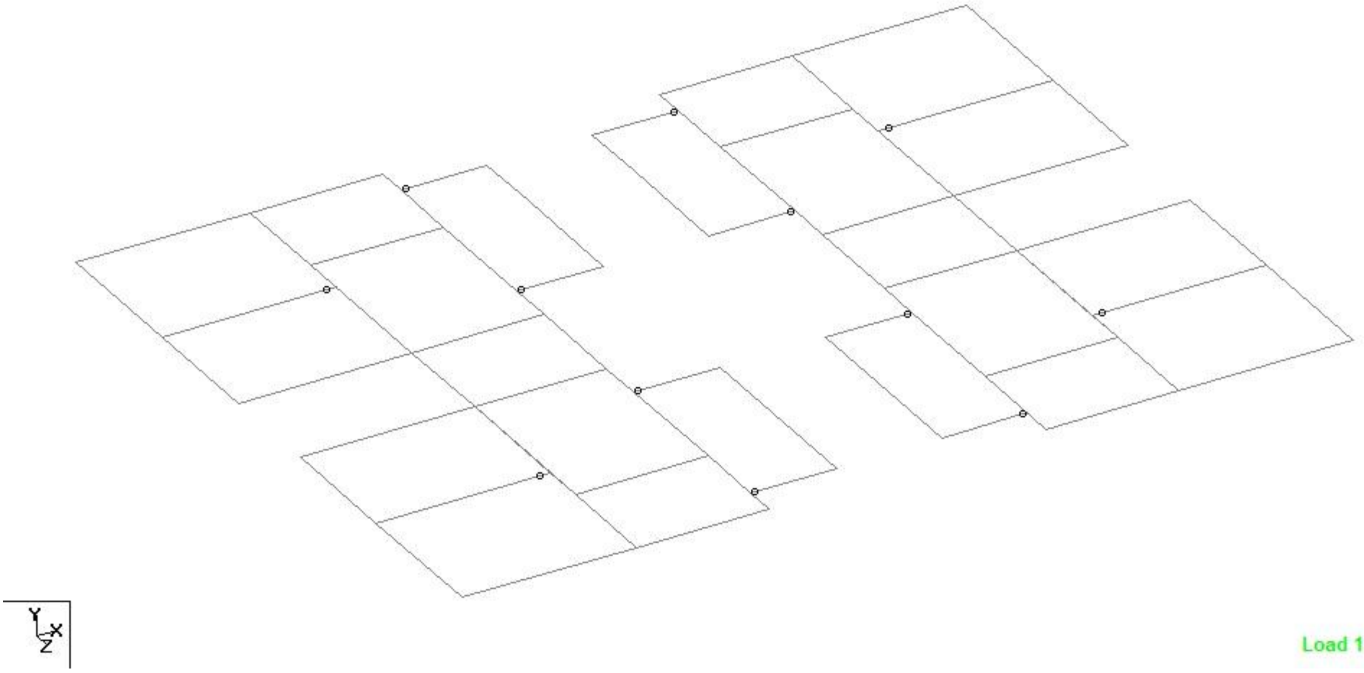


Fig 4.2 Isometric view of a typical floor showing member releases

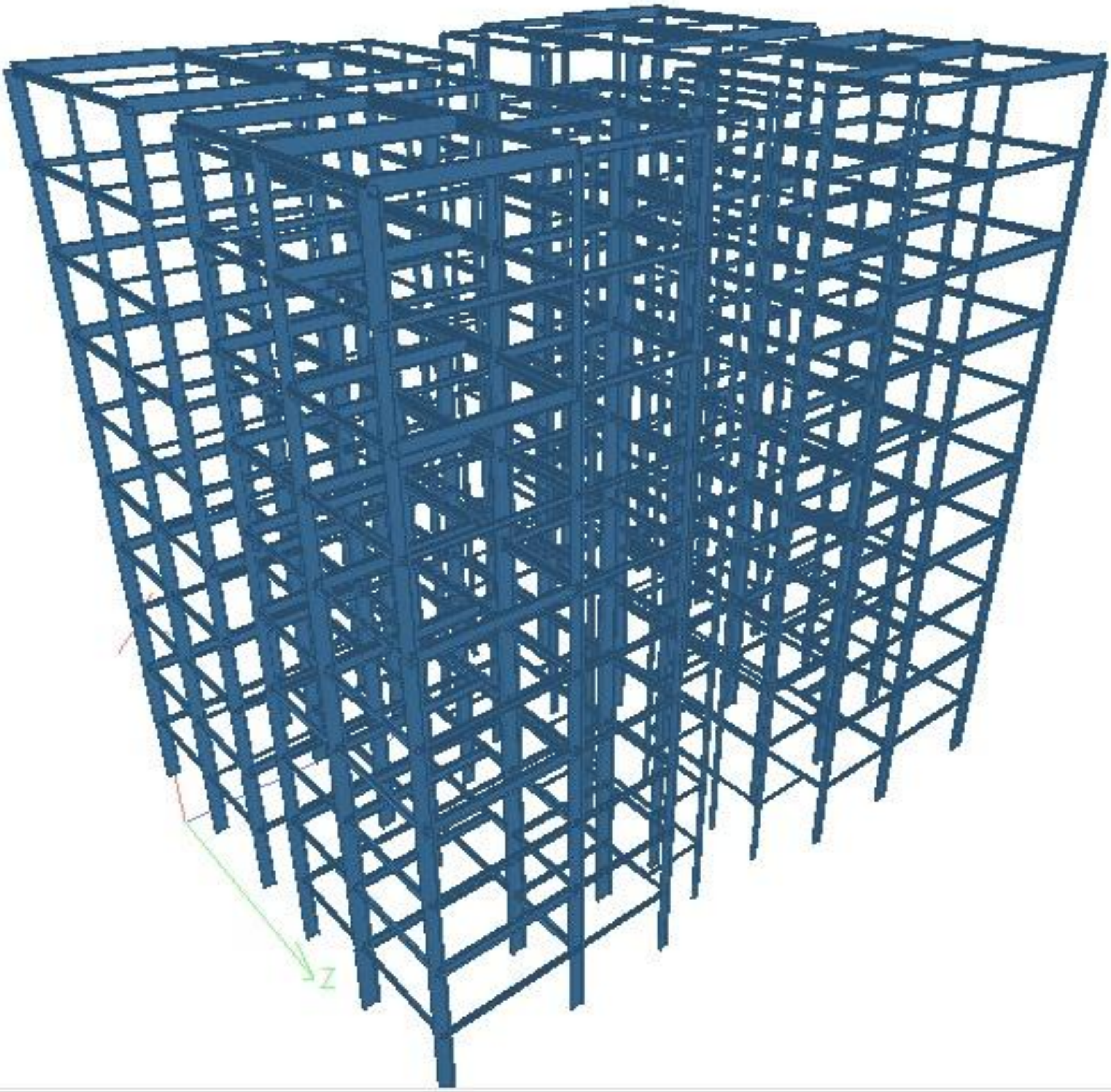


Fig 4.3 3-D Rendered view of Staad Model

4.2 Load Definitions

- Self-weight: It includes the self-weight of all the members according to their material property assigned.
- Live Load: It includes the total floor load coming from the slab(including the dead load due to slab) which will be transferred to the beam through appropriate distribution i.e. either One-way or Two-way.

