Faculty of Engineering & Information Technology
Department of Civil Engineering

Graduation Project Report II

Analysis and Design of Princess Tower in Dubai, United Arab Emirates

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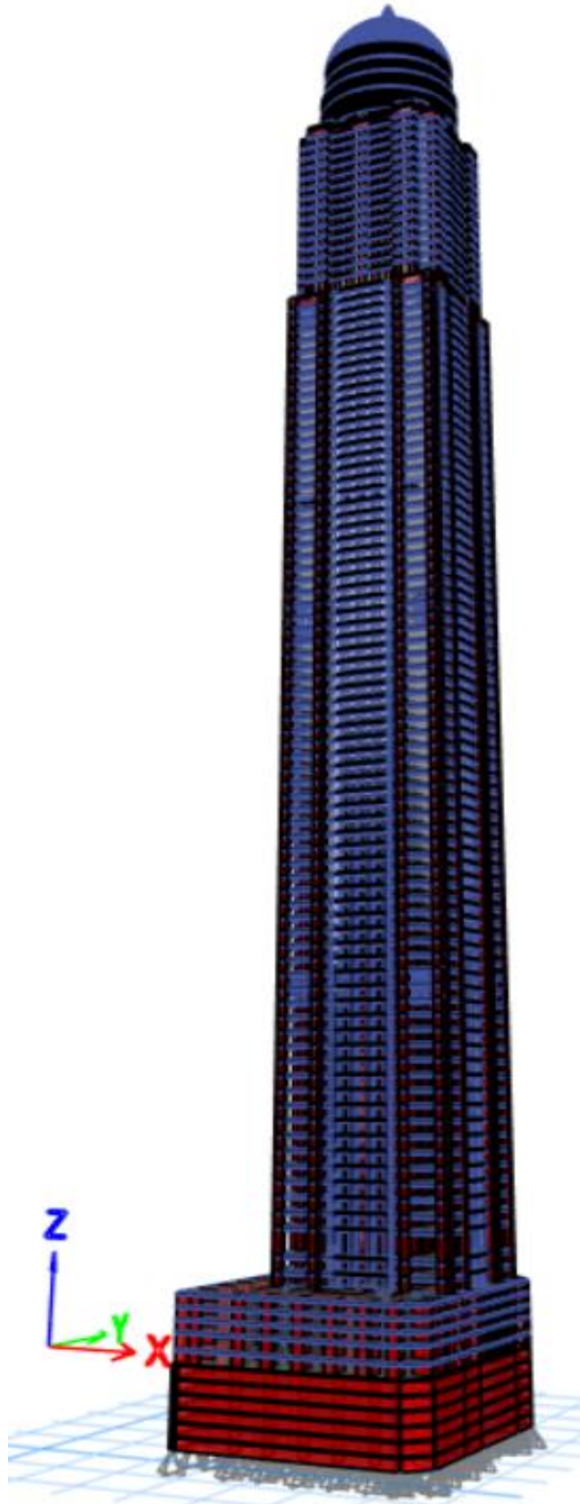
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DISCLAIMER

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ABSTRACT

The Princess Tower, an iconic architectural landmark, has graced the skyline for decades, symbolizing both heritage and modernity. This paper presents a comprehensive exploration of the concept and process of redesigning the Princess Tower to transform it into a symbol of contemporary elegance. The aim is to merge its rich historical significance with cutting-edge design.

Since it is in Dubai, this made it one of the most well – known buildings around the globe for being one of the highest residential structures all over the world.

Additionally, this skyscraper boasts an architectural height of 414 meters above ground level and a total floor area spanning 171,175 square meters. It comprises 101 stories, including 6 basement levels.

To elaborate further, the tower is structured as follows: 6 basement floors, a ground floor, and 2 health clubs. Above that, there are 16 floors with a uniform design, followed by 2 service floors. Continuing upward, there are 26 more floors identical in design to the previous ones, each featuring 2 service floors. The tower then concludes with 18 more floors of the same typical design.

In another section of the building, 8 floors were uniquely designed, distinct from the stories below, and included 1 service floor. Furthermore, 11 floors were constructed with a design that sets them apart from the floors beneath, and these floors house 2 entertainment clubs along with a business center.

Lastly, at the pinnacle of the skyscraper, there are 4 floors characterized by a completely contrasting design, topped off with 1 service floor.

Throughout the first part of this project, it focused on structural analysis under gravity loads effect and some additions and modification that had been added to the original tower plans.

To continue with the design part, seismic analysis must be taken into consideration with different methodologies such as: Equivalent Static Method, Response Spectrum, … etc.

Also, performance-based analysis must be undergone along with push-over analysis.

All these analyses methods and results will be used in full structural design for all structural elements with detailed drawings.

Finally, Palestinian Building Code will be referenced as per Graduation Project I to ensure safety, structural integrity, and serviceability.

1. INTRODUCTION

The structural integrity of high-rise buildings plays a pivotal role in the safety and functionality of urban landscapes. As one of the most iconic structures on the Dubai skyline, the Princess Tower stands tall and exemplifies modern architectural excellence. With its stunning height and unique design, this residential skyscraper not only offers breathtaking views but also poses numerous engineering challenges. In this report, delve into the comprehensive structural analysis of the Princess Tower, exploring the methodologies used to ensure its safety, stability, and resilience in the face of various environmental and structural forces. By studying this remarkable feat of engineering, valuable insights are shown into the intricate balance between design, construction, and safety considerations in the creation of skyscrapers that stand the test of time.

1.1 Background

Princess Tower, completed in 2012, is a luxurious residential building that stands as an architectural marvel in Dubai's skyline. It was developed by Tameer Holding Investment LLC, a prominent real estate developer in the Middle East. The tower's design was created by architects from Eng. Adnan Saffarini Office, known for their innovative and iconic projects in the region.

Situated in Dubai Marina, the Princess Tower offers stunning panoramic views of the Arabian Gulf, Palm Jumeirah, and the bustling cityscape of Dubai. The Marina district itself is a vibrant waterfront community that features a picturesque promenade, yacht club, retail outlets, restaurants, and various entertainment options, making it an attractive place to live.

1.2 Objectives

The main objective of this part of the graduation project is to ensure that the building still sustains its required serviceability, undergoing applied gravity loads only such as: live, dead, and superimposed dead loads, so the project is well prepared in structural terms to undergo seismic analysis and full design process. More detailed calculations to be explained throughout the report.

1.3 Significance of the project

This project focuses on high rise buildings standards and structural systems. It helps in understanding the way high rise structures are constructed and designed and how different they are from traditional buildings. Moreover, this project case of high-rise buildings behaves completely in an unusual way, so more advanced and complex structural systems precisely imposed for high rise buildings take place.

1.4 Plans Allocation

In order to start project analysis process, full structural and architectural plans should be obtained and understood. This was done by official contact to construction executing office (Eng. Adnan Saffarini Engineering Office) and kindly full plans were sent. Furthermore, structural plans were displayed on AutoCAD drawing software and some modification was done to allow uploading plans onto structural modelling software.

1.5 Codes and Specifications

Structures are designed by using different practice codes and specifications that control the design processes and variables, also qualify design variables at different loading conditions.

The following codes and standards are used:

- ACI 318-19, (American Concrete Institute): building code requirements for structural concrete and commentary.
- ASCE 7-22, (American Society of Civil Engineers)
- ASCE 7-16, (American Society of Civil Engineers)
- TBI 2017, (Tall Buildings Initiative): guidelines for performance – based seismic design of tall buildings.

1.6 Materials

Materials are divided into two types: Structural materials and non-structural materials.

1. Structural Materials:
    - Concrete, $f'c = 55\ MPa (For\ the\ \ first\ model)$

      $70\ MPa\ (For\ the\ second\ model)$

      $\gamma_{concrete} = 25\ KN/m^3$.
    - Steel, $fy = 420\ MPa\ , \gamma = 77 \frac{KN}{m^3}$.

