STRUCTURAL ANALYSIS AND DESIGN REPORT

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1. INTRODUCTION

The basic aim of the structural design is to build a structure, which is safe, fulfilling the intended purpose during its estimated life span, economical in terms of initial and maintenance cost, durable and also maintaining a good aesthetic appearance.

A building is considered to be structurally sound, if the individual elements and the building as a whole satisfy the criteria for strength, stability and serviceability and in seismic areas additional criteria for ductility and energy absorption capabilities. The overall building must be strong enough to transfer all loads through the structure to the ground without collapsing or losing structural integrity by rupture of the material at the critical sections, by transformation of the whole or parts into mechanisms or by instability.

2. SEISMIC VULNERABILITY OF NEPAL

Nepal is located in the boundary of two colliding tectonic plates, namely, the Indian Plate (Indo-Australian Plate) and the Tibetan Plate (Eurasian Plate). The Indian Plate is constantly moving under the Tibetan Plate causing many minor and major earthquakes in this region. As a result, Nepal has witnessed many major as well as minor earthquakes during the past. Records of earthquakes are available in Nepal since 1255 A.D. Those records show that around 18 major earthquakes have shaken Nepal since then. The 1833 A.D. earthquake and 1934 A.D Bihar-Nepal earthquakes and 2015 Gorkha earth quake were the most destructive ones in the history of Nepal.

Thus structures to be built in Nepal need to be suitably designed and detailed, so as to counteract the forces due to earthquakes.

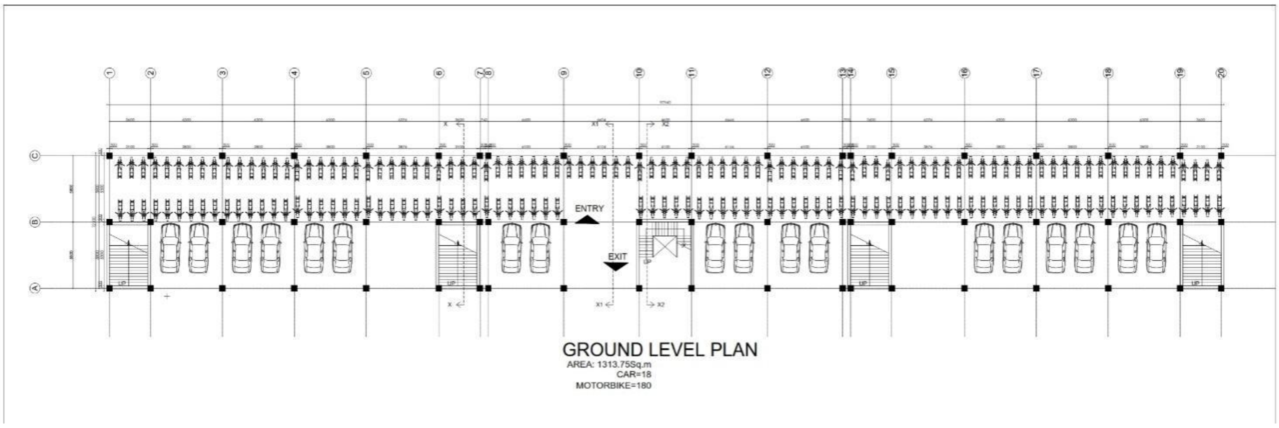
3. PHILISOPHY OF SEISMIC DESIGN

The probability of occurrence of severe earthquakes is much less than that of minor earthquakes at a given site. Many of the structures may never experience severe earthquakes during its lifetime. Construction of any ordinary structures to resist such severe earthquakes without undergoing any damage may not be considered economically feasible, as it may be far cheaper to repair or even rebuild the structure after having severe and strong shaking. On the other hand, structures located in seismic areas experience minor earthquakes rather frequently. Thus, in the event of severe and strong shaking, the structure is allowed to have some damage which may be repairable or even irreparable, but the structure will not be allowed to collapse completely, thereby ensuring the safety of life and the property in the structure. In order that one does not have to undertake frequent repair and retrofitting of the structure, the structure should not have any damage during minor level of shaking. In case of moderate shaking the structure is allowed to have some non-structural damage without endangering life and property within the structure. During such event the level of damage should be such that it can be economically repaired.

The structures are generally designed for much lower seismic forces than what it may actually experience during its life time. Since the structure is expected to undergo damage in the event of a severe shaking, reliance is placed on the inelastic response of the structure beyond yield. Therefore, structures have to be ductile and capable of dissipating energy through inelastic actions. Ductility can be achieved by avoiding brittle modes of failures. Brittle modes of failures include, shear and bond failure. Thus, structures should be designed on Weak beam-Strong column philosophy.

4. BUILDING DESCRIPTION

Type:	Stadium
Building Typology:	Reinforcement Concrete Frame Building
Form:	
Plan Shape:	Irregular shaped
Plan Configuration:	Irregular
Vertical Configuration:	Irregular
Plinth Area:	1019.25m^2
Number of Stories	Block A1,A3 (3 Storey), Block A2 (4 Storey) Block B (5 Storey) Block C (3 Storey)
Position of the Building:	Free Standing
Total Height:	Block A1,A3 (10m), Block A2 (13m) Block B (17.266) Block C (10m)
Inter Storey Height:	GF – 4m & Rest 3m (For Block A) GF-1.5m, First Floor-4m,Top Floor-2.766 & Rest-3m (Block B) GF – 4m & Rest 3m (For Block C)
Maximum length of Beam:	6.533m
Size of Columns:	525 x 525 mm^2
Wall Thickness:	230mm
Floor/Roof structure:	150mm slab floor and 175mm Slant Slab
Location of site:	Suryabinayak



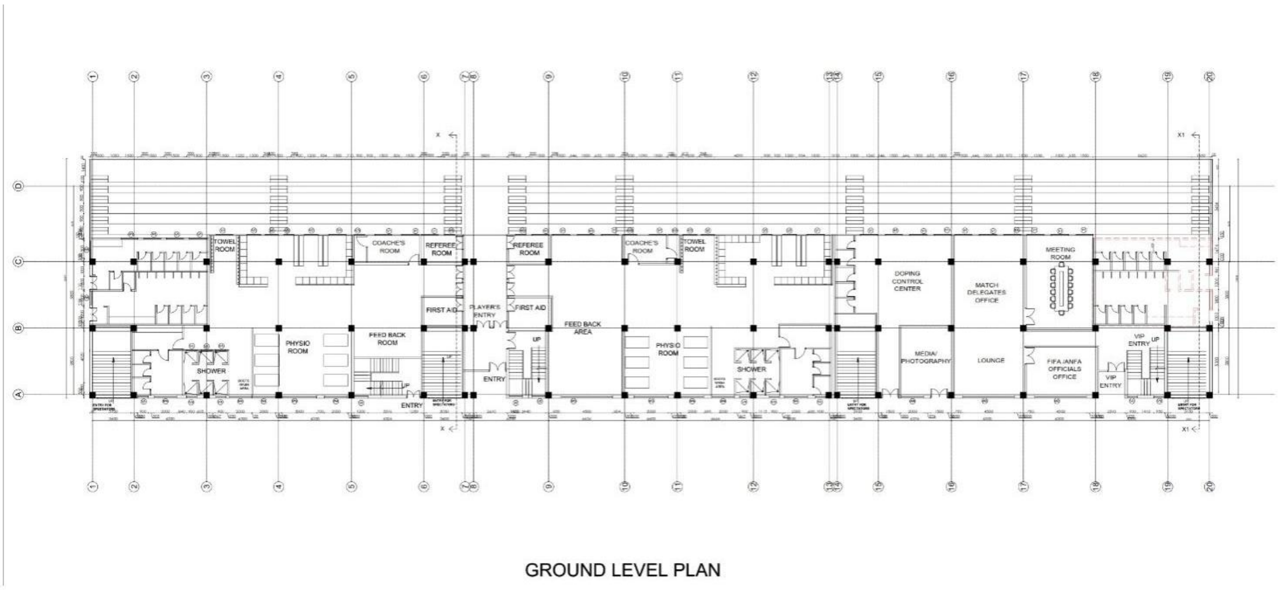


Figure 1: Ground Floor Plan Block A and Block B (Refer Drawing for Detail)

5. STRUCTURAL SYSTEM

Material:	Reinforced Cement Concrete
Frame System:	Special Moment Resisting Frame
Floor System:	Two way Solid Slab
Foundation System:	Mat footing

Material Strengths:

Member	Concrete Grade
Columns	M25
Beams	M25
Slabs	M25
Foundation	M25

Steel

Steel Type	Grade
Thermo mechanically Treated Bar(TMT)	Fe 500

6. LOADS ADOPTED

Load calculation is done using the NBC 102:1994 as reference. At first type of material is selected and value of unit weight of the materials is taken from the above mentioned code. Thickness of the material is selected as per the design requirement. Knowing area, thickness and unit weight of materials, loads on each section is found.

The following are assumed for detail load calculation.

- R.C.C Slab, Beam and Column = 25.0 KN/m^3
- Screed (25mm thick) = 19.2 KN/m^3
- Cement Plaster (20mm thick) = 20.40 KN/m^3
- Marble Dressed = 26.50 KN/m^3
- Telia Brick = 19 KN/m^3

Live Load

Live load for the floor and Roof is taken from IS 875 part 2 as referred by National building code. For Stadium, Following load has be taken (Table 1, IS 875 Part 2)

Corridors, passages, staircases including fire escapes - 3 KN/m2
Slant Slab for Sitting Stadium- 5 KN/m2
Office rooms– 3 KN/m2
Kitchen, Cafeteria - 3 KN/m2

For Roof Load, Table 2 of IS 875 part 2 has been taken for the estimation

Flat, sloping or curved roof with slopes up to and including 10 degrees
Access not provided except for maintenance –0.75 KN/m^2

Floor Finish

Floor Finish Load is calculated Simple Marble Finishes. With load calculation

Depth of Finishes = 0.055 m
Marble Dressed = 26.50 KN/m^3
Weight per Square meter = 0.055 * 26.5 = 1.458 KN/m^2 (Assume 1.5 KN/m^2)
Slant Slab for Sitting Stadium = 5.75 KN/m^2

Wall Loads

Wall loads are applied on underneath beam if wall is rested on the beam. For partition wall load is applied as the area load intensity. Load intensity is calculated by dividing total weight of partition wall by the area of given slab portion.

7. SEISMIC DESIGN PARAMETERS

The seismic design parameters have been considered in reference with NBC 105-2020 and are presented as follows:

Seismic Zone Factor

Seismic Zone Factor (Z)	Z
	0.35

Importance Factor

Building Occupancy Type	I
Stadium	1.25

Site Soil Category

Soil Type	Soft Soil(Type D)

8. PRELIMINARY DESIGN

For the analysis, dead load is also necessary which depends upon the size of member itself. So it is necessary to pre-assume logical size of member which will neither overestimate the load nor under estimate the stiffness of the building. So, the tentative sizes of the structural elements are determined through the preliminary design so that the pre-assumed dimensions may not deviate considerably after analysis thus making the final design both safe and economical. Tentative sizes of various elements have been determined as follows:

Slab:

Preliminary design of slab is done as per the deflection criteria as directed by code Clause 23.2.1 of [IS 456: 2000]. The cover provided is 20 mm and the grade of concrete used in the design is M25.

According to which,

$$\frac{\text{Span}}{\text{Eff. Depth}} \leq (M_{ft} \times M_{fc}) \times \text{Basic Value}$$

Where, the critical span is selected which is the maximum shorter span among the all slab element. This is done to make uniformity in slab thickness. The amount of reinforcement will be varied slab to slab but the thickness will be adopted corresponding to the entire slab.

Beam:

Preliminary design of the beam is done as per the deflection criteria as directed by code Clause 23.2.1 of [IS 456: 2000] and ductility criteria of ACI code. The cover provided is 30 mm and the grade of concrete used in the design is M25.

According to which,

$$\frac{\text{Span}}{\text{Eff. Depth}} \leq (M_{ft} \times M_{fc}) \times \text{Basic Value} \times \text{Correction Factor for span} \times \text{Correction Factor for Flange}$$

But,

According to Ductility code, Spacing of Stirrups in beam should not exceed d/4 or 8 times diameter of minimum size of bar adopted and should not greater than 100mm. So, for considering construction difficulties in actual field, it is logical to use d/4 as spacing as per the construction practice in Nepal.

COLUMN:

Preliminary design of column is done from the assessment of approximate factored gravity loads and live loads coming up to the critical section. To compensate the possible eccentric loading and earthquake loads the size is increased by about 25% in design. For the load acting in the column, live load is decreased according to IS 875: 1978. Initially a rectangular column is adopted in this building project so as to provide internal aesthetics required from architecture point of view but the column size and shape will vary as per the requirement for the analysis, design and aesthetic value. The cover provided is 40 mm and the grade of concrete used in the column design is M25.

9. FINITE ELEMENT MODELING AND ANALYSIS OF BUILDING USING ETABS V19

The FE model of building is developed in ETABS V19, a general purpose FE analysis and design software. The size of beams and columns as obtained from preliminary analysis are adjusted according to architectural need. Beam and columns are modeled as frame element. Slabs are also modeled as shell element.

Beam and column are assumed to be line element having six degree of freedom at each node and slab is assumed to be shell element having six degree of freedom at each node. Floor diaphragm is used in the structure to reduce degree of freedom to three for each floor level.

Imposed loads have been modeled as uniform distributed loads. Similarly, wall loads are modeled as uniformly distributed line loads. The columns and walls were “fixed” at their base.

The 3D model is assumed to be fixed at tie beam level. Suitable assumptions are made and FE model as shown in Fig 2 is developed.

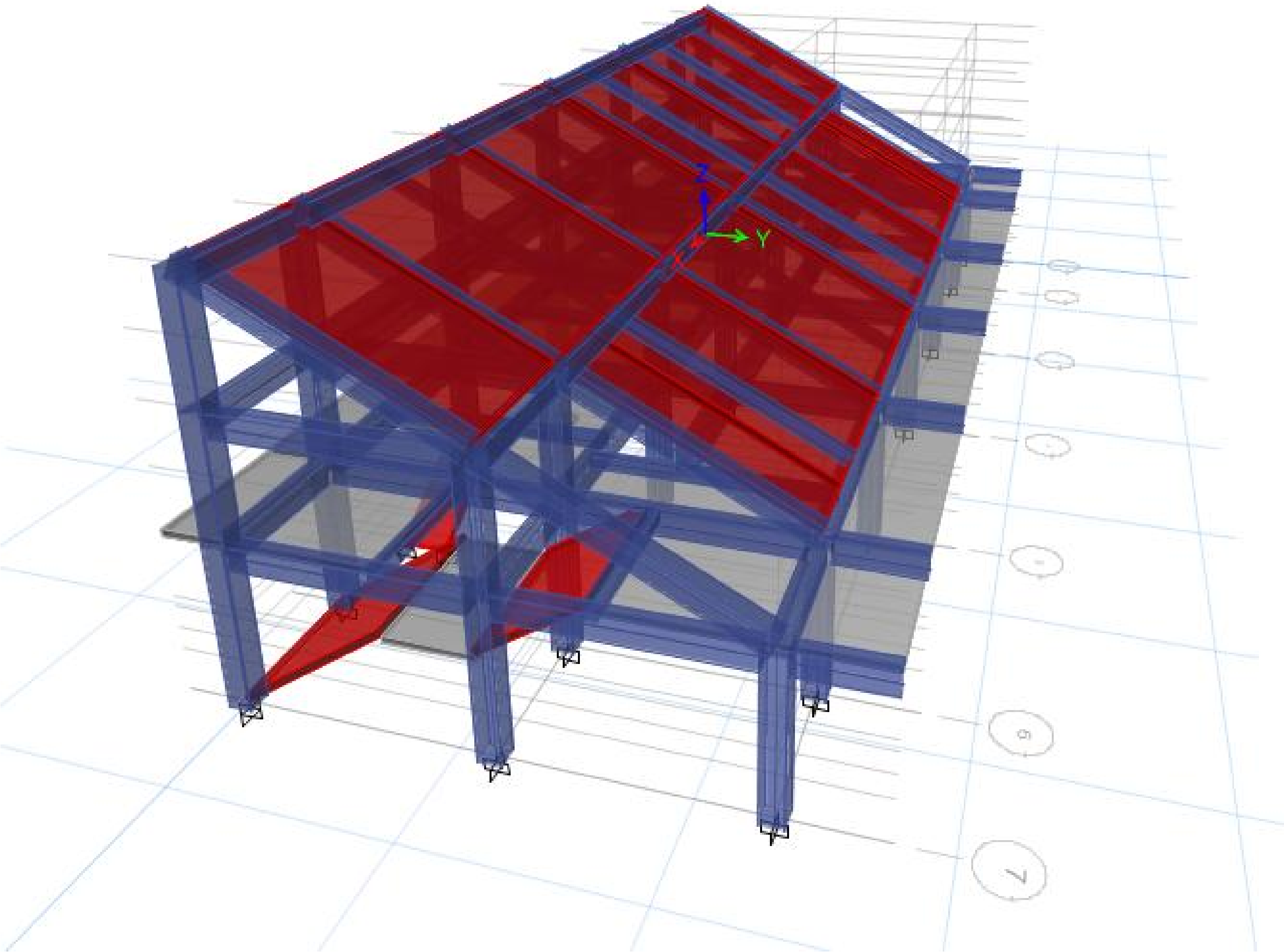


Figure 2: Finite Elemental Modeling in ETABS V19 (Block A 1 & 3)

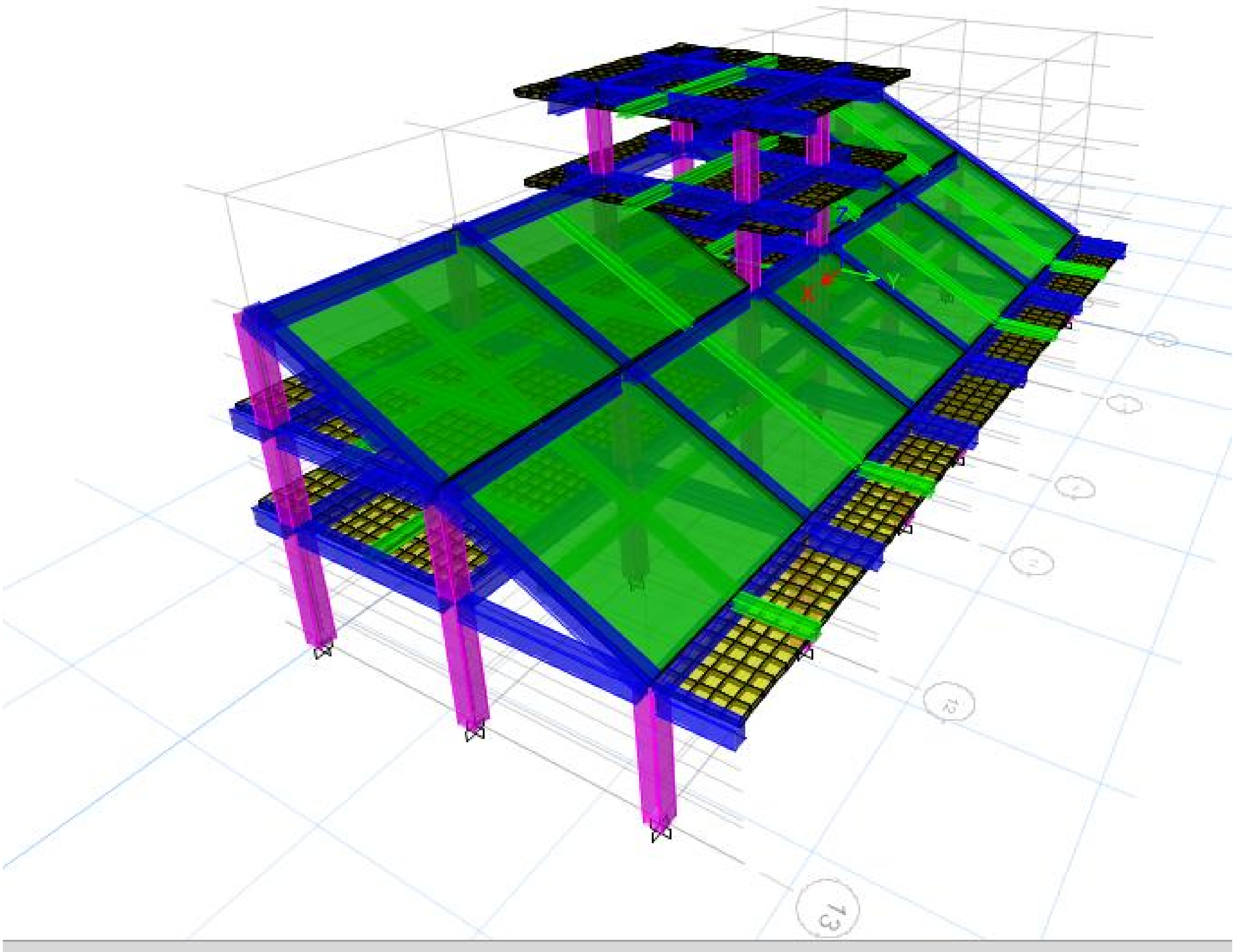


Figure 3: Finite Elemental Modeling in ETABS V19 (Block A 2)

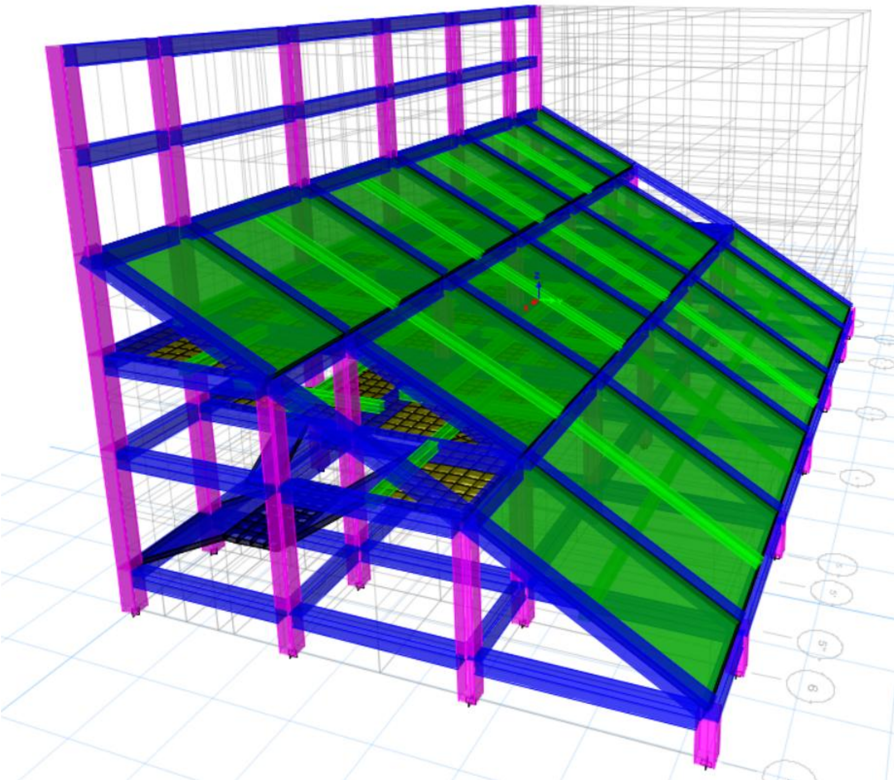


Figure 4: Finite Elemental Modeling in ETABS V19 (Block B 1 & 3)

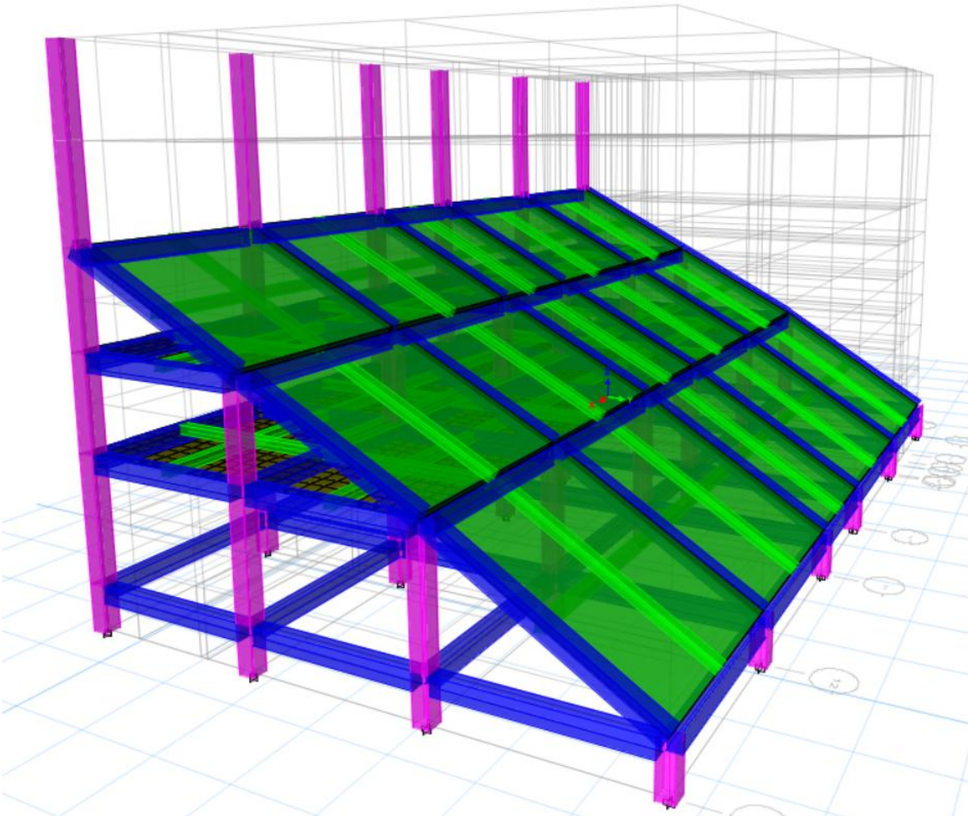


Figure 5: Finite Elemental Modeling in ETABS V19 (Block B 2)

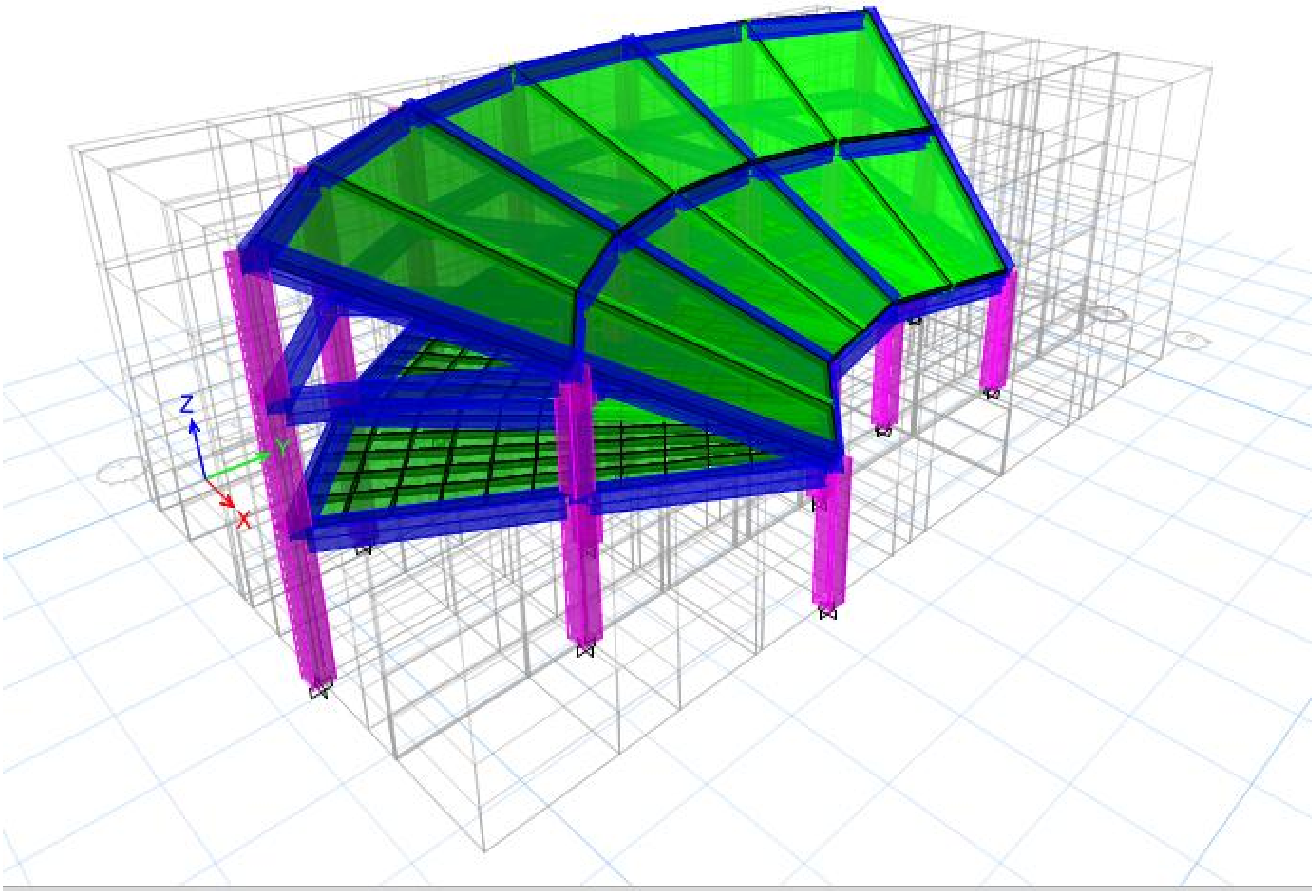


Figure 5: Finite Elemental Modeling in ETABS V19 (Block C)

Loading due to wall, floor finish and live load is as shown in figure below and analysis is done by static method (seismic coefficient method). Following forces is observed during Analysis:

9.1 LOADS APPLIED ON BUILDING:

9.1.1 Floor Finish

This load is applied all over the slab. Load application is shown in figure below.

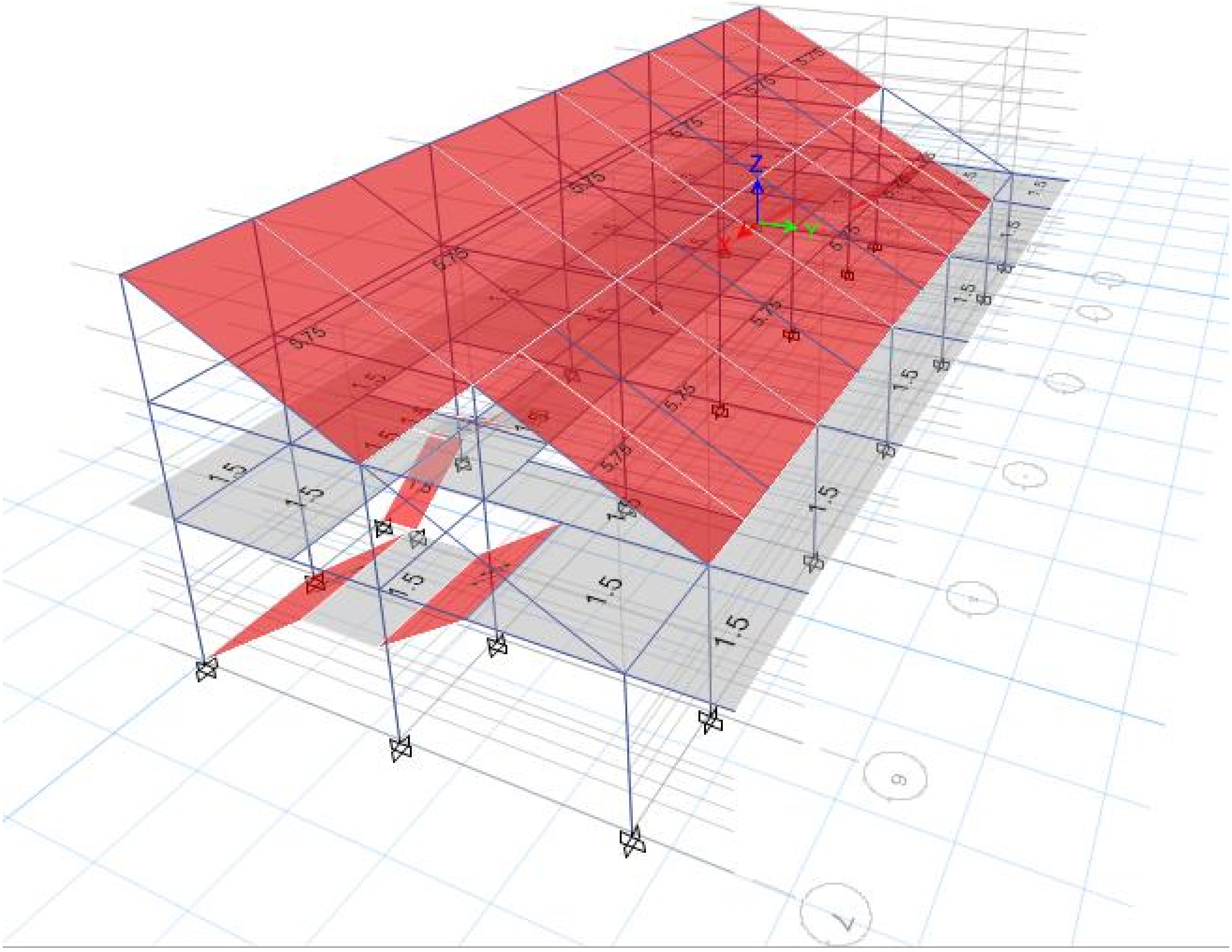


Figure 6: Floor Finish load (Block A 1 & 3)

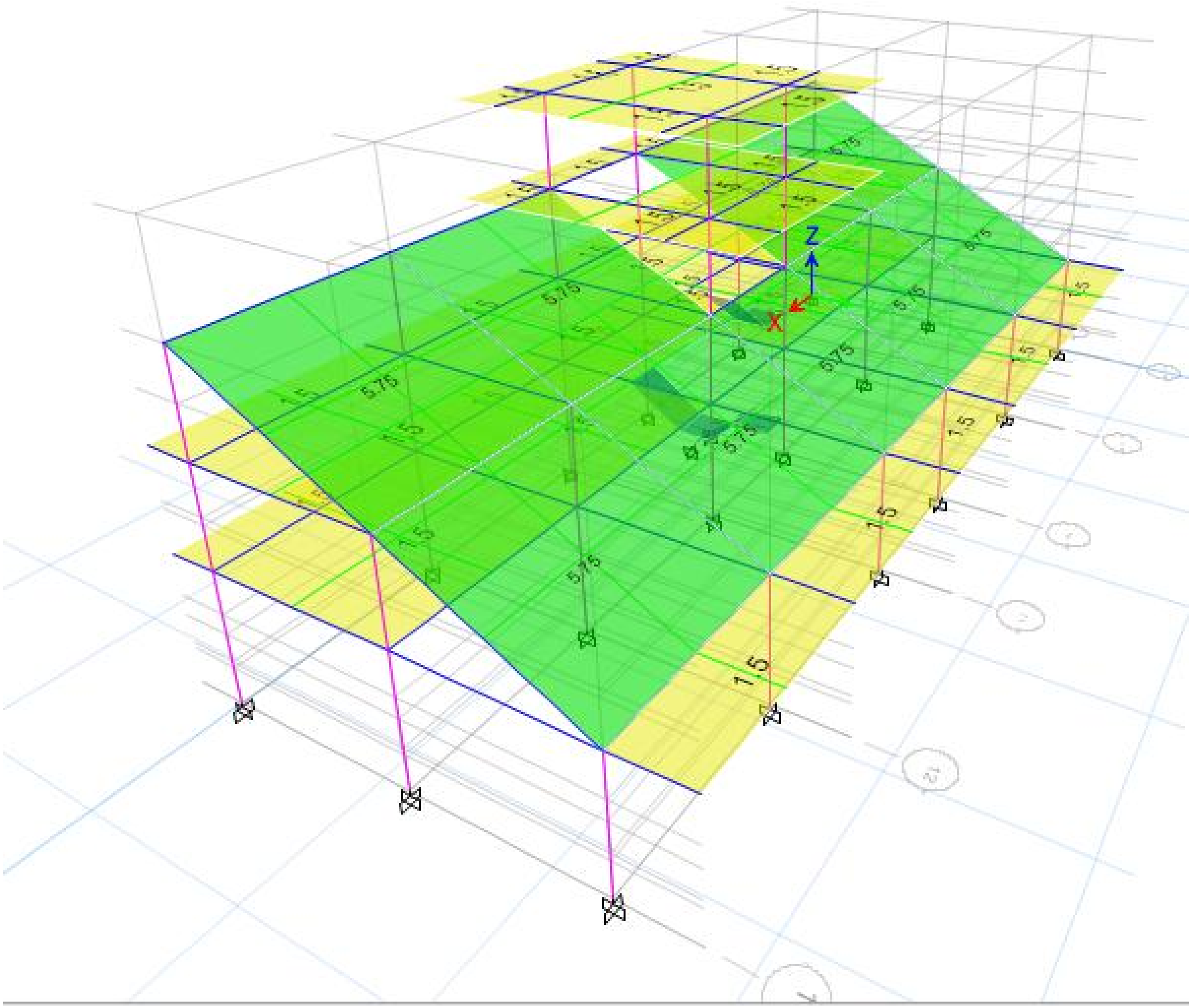


Figure 7: Floor Finish load (Block A 2)

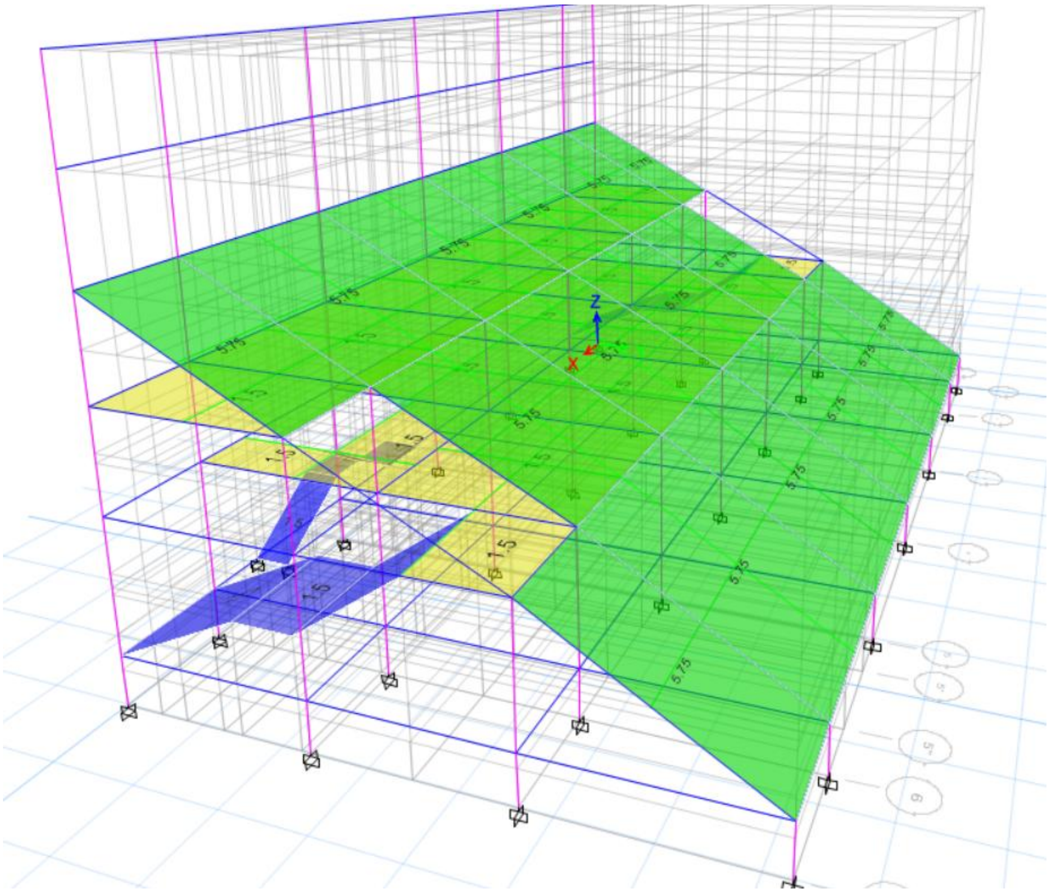


Figure 8: Floor Finish load (Block B 1 & 3)

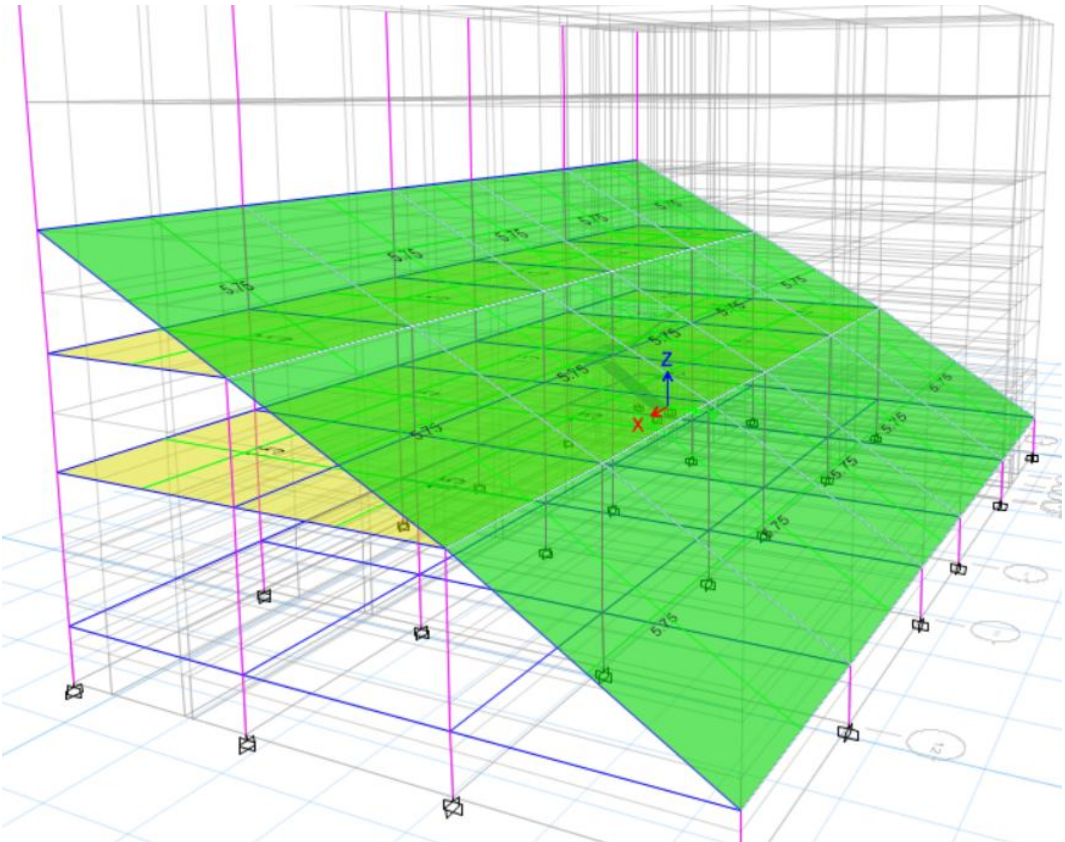


Figure 9: Floor Finish load (Block B 2)

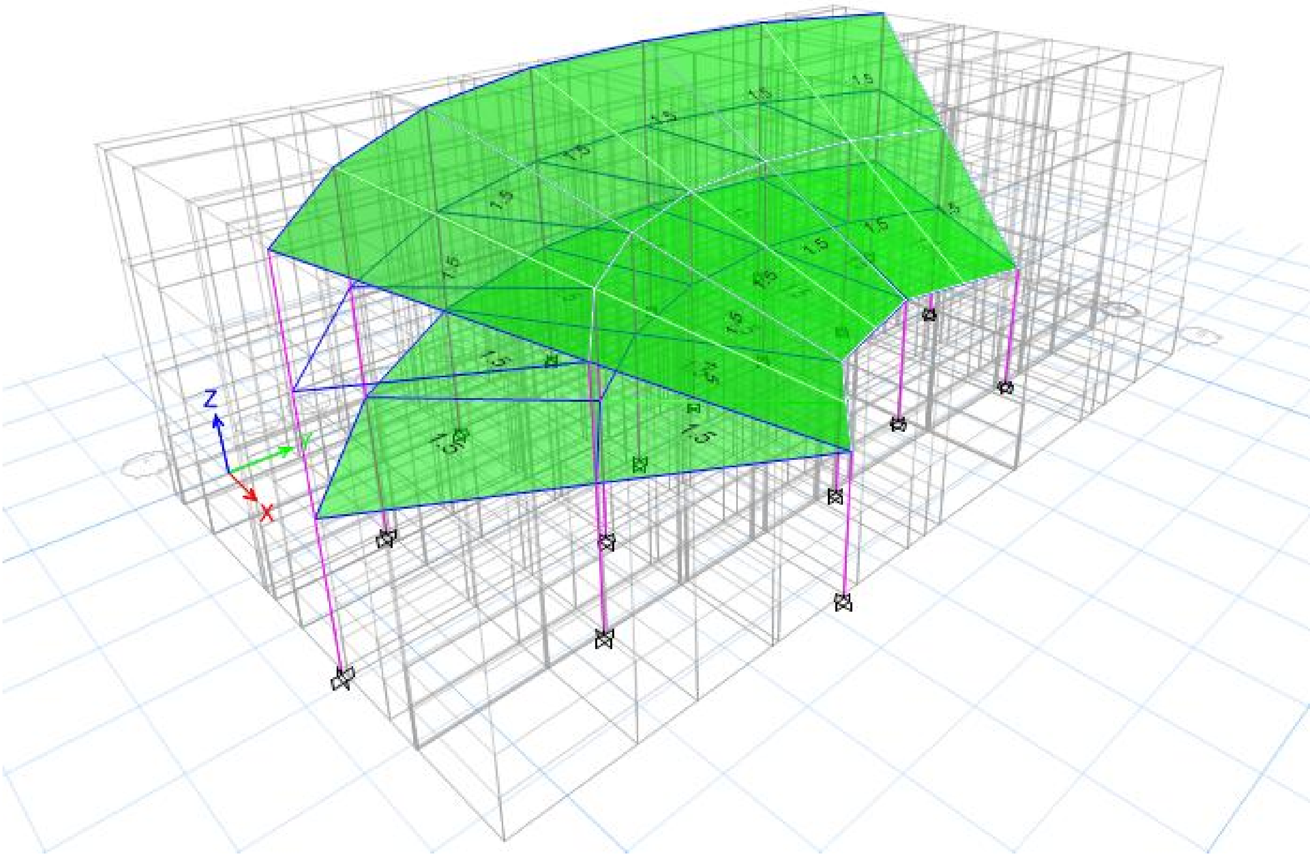


Figure 9: Floor Finish load (Block C)

9.1.2 Live Load

Application of live load is shown in figure below.

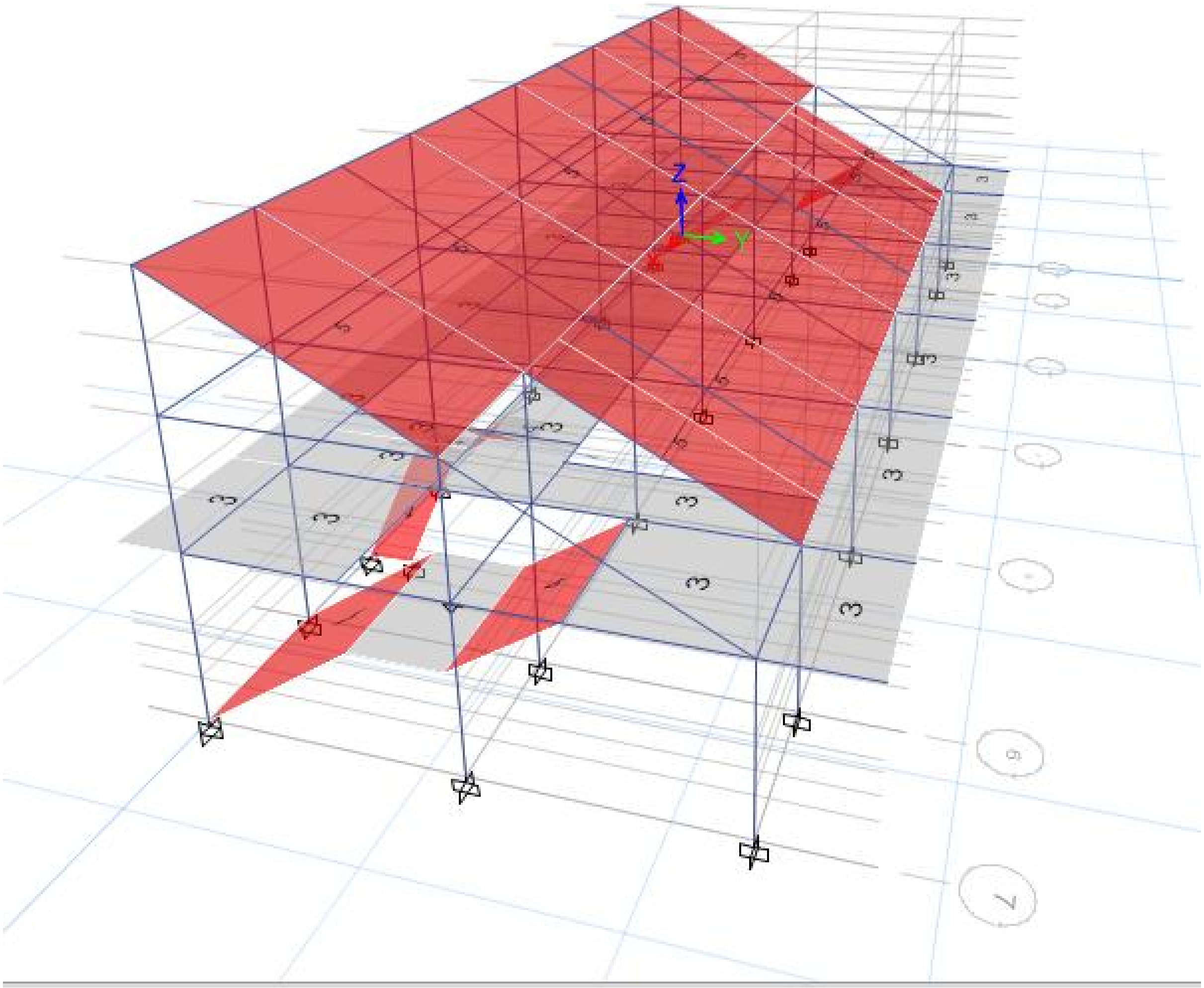


Figure 10: Sample Live Load < 3 KN/m2 (Block A 1 & 3)

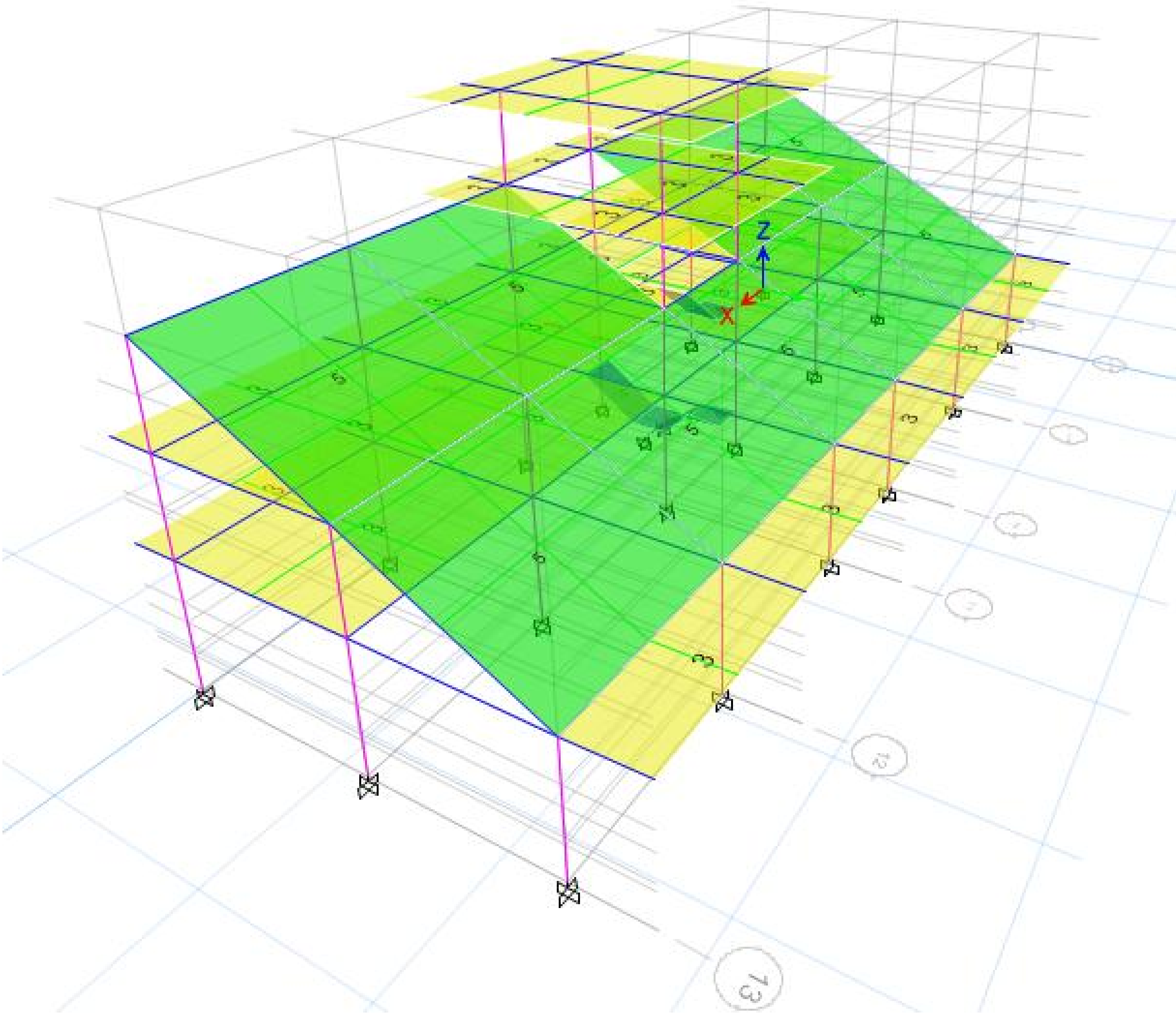


Figure 11: Sample Live Load < 3 KN/m2 (Block A 2)