  Calculation of Floor Load:

  Considering the thickness of the slab as 120 mm as obtained in Chapter 3 and taking the density of concrete as 25 KN/m^3, load due to floor finish as 1 KN/m^2, and the live load due to residential building as per IS 875(part 2) equal to 3 KN/m^2.

  Floor load = 1.5(25x0.12 + 3 +1) = 9 KN/m^2

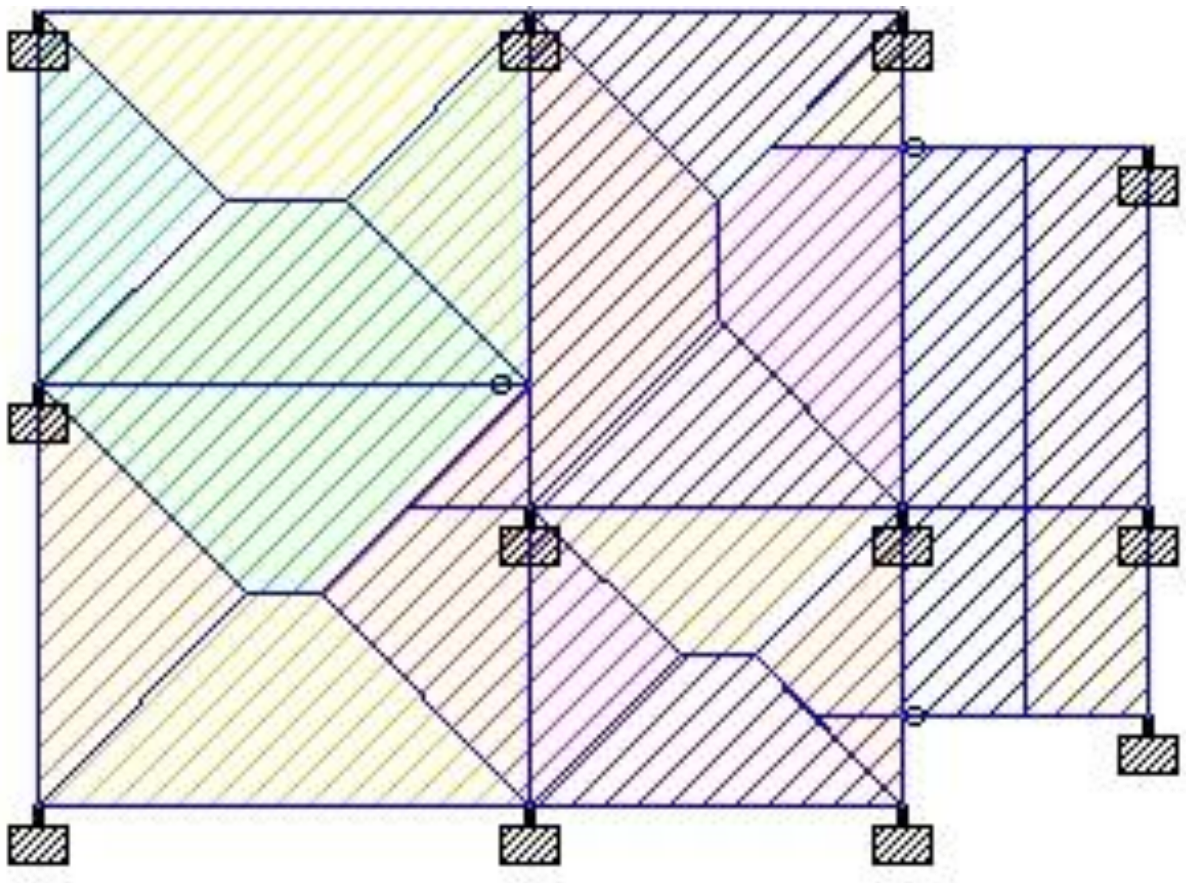


Fig 4.4 Loading on a typical floor section

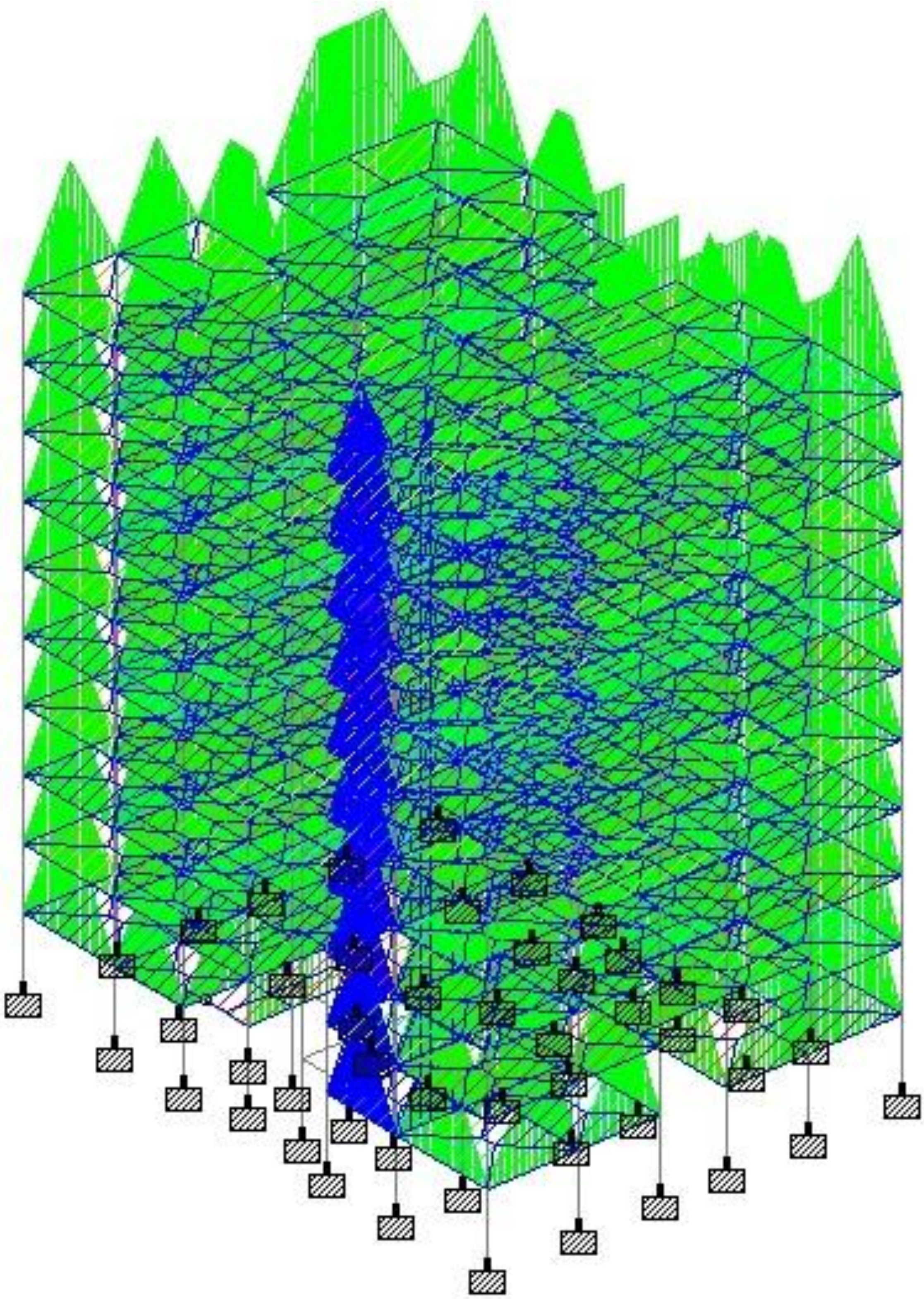


Fig 4.5 Two Loading on the Staad Model

4.3 Analysis and Results

The frame was analysed for the load combination of Self-weight and Live load combinations. Steel design was done according to IS 800-2007 and the member properties were updated as optimized sections after analysis.

Beam Number	Moment(KN-m)	Shear Force(KN)	Section (Manual Calc.)	Section (Staad Pro)
101	29.074	39.00	ISMB175	ISMB200
102	23.292	25.15	ISMB175	ISMB200
105	45.328	39.44	ISMB200	ISMB225
106	140.442	101.52	ISMB300	ISMB400
107	43.367	37.90	ISMB200	ISMB225
111	76	66.00	ISMB225	ISMB300
112	95.14	115.00	ISMB250	ISMB300
118	19.357	22.29	ISMB150	ISMB175

It can be noticed that Staad Pro designs the sections to be bit heavier than the required. Though the structure won’t be as economical but it would be safer structurally. Therefore it can be said that the results given by Staad Pro are verified.

SUPPORT REACTIONS

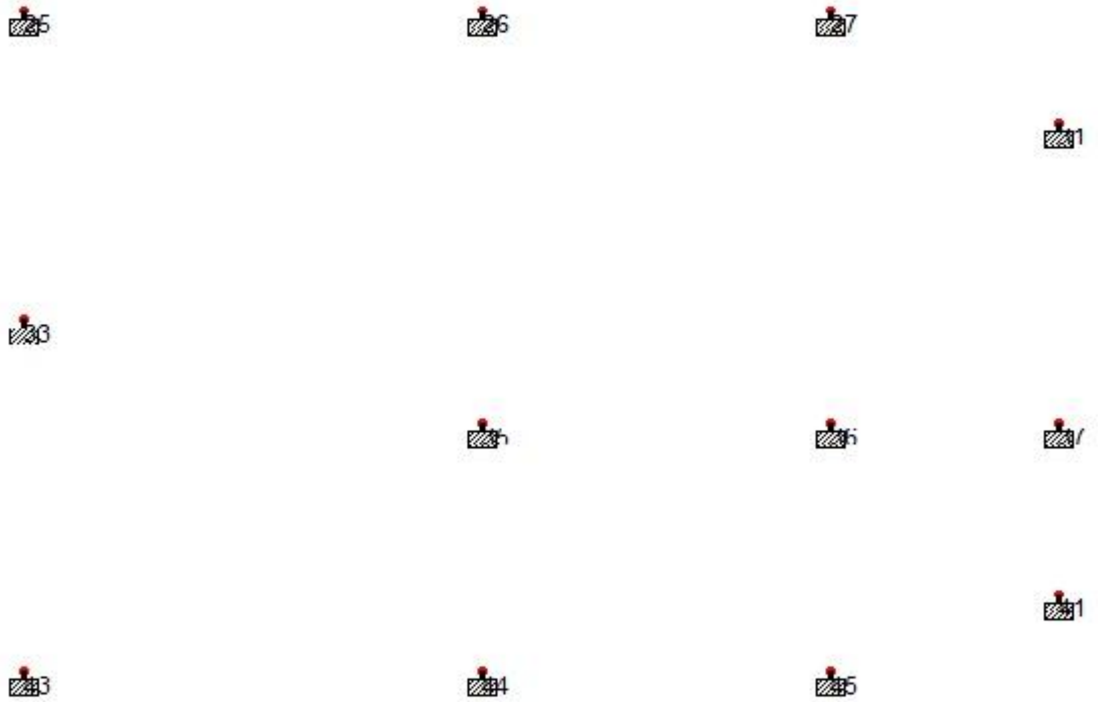