2. Non-Structural Materials

Material	Unit Weight (KN/m^3)
Gypsum in Partition Walls	**6**
Metal Studs in Partition Walls	**0.3**
Aluminum Coating	**27**
Plywood in Parking Floors	**6.3**
Contact Cement in Parking Floors	**8.6**
Ethylene Propylene Diene Monomer (EPDM) Rubber in Parking Floors	**15.75**
Oak Wood	**7.7**
Modified Silane (MS) Polymer Adhesive	**9**

Table 1.6.1 – Materials Unit Weights

1.7 Load cases and combinations

1.7.1 Load Cases

There are three main load types to be considered in this part of the graduation project:

- Dead load, which is the **structure's self-weight.**
- Superimposed dead load is calculated as follows:
- **S.D Load for Partition Walls**

$$Gypsum\ in\ Partition\ Walls\ Unit\ Weight = 6\ kN/m^3$$

$$Metal\ Studs\ in\ Patition\ Walls\ Weight = 0.3\ kN/m^2$$

$$Thickness\ of\ Partitions = \frac{0.1 + 0.2}{2} = 0.15\ m$$

$$Total\ Length\ in\ the\ most\ partitions - intensive\ floor = 382.3\ m$$

$$Area\ of\ Floors = 1354\ m^2$$

$Average\ Floor\ Height = 3.5\ m$

$Total\ Load\ from\ Partitions = (0.3 + 6 * 0.15) * 3.5 * 382.3 = 1605.66\ kN$

$$\boldsymbol{Total\ Distributed\ Load\ on\ Floors\ from\ Partitions = \frac{1605.66}{1354} = 1.2\ kN/m^2}$$

- **S.D Load for Parking Floors**

Rubber Flooring is used in Parking Floors, EPDM Rubber flooring rolls are installed on a plywood (OSB Panel Type) using contact cement.

$Thickness\ of\ the\ Plywood\ is\ taken\ to\ be\ 0.75\ inches.$

$Weight\ of\ the\ Plywood\ is\ 80\ pounds\ per\ standard\ sheet\ (4\ feet * 8\ feet)$

$Weight\ of\ the\ plywood\ is\ 2.5\ psf$ thus $weight\ of\ the\ plywood\ is\ 0.12\ kN/m^2$

$Unit\ Weight\ of\ the\ Contact\ Cement\ is\ 8.6\ kN/m^3$

$The\ Thickness\ of\ the\ Contact\ Cement\ is\ taken\ to\ be\ 0.2\ mm$

$Contact\ Cement\ Weight = 8.6 * 0.0002 = 0.00172\ kN/m^2$

$Ethylene\ Propylene\ Diene\ Monomer\ (EPDM)\ Rubber\ Rolls\ are\ used\ for\ parking\ floors$

$The\ Rubber\ Flooring\ thickness\ is\ taken\ to\ be\ 0.5\ inch\ (12.7mm)\ (due\ to\ heavy\ traffic)$

$Rubber\ Rolls\ Weight = 4\ psf = 0.2\ kN/m^2$

$\boldsymbol{S.D\ Load\ for\ Parking\ Floors = 0.12 + 0.00172 + 0.2 = 0.322\ kN/m^2}$

- **S.D Load for False Ceilings**

$False\ Ceilings\ are\ composed\ of\ gypsum\ boards\ and\ metal\ framework$

$\boldsymbol{Weight\ of\ false\ Ceilings\ is\ 10\ psf = 0.4788\ kN/m^2}$

- **S.D Load for Parquet Tiles**

  First thing to begin with, is that parquet tiles have different types based on the type of wood used, there are several types of wood that can be used in parquet tiles, such as, oak wood, maple, cherry, walnut, etc.... To complete the installation process of the parquet tiles certain floor adhesives shall be used, such as, polyurethane adhesive, modified silane polymer adhesive, water-based wood adhesive, etc...

In this project, oak wood parquet tiles with modified silane (MS) polymer adhesive were chosen.

The thickness of the MS adhesive is taken to be 22 mm, and the thickness of the parquet tiles is taken to be 20mm.

The unit weight of the oak wood is 7.7 kN/m^3
The unit weight of the MS adhesive is 9 kN/m^3

So, the superimposed dead load = $(7.7 * 0.02) + (9 * 0.022) = \mathbf{0.352\ kN/m^2}$

Element Type	S.D Load (kN/m^2)
Partition Walls	1.2
Parking Floors	0.322
False Ceilings	0.4788
Parquet Tiles	0.352

Table 2.7.1.1 – Elements Superimposed Loads

- **Live load could be taken 3 KN/m^2 for high-rise residential buildings.**
- Finally, for load combinations, the envelope used for analysis in this project is named **"Gravity Envelope**," since in Graduation Project I the objective is to ensure structure serviceability and safety for **gravity loads only.** For this reason, two load combination were taken into consideration:

  Ultimate load = 1.4D+1.4SD

  Ultimate load = 1.2D+1.2SD+1.6L.

1.7.2 Seismic Design Loads

The Risk Category is III

The design spectral response acceleration parameter at short period, S_{ds} and S_{d1} are needed.

But $S_{ds} = \frac{2}{3} * S_{MS}$ and $S_{d1} = \frac{2}{3} * S_{M1}$

But $S_{MS} = F_a * S_s$ and $S_{M1} = F_v * S_1$

According to a site investigation report, the soil in Al-Nahda, Dubai, United Arab Emirates is class C.

Location	S_s	S_1	T_L (s)
Dubai	0.51	0.18	24

Table F.13 Enhanced performance seismic ground motion parameters for Dubai (site class B)

Table 3.7.2.1 – Seismic Ground Motion Parameters for Dubai

Table F.13 in Dubai Building Code gives the values of S_s, S_1 and the Long Period Transition Period T_L

$$\boldsymbol{S_s = 0.51}$$

$$\boldsymbol{S_1 = 0.18}$$

$$\boldsymbol{T_L = 24\ s}$$

Site class	Short-period F_a	Long-period F_v
A	0.80	0.80
B	0.90	0.80
C	1.296	1.50
D	1.392	2.24
E	1.684	See F.7.13.9
F	See F.7.13.9	See F.7.13.9

Table F.14 Short- and long-period site coefficients for building design in Dubai

Table 4.7.2.2 – Short Period and Long Period Site Coefficients

From Table F.14 in Dubai Building Code:

$$F_a = 1.296$$
$$F_v = 1.5$$

$$S_{MS} = 1.296 * 0.51 = 0.661$$

$$\mathrm{S_{ds}} = \frac{2}{3} * 0.661 = 0.44$$

Short period value S_{DS}	Risk level		Long-period value S_{D1}	Risk level	
	I, II & III	IV		I, II & III	IV
$S_{DS} < 0.167$	A	A	$S_{D1} < 0.067$	A	A
$0.167 \leq S_{DS} < 0.33$	B	C	$0.067 \leq S_{D1} < 0.133$	B	C
$0.33 \leq S_{DS} < 0.50$	C	D	$0.133 \leq S_{D1} < 0.20$	C	D
$0.50 \leq S_{DS}$	D	D	$0.20 \leq S_{D1}$	D	D

Table F.15 Seismic design category based on short- and long-period response acceleration parameters

Table 5.7.2.3 – Seismic Design Category Determination by Short Period Parameters

From Table F.15 in Dubai Building Code, the Seismic Design Category is **C.**

Site class	Short-period F_a	Long-period F_v
A	0.80	0.80
B	0.90	0.80
C	1.296	1.50
D	1.392	2.24
E	1.684	See F.7.13.9
F	See F.7.13.9	See F.7.13.9