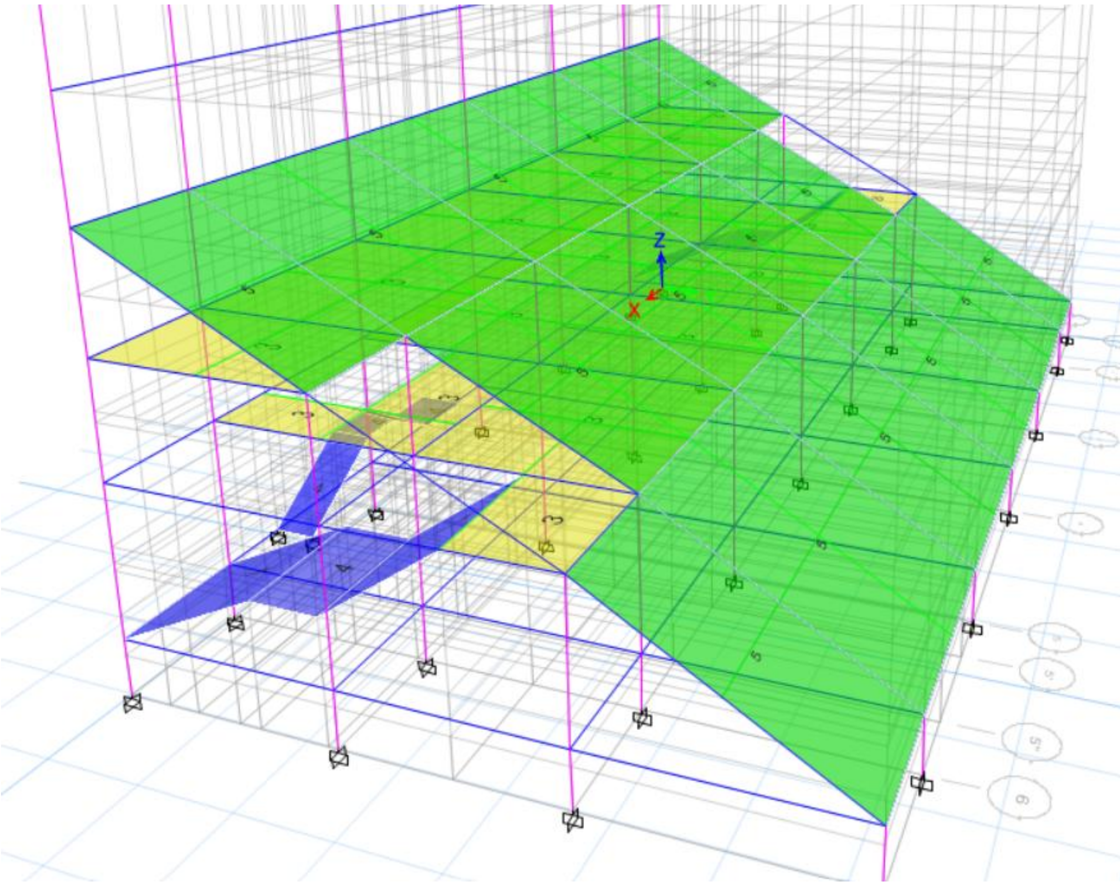


Figure 12: Sample Live Load < 3 KN/m2 (Block B 1 & 3)

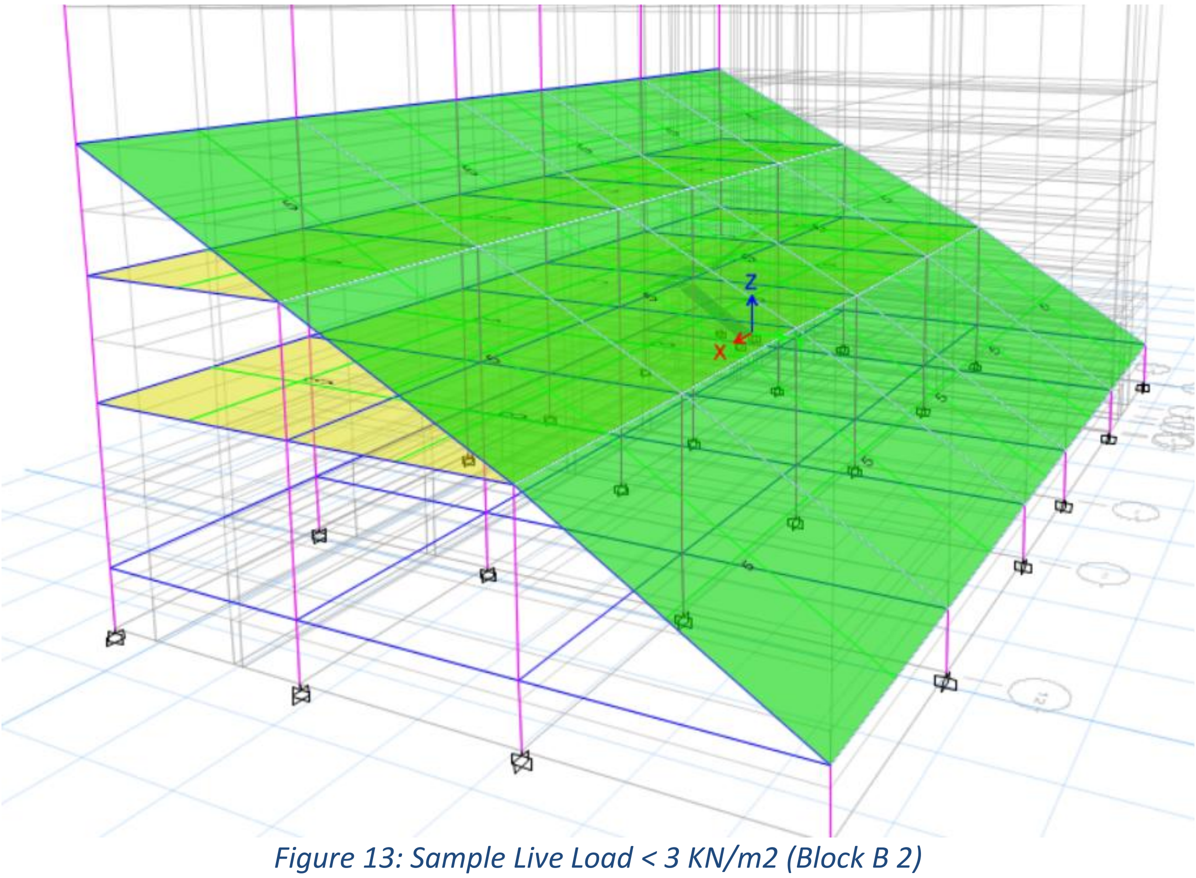
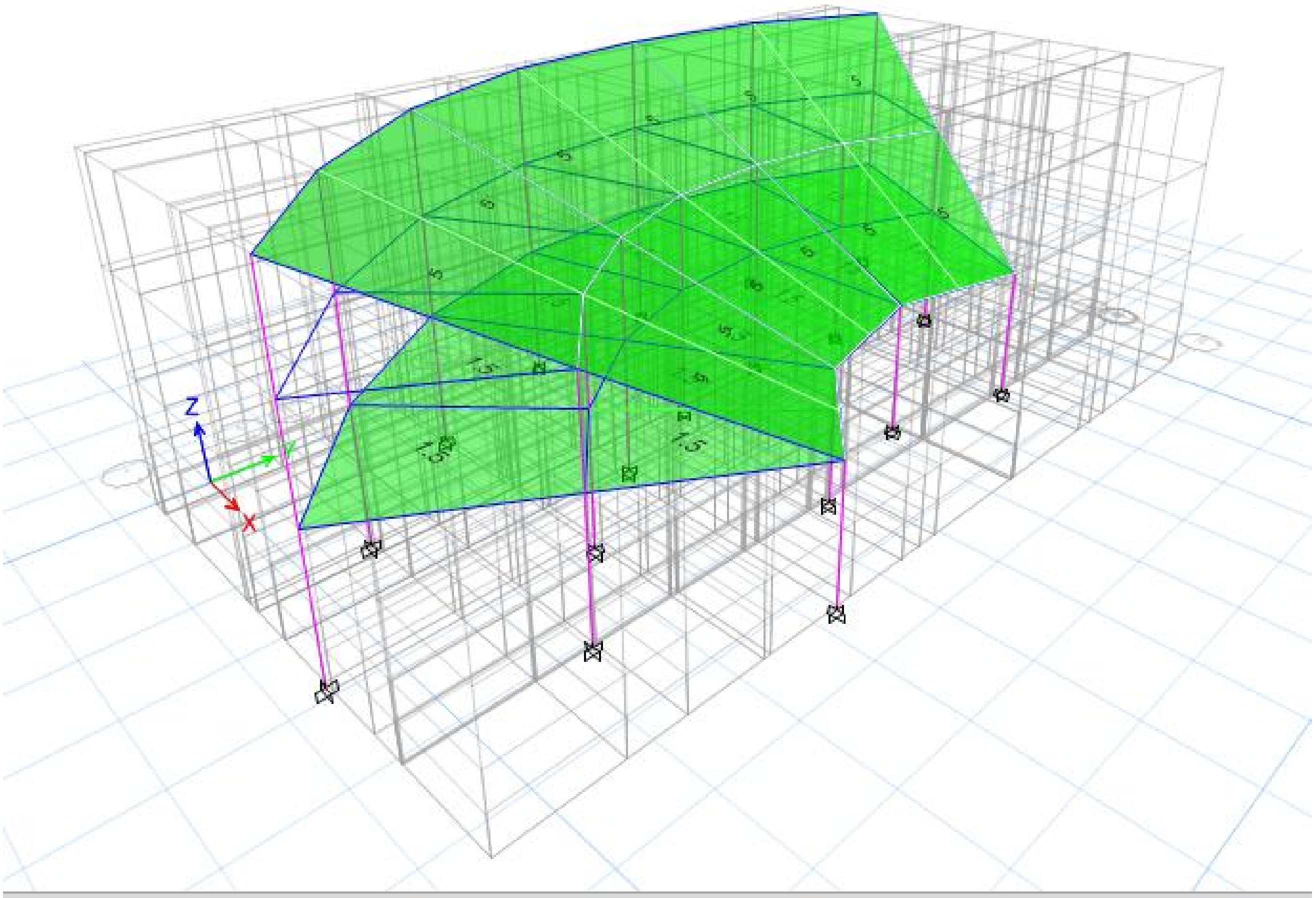


Figure 13: Sample Live Load < 3 KN/m2 (Block B 2)



Please Refer Model Provided along with the Report for Detail

9.1.3 Wall load

Load coming from the weight of wall is applied on the beam underneath the wall. If there is not any beam below the wall, load is applied to nearby beam in the direction of wall. Application of wall load is shown in figure below. Detail Calculation of the wall load is presented in Annex.

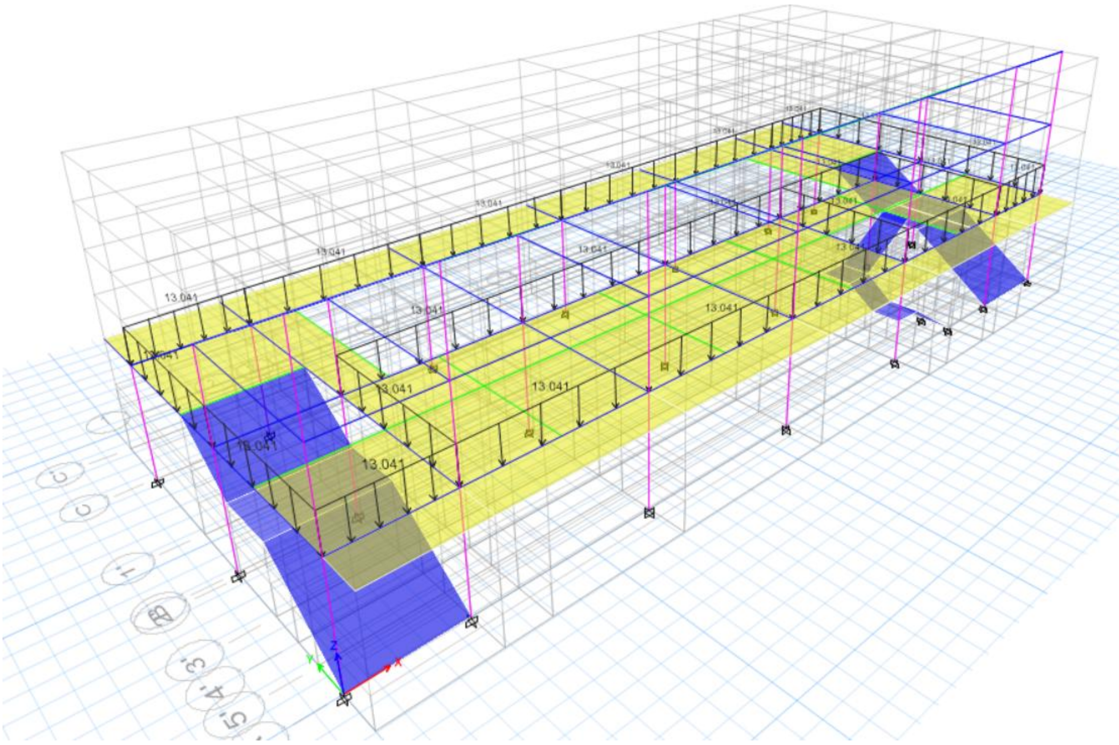


Figure 14: Sample Wall Load (Block A 1 & 3)

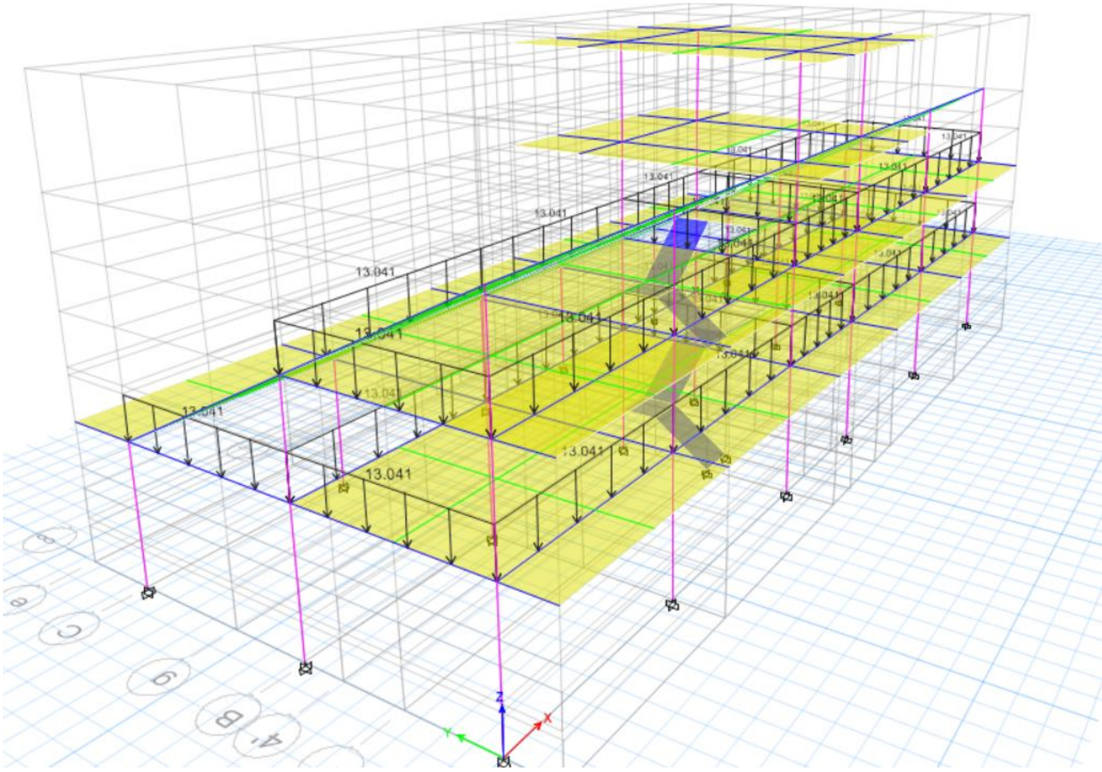


Figure 15: Sample Wall Load (Block A 2)

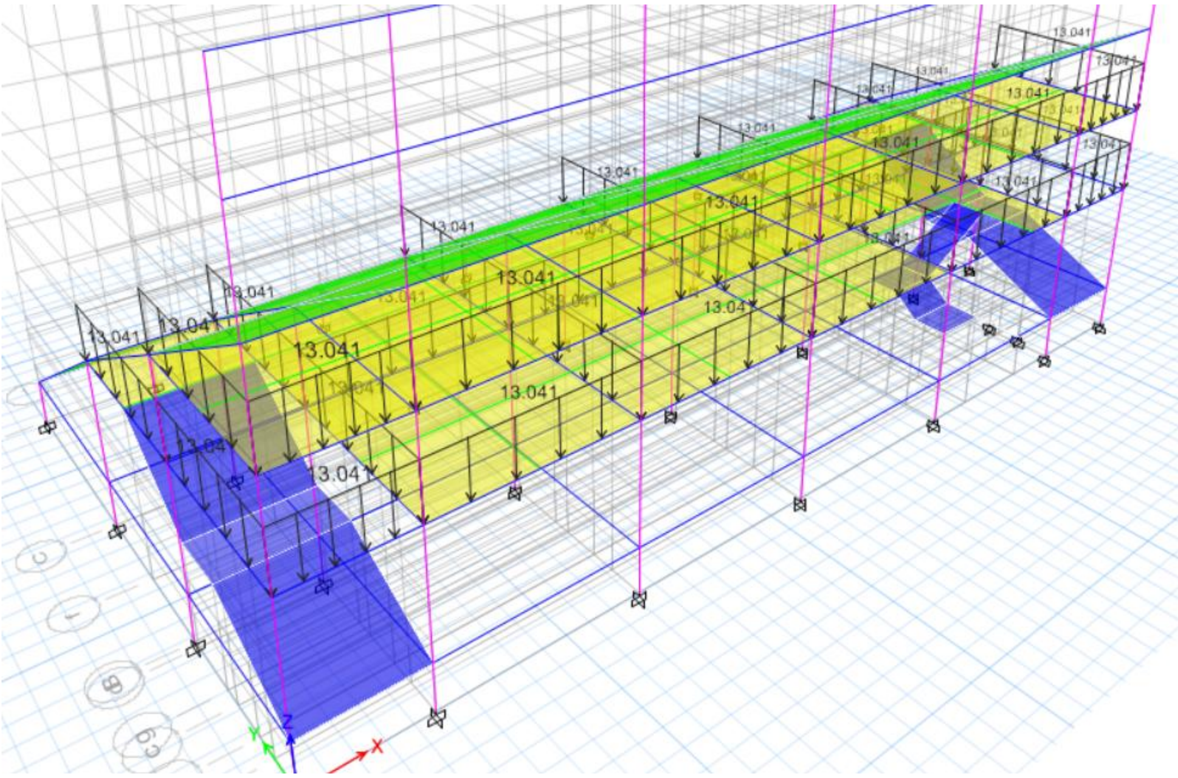


Figure 16: Sample Wall Load (Block B 1 & 3)

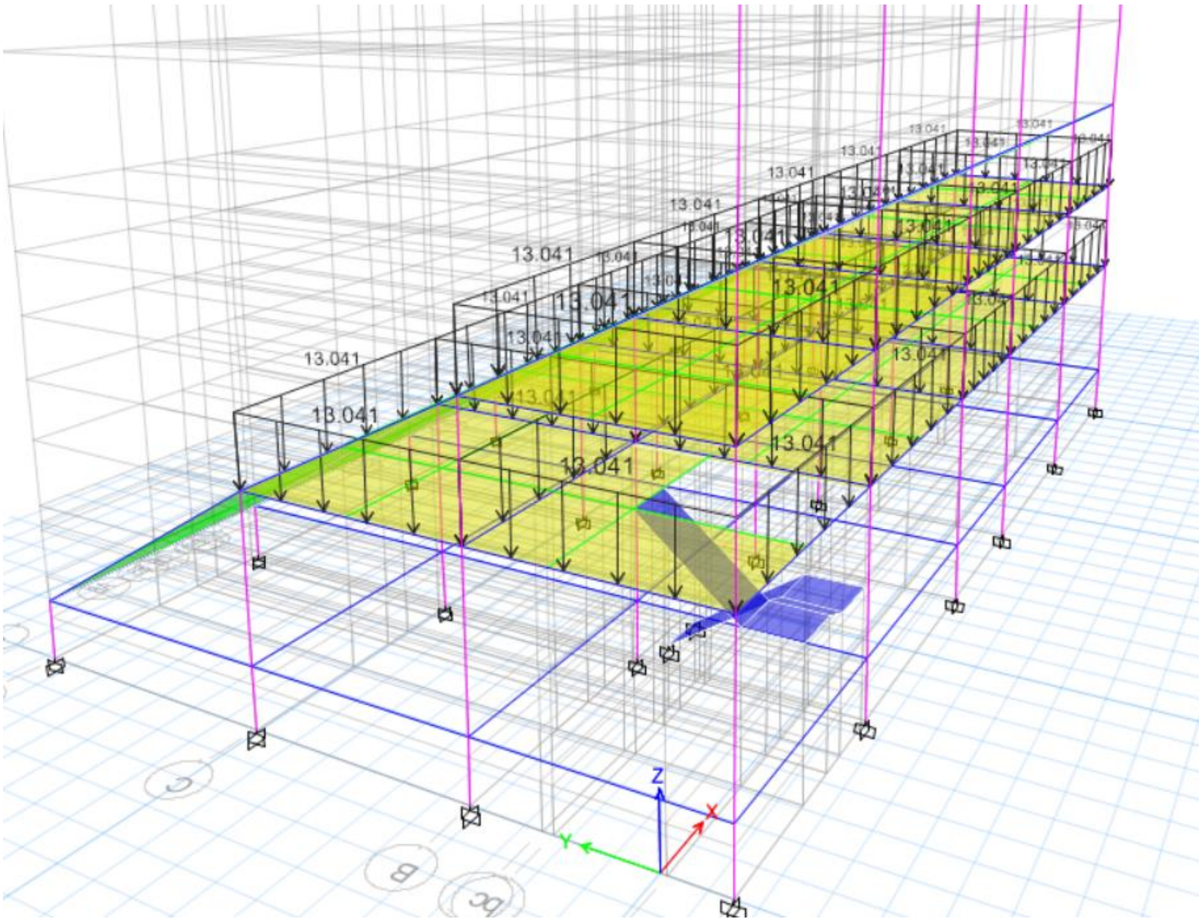


Figure 17: Sample Wall Load (Block B 2)

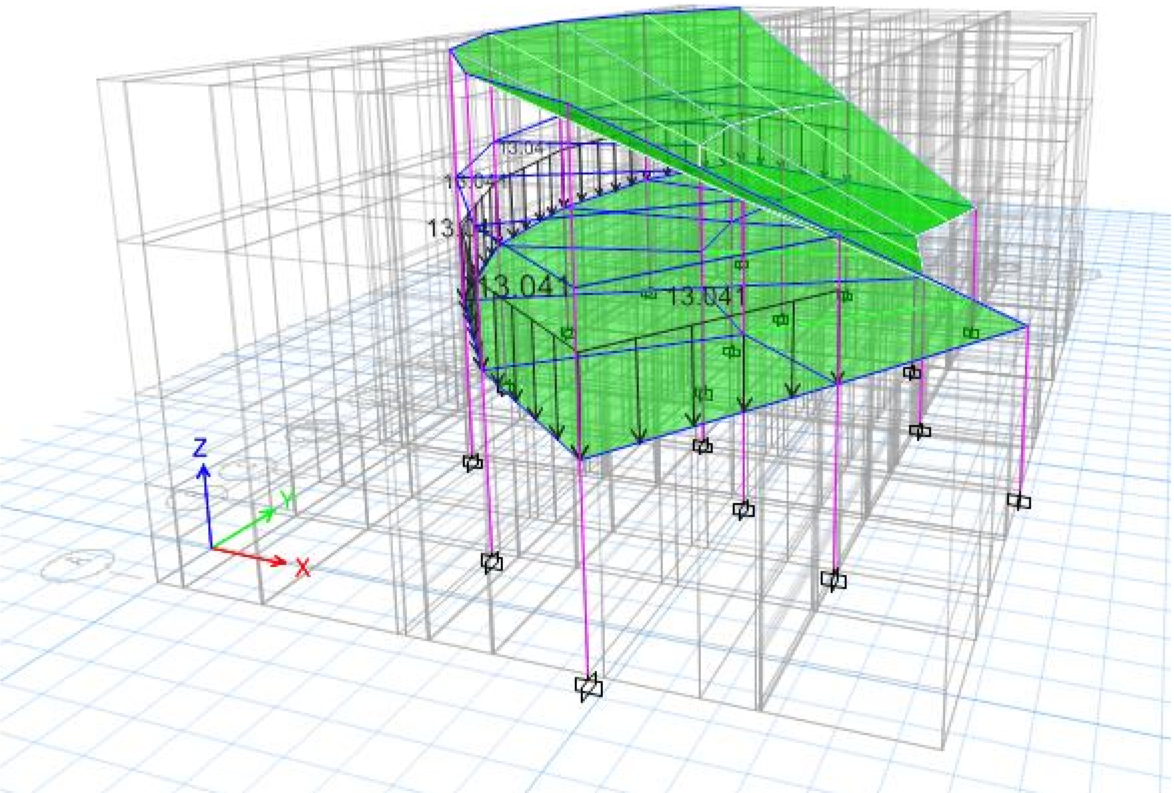


Figure 17: Sample Wall Load (Block C)

Please Refer Model Provided along with the Report for Detail

9.2 LATERAL LOAD ESTIMATION ACCORDING TO NBC 105-2020

Lateral loads on the building frames are caused primarily by wind pressure. In addition; earthquake shocks produce horizontal sway, which results in inertia forces acting horizontally on the structure. But in an area wind load is not so vital so, only the lateral load due to earthquake shock is considered in this case.

9.2.1 Block A 1 & 3

For the analysis and design of earthquake action following method has been applied in this building.

(a) The seismic co-efficient method

Following assumptions have been made to estimate the total base shear in the buildings:

Seismic Zone Factor (Z)	=	0.35
Importance Factor (I)	=	1.25
Height of Building (h)	=	10
Soil Type	=	D

Period Of Vibration:

For Reinforced Moment resisting Frame Kt	=	0.075
T1 = 1.25*Kt*h^0.75	=	0.52719
Lower Part of the Flat Part Of Spectrum, Ta	=	0.5
Upper Part of the Flat Part Of Spectrum, Tc	=	2.0
Peak Spectral Acceleration Normalised by PGA	=	2.25
K	=	0.8

Calculation For Specral Shape Factor: Ch(T)

Since Ta<=T1<=Tc

Ch(T)	=	2.25

Elastic Site Spectral for the horizontal loading (4.1.1 105:2020)

C(T) = Ch(T)ZI	=	0.984375

Elastic Site Spectral for SLS State (4.2 105:2020)

Cs(T1) = 0.2*C(T)	=	0.196875

Horizontal Base Shear Coefficient for Equivalent Static Method:

5.2 NBC 105:2020

Ductility Factor for ULS State (Ru)	=	4
Overstrength Factor for ULS State (Ωu)	=	1.5
Overstrength Factor for SLS State (Ωs)	=	1.25
Horizontal Base Shear Coefficient at the ULS State		
Cd(T1) = C(T)/(Ru*Ωu)	=	0.164063
Horizontal Base Shear Coefficient at the SLS State		
Cd(T1) = Cs(T1)/Ωs	=	0.1575
Exponent Related to the structural period (k)	=	1.01359
So, K	=	1.01359

9.2.2 Block A 2

For the analysis and design of earthquake action following method has been applied in this building.

(a) The seismic co-efficient method

Following assumptions have been made to estimate the total base shear in the buildings:

Seismic Zone Factor (Z)	=	0.35
Importance Factor (I)	=	1.25
Height of Building (h)	=	13
Soil Type	=	D

Period Of Vibration:

For Reinforced Moment resisting Frame Kt	=	0.075
T1 = 1.25*Kt*h^0.75	=	0.64184
Lower Part of the Flat Part Of Spectrum, Ta	=	0.5
Upper Part of the Flat Part Of Spectrum, Tc	=	2.0
Peak Spectral Acceleration Normalised by PGA	=	2.25
K	=	0.8

Calculation For Specral Shape Factor: Ch(T)

Since Ta<=T1<=Tc

Ch(T) = 2.25

Elastic Site Spectral for the horizontal loading (4.1.1 105:2020)

C(T) = Ch(T)ZI = 0.984375

Elastic Site Spectral for SLS State (4.2 105:2020)

Cs(T1) = 0.2*C(T) = 0.196875

Horizontal Base Shear Coefficient for Equivalent Static Method:

5.2 NBC 105:2020

Ductility Factor for ULS State (Ru) = 4

Overstrength Factor for ULS State (Ωu) = 1.5

Overstrength Factor for SLS State (Ωs) = 1.25

Horizontal Base Shear Coefficient at the ULS State

Cd(T1) = C(T)/(Ru*Ωu) = 0.164063

Horizontal Base Shear Coefficient at the SLS State

Cd(T1) = Cs(T1)/Ωs = 0.1575

Exponent Related to the structural period (k) = 1.070921

So, K = 1.070921

9.2.3 Block B 1 2 & 3

For the analysis and design of earthquake action following method has been applied in this building.

(a) The seismic co-efficient method

Following assumptions have been made to estimate the total base shear in the buildings:

Seismic Zone Factor (Z) = 0.3

Importance Factor (I) = 1.5

Height of Building (h) = 17.266

Soil Type = D

Period Of Vibration:

For Reinforced Moment resisting Frame Kt = 0.075

0.79408

T1 = 1.25*Kt*h^0.75 = 2

Lower Part of the Flat Part Of Spectrum, Ta = 0.5

Upper Part of the Flat Part Of Spectrum, Tc = 2.0

Peak Spectral Acceleration Normalised by PGA = 2.25

K = 0.8

Calculation For Specral Shape Factor: Ch(T)

Since Ta<=T1<=Tc

	Ch(T)	=	2.25
Elastic Site Spectral for the horizontal loading (4.1.1 105:2020)			
	C(T) = Ch(T)ZI	=	0.98437 5
Elastic Site Spectral for SLS State (4.2 105:2020)			
	Cs(T1) = 0.2*C(T)	=	0.19687 5
Horizontal Base Shear Coefficient for Equivalent Static Method:			
5.2 NBC 105:2020			
Ductility Factor for ULS State (Ru)		=	4
Overstrength Factor for ULS State (Ωu)		=	1.5
Overstrength Factor for SLS State (Ωs)		=	1.25
Horizontal Base Shear Coefficient at the ULS State			
	Cd(T1) = C(T)/(Ru*Ωu)	=	0.16406 3
Horizontal Base Shear Coefficient at the SLS State			
	Cd(T1) = Cs(T1)/Ωs	=	0.1575
Exponent Related to the structural period (k)		=	1.14704 1
	So, K	=	1.14704 1

9.2.4 Block C

For the analysis and design of earthquake action following method has been applied in this building.

(a) The seismic co-efficient method

Following assumptions have been made to estimate the total base shear in the buildings:

Seismic Zone Factor (Z)	=	0.35
Importance Factor (I)	=	1.25
Height of Building (h)	=	10
Soil Type	=	D
Period Of Vibration:		
For Reinforced Moment resisting Frame Kt	=	0.075
T1 = 1.25*Kt*h^0.75	=	0.52719
Lower Part of the Flat Part Of Spectrum, Ta	=	0.5
Upper Part of the Flat Part Of Spectrum, Tc	=	2.0
Peak Spectral Acceleration Normalised by PGA	=	2.25
K	=	0.8
Calculation For Specral Shape Factor: Ch(T)		
Since Ta<=T1<=Tc		
Ch(T)	=	2.25
Elastic Site Spectral for the horizontal loading (4.1.1 105:2020)		
C(T) = Ch(T)ZI	=	0.98437 5
Elastic Site Spectral for SLS State (4.2 105:2020)		
Cs(T1) = 0.2*C(T)	=	0.19687 5
Horizontal Base Shear Coefficient for Equivalent Static Method:		
5.2 NBC 105:2020		
Ductility Factor for ULS State (Ru)	=	4
Overstrength Factor for ULS State (Ωu)	=	1.5
Overstrength Factor for SLS State (Ωs)	=	1.25
Horizontal Base Shear Coefficient at the ULS State		
Cd(T1) = C(T)/(Ru*Ωu)	=	0.16406 3
Horizontal Base Shear Coefficient at the SLS State		
Cd(T1) = Cs(T1)/Ωs	=	0.1575
Exponent Related to the structural period (k)	=	1.01359
So, K	=	1.01359

9.3 LOAD CASES AND COMBINATION

9.3.1 Load Cases

Load cases are the independent loadings for which the structure is explicitly analyzed. Earthquake forces occur in random fashion in all directions. For buildings whose lateral load resisting elements are oriented in two principal directions, it is usually sufficient to analyze in these two principal directions (X – and Y – direction) separately one at a time. Thus, the load cases adopted are as follows:

i. Dead Load (DL)
ii. Live Load (LL)
iii. EQX
iv. EQY

9.3.2 Load Combinations

Load combinations are the loadings formed by the linear combination of the independent loading conditions. The different load cases have been combined as per NBC Code .The load combinations are as follows:

i. 1.2 DL + 1.5 LL
ii. (DL+ λLL+- E)

Where, λ = 0.6 for storage facilities

= 0.3 for other usage

DL= Dead Load

LL= Live load

E= Earthquake Load

9.4 Base Reaction

TABLE: Base Reactions (Block A1 & 3)					
Output Case	**Case Type**	**Step Type**	**Global FX**	**Global FY**	**Global FZ**
Text	Text	Text	KN	KN	KN
EQ_X ULS	LinStatic		-2017.8391	0	0
EQ_Y ULS	LinStatic			-2017.8391	0
EQ_X SLS	LinStatic		-1937.1256	0	0
EQ_Y SLS	LinStatic			-1937.1256	0

RSX ULS	LinStatic		2017.8391	0	0
RSX SLS	LinStatic		1937.1256	0	0
RSY ULS	LinStatic			2017.8391	0
RSY SLS	LinStatic			1937.1256	0

TABLE: Base Reactions (Block A2)					
Output Case	**Case Type**	**Step Type**	**Global FX**	**Global FY**	**Global FZ**
Text	Text	Text	KN	KN	KN
EQ_X ULS	LinStatic		-2434.2377	0	0
EQ_Y ULS	LinStatic		0	2434.2377	0
EQ_X SLS	LinStatic		-2336.8681	0	0
EQ_Y SLS	LinStatic		0	-2336.8681	0
RSX ULS	LinStatic		2434.2377	0	0
RSX SLS	LinStatic		2336.8681	0	0
RSY ULS	LinStatic		0	2434.2377	0
RSY SLS	LinStatic		0	2336.8681	0

TABLE: Base Reactions (Block B 1 & 3)					
Output Case	**Case Type**	**Step Type**	**Global FX**	**Global FY**	**Global FZ**
Text	Text	Text	KN	KN	KN
EQ_X ULS	LinStatic		-3428.7786	0	0
EQ_Y ULS	LinStatic		0	-3428.7786	0
EQ_X SLS	LinStatic		-3291.6275	0	0
EQ_Y SLS	LinStatic		0	-3291.6275	0
RSX ULS	LinStatic		3428.7786	0	0
RSX SLS	LinStatic		3291.6275	0	0
RSY ULS	LinStatic		0	3428.7786	0
RSY SLS	LinStatic		0	3291.6275	0

TABLE: Base Reactions (Block B 2)					
Output Case	**Case Type**	**Step Type**	**Global FX**	**Global FY**	**Global FZ**
Text	Text	Text	KN	KN	KN
EQ_X ULS	LinStatic		-3171.6626	0	0
EQ_Y ULS	LinStatic		0	-3171.6626	0
EQ_X SLS	LinStatic		-3044.7961	0	0
EQ_Y SLS	LinStatic		0	-3044.7961	0
RSX ULS	LinStatic		33171.6626	0	0
RSX SLS	LinStatic		3044.7961	0	0
RSY ULS	LinStatic		0	33171.6626	0
RSY SLS	LinStatic		0	3044.7961	0

TABLE: Base Reactions (Block C)					
Output Case	Case Type	Step Type	Global FX	Global FY	Global FZ
Text	Text	Text	KN	KN	KN
EQ_X ULS	LinStatic		-1102.2307	0	0
EQ_Y ULS	LinStatic		0	-1102.2307	0
EQ_X SLS	LinStatic		-1058.1418	0	0
EQ_Y SLS	LinStatic		0	-1058.1418	0
RSX ULS	LinStatic		1102.2307	0	0
RSX SLS	LinStatic		1058.1418	0	0
RSY ULS	LinStatic		0	1102.2307	0
RSY SLS	LinStatic		0	1058.1418	0

9.5 MODAL RESULT

Free vibration analysis was performed to determine the natural periods and mode shapes of the buildings. The number of modes, corresponding natural periods and mass participation ration of the building is tabulated in Tables below.

Table 1: Mode numbers, natural periods and mass participation (Block A)

TABLE: Modal Participating Mass Ratios (Block A 1,3)								
Case	Mode	Period	UX	UY	SumUX	SumUY	RZ	SumRZ
		sec						
Modal	1	0.327	0.005	0.9853	0.005	0.9853	0.0046	0.0046
Modal	2	0.301	0.9339	0.0071	0.9389	0.9924	0.0431	0.0476
Modal	3	0.269	0.0473	0.0024	0.9862	0.9948	0.945	0.9927
Modal	4	0.049	0.0005	0.00004547	0.9867	0.9949	0.0001	0.9928
Modal	5	0.046	0.0007	0	0.9874	0.9949	0.0017	0.9945
Modal	6	0.036	8.593E-07	0.0013	0.9874	0.9961	0.000002688	0.9945
Modal	7	0.033	0.0002	0.00001264	0.9876	0.9962	0.0019	0.9964
Modal	8	0.03	0.00001241	0.0031	0.9876	0.9992	0.0001	0.9965
Modal	9	0.03	0.0000239	0.00004679	0.9876	0.9993	0.0004	0.9969
Modal	10	0.029	0.0001	0.00001051	0.9877	0.9993	0.0023	0.9992
Modal	11	0.024	0	0.000002518	0.9877	0.9993	0	0.9992
Modal	12	0.024	0.0004	0	0.9881	0.9993	0.000009753	0.9992

TABLE: Modal Participating Mass Ratios (Block A2)								
Case	Mode	Period	UX	UY	SumUX	SumUY	RZ	SumRZ
		sec						
Modal	1	0.466	0.0013	0.993	0.0013	0.993	0.0012	0.0012
Modal	2	0.459	0.579	0.0025	0.5804	0.9955	0.4178	0.419
Modal	3	0.383	0.4143	0.00001027	0.9946	0.9955	0.5801	0.9992
Modal	4	0.198	0	0.0029	0.9946	0.9984	0	0.9992
Modal	5	0.189	0.0025	0	0.9971	0.9984	0.0004	0.9996
Modal	6	0.158	0.0007	0	0.9978	0.9984	0.0001	0.9997

Modal	7	0.046	0.0004	8.456E-07	0.9982	0.9984	0.0001	0.9998
Modal	8	0.037	7.919E-07	0.0002	0.9982	0.9986	0	0.9998
Modal	9	0.035	0.00001929	0	0.9982	0.9986	0.0001	0.9999
Modal	10	0.031	0	0.000009047	0.9982	0.9986	0	0.9999
Modal	11	0.028	0.00001135	0.00000053	0.9982	0.9986	0	0.9999
Modal	12	0.026	0.00002411	0.00001002	0.9982	0.9986	0.000006694	0.9999

Table 2: Mode numbers, natural periods and mass participation (Block B)

TABLE: Modal Participating Mass Ratios (Block B 1,3)								
Case	**Mode**	**Period**	**UX**	**UY**	**SumUX**	**SumUY**	**RZ**	**SumRZ**
		sec						
Modal	1	0.238	0.7732	0	0.7732	0	0.1492	0.1492
Modal	2	0.147	0	0.9637	0.7732	0.9637	0	0.1492
Modal	3	0.116	0.1447	0	0.9178	0.9637	0.793	0.9421
Modal	4	0.049	0.0058	0	0.9236	0.9637	0.002	0.9441
Modal	5	0.039	0.000003294	0.0053	0.9236	0.9689	0.00001008	0.9441
Modal	6	0.038	0.0039	0.000004709	0.9275	0.9689	0.0144	0.9584
Modal	7	0.035	0.0509	0	0.9785	0.9689	0.0006	0.9591
Modal	8	0.034	0.0004	0	0.9789	0.9689	0.0033	0.9623
Modal	9	0.031	0.018	0	0.9969	0.9689	0.006	0.9683
Modal	10	0.03	0	0.004	0.9969	0.9729	0	0.9683
Modal	11	0.029	0.0005	7.372E-07	0.9974	0.9729	0.0027	0.971
Modal	12	0.029	0	0.0018	0.9974	0.9747	0	0.971

TABLE: Modal Participating Mass Ratios (Block B2)								
Case	**Mode**	**Period**	**UX**	**UY**	**SumUX**	**SumUY**	**RZ**	**SumRZ**
		sec						
Modal	1	0.318	0.7732	0.000001478	0.7732	0.000001478	0.1777	0.1777
Modal	2	0.19	0.00001727	0.9761	0.7732	0.9761	0.00003998	0.1778
Modal	3	0.149	0.1748	0.00004069	0.948	0.9761	0.788	0.9657
Modal	4	0.08	0.0009	0.000004703	0.9489	0.9761	0.0002	0.9659
Modal	5	0.05	0.0016	0.000002908	0.9505	0.9761	0.0005	0.9664
Modal	6	0.044	0	0.0011	0.9505	0.9773	0	0.9664
Modal	7	0.04	0.0025	0.000004088	0.953	0.9773	0.0042	0.9706
Modal	8	0.039	0.0001	0.0004	0.9531	0.9776	0.0007	0.9712
Modal	9	0.035	0.0329	0	0.986	0.9776	0.0001	0.9713
Modal	10	0.035	0.0005	0.000001102	0.9865	0.9776	0.0028	0.9741
Modal	11	0.034	0.0009	0.0001	0.9874	0.9778	0.00004369	0.9742
Modal	12	0.032	0.0116	0	0.9989	0.9778	0.0047	0.9789

Table 3: Mode numbers, natural periods and mass participation (Block C)

TABLE: Modal Participating Mass Ratios (Block C)								
Case	**Mode**	**Period**	**UX**	**UY**	**SumUX**	**SumUY**	**RZ**	**SumRZ**
		sec						
Modal	1	0.335	0.01	0.55	0.01	0.55	0.4391	0.4391

Modal	2	0.318	0.9714	0.027	0.9814	0.577	0.0012	0.4403
Modal	3	0.285	0.0181	0.4214	0.9996	0.9984	0.5568	0.9971
Modal	4	0.06	0.0001	0.0016	0.9996	0.9999	0.0028	0.9999
Modal	5	0.033	0.0003	0.00001423	1	0.9999	0	0.9999
Modal	6	0.027	0	0.0001	1	1	0.0001	1
Modal	7	0.022	0.000008488	0.000001545	1	1	0.00001816	1
Modal	8	0.021	0.00001014	0	1	1	0.000005407	1
Modal	9	0.017	0	0.000003785	1	1	0	1
Modal	10	0.007	0	0	1	1	0	1
Modal	11	0.007	0	0	1	1	0	1
Modal	12	0.007	0	0	1	1	0	1

9.6 DRIFT AND DISPLACEMENT OF THE BUILDING

The deformation of the buildings is also determined and found that the drift limit is compliance with the provision of NBC 105-2020. The story drift of the building along x and y-direction is tabulated below.

Displacement and Drift Calculation for Block A 1 & 3

Displacement Criteria For Equivalent Static Method:		
Allowable Ratio ULS	=	0.025
Allowable Displacement ULS = 0.025*H/Ru	=	62.5mm
Allowable Ratio SLS	=	0.006
Allowable Displacement SLS = 0.006*H/Rs	=	60mm
Drift Criteria for Equivalent Static Method:		
Allowable Drift = 0.025/Ru	=	0.00625
Allowable Drift	=	0.006

Displacement and Drift Calculation for Block A 2

Displacement Criteria For Equivalent Static Method:		
Allowable Ratio ULS	=	0.025
Allowable Displacement ULS = 0.025*H/Ru	=	81.25mm
Allowable Ratio SLS	=	0.006
Allowable Displacement SLS = 0.006*H/Rs	=	78mm
Drift Criteria for Equivalent Static Method:		
Allowable Drift = 0.025/Ru	=	0.00625
Allowable Drift	=	0.006

Displacement and Drift Calculation for Block B 1 2 & 3

Displacement Criteria For Equivalent Static Method:		
Allowable Ratio ULS	=	0.025
Allowable Displacement ULS = 0.025*H/Ru	=	107.9125mm
Allowable Ratio SLS	=	0.006
Allowable Displacement SLS = 0.006*H/Rs	=	103.59mm
Drift Criteria for Equivalent Static Method:		
Allowable Drift = 0.025/Ru	=	0.00625
Allowable Drift	=	0.006

Displacement and Drift Calculation for Block A 1 & 3

Displacement Criteria For Equivalent Static Method:		
Allowable Ratio ULS	=	0.025

Allowable Displacement ULS = 0.025*H/Ru	=	62.5mm
Allowable Ratio SLS	=	0.006
Allowable Displacement SLS = 0.006*H/Rs	=	60mm
Drift Criteria for Equivalent Static Method:		
Allowable Drift = 0.025/Ru	=	0.00625
Allowable Drift	=	0.006

9.7 Forces in Beams & Columns

The axial force, bending moment and shear force in all members are shown below:

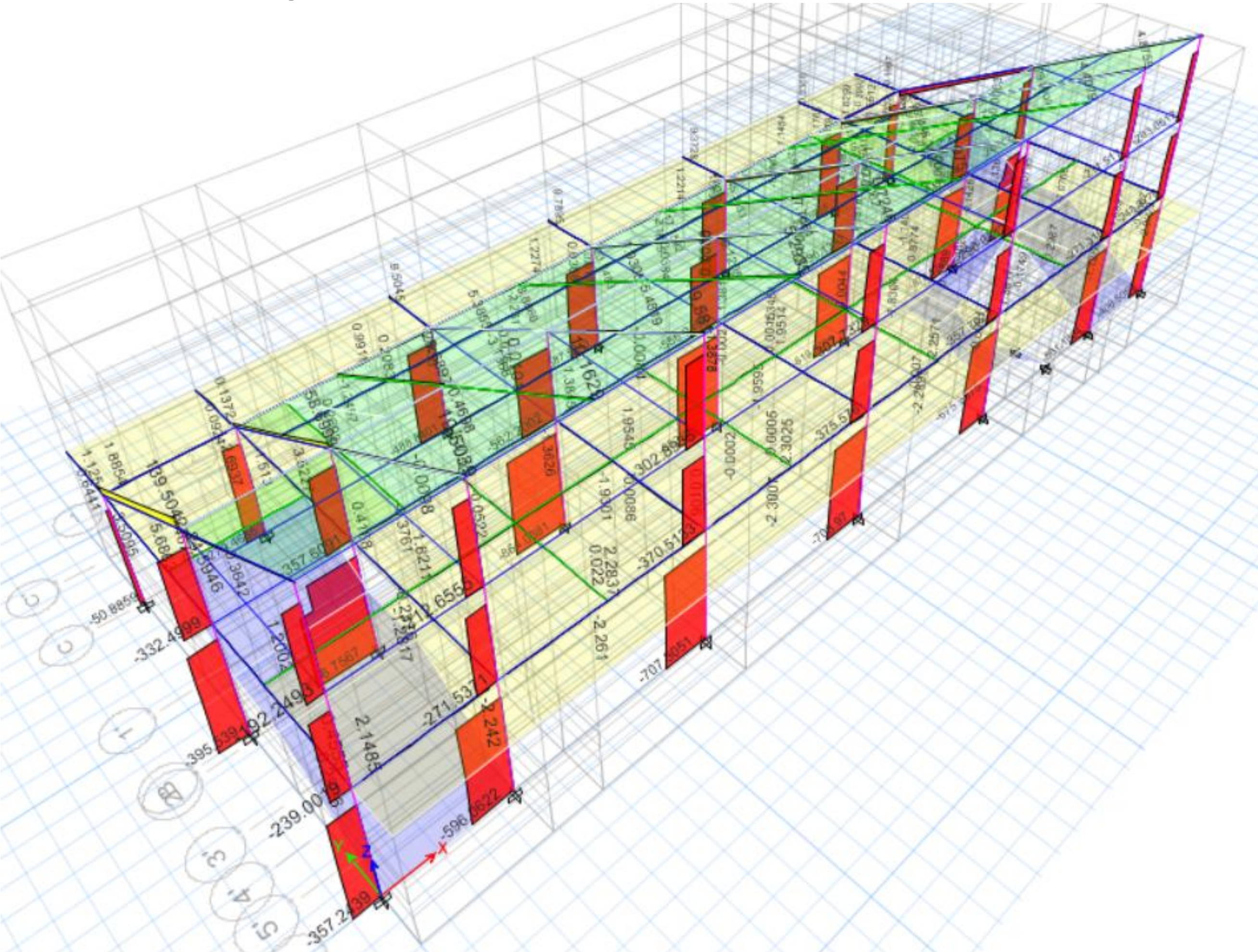


Figure 18: Axial force in Block A 1 & 3

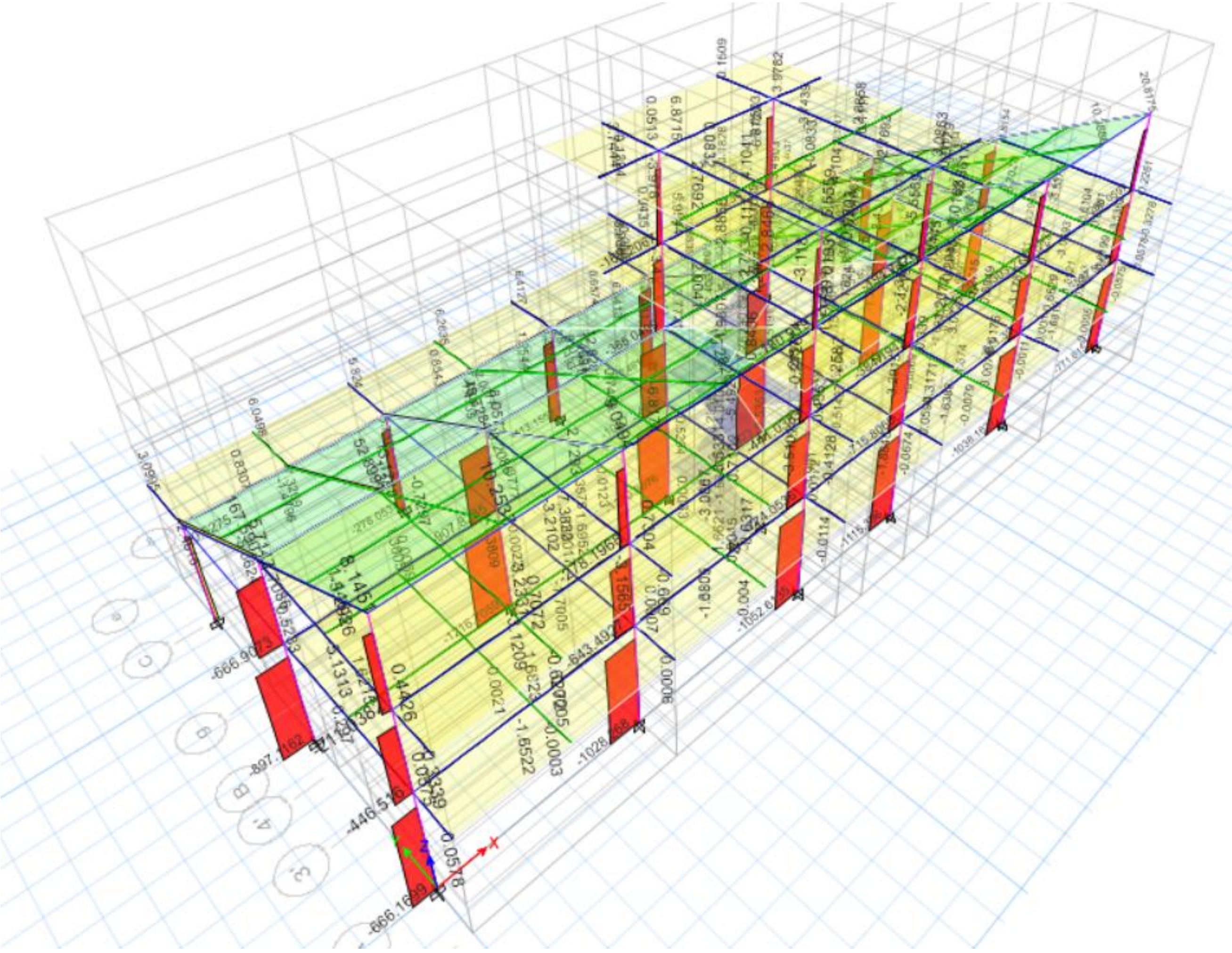


Figure 19: Axial force in Block A 2

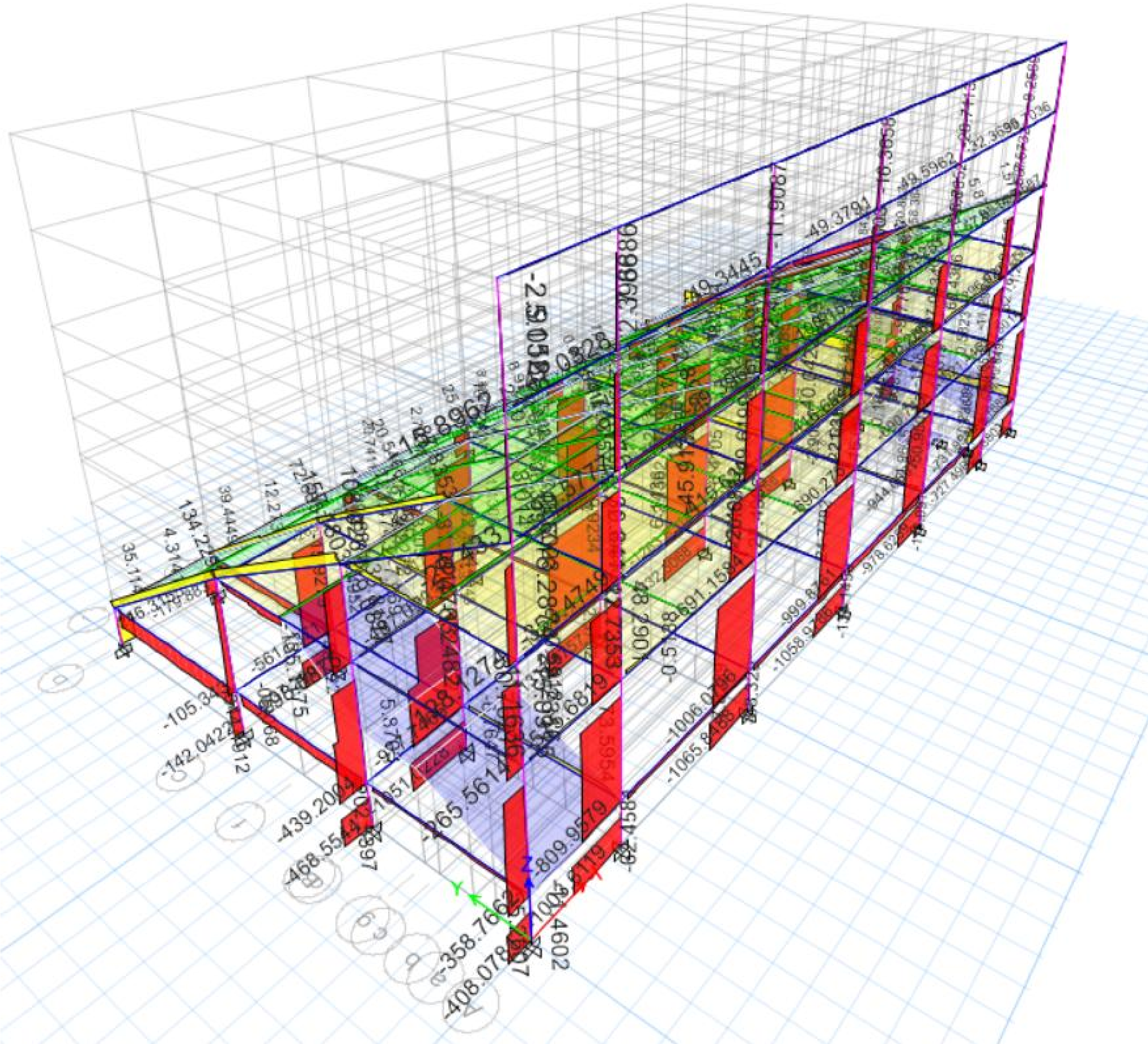


Figure 20: Axial force in Block B 1 & 3

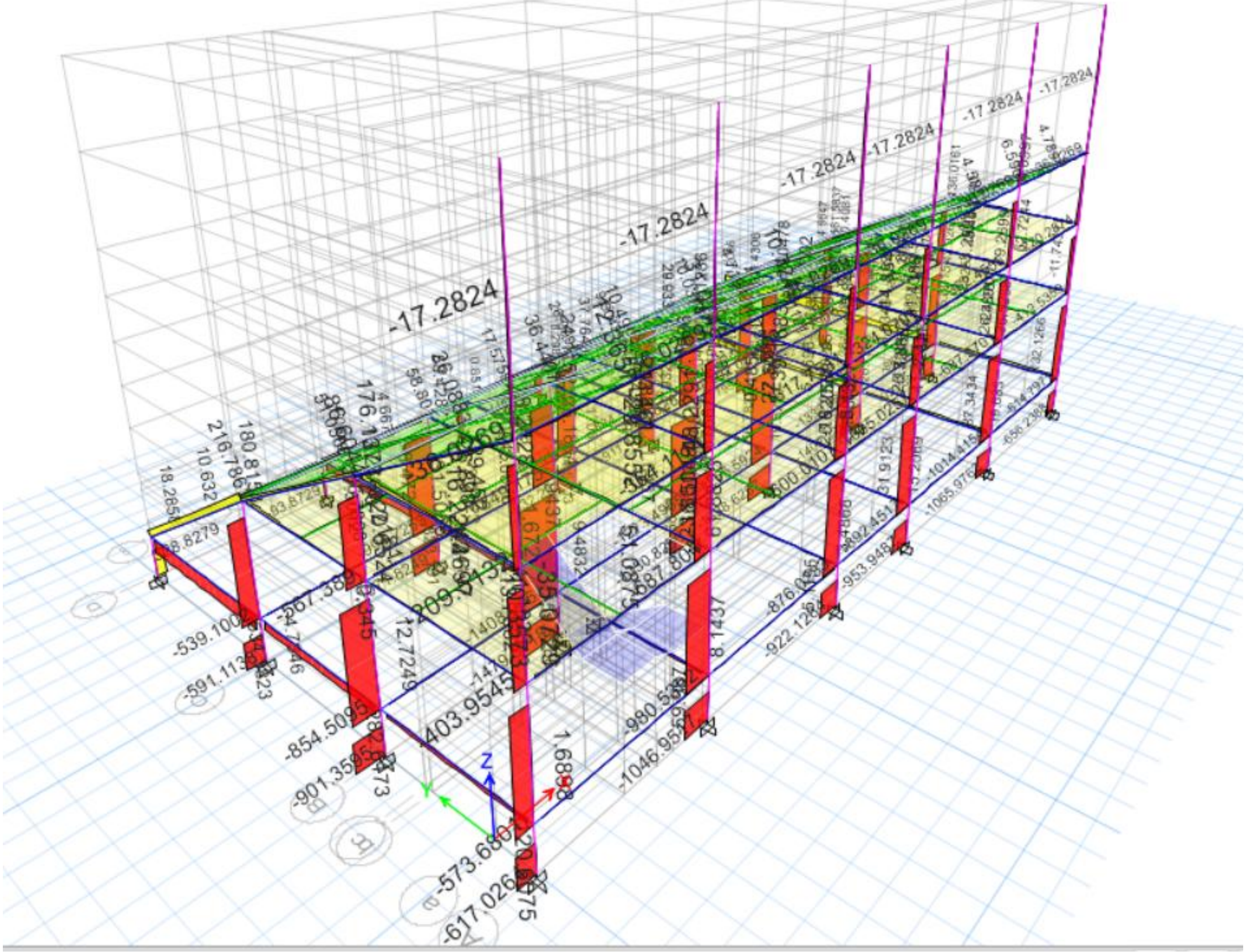


Figure 21: Axial force in Block B 2

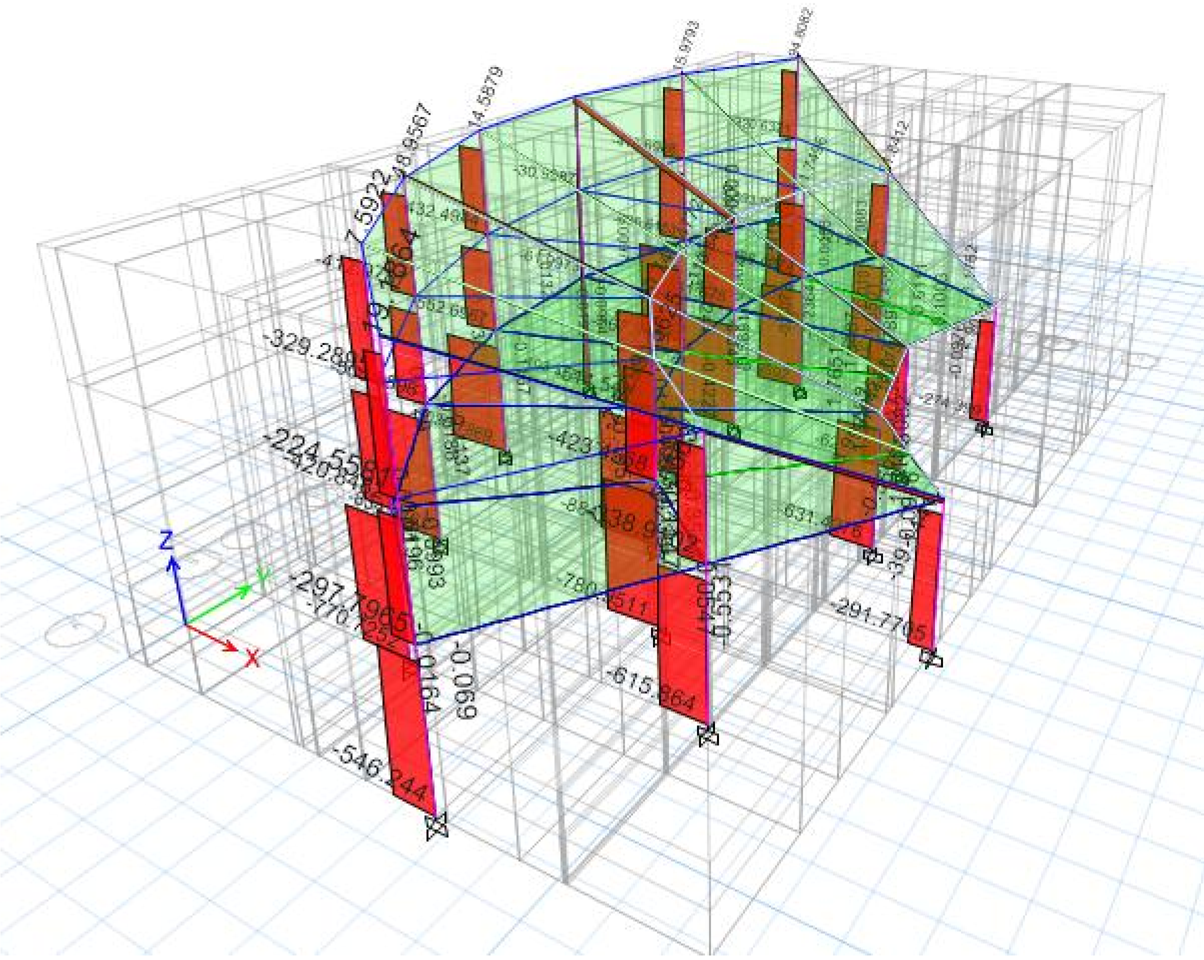


Figure 21: Axial force in Block C

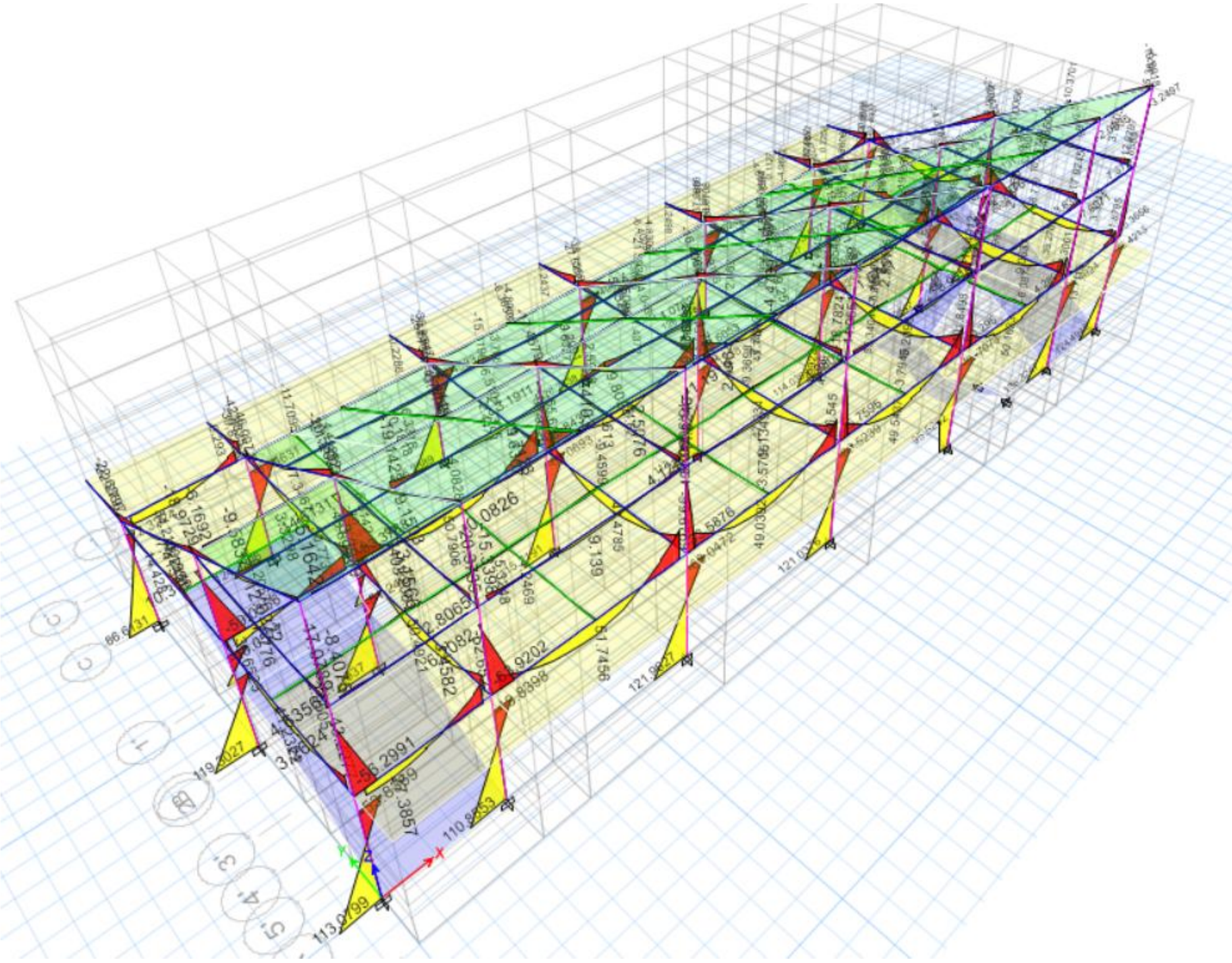


Figure 22: Bending Moment Diagram in Block A 1 & 3

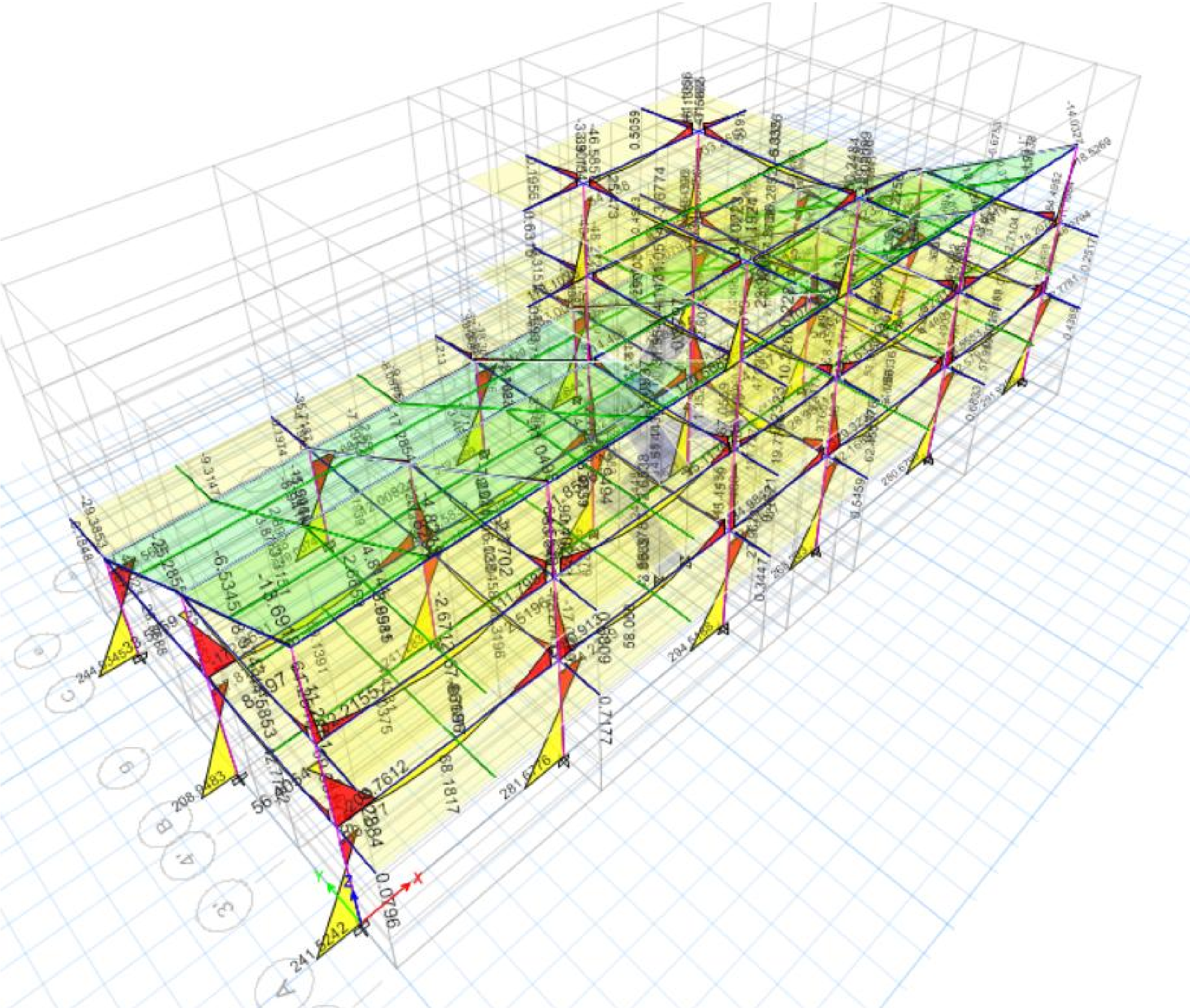


Figure 23: Bending Moment Diagram in Block A 2

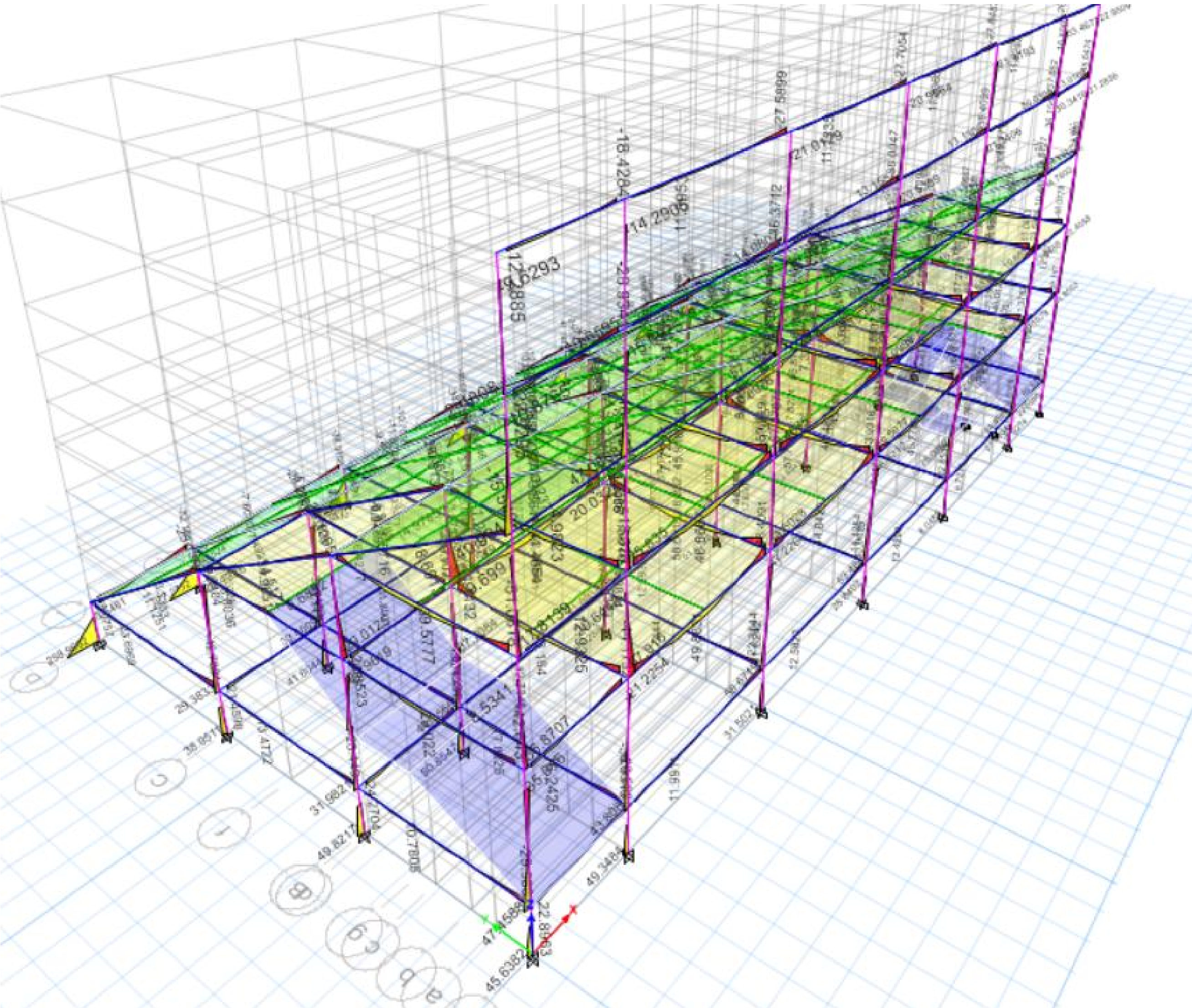


Figure 24: Bending Moment Diagram in Block B 1 & 3

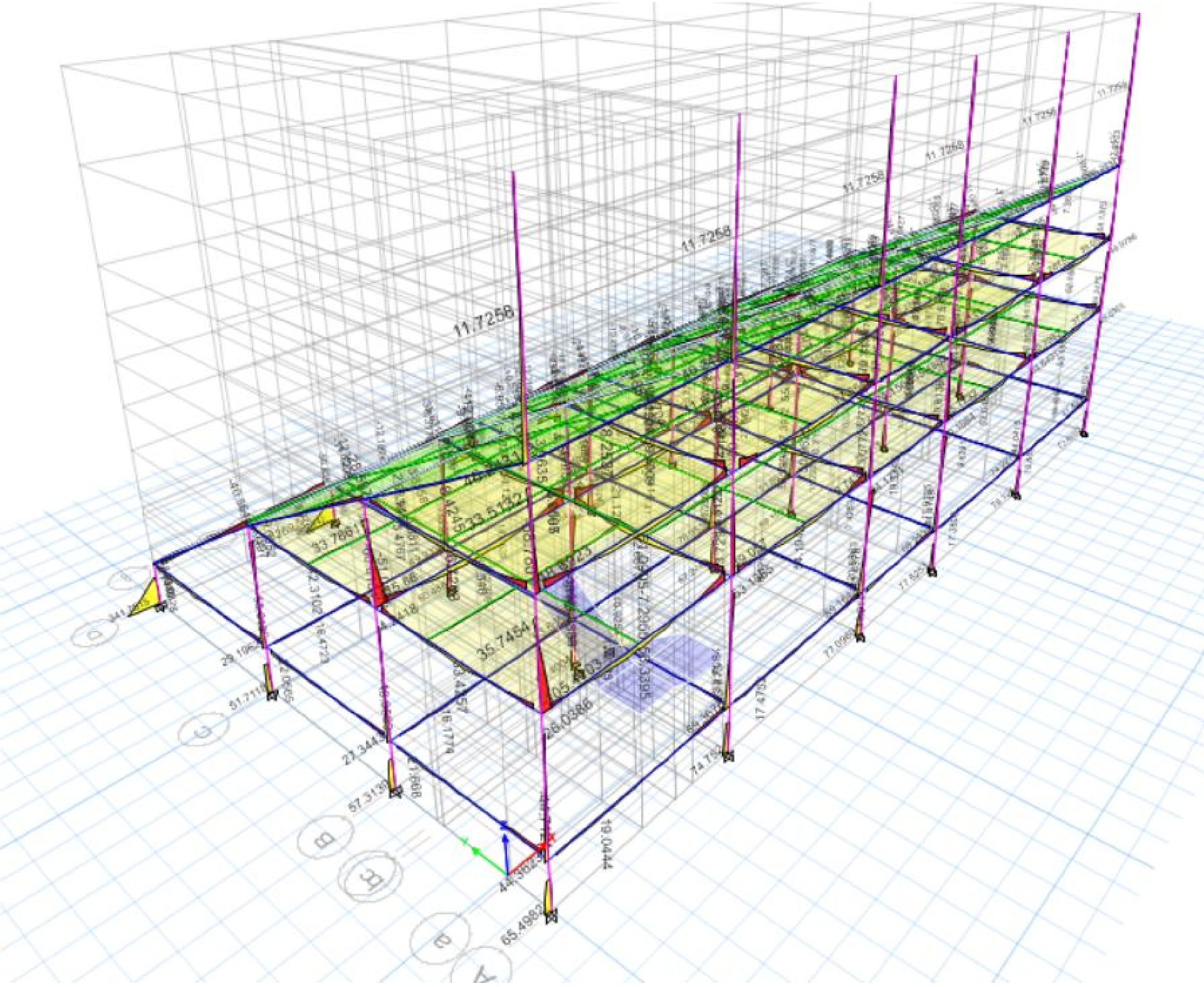


Figure 25: Bending Moment Diagram in Block B 2

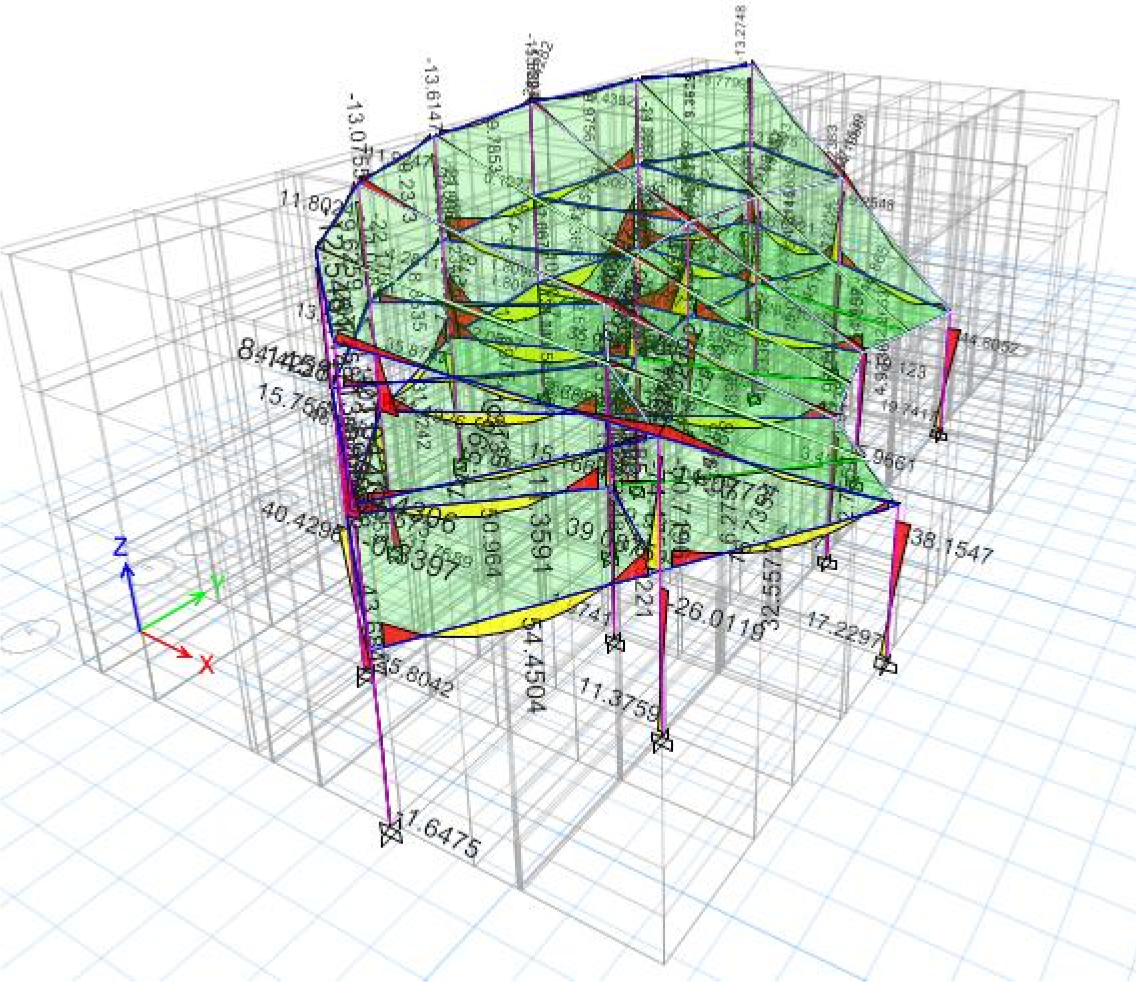


Figure 25: Bending Moment Diagram in Block C

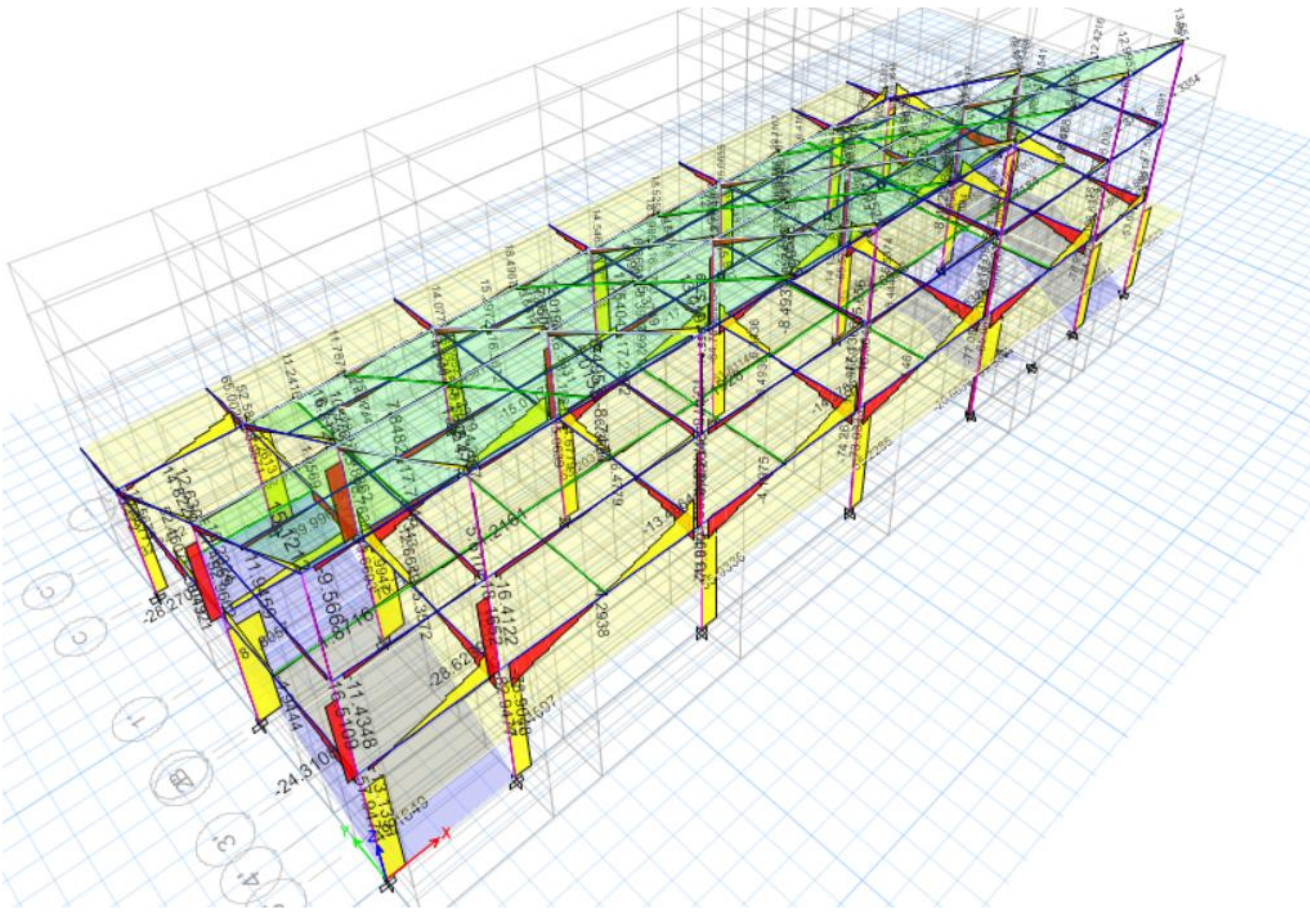


Figure 26: Shear Force Diagram in Block A 1 & 3

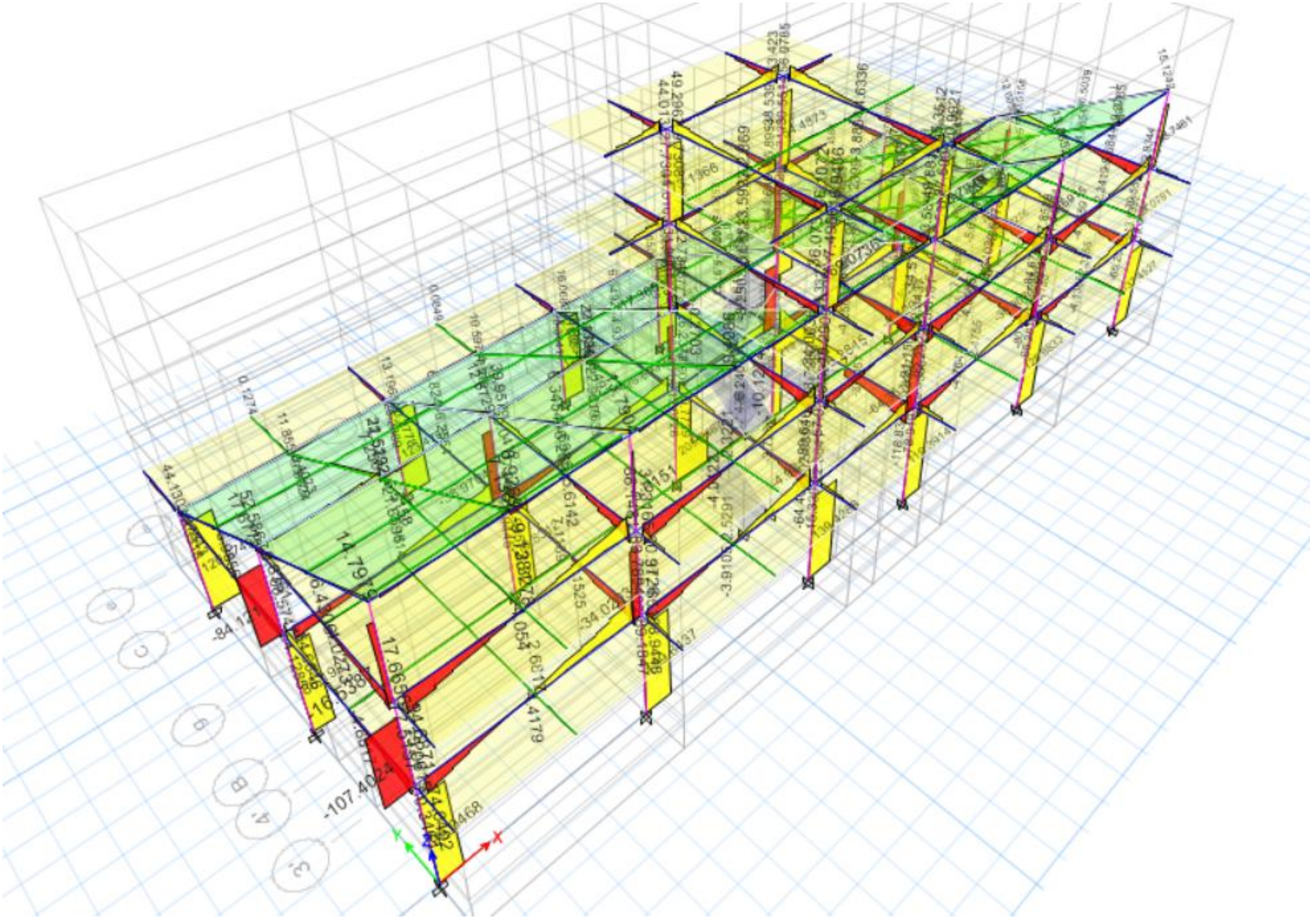


Figure 27: Shear Force Diagram in Block A 2

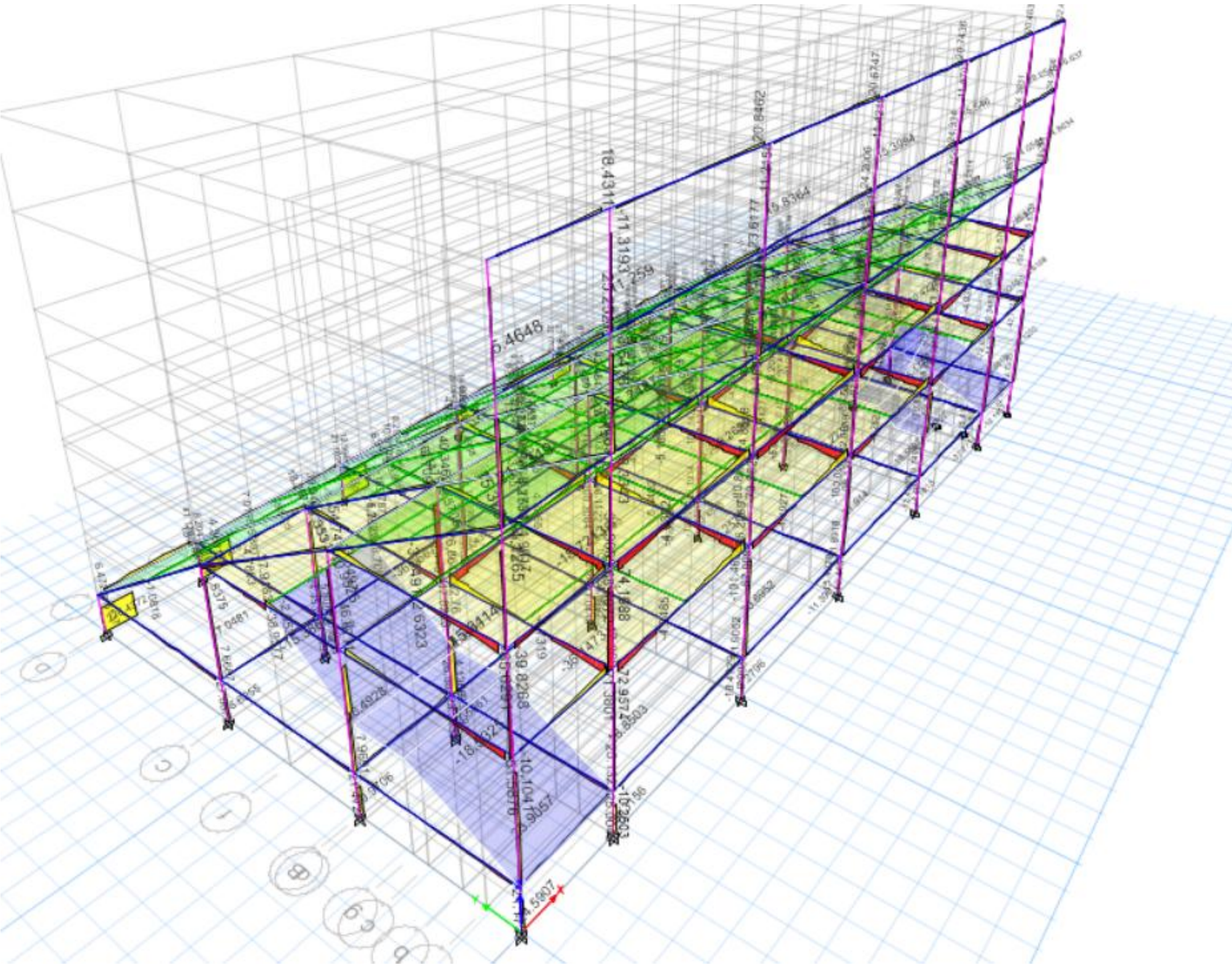


Figure 28: Shear Force Diagram in Block B 1 & 3

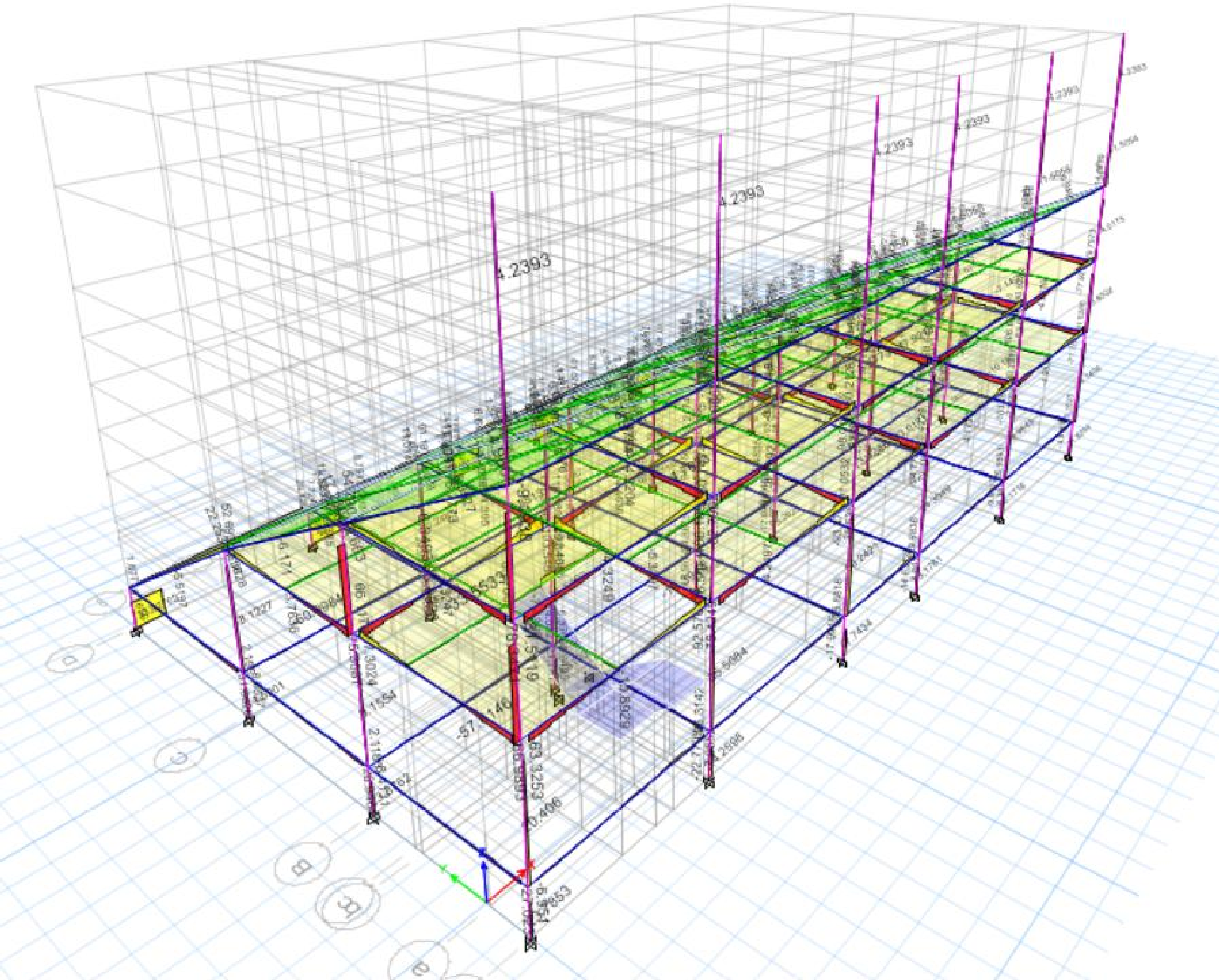


Figure 29: Shear Force Diagram in Block B 2

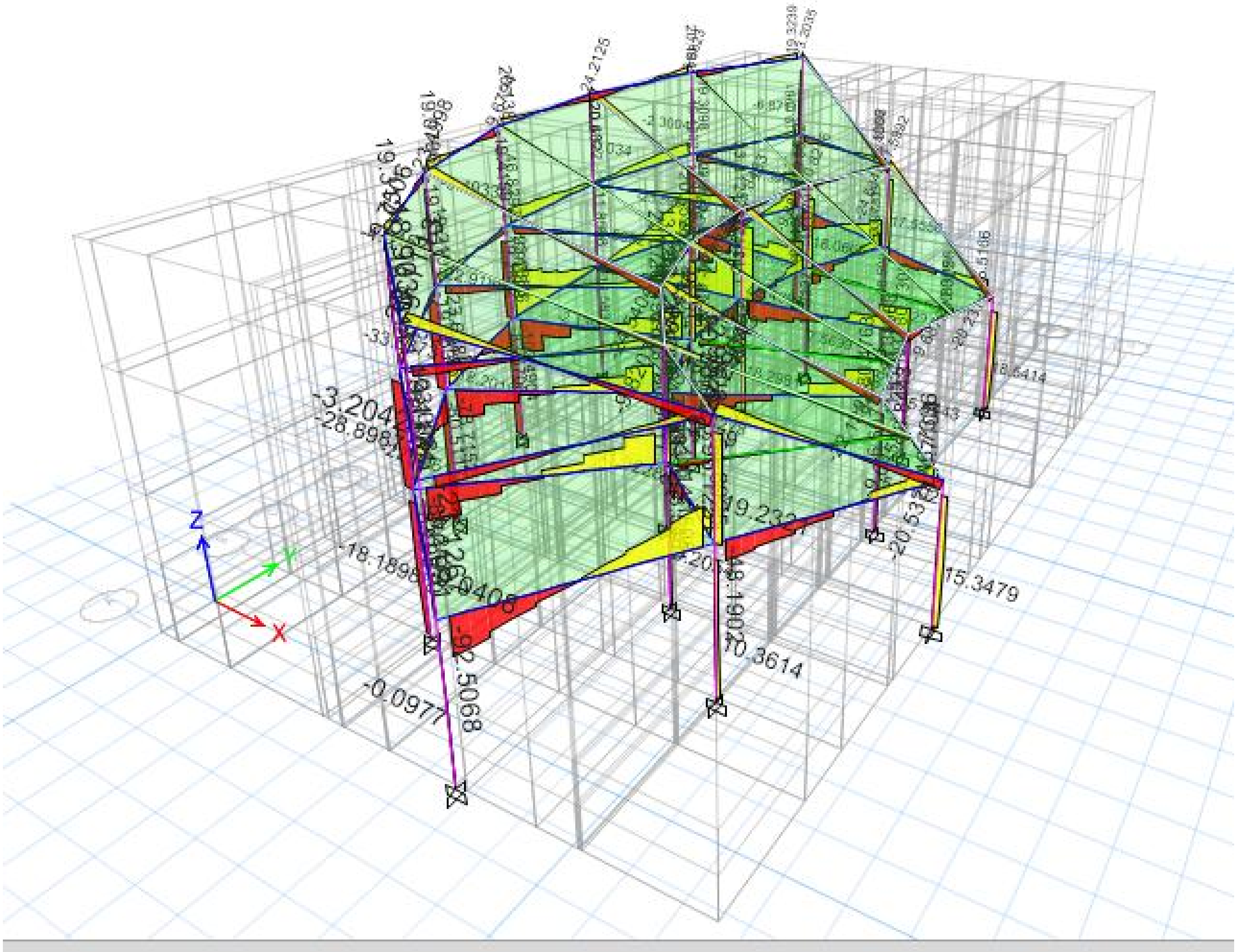


Figure 29: Shear Force Diagram in Block C

10. DESIGN OF STRUCTURL MEMBERS

10.1 Design of Slab

The slabs are kept in such a way that ly/lx is kept less than 2 such that it can be designed as two way slab. The slab is designed using ETABS V19 and checked manually on excel sheet based on IS 456:2000 and is presented in Annex.

10.2 Design of Beam

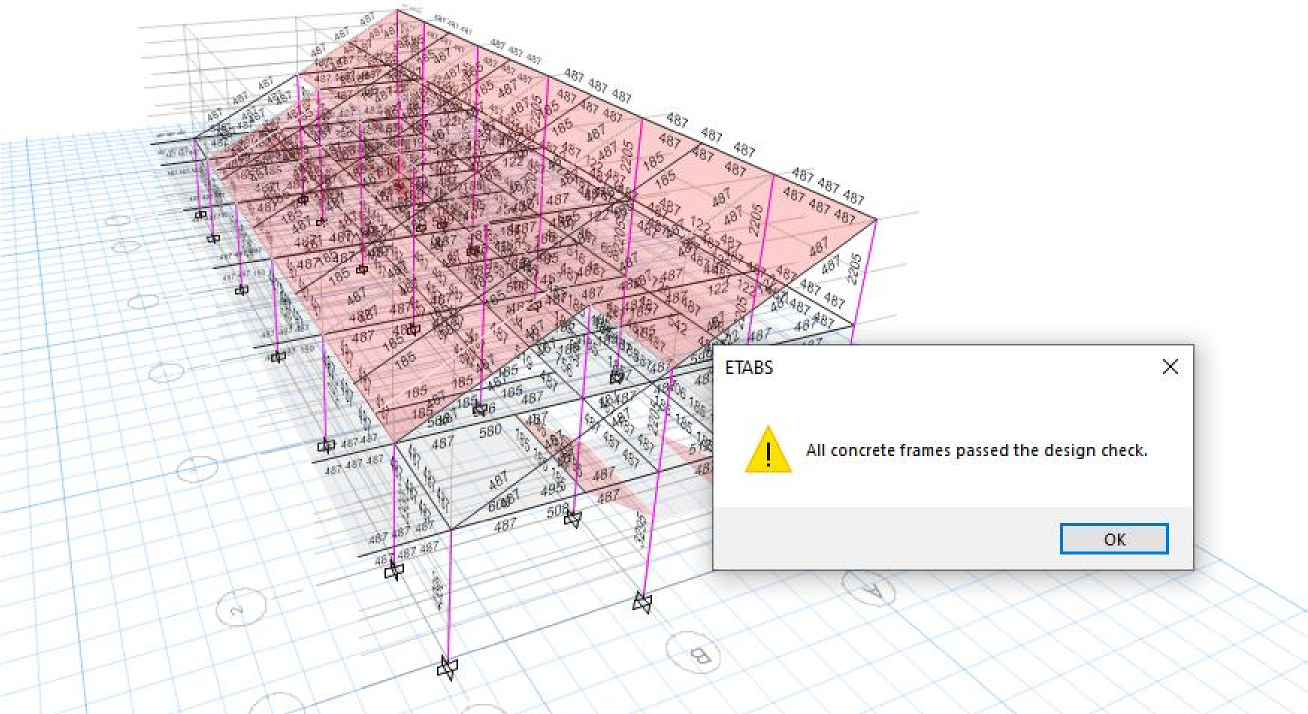


Figure 30: Showing all members passed (Block A 1 & 3)

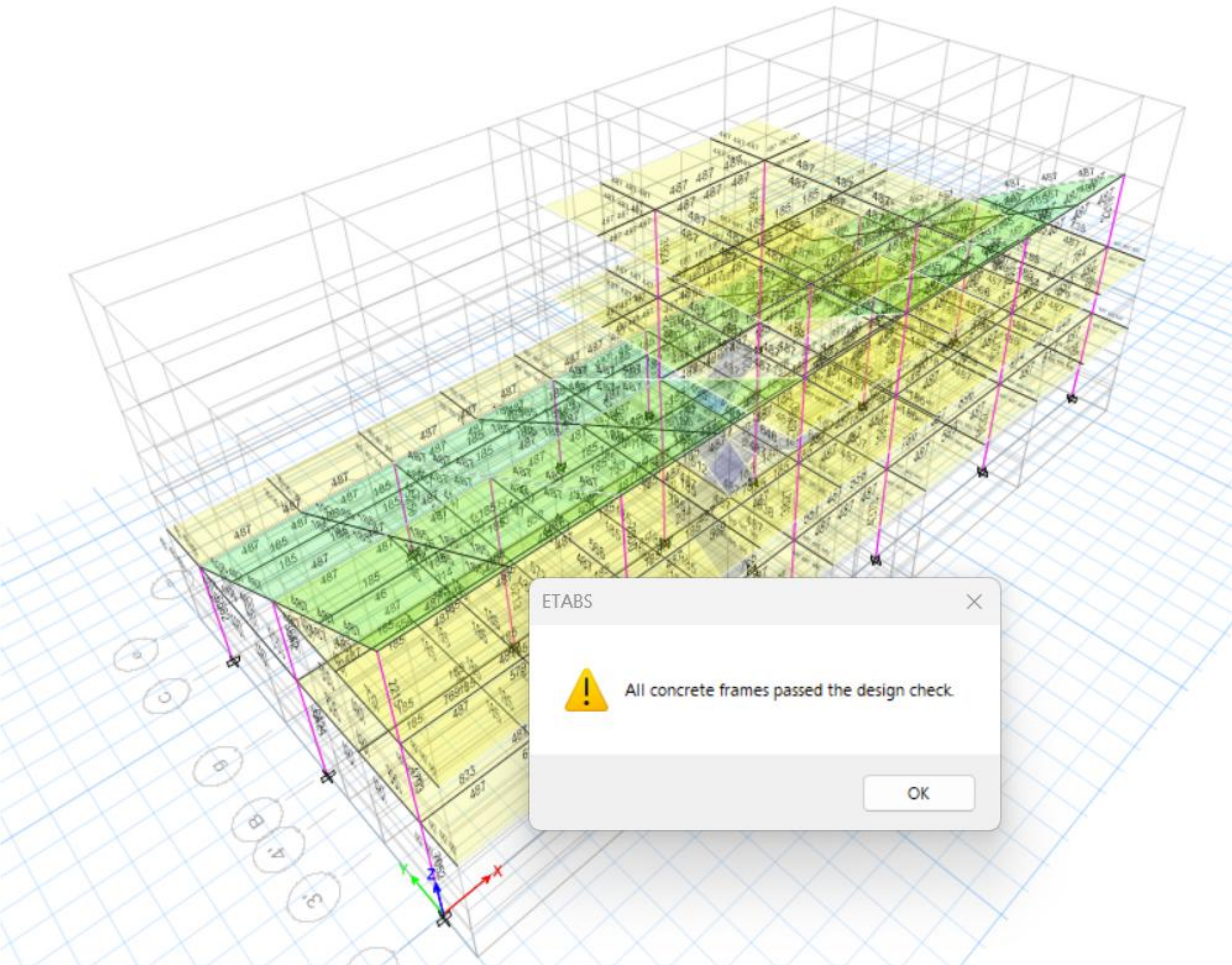


Figure 31: Showing all members passed (Block A 2)

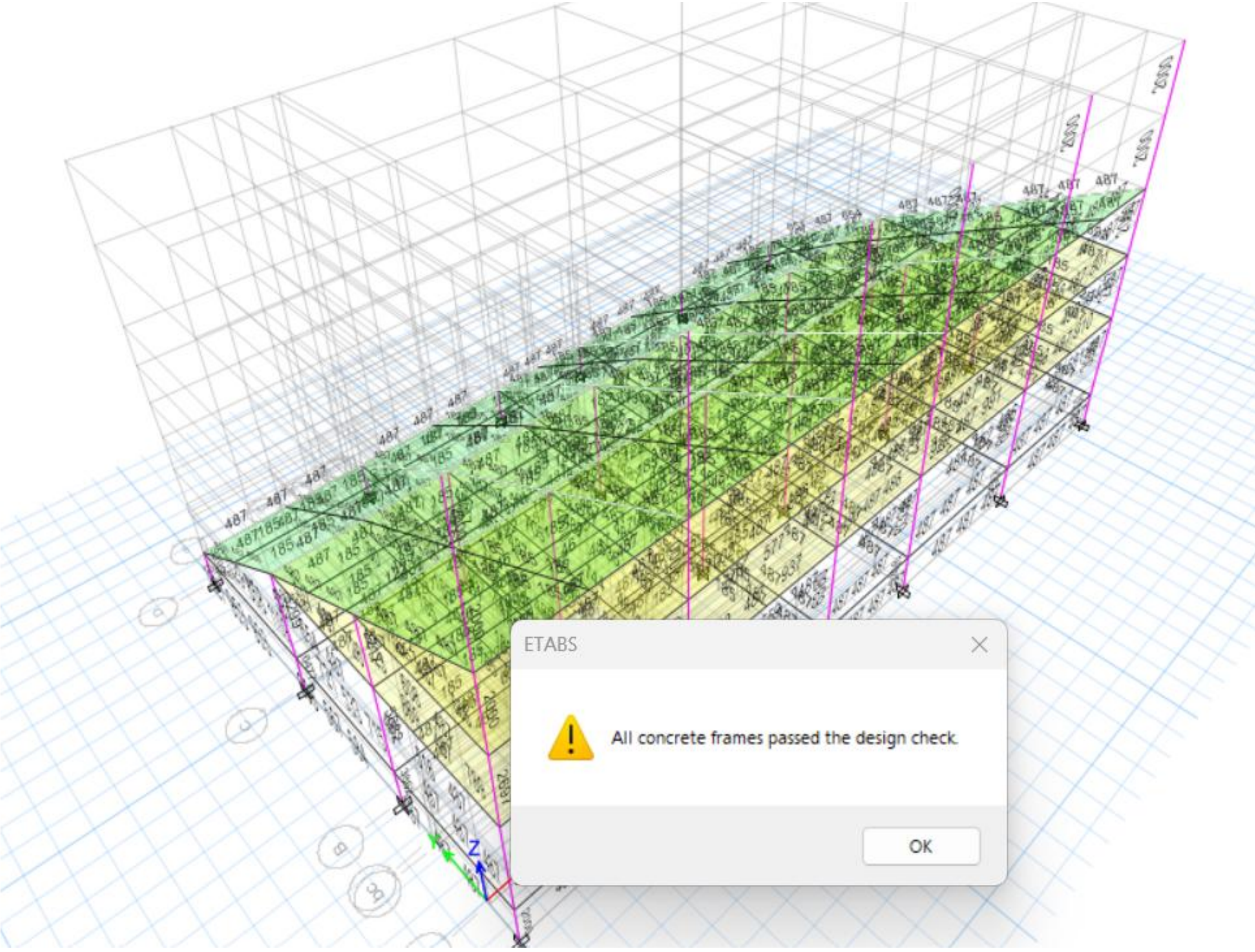


Figure 32: Showing all members passed (Block B 2)

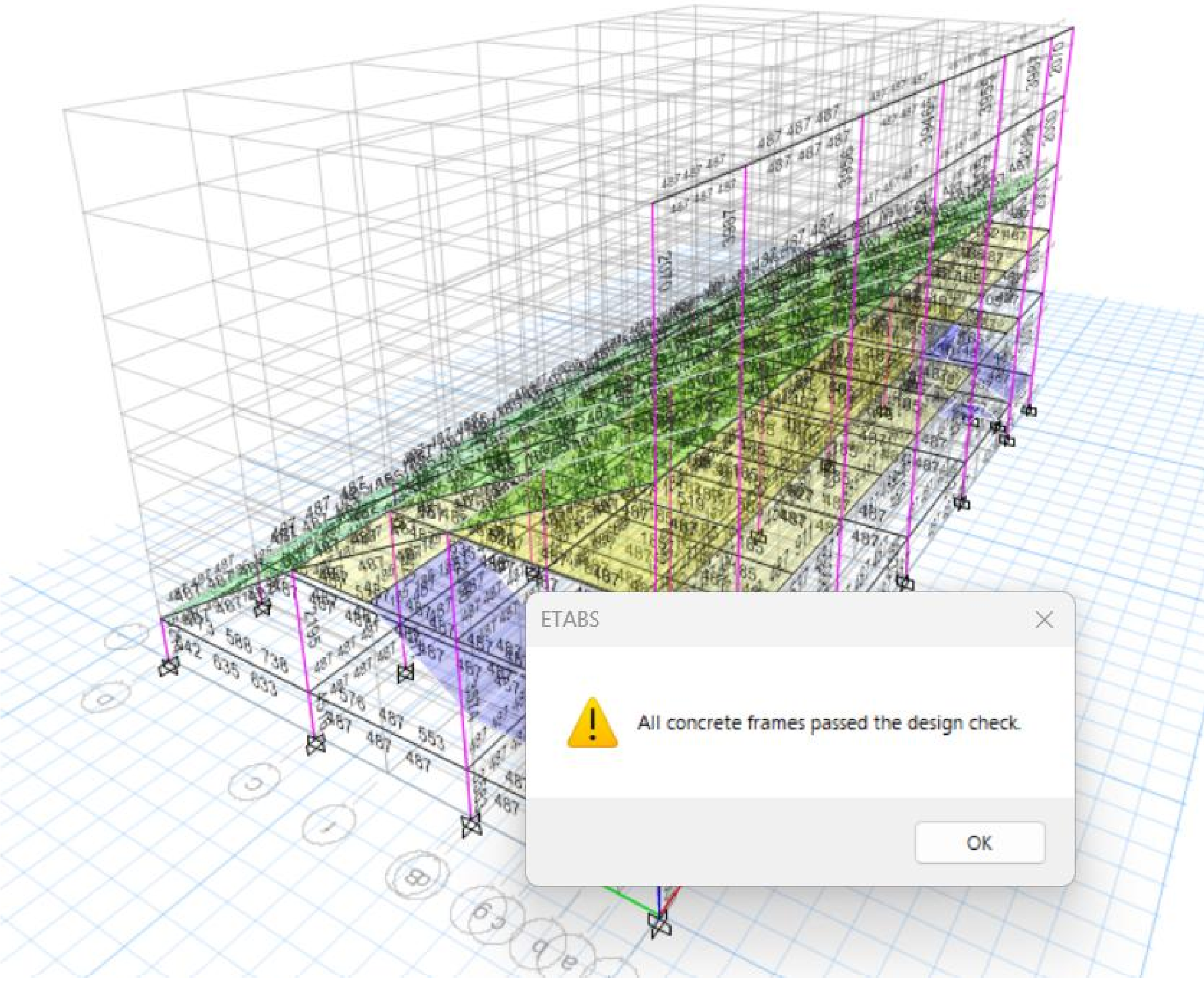


Figure 33: Showing all members passed (Block B 1 & 3)

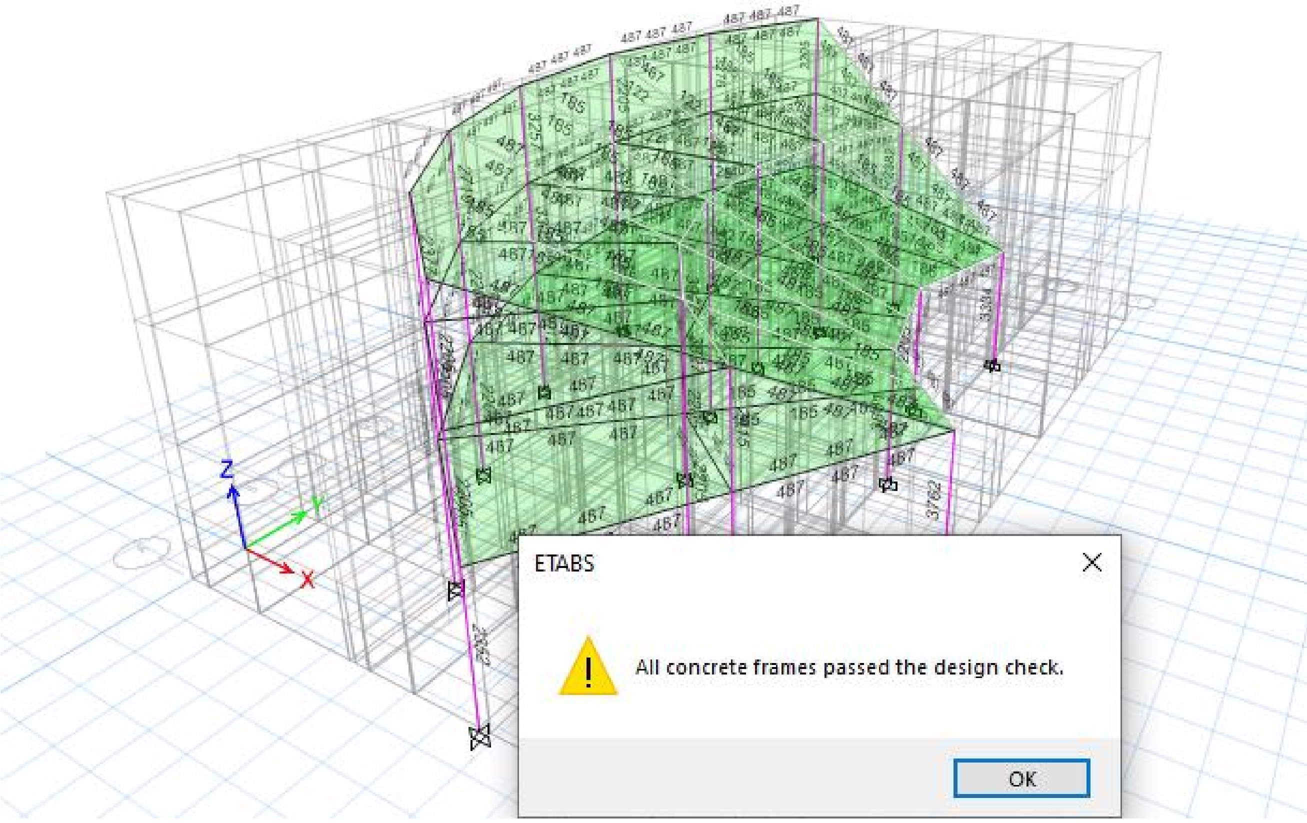


Figure 33: Showing all members passed (Block C)

The beams are designed with the help of ETABS V19 and checked manually. The calculation of reinforcement on typical section of beam is obtained by ETABS V19 as shown below in Fig.