Fig 4.6 Fixed Supports

Node	Force-Y (KN)
25	570.277
26	1361.453
27	767.379
31	348.251
33	1471.686
35	1996.264
36	1641.296
37	448.94
41	172.575
43	659.154
44	841.324
45	445.056

Chapter 5

SEISMIC ANALYSIS

5.1 Statics Method

The seismic load values were calculated as per IS 1893-2002. STAAD.Pro has a seismic load generator in accordance with the IS code mentioned.The seismic load generator can be used to generate lateral loads in the X and Z directions only. Y is the direction of gravity loads. This facility has not been developed for cases where the Z axis is set to be the vertical direction using the “SET Z UP” command.

Methodology:

The design base shear is computed by STAAD in accordance with the IS: 1893(Part 1)-2002.

$$\mathbf{V = A_h * W}$$

Where, $A_h = (Z*I*Sa)/ (2*R*g)$

- STAAD utilizes the following procedure to generate the lateral seismic loads.

i. User provides seismic zone co-efficient and desired "1893(Part 1)-2002 specs" through the DEFINE 1893 LOAD command.

ii. Program calculates the structure period (T).

iii. Program calculates Sa/g utilizing T.

iv. Program calculates V from the above equation. W is obtained from the weight data provided by the user through the DEFINE 1893 LOAD command.

v. The total lateral seismic load (base shear) is then distributed by the program among different levels of the structure per the IS: 1893(Part 1)-2002 procedures.