Table F.14 Short- and long-period site coefficients for building design in Dubai

Table 6.7.2.4 – Short period and Long period Site Coefficients

Table F.14 in Dubai Building Code, $F_v = 1.5$

$$S_{M1} = 1.5 * 0.18 = 0.27$$

$$\mathrm{S}_{\mathrm{d1}} = \frac{2}{3} * 0.27 = 0.18$$

Short period value S_{DS}	Risk level		Long-period value S_{D1}	Risk level	
	I, II & III	IV		I, II & III	IV
$S_{DS} < 0.167$	A	A	$S_{D1} < 0.067$	A	A
$0.167 \le S_{DS} < 0.33$	B	C	$0.067 \le S_{D1} < 0.133$	B	C
$0.33 \le S_{DS} < 0.50$	C	D	$0.133 \le S_{D1} < 0.20$	C	D
$0.50 \le S_{DS}$	D	D	$0.20 \le S_{D1}$	D	D

Table F.15 Seismic design category based on short- and long-period response acceleration parameters

Table 7.7.2.5 – Seismic Design Long Period Parameters

Table F.15 in Dubai Building Code, the Seismic Design Category is C

Use Table 1.5-2 to find the Importance Factor of snow, ice, and earthquake loads:

Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads

Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.15	1.00	1.25
IV	1.20	1.25	1.00	1.50

Table 8.7.2.6 – Load Importance Factors

$I_{snow} = 1.1$

$\mathbf{I_{earthquake} = 1.25}$

$\mathbf{I_{wind} = 1.0}$

From ASCE 7-16 Table 12.2-1, the Response Modification Factor **R**, Strength Modification Factor $\mathbf{\Omega_0}$, and Deflection Amplification Factor $\mathbf{C_d}$**:**

Considering Bearing Wall System, Reinforced Concrete Shear Walls.

Table 12.2-1 Design Coefficients and Factors for Seismic Force-Resisting Systems

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, $\Omega_0{}^b$	Deflection Amplification Factor, $C_d{}^c$
A. BEARING WALL SYSTEMS				
1. Special reinforced concrete shear walls[g,h]	14.2	5	2½	5
2. Ordinary reinforced concrete shear walls[g]	14.2	4	2½	4
3. Detailed plain concrete shear walls[g]	14.2	2	2½	2
4. Ordinary plain concrete shear walls[g]	14.2	1½	2½	1½
5. Intermediate precast shear walls[g]	14.2	4	2½	4
6. Ordinary precast shear walls[g]	14.2	3	2½	3
7. Special reinforced masonry shear walls	14.4	5	2½	3½
8. Intermediate reinforced masonry shear walls	14.4	3½	2½	2¼
9. Ordinary reinforced masonry shear walls	14.4	2	2½	1¾
10. Detailed plain masonry shear walls	14.4	2	2½	1¾
11. Ordinary plain masonry shear walls	14.4	1½	2½	1¼
12. Prestressed masonry shear walls	14.4	1½	2½	1¾
13. Ordinary reinforced AAC masonry shear walls	14.4	2	2½	2
14. Ordinary plain AAC masonry shear walls	14.4	1½	2½	1½
15. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	6½	3	4
16. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	6½	3	4
17. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2	2½	2
18. Light-frame (cold-formed steel) wall systems using flat strap bracing	14.1	4	2	3½
B. BUILDING FRAME SYSTEMS				
1. Steel eccentrically braced frames	14.1	8	2	4
2. Steel special concentrically braced frames	14.1	6	2	5
3. Steel ordinary concentrically braced frames	14.1	3¼	2	3¼
4. Special reinforced concrete shear walls[g,h]	14.2	6	2½	5
5. Ordinary reinforced concrete shear walls[g]	14.2	5	2½	4½
6. Detailed plain concrete shear walls[g]	14.2 and 14.2.2.7	2	2½	2
7. Ordinary plain concrete shear walls[g]	14.2	1½	2½	1½
8. Intermediate precast shear walls[g]	14.2	5	2½	4½
9. Ordinary precast shear walls[g]	14.2	4	2½	4
10. Steel and concrete composite eccentrically braced frames	14.3	8	2½	4
11. Steel and concrete composite special concentrically braced frames	14.3	5	2	4½
12. Steel and concrete composite ordinary braced frames	14.3	3	2	3
13. Steel and concrete composite plate shear walls	14.3	6½	2½	5½
14. Steel and concrete composite special shear walls	14.3	6	2½	5
15. Steel and concrete composite ordinary shear walls	14.3	5	2½	4½
16. Special reinforced masonry shear walls	14.4	5½	2½	4
17. Intermediate reinforced masonry shear walls	14.4	4	2½	4

STANDARD 7-16

Table 9.7.2.7 – Design Coefficients and Factors Based on Seismic Force Resisting System

$R^a = 5.0$

$\Omega_0 = 2.5$

$C^b{}_d = 5.0$

According to the design category C, the structural height is NL (Not Limited).

To determine the Approximate Period Parameters $\boldsymbol{C_t}$, and **X**, it is needed to go to table 12.8-2 in ASCE 7-16 code:

Table 12.8-2 Values of Approximate Period Parameters C_t and x

Structure Type	C_t	x
Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces:		
Steel moment-resisting frames	0.028 (0.0724)[a]	0.8
Concrete moment-resisting frames	0.016 (0.0466)[a]	0.9
Steel eccentrically braced frames in accordance with Table 12.2-1 lines B1 or D1	0.03 (0.0731)[a]	0.75
Steel buckling-restrained braced frames	0.03 (0.0731)[a]	0.75
All other structural systems	0.02 (0.0488)[a]	0.75

[a]Metric equivalents are shown in parentheses.

Table 10.7.2.8 – Approximate Period Parameters

C_t= 0.02 ft (0.0488 meters)

x = 0.75

1.7.3 Wind Load Design

Note: Throughout these calculations, the dome was totally neglected.

Velocity Pressure (q_z)

$$q_z = 0.613 * K_z * K_{zt} * K_e * v^2 \quad (\frac{N}{m^2})$$

1.7.3.1 – Wind Speed

To find the wind speed in a region, refer to the building code of that region, since the princess tower is built in Dubai, refer to the Dubai Building Code.

According to ASCE 7-22 Code, the Mean Recurrence Interval (MRI) is taken to be 1700 years.

and associated reliability. The subcommittee found that the following return periods for each risk category are consistent with the target reliabilities in the first row of Table 1.3-1: Risk Category I, 300 years; Risk Category II, 700 years; Risk Category III, 1,700 years; Risk Category IV, 3,000 years.

Figure 1.7.3.1.1 – Code Recommendation for Wind Mean Recurrence Interval

ASCE/SEI 7-16 MRI (years)	Reference wind speed for 3 s gust at 10 m height on open terrain, $V = V_{ref}$ (m/s)	Application
1	22	Serviceability – occupancy comfort (refer to F.7.12.4.2)
10	30	Serviceability – displacement (refer to F.7.12.4.1)
50	38	Strength in accordance with Clause 5.3.5 of ACI 318 19
300	44	Strength – category I
700	47	Strength – category II
1,700	51	Strength – category III
3,000	53	Strength – category IV

Table F.10 Reference wind speeds per risk category as defined in Clause 1.5 of ASCE/SEI 7-16 and mean recurrence interval (MRI) as defined in the RWDI report **[Ref. F.5]**

Table 1.7.3.1.1 – Wind Speed Based on Mean Recurrence Interval

According to table F.10 in Dubai Building Code ,wind velocity is 51 m/s

1.7.3.2 - K_d $(Wind\ Directionality\ Factor)$

To find K_d, table 26.6-1 is needed.

Table 26.6-1. Wind Directionality Factor, K_d.

Structure Type	Directionality Factor K_d
Buildings	
Main wind force resisting system	0.85
Components and cladding	0.85
Arched roofs	0.85
Circular domes	1.0*
Chimneys, tanks, and similar structures	
Square	0.90
Hexagonal	0.95
Octagonal	1.0*
Round	1.0*
Solid freestanding walls, roof top equipment, and solid freestanding and attached signs	0.85
Open signs and single-plane open frames	0.85
Trussed towers	
Triangular, square, or rectangular	0.85
All other cross sections	0.95

*Directionality factor $K_d = 0.95$ shall be permitted for round or octagonal structures with nonaxisymmetric structural systems.