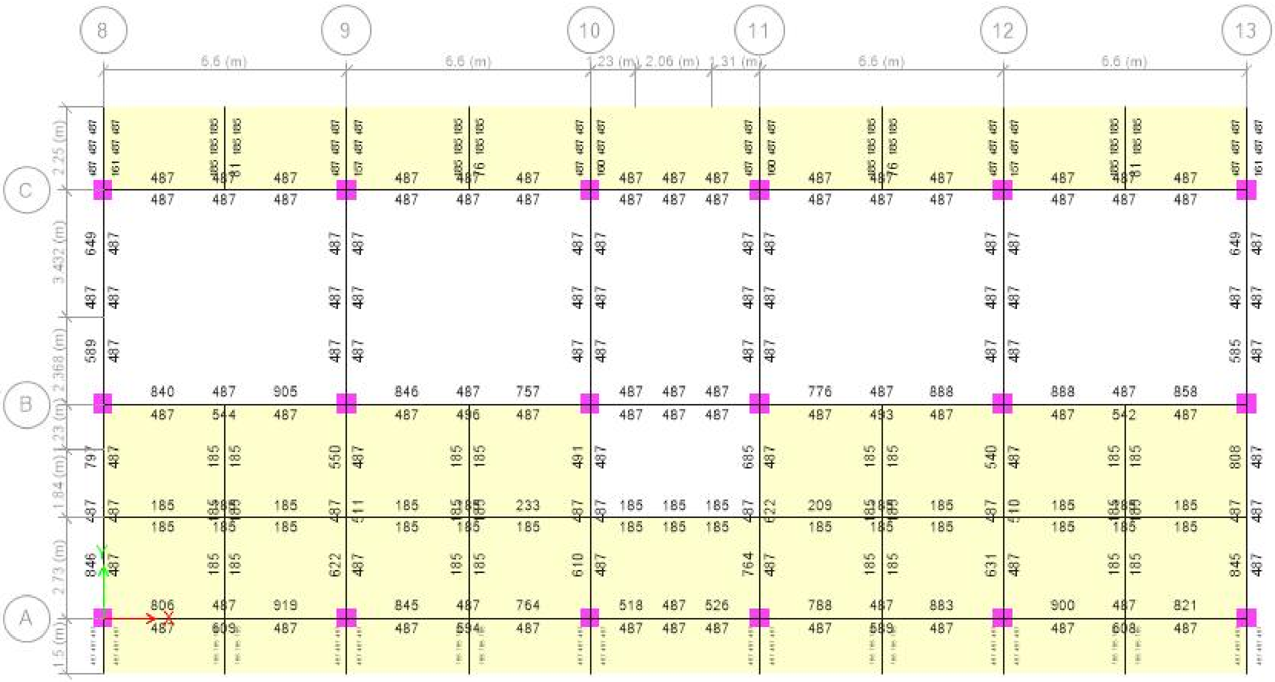


Figure 34: Sample Reinforcement at First floor beam (Block A 2)

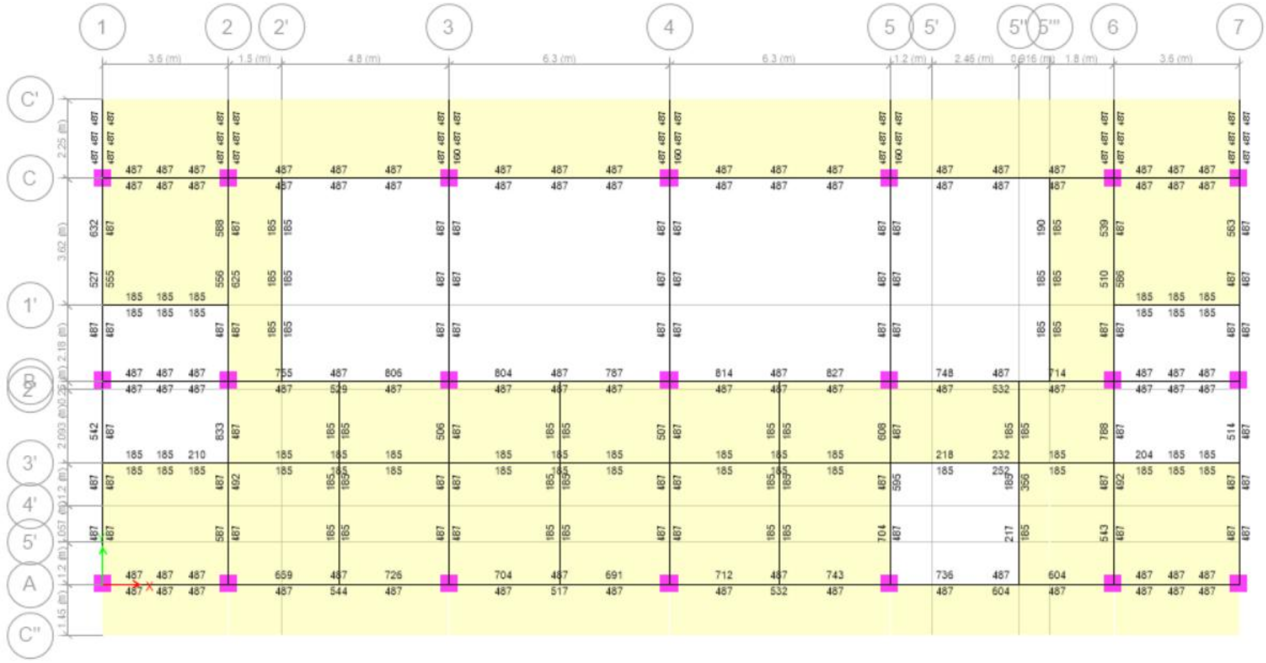


Figure 35: Sample Reinforcement at First floor beam (Block A 1 & 3)

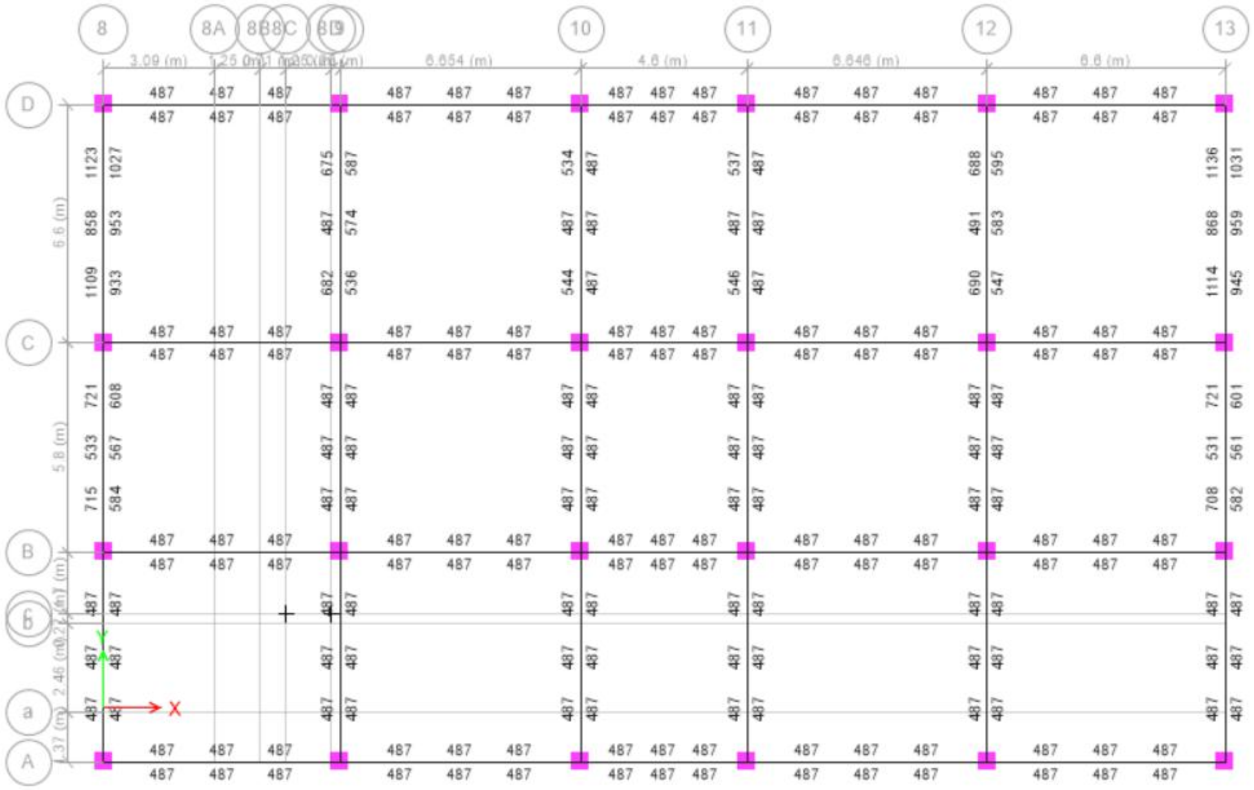


Figure 36: Sample Reinforcement at First floor beam (Block B 2)

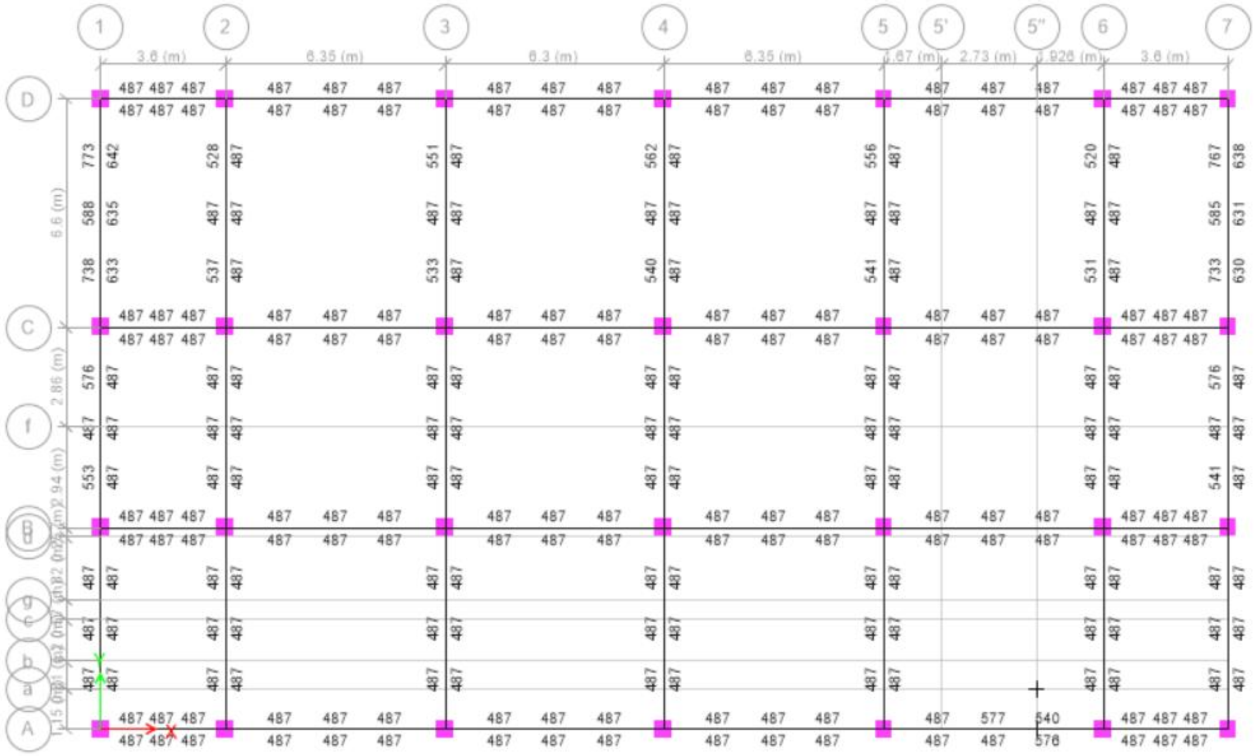


Figure 37: Sample Reinforcement at First floor beam (Block B 1 & 3)

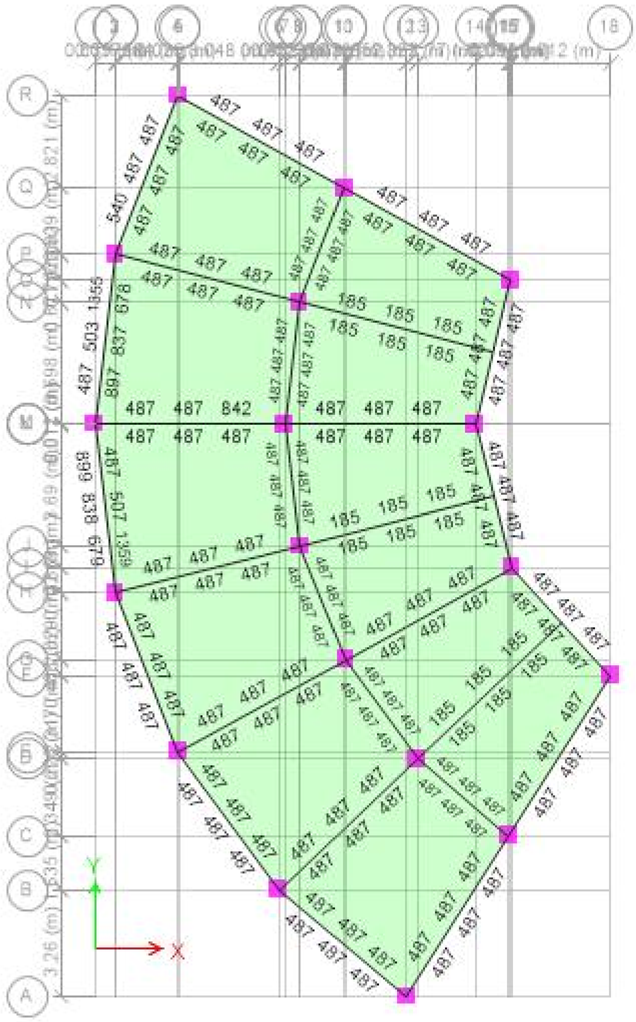


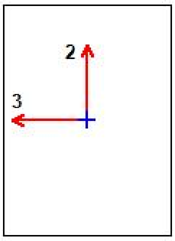
Figure 37: Sample Reinforcement at First floor beam (Block C)

Please Refer Model Provided along with the Report for Detail

The sample design of beam at first floor grid is presented below:

ETABS Concrete Frame Design

IS 456:2000 + IS 13920:2016 Beam Section Design



Beam Element Details Type: Ductile Frame (Summary)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	LLRF
Story1	B31	119	beam 400x550	DL+yLL+RSY ULS	5537.5	5800	1

Section Properties

b (mm)	h (mm)	b_f (mm)	d_s (mm)	d_{ct} (mm)	d_{cb} (mm)
400	550	400	0	43	43

Material Properties

E_c (MPa)	f_{ck} (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
25000	25	1	500	500

Design Code Parameters

γ_C	γ_S
1.5	1.15

Factored Forces and Moments

Factored M_{u3} kN-m	Factored T_u kN-m	Factored V_{u2} kN	Factored P_u kN
60.0361	3.4006	104.7296	2.5056

Design Moments, M_{u3} & M_t

Factored Moment kN-m	Factored M_t kN-m	Positive Moment kN-m	Negative Moment kN-m
60.0361	4.7509	64.787	-128.4853

Design Moment and Flexural Reinforcement for Moment, M_{u3} & T_u

	Design -Moment kN-m	Design +Moment kN-m	-Moment Rebar mm²	+Moment Rebar mm²	Minimum Rebar mm²	Required Rebar mm²
Top (+2 Axis)	-128.4853		605	0	605	487
Bottom (-2 Axis)		64.787	487	297	0	487

Shear Force and Reinforcement for Shear, V_{u2} & T_u

Shear V_e kN	Shear V_c kN	Shear V_s kN	Shear V_p kN	Rebar A_{sv} /s mm²/m
108.7002	0	122.3027	61.4955	668.46

Torsion Force and Torsion Reinforcement for Torsion, T_u & V_{U2}

T_u kN-m	V_u kN	Core b_1 mm	Core d_1 mm	Rebar A_{svt} /s mm²/m
1.3804	104.7296	334	484	385.54

10.3 Design of Column

The rectangular columns are designed with the help of ETABS V19 and checked manually. Calculation of longitudinal reinforcement of typical elements is shown below in Fig. below. The method carried out during the structural analysis is verified with other possible methods. There is not significant change in the design values. The interaction curve provided in literature is then used to design these columns.

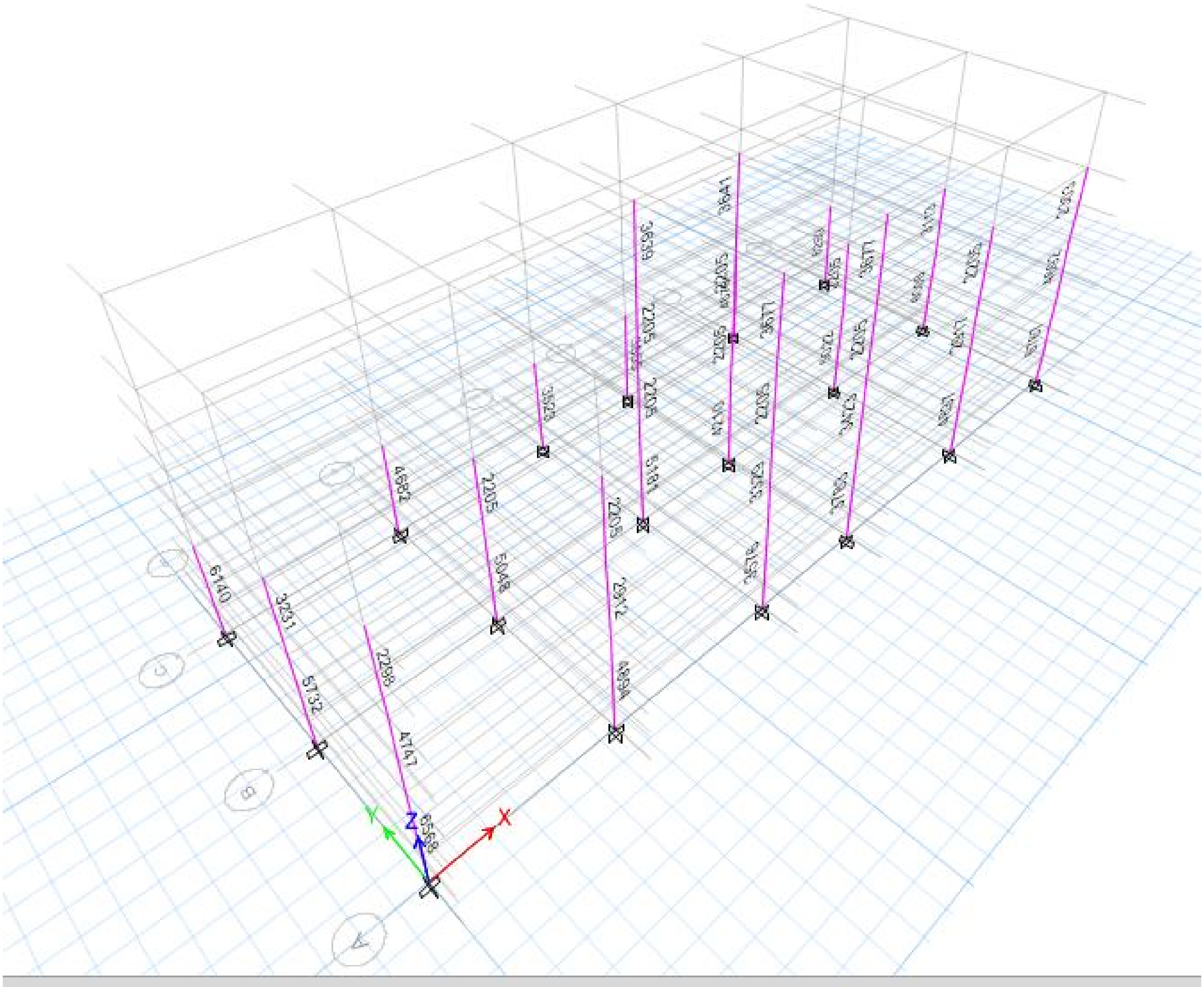


Figure 38: Column Reinforcement in Block A 2 (Columns only shown for clarity)

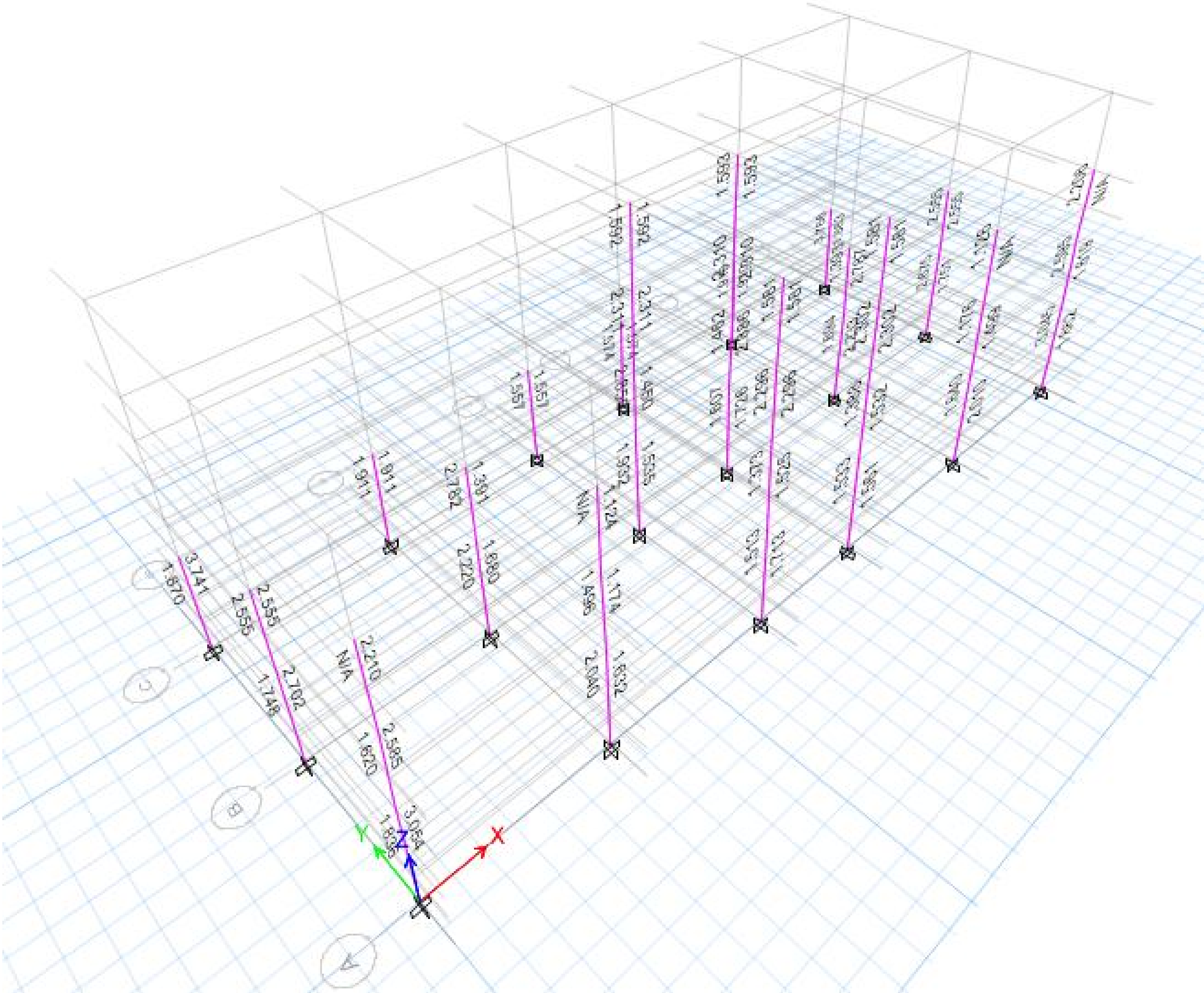


Figure 39: Column/ Beam Capacity Ratio in Block A2 (Columns only shown for clarity)

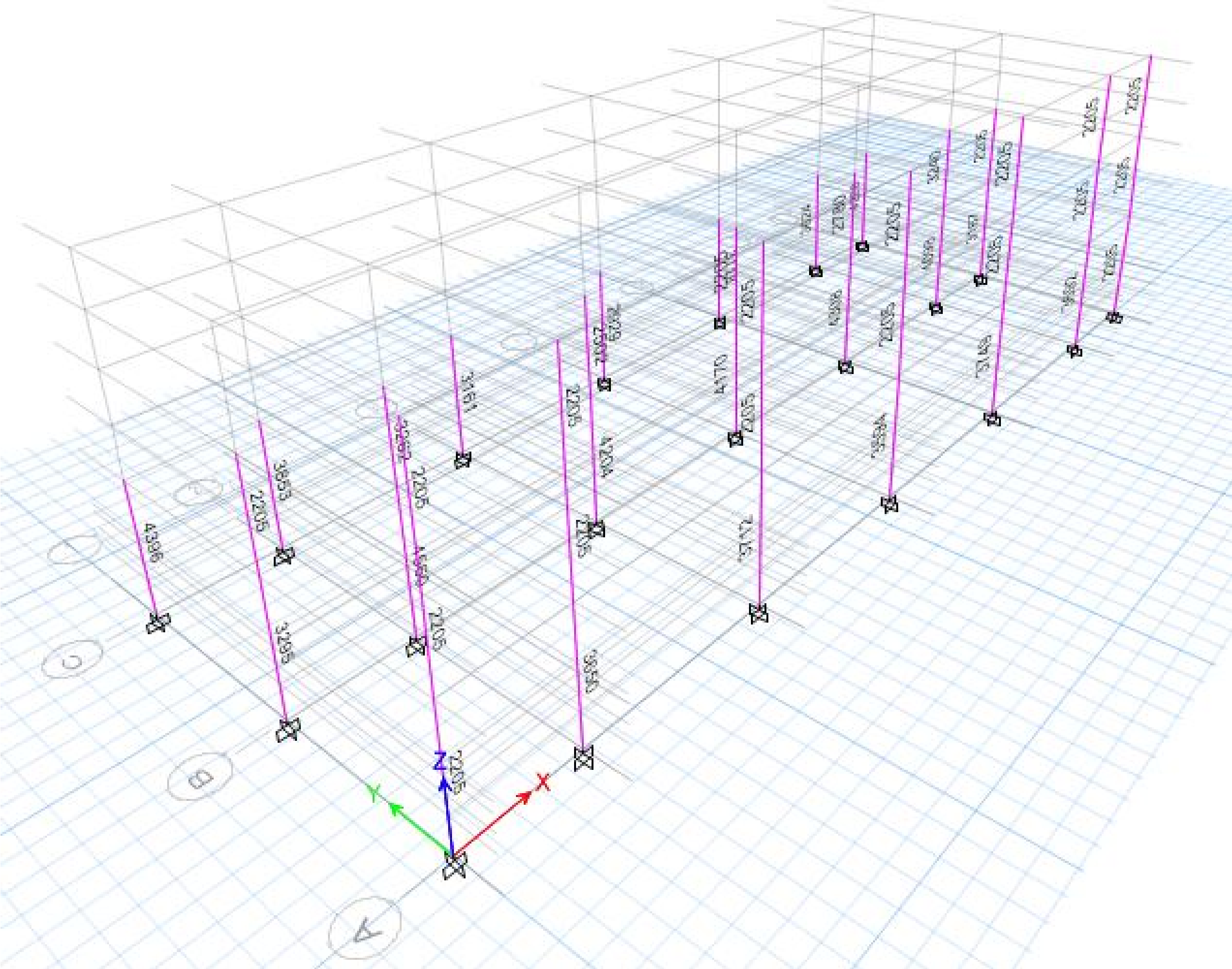


Figure 40: Column Reinforcement in Block A 1 & 3 (Columns only shown for clarity)

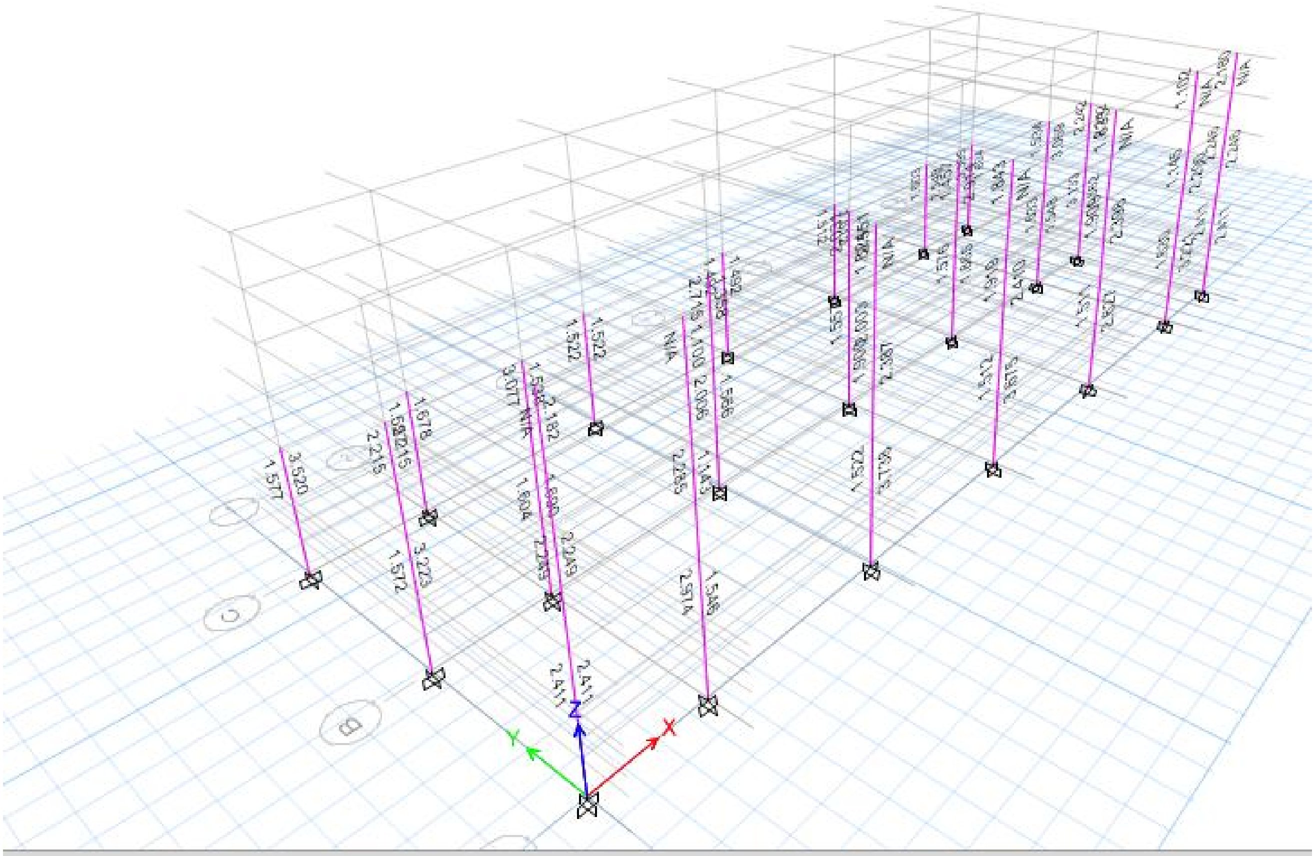


Figure 41: Column/ Beam Capacity Ratio in Block A 1 & 3 (Columns only shown for clarity)

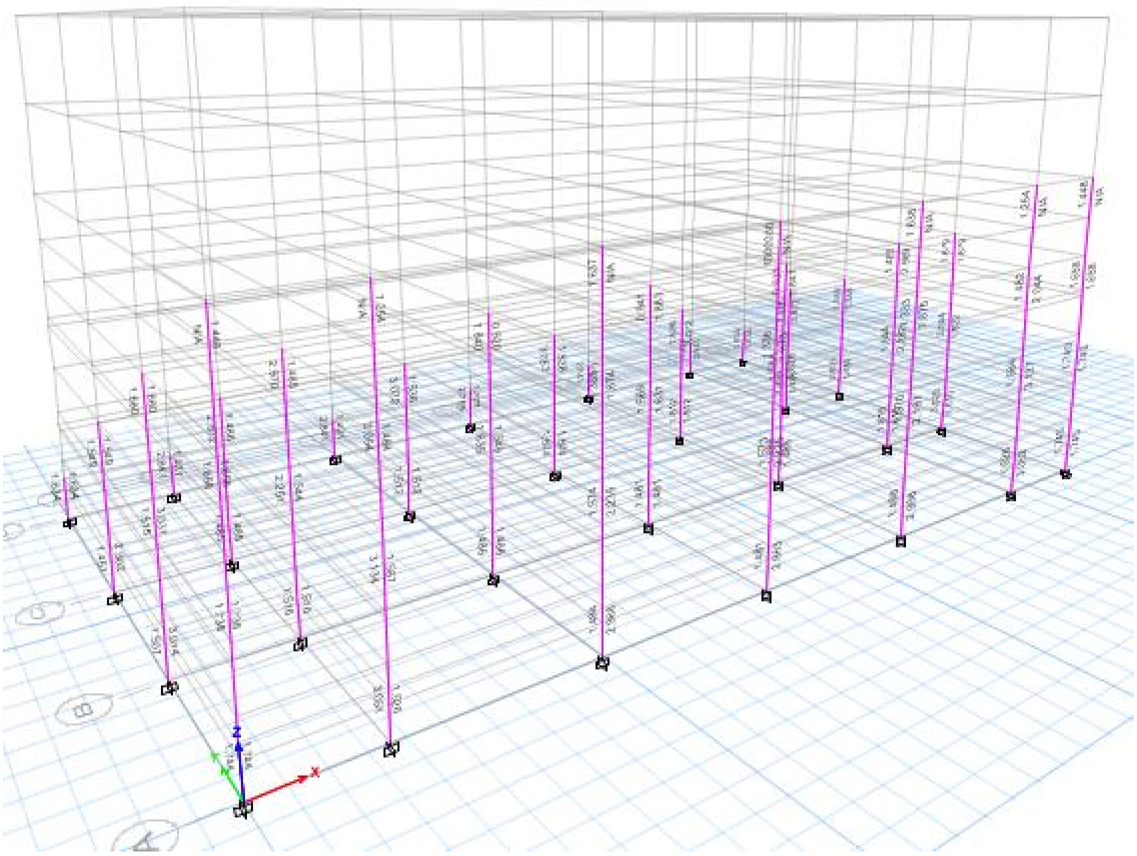


Figure 42: Column/ Beam Capacity Ratio in Block B 1 & 3 (Columns only shown for clarity)

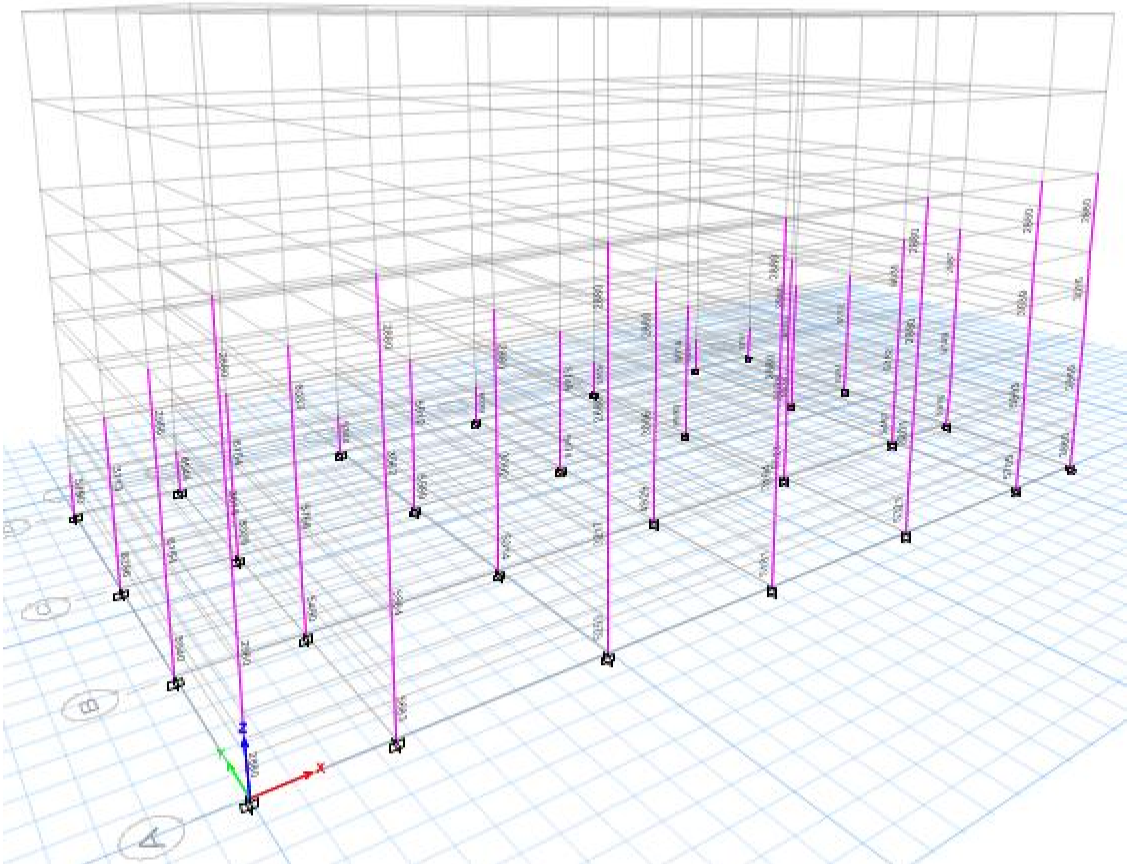


Figure 43: Column Reinforcement in Block B 1 & 3 (Columns only shown for clarity)

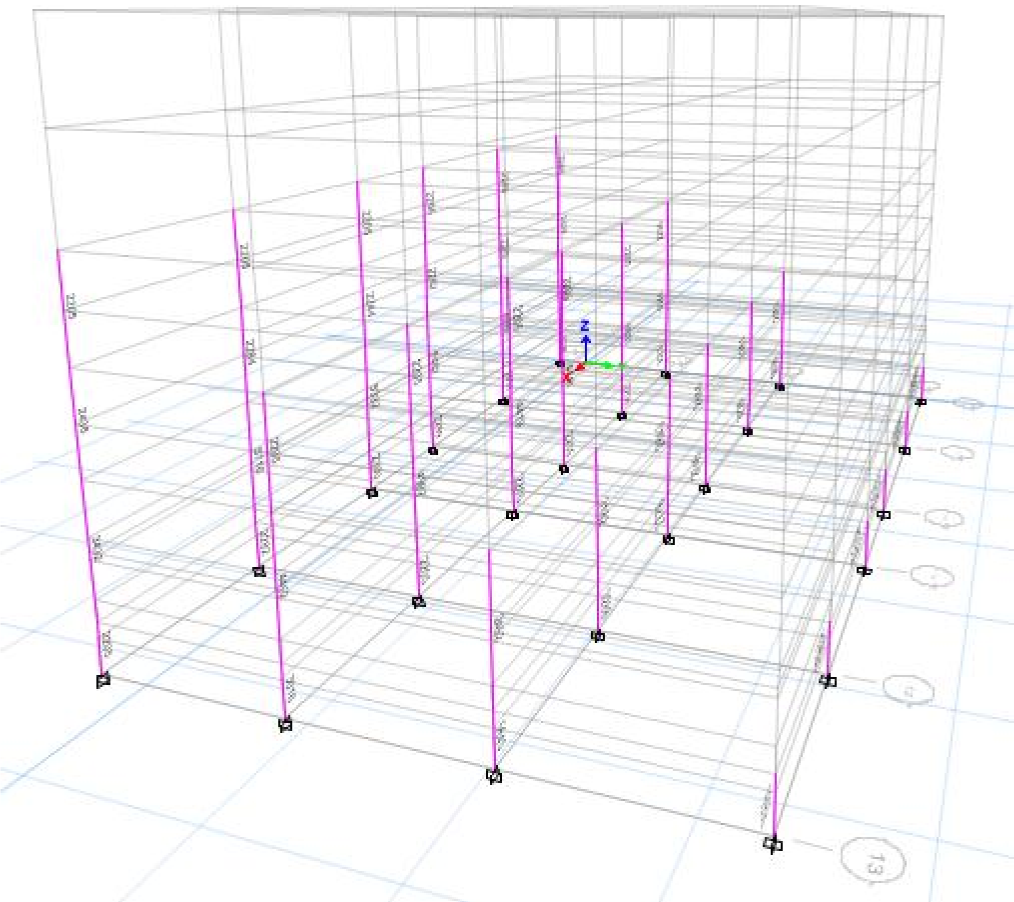


Figure 44: Column Reinforcement in Block B 2 (Columns only shown for clarity)

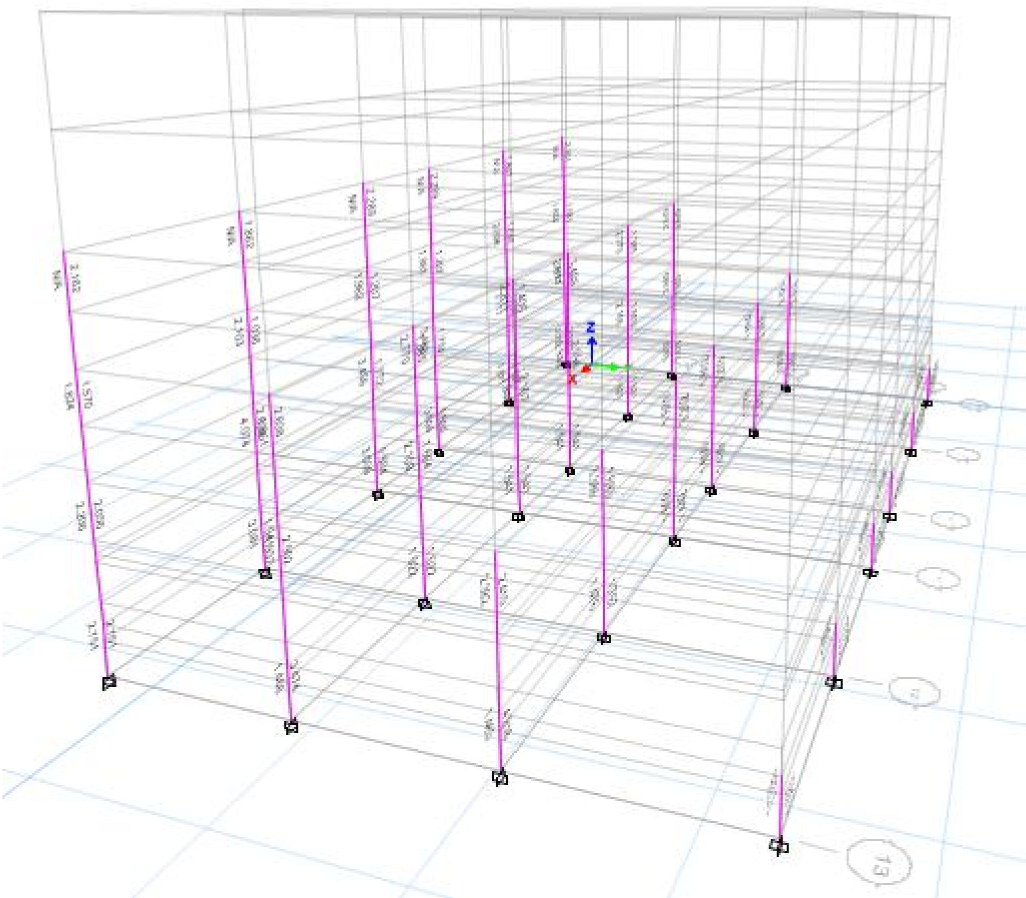


Figure 45: Column/ Beam Capacity Ratio in Block B 2(Columns only shown for clarity)

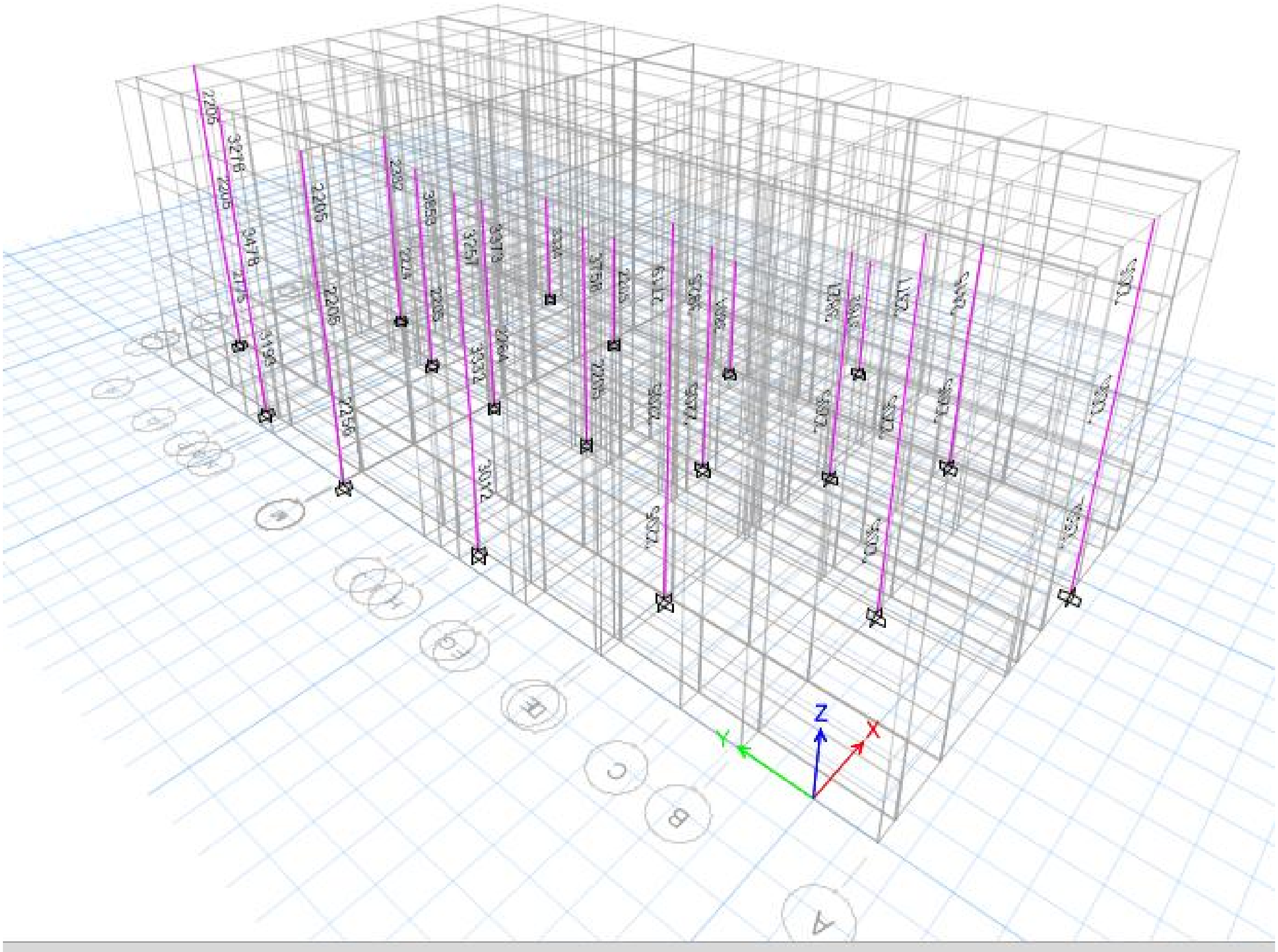


Figure 44: Column Reinforcement in Block C (Columns only shown for clarity)

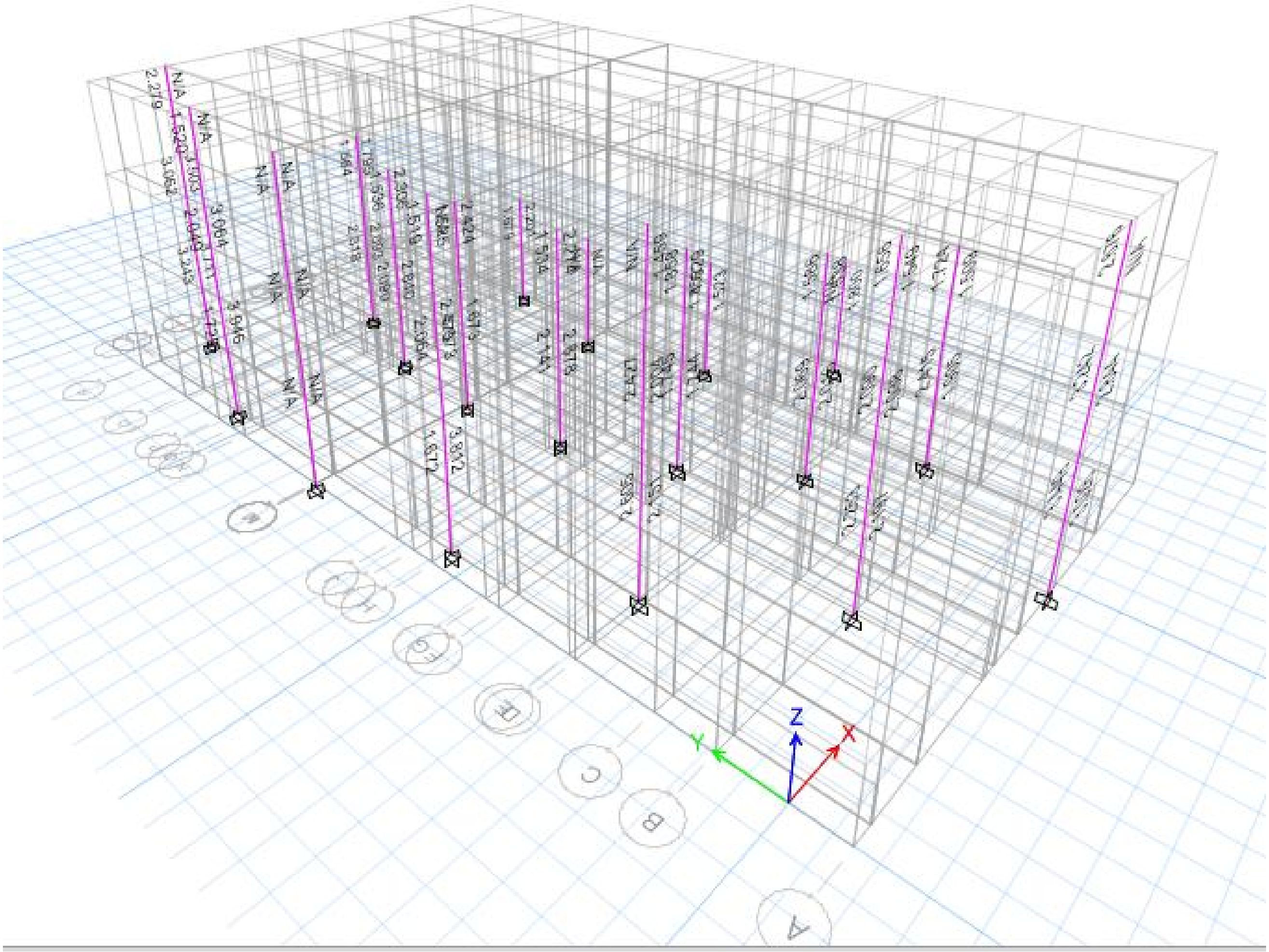
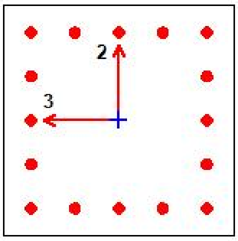


Figure 45: Column/ Beam Capacity Ratio in Block C(Columns only shown for clarity)

Please Refer Model Provided along with the Report for Detail. Sample design of column of ground floor at grid is shown below:

ETABS Concrete Frame Design

IS 456:2000 + IS 13920:2016 Column Section Design



Column Element Details Type: Ductile Frame (Summary)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	LLRF
Story1	C1	1	C 525X525	DL+yLL+RSY ULS	0	4000	1

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
525	525	62.5	30

Material Properties

E_c (MPa)	f_{ck} (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
25000	25	1	500	500

Design Code Parameters

γ_C	γ_S
1.5	1.15

Axial Force and Biaxial Moment Design For P_u , M_{u2} , M_{u3}

Design P_u kN	Design M_{u2} kN-m	Design M_{u3} kN-m	Minimum M_2 kN-m	Minimum M_3 kN-m	Rebar Area mm^2	Rebar % %
362.1349	204.4236	-32.5648	8.8361	8.8361	2205	0.8

Axial Force and Biaxial Moment Factors

	K Factor Unitless	Length mm	Initial Moment kN-m	Additional Moment kN-m	Minimum Moment kN-m
Major Bend(M3)	0.653724	3450	-13.0259	0	8.8361
Minor Bend(M2)	0.667304	3450	85.794	0	8.8361

Shear Design for V_{u2} , V_{u3}

	Shear V_u kN	Shear V_c kN	Shear V_s kN	Shear V_p kN	Rebar A_{sv} /s mm^2/m
Major, V_{u2}	72.6649	131.0211	97.1255	72.6649	581.93
Minor, V_{u3}	85.9634	131.0211	97.1255	72.6649	581.93

Joint Shear Check/Design

	Joint Shear Force kN	Shear V_{Top} kN	Shear $V_{u,Tot}$ kN	Shear V_c kN	Joint Area cm^2	Shear Ratio Unitless
Major Shear, V_{u2}	N/A	N/A	N/A	N/A	N/A	N/A
Minor Shear, V_{u3}	N/A	N/A	N/A	N/A	N/A	N/A

(1.4) Beam/Column Capacity Ratio

Major Ratio	Minor Ratio
N/A	N/A

Additional Moment Reduction Factor k (IS 39.7.1.1)

A_g cm^2	A_{sc} cm^2	P_{uz} kN	P_b kN	P_u kN	k Unitless
2756.3	22.1	3927.6563	1374.92	362.1349	1

Additional Moment (IS 39.7.1)

	Consider M_a	Length Factor	Section Depth (mm)	KL/Depth Ratio	KL/Depth Limit	KL/Depth Exceeded	M_a Moment (kN-m)
Major Bending (M_3)	Yes	0.863	525	4.296	12	No	0
Minor Bending (M_2)	Yes	0.863	525	4.385	12	No	0

Notes:

N/A: Not Applicable

N/C: Not Calculated

N/N: Not Needed

10.4 Design of foundation

The foundations used in the building are of Mat type as per the requirements. The depth of the footing is governed by one way and two way shear (punching shear).The soil type is assumed to be of medium type. So the allowable bearing capacity of soil is taken from the soil test report. Bearing capacity of the isolated footing is at the depth of 2m.