5.2 Seismic Definitions

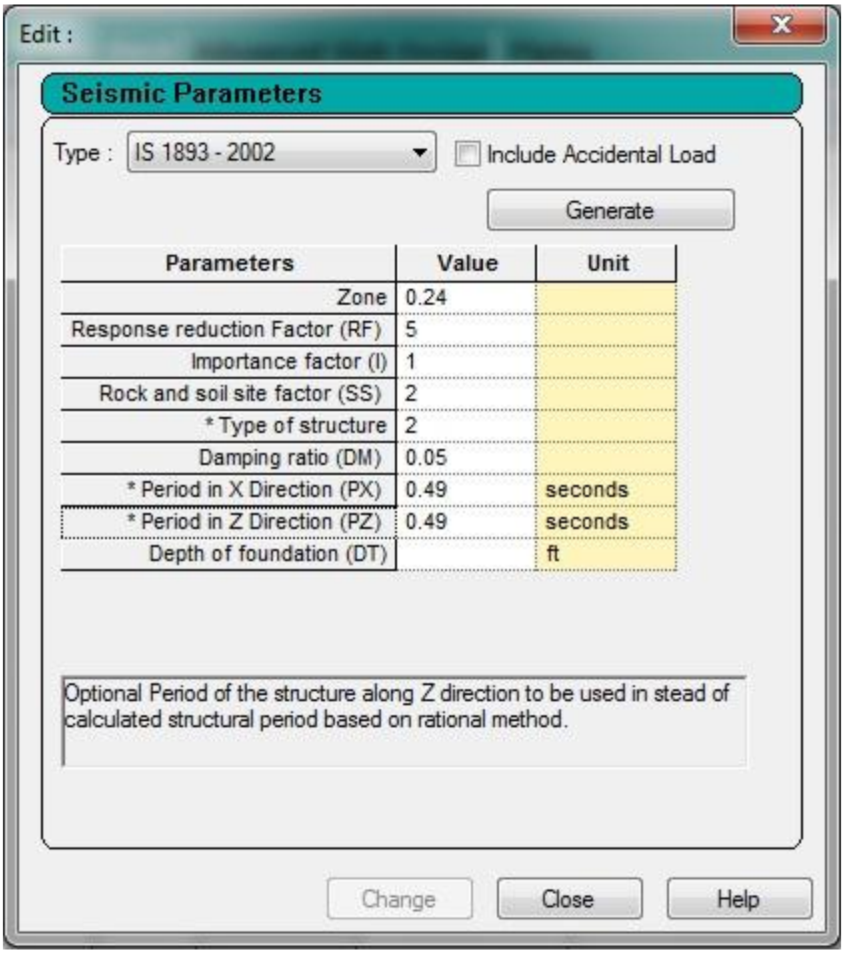


Fig 5.1 Seismic Definition input in Staad Pro

5.3 Loading

Seismic Loading : 1 SEISMIC

Code	Direction	Factor
	X	1.000

Selfweight : 2 DL

Direction	Factor
Y	-1.000

Floor Loads : 3 LL

Load (N/mm^2)	Min Ht. (ft)	Max Ht. (ft)	Min X (ft)	Max X (ft)	Min Y (ft)	Max Y (ft)
-0.011	10.000	110.000	0.000	17.750	0.000	28.584
-0.011	10.000	110.000	17.750	31.250	0.000	10.750
-0.011	10.000	110.000	17.750	31.250	10.750	28.584
-0.011	10.000	110.000	0.000	17.750	39.584	68.168
-0.011	10.000	110.000	17.750	31.250	57.418	68.168
-0.011	10.000	110.000	17.750	31.250	39.584	57.418
-0.011	10.000	110.000	73.000	90.750	39.584	68.168
-0.011	10.000	110.000	59.500	73.000	57.418	68.168
-0.011	10.000	110.000	59.500	73.000	39.584	57.418
-0.011	10.000	110.000	73.000	90.750	0.000	28.584
-0.011	10.000	110.000	59.500	73.000	0.000	10.750
-0.011	10.000	110.000	59.500	73.000	10.750	28.584

One Way Loads : 3 LL

Load (N/mm^2)	Min Ht. (ft)	Max Ht. (ft)	Min X (ft)	Max X (ft)	Min Y (ft)	Max Y (ft)
-0.011	10.000	110.000	31.250	40.000	3.250	23.750
-0.011	10.000	110.000	50.750	59.500	3.250	23.750
-0.011	10.000	110.000	31.250	40.000	44.418	64.918
-0.011	19.000	110.000	50.750	59.500	44.418	64.918