Table 1.7.3.2.1 – Wind Directionality Factor Determination

According to the above table , $\boldsymbol{K_d = 0.85}$.

1.7.3.3 - Establish the Surface Roughness Category and Exposure Category

Urban area, Surface Roughness Category (B), Exposure Category (B).

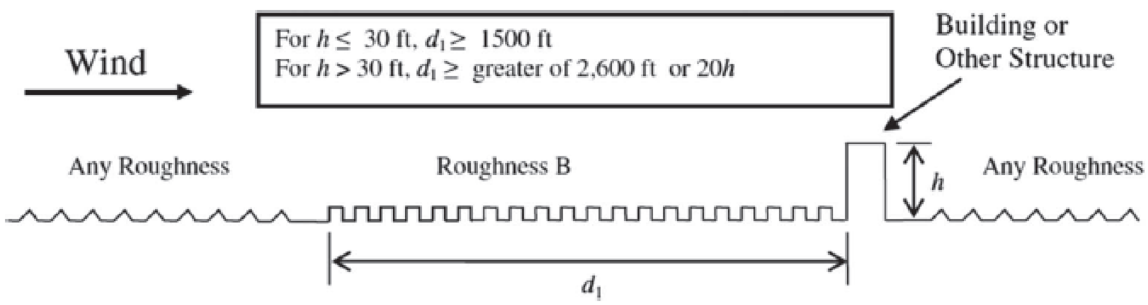


Figure C26.7-1. Upwind surface roughness conditions required for Exposure B.

Figure 1.7.3.3.1 – Surface Roughness Category B Conditions

1.7.3.4 - K_{zt} $(Topographic\ Factor)$

Because there are no significant topographic changes, K_{zt} = 1.0

1.7.3.5 - $K_e(Ground\ Elevation\ Factor)$

Table 26.9-1 Ground Elevation Factor, K_e

Ground Elevation above Sea Level		Ground Elevation Factor
ft	m	K_e
<0	<0	See note 2
0	0	1.00
1,000	305	0.96
2,000	610	0.93
3,000	914	0.90
4,000	1,219	0.86
5,000	1,524	0.83
6,000	1,829	0.80
>6,000	>1,829	See note 2

Notes

1. The conservative approximation $K_e = 1.00$ is permitted in all cases.
2. The factor K_e shall be determined from the above table using interpolation or from the following formula for all elevations:
   $K_e = e^{-0.0000362 z_g}$ (z_g = ground elevation above sea level in ft).
   $K_e = e^{-0.000119 z_g}$ (z_g = ground elevation above sea level in m).
3. K_e is permitted to be take as 1.00 in all cases.

Table 11.7.3.5.1 – Ground Elevation Factor Determination

Take $K_e = 1$

1.7.3.6 - K_z (*Velocity pressure Exposure Coefficient*)

Table 26.10-1. Velocity Pressure Exposure Coefficients, K_h and K_z.

Height above Ground Level, z or h		Exposure		
ft	m	B	C	D
0–15	0–4.6	0.57 (0.70)*	0.85	1.03
20	6.1	0.62 (0.70)*	0.90	1.08
25	7.6	0.66 (0.70)*	0.94	1.12
30	9.1	0.70	0.98	1.16
40	12.2	0.74	1.04	1.22
50	15.2	0.79	1.09	1.27
60	18.3	0.83	1.13	1.31
70	21.3	0.86	1.17	1.34
80	24.4	0.90	1.21	1.38
90	27.4	0.92	1.24	1.40
100	30.5	0.95	1.26	1.43
120	36.6	1.00	1.31	1.48
140	42.7	1.04	1.34	1.52
160	48.8	1.08	1.39	1.55
180	54.9	1.11	1.41	1.58
200	61.0	1.14	1.44	1.61
250	76.2	1.21	1.51	1.68
300	91.4	1.27	1.57	1.73
350	106.7	1.33	1.62	1.78
400	121.9	1.38	1.66	1.82
450	137.2	1.42	1.70	1.86
500	152.4	1.46	1.74	1.89

Table 1.7.3.6.1 –Velocity Pressure Exposure Coefficient

1. Velocity pressure exposure coefficient K_z may be determined from the following formula:

For z < 15 ft	$K_z = 2.41\,(15/z_g)^{2/\alpha}$
For z < 4.6 m	$K_z = 2.41\,(4.6/z_g)^{2/\alpha}$
For 15 ft (4.6 m) $\leq z \leq z_g$	$K_z = 2.41\,(z/z_g)^{2/\alpha}$
For $z_g < z \leq$ 3,280 ft (1,000 m)	$K_z = 2.41$

2. α and z_g are tabulated in Table 26.11-1.
3. Linear interpolation for intermediate values of height z is acceptable.
4. Exposure categories are defined in Section 26.7.

Table 12.7.3.6.2 –Velocity Pressure Exposure Formula

$$\alpha = 7.5$$

$$z_g = 1000\ m$$

Table 26.11-1. Terrain Exposure Constants.

Customary Units										
Exposure	α	z_g (ft)	$\hat{\alpha}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	l (ft)	$\bar{\epsilon}$	z_{min} (ft)*
B	7.5	3,280	1/7.5	0.84	1/4.5	0.47	0.30	320	1/3.0	30
C	9.8	2,460	1/9.8	1.00	1/6.4	0.66	0.20	500	1/5.0	15
D	11.5	1,935	1/11.5	1.09	1/8.0	0.78	0.15	650	1/8.0	7
SI Units										
Exposure	α	z_g (m)	$\hat{\alpha}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	l (m)	$\bar{\epsilon}$	z_{min} (m)*
B	7.5	1,000	1/7.5	0.84	1/4.5	0.47	0.30	97.54	1/3.0	9.14
C	9.8	750	1/9.8	1.00	1/6.4	0.66	0.20	152.40	1/5.0	4.57
D	11.5	590	1/11.5	1.09	1/8.0	0.78	0.15	198.12	1/8.0	2.13

* z_{min} = Minimum height used to ensure that the equivalent height $\bar{z}$ is the greater of $0.6h$ or z_{min}. For buildings or other structures with $h \le z_{min}$, $\bar{z}$ shall be taken as z_{min}.

Table 13.7.3.6.3 –Terrain Exposure Constants

Floor No.	**X Dimension**	**Y Dimension**
1st Parking Floor to 5th Parking Floor	56.41	62.5
1st Typical Floor (6th Floor to 80th Floor)	41.35	37.85
2nd Typical Floor (81st Floor to 96th Floor)	40.35	36.85
97th and 98th Floors	37	37
99th and 100th Floors	34.8	34.8

Table 14.7.3.6.4 –Floor Horizontal Dimensions

For 1st Parking Floor to 5th Parking Floor, z is 15.78, $\boldsymbol{K_z = 0.797.}$

$q_z = 0.613 * 0.797 * 1 * 1 * 51^2 = 1270.7\ N/m^2$

$\boldsymbol{q_z\ for\ 1st\ Parking\ Floor\ to\ 5th\ Parking\ Floor = 1.27\ kN/m^2}$

1st Parking Floor to 5th Parking Floor have:

$B_x = 62.5\ m$ and $L_x = 56.41\ m$

$B_y = 56.41\ m$ and $L_y = 62.5\ m$

- B_x is the dimension of the floor's plane that is normal to wind direction (while wind is in x direction)
- L_x is the dimension of the floor's plane that is parallel to wind direction (while wind is in x direction)
- B_y and L_y are the same as before but when the wind is in y direction.