Allowable bearing capacity = 80KN/m2

The design of isolated footing has been carried out manually as per IS 456 2000 and is presented in Annex.

10.5 Design of staircase

The staircase used in the building is of open well type. The design of staircase is done manually using IS 456 2000 as presented in Annex.

11. CONCLUDING REMARKS

Reinforced concrete construction is common all over the world. It is used extensively for construction of variety of structures such as buildings, bridges, dams, water tanks, stadium, towers, chimneys, tunnels and so on.

Experiences from past earthquakes and extensive laboratory works have shown that a well-designed and detailed reinforced concrete structure is suitable for earthquake resistant structure. Ductility and strength required to resist major earthquake can be achieved by following the recommendations made in the standard codes of practice for earthquake resistant design.

Detailing of steel reinforcement is an important aspect of structural design. Poor reinforcement detailing can lead to structural failures. Detailing plays an important role in seismic resistant design. In seismic resistant design, actual forces experienced by the structure are reduced and reliance is placed on the ductility of the structure. And, ductility can be achieved by proper detailing only. Thus, in addition to design, attention should be paid on amount, location and arrangement of reinforcement to achieve ductility as well as strength.

Design and construction of the structure are inter – related jobs. A building behaves in a manner how it has been built rather than what the intensions is during designing. A large percentage of structural failures are attributed due to poor quality of construction. Therefore, quality assurance is needed in both design and construction.

In earthquake resistant construction quality of materials and workmanship plays a very important role. It has been observed that damages during earthquakes are largely dependent on the quality and workmanship. Hence, quality assurance is the most important factor in the good seismic behavior of the structure.

12. REFERENCE CODE

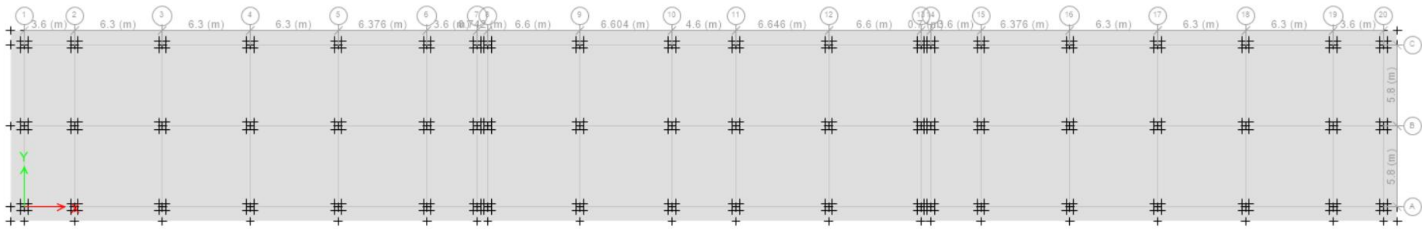
NBC 110: 1994	Plain and Reinforced Concrete
NBC 102: 1994	Unit Weights of Materials
NBC 103: 1994	Occupancy Load (Imposed Load)
NBC 104: 1994	Wind Load
NBC105: 2020	Seismic Design of Buildings in Nepal
NS: 501-2058	Code of Practice for Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces
SP: 16-1980	Design Aids for Reinforced Concrete to IS: 456-1978
SP: 34-1987	Handbook on Concrete Reinforcement Detailing
IS: 456-2000	Plain and reinforced concrete code
IS: 1893-2002	Earthquake resistant design of structure
IS: 13920	Ductility code

ANNEX

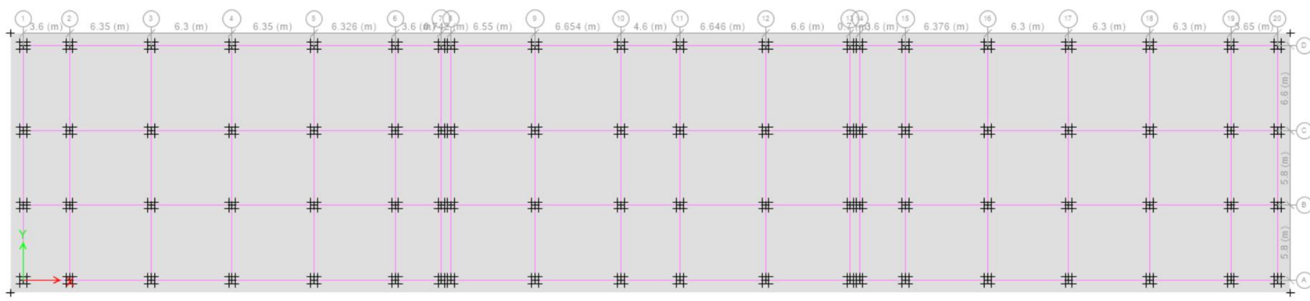
Annex 1: Design of Staircase

Open well type staircase has been designed manually as per IS 456: 2000 and is presented below:

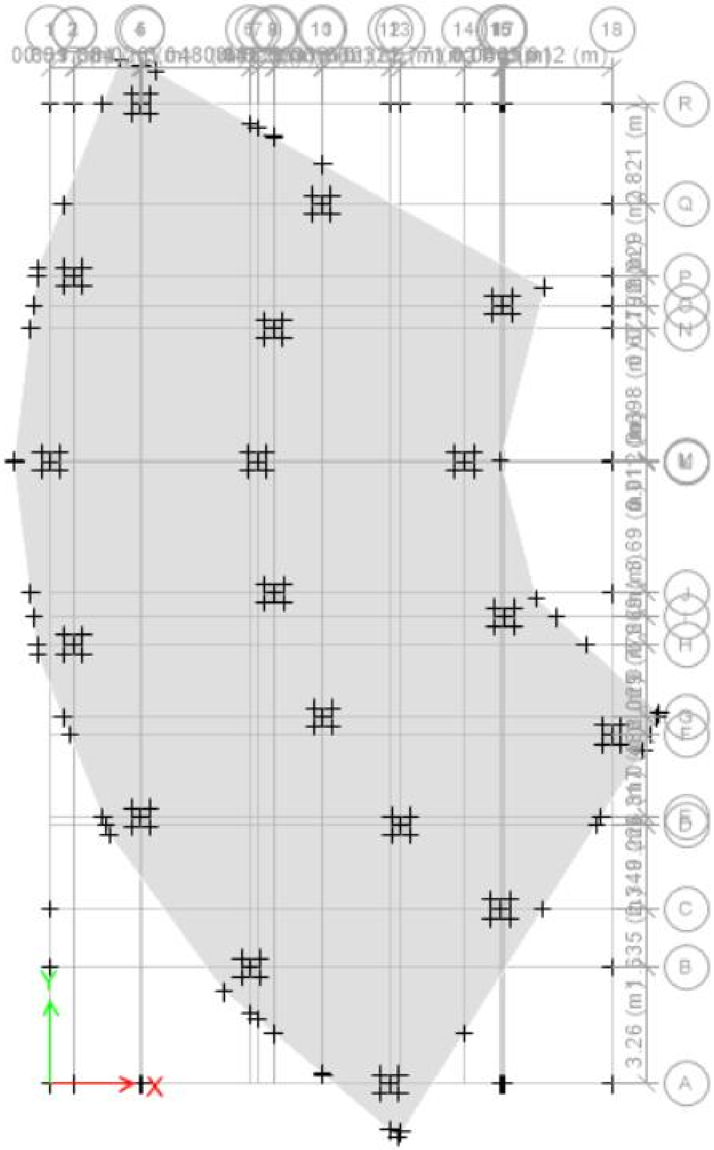
Design of Raft Footing:



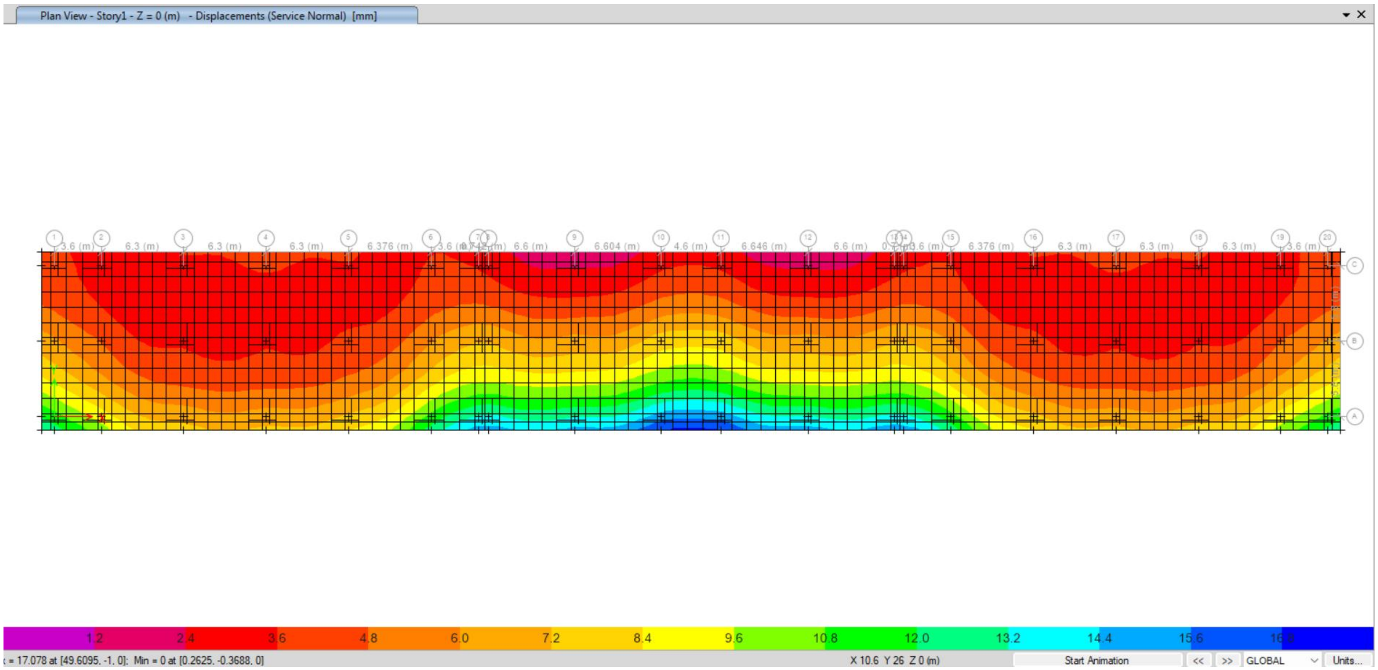
Block A Raft Footing Dimension



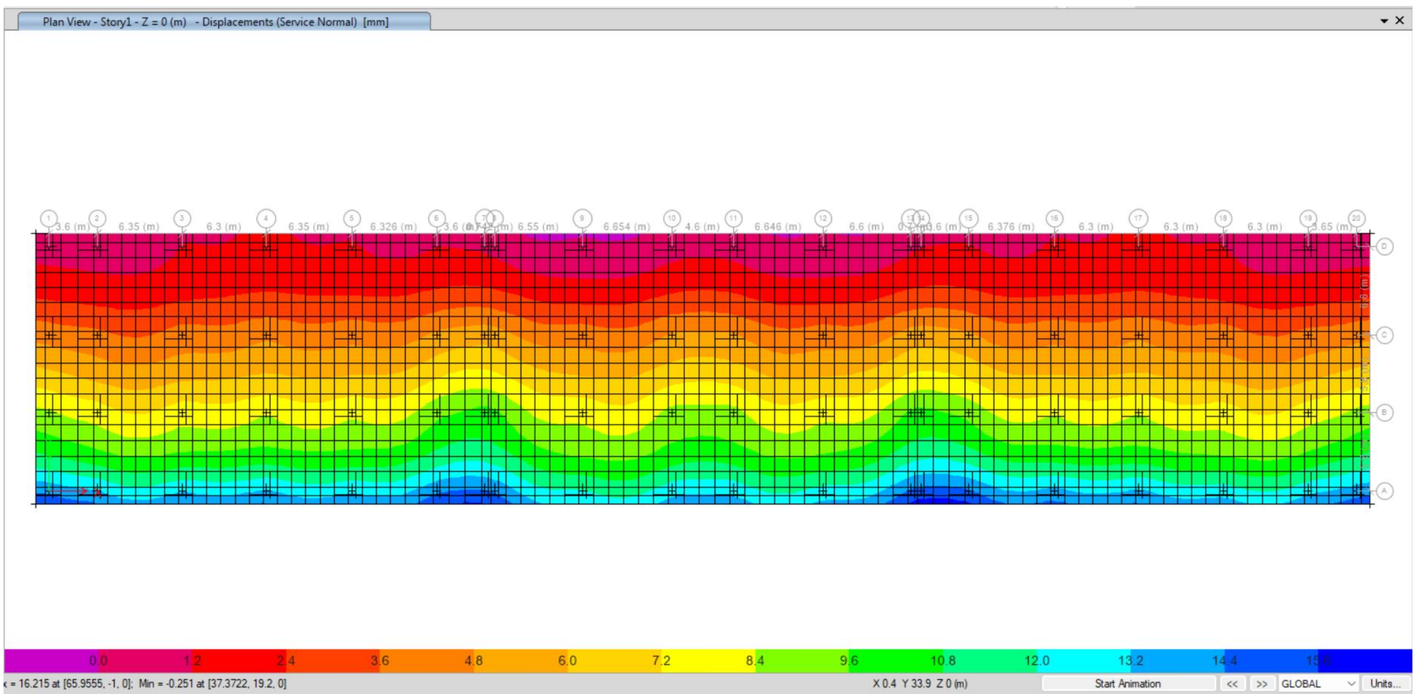
Block B Raft Footing Dimension



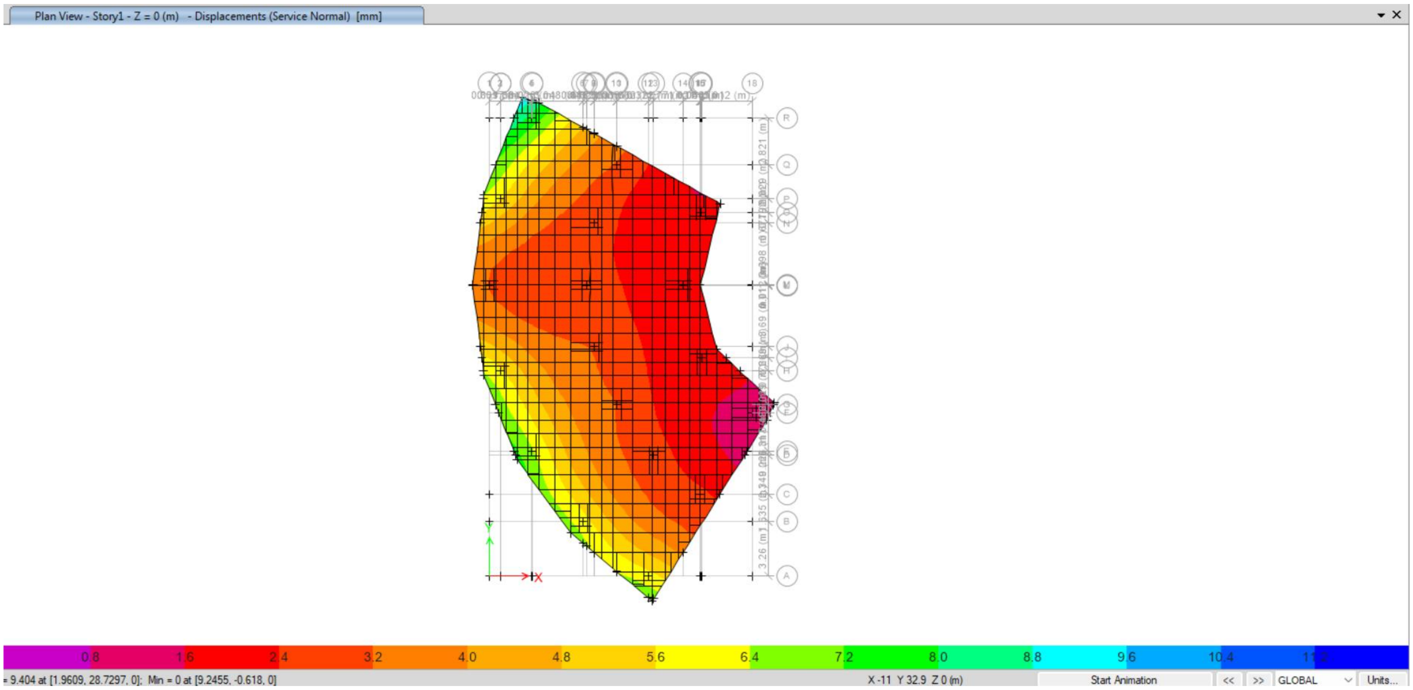
Block C Raft Footing Dimension



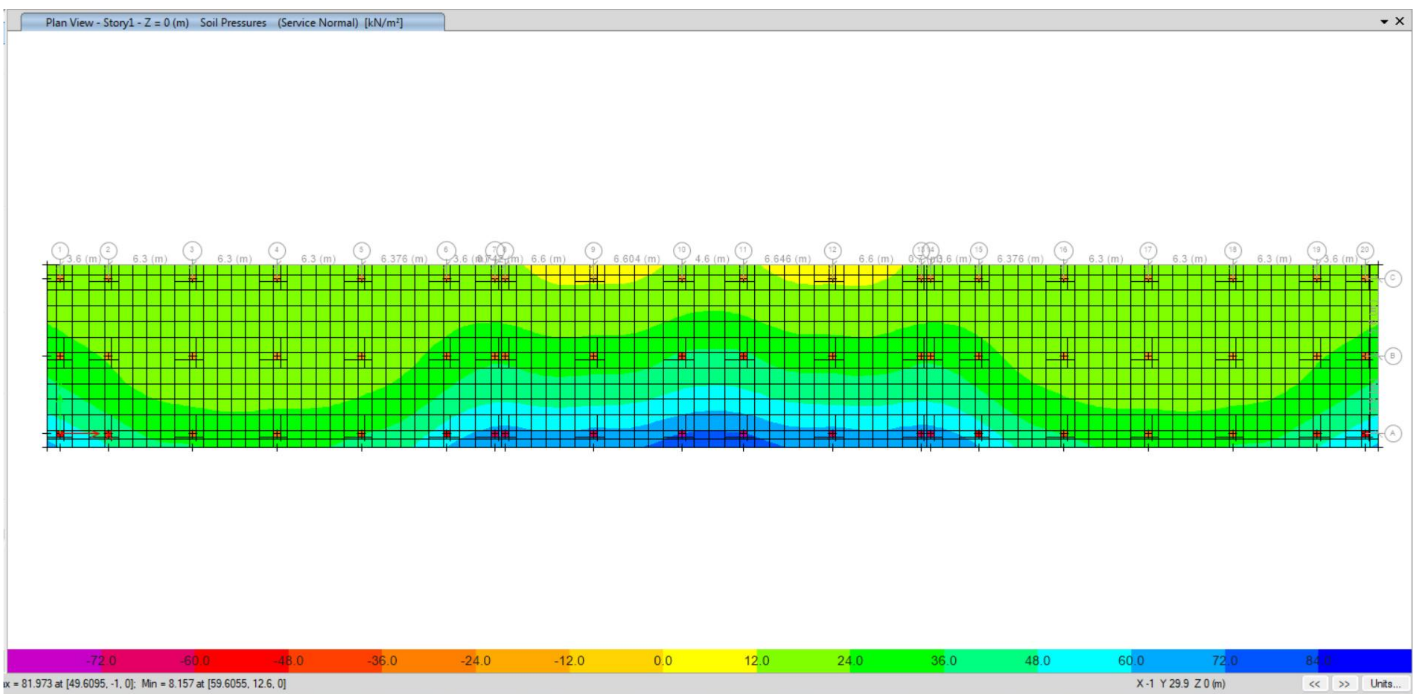
Block A Raft Footing Settlement < 25 mm



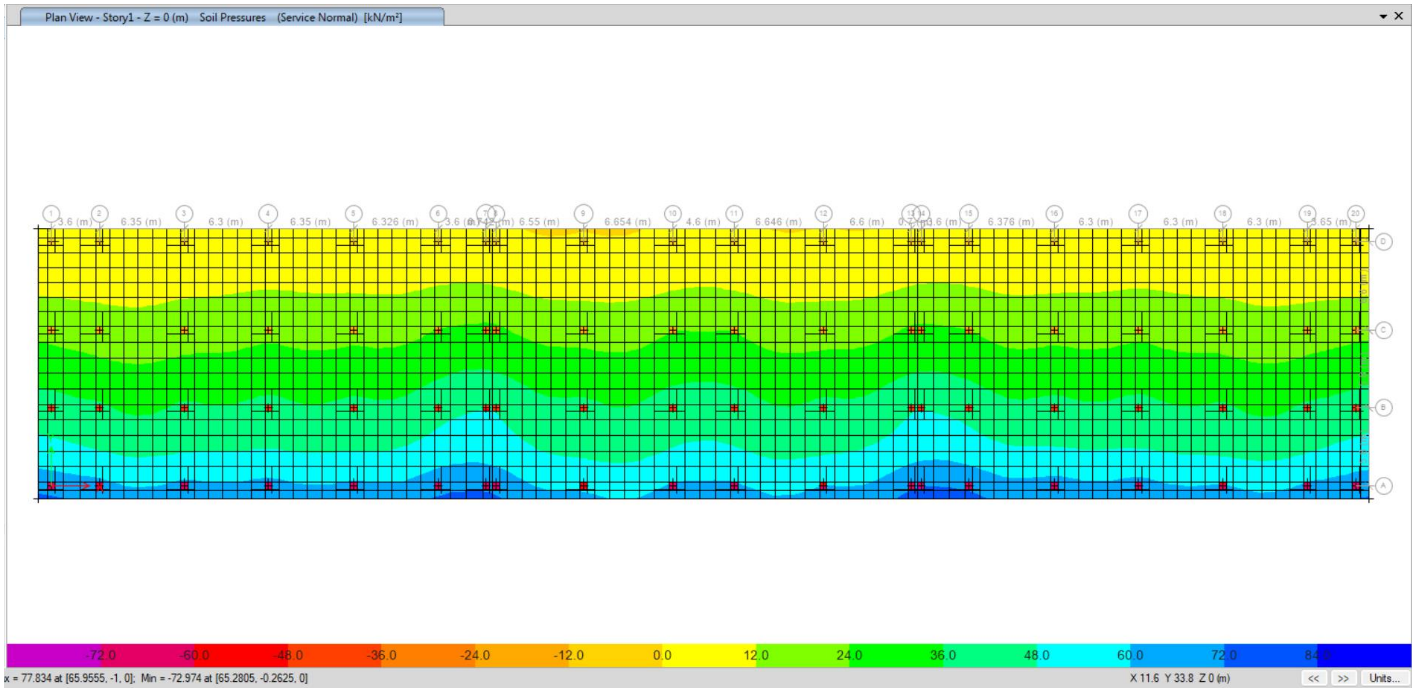
Block B Raft Footing Settlement < 25 mm



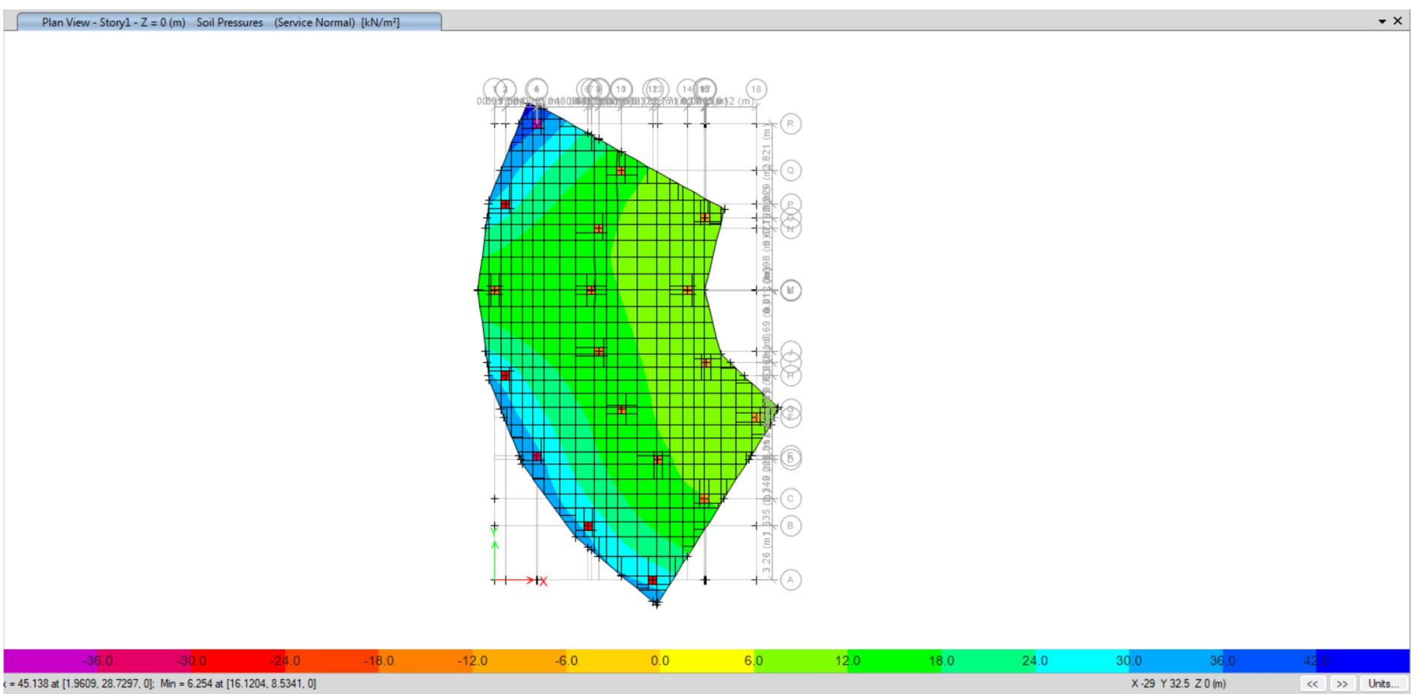
Block C Raft Footing Settlement < 25 mm



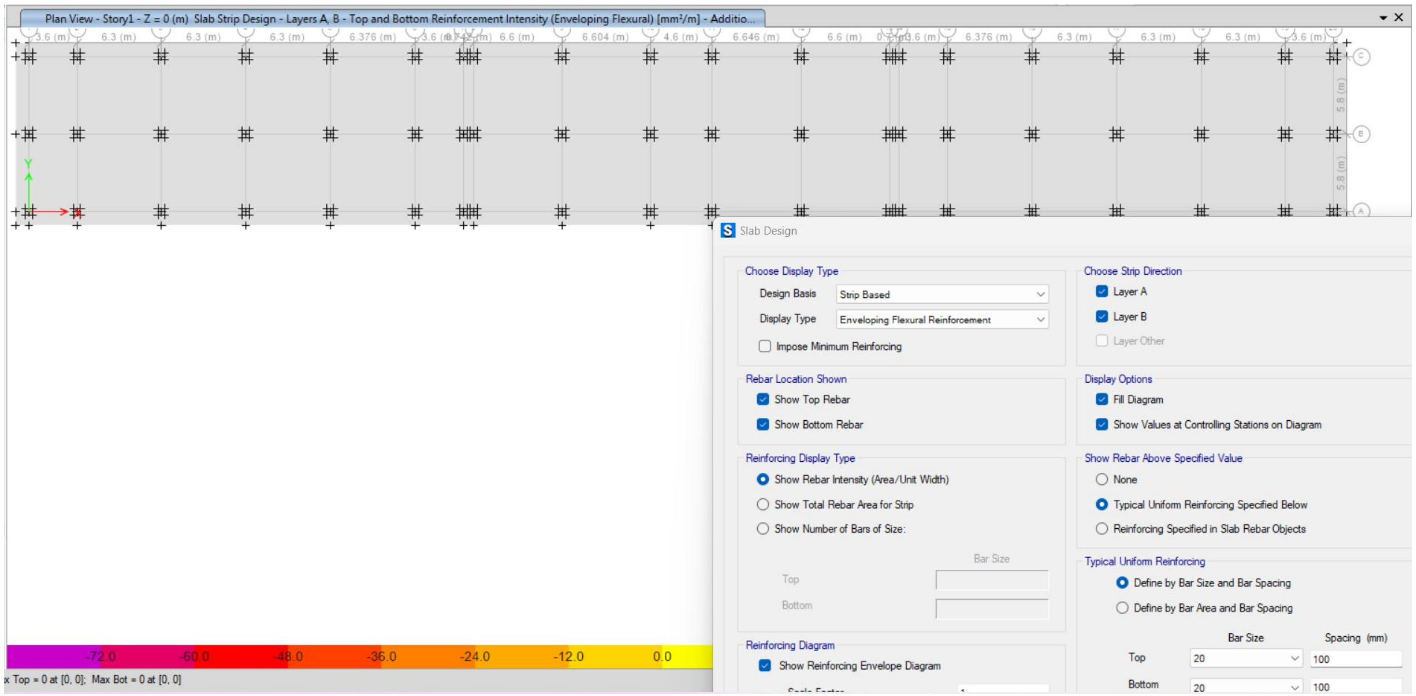
Block A Soil Pressure < 80KN/m3



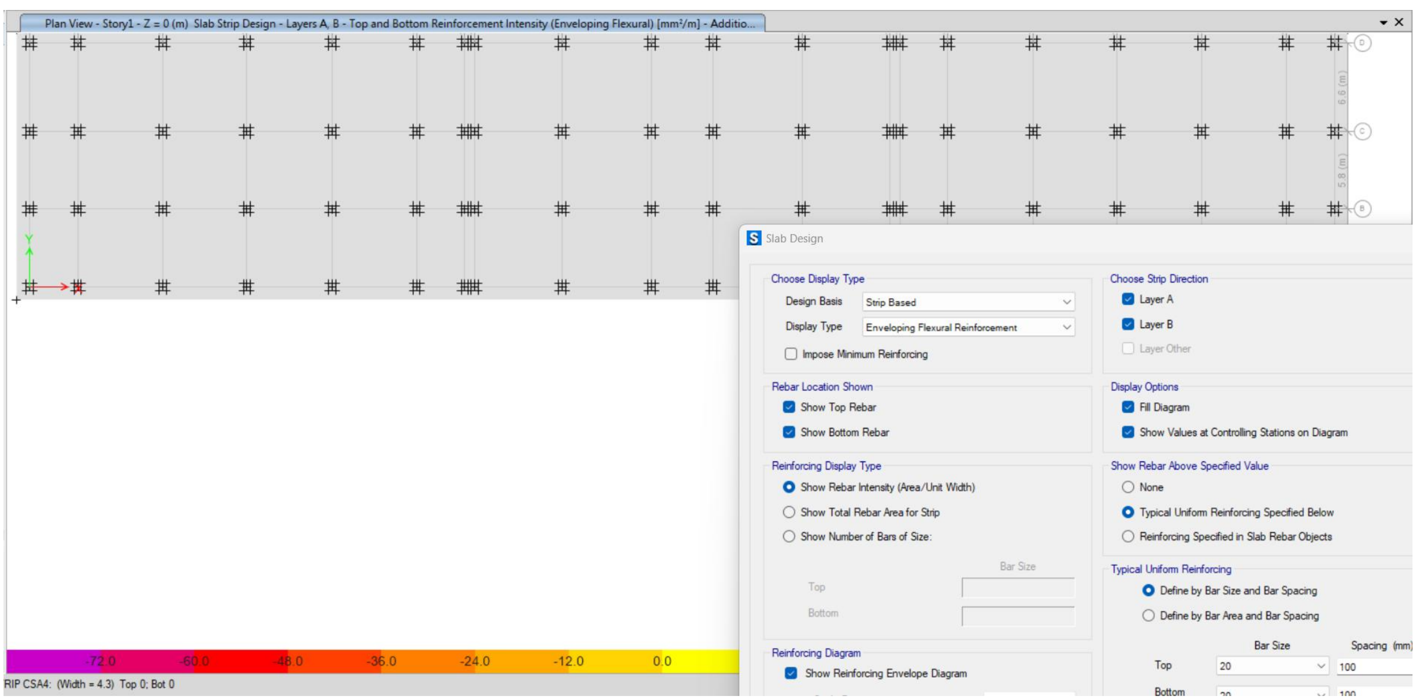
Block B Soil Pressure < 80KN/m3



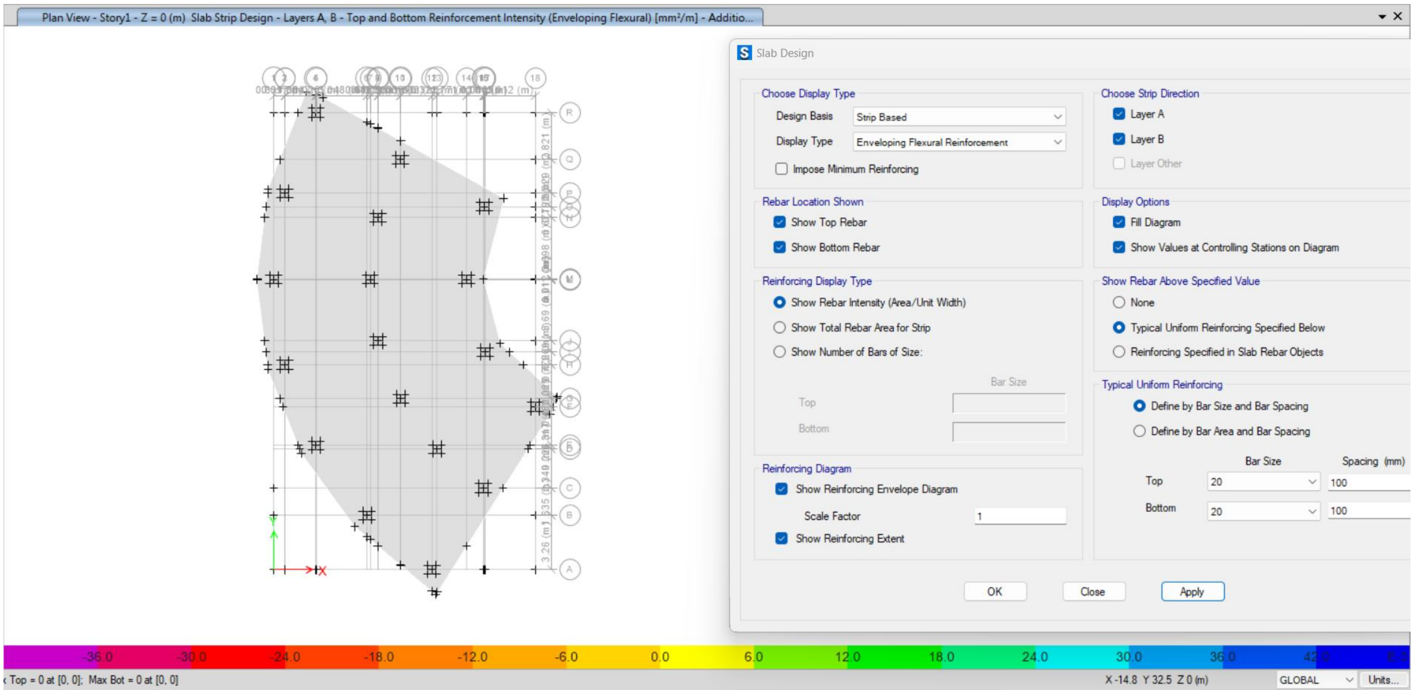
Block C Soil Pressure < 80KN/m3



Block A Raft Footing Slab Reinforcement



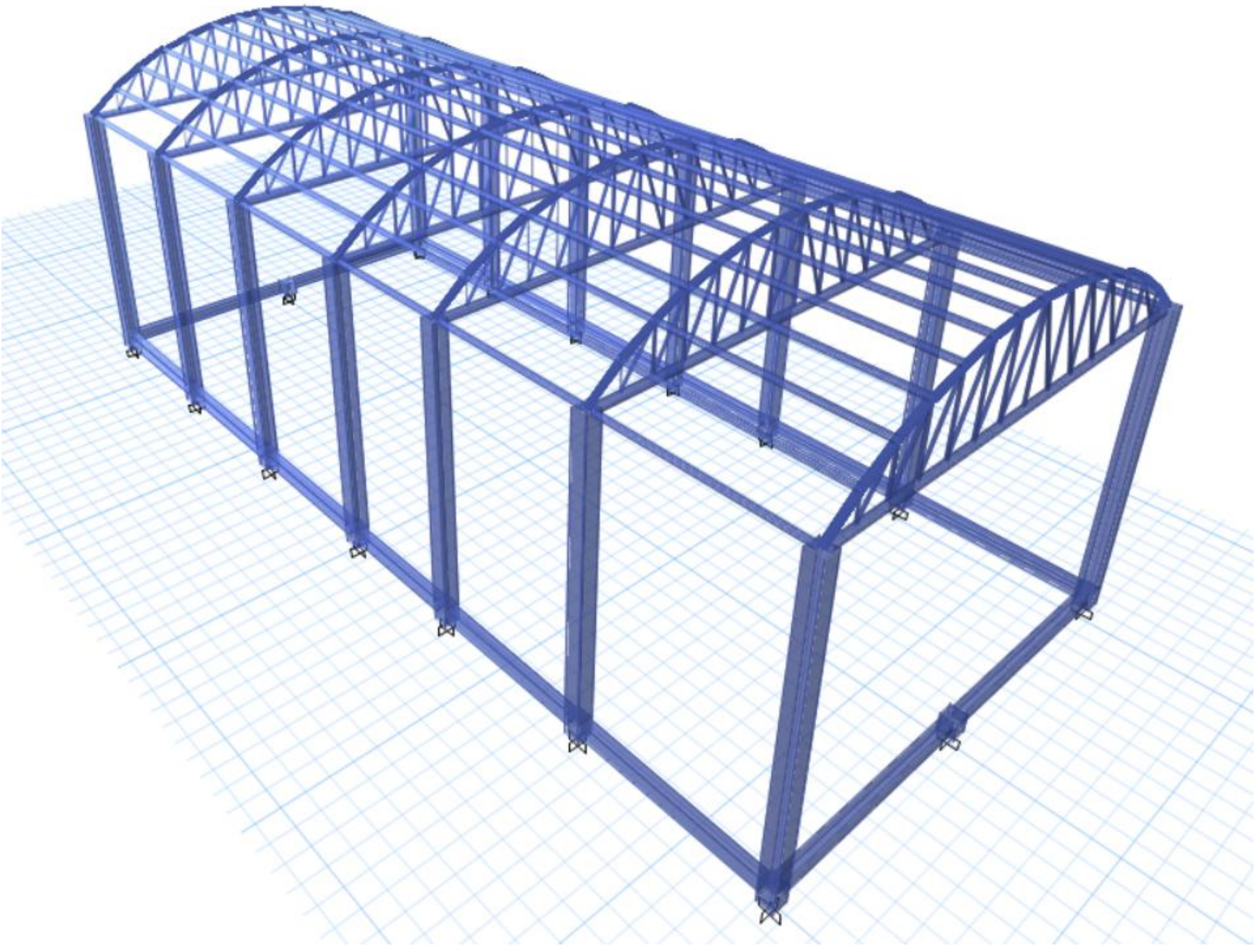
Block B Raft Footing Slab Reinforcement



Block C Raft Footing Slab Reinforcement

Raft Footing =20 mm dia @ 100 mm c/c All Direction top and bottom bar
Footing Depth = 750mm

STRUCTURAL ANALYSIS & DESIGN REPORT
OF
COVERED HALL



2023

Table of Contents:

1.0 Introduction

1.1 Executive Summary

This report has been prepared as a part of the structural engineering analysis and design of **Covered Hall Building**, as a partial requirement of application for permit to construct the building in **Suryabinayak Municipality, Bhaktapur**. This report describes in brief the Structural Aspects and Design Report of the proposed building. The analysis and design have been carried out using finite element software ETABS 2019. It provides the Structural Engineer with all the tools necessary to create, modify, analyze, design, and optimize the structural elements in a building model. The structure design is intended to be based primarily on the current National Building Code of Practice of Nepal taking account of relevant Indian Codes for the provisions not covered in this.

1.2 Structural Modeling

A three-dimensional mathematical model of the physical structure should be used that represents the spatial distribution of the mass and stiffness of the structure to an extent that is adequate for the calculation of the significant features of its dynamic response. Thus, the essential requirements of the model are that, it should include the sufficient detail in geometry, support, material, members, loading, strength, rigidity, stability etc. such that it reflects the real and true prototype of a physical structure. In modeling, for the vertical loading system, the deflection on the column in axial direction is so minimal that we can neglect it. It is because of high rigidity of column in axial direction whereas in horizontal loading system, the in-plane stiffness of floor is assumed to be very high compared to the stiffness of other frame members in that plane. It is because of the presence of floor slab. Since, floor slab has very high in-plane rigidity, the member like column, wall and braces connected to that plane are assumed to move as a single unit in the lateral direction. This system is known as rigid floor diaphragm in which beam is monolithically connected with slab providing negligible bending in the vertical plane.

For the modeling of this building, ETABS 2019software was used. ETABS 2019 is a sophisticated, yet easy to use, special purpose analysis and design program developed specifically for building systems. ETABS 2019 features an intuitive and powerful graphical interface coupled with unmatched modeling, analytical, design, detailing procedure, powerful numerical methods and many international design codes all integrated using a common database. Although quick and easy for simple structures ETABS 2019can also handle the largest and most complex building models, including the wide range of nonlinear behaviors, making it the tool of choice for structural engineers in the building industry. For the design of foundation, excel sheet has been used.

1.3 Structural System

This report deals with the methodology of the Structural Analysis and Design of Building. The Structure is RCC frame Structure and Foundation is **Isolated footing**.

2.0 Geometrical Configuration

a. Nos. of Storey	= G.F+1.+ Truss
b. Floor Height	= 1.95+6.7+2 M
c. No of Lifts	= 0
d. No. of Staircase	= 0
e. Total Height of Structure	= 10.65 m above the Plinth Beam

3.0 Basic Data

a. Density of Concrete = 20 kN/m^3
b. Live load = 0.23kN/m on end purlin and 0.46 kN/m on all Intermediate purlin as Roof live
c. CGI Load = 0.1 kN/m on end purlin and 0.05 kN/m on all intermediate purlin
d. Wind Load = 2.16 kN/m 0.1 kN/m on end purlin and 4.32 kN/m on all intermediate purlin
e. Snow Load = 2.05 kN/m on end purlin and 4.1 kN/m on all intermediate Purlin
f. Density of Brick = 19.2 kN/m^3
g. Soil Bearing Capacity = 131 kN/m^2

4.0 Relevant Code followed

Many international standard codes of practices were adopted for the creation of mathematical model, its analysis and design. As per the requirements, National Building Code of India was used for the load combination in order to check for the worse case during analysis.

Some of the codes used are enlisted below:

A. Loading

Code	Description
IS 875: 1987 Part I	Dead Loads
IS 875: 1987 Part II	Imposed Loads
IS 875: 1987 Part V	Special Loads and Combinations

B. Design of Earthquake Resistance

Code	Description
NBC 105:2020	Seismic Design Buildings in Nepal
IS 4326:2013	Code of practice for earthquake resistant design and construction of buildings

C. Design of Concrete Elements

Code	Description
IS 456:2000	Code of practice for plain and reinforced concrete (Reaffirmed in 2016)
IS 1786:2008	Specification for high strength deformed steel bars and wires for concrete reinforcement
SP-16	Design aids for reinforced concrete
SP-34	Handbook on concrete reinforcement and detailing

E. Design of Foundations

Code	Description
IS 1904	Indian Standard code of practice for design and construction of foundations in soil - General requirements
IS 2950	Indian Standard code of practice for design and construction of raft foundation (Part - I)
IS 2911	Indian Standard code of practice for design and construction of pile foundations
IS 2974	Code of practice for design and construction of machine foundation

F. Detailing of Structures

Code	Description
IS 13920:2016	Ductile Design and Detailing of Reinforced Concrete structures subjected to lateral forces (Reaffirmed in 2017)

The design was carried out using relevant Indian Codes of Practice. The earthquake loading was carried out using NBC 105:2020. The structure design of foundation, slabs, beams, columns, and tie beams was based on IS456:2000. Also, the system has been designed to meet the ductility requirements of IS 13920:2016 and ANNEX B of NBC 105:2020.

5.0 Basic Principal of Analysis of the structure Step for earthquake load

1. Earthquake load with seismic coefficient method (NBC 105: 2020)
2. Modal Response Spectrum Method (NBC 105: 2020)

6.0 Basic Principal of Design of Foundation

1. Isolated Footing

7.0 Software used for Analysis and Design

a. ETABS V 19.0.0
b. SAFE 2016
c. Excel Sheets

8.0 Concrete and Steel Grade

Concrete Grade = **M 20** for Columns and Beams
M 20 for Footing, Staircase and Slabs
Steel Grade = Fe 500

9.0 Preliminary Design for proportioning of the Structural Elements

The tentative sizes of the Structural elements are determined through preliminary design so that after analysis, the presumed dimensions may not deviate considerably, thus making the final design both safe and economical. The tentative sizes have been determined as follows:
Slab: From Deflection Criteria [Effective Depth = Effective Shorter Span / (26 * MF)]

Beam: For Practical rule as 25mm Depth for 300mm of Span covering deflection Criteria

Column: From evaluation of approximate gravity loading coming up to the critical Column. To compensate for the possible eccentric Loading and earthquake loads the size is increased by about 25 % in design.

10.0 Load on Structures

The following Loads were assumed to occur in Structural System.

a. Dead Load
b. Live Load
c. Seismic Load
d. Wind Load
e. Snow Load

a) Dead Load:

Dead Load on the structure comprise the self-weight of the member; weight of the finishes and partition walls. The Wall Load is taken for thickness of either 230 mm or 115 mm as per Architectural Drawing and suitable reduction is made for Window and Door Opening.

Dead loads are as per IS 875: 1987 Part I

As the software, we have used, generates the self-weight of the Structural member by itself, we have not calculated the self-weight.

b) Live Loads:

Live loads are as per IS 875: 1987 Part II

c) Seismic Load:

For Earthquake Load, 100% of Dead Load and i) 30% of Live Load for other usage and ii) 60% of Live load for storage facilities are taken into account.

Seismic Coefficient method & Response spectrum method using NBC 105: 2020 are applied for Earthquake Analysis of the Structure and the Parameters taken are:

Importance Factor, I	=	1
Zone factor, Z	=	0.30

Soil Type	=	B
Response Reduction Factor	=	Rμ = 4, Ωu =1.5 and Ωs = 1.25
Damping	=	0.05

d) Wind Load:

Wind loads are as per IS 875: 1987 Part III

e) Snow Load:

Snow loads are as per IS 875: 1987 Part IV

11.0 Design Methods of Structural Elements

We have followed Indian Standard Code of Practice for Plain and Reinforced Concrete, IS: 456 -2000 for design of Structural Elements. This incorporates the two methods of Structural Design of RC structures specified as:

a. *Working Stress Method* based on the Working loads in conjunction with permissible stresses in the materials.
b. *Limit State Method* based on safety and serviceability requirements associated with the design loads and design strengths of the materials. These design loads and design strengths are obtained by applying partial safety factors for characteristic loads and strengths of the materials concrete and steel.

We have followed the limit state method which is incorporated in IS:456-2000. It is consistent with the new philosophy of design termed *limit state approach* which was incorporated in the Russian Code – 1954, the British code BS 8110 – 1985 and the American Code ACI 318 – 1989.

12.0 Limit State Method

- **Limit States**
  The Limit State method of design covers the various forms of failure. There are several limit states at which the structure ceases to function, the most important among them being,

a. The limit state of collapse or total failure of structure.
   It corresponds to the maximum load carrying capacity. Violation of collapse implies failure. This limit state corresponds to Flexure, Compression, Shear and Torsion.

b. The limit state of serviceability which includes excessive deflection and excessive local damage.

- **Load Combinations in Limit State Method**

Various Load Combinations are done for critical conditions.

As per clause 3.6.1 of NBC 105:2020,
When seismic load effect is combined with other load effects, the following load combination shall be adopted.

1.2DL + 1.5LL
DL + λLL +- E

Where, λ = 0.6 for storage facilities
= 0.3 for other usage

- **Analysis and Design of the Structural Elements:**

The Structure is analyzed and designed by standard software ETABS 2019.

13.0 Detailing of the Structural Elements

The Reinforcement detailing of most of the important structural components have been shown in drawing which is based on NBC 105:2020. The provisions not covered in NBC 105:2020 confirm with the relevant sections of the IS Codes IS 456-2000, IS 1893-2016, IS 13920:2016 For Ductile Detailing, SP-16, and SP –34.

14.0 Calculation & Output details

14.1 Static Analysis (Seismic Coefficient Method)

Clause 3.2.1 of NBC 105:2020 contains provisions for Static analysis using equivalent lateral force procedure. Equivalent Static Method may be used for all serviceability limit state (SLS) calculations regardless of building characteristics. For ultimate limit state (ULS), the Equivalent Static Method may be used when at least one of the following criteria is satisfied:

i) The height of the structure is less than or equal to 15m.
ii) The natural time period of the structure is less than 0.5 secs.
iii) The structure is not categorized as irregular as per 5.5 and the height is less than 40m.