5.4 Results

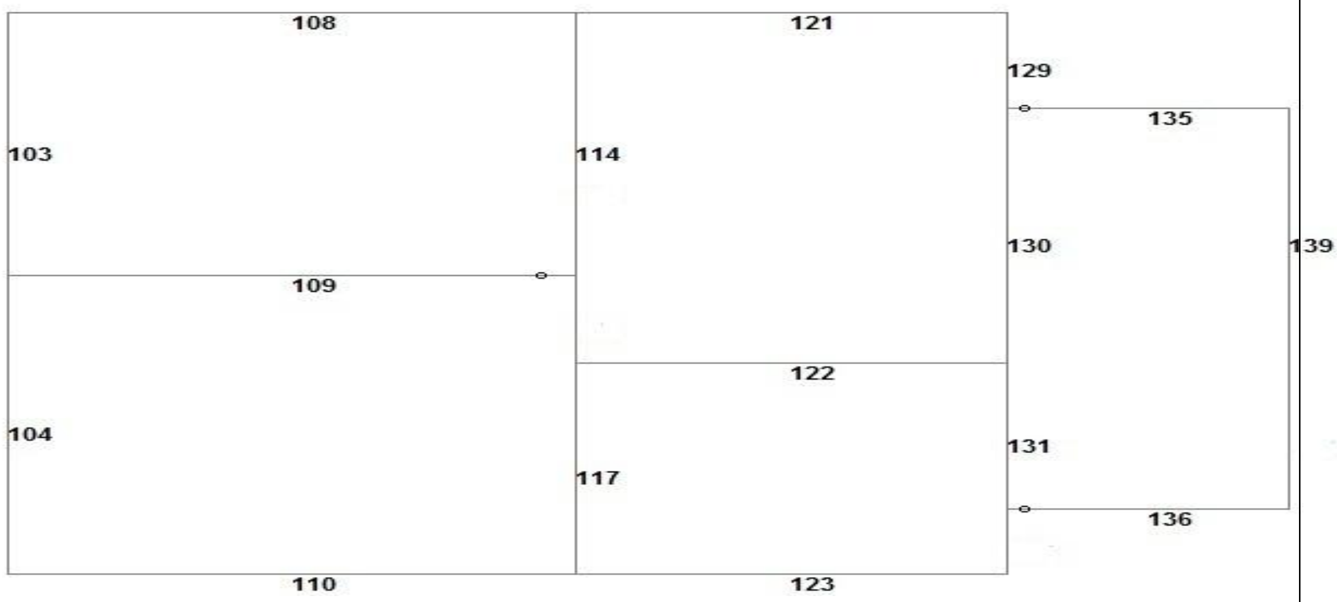


Fig 5.2 Beam Numbers of 1st Floor

Beam	Length ft	Moment(KN-m)	Profile
103	13.417	34.54848	ISMB225
104	15.167	42.11818	ISMB225
108	17.75	231.1455	ISMB600
110	17.75	107.0515	ISMB450
114	13.417	45.82273	ISMB225
117	10.75	75.42424	ISMB300
121	13.5	188.4848	ISMB500
122	13.5	195.2545	ISMB450
123	13.5	131.2121	ISMB500
130	13	86.36364	ISMB300
131	7.5	49.57576	ISMB250
135	8.75	181.4818	ISMB500
136	8.75	27.90606	ISMB225
139	13	25.10909	ISMB225

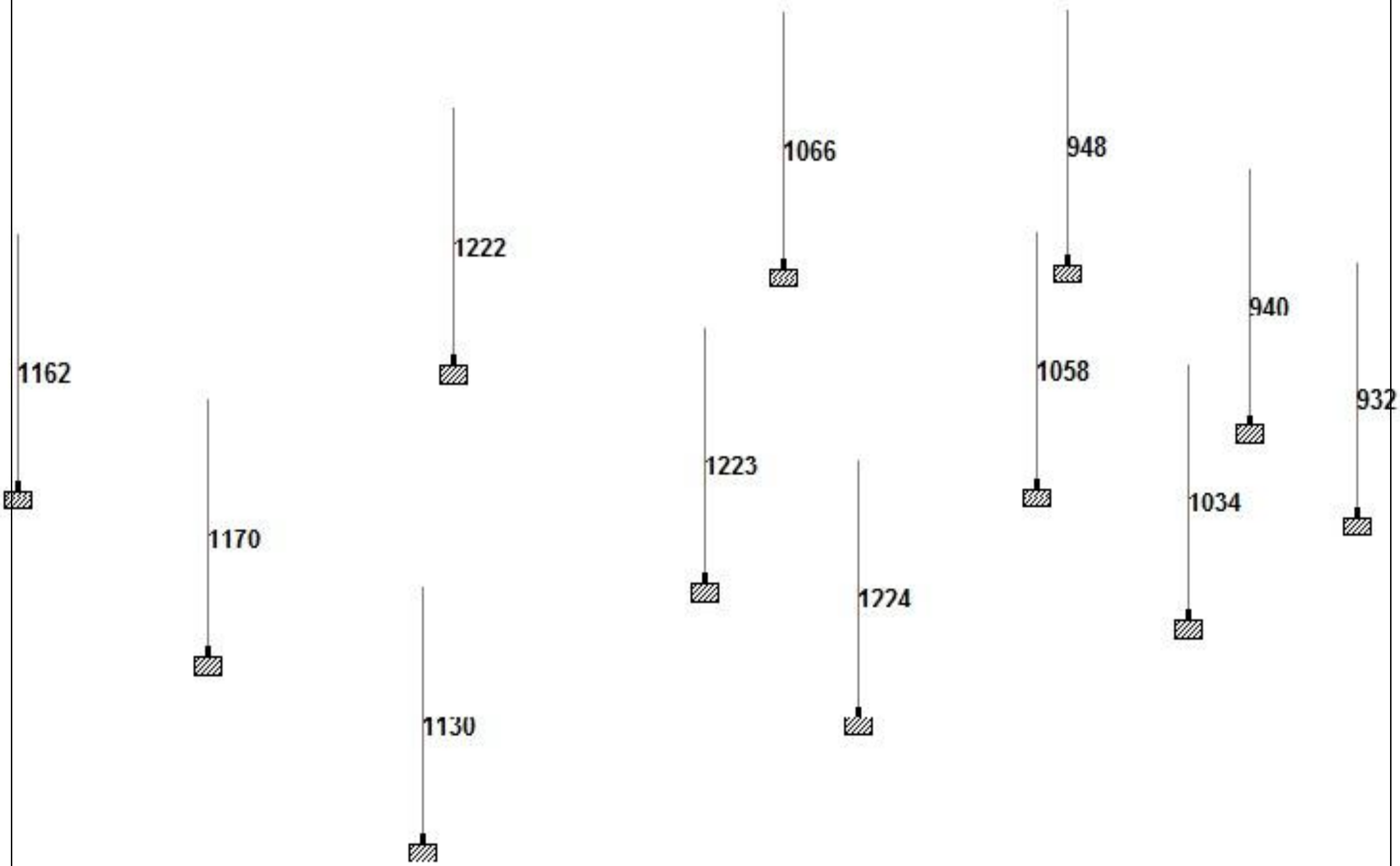


Fig 5.3 Column Numbers on Ground Level

Column	Profile	Axial Force(KN)
932	ISHB200	394.2
940	ISHB200	678.97
948	ISHB350	1028.83
1034	ISWB400	918.65
1058	ISWB600	2729.94
1066	ISHB450	1459.44
1130	ISHB450	1344.5
1162	ISHB400	1328.9
1170	ISWB550	2153
1222	ISWB500	1591.47
1223	ISWB600	2890.9
1224	ISHB400	1180.63

Chapter 6

WIND LOAD

6.1 Introduction

The wind load was calculated according to the IS 875 part 3. The wind pressure varies with

the height of the building. The following formulae are used to calculate the design wind pressure at various height of the building.

$$V_z = V_b \,.\, k_1 .\; k_2 .\, k_3 .\, k_4$$

V_z = design wind speed at any height z in m/s,

k_1 = probability factor (risk coefficient) (see 5.3.1),

k_2= terrain roughness and height factor(see 5.3.2),

k_3= topography factor (see 5.3.3), and

k_4= importance factor for the cyclonic region (see 5.3.4).

$$p_z = \; V_z \, 0.6z^2$$

pz= wind pressure in N/mat height z, and

V_z = design wind speed in m/s at height z.

$$Pd{=}k_d \, k_a \, k_c \;\; p_z$$

Where,

K_d= Wind directionality factor

K_a= Area averaging factor

K_c = Combination factor (See 6.2.3.13)