6th Floor to 22nd Floor, z is 80.85 m, $\boldsymbol{K_z = 1.2288.}$

$q_z = 0.613 * 1.2288 * 1 * 1 * 51^2 = 1959.2\frac{N}{m^2}$

$\boldsymbol{q_z\ for\ 6th\ Floor\ to\ 22nd\ Floor = 1.96\frac{kN}{m^2}.}$

6th Floor to 22nd Floor have:

$B_x = 37.85\ m$ and $L_x = 41.35\ m$

$B_y = 41.35\ m$ and $L_y = 37.85\ m$

23rd Floor to 36th Floor**, z is 131.95 m,** $\boldsymbol{K_z = 1.406}$

$q_z = 0.613 * 1.406 * 1 * 1 * 51^2 = 2241.74\frac{N}{m^2}$

$\boldsymbol{q_z\ for\ 23rd\ Floor\ to\ 36th\ Floor = 2.24\ kN/m^2}$

23rd Floor to 36th Floor have:

$B_x = 37.85\ m$ and $L_x = 41.35\ m$

$B_y = 41.35\ m$ and $L_y = 37.85\ m$

37th Floor to 50th Floor, z is 183.05 m, $K_z = 2.41 * \left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}} = 2.41\left(\frac{183.05}{1000}\right)^{\frac{2}{7.5}} = \mathbf{1.532}$

$q_z = 0.613 * 1.532 * 1 * 1 * 51^2 = 2442.6 \frac{N}{m^2}$

$q_z\ for\ 37th\ Floor\ to\ 50th\ Floor = 2.4426\ kN/m^2$

37th Floor to 50th Floor have:

$B_x = 37.85\ m$ and $L_x = 41.35\ m$

$B_y = 41.35\ m$ and $L_y = 37.85\ m$

51st Floor to 65th Floor, z is 237.8 m, $K_z = 2.41 * \left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}} = 2.41\left(\frac{237.8}{1000}\right)^{\frac{2}{7.5}} = 1.643$

$q_z = 0.613 * 1.643 * 1 * 1 * 51^2 = 2619.6 \frac{N}{m^2}$

$\boldsymbol{q_z\ for\ 51st\ Floor\ to\ 65th\ Floor = 2.6196\ kN/m^2}$

51st Floor to 65th Floor have:

$B_x = 37.85\ m$ and $L_x = 41.35\ m$

$B_y = 41.35\ m$ and $L_y = 37.85\ m$

66th Floor to 80th Floor, z is 294.1 m, $K_z = 2.41 * \left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}} = 2.41\left(\frac{294.1}{1000}\right)^{\frac{2}{7.5}} = 1.74$

$q_z = 0.613 * 1.74 * 1 * 1 * 51^2 = 2774.28\frac{N}{m^2}$

$\boldsymbol{q_z\ for\ 66th\ Floor\ to\ 80th\ Floor = 2.77\ kN/m^2}$

66th Floor to 80th Floor have:

$B_x = 37.85\ m$ and $L_x = 41.35\ m$

$B_y = 41.35\ m$ and $L_y = 37.85\ m$

81st Floor to 96th Floor, z is 352.2 m, $K_z = 2.41 * \left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}} = 2.41\left(\frac{352.2}{1000}\right)^{\frac{2}{7.5}} = 1.824$

$q_z = 0.613 * 1.824 * 1 * 1 * 51^2 = 2908.21\frac{N}{m^2}$

$\boldsymbol{q_z\ for\ 81st\ Floor\ to\ 96th\ Floor = 2.908\ kN/m^2}$

81st Floor to 96th Floor have:

$B_x = 36.85\ m$ and $L_x = 40.35\ m$

$B_y = 40.35\ m$ and $L_y = 36.85\ m$

97th Floor and 98th, z is 364.2 m, $K_z = 2.41 * \left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}} = 2.41\left(\frac{364.2}{1000}\right)^{\frac{2}{7.5}} = 1.841$

$q_z = 0.613 * 1.841 * 1 * 1 * 51^2 = 2935.3\frac{N}{m^2}$

$q_z\ for\ 97th\ Floor\ and\ 98th\ Floor = 2.935\ kN/m^2$

97th Floor and 98th have:

$B_x = 37\ m$ and $L_x = 37\ m$

$B_y = 37\ m$ and $L_y = 37\ m$

99^{th} Floor and 100^{th} Floor, z is 376.8 m, $K_z = 2.41 * \left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}} = 2.41 \left(\frac{376.8}{1000}\right)^{\frac{2}{7.5}} = 1.8577$

$q_z = 0.613 * 1.8577 * 1 * 1 * 51^2 = 2962 \frac{N}{m^2}$

$q_z\ for\ 99th\ Floor\ and\ 100th\ Floor = 2.962\ kN/m^2$

99^{th} Floor and 100^{th} Floor have:

$B_x = 34.8\ m$ and $L_x = 34.8\ m$

$B_y = 34.8\ m$ and $L_y = 34.8\ m$

1.7.3.7 - Design Wind Pressure

There are several procedures to determine the design wind pressure for a building, as for our building, it is a high rise building and it does not have a complex shape, so the procedure used is Directional Procedure.

Design Wind Pressure (p):

$$p = qK_dGC_p - q_iK_d(GC_{pi}) \quad \left(\frac{N}{m^2}\right)$$

q = q_z For windward walls evaluated at height Z above the ground.

q = q_h For leeward walls, sidewalls, and roofs evaluated at height h

$$q = q_z$$

q_i = q_h For windward walls, sidewalls, leeward walls, and roofs of enclosed buildings, partially open buildings, and for negative internal pressure evaluation in partially enclosed buildings.

q_i = q_z For positive internal pressure evaluation in partially enclosed buildings, where height z is defined as the level of the highest opening in the building that could affect the positive internal pressure.

$$q_i = q_h$$

$q_h\ is\ the\ velocity\ pressure\ at\ height\ h$

h is the mean roof height of the building, which is the height of the centroid of the dome located at the roof.

The height to the roof is 376.8 m

Mean Roof Height = 376.8 m

$K_z = 1.8577$

$$q_h = 0.613 * 1.8577 * 1 * 1 * 51^2 = 2962 \frac{N}{m^2}$$

$$\boldsymbol{q_h = 2.962\ kN/m^2}$$

To find the external wind pressure for walls, use the Table in Figure 27.3-1 in ASCE 7-22

Wall Pressure Coefficients, C_p

Surface	L/B	C_p	Use with
Windward wall	All values	0.8	q_z
Leeward wall	0–1	–0.5	q_h
	2	–0.3	q_h
	≥ 4	–0.2	q_h
Sidewall	All values	–0.7	q_h
Parapet	All values	See Section 27.3.4 for GC_{pn}	q_p

Table 15.7.3.7.1 –Wall External Pressure Coefficients

Floors	Windward Cp_x	Windward Cp_y	Leeward Cp_x	Leeward Cp_y
1st Parking Floor to 5th Parking Floor	0.8	0.8	-0.5	-0.478408
1st Typical Floor (6th Floor to 80th Floor)	0.8	0.8	-0.481506	-0.5
2nd Typical Floor (81st Floor to 96th Floor)	0.8	0.8	-0.481004	-0.5
97th and 98th Floors	0.8	0.8	-0.5	-0.5
99th and 100th Floors	0.8	0.8	-0.5	-0.5

Table 16.7.3.7.2 – Floors Wall External Pressure

To find Internal Pressure Coefficient (GC_{pi}), Table 26.13-1 in ASCE 7-22.

Table 26.13-1. Main Wind Force Resisting System and Components and Cladding (All Heights): Internal Pressure Coefficient, (GC_{pi}), for Enclosed, Partially Enclosed, Partially Open, and Open Buildings (Walls and Roof).