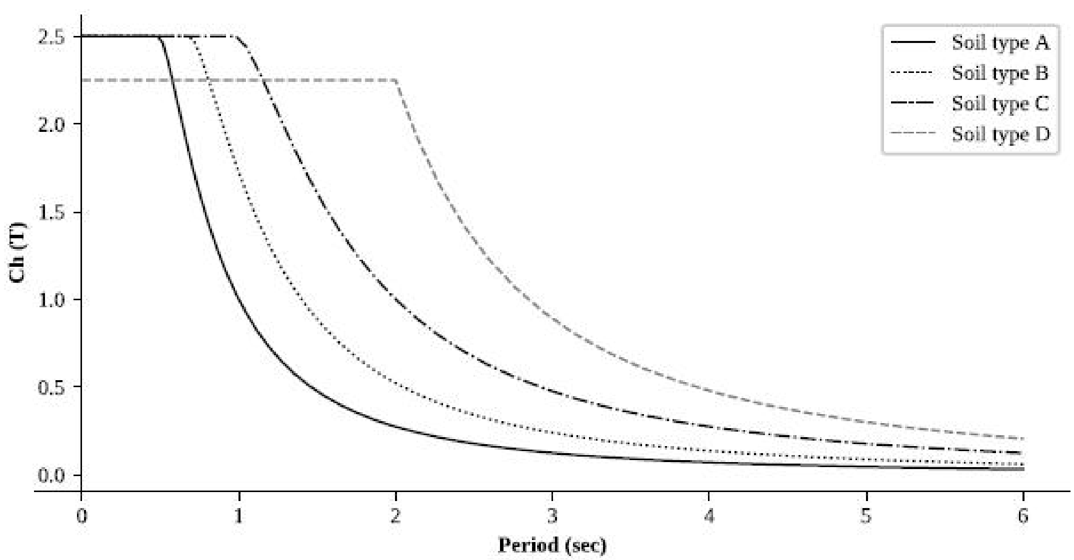


Fig : Spectral Shape Factor, Ch(T) for Equivalent Static Method
(Figure 4-1 from NBC 105:2020)

HORIZONTAL BASE SHEAR COEFFICIENT

Ultimate Limit State

For the ultimate limit state, the horizontal base shear coefficient (design coefficient), Cd (T1), shall be given by:

$Cd\,(T1) = C\,(T1) / (R\mu \times \Omega u)$

Where,

C (T1) = Elastic Site Spectra as per 4.1.1

Rμ = Ductility Factor as per 5.3 = 4

Ωu = Over strength Factor for ULS as per 5.4 = 1.5

Serviceability Limit State

For the serviceability limit state, the horizontal base shear coefficient (design coefficient), Cd (T1), shall be given by:

$Cd\,(T1) = Cs\,(T1) / (\Omega s)$

Where,

Cs (T1) = Elastic Site Spectra determined for Serviceability Limit State as per 4.2

Ωs = Over strength Factor for SLS as per 5.4 = 1.25

The seismic weight at each level, Wi, shall be taken as the sum of the loads and the factored seismic live loads between the mid-height adjacent stories.

Table: Live Load Categories and Factors (NBC 105:2020)

Live Load Category	Factor (λ)
Storage	0.6
For other purpose	0.3
Roof	Nil

Periods of Vibration

As per IS NBC 105:2020, Clause 5.1 the periods of vibration, Ti, shall be established from properly substantiated data, or computation, or both. The fundamental translation period shall be determined using following methods:

- Rayleigh Method
- Empirical Equations

The fundamental translation period of a building shall be estimated using the appropriate empirical equations listed in section 5.1.2. The approximate time period calculated in section 5.1.2 shall be modified as per section 5.1.3. The time period so modified shall be compared with the translation period computed from section 5.1.1 and the lesser value of the two shall be adopted for determining the design action.

Rayleigh Method

The fundamental translation period in the direction under consideration, T1, shall be calculated as:

$$T_1 = 2\pi\sqrt{\frac{\sum_{i=1}^{n}(W_i d_i^2)}{g\sum_{i=1}^{n}(F_i d_i)}}$$

Where, di = elastic horizontal displacement of center of mass at level i

Fi = lateral force acting at level i

g = acceleration due to gravity

i = level under consideration

n = number of levels in the structure

Wi = seismic weight at level i

Table: Calculation of Time period for EQX ULS

for Eqx ULS							
SN	STORY	HEIGHTH	Fi(KN)	di(mm)	Wi(KN)	Wi di^2	Fi di
1	Story3	10.65	84.2169	32.564	7.177192	7610.796	2742.439
2	Story2	8.65	70.9266	32.218	142.8877	148317.4	2285.113
3	Story1	1.95	4.4786	1.948	873.5033	3314.686	8.724313
4	Base	0	0	0	62.40288	0	0
	Total		159.6221	66.73	1085.971	159242.8	5036.277
						T	0.356714

Table: Calculation of Time period for EQY ULS

For EQy ULS							
SN	STORY	HEIGHT	Fi(KN)	di(mm)	Wi(KN)	Wi di^2	Fi di
1	Story3	10.65	84.2169	46.342	7.177192	15413.6	3902.78
2	Story2	8.65	70.9266	46.968	142.8877	315209.3	3331.281
3	Story1	1.95	4.4786	1.438	873.5033	1806.269	6.440227
4	Base	0	0	0	62.40288	0	0
	total		159.6221	94.748	1085.971	332429.1	7240.5
						T	0.429844

Table: Calculation of Time period for EQX SLS

For Eqx SLS							
SN	STORY	HEIGHT	Fi(KN)	di(mm)	Wi(KN)	Wi di^2	Fi di
1	Story3	10.65	80.9778	31.311	7.177192	7036.366	2535.496
2	Story2	8.65	68.1987	30.979	142.8877	137129.1	2112.728
3	Story1	1.95	4.3063	1.873	873.5033	3064.362	8.0657
4	Base	0	0	0	62.40288	0	0
	Total		153.4828	64.163	1085.971	147229.8	4656.289
						T	0.356717

Table: Calculation of Time period for EQY SLS

For Eqy SLS							
SN	STORY	HEIGHT	Fi(KN)	di(mm)	Wi(KN)	Wi di^2	Fi di
1	Story3	10.65	80.9778	44.56	7.177192	14250.99	3608.371
2	Story2	8.65	68.1987	45.162	142.8877	291434.6	3079.99
3	Story1	1.95	4.3063	1.382	873.5033	1668.325	5.951307
4	Base	0	0	0	62.40288	0	0
	total		153.4828	91.104	1085.971	307354	6694.312
						T	0.429845

Empirical Equations

The approximate fundamental period of vibration, T_1, in seconds is determined from following empirical equation:

$T_1 = kt\,H^{0.75}$

Where, kt = 0.075 for Moment resisting concrete frame

= 0.085 for Moment resisting structural steel frame

= 0.075 for eccentrically braced structural steel frame

= 0.05 for all other structural systems

Where, H = Height of the building from foundation or from top of a rigid basement.

Amplification of Approximate Period:

The approximate fundamental time period calculated using empirical equation in section 5.1.2 shall be increased by a factor of 1.25.

The lateral seismic force (Fi) induced at each level 'i' shall be calculated as:

$$Fi = \frac{W_i h_i^k}{\sum_i^n W_i h_i^k} * V$$

Where,

Wi = seismic weight of the structure assigned to level 'i';

hi= height (m) from the base to level ‘i’;
n= total number of floors/levels
V= horizontal seismic base shear calculated as per Clause 6.2 of NBC 105:2020

k= an exponent related to the structural period as follows:

- for structure having time period T≤0.5sec, k=1
- for structure having time period T≥2.5sec, k=2
- for structure having period between 0.5 sec and 2.5 sec, k shall be determined by linear interpolation between 1 and 2.

from releigh method	
summary of time period	
Eqx ULS	0.356714318
EQX SLS	0.356716586
EQY ULS	0.429843751
EQY SLS	0.429844923
FROM EMPERICAL METHOD	
Summary of time period	
T=	0.626384705

14.2 Storey Drift

Story Response - Maximum Story Drifts

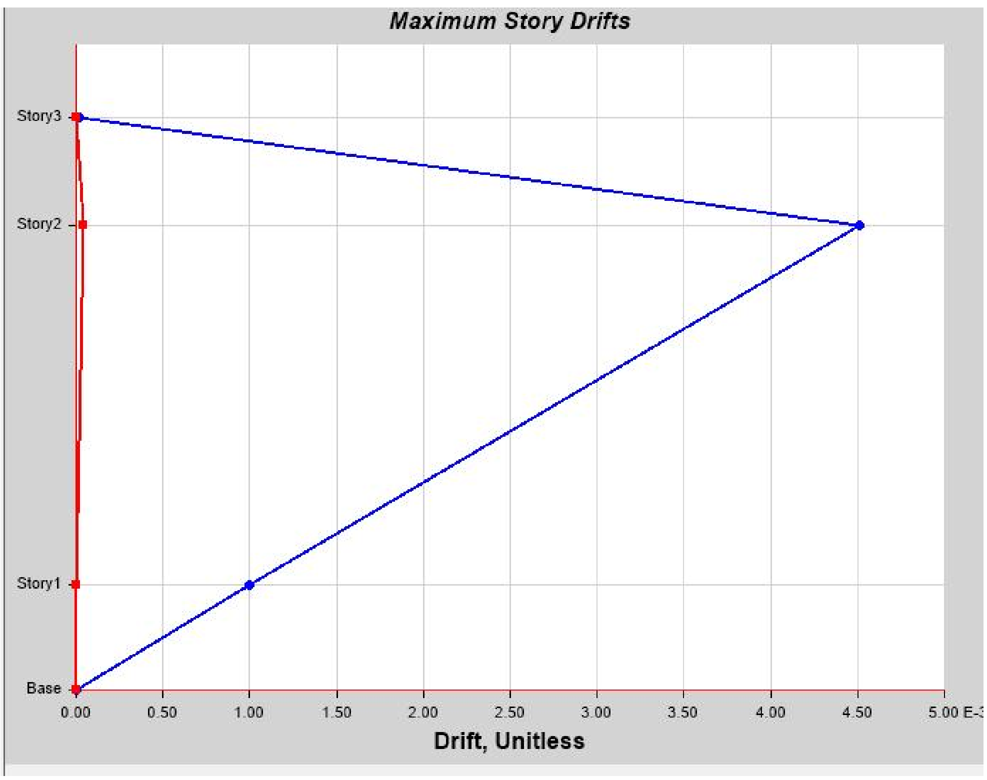
Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

Input Data

Name	StoryResp1		
Display Type	Max story drifts	Story Range	All Stories
Load Case	EQX ULS	Top Story	Story3
Output Type	Step Number 1	Bottom Story	Base

Plot



Tabulated Plot Coordinates

Story Response Values

Story	Elevation m	Location	X-Dir	Y-Dir
Base	0	Top	0	0
Story1	1.95	Top	0.000999	4E-06
Story2	8.65	Top	0.004508	3.9E-05
Story3	10.65	Top	1.8E-05	0

Maximum Drift in ULS EQX = 0.005038
Design Drift = 0.018032< 0.025, Hence Safe.

Story Response - Maximum Story Drifts

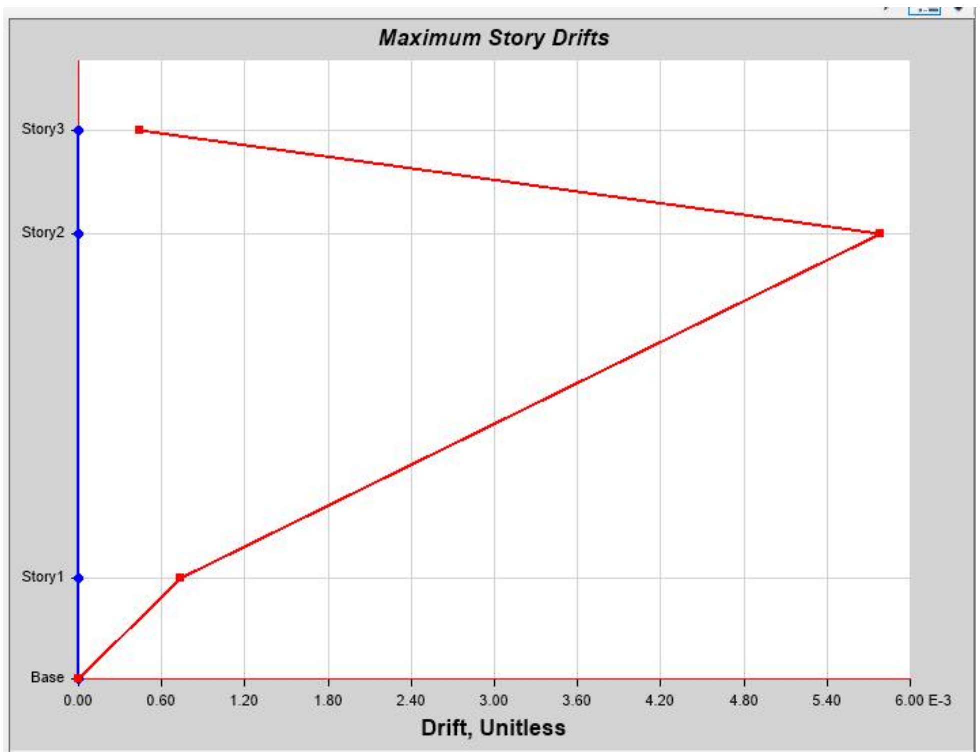
Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

Input Data

Name	StoryResp1		
Display Type	Max story drifts	Story Range	All Stories
Load Case	EQY ULS	Top Story	Story3
Output Type	Step Number 1	Bottom Story	Base

Plot



Tabulated Plot Coordinates

Story Response Values

Story	Elevation m	Location	X-Dir	Y-Dir
Story3	10.65	Top	0	0.000438
Story2	8.65	Top	3E-06	0.005785
Story1	1.95	Top	1E-06	0.000737
Base	0	Top	0	0

Maximum Drift in ULS EQY = 0.005785

Design Drift = 0.02314< 0.025, Hence Safe.

Story Response - Maximum Story Drifts

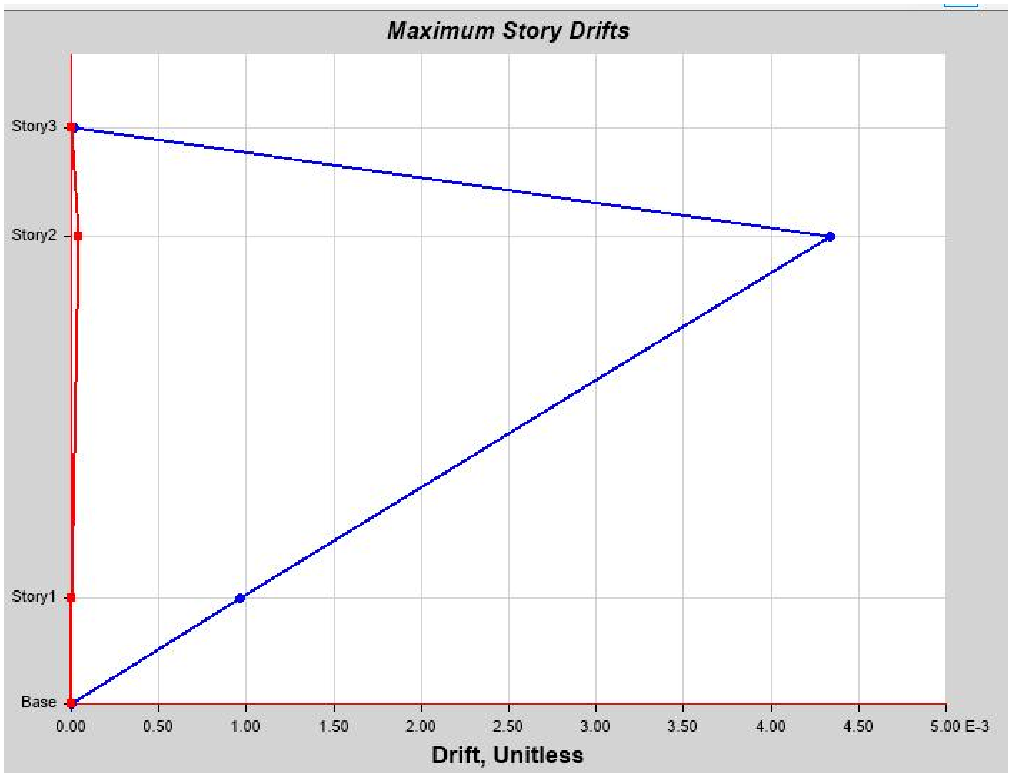
Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

Input Data

Name	StoryResp1		
Display Type	Max story drifts	Story Range	All Stories
Load Case	EQX SLS	Top Story	Story3
Output Type	Step Number 1	Bottom Story	Base

Plot



Tabulated Plot Coordinates

Story Response Values

	Story	Elevation m	Location	X-Dir	Y-Dir
▶	Story3	10.65	Top	1.7E-05	0
	Story2	8.65	Top	0.004335	3.8E-05
	Story1	1.95	Top	0.00096	4E-06
	Base	0	Top	0	0

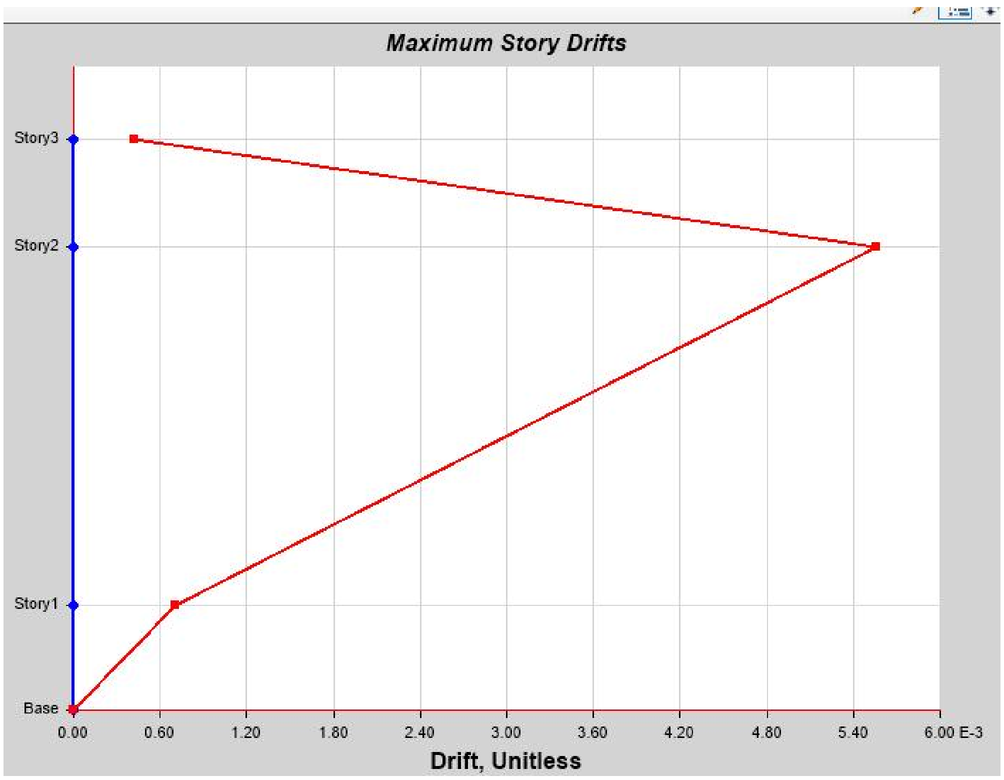
Maximum Drift in SLS EQX = 0.00435
Design Drift = 0.00435< 0.006, Hence Safe

Story Response - Maximum Story Drifts

Summary Description
This is story response output for a specified range of stories and a selected load case or load combination.
Input Data

Name	StoryResp1		
Display Type	Max story drifts	Story Range	All Stories
Load Case	EQY SLS	Top Story	Story3
Output Type	Step Number 1	Bottom Story	Base

Plot



Tabulated Plot Coordinates
Story Response Values

Story	Elevation m	Location	X-Dir	Y-Dir
Story3	10.65	Top	0	0.000421
Story2	8.65	Top	3E-06	0.005563
Story1	1.95	Top	1E-06	0.000709
Base	0	Top	0	0

Maximum Drift in SLS EQY = 0.005563
Design Drift = 0.005563< 0.006, Hence Safe.

14.3 Storey Displacement

Story Response - Maximum Story Displacement

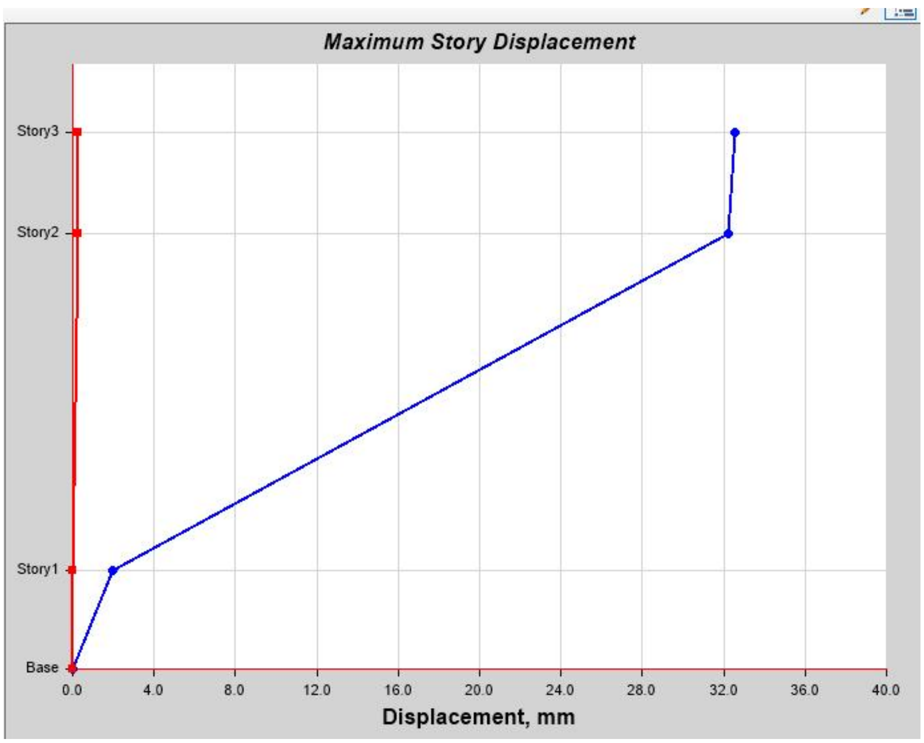
Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

Input Data

Name	StoryResp1		
Display Type	Max story displ	Story Range	All Stories
Load Case	EQX ULS	Top Story	Story3
Output Type	Step Number 1	Bottom Story	Base

Plot



Tabulated Plot Coordinates

Story Response Values

Story	Elevation m	Location	X-Dir mm	Y-Dir mm
Story3	10.65	Top	32.564	0.242
Story2	8.65	Top	32.218	0.293
Story1	1.95	Top	1.948	0.008
Base	0	Top	0	0

Maximum Displacement in ULS EQX = 32.563696 mm. <66.5625mm. Hence safe

Story Response - Maximum Story Displacement

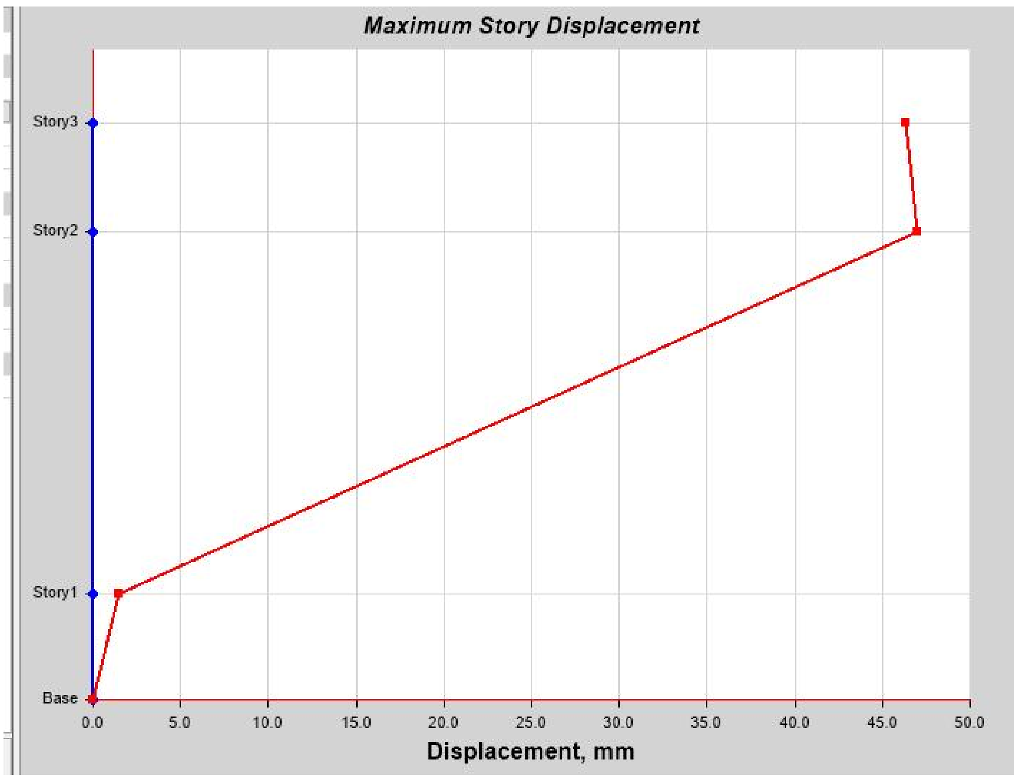
Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

Input Data

Name	StoryResp1		
Display Type	Max story displ	Story Range	All Stories
Load Case	EQY ULS	Top Story	Story3
Output Type	Step Number 1	Bottom Story	Base

Plot



Tabulated Plot Coordinates

Story Response Values

Story	Elevation m	Location	X-Dir mm	Y-Dir mm
Story3	10.65	Top	0.005	46.342
Story2	8.65	Top	0.018	46.968
Story1	1.95	Top	0.002	1.438
Base	0	Top	0	0

Maximum Displacement in ULS EQY = 46.968197 mm. <66.5625mm. Hence safe

Story Response - Maximum Story Displacement

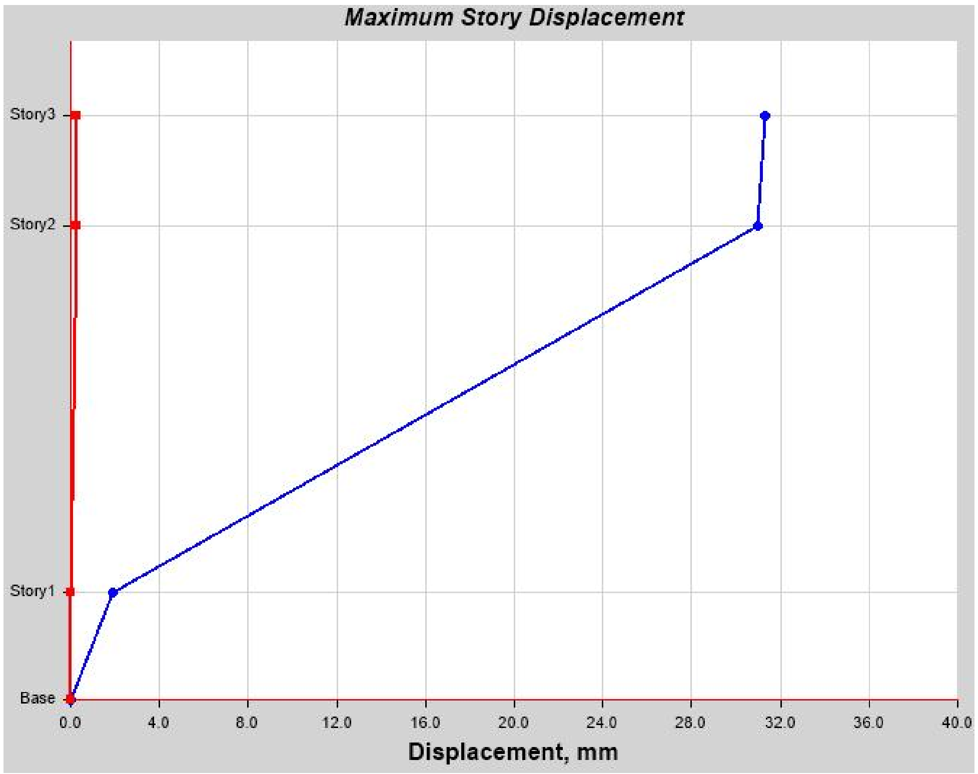
Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

Input Data

Name	StoryResp1		
Display Type	Max story displ	Story Range	All Stories
Load Case	EQX SLS	Top Story	Story3
Output Type	Step Number 1	Bottom Story	Base

Plot



Tabulated Plot Coordinates

Story Response Values

	Story	Elevation m	Location	X-Dir mm	Y-Dir mm
▶	Story3	10.65	Top	31.311	0.232
	Story2	8.65	Top	30.979	0.282
	Story1	1.95	Top	1.873	0.007
	Base	0	Top	0	0

Maximum Displacement in SLS EQX = 32.563696 mm. <66.5625mm. Hence safe

Story Response - Maximum Story Displacement

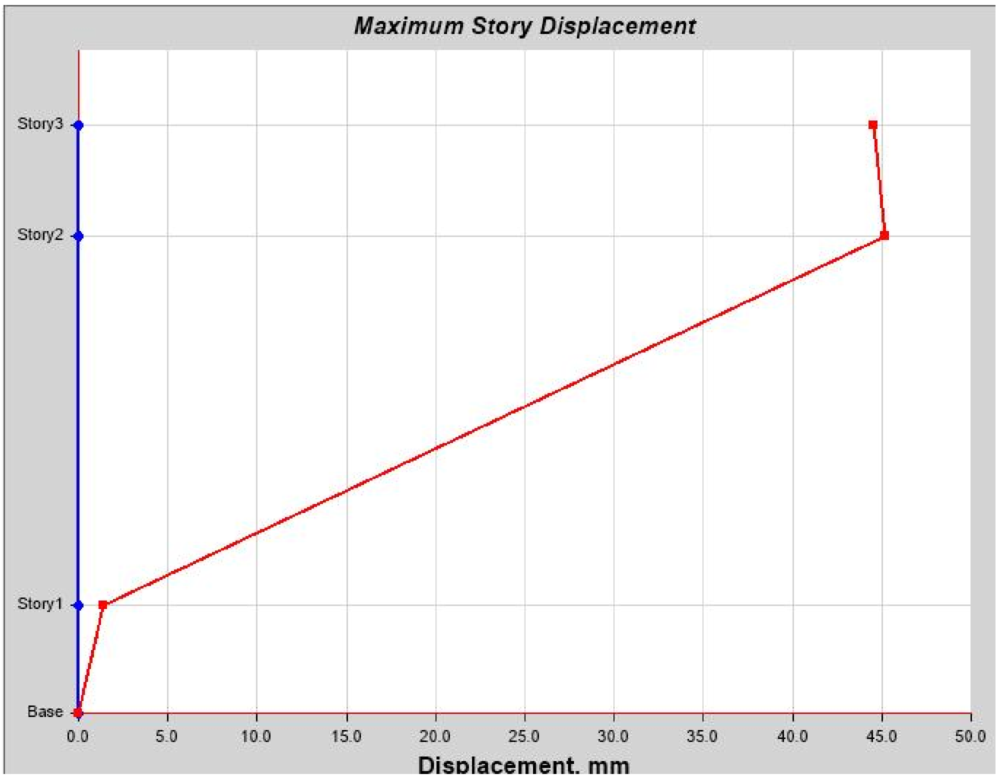
Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

Input Data

Name	StoryResp1		
Display Type	Max story displ	Story Range	All Stories
Load Case	EQY SLS	Top Story	Story3
Output Type	Step Number 1	Bottom Story	Base

Plot



Tabulated Plot Coordinates

Story Response Values

Story	Elevation m	Location	X-Dir mm	Y-Dir mm
Story3	10.65	Top	0.005	44.56
Story2	8.65	Top	0.018	45.162
Story1	1.95	Top	0.002	1.382
Base	0	Top	0	0

Maximum Displacement in SLS EQY = 45.161727 mm. <66.5625mm. Hence safe

15.0 Design Results

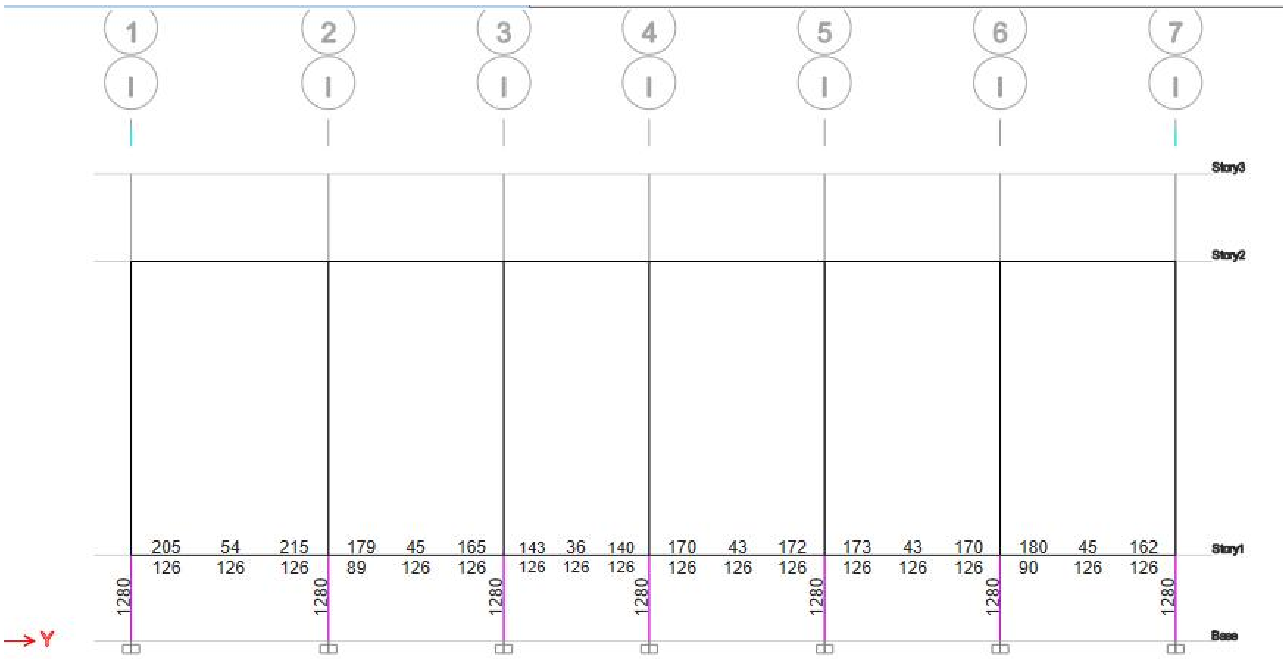


Fig: Reinforcement details along Grid I-I

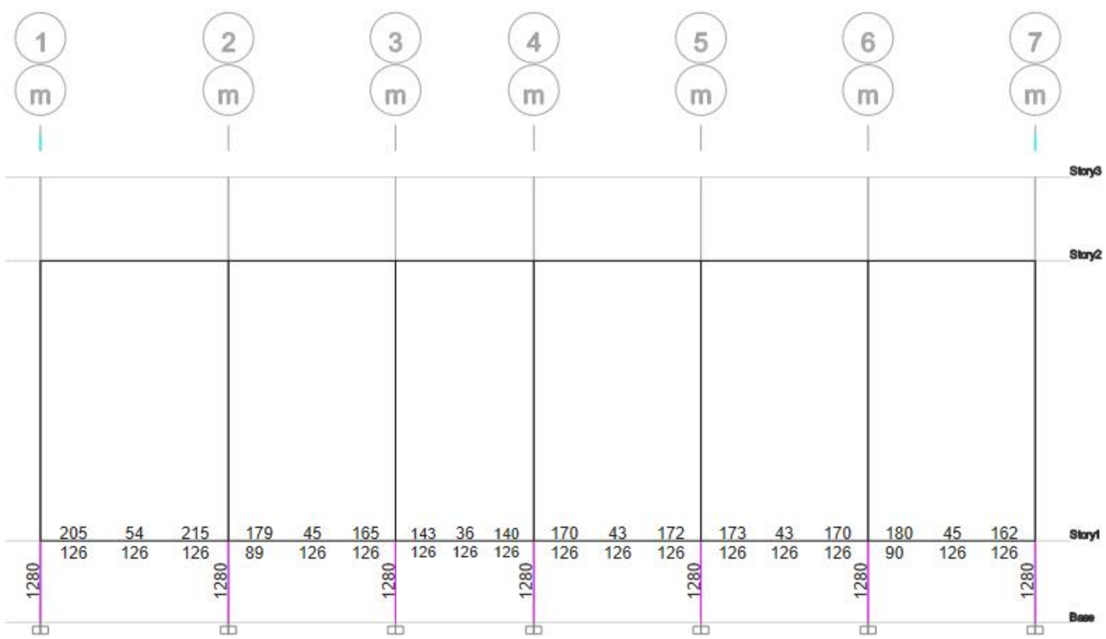


Fig: Reinforcement details along Grid M-M

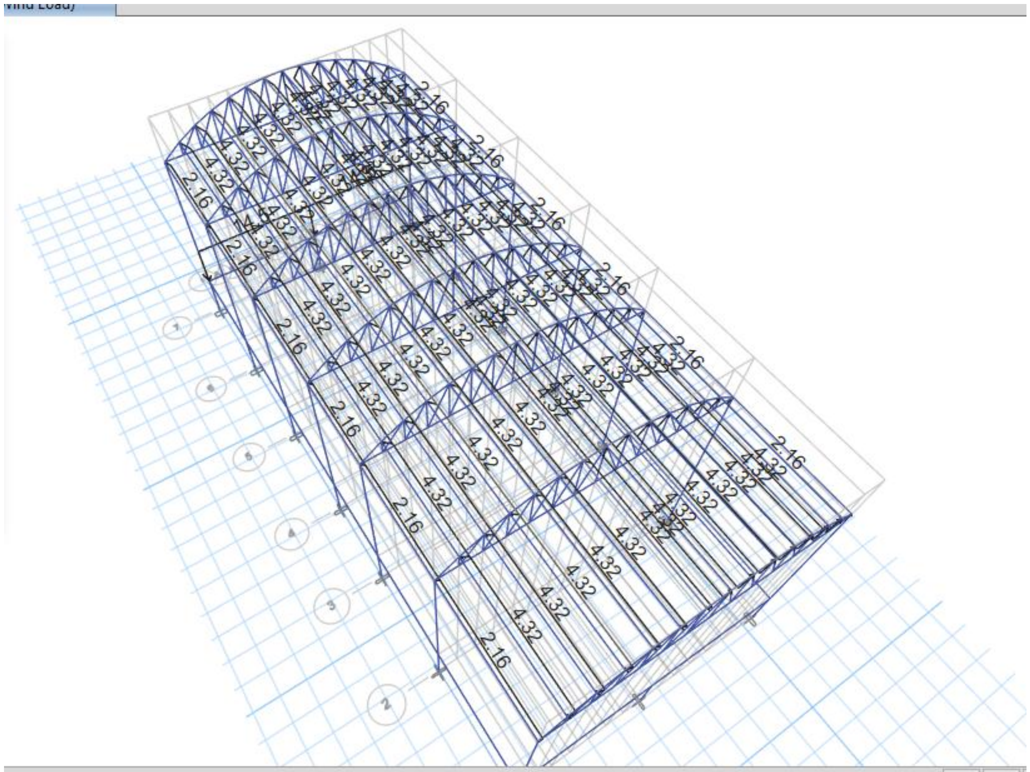


Fig: WIND Load Assign Details

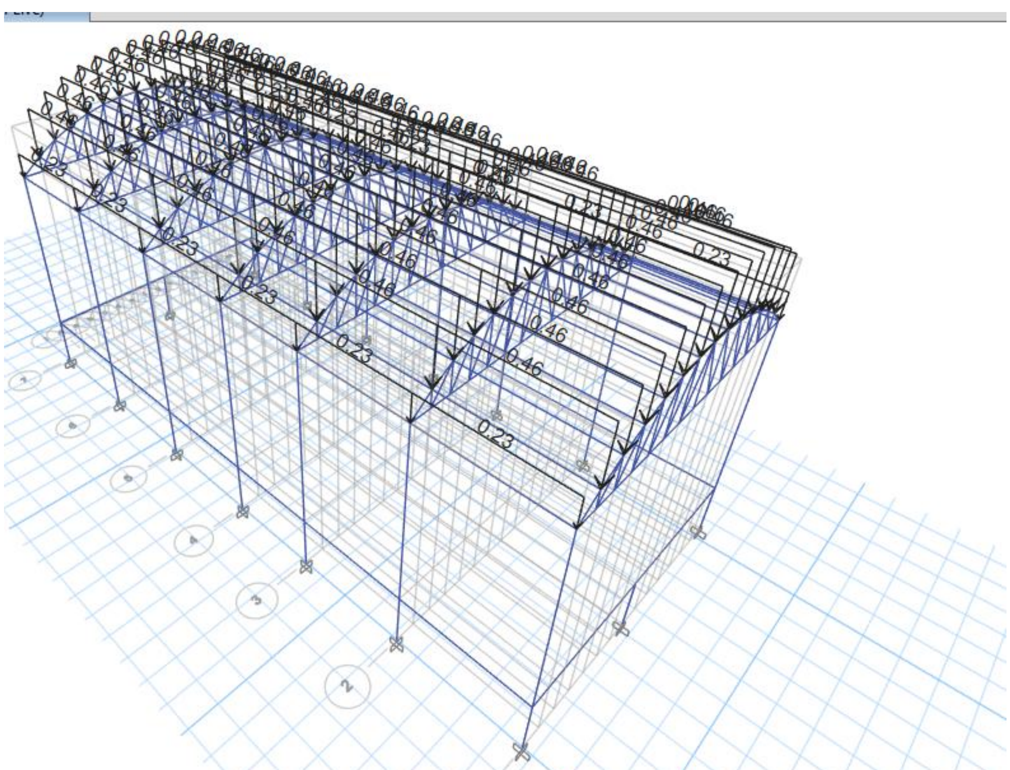


Fig: Roof Live Load Assign Details

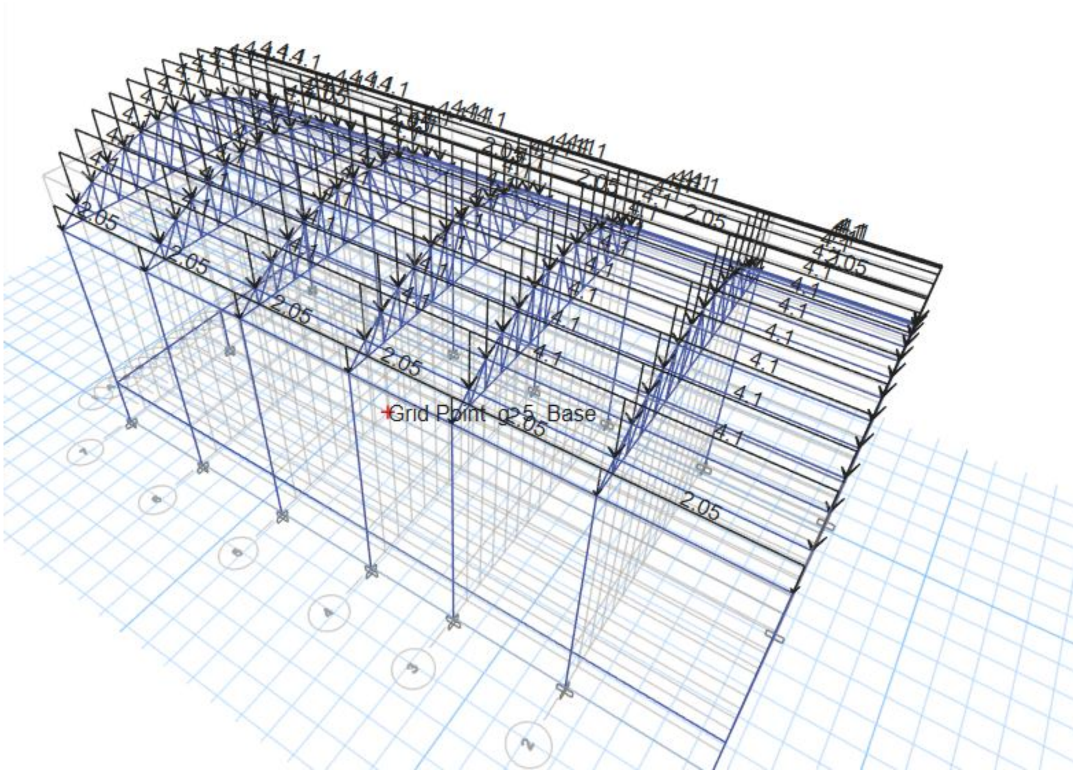


Fig: Snow Load Assign Details

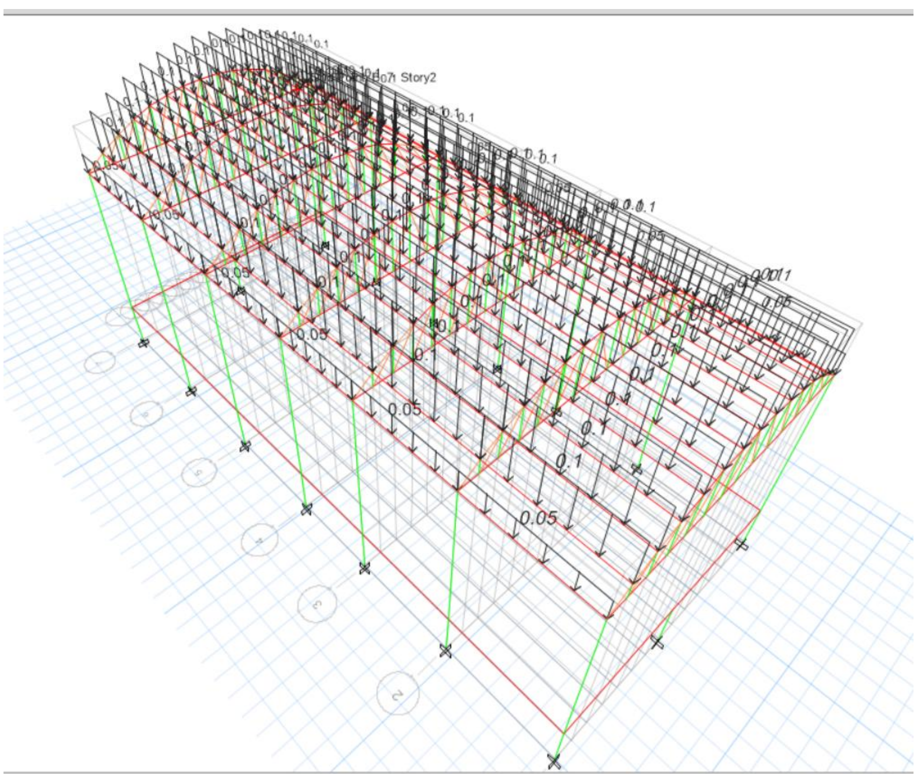
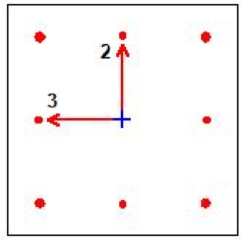


Fig: CGI Load Assign Details

15.1 Column Design Summary

ETABS Concrete Frame Design



Column Element Details Type: Ductile Frame (Summary)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	LLRF
Story1	C12	21	C400*400	DL+0.3LL-EQx ULS	0	1950	1

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
400	400	56	30

Material Properties

E_c (MPa)	f_{ck} (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
22360.68	20	1	500	500

Design Code Parameters

γ_C	γ_S
1.5	1.15

Axial Force and Biaxial Moment Design For P_u , M_{u2} , M_{u3}

Design P_u kN	Design M_{u2} kN-m	Design M_{u3} kN-m	Minimum M_2 kN-m	Minimum M_3 kN-m	Rebar Area mm²	Rebar % %
87.2611	3.0788	-44.278	1.7452	2.6393	1280	0.8

Axial Force and Biaxial Moment Factors

	K Factor Unitless	Length mm	Initial Moment kN-m	Additional Moment kN-m	Minimum Moment kN-m
Major Bend(M3)	0.642656	8456.3	-33.4337	3.2214	2.6393
Minor Bend(M2)	0.672116	1650	-1.3415	0	1.7452

Shear Design for V_{u2} , V_{u3}

	Shear V_u kN	Shear V_c kN	Shear V_s kN	Shear V_p kN	Rebar A_{sv} /s mm²/m
Major, V_{u2}	11.5498	68.9584	55.0394	0	443.37
Minor, V_{u3}	10.3508	68.9584	55.0394	10.3508	443.37

Joint Shear Check/Design

	Joint Shear Force kN	Shear V_{Top} kN	Shear $V_{u,Tot}$ kN	Shear V_c kN	Joint Area cm²	Shear Ratio Unitless
Major Shear, V_{u2}	N/A	N/A	N/A	N/A	N/A	N/A
Minor Shear, V_{u3}	N/A	N/A	N/A	N/A	N/A	N/A

(1.4) Beam/Column Capacity Ratio

Major Ratio	Minor Ratio
N/A	N/A

Additional Moment Reduction Factor k (IS 39.7.1.1)

A_g cm²	A_{sc} cm²	P_{uz} kN	P_b kN	P_u kN	k Unitless
1600	12.8	1920	605.9329	87.2611	1

Additional Moment (IS 39.7.1)

	Consider M_a	Length Factor	Section Depth (mm)	KL/Depth Ratio	KL/Depth Limit	KL/Depth Exceeded	M_a Moment (kN-m)
Major Bending (M_3)	Yes	4.337	400	13.586	12	Yes	3.2214
Minor Bending (M_2)	Yes	0.846	400	2.772	12	No	0

Notes:

N/A: Not Applicable

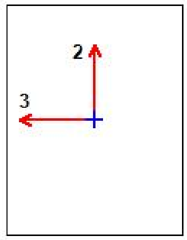
N/C: Not Calculated

N/N: Not Needed

15.2 Beam Design Summary

ETABS Concrete Frame Design

IS 456:2000 + IS 13920:2016 Beam Section Design



Beam Element Details Type: Ductile Frame (Summary)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	LLRF
Story1	B4	32	B230*300	DL+0.3LL-EQy ULS	200	3310	1

Section Properties

b (mm)	h (mm)	b_f (mm)	d_s (mm)	d_{ct} (mm)	d_{cb} (mm)
230	300	230	0	45	45

Material Properties

E_c (MPa)	f_{ck} (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
22360.68	20	1	500	500

Design Code Parameters

γ_C	γ_S
1.5	1.15

Factored Forces and Moments

Factored M_{u3} kN-m	Factored T_u kN-m	Factored V_{u2} kN	Factored P_u kN
-15.6266	0.001	24.4846	3.3123

Design Moments, M_{u3} & M_t

Factored Moment kN-m	Factored M_t kN-m	Positive Moment kN-m	Negative Moment kN-m
-15.6266	0.0013	0	-15.6279

Design Moment and Flexural Reinforcement for Moment, M_{u3} & T_u

	Design -Moment kN-m	Design +Moment kN-m	-Moment Rebar mm²	+Moment Rebar mm²	Minimum Rebar mm²	Required Rebar mm²
Top (+2 Axis)	-15.6279		143	0	143	126
Bottom (-2 Axis)		0	72	0	0	72

Shear Force and Reinforcement for Shear, V_{u2} & T_u

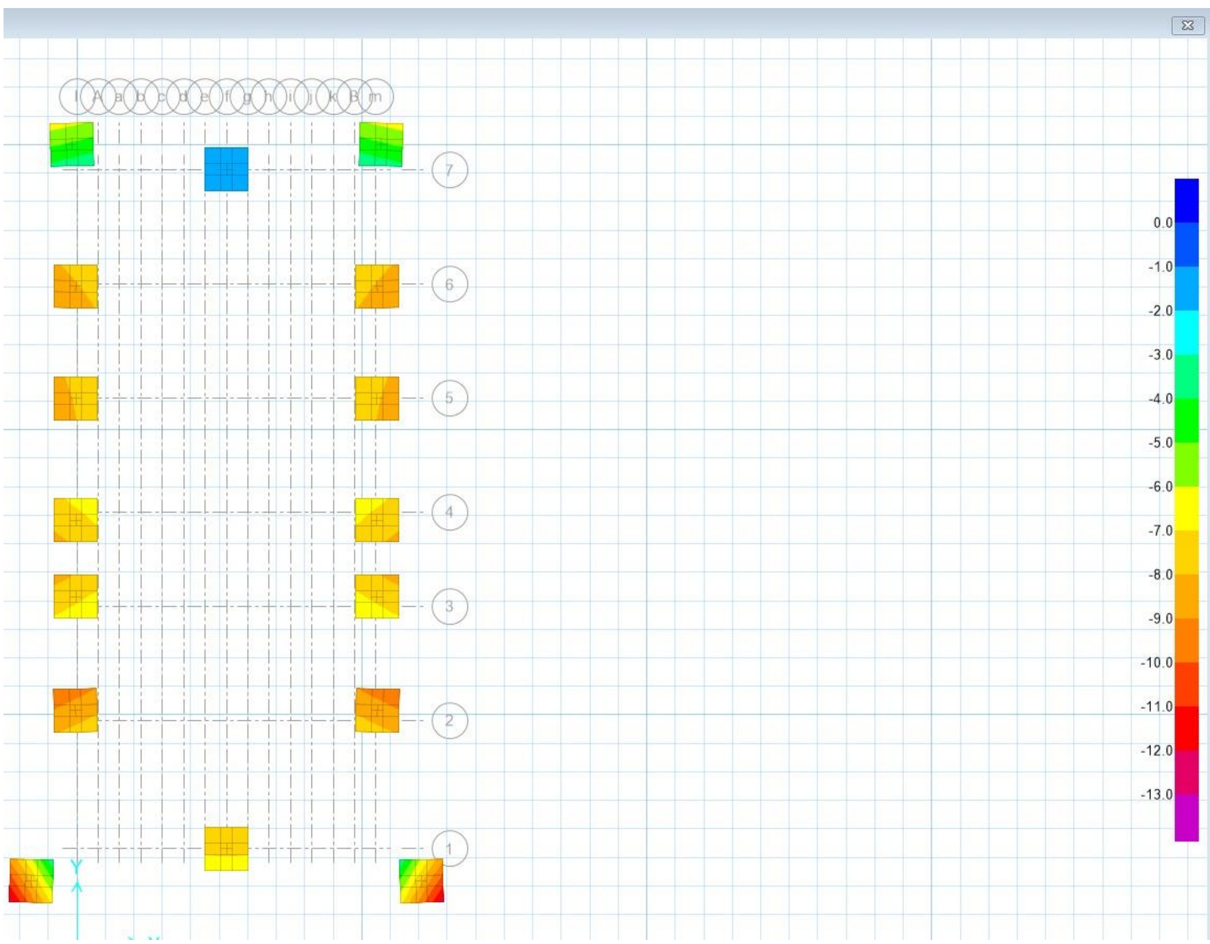
Shear V_e kN	Shear V_c kN	Shear V_s kN	Shear V_p kN	Rebar A_{sv} /s mm²/m
30.3172	0	30.3238	10.9607	329.53