6.2 Calculations

Basic wind speed, V_b= 47 m/s

Height(m)	V_z(m/s)	P_z(KN/m^2)	P_d(KN/m^2)
10	47	1546.3	1391.67
15	49.33	1703.4	1533.06
20	50.29	1770.35	1593.32
30	52.64	1939.6	1745.45

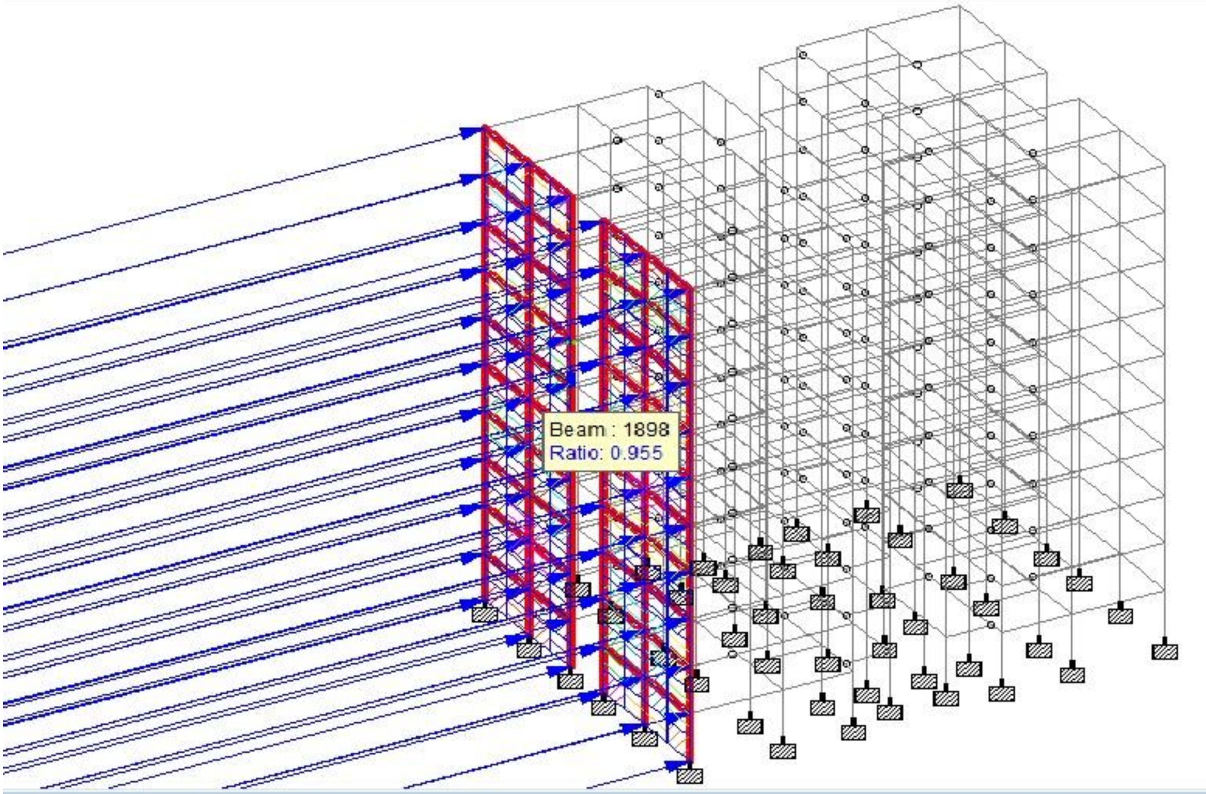


Fig 6.1 Wind loading in X-Direction

6.3 Results

Beam	Length ft	Moment(KN-m)	Profile
103	13.417	24.7	ISMB200
104	15.167	28.2	ISMB200
108	17.75	141.4	ISMB500
109	17.75	102.8	ISMB450
110	17.75	140.5	ISMB500
114	13.417	35.45	ISMB225
117	10.75	54.76	ISMB250
121	13.5	39.25	ISMB250
122	13.5	61.14	ISMB250
123	13.5	38	ISMB300
130	13	64.6	ISMB300
131	7.5	32.2	ISMB125
135	8.75	19.8	ISMB200
136	8.75	12.8	ISMB175
139	13	14.8	ISMB225

Column	Axial Force (KN)	Section
932	261.8	ISWB250
940	486.12	ISHB200
948	442.73	ISHB225
1034	531.49	ISWB350
1058	1924.75	ISHB450
1066	962.94	ISHB350
1130	487.01	ISHB250
1162	383.04	ISHB250
1170	1467.45	ISHB450
1222	1504.9	ISHB450
1223	1915.74	ISHB450
1224	1013.76	ISWB350

Chapter 7

CONCLUSION

The results obtained by analyzing and designing the wind structure for a typical floor are shown below. Section profiles obtained due to seismic and wind loading are compared.

Beam Details		Seismic		Wind	
Beam	Length(ft)	Moment(KN-m)	Section Profile	Moment(KN-m)	Section Profile
103	13.417	34.54848	ISMB225	24.7	ISMB200
104	15.167	42.11818	ISMB225	28.2	ISMB200
108	17.75	231.1455	ISMB600	141.4	ISMB500
110	17.75	107.0515	ISMB450	140.5	ISMB500
114	13.417	45.82273	ISMB225	35.45	ISMB225
117	10.75	75.42424	ISMB300	54.76	ISMB250
121	13.5	188.4848	ISMB500	39.25	ISMB250
122	13.5	195.2545	ISMB450	61.14	ISMB250
123	13.5	131.2121	ISMB500	38	ISMB300
130	13	86.36364	ISMB300	64.6	ISMB300
131	7.5	49.57576	ISMB250	32.2	ISMB125
135	8.75	181.4818	ISMB500	19.8	ISMB200
136	8.75	27.90606	ISMB225	12.8	ISMB175
139	13	25.10909	ISMB225	14.8	ISMB225

Column	Profile Seismic	Axial(KN)	Profile WIND	Axial Force(KN)
932	ISHB200	394.2	ISWB250	261.8
940	ISHB200	678.97	ISHB200	486.12
948	ISHB350	1028.83	ISHB225	442.73
1034	ISWB400	918.65	ISWB350	531.49
1058	ISWB600	2729.94	ISHB450	1924.75
1066	ISHB450	1459.44	ISHB350	962.94
1130	ISHB450	1344.5	ISHB250	487.01
1162	ISHB400	1328.9	ISHB250	383.04
1170	ISWB550	2153	ISHB450	1467.45
1222	ISWB500	1591.47	ISHB450	1504.9
1223	ISWB600	2890.9	ISHB450A	1915.74
1224	ISHB400	1180.63	ISWB350	1013.76

It is evident from the above results that the frame designed for the seismic load is also able to sustain the wind load separately as it is evident that seismic load and wind load do not occur simultaneously. Therefore the sections obtained for seismic loading are considered as final and should be considered for the design of the frame.