Enclosure Classification	Criteria for Enclosure Classification	Internal Pressure	Internal Pressure Coefficient (GC_{pi})
Enclosed buildings	A_o is less than the smaller of $0.01A_g$ or 4 ft² (0.37 m²), and $A_{oi}/A_{gi} \leq 0.2$	Moderate	+0.18 −0.18
Partially enclosed buildings	$A_o > 1.1A_{oi}$, and $A_o >$ the lesser of $0.01A_g$ or 4 ft² (0.37 m²), and $A_{oi}/A_{gi} \leq 0.2$	High	+0.55 −0.55
Partially open buildings	A building that does not comply with Enclosed, Partially Enclosed, or Open classifications	Moderate	+0.18 −0.18
Open buildings	Each wall is at least 80% open	Negligible	0.00

Notes:

1. Plus and minus signs signify pressures acting toward and away from the internal surfaces, respectively.
2. Values of (GC_{pi}) shall be used with q_z or q_h as specified.
3. Two cases shall be considered to determine the critical load requirements for the appropriate condition:
   (a) A positive value of (GC_{pi}) applied to all internal surfaces, or
   (b) A negative value of (GC_{pi}) applied to all internal surfaces.

Table 1.7.3.7.3 –Internal Pressure Coefficient

This indicates that for suction (negative) internal pressure the value (-0.18) shall be taken and for push (positive) internal pressure the value (+0.18) shall be taken.

To take the worst case, take suction internal pressure for windward pressure and push internal pressure for leeward pressure.

For Windward: GC_{pi} = -0.18

For Leeward: GC_{pi} = +0.18

To Find the Gust effect factor (G or G_f):

$$G_f = 0.925\left(\frac{1 + 1.7I_{Z'}\sqrt{g_Q{}^2Q^2 + g_R{}^2R^2}}{1 + 1.7g_vI_{Z'}}\right)$$

The background response factor Q is given by:

$$Q = \sqrt{\frac{1}{1 + 0.63 * \left(\frac{B + h}{L_{Z'}}\right)^{0.63}}}$$

$$L_{Z'} = l * \left(\frac{Z'}{10}\right)^{\varepsilon'}$$

z' is the effective height of the building, can be taken as 0.6h,$Z' = 0.6 * 376.8 = \mathbf{226.08\ m}$.

To find ε' and l, refer to table 1.7.3.6.3 above.

$l = 97.54$

$$\varepsilon' = \frac{1}{3}$$

$$L_{Z'} = 97.54 * \left(\frac{226.08}{10}\right)^{\frac{1}{3}} = 275.8\ m.$$

For 1st Parking Floor to 5th Parking Floor

$$Q_x = \sqrt{\frac{1}{1 + 0.63\left(\frac{62.5 + 376.8}{275.8}\right)^{0.63}}} = 0.73627$$

$$Q_y = \sqrt{\frac{1}{1 + 0.63\left(\frac{56.41 + 376.8}{275.8}\right)^{0.63}}} = 0.73775$$

For 6th Floor to 80th Floor

$$Q_x = \sqrt{\frac{1}{1 + 0.63\left(\frac{37.85 + 376.8}{275.8}\right)^{0.63}}} = 0.74237$$

$$Q_y = \sqrt{\frac{1}{1 + 0.63\left(\frac{41.35 + 376.8}{275.8}\right)^{0.63}}} = 0.74148$$

For 81st Floor to 96th Floor

$$Q_x = \sqrt{\frac{1}{1 + 0.63\left(\frac{36.85 + 376.8}{275.8}\right)^{0.63}}} = 0.74262$$

$$Q_y = \sqrt{\frac{1}{1 + 0.63\left(\frac{40.35 + 376.8}{275.8}\right)^{0.63}}} = 0.74173$$

For 97th Floor and 98th Floor

$$Q_x = \sqrt{\frac{1}{1 + 0.63\left(\frac{37 + 376.8}{275.8}\right)^{0.63}}} = 0.74258$$

$$Q_y = \sqrt{\frac{1}{1 + 0.63\left(\frac{37 + 376.8}{275.8}\right)^{0.63}}} = 0.74258$$

For 99th Floor and 100th Floor

$$Q_x = \sqrt{\frac{1}{1 + 0.63\left(\frac{34.8 + 376.8}{275.8}\right)^{0.63}}} = 0.74314$$

$$Q_y = \sqrt{\frac{1}{1 + 0.63\left(\frac{34.8 + 376.8}{275.8}\right)^{0.63}}} = 0.74314$$

The intensity of turbulence at height Z,' $I_{z'}$ is found by the following equation:

$I_{Z'} = c * \left(\frac{10}{Z'}\right)^{\frac{1}{6}}$, c is found in table 1.7.3.6.3 above: C = 0.3

$$I_{Z'} = 0.3 * \left(\frac{10}{226.08}\right)^{\frac{1}{6}} = 0.1784$$

g_Q *and* g_v *shall be taken as* 3.4

g_R *as follows*:

$$g_R = \sqrt{2\ln(3600 * n_1)} + \frac{0.577}{\sqrt{2\ln(3600 * n_1)}}$$

n_1 is the fundamental natural frequency

To find n_1, the period in each direction is needed.

$T_x = 9.35$ Seconds.

$T_y = 10.1$ Seconds.

$$n_{1x} = \frac{1}{9.35} = 0.10695\ Hz$$

$$n_{1y} = \frac{1}{10.1} = 0.09901\ Hz$$

$$g_{Rx} = \sqrt{2\ln(3600 * 0.10695)} + \frac{0.577}{\sqrt{2\ln(3600 * 0.10695)}} = 3.61782$$

$$g_{Ry} = \sqrt{2\ln(3600 * 0.09901\ )} + \frac{0.577}{\sqrt{2\ln(3600 * 0.09901\ )}} = 3.59648$$

The Resonant Response Factor R is given by:

$R = \sqrt{\left(\frac{1}{\beta}\right) R_n R_h R_B (0.53 + 0.47R_L)}$, β is the damping ratio, taken as 0.05

- R_h *is the size factor related to height.*
- R_B *is the size factor related to breadth (width).*
- R_L *is the size factor related to Length.*

$$R_h = \left(\frac{1}{n_h}\right) - \left(\frac{1}{2 * {n_h}^2}\right)(1 - e^{-2*n_h})$$

$$R_B = \left(\frac{1}{n_B}\right) - \left(\frac{1}{2 * {n_B}^2}\right)(1 - e^{-2*n_B})$$

$$R_L = \left(\frac{1}{n_L}\right) - \left(\frac{1}{2 * {n_L}^2}\right)(1 - e^{-2*n_L})$$

- n_h *is the turbulent coherence (correlation) factor in the Height direction.*
- n_B *is the turbulent coherence (correlation) factor in the Breadth direction.*
- n_L *is the turbulent coherence (correlation) factor in the Length direction*

$$n_h = \frac{4.6 n_1 h}{V'_Z}$$

$$n_B = \frac{4.6 n_1 B}{V'_Z}$$

$$n_L = \frac{4.6 n_1 L}{V'_Z}$$

$V'_{Z'}$ is the mean hourly speed at the at the equivalent structure height Z,' and it is given by:

$$V'_{Z'} = b' * \left(\frac{Z'}{10}\right)^{\alpha'} * V$$

b' and α' are taken from Table 26.11-1

b' = 0.47

$$\alpha' = \frac{1}{4.5}$$

$$V'_{Z'} = 0.47 * \left(\frac{226.08}{10}\right)^{\frac{1}{4.5}} * 51 = 47.93\ m/s$$

$$n_{hx} = \frac{4.6 * 0.10695 \ * 376.8}{47.93} = 3.75$$

$$n_{hy} = \frac{4.6 * 0.09901 * 376.8}{47.93} = 3.473$$

$$R_{hx} = \left(\frac{1}{3.473}\right) - \left(\frac{1}{2 * (3.473)^2}\right) * (1 - e^{-2*3.473}) = 0.23105$$

$$R_{hy} = \left(\frac{1}{3.473}\right) - \left(\frac{1}{2 * (3.473)^2}\right)(1 - e^{-2*3.473}) = 0.24652$$