Torsion Force and Torsion Reinforcement for Torsion, T_u & V_{U2}

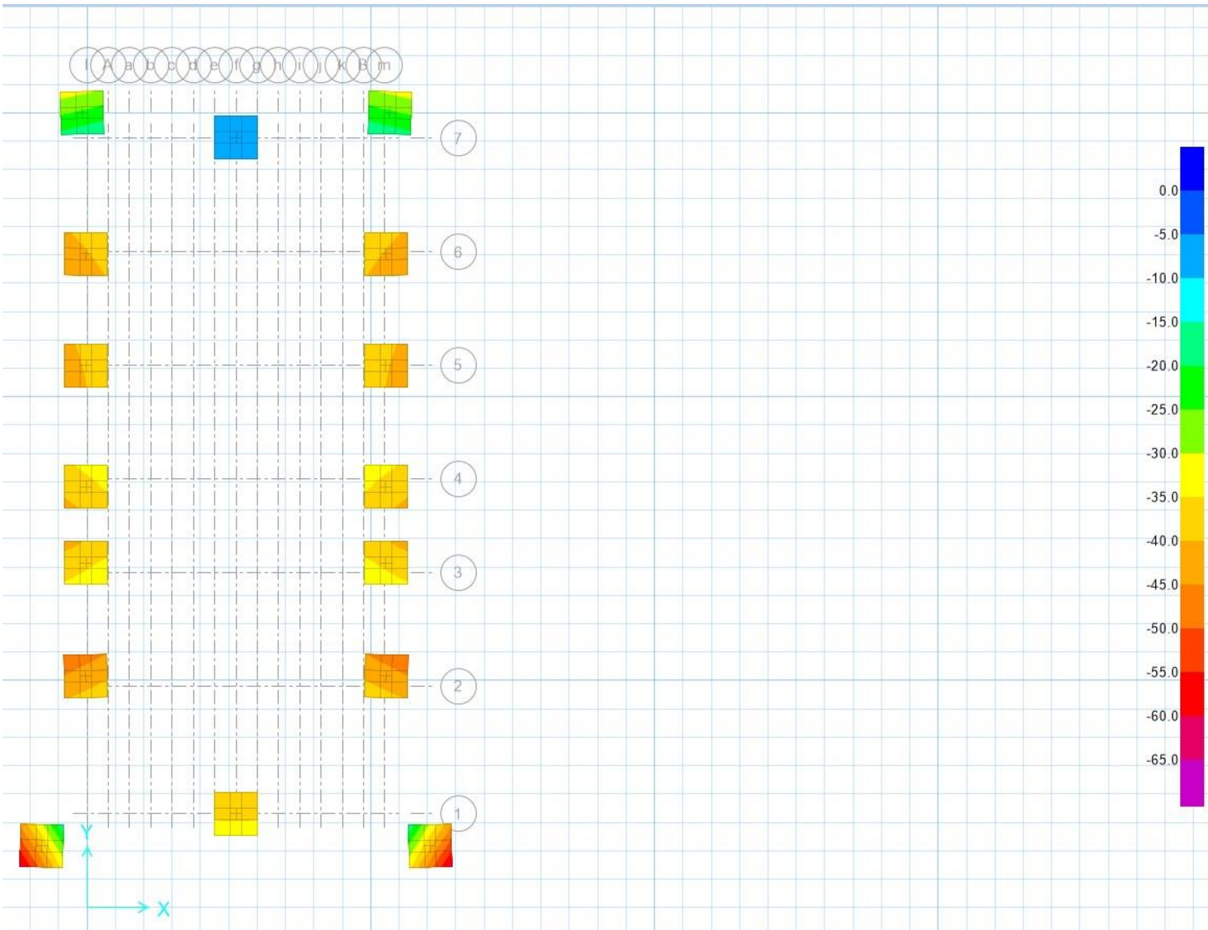
T_u kN-m	V_u kN	Core b_1 mm	Core d_1 mm	Rebar A_{svt} /s mm²/m
0.001	24.4846	160	230	0

16.0 Design Calculations

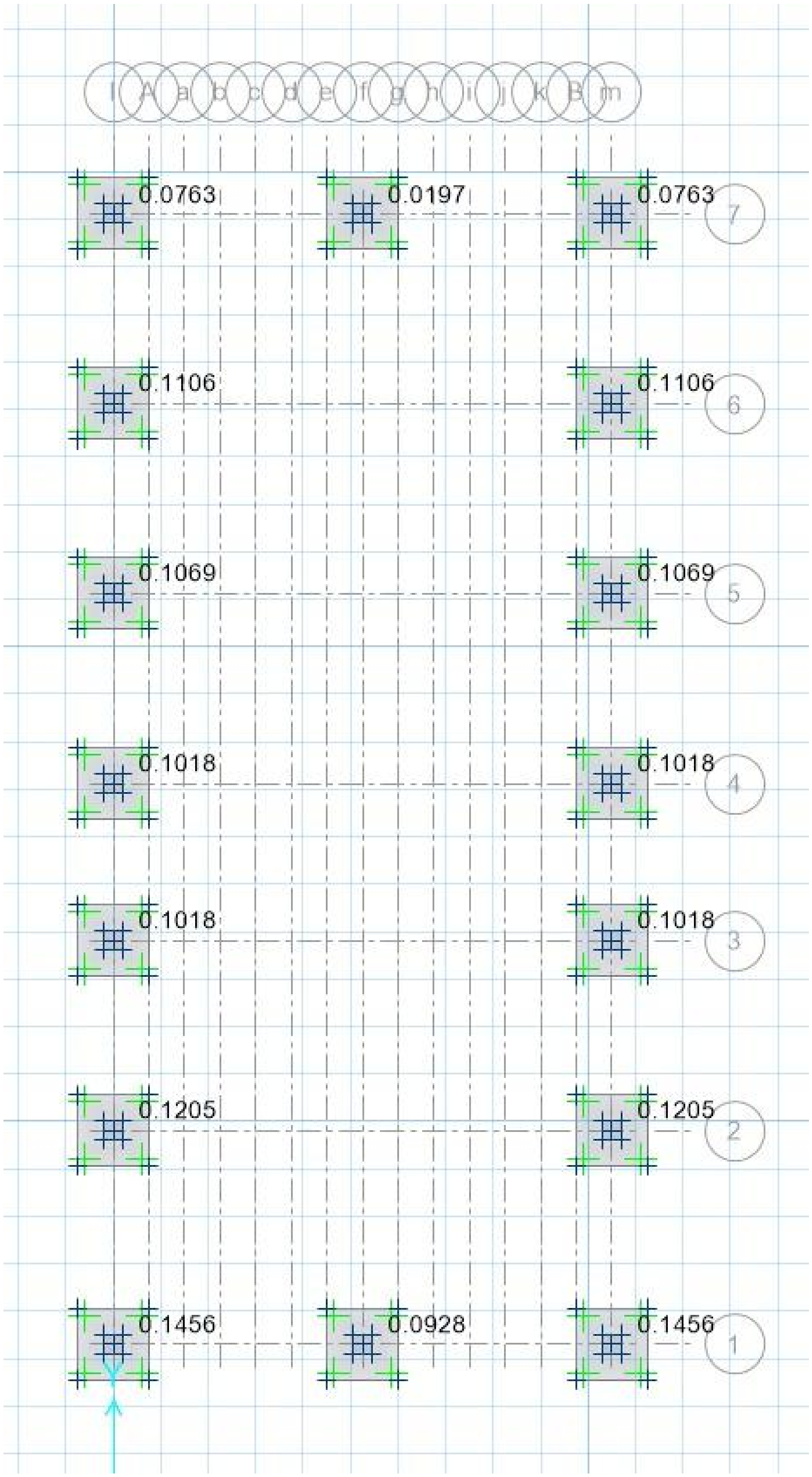
16.1 Design of Footing



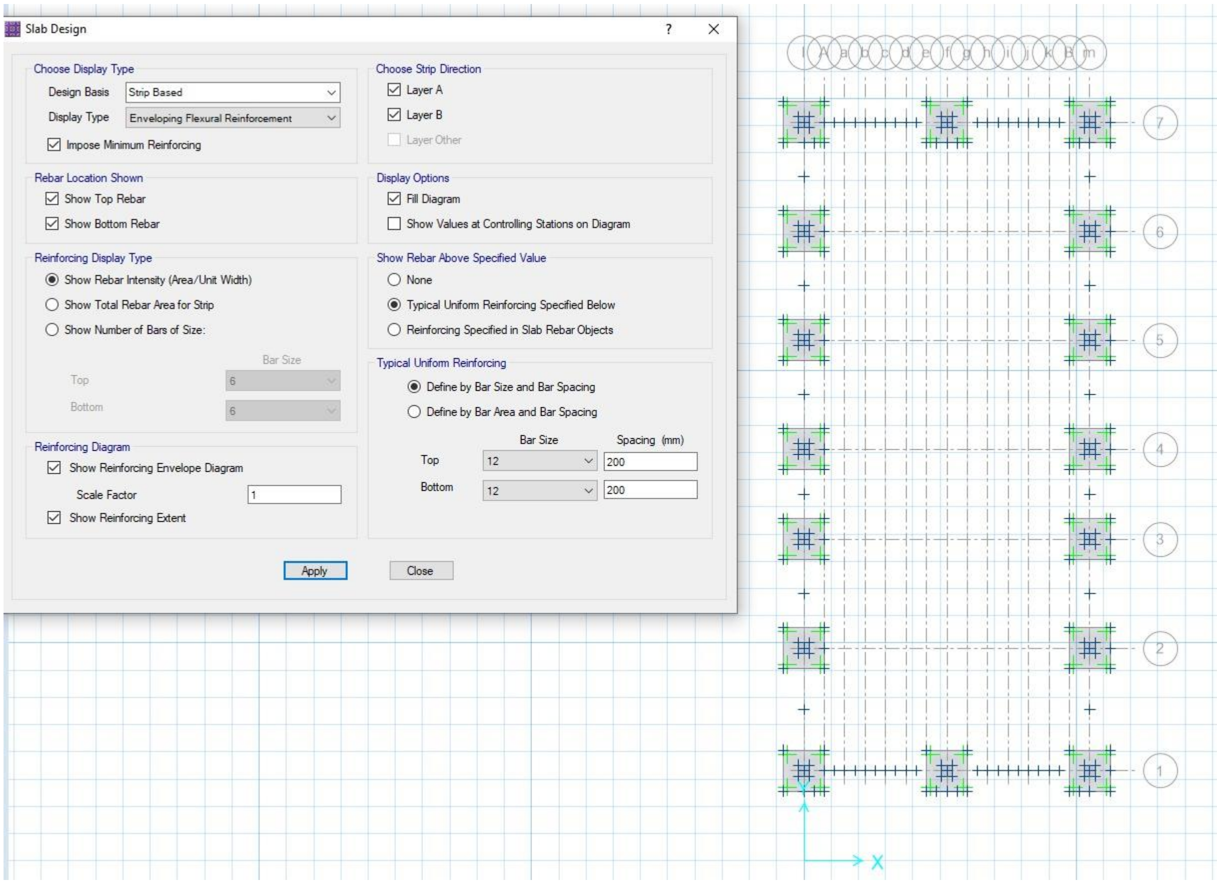
Footing Settlement < 25 mm



Bearing Capacity < 131 Kn/m2



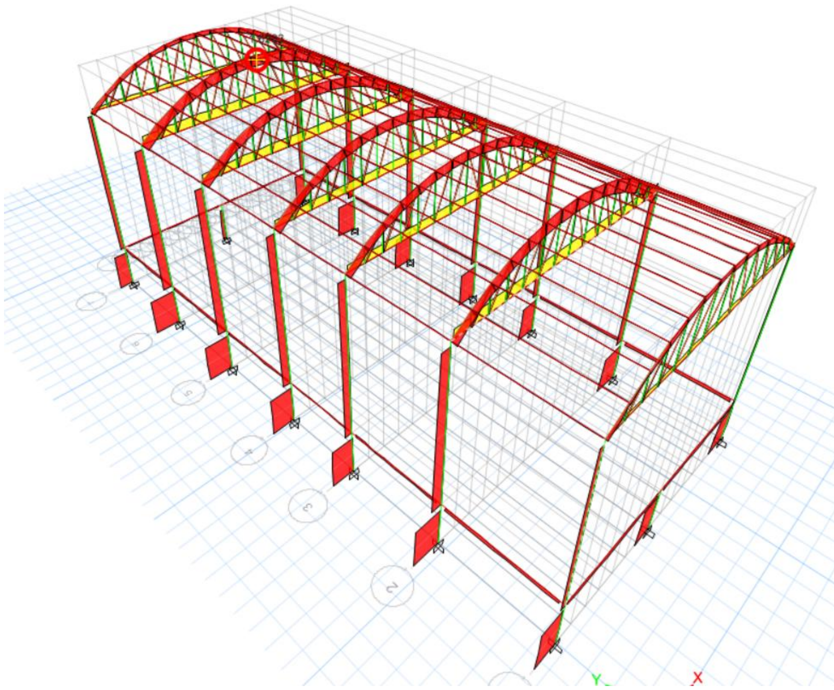
Punching Shear Design < 1



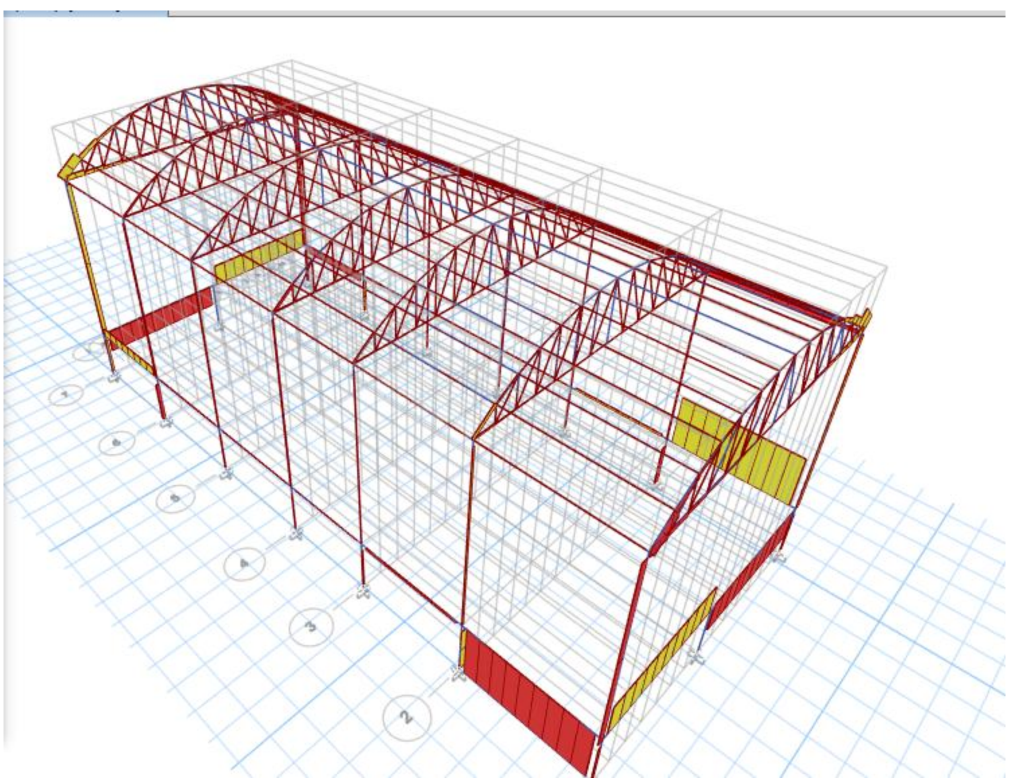
Footing Slab Reinforcement

FOOTING DEPTH =300 MM
FOOTING BOTTOM REINFORCEMENT =12 MM DIA @ 200 MM C/C BOTHWAYS

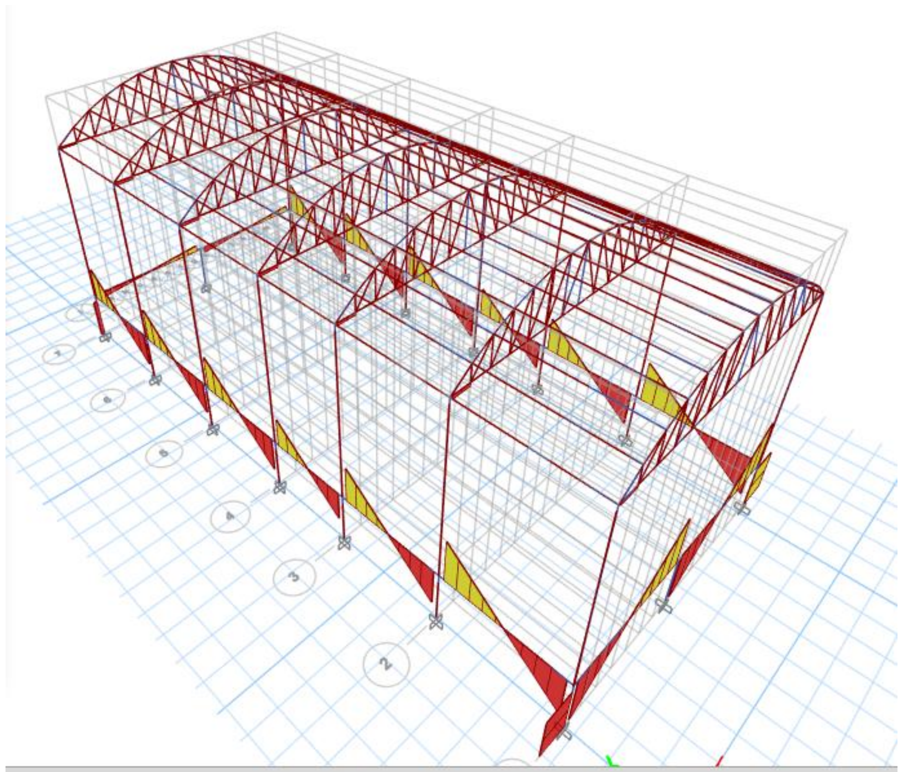
17.0 Force Diagram



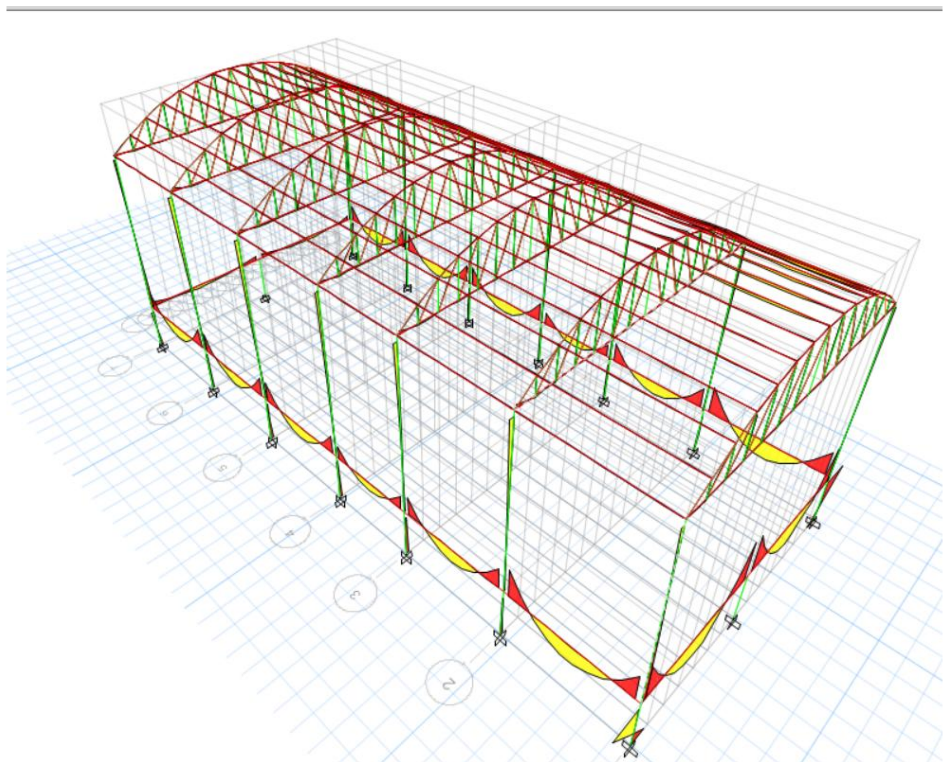
Axial Force Diagram



Torsional Force Diagram

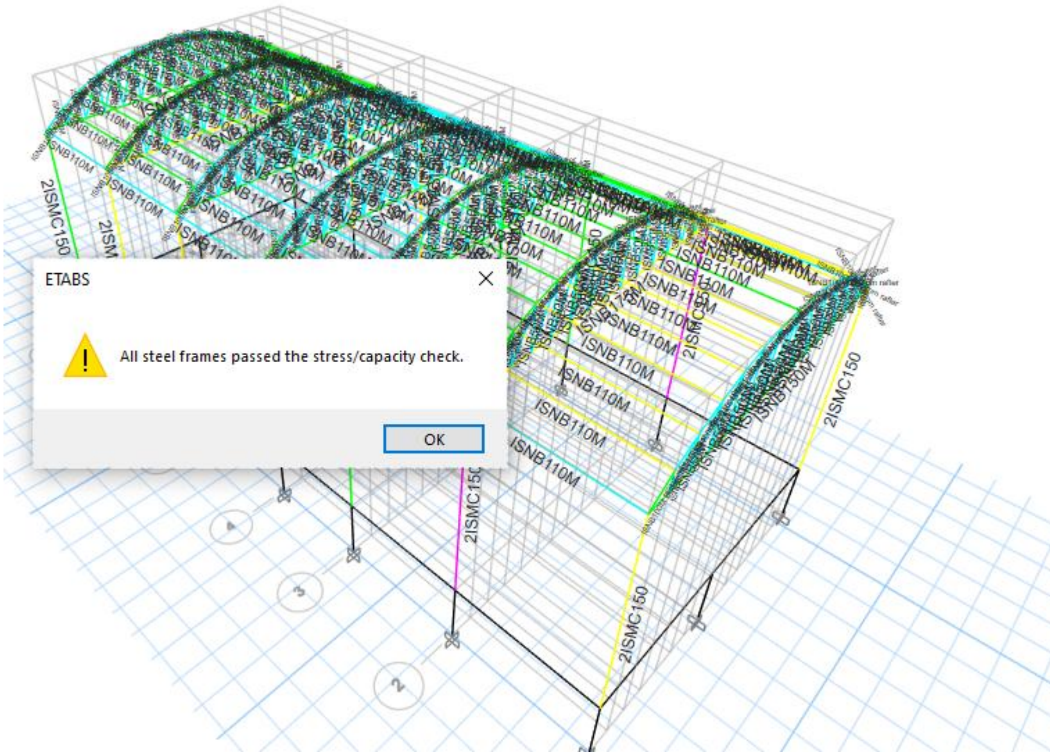


Shear Force Diagram



Moment Diagram

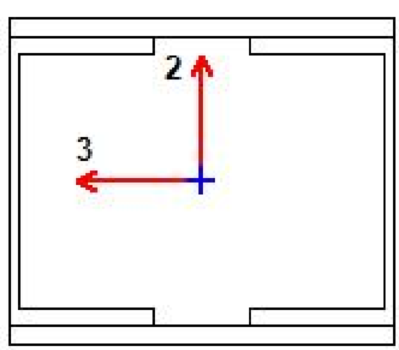
18.0 Steel Structure



19.0 Steel Design Summary

ETABS Steel Frame Design

IS 800:2007 Steel Section Check (Strength Summary)



Element Details (Part 1 of 2)

Level	Element	Unique Name	Location (mm)	Combo	Design Type	Element Type
Story2	C10	18	3253.2	DL+0.3LL+Snow load	Column	Special Moment Frame

Element Details (Part 2 of 2)

Section	Classification	Rolled
2ISMC150	Class 3	No

Design Code Parameters

γ_{M0}	γ_{M1}	A_n / A_g	LLRF	PLLF	Stress ratio Limit
1.1	1.25	1	1	0.75	0.95

Section Properties

A (cm^2)	I_{zz} (cm^4)	r_{zz} (mm)	$Z_{e,zz}$ (cm^3)	$A_{v,z}$ (cm^2)	$Z_{p,zz}$ (cm^3)	I_{yz} (cm^4)	I_t (cm^4)
81.3	4114.1	71.2	484	51.8	557.4	0	4651.9

J (cm^4)	I_{yy} (cm^4)	r_{yy} (mm)	$Z_{e,yy}$ (cm^3)	$A_{v,y}$ (cm^2)	$Z_{p,yy}$ (cm^3)	I_w (cm^6)	h (mm)
4651.9	3864.6	69	386.5	18.8	507.5		170

Material Properties

J (cm^4)	I_{yy} (cm^4)	r_{yy} (mm)
4651.9	3864.6	69
E (MPa)	f_y (MPa)	f_u (MPa)
210000	250	410

Stress Check Forces and Moments

Location (mm)	N (kN)	M_{zz} (kN-m)	M_{yy} (kN-m)	V_y (kN)	V_z (kN)	T_o (kN-m)
3253.2	-113.3182	2.8209	-0.0481	-2.0629	0.2482	0.1013

PMM Demand/Capacity (D/C) Ratio 9.3.2.2(a)

D/C Ratio =	$P / P_{dy} + K_y * C_{my} * (M_{y,span} / M_{dy;}) + K_{LT} * (M_{z,span} / M_{dz;})$
0.62 =	0.53 + 0.086 + 0.004

Basic Factors

Buckling Mode	K Factor	L Factor	L Length (mm)	KL/r
Major (z-z)	1.497	1.262	8456.3	177.927
Major Braced	0.649	1.262	8456.3	77.078
Minor (y-y)	2.598	0.971	6506.3	245.142
Minor Braced	0.858	0.971	6506.3	80.94
LTB	2.598	0.971	6506.3	245.142

Axial Force Design

	N Force kN	T_d Capacity kN	N_d Capacity kN	P_{dy} Capacity kN	P_z Capacity kN	P_d Capacity kN

	N Force kN	T_d Capacity kN	N_d Capacity kN	P_{dy} Capacity kN	P_z Capacity kN	P_d Capacity kN
Axial	-113.3182	1846.7273	1846.7273	213.991	376.9903	213.991

T_{dg} kN	T_{dn} kN	$N_{cr,T}$ kN	$N_{cr,TF}$ kN	A_n /A_g Unitless	N /N_d Unitless
1846.7273	2398.6771	382643.8318	280.2461	1	0.061

Design Parameters for Axial Design

	Curve	α	f_{cc} (MPa)	λ	Φ	χ	f_{cd} (MPa)
Major (z-z)	c	0.49	65.47	1.954	2.839	0.204	46.4
MajorB (z-z)	c	0.49	348.87	0.847	1.017	0.633	143.86
Minor (y-y)	c	0.49	34.49	2.692	4.735	0.116	26.34
MinorB (y-y)	c	0.49	316.37	0.889	1.064	0.607	26.34
Torsional TF	c	0.49	34.49	2.692	4.735	0.116	26.34

Moment Designs

	M Moment kN-m	M_{span} Moment kN-m	$M_{d(yield)}$ Capacity kN-m	M_{dv} Capacity kN-m	M_{nd} Capacity kN-m	$M_{d(LTB)}$ Capacity kN-m
Major (z-z)	2.8209	9.5317	110.0032	110.0032	110.0032	109.1313
Minor (y-y)	-0.0481	-0.8556	87.8315	87.8315	87.8315	

	Curve	α_{LT}	λ_{LT}	Φ_{LT}	χ_{LT}	C_1	M_{cr} (kN-m)
LTB	c	0.49	0.216	0.527	0.992	2.538	2604.2877

	C_{my}	C_{mz}	C_{mLT}	k_z	k_y	K_{LT}	M_y / M_{dy}	M_z / M_{dz}	α_1	α_2
Factors	0.4	1	0.437	1.063	1.07	0.988	-0.001	0.026	2	2

Shear Design

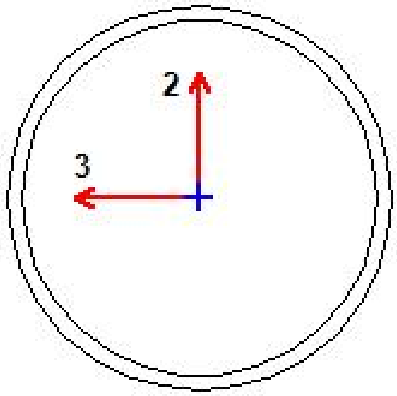
	V Force (kN)	V_d Capacity (kN)	T_o Capacity (kN-m)	Stress Ratio	Status Check
Major (y)	2.0629	247.0631	0.1013	0.008	OK
Minor (z)	0.2482	679.2082	0.1013	3.655E-04	OK

Shear Design

	V_p (kN)	k_v (Unitless)	Λ_W (Unitless)	T_b (MPa)
Reduction	247.0631	0	0	1

ETABS Steel Frame Design

IS 800:2007 Steel Section Check (Strength Summary)



Element Details (Part 1 of 2)

Level	Element	Unique Name	Location (mm)	Combo	Design Type	Element Type	Section
Story2	B5	322	3936.2	DL+0.3LL+EQy ULS	Beam	Special Moment Frame	ISNB110M

Element Details (Part 2 of 2)

Classification	Rolled
Class 1	Yes

Seismic Parameters

MultiResponse	P-Δ Done?	Ignore Seismic Code?	Ignore Special EQ Load?	D/P Plug Welded?
Envelopes	No	No	No	Yes

Design Code Parameters

γ_{M0}	γ_{M1}	A_n/A_g	LLRF	PLLF	Stress ratio Limit
1.1	1.25	1	1	0.75	0.95

Section Properties

A (cm^2)	I_{zz} (cm^4)	r_{zz} (mm)	$Z_{e,zz}$ (cm^3)	$A_{v,z}$ (cm^2)	$Z_{p,zz}$ (cm^3)	I_{yz} (cm^4)	I_t (cm^4)
18.4	344.6	43.3	54.3	11.7	54.3	0	689.2

J (cm^4)	I_{yy} (cm^4)	r_{yy} (mm)	$Z_{e,yy}$ (cm^3)	$A_{v,y}$ (cm^2)	$Z_{p,yy}$ (cm^3)	I_w (cm^6)	h (mm)
689.2	344.6	43.3	54.3	11.7	54.3		127

Material Properties

J (cm^4)	I_{yy} (cm^4)	r_{yy} (mm)
689.2	344.6	43.3
E (MPa)	f_y (MPa)	f_u (MPa)

J (cm^4)	I_{yy} (cm^4)	r_{yy} (mm)
210000	250	410

Stress Check Forces and Moments

Location (mm)	N (kN)	M_{zz} (kN-m)	M_{yy} (kN-m)	V_y (kN)	V_z (kN)	T_o (kN-m)
3936.2	0.8847	-3.7355	-4.3261	2.1985	2.4614	0.0089

PMM Demand/Capacity (D/C) Ratio 9.3.2.2(a)

D/C Ratio =	P / P_{dy} + Sqrt[(K_y * C_{my} * ($M_{y,span}$ / $M_{dy;}$)^2 + (K_{LT} * ($M_{z,span}$ / $M_{dz;}$)^2]
0.463 =	0 + Sqrt[$(0.303)^2$ + $(0.351)^2$]

Basic Factors

Buckling Mode	K Factor	L Factor	L Length (mm)	KL/r
Major (z-z)	1	0.968	3872.4	89.485
Major Braced	1	0.968	3872.4	89.485
Minor (y-y)	1	0.968	3872.4	89.485
Minor Braced	1	0.968	3872.4	89.485
LTB	1	0.968	3872.4	89.485

Axial Force Design

	N Force kN	T_d Capacity kN	N_d Capacity kN	P_{dy} Capacity kN	P_z Capacity kN	P_d Capacity kN
Axial	0.8847	418.1818	418.1818	283.3657	283.3657	283.3657

T_{dg} kN	T_{dn} kN	$N_{cr,T}$ kN	$N_{cr,TF}$ kN	A_n /A_g Unitless	N /N_d Unitless
418.1818	543.168	148615.418	476.256	1	0.002

Design Parameters for Axial Design

	Curve	α	f_{cc} (MPa)	λ	Φ	χ	f_{cd} (MPa)
Major (z-z)	a	0.21	258.83	0.983	1.065	0.678	154
MajorB (z-z)	a	0.21	258.83	0.983	1.065	0.678	154
Minor (y-y)	a	0.21	258.83	0.983	1.065	0.678	154
MinorB (y-y)	a	0.21	258.83	0.983	1.065	0.678	154
Torsional TF	a	0.21	258.83	0.983	1.065	0.678	154

Moment Designs

	M Moment kN-m	M_{span} Moment kN-m	$M_{d(yield)}$ Capacity kN-m	M_{dv} Capacity kN-m	M_{nd} Capacity kN-m	$M_{d(LTB)}$ Capacity kN-m
Major (z-z)	-3.7355	-3.7355	12.3341	12.3341	12.3341	12.3341
Minor (y-y)	-4.3261	-4.3261	12.3341	12.3341	12.3341	

	Curve	$α_{LT}$	$λ_{LT}$	$Φ_{LT}$	$χ_{LT}$	C_1	M_{cr} (kN-m)
LTB	a	0.21	0.107	0.496	1	2.301	1184.7527

	C_{my}	C_{mz}	C_{mLT}	k_z	k_y	K_{LT}	M_y / M_{dy}	M_z / M_{dz}	$α_1$	$α_2$
Factors	1	1	1	1	1	1	-0.351	-0.303	2	2

Shear Design

	V Force (kN)	V_d Capacity (kN)	T_o Capacity (kN-m)	Stress Ratio	Status Check
Major (y)	2.1985	153.7037	0.0089	0.014	OK
Minor (z)	2.4614	153.7037	0.0089	0.016	OK

Shear Design

	V_p (kN)	k_v (Unitless)	Λ_W (Unitless)	T_b (MPa)
Reduction	153.7037	0	0	1

End Reaction Major Shear Forces

Left End Reaction (kN)	Load Combo	Right End Reaction (kN)	Load Combo
-5.0764	DL+LL-	5.0358	DL+LL-

20.0 Column rafter connection

Material

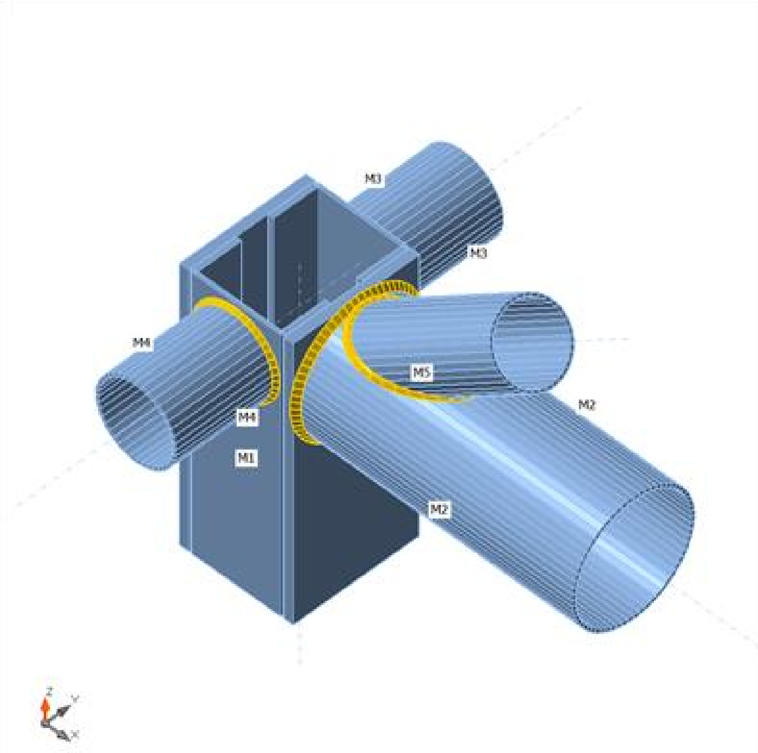
Steel E 250 (Fe 410 W) B
Concrete M30

Design

Name column rafter connection
Description
Analysis Stress, strain/ simplified loading

Beams and columns

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [mm]	Offset ey [mm]	Offset ez [mm]	Forces in
M1	1 - General	0.0	90.0	0.0	0	0	0	Node
M2	2 - ISTube 175x5.4	0.0	0.0	0.0	50	0	-100	Node
M3	3 - ISTube 110x4.8	90.0	0.0	0.0	0	0	-75	Node
M4	3 - ISTube 110x4.8	-90.0	0.0	0.0	0	0	-75	Node
M5	4 - ISTube 100x4.5	0.0	-37.0	0.0	0	0	-120	Node



Cross-sections

Name	Material
1 - General	E 250 (Fe 410 W) B, E 250 (Fe 410 W) B, E 250 (Fe 410 W) B, E 250 (Fe 410 W) B
2 - ISTube 175x5.4	E 250 (Fe 410 W) B
3 - ISTube 110x4.8	E 250 (Fe 410 W) B
4 - ISTube 100x4.5	E 250 (Fe 410 W) B

Load effects (equilibrium not required)

Name	Member	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
LE1	M2	188.1	24.6	0.0	-13.9	0.0	0.0
	M3	1.3	4.3	0.0	-4.0	0.0	0.0
	M4	1.5	5.7	0.0	-4.2	0.0	0.0
	M5	148.7	2.5	0.0	-2.5	0.0	0.0

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	2.4 < 5.0%	OK
Loc. deformation	0.5 < 3%	OK
Welds	98.6 < 100%	OK
Buckling	Not calculated	

Plates

Name	Material	f_{yd} [MPa]	Thickness [mm]	Loads	σ [MPa]	ε_{Pl} [%]	σ_{cEd} [MPa]	Check status
M1-bfl 1	E 250 (Fe 410 W) B	227.3	9.0	LE1	227.5	0.1	0.0	OK
M1-tfl 1	E 250 (Fe 410 W) B	227.3	9.0	LE1	228.0	0.4	0.0	OK
M1-w 1	E 250 (Fe 410 W) B	227.3	5.7	LE1	230.9	1.8	0.0	OK
M1-bfl 2	E 250 (Fe 410 W) B	227.3	15.0	LE1	229.0	0.9	0.0	OK
M1-bfl 3	E 250 (Fe 410 W) B	227.3	9.0	LE1	227.8	0.3	0.0	OK
M1-tfl 2	E 250 (Fe 410 W) B	227.3	9.0	LE1	227.5	0.1	0.0	OK
M1-w 2	E 250 (Fe 410 W) B	227.3	5.7	LE1	230.7	1.7	0.0	OK
M1-bfl 4	E 250 (Fe 410 W) B	227.3	15.0	LE1	229.0	0.9	0.0	OK
M2	E 250 (Fe 410 W) B	250.0	5.4	LE1	232.1	2.4	0.0	OK
M3	E 250 (Fe 410 W) B	250.0	4.8	LE1	227.4	0.1	0.0	OK
M4	E 250 (Fe 410 W) B	250.0	4.8	LE1	227.8	0.3	0.0	OK
M5	E 250 (Fe 410 W) B	250.0	4.5	LE1	229.6	1.2	0.0	OK

Symbol explanation

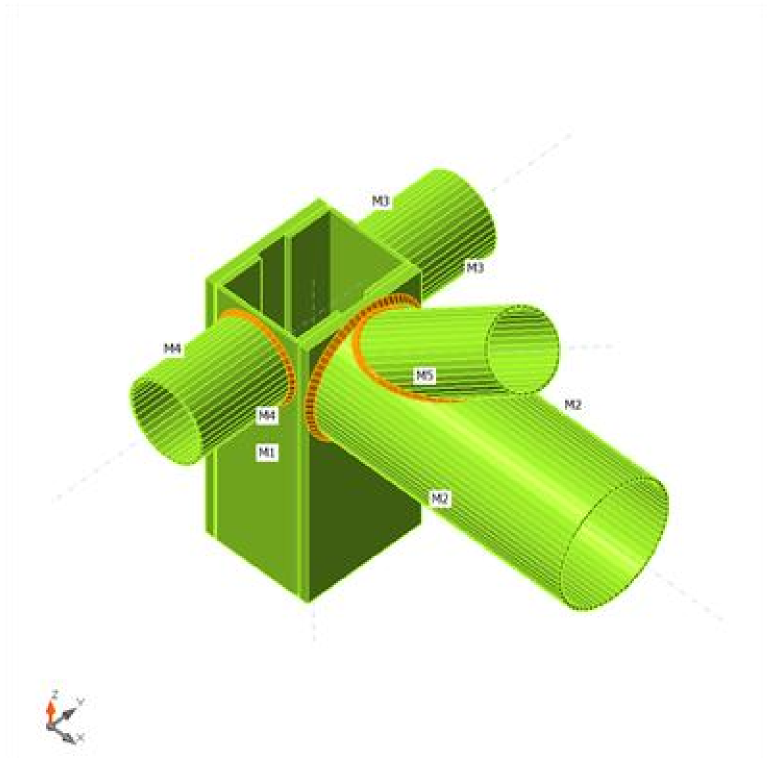
ε_{Pl} Plastic strain
σ Equivalent stress
f_{yd} Design yield strength
σ_{cEd} Contact stress

Loc. deformation

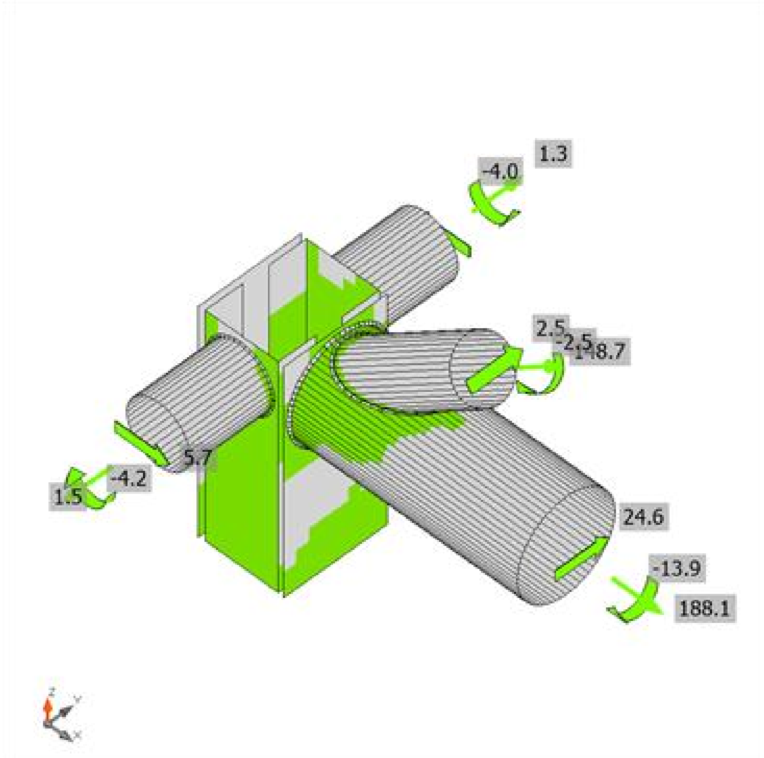
Name	d0 [mm]	Loads	δ [mm]	δ lim [mm]	δ/d0 [%]	Check status
M2	194	LE1	1	6	0.5	OK
M3	127	LE1	0	4	0.1	OK
M4	127	LE1	0	4	0.2	OK
M5	114	LE1	0	3	0.2	OK

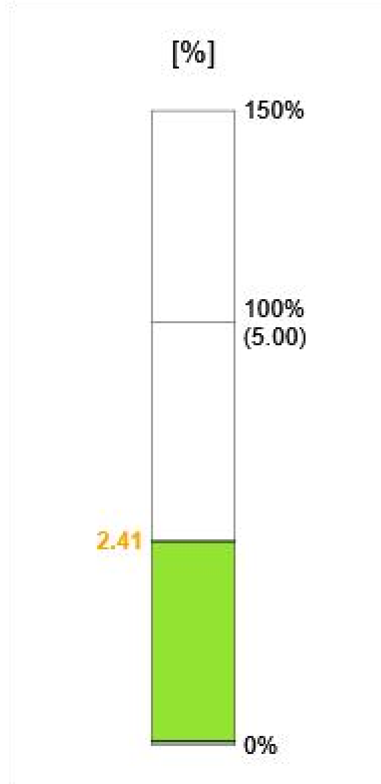
Symbol explanation

d_0 Cross-section size
δ Local cross-section deformation
$δ_{lim}$ Allowed deformation

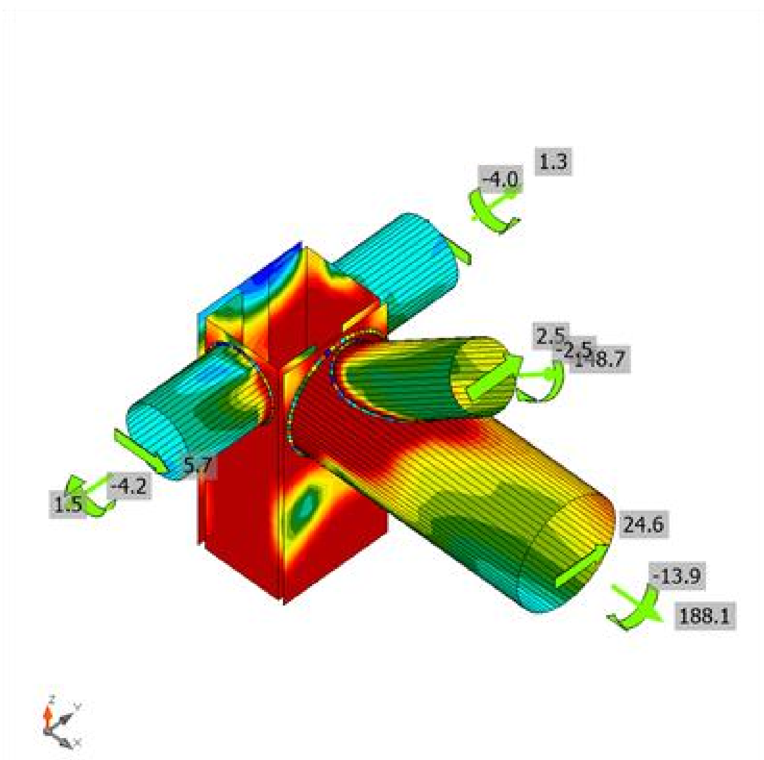


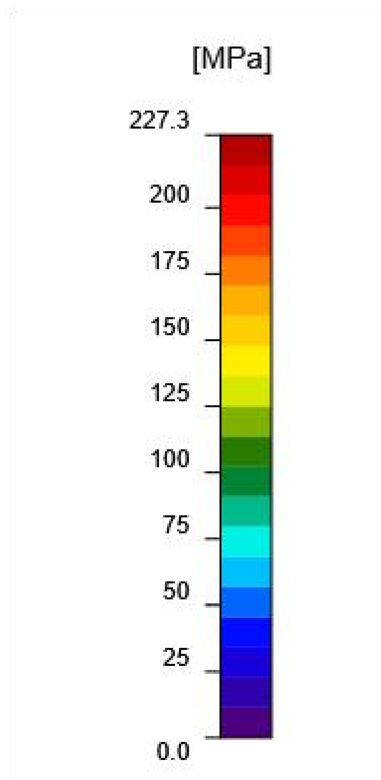
Overall check, LE1





Strain check, LE1





Equivalent stress, LE1

Weld sections

Item	Edge	Electrode	t_t [mm]	l_j [mm]	l_{je} [mm]	Loads	f_e [MPa]	f_{wd} [MPa]	U_t [%]	Status
M1-bfl 2	M2	E 250 (Fe 410 W) B	◢6.0◣	591	9	LE1	185.8	189.4	98.1	OK
M2-arc 23	M5	E 250 (Fe 410 W) B	◢4.0◣	474	8	LE1	186.6	189.4	98.6	OK
M1-w 1	M4	E 250 (Fe 410 W) B	◢4.0◣	384	10	LE1	185.7	189.4	98.1	OK
M1-w 2	M3	E 250 (Fe 410 W) B	◢4.0◣	384	10	LE1	185.6	189.4	98.0	OK
		E 250 (Fe 410 W) B	◢6.0◣	591	9	LE1	186.0	189.4	98.2	OK
		E 250 (Fe 410 W) B	◢4.0◣	474	8	LE1	185.8	189.4	98.1	OK
		E 250 (Fe 410 W) B	◢4.0◣	384	10	LE1	185.7	189.4	98.1	OK
		E 250 (Fe 410 W) B	◢4.0◣	384	10	LE1	185.7	189.4	98.0	OK

Symbol explanation

t_t Fillet weld throat thickness
l_j Weld length
l_{je} Weld element length
f_e Equivalent stress in the weld
f_{wd} Design strength of a fillet weld
U_t Utilization

Detailed result for M2-arc 23 / M5

Weld resistance check (IS 800, Cl. 10.5.10.1.1)

$f_e =$ 186.6 MPa ≤ $f_{wd} =$ 189.4 MPa

Where:

$f_e =$ 186.6 MPa – equivalent stress in weld

$f_a =$ 31.9 MPa – normal stresses, compression or tension, due to axial force or bending moment

$q =$ 106.2 MPa – shear stress due to shear force or tension

$f_{wd} =$ 189.4 MPa – design strength of a fillet weld

$$f_{wd} = \frac{f_u}{\sqrt{3} \cdot \gamma_{mw}}$$

, where:

$f_u =$
410.0 MPa – smaller of the ultimate stress of the weld or of the parent metal

$\gamma_{mw} =$
1.25 – partial safety factor for welds

Buckling

Buckling analysis was not calculated.