APPENDIX A

Section Property Table

Destination	Weight per Metre	Sectional Area	Depth of Section (D)	Width of Flange (b_f)	Thickness of Flange (t_f)	Thickness of Web (t_w)	Radii of Gyration (r_z)	(r_y)	Section Modulus (Z_{ez})	Plastic modulus (Z_{pz})
	kg/m	cm^2	mm	mm	mm	cm	cm	cm	cm^3	cm^3
ISWB600	145.1	184.86	600	250	23.6	11.8	25.01	5.35	3854.2	4341.63
ISWB600	133.7	170.38	600	250	21.3	11.2	24.97	5.25	3540	3986.66
ISMB600	122.6	156.21	600	210	20.8	12	24.24	4.12	3060.4	3510.63
ISWB550	112.5	143.34	550	250	17.6	10.5	22.86	5.11	2723.9	3066.29
ISMB500	103.7	132.11	550	190	19.3	11.2	22.16	3.73	2359.8	2711.98
ISWB500	95.2	121.22	500	250	14.7	9.9	20.77	4.96	2091.6	2351.35
ISMB500	86.9	110.74	500	180	17.2	10.2	20.21	3.52	1808.7	2074.67
ISHB450	92.5	117.89	450	250	13.7	11.3	18.5	5.08	1793.3	2030.95
ISHB450	87.2	111.14	450	250	13.7	9.8	18.78	5.18	1742.7	1955.03
ISWB450	79.4	101.15	450	200	15.4	9.2	18.63	4.11	1558.1	1760.59
ISHB400	82.2	104.66	400	250	12.7	10.6	16.61	5.16	1444.2	1626.36
ISHB400	77.4	98.66	400	250	12.7	9.1	16.87	5.26	1404.2	1556.33
ISMB450	72.4	92.27	450	150	17.4	9.4	18.15	3.01	1350.7	1533.36
ISLB450	65.3	83.14	450	170	13.4	8.6	18.2	3.2	1223.8	1401.35
ISWB400	66.7	85.01	400	200	13	8.6	16.6	4.04	1171.3	1290.19
ISHB350	72.4	92.21	350	250	11.6	10.1	14.65	5.22	1131.6	1268.69
ISHB350	67.4	85.91	350	250	11.6	8.3	14.93	5.34	1094.8	1213.53
ISMB400	61.6	78.46	400	140	16	8.9	16.15	2.82	1022.9	1176.18
ISLB400	56.9	72.43	400	165	12.5	8	16.33	3.15	965.3	1099.45
ISWB350	56.9	72.5	350	200	11.4	8	14.63	4.03	887	995.49
ISHB300	63	80.25	300	250	10.6	9.4	12.7	5.29	863.3	962.18
ISHB300	58.8	74.85	300	250	10.6	7.6	12.95	5.41	836.3	921.68
ISMB350	52.4	66.71	350	140	14.2	8.1	14.29	2.84	778.9	889.57
ISWB300	48.1	61.33	300	200	10	7.4	12.66	4.02	654.8	731.21

ISHB250	54.7	69.71	250	250	9.7	8.8	10.7	5.37	638.7	708.43
ISLB325	43.1	54.9	325	165	9.8	7	13.41	3.05	607.7	687.76
ISHB250	51	64.96	250	250	9.7	6.9	10.91	5.49	618.9	678.73
ISMB300	44.2	56.26	300	140	12.4	7.5	12.37	2.84	573.6	651.74
ISHB225	46.8	59.66	225	225	9.1	8.6	9.58	4.84	487	542.22
ISWB250	40.9	52.05	250	200	9	6.7	10.69	4.06	475.4	527.57
ISHB225	43.1	54.94	225	225	9.1	6.5	9.8	4.96	469.3	515.82
ISHB200	40	50.94	200	200	9	7.8	8.55	4.42	372.2	414.23
ISHB200	37.3	47.54	200	200	9	6.1	8.71	4.51	360.8	397.23
ISWB225	33.9	43.24	225	150	9.9	6.4	9.52	3.22	348.5	389.93
ISMC250	30.4	38.67	250	80	14.1	7.1	9.94	2.38	305.3	356.72
ISMB225	31.2	39.72	225	110	11.8	6.5	9.31	2.34	305.9	348.27
ISLB250	27.9	35.53	250	125	8.2	6.1	10.23	2.33	297.4	338.69
ISWB200	28.8	36.71	200	140	9	6.1	8.46	2.99	262.5	293.99
ISMB200	25.4	32.33	200	100	10.8	5.7	8.32	2.15	223.5	253.86
ISHB150	34.6	44.08	150	150	9	11.8	6.09	3.35	218.1	251.64
ISHB150	30.6	38.98	150	150	9	8.4	6.29	3.44	205.3	232.52
ISHB150	27.1	34.48	150	150	9	5.4	6.5	3.54	194.1	215.64
ISWB175	22.1	28.11	175	125	7.4	5.8	7.33	2.59	172.5	194.2
ISLB200	19.8	25.27	200	100	7.3	5.4	8.19	2.13	169.7	184.34
ISMB175	19.3	24.62	175	90	8.6	5.5	7.19	1.86	145.4	166.08
ISWB150	17	21.67	150	100	7	5.4	6.22	2.09	111.9	126.86
ISMB150	14.9	19	150	80	7.6	4.8	6.18	1.66	96.9	110.48
ISMB125	13	16.6	125	75	7.6	4.4	5.2	1.62	71.8	81.85
ISMB100	11.5	14.6	100	75	7.2	4	4.2	1.67	51.5	58.65

APPENDIX B

Risk coefficients for different classes of structures in different wind speed zones

Class of Structure	Mean Probable design life of structure in years	k_1 factor for Basic Wind Speed (m/s) of					
		33	39	44	47	50	55
All general buildings and structures	50	1.0	1.0	1.0	1.0	1.0	1.0
Temporary sheds, structures such as those used during construction operations (for example, formwork and false work), structures during construction stages, and boundary walls	5	0.82	0.76	0.73	0.71	0.70	0.67
Buildings and structures presenting a low degree of hazard to life and property in the event of failure, such as isolated towers in wooded areas, farm buildings other than residential buildings, etc.	25	0.94	0.92	0.91	0.90	0.90	0.89
Important buildings and structures such as hospitals, communication buildings, towers and power plant structures	100	1.05	1.06	1.07	1.07	1.08	1.08