For 1st Parking Floor to 5th Parking Floor

$$n_{Bx} = \frac{4.6 * 0.10695 \ * 62.5}{47.93} = 0.61313$$

$$n_{Lx} = \frac{15.4 * 0.10695 \ * 56.41}{47.93} = 1.85265$$

$$n_{By} = \frac{4.6 * 0.09901 * 56.41}{47.93} = 0.5123$$

$$n_{Ly} = \frac{15.4 * 0.09901 * 62.5}{47.93} = 1.90024$$

$$R_{Bx} = \left(\frac{1}{0.61313}\right) - \left(\frac{1}{2 * (0.61313)^2}\right) * (1 - e^{-2*0.61313}) = 0.69115$$

$$R_{Lx} = \left(\frac{1}{1.85265}\right) - \left(\frac{1}{2*(1.85265)^2}\right)*(1 - e^{-2*1.85265}) = 0.39767$$

$$R_{By} = \left(\frac{1}{0.5123}\right) - \left(\frac{1}{2*(0.5123)^2}\right)*(1 - e^{-2*0.5123}) = 0.73069$$

$$R_{Ly} = \left(\frac{1}{1.90024}\right) - \left(\frac{1}{2*(1.90024)^2}\right)*(1 - e^{-2*1.90024}) = 0.39088$$

For 6th Floor to 80th Floor

$$n_{Bx} = \frac{4.6*0.10695*37.85}{47.93} = 0.37131$$

$$n_{Lx} = \frac{15.4*0.10695*41.35}{47.93} = 1.35804$$

$$n_{By} = \frac{4.6*0.09901*41.35}{47.93} = 0.37553$$

$$n_{Ly} = \frac{15.4*0.09901*37.85}{47.93} = 1.15079$$

$$R_{Bx} = \left(\frac{1}{0.37131}\right) - \left(\frac{1}{2*(0.37131)^2}\right)*(1 - e^{-2*0.37131}) = 0.79235$$

$$R_{Lx} = \left(\frac{1}{1.35804}\right) - \left(\frac{1}{2*(1.35804)^2}\right)*(1 - e^{-2*1.35804}) = 0.48317$$

$$R_{By} = \left(\frac{1}{0.37553}\right) - \left(\frac{1}{2*(0.37553)^2}\right)*(1 - e^{-2*0.37553}) = 0.79039$$

$$R_{Ly} = \left(\frac{1}{1.15079}\right) - \left(\frac{1}{2*(1.15079)^2}\right)*(1 - e^{-2*1.15079}) = 0.52921$$

For 81st to 96th Floor

$$n_{Bx} = \frac{4.6*0.10695*36.85}{47.93} = 0.3615$$

$$n_{Lx} = \frac{15.4*0.10695*40.35}{47.93} = 1.3252$$

$$n_{By} = \frac{4.6*0.09901*40.35}{47.93} = 0.36645$$

$$n_{Ly} = \frac{15.4 * 0.09901 * 36.85}{47.93} = 1.12038$$

$$R_{Bx} = \left(\frac{1}{0.3615}\right) - \left(\frac{1}{2 * (0.3615)^2}\right) * (1 - e^{-2*0.3615}) = 0.79695$$

$$R_{Lx} = \left(\frac{1}{1.3252}\right) - \left(\frac{1}{2 * (1.3252)^2}\right) * (1 - e^{-2*1.3252}) = 0.49$$

$$R_{By} = \left(\frac{1}{0.36645}\right) - \left(\frac{1}{2 * (0.36645)^2}\right) * (1 - e^{-2*0.36645}) = 0.79463$$

$$R_{Ly} = \left(\frac{1}{1.12038}\right) - \left(\frac{1}{2 * (1.12038)^2}\right) * (1 - e^{-2*1.12038}) = 0.5366$$

For 97th Floor and 98th Floor

$$n_{Bx} = \frac{4.6 * 0.10695 * 37}{47.93} = 0.36298$$

$$n_{Lx} = \frac{15.4 * 0.10695 * 37}{47.93} = 1.21518$$

$$n_{By} = \frac{4.6 * 0.09901 * 37}{47.93} = 0.33602$$

$$n_{Ly} = \frac{15.4 * 0.09901 * 37}{47.93} = 1.1294$$

$$R_{Bx} = \left(\frac{1}{0.36298}\right) - \left(\frac{1}{2 * (0.36298)^2}\right) * (1 - e^{-2*0.36298}) = 0.79626$$

$$R_{Lx} = \left(\frac{1}{1.21518}\right) - \left(\frac{1}{2 * (1.21518)^2}\right) * (1 - e^{-2*1.21518}) = 0.51412$$

$$R_{By} = \left(\frac{1}{0.33602}\right) - \left(\frac{1}{2 * (0.33602)^2}\right) * (1 - e^{-2*0.33602}) = 0.80908$$

$$R_{Ly} = \left(\frac{1}{1.1294}\right) - \left(\frac{1}{2 * (1.1294)^2}\right) * (1 - e^{-2*1.1294}) = 0.53548$$

For 99th Floor and 100th Floor

$$n_{Bx} = \frac{4.6 * 0.10695 * 34.8}{47.93} = 0.34139$$

$$n_{Lx} = \frac{15.4 * 0.10695 * 34.8}{47.93} = 1.14292$$

$$n_{By} = \frac{4.6 * 0.09901 * 34.8}{47.93} = 0.31604$$

$$n_{Ly} = \frac{15.4 * 0.09901 * 34.8}{47.93} = 1.05805$$

$$R_{Bx} = \left(\frac{1}{0.34139}\right) - \left(\frac{1}{2 * (0.34139)^2}\right) * (1 - e^{-2*0.34139}) = 0.8065$$

$$R_{Lx} = \left(\frac{1}{1.14292}\right) - \left(\frac{1}{2 * (1.14292)^2}\right) * (1 - e^{-2*1.14292}) = 0.5311$$

$$R_{By} = \left(\frac{1}{0.31604}\right) - \left(\frac{1}{2 * (0.31604)^2}\right) * (1 - e^{-2*0.31604}) = 0.8188$$

$$R_{Ly} = \left(\frac{1}{1.05805}\right) - \left(\frac{1}{2 * (1.05805)^2}\right) * (1 - e^{-2*1.05805}) = 0.55231$$

As for R_n, it is the power spectral density of turbulence at the equivalent structure height Z,' evaluated at the structure's natural reduced frequency N_1, and it is given by:

$$R_n = \frac{7.47 * N_1}{(1 + 10.3 * N_1)^{\frac{5}{3}}}$$

$$N_1 = \frac{n_1 * L_{Z'}}{V'_{Z'}}$$

$$N_{1x} = \frac{0.10695 * 275.8}{47.93} = 0.5911$$

$$N_{1y} = \frac{0.09901 * 275.8}{47.93} = 0.54721$$

$$R_{nx} = \frac{7.47 * 0.56}{(1 + 10.3 * 0.56)^{\frac{5}{3}}} = 0.16881$$

$$R_{ny} = \frac{7.47 * 0.517}{(1 + 10.3 * 0.517)^{\frac{5}{3}}} = 0.17442$$

For 1st Parking Floor to 5th Parking Floor

$$R_x = \sqrt{\frac{1}{0.05} * 0.16881 * 0.23105 * 0.69115(0.53 + 0.47 * 0.39767)}$$

$R_x = 0.6217$

$$R_y = \sqrt{\frac{1}{0.05} * 0.17442 * 0.24652 * 0.73069(0.53 + 0.47 * 0.39088)}$$

$R_y = 0.6696$

$$G_{fx} = 0.925 * \left(\frac{1 + \left(1.7 * 0.1724\sqrt{(3.4^2) * (0.735^2) + (3.599^2) * (0.538^2)}\right)}{1 + (1.7 * 3.4 * 0.1724)}\right)$$

$G_{fx} = 0.92$

$$G_{fy} = 0.925 * \left(\frac{1 + \left(1.7 * 0.1724\sqrt{((3.4)^2) * ((0.737)^2) + ((3.56)^2) * ((0.655)^2)}\right)}{1 + (1.7 * 3.4 * 0.1724)}\right)$$

$G_{fy} = 0.9355$

for 6th Floor to 80th Floor

$$R_x = \sqrt{\frac{1}{0.05} * 0.16881 * 0.23105 * 0.79235(0.53 + 0.47 * 0.48317)}$$

$R_x = 0.684$

$$R_y = \sqrt{\frac{1}{0.05} * 0.17442 * 0.24652 * 0.79(0.53 + 0.47 * 0.52921)}$$

$R_y = 0.7275$

$$G_{fx} = 0.925 * \left(\frac{1 + \left(1.7 * 0.1724\sqrt{((3.4)^2) * ((0.742)^2) + ((3.599)^2) * ((0.7385)^2)}\right)}{1 + (1.7 * 3.4 * 0.1724)}\right)$$

$G_{fx} = 0.94$

$$G_{fy} = 0.925 * (\frac{1 + \left(1.7 * 0.1724\sqrt{((3.4)^2) * ((0.741)^2) + ((3.56)^2) * ((0.7356)^2)}\right)}{1 + (1.7 * 3.4 * 0.1724)})$$

$$G_{fy} = 0.956$$

For 81st Floor to 96th Floor

$$R_x = \sqrt{\frac{1}{0.05} * 0.16881 * 0.23105 * 0.797(0.53 + 0.47 * 0.49)}$$

$$R_x = 0.687$$

$$R_y = \sqrt{\frac{1}{0.05} * 0.17442 * 0.24652 * 0.794(0.53 + 0.47 * 0.5366)}$$

$$R_y = 0.7311$$

$$G_{fx} = 0.925 * (\frac{1 + \left(1.7 * 0.1724\sqrt{((3.4)^2) * ((0.7478)^2) + ((3.599)^2) * ((0.765)^2)}\right)}{1 + (1.7 * 3.4 * 0.1724)})$$

$$G_{fx} = 0.944$$

$$G_{fy} = 0.925 * \left(\frac{1 + \left(1.7 * 0.1724\sqrt{((3.4)^2) * ((0.746681)^2) + ((3.56)^2) * ((0.715)^2)}\right)}{1 + (1.7 * 3.4 * 0.1724)}\right)$$

$$G_{fy} = 0.958$$

For 97th Floor and 98th Floor

$$R_x = \sqrt{\frac{1}{0.05} * 0.16881 * 0.23105 * 0.796(0.53 + 0.47 * 0.514)}$$

$$R_x = 0.692$$

$$R_y = \sqrt{\frac{1}{0.05} * 0.17442 * 0.24652 * 0.809(0.53 + 0.47 * 0.535)}$$

$$R_y = 0.737$$

$$G_{fx} = 0.925 * (\frac{1 + \left(1.7 * 0.1724\sqrt{((3.4)^2) * ((0.74783)^2) + ((3.599)^2) * ((0.771)^2)}\right)}{1 + (1.7 * 3.4 * 0.1724)})$$

$G_{fx} = 0.946$

$$G_{fy} = 0.925 * (\frac{1 + \left(1.7 * 0.1724\sqrt{((3.4)^2) * ((0.7479)^2) + ((3.56)^2) * ((0.722)^2)}\right)}{1 + (1.7 * 3.4 * 0.1724)})$$

$G_{fy} = 0.96$

For 99th Floor and 100th Floor

$$R_x = \sqrt{\frac{1}{0.05} * 0.16881 * 0.23105 * 0.8065(0.53 + 0.47 * 0.5311)}$$

$R_x = 0.7$

$$R_y = \sqrt{\frac{1}{0.05} * 0.17442 * 0.24652 * 0.8188(0.53 + 0.47 * 0.552)}$$

$R_y = 0.745$

$$G_{fx} = 0.925 * (\frac{1 + \left(1.7 * 0.1724\sqrt{((3.4)^2) * ((0.74783)^2) + ((3.599)^2) * ((0.778)^2)}\right)}{1 + (1.7 * 3.4 * 0.1724)})$$

$G_{fx} = 0.949$

$$G_{fy} = 0.925 * (\frac{1 + \left(1.7 * 0.1724\sqrt{((3.4)^2) * ((0.7479)^2) + ((3.56)^2) * ((0.73)^2)}\right)}{1 + (1.7 * 3.4 * 0.1724)})$$

$$G_{fy} = 0.96$$

To determine the design wind pressure (Windward) for each floor interval

$$p = qK_dGC_p - q_iK_d(GC_{pi}) \quad \left(\frac{N}{m^2}\right)$$

1st Parking Floor to 5th Parking Floor

$p_x = q_z K_d G_x C_{px} - q_h K_d (GC_{pi}) \quad \left(\frac{N}{m^2}\right)$

$p_x = (1.27 * 0.85 * 0.92 * (0.8)) - (2.962 * 0.85 * -0.18)$, $\boldsymbol{p_x = 1.247\ kN/m^2}$

$p_y = q_z K_d G_y C_{py} - q_h K_d (GC_{pi}) \quad \left(\frac{N}{m^2}\right)$

$p_y = (1.27 * 0.85 * 0.9355 * (0.8)) - (2.962 * 0.85 * -0.18)$

$\boldsymbol{p_y = 1.26\ kN/m^2}$

6th Floor to 22nd Floor

$p_x = (1.96 * 0.85 * 0.943 * (0.8)) - (2.962 * 0.85 * -0.18)$

$\boldsymbol{p_x = 1.71\ kN/m^2}$

$p_y = (1.96 * 0.85 * 0.956 * (0.8)) - (2.962 * 0.85 * -0.18)$

$\boldsymbol{p_y = 1.728\ kN/m^2}$

23rd Floor to 36th Floor

$p_x = (2.24 * 0.85 * 0.943 * (0.8)) - (2.962 * 0.85 * -0.18)$

$\boldsymbol{p_x = 1.89\ kN/m^2}$

$p_y = (2.24 * 0.85 * 0.956 * (0.8)) - (2.962 * 0.85 * -0.18)$

$\boldsymbol{p_y = 1.91\ kN/m^2}$

37th Floor to 50th Floor

$p_x = (2.442 * 0.85 * 0.943 * (0.8)) - (2.962 * 0.85 * -0.18)$

$\boldsymbol{p_x = 2.02\ kN/m^2}$

$p_y = (2.442 * 0.85 * 0.956 * (0.8)) - (2.962 * 0.85 * -0.18)$

$\boldsymbol{p_y = 2.04\ kN/m^2}$

51st Floor to 65th Floor

$p_x = (2.62 \ * 0.85 * 0.943 * (0.8)) - (2.962 \ * 0.85 * -0.18)$

$\boldsymbol{p_x = 2.133 kN/m^2}$

$p_y = (2.62 \ * 0.85 * 0.956 * (0.8)) - (2.962 \ * 0.85 * -0.18)$

$\boldsymbol{p_y = 2.157\ kN/m^2}$

66th Floor to 80th Floor

$p_x = (2.77 * 0.85 * 0.943 * (0.8)) - (2.962 \ * 0.85 * -0.18)$

$\boldsymbol{p_x = 2.23\ kN/m^2}$

$p_y = (2.77 * 0.85 * 0.956 * (0.8)) - (2.962 \ * 0.85 * -0.18)$

$\boldsymbol{p_y = 2.25 kN/m^2}$

81st Floor to 96th Floor

$p_x = (2.908 * 0.85 * 0.944 * (0.8)) - (2.962 \ * 0.85 * -0.18)$

$\boldsymbol{p_x = 2.32\ kN/m^2}$

$p_y = (2.908 * 0.85 * 0.958 * (0.8)) - (2.962 \ * 0.85 * -0.18)$

$\boldsymbol{p_y = 2.347\ kN/m^2}$

97th Floor and