Code settings

Item	Value	Unit	Reference
Friction coefficient - concrete	0.45	-	IS 800, Cl. 7.4.1
Friction coefficient in slip-resistance	0.30	-	IS 800, Cl. 10.4.3
Limit plastic strain	0.05	-	
Detailing	No		
Distance between bolts [d]	2.50	-	IS 800, Cl. 10.2.2
Distance between bolts and edge [d]	1.50	-	IS 800, Cl. 10.2.4
Bolt maximum grip length [d]	8.00	-	Limit grip length of bolts as a multiple of bolt diameter - IS 800, Cl. 10.3.3.2
Local deformation check	Yes		
Local deformation limit	0.03	-	CIDECT DG 1, 3 - 1.1
Geometrical nonlinearity (GMNA)	Yes		Analysis with large deformations for hollow section joints
Concrete in compression check	IS800, Cl 7.4		
Braced system (EC stiffness classification)	No		EN1993-1-8 - Cl. 5.2.2.5

21.0 Baseplate Design

Material

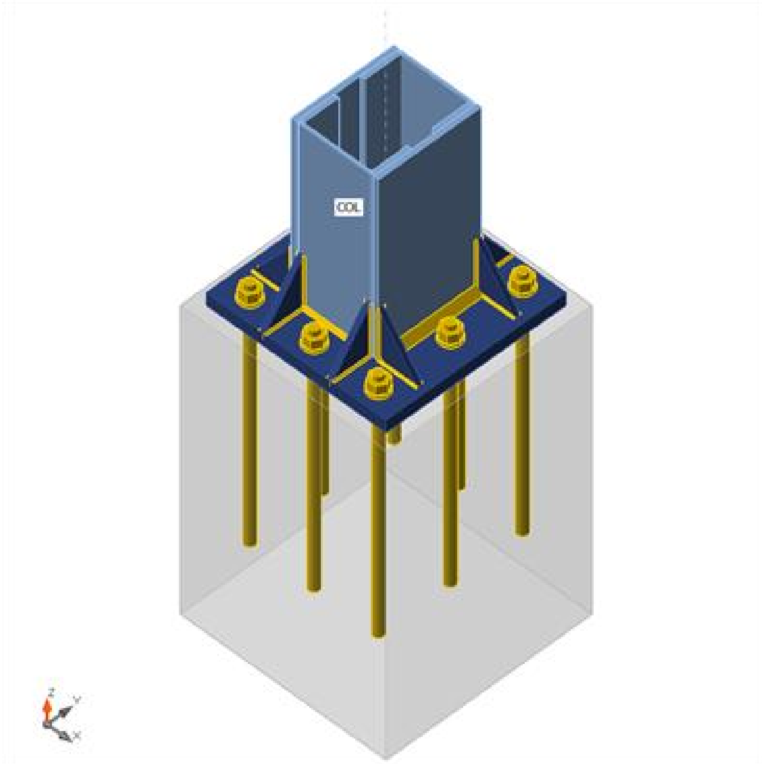
Steel A36
Concrete 3000 psi, 4000 psi

Design

Name Baseplate
Description
Analysis Stress, strain/ simplified loading
Design code AISC - LRFD 2016

Beams and columns

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [mm]	Offset ey [mm]	Offset ez [mm]	Forces in
COL	2 - General	0.0	-90.0	0.0	0	0	0	Node



Cross-sections

Name	Material
2 - General	A36, A36, A36, A36

Anchors

Name	Bolt assembly	Diameter [mm]	fu [MPa]	Gross area [mm^2]
3/4 A325	3/4 A325	19	827.4	285

Load effects (equilibrium not required)

Name	Member	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
LE1	COL	-117.0	15.0	20.0	-7.9	-11.5	-0.2

Foundation block

Item	Value	Unit
CB 1		
Dimensions	400 x 400	mm
Depth	600	mm
Anchor	3/4 A325	
Anchoring length	450	mm
Shear force transfer	Friction	

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	0.0 < 5.0%	OK
Anchors	46.9 < 100%	OK
Welds	42.8 < 100%	OK
Concrete block	10.8 < 100%	OK
Shear	74.7 < 100%	OK
Buckling	Not calculated	

Plates

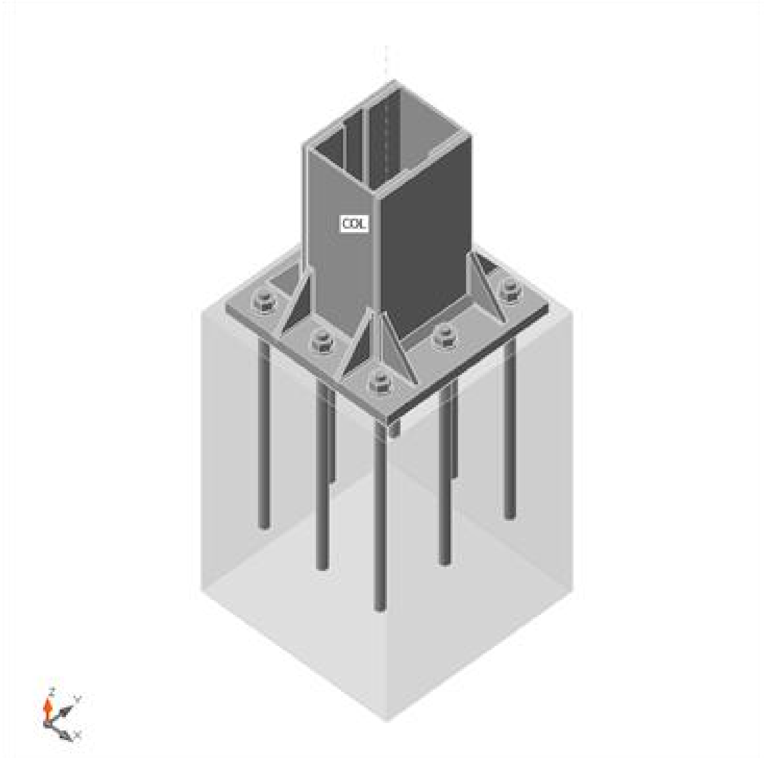
Name	f_y [MPa]	Thickness [mm]	Loads	σ_{Ed} [MPa]	ε_{Pl} [%]	σc_{Ed} [MPa]	Check status
COL-bfl 1	248.2	9.0	LE1	26.0	0.0	0.0	OK
COL-tfl 1	248.2	9.0	LE1	37.8	0.0	0.0	OK
COL-w 1	248.2	5.4	LE1	80.2	0.0	0.0	OK
COL-bfl 2	248.2	10.0	LE1	50.6	0.0	0.0	OK
COL-bfl 3	248.2	9.0	LE1	11.8	0.0	0.0	OK
COL-tfl 2	248.2	9.0	LE1	37.0	0.0	0.0	OK
COL-w 2	248.2	5.4	LE1	53.1	0.0	0.0	OK
COL-bfl 4	248.2	10.0	LE1	26.4	0.0	0.0	OK
BP1	248.2	24.0	LE1	29.1	0.0	0.0	OK
RIB1a	248.2	10.0	LE1	17.6	0.0	0.0	OK
RIB1b	248.2	10.0	LE1	34.7	0.0	0.0	OK
RIB2a	248.2	10.0	LE1	14.7	0.0	0.0	OK
RIB2b	248.2	10.0	LE1	52.1	0.0	0.0	OK
RIB3a	248.2	10.0	LE1	62.9	0.0	0.0	OK
RIB3b	248.2	10.0	LE1	72.1	0.0	0.0	OK
RIB4a	248.2	10.0	LE1	18.5	0.0	0.0	OK
RIB4b	248.2	10.0	LE1	21.0	0.0	0.0	OK

Design data

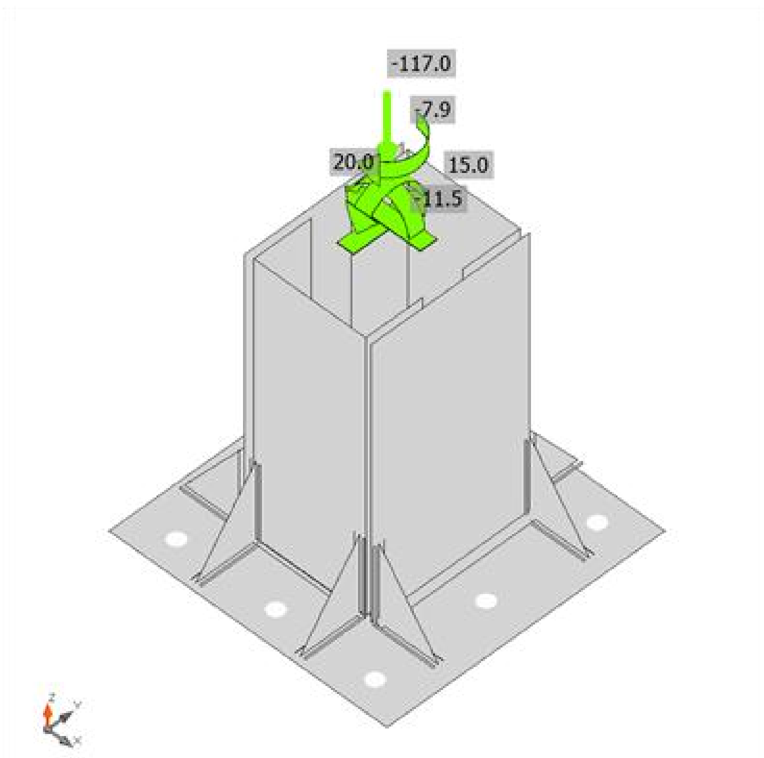
Material	f_y [MPa]	ε_{lim} [%]
A36	248.2	5.0

Symbol explanation

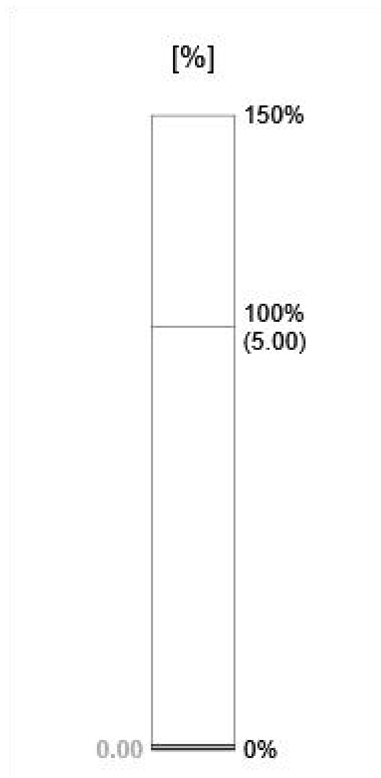
ε_{Pl}	Plastic strain
σc_{Ed}	Contact stress
σ_{Ed}	Eq. stress
f_y	Yield strength
ε_{lim}	Limit of plastic strain

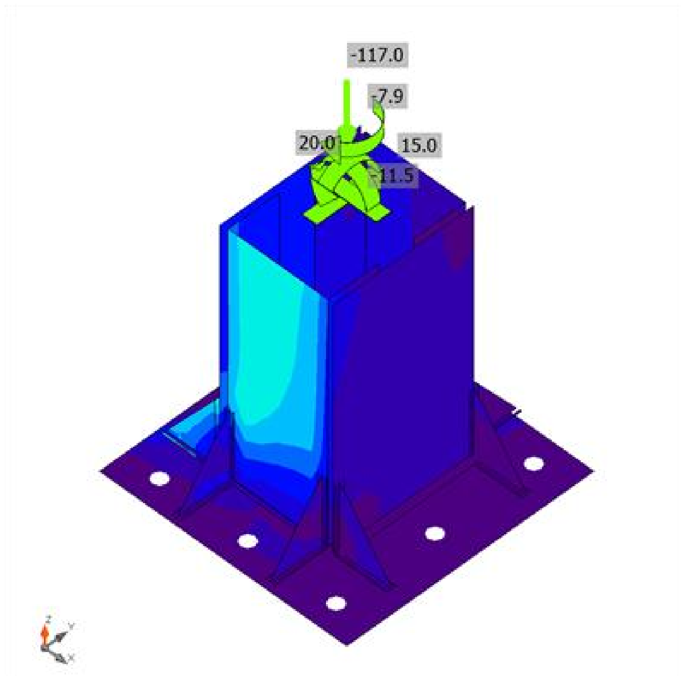


Overall check, LE1

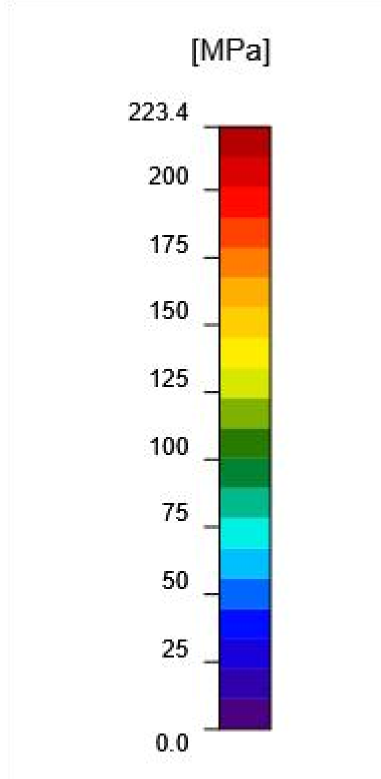


Strain check, LE1

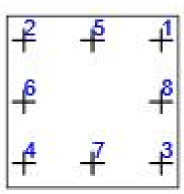




Equivalent stress, LE1



Anchors

Shape	Item	Loads	N_f [kN]	V [kN]	ϕN_{cbg} [kN]	ϕV_{cp} [kN]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Status
	A1	LE1	0.0	0.0	-	63.9	0.0	0.0	0.0	OK
	A2	LE1	0.0	0.0	-	63.9	0.0	0.0	0.0	OK
	A3	LE1	3.9	0.0	24.6	63.9	46.9	0.0	28.3	OK
	A4	LE1	3.1	0.0	24.6	63.9	46.9	0.0	28.3	OK
	A5	LE1	0.0	0.0	-	63.9	0.0	0.0	0.0	OK
	A6	LE1	0.0	0.0	-	63.9	0.0	0.0	0.0	OK
	A7	LE1	4.6	0.0	24.6	63.9	46.9	0.0	28.3	OK
	A8	LE1	0.0	0.0	-	63.9	0.0	0.0	0.0	OK

Design data

Grade	ϕN_{sa} [kN]	ϕV_{sa} [kN]
3/4 A325 - 1	124.8	69.5

Symbol explanation

N_f Tension force
V Resultant of shear forces Vy, Vz in bolt
ϕN_{cbg} Concrete breakout strength in tension – ACI 318-14 – 17.4.2
ϕV_{cp} Concrete pryout strength in shear – ACI 318-14 – 17.5.3
Ut_t Utilization in tension
Ut_s Utilization in shear
Ut_{ts} Utilization in tension and shear
ϕN_{sa} Steel strength of anchor in tension - ACI 318-14 – 17.4.1

ϕV_{sa} Steel strength of anchor in shear - ACI 318-14 – 17.5.1

Detailed result for A3

Anchor tensile resistance (ACI 318-14 – 17.4.1)

$\phi N_{sa} = \phi \cdot A_{se,N} \cdot$ 124.8 kN ≥ $N_f =$ 3.9 kN

Where:

$\phi = 0.70$ – resistance factor

$A_{se,N} = 215\ \text{mm}^2$ – tensile stress area

$f_{uta} = 827.4\ \text{MPa}$ – specified tensile strength of anchor steel:
$f_{uta} = \min(860\ \text{MPa}, 1.9 \cdot f_{ya}, f_u)$
, where:
$f_{ya} =$
634.3 MPa – specified yield strength of anchor steel
$f_u =$
827.4 MPa – specified ultimate strength of anchor steel

Concrete breakout resistance of anchor in tension (ACI 318-14 – 17.4.2)
The check is performed for group of anchors that form common tension breakout cone: A3, A4, A7

$\phi N_{cbg} = \phi \cdot \frac{A_{Nc}}{A_{Nc0}} \cdot \Psi_{ed,N} \cdot \Psi_{ec,N} \cdot \Psi_{c,N} \cdot$ 24.6 kN ≥ $N_{fg} =$ 11.5 kN

Where:

$N_{fg} = 11.5\ \text{kN}$ – sum of tension forces of anchors with common concrete breakout cone area

$\phi = 0.70$ – resistance factor

$A_{Nc} = 59850\ \text{mm}^2$ – concrete breakout cone area for group of anchors

$A_{Nc0} = 22500\ \text{mm}^2$ – concrete breakout cone area for single anchor not influenced by edges

$\Psi_{ed,N} = 0.94$ – modification factor for edge distance:

$\Psi_{ed,N} = \min(0.7 + \frac{0.3 \cdot c_{a,min}}{1.5 \cdot h_{ef}}, 1)$
, where:
$c_{a,min} =$
60 mm – minimum distance from the anchor to the edge
$h_{ef} = \min(h_{emb}, \max(\frac{c_{a,max}}{1.5}, \frac{s}{3})) =$
50 mm – depth of embedment, where:
$h_{emb} =$
450 mm – anchor length
$c_{a,max} =$
75 mm – maximum distance from the anchor to one of the three closest edges
$s =$
140 mm – maximum spacing between anchors

$\Psi_{ec,N} = 0.88$ – modification factor for eccentrically loaded group of anchors

$\Psi_{ec,N} = \Psi_{ecx,N} \cdot \Psi_{ecy,N}$
, where:

$\Psi_{ecx,N} = \frac{1}{1+\frac{2 \cdot e_{x,N}}{3 \cdot h_{ef}}} =$
1.00 – modification factor that depends on eccentricity in x-direction
$e_{x,N} =$
0 mm – tension load eccentricity in x-direction
$\Psi_{ecy,N} = \frac{1}{1+\frac{2 \cdot e_{y,N}}{3 \cdot h_{ef}}} =$
0.88 – modification factor that depends on eccentricity in y-direction
$e_{y,N} =$
11 mm – tension load eccentricity in y-direction
$h_{ef} =$
50 mm – depth of embedment

$\Psi_{c,N} = 1.00$ – modification factor for concrete conditions

$N_b = 16.1$ kN – basic concrete breakout strength of a single anchor in tension:

$N_b = k_c \cdot \lambda_a \cdot \sqrt{f_c} \cdot h_{ef}^{1.5}$
, where:
$k_c =$
10.0 – coefficient for cast-in anchors
$\lambda_a =$
1.00 – modification factor for lightweight concrete
$f_c =$
20.7 MPa – concrete compressive strength
$h_{ef} =$
50 mm – depth of embedment

Shear resistance (ACI 318-14 – 17.5.1)

$\phi V_{sa} = \phi \cdot 0.6 \cdot A_{se,V} \cdot f_{uta} =$ 69.5 kN ≥ $V =$ 0.0 kN

Where:

$\phi = 0.65$ – resistance factor

$A_{se,V} = 215\ mm^2$ – tensile stress area

$f_{uta} = 827.4$ MPa – specified tensile strength of anchor steel:

$f_{uta} = \min(860\ \text{MPa}, 1.9 \cdot f_{ya}, f_u)$
, where:
$f_{ya} =$
634.3 MPa – specified yield strength of anchor steel
$f_u =$
827.4 MPa – specified ultimate strength of anchor steel

Concrete pryout resistance (ACI 318-14 – 17.5.3)

The check is performed for group of anchors on common base plate

$\phi V_{cp} = \phi \cdot k_{cp} \cdot$ 63.9 kN ≥ $V_g =$ 0.0 kN

Where:

$\phi = 0.65$ – resistance factor

$k_{cp} = 1.00$ – concrete pry-out factor

$N_{cp} =$ 98.2 kN – concrete cone tension break-out resistance in case all anchors are in tension

$V_g =$ 0.0 kN – sum of shear forces of anchors on common base plate

Interaction of tensile and shear forces (ACI 318-14 – R17.6)

$U_{tt}{}^{5/3} + U_{ts}{}^{5/3} =$ 0.28 ≤ 1.0

Where:

$U_{tt} =$ 0.47 – maximum ratio of factored tensile force and tensile resistance determined from all appropriate failure modes

$U_{ts} =$ 0.00 – maximum ratio of factored shear force and shear resistance determined from all appropriate failure modes

Weld sections

Item	Edge	Xu	T_h [mm]	L_s [mm]	L [mm]	L_c [mm]	Loads	F_n [kN]	ϕR_n [kN]	Ut [%]	Status
BP1	COL-bfl 1	E70xx	◢9.0	◢12.7	72	24	LE1	1.3	64.5	2.1	OK
BP1	COL-tfl 1	E70xx	◢9.0	◢12.7	72	24	LE1	4.1	64.6	6.4	OK
BP1	COL-w 1	E70xx	◢4.2	◢6.0	140	23	LE1	2.5	28.7	8.7	OK
BP1	COL-bfl 2	E70xx	◢10.0	◢14.1	199	22	LE1	7.5	68.6	10.9	OK
BP1	COL-bfl 3	E70xx	◢9.0	◢12.7	72	24	LE1	2.0	63.1	3.2	OK
BP1	COL-tfl 2	E70xx	◢9.0	◢12.7	72	24	LE1	7.4	64.9	11.4	OK
BP1	COL-w 2	E70xx	◢4.2	◢6.0	141	23	LE1	4.4	22.7	19.5	OK
BP1	COL-bfl 4	E70xx	◢10.0	◢14.1	200	22	LE1	2.2	63.3	3.5	OK
BP1	RIB1a	E70xx	◢2.1◣	◢3.0◣	74	9	LE1	0.6	5.4	10.7	OK
		E70xx	◢2.1◣	◢3.0◣	75	9	LE1	0.3	6.2	5.5	OK
COL-w 1	RIB1a	E70xx	◢2.1◣	◢3.0◣	100	10	LE1	0.2	5.9	3.3	OK
		E70xx	◢2.1◣	◢3.0◣	99	10	LE1	0.4	5.8	6.2	OK
BP1	RIB1b	E70xx	◢2.1◣	◢3.0◣	75	9	LE1	1.2	6.3	18.6	OK
		E70xx	◢2.1◣	◢3.0◣	74	9	LE1	0.4	6.1	5.8	OK
COL-w 1	RIB1b	E70xx	◢2.1◣	◢3.0◣	100	10	LE1	1.1	5.3	20.8	OK
		E70xx	◢2.1◣	◢3.0◣	100	10	LE1	1.3	5.3	24.0	OK
BP1	RIB2a	E70xx	◢2.1◣	◢3.0◣	74	9	LE1	0.4	5.1	7.3	OK
		E70xx	◢2.1◣	◢3.0◣	75	9	LE1	0.4	6.3	6.8	OK
COL-w 2	RIB2a	E70xx	◢2.1◣	◢3.0◣	100	10	LE1	0.4	5.3	7.4	OK
		E70xx	◢2.1◣	◢3.0◣	100	10	LE1	0.3	5.5	5.7	OK
BP1	RIB2b	E70xx	◢2.1◣	◢3.0◣	74	9	LE1	1.2	6.0	19.9	OK
		E70xx	◢2.1◣	◢3.0◣	74	9	LE1	1.2	5.6	20.6	OK
COL-w 2	RIB2b	E70xx	◢2.1◣	◢3.0◣	100	10	LE1	1.4	5.4	25.1	OK
		E70xx	◢2.1◣	◢3.0◣	100	10	LE1	1.4	5.4	26.3	OK
BP1	RIB3a	E70xx	◢2.1◣	◢3.0◣	74	9	LE1	1.8	5.7	31.1	OK
		E70xx	◢2.1◣	◢3.0◣	74	9	LE1	1.5	5.3	28.5	OK
COL-bfl	RIB3a	E70xx	◢2.1◣	◢3.0◣	100	10	LE1	1.7	5.3	31.3	OK

2											
		E70xx	◢2.1◣	◢3.0◣	100	10	LE1	2.1	5.2	39.8	OK
BP1	RIB3b	E70xx	◢2.1◣	◢3.0◣	74	9	LE1	2.3	5.5	41.8	OK
		E70xx	◢2.1◣	◢3.0◣	74	9	LE1	2.1	6.2	33.6	OK
COL-bfl 2	RIB3b	E70xx	◢2.1◣	◢3.0◣	100	10	LE1	2.3	5.3	42.8	OK
		E70xx	◢2.1◣	◢3.0◣	100	10	LE1	2.2	5.3	41.7	OK
BP1	RIB4a	E70xx	◢2.1◣	◢3.0◣	75	9	LE1	0.3	6.4	4.8	OK
		E70xx	◢2.1◣	◢3.0◣	74	9	LE1	0.4	5.6	6.5	OK
COL-bfl 4	RIB4a	E70xx	◢2.1◣	◢3.0◣	100	10	LE1	0.6	5.2	12.2	OK
		E70xx	◢2.1◣	◢3.0◣	100	10	LE1	0.4	5.5	7.6	OK
BP1	RIB4b	E70xx	◢2.1◣	◢3.0◣	74	9	LE1	0.5	6.1	7.8	OK
		E70xx	◢2.1◣	◢3.0◣	74	9	LE1	0.6	6.0	9.4	OK
COL-bfl 4	RIB4b	E70xx	◢2.1◣	◢3.0◣	99	10	LE1	0.7	5.6	12.2	OK
		E70xx	◢2.1◣	◢3.0◣	99	10	LE1	0.5	5.9	8.7	OK

Symbol explanation

T_h Throat thickness of weld
L_s Leg size of weld
L Length of weld
L_c Length of weld critical element
F_n Force in weld critical element
ϕR_n Weld resistance AISC 360-16 J2.4
Ut Utilization

Detailed result for COL-bfl 2 / RIB3b

Weld resistance check (AISC 360-16: J2-4)

$\phi R_n = \phi \cdot F_{nw} \cdot$ 5.3 kN ≥ $F_n =$ 2.3 kN

Where:

$F_{nw} = 334.9$ MPa – nominal stress of weld material:

$F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$

, where:

$F_{EXX} =$

482.6 MPa – electrode classification number, i.e. minimum specified tensile strength

$\theta =$

27.5° – angle of loading measured from the weld longitudinal axis

$A_{we} = 21\ mm^2$ – effective area of weld critical element

$\phi = 0.75$ – resistance factor for welded connections

Concrete block

Item	Loads	A_1 [mm2]	A_2 [mm2]	σ [MPa]	Ut [%]	Status
CB 1	LE1	87956	123718	1.5	10.8	OK

Symbol explanation

A_1 Loaded area
A_2 Supporting area
σ Average stress in concrete
Ut Utilization

Detailed result for CB 1

Concrete block compressive resistance check (AISC 360-16 Section J8)

$\phi_c f_{p,max}$ =	13.6	MPa	≥	σ =	1.5	MPa

Where:

$f_{p,max}$ = 20.9 MPa – concrete block design bearing strength:

$f_{p,max} = 0.85 \cdot f_c \cdot \sqrt{\frac{A_2}{A_1}} \leq 1.7 \cdot f_c$

, where:

f_c =

20.7 MPa – concrete compressive strength

A_1 =

87956 mm^2 – base plate area in contact with concrete surface

A_2 =

123718 mm^2 – concrete supporting surface

ϕ_c = 0.65 – resistance factor for concrete

Shear in contact plane

Item	Loads	V [kN]	ϕV$_r$ [kN]	μ [-]	Ut [%]	Status
BP1	LE1	25.0	33.5	0.40	74.7	OK

Symbol explanation

V Shear force
ϕV$_r$ Shear resistance
μ Coefficient of friction between base plate and concrete block
Ut Utilization

Detailed result for BP1

Base plate shear resistance check (ACI 349 – B.6.1.4)

$\phi_c V_r = \phi_c \cdot \mu \cdot$ 33.5 kN ≥ V = 25.0 kN

Where:

ϕ_c = 0.65 – resistance factor for concrete

μ = 0.4 – coefficient of friction between base plate and concrete

C = 128.7 kN – compressive force

Buckling

Buckling analysis was not calculated.

Code settings

Item	Value	Unit	Reference
Friction coefficient - concrete	0.40	-	ACI 349 – B.6.1.4
Friction coefficient in slip-resistance	0.30	-	AISC 360-16 J3.8
Limit plastic strain	0.05	-	
Weld stress evaluation	Plastic redistribution		
Detailing	No		
Distance between bolts [d]	2.66	-	AISC 360-16 – J3.3
Distance between bolts and edge [d]	1.25	-	AISC 360-16 – J.3.4
Concrete breakout resistance check	Both		
Base metal capacity check at weld fusion face	No		AISC 360-16: J2-2
Cracked concrete	Yes		ACI 318-14 – Chapter 17
Local deformation check	No		
Local deformation limit	0.03	-	CIDECT DG 1, 3 - 1.1
Geometrical nonlinearity (GMNA)	Yes		Analysis with large deformations for hollow section joints

STRUCTURAL ANALYSIS AND DESIGN OF ROOF TRUSS

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1 INTRODUCTION

1.1 Background:

Structural analysis, design, detailing of the different structural components of the stadium and separate report preparation has already been carried out.

1.2 About the report:

This report elaborates the following works, which is a part of detailed structural analysis and design of the roof truss:

- Modelling of the roof for structural analysis.
- Structural analysis using structural analysis software ETABS 2019.
- Sectional design of structural component.
- Calculations for detailing as necessary.

1.3 Analysis overview:

The structure has been primarily modelled and analyzed in a computer software “ETABS 2019”. The software has very good analysis and design capability which are verified in the verification problems included in the package. It is a Finite Element Method (FEM) based software and requires modelling of the structure by finite-elements. Global analysis of the truss was carried out with the help of the software, while element level design is further done with additional manual calculations, using spreadsheets. All the details are elaborated in this report.

2 DETAIL PARAMETERS OF STRUCTURE

The various details and parameters used in the analysis of the building and roof truss are presented in this section.

3.1. General parameters of the building

Building type	Stadium
Location	Suryabinayak
Number of storey	3 storey+ roof truss
Basement	none
Floor height	4m (Plinth-first floor level)
	3m (first to second floor level)
	3m (second to third floor level)
	5.766m (third to Top of the roof)
Height of the building	15.766m

3.2. Geometry of Truss

Truss Type	Cantilever truss
Spacing of truss	6300 mm
Span of Truss	6823 mm
Slope of top chord	6.7°
Slope of bottom chord	11.6°
RCC Column	525mm*525mm

3.3 Material properties

Rebar grade	Fe 500(Ductile) for all reinforcement (Minimum 15% elongation required for rebar)
Structural Steel Grade	Fe410 (E250)

2.1 Design Basis:

The building is designed following the standard codes and norms as follows:

- IS 875 Part-1 :1987 (Code of Practice for Design Loads-Part 1: Dead Loads) for adaptation of specific weight of building materials.
- IS 875 :1987 (Code of Practice for Design live Loads-Part 2: Live loads) for adaptation of live load.
- IS 875 Part-3 :2015 and NBC104:1994 for wind load calculation.
- IS800:2007 for structural design of steel members

3 LOADING

3.1 Dead load:

The dead load included in the design are dead load of the CGI sheet and self-weight of the purlin. The adopted spacing of truss is 6300mm as mentioned in the architectural drawing, and maximum value spacing of purlin 1132.5mm is taken for load calculation at each joint of purlin.

Table 4-1: Dead load calculation at purlin joint

DL of CGI sheet	=	0.25	kN/m^2
DL of CGI along purlin	=	0.28	kN/m
Corresponding joint load	=	**1.78**	kN
Purlin size (ISMC 125)	=	12.7	kg/m
Self wt. of purlin	=	0.14	kN/m
Self wt. of purlin in each joint	=	**0.89**	kN
Total wt. along the purlin	=	0.42	kN/m
Corresponding joint load	=	2.67	kN

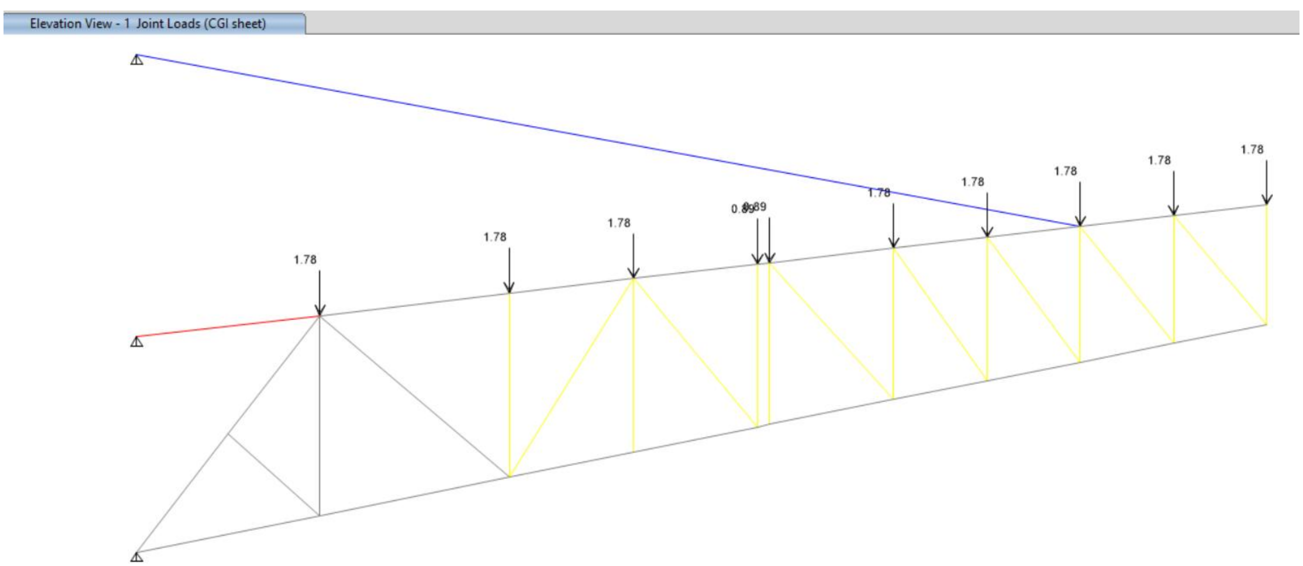


Figure 4- 1: Application of joint load at each joint

3.2 Live loads:

These are the loads that may vary its intensity and/or position during design life. Live loads for the roof are calculated as per the functional requirement as specified in IS875 code.

Table 4-2 Live load calculation

1	Floor live load	Not applicable
2	Roof @ 6.7 degrees slope (access is not provided)	0.75 kN/m^2
3	Corresponding joint load	5.351 kN
4	Snow and other	Not applicable

3.3 Wind Load

The wind load design parameters have been considered in reference with NBC104:1994 and IS875 (part3):2015 and are presented as follows.

Table 4-3: Wind load calculation for roof truss

B. Wind load in Building

Individual structural elements (roofs.)

Vb	47	m/s		Vz	**50.29**	m/s
Structure Class	Important			K1	1.07	
Life	50	yrs				
Terrian cat	Cat 2			K2	1	
Height	<10	m				
Topography slope	<3	deg		K3	1	
Importance factor	off-coast, Imp.			K4	1	

Design Wind Pressure		
$P_z=0.6*V_z^2$	**1517.45**	N/m^2

For Roof (considering, Monoslope roof)		
h	12.883	m
w	6.843	m
h/w	1.88	<2
roof angle	6.7	deg

Internal pressure coeff.(CL 7.3.2)	
Cpi	**0.2**
	-0.2
External pressure ceoff. (Table 7)	
Cpe	-1

Forrce Cacualtion per unit area

$F= (C_{pe}-C_{pi})*P_z$		
Force per unit area =	-1.82	kN/m^2

3.4 Load combination

The load combination has been adopted as per the codal provision of IS800:2007.

Table 4 Partial Safety Factors for Loads, γ_f, for Limit States

(*Clauses* 3.5.1 *and* 5.3.3)

Combination	Limit State of Strength					Limit State of Serviceability			
	DL	LL[1]		WL/EL	AL	DL	LL[1]		WL/EL
		Leading	Accompanying				Leading	Accompanying	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
DL+LL+CL	1.5	1.5	1.05	—	—	1.0	1.0	1.0	—
DL+LL+CL+	1.2	1.2	1.05	0.6	—	1.0	0.8	0.8	0.8
WL/EL	1.2	1.2	0.53	1.2					
DL+WL/EL	1.5 (0.9)[2]	—	—	1.5	—	1.0	—	—	1.0
DL+ER	1.2 (0.9)[2]	1.2	—	—	—	—	—	—	—
DL+LL+AL	1.0	0.35	0.35	—	1.0	—	—	—	—

[1] When action of different live loads is simultaneously considered, the leading live load shall be considered to be the one causing the higher load effects in the member/section.

[2] This value is to be considered when the dead load contributes to stability against overturning is critical or the dead load causes reduction in stress due to other loads.

Abbreviations:

DL = Dead load, LL = Imposed load (Live loads), WL = Wind load, CL = Crane load (Vertical/Horizontal), AL = Accidental load, ER = Erection load, EL = Earthquake load.

NOTE — The effects of actions (loads) in terms of stresses or stress resultants may be obtained from an appropriate method of analysis as in **4**.

4 MODELLING OF ROOF TRUSS

The Truss structure has been modeled and analyzed in ETABS 2019. FEM has been utilized for modeling. All the members of truss are modeled using line element. The section properties and loads have been appropriately specified. The supports of truss are assumed to be pinned connection with the RCC column. All the load combinations are converted to non-linear static to account tension only load on the steel rod of dia. 40mm.

4.1 3D view of Model:

The 3D view of FEM model of the roof truss is as shown.

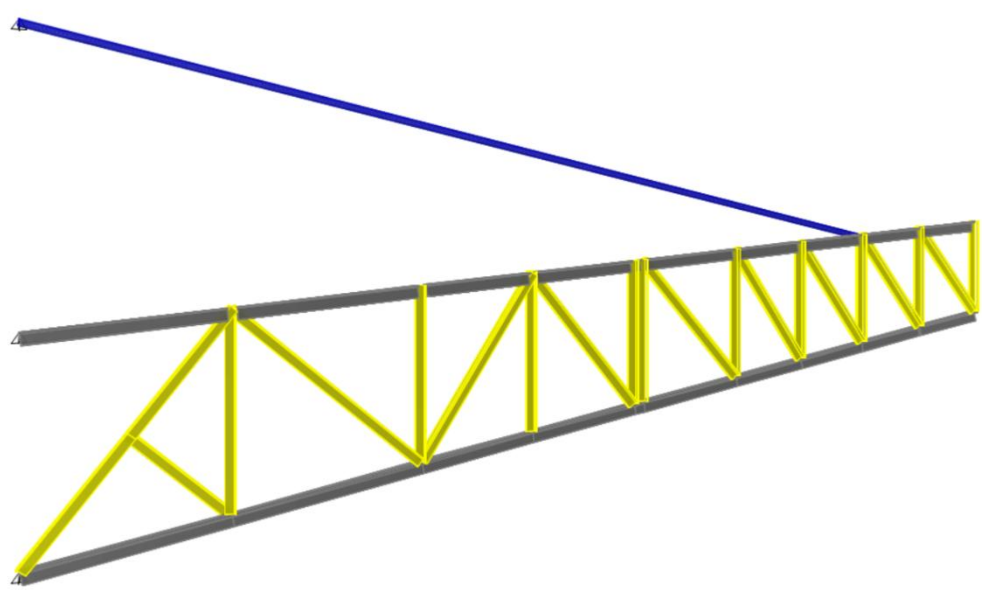


Figure 5- 1:3D view of FEM Model OF ROOF TRUSS

4.2 Rafter and purlin sections

Standard steel sections, and built-up sections (double angles) are used mostly. The used sections are as listed below.

- Double ISA 100x65x8,
- Double ISA 90x60x8
- Double ISA 75x45x8
- Double ISA 60x40x5,
- Steel rod of dia. 40mm
- ISMC 150

5 ANALYSIS RESULTS AND DESIGN OF ROOF TRUSS

The roof truss has been analyzed with appropriate geometry, material properties, loads and analysis procedure using FEM software. The major analysis results have been presented below.

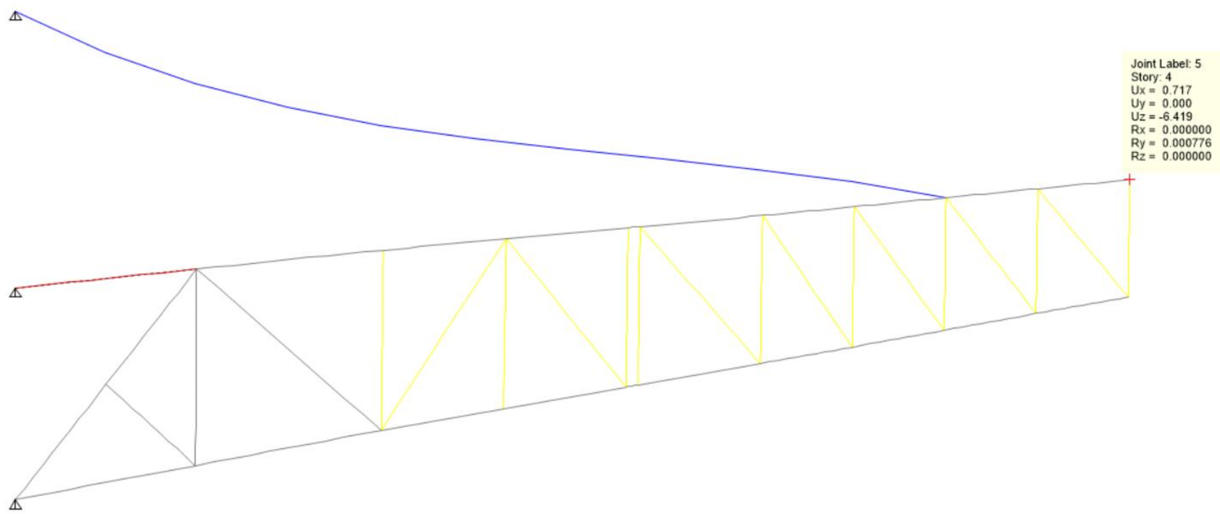


Figure 6- 1: Deformed shape of the truss (DL+LL)

Similarly, maximum deflection of roof was found to be about 7mm under DL+LL. Considering span of 6.843m, the deflection is 1:977, which is well below the limit of span/300 as per IS800:2007, table 6.

5.1 Sample Internal Forces:

Few representative internal force diagrams are presented in this section.

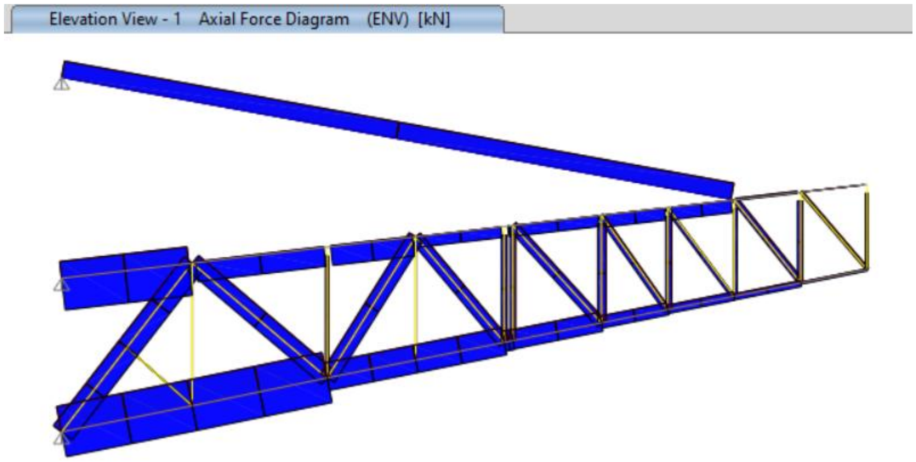


Figure 6- 2: Envelope of axial force diagram

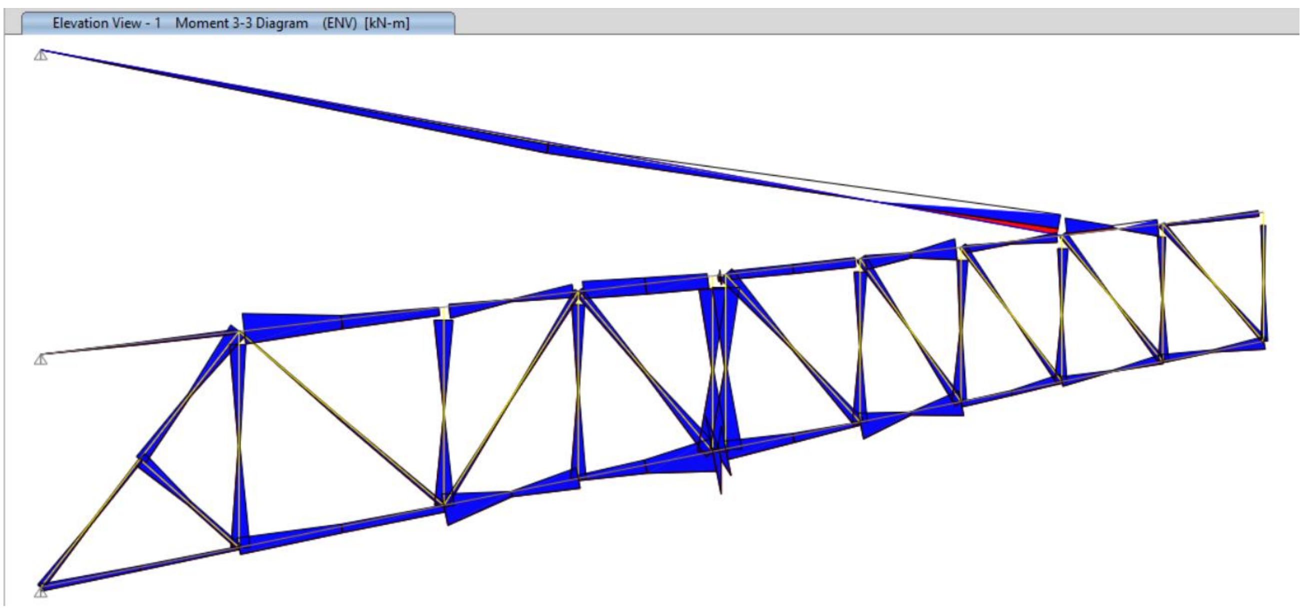
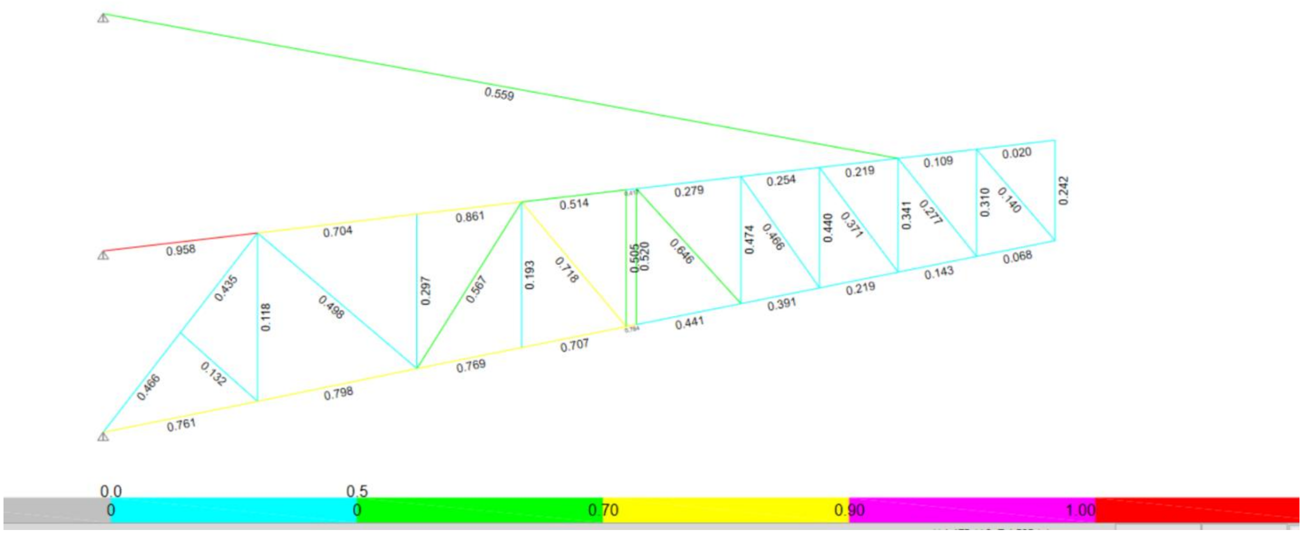


Figure 6- 3: Envelope of M3-3 diagram

6 DESIGN OF STRUCTURE

The structure has been designed using computer software in accordance with IS800:2007. The detailed design results are utilized for the design and the different design outputs are discussed in the following sections.



6.1 Design of purlins:

Purlins span from truss to truss. ISMC 150 channel purlins are used, continuous over the trusses. The design is based on the minimum slenderness ratio, and the strength requirements under various load combinations.

6.2 Sample Design Detail of a Purlin:

The sample design calculation with maximum spacing of 1132.5 mm is presented below.

Table 7-1: Wind load calculation to design purlin

Perpendicular to sheeting	
Load combination	Load (kN/m)
$1.2DL+1.2LL+1.2WL_u$	-0.93
$1.5DL+1.5WL_u$	-2.42
$0.9DL+1.5WL_u$	**-2.69**
$1.2DL+1.2LL+0.6WL_u$	0.31

Parallel to sheeting	
Load combination	Load (kN/m)
$1.2DL+1.2LL+1.2WL_u$	0.17
$1.5DL+1.5WL_u$	0.07
$0.9DL+1.5WL_u$	**0.35**
$1.2DL+1.2LL+0.6WL_u$	0.17

Table 7-2: Sample design calculation of purlin design

S.N.	Design of purlin (ISMC 150)			
1)	**Geometry of truss**			
	Spacing of truss	=	6.3	m
	Span of truss	=	6.843	m
	Spacing of purlins	=	1.1325	m
	f_y	=	250	mpa
	γ_{mo}	=	1.1	
2)	**Load calculation**			
a)	**CGI and purlin**			
	DL of CGI along purlin	=	0.28	kN/m
	Self-weight of purlin (ISMC150)	=	0.16	kN/m
	Total DL	=	0.45	kN/m
	DL along the sheet	=	0.05	kN/m
	DL perpendicular to sheet	=	0.44	kN/m
b)	**Live load**			
	Live load along the purlin (LL)	=	0.85	kN/m
	LL along the sheet	=	0.10	kN/m
	LL perpendicular to sheet	=	0.84	kN/m
c)	**Wind load**			
	Uplift along purlin (Wl_u)	=	-2.06	kN/m
d)	**Factored load**			
	Factored load perpendicular to sheet (w_v)	=	2.69	kN/m
	Factored load parallel to sheet(w_h)	=	0.40	kN/m
3)	**Bending moment calculation**			
	M_{zz}=Mv	=	10.68	kN-m
	M_{yy}=Mh	=	1.60	kN-m
4)	**Section classification (ISMC 150)**			
	h	=	150	mm
	bf	=	75	mm
	t_f	=	9.00	mm
	t_w	=	5.40	mm
	r_1	=	10.00	mm
	d [h-(2*t_f+r_1)]	=	122.00	mm
	b= b_f/2	=	37.50	mm
	ε	=	15.08	
	b/t_f	=	4.17	<9.4
	d/t_w	=	22.59	<42
	So, the section is plastic			

4)	Assuming ISMC 150 Section			
	Shape factor	=	1.1533	
	Z_{yy}	=	19400	mm^3
	Provided plastic section modulus required (Z_{pz})	=	119820	mm^3
	Provided plastic section modulus required (Z_{py})	=	22374.02	mm^3
	M_{dz}	=	27.23	kN-m
	M_{dy}	=	5.09	kN-m
5)	Check for combined bending capacity			
	$\left(\frac{M_y}{M_{ndy}}\right)^{\alpha_1} + \left(\frac{M_z}{M_{ndz}}\right)^{\alpha_2} \leq 1.0$	=	**0.707**	
			SAFE	

6.3 Design of Truss:

The truss is modelled combined with the building. Truss is anchored to columns at both ends. Appropriate sections are provided for different elements to obtain most economical and optimum results. The member details and utilization ratio of the final design is depicted by the following figure.

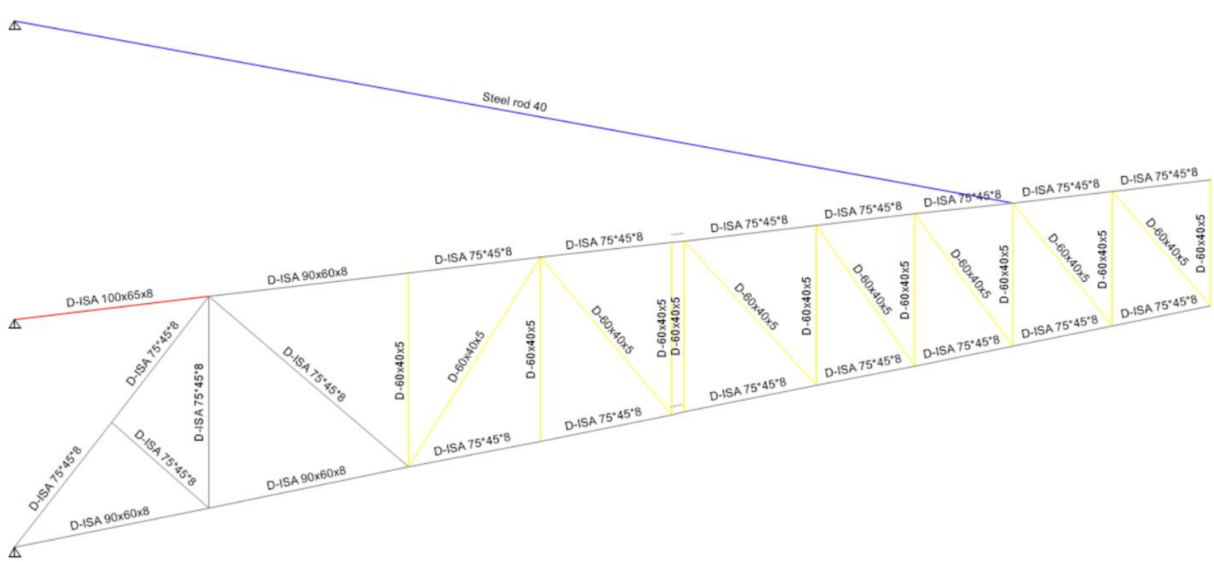


Figure 7- 1: Member details of roof truss

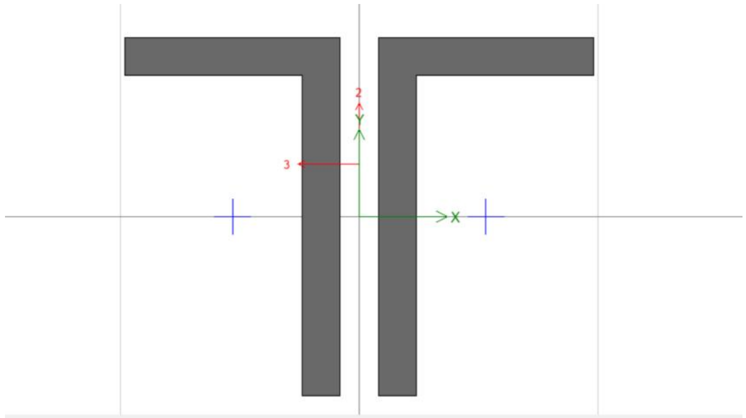


Figure 7- 2: Double angle section ISA 100x65x8

The member details for truss are as follows:

- Top chord: Double ISA 75x45x8, Double ISA 90x60x8, Double ISA 100x65x8 with 12mm gap for gusset connection.
- Bottom chord: Double ISA 75x45x8 with 12mm gap
- Diagonal members: Double ISA 60x40x5, Double ISA 75x45x8 with 12mm gap
- Middle vertical and additional tie: Double ISA 60x40x5, Double ISA 75x45x8

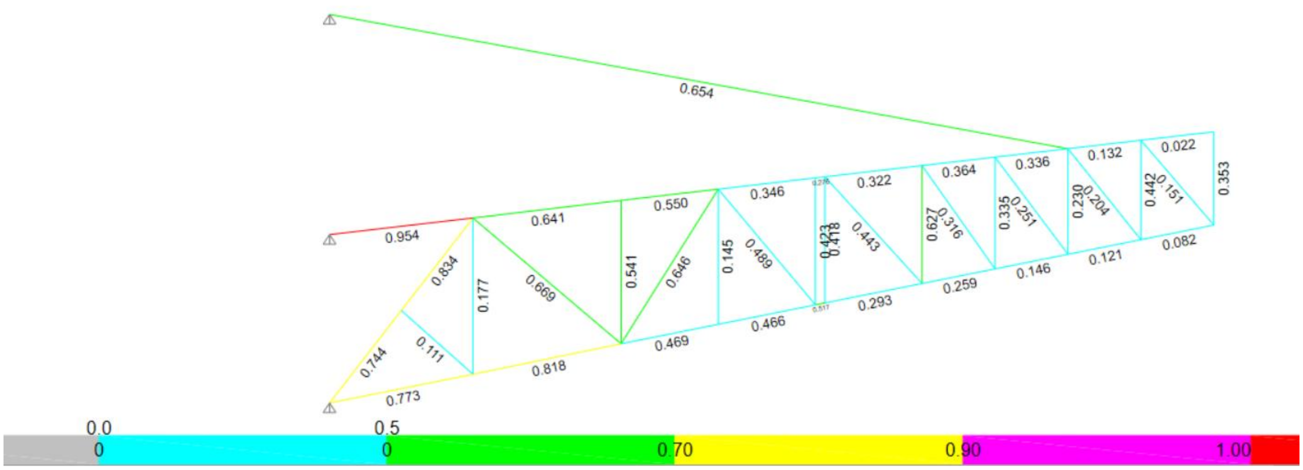


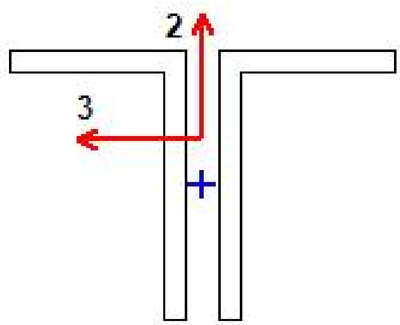
Figure 7- 3: Design utilization ratio in truss

Above figure shows the utilization ration in truss members that shows utilization upto about 95% indicating efficient design. Several other members have low utilization ratio, but the sizes are determined for the necessary slenderness ratios, uniformity in design for practicality in construction.

7.3.1 Sample Design Detail of a truss top-chord:

ETABS Steel Frame Design

IS 800:2007 Steel Section Check (Strength Summary)



Element Details (Part 1 of 2)

Level	Element	Unique Name	Location (mm)	Combo	Design Type	Element Type	Section
4	B3	11	0	DStlS17	Beam	Special Moment Frame	D-ISA 100x65x8

Element Details (Part 2 of 2)

Classification	Rolled
Class 3	No

Design Code Parameters

γ_{M0}	γ_{M1}	A_n / A_g	LLRF	PLLF	Stress ratio Limit
1.1	1.25	1	1	0.75	0.95

Section Properties

A (cm^2)	I_{zz} (cm^4)	r_{zz} (mm)	$Z_{e,zz}$ (cm^3)	$A_{v,z}$ (cm^2)	$Z_{p,zz}$ (cm^3)	I_{yz} (cm^4)	I_t (cm^4)
25.1	256.7	32	38.5	16.7	69	0	5.3

J (cm^4)	I_{yy} (cm^4)	r_{yy} (mm)	$Z_{e,yy}$ (cm^3)	$A_{v,y}$ (cm^2)	$Z_{p,yy}$ (cm^3)	I_w (cm^6)	h (mm)
5.3	206.3	28.7	29.1	14.2	54.8		100

Material Properties

J (cm^4)	I_{yy} (cm^4)	r_{yy} (mm)
5.3	206.3	28.7
E (MPa)	f_y (MPa)	f_u (MPa)
210000	250	410

Stress Check Forces and Moments

Location (mm)	N (kN)	M_{zz} (kN-m)	M_{yy} (kN-m)	V_y (kN)	V_z (kN)	T_o (kN-m)
0	-454.3904	0	0	-0.9306	0	0

PMM Demand/Capacity (D/C) Ratio 7.1.2

D/C Ratio = P / P_d
0.958 = 0.958

Basic Factors

Buckling Mode	K Factor	L Factor	L Length (mm)	KL/r
Major (z-z)	1	0.967	1077.9	33.716
Major Braced	1	0.967	1077.9	33.716
Minor (y-y)	1	0.967	1077.9	37.614
Minor Braced	1	0.967	1077.9	37.614
LTB	1	0.967	1077.9	37.614

Axial Force Design

	N Force kN	T_d Capacity kN	N_d Capacity kN	P_{dy} Capacity kN	P_z Capacity kN	P_d Capacity kN
Axial	-454.3904	570.9091	570.9091	508.3183	521.196	474.383

T_{dg} kN	T_{dn} kN	$N_{cr,T}$ kN	$N_{cr,TF}$ kN	A_n /A_g Unitless	N /N_d Unitless
570.9091	741.5424	2309.6713	2309.6713	1	0.796

Design Parameters for Axial Design

	Curve	α	f_{cc} (MPa)	λ	Φ	χ	f_{cd} (MPa)
Major (z-z)	c	0.49	1823.27	0.37	0.61	0.913	207.48
MajorB (z-z)	c	0.49	1823.27	0.37	0.61	0.913	207.48
Minor (y-y)	c	0.49	1464.91	0.413	0.638	0.89	202.36
MinorB (y-y)	c	0.49	1464.91	0.413	0.638	0.89	202.36
Torsional TF	c	0.49	919.46	0.521	0.715	0.831	188.85

Moment Designs

	M Moment kN-m	M_{span} Moment kN-m	$M_{d(yield)}$ Capacity kN-m	M_{dv} Capacity kN-m	M_{nd} Capacity kN-m	$M_{d(LTB)}$ Capacity kN-m
Major (z-z)	0	0.059	8.7479	8.7479	8.7479	8.4846
Minor (y-y)	0	0	6.6029	6.6029	6.6029	

	Curve	α_{LT}	λ_{LT}	Φ_{LT}	χ_{LT}	C_1	M_{cr} (kN-m)
LTB	c	0.49	0.259	0.548	0.97	1.144	143.2097

	C_{my}	C_{mz}	C_{mLT}	k_z	k_y	K_{LT}	M_y / M_{dy}	M_z / M_{dz}	α_1	α_2
Factors	1	0.903	0.903	1.148	1.19	0.964	0	0	2	2

Shear Design

	V Force (kN)	V_d Capacity (kN)	T_o Capacity (kN-m)	Stress Ratio	Status Check
Major (y)	0.9306	186.6339	0	0.005	OK
Minor (z)	0	219.6436	0	0	OK

Shear Design				
	V_p (kN)	k_v (Unitless)	Λ_W (Unitless)	τ_b (MPa)
Reduction	186.6339	0	0	1

End Reaction Major Shear Forces

Left End Reaction (kN)	Load Combo	Right End Reaction (kN)	Load Combo
		0.7443	DStlS17

6.4 Protection of steel works:

It is recommended that, all steel sections be galvanized, after shop fabrication, and bolted at site. Provisions shall be made to avoid water running down into any structural members. Gutters shall be provided in roof with proper run-down pipes, to avoid any splashing of water to any structural members. Colored metal roofing (min 0.5mm thick, with thick galvanization) is preferred to avoid creep, fatigue and deterioration in long run.

7 CONCLUSION

A parapet roof structure is designed using a computer program, ETABS 2019. All the required design details of the buildings have been calculated and presented.

Best-approaches have been adopted for economical design, yet fulfilling all the requirements for building design. Hence the structure is appropriately designed satisfying the design criteria. However, it is very important to follow all the specifications, various detailing specifications and requirements of ductile detailing during construction to actually make the structure perform as expected. Strict Control over quality of materials and workmanship is required.