NOTE – The factor k_1 is based on statistical concepts, which take account of the degree of reliability required, and period of time in years during which there will be exposure to wind, that is, life of the structure. Whatever wind speed is adopted for design purposes, there is always a probability (howsoever small) that it may be exceeded in a storm of exceptional violence; the greater the number of years over which there will be exposure to wind, the greater is the probability. High return periods ranging from 100 to 1000 years (implying lower risk level) in association with greater period of exposure may have to be selected for exceptionally important structures, such as, nuclear power reactors and satellite communication towers. Equation given below may be used in such cases to estimate k_1 factors for different periods of exposure and chosen probability of exceedence (risk level). The probability level of 0.63 is normally considered sufficient for design of buildings and structures against wind effects and the values of k_1 corresponding to this risk level are given above.

$$k_1 = \frac{X_{N,P_N}}{X_{50,\,0.63}} = \frac{A - B\left[\ln\left\{-\frac{1}{N}\ln\left(1 - P_N\right)\right\}\right]}{A + 4B}$$

where

N = mean probable design life of the structure in years;

P_N = risk level in N consecutive years (probability that the design wind speed is exceeded at least once in N successive years), nominal value = 0.63;

$X_{N,P}$ = extreme wind speed for given value of N and P_N; and

$X_{50,0.63}$ = extreme wind speed for N = 50 years and P_N = 0.63

A and B are coefficients having the following values for different basic wind speed zones:

Zone	A	B
33 m/s	83.2	9.2
39 m/s	84.0	14.0
44 m/s	88.0	18.0
47 m/s	88.0	20.5
50 m/s	88.8	22.8
55 m/s	90.8	27.3

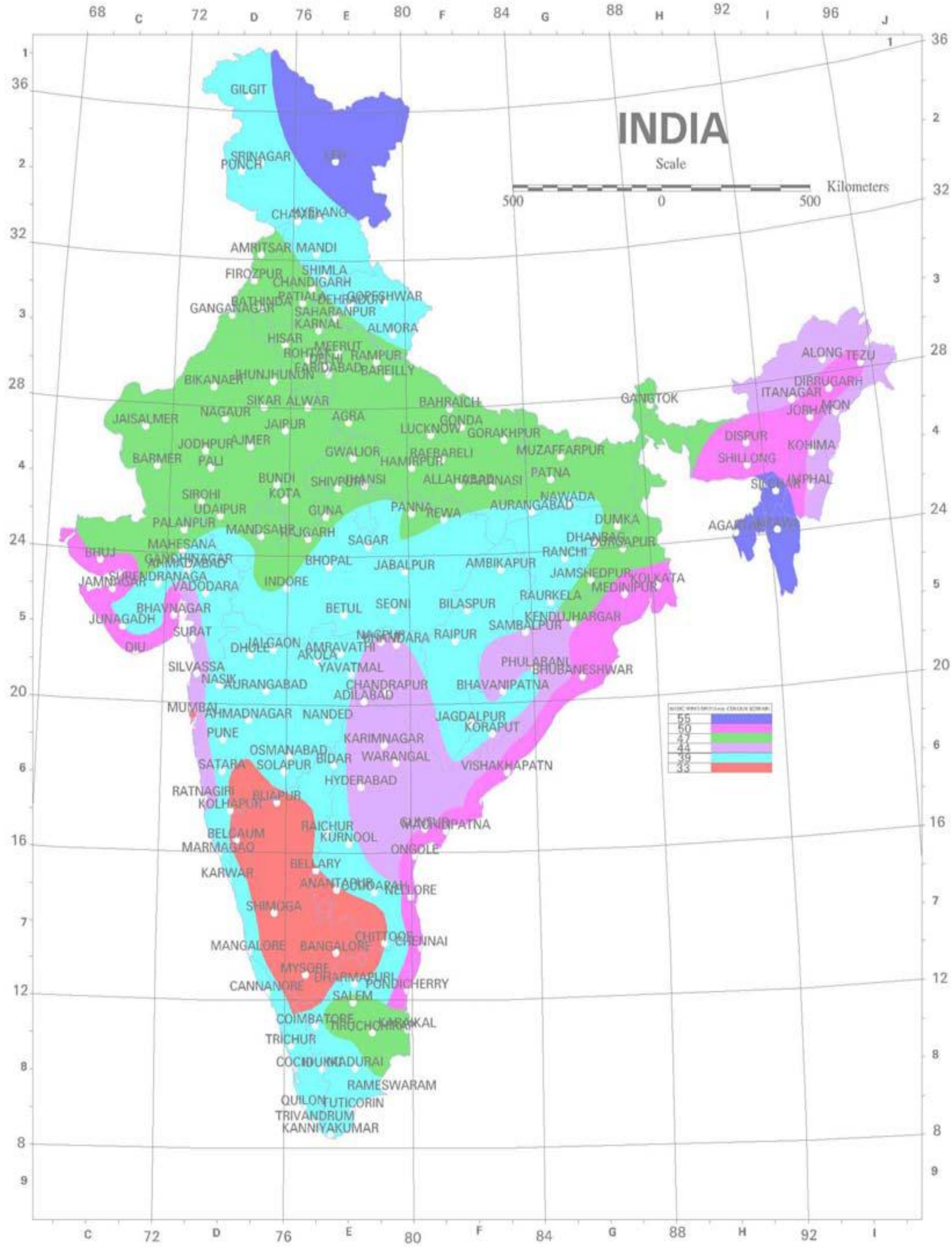
k_2 factors to obtain design wind speed variation with height in different terrains

Height (z) (m)	Terrain and height multiplier (k_2)			
	Terrain Category 1	Terrain Category 2	Terrain Category 3	Terrain Category 4
10	1.05	1.00	0.91	0.80
15	1.09	1.05	0.97	0.80
20	1.12	1.07	1.01	0.80
30	1.15	1.12	1.06	0.97
50	1.20	1.17	1.12	1.10
100	1.26	1.24	1.20	1.20
150	1.30	1.28	1.24	1.24
200	1.32	1.30	1.27	1.27
250	1.34	1.32	1.29	1.28
300	1.35	1.34	1.31	1.30
350	1.37	1.36	1.32	1.31
400	1.38	1.37	1.34	1.32
450	1.39	1.38	1.35	1.33
500	1.40	1.39	1.36	1.34

NOTE: For intermediate values of height z and terrain category, use linear interpolation.

Tributary Area (A) (m^2)	Area Averaging Factor (K_a)
≤ 10	1.0
25	0.9
≥ 100	0.8

Area averaging factor (K_a)



Map Showing the Basic Wind Speed for Different Zones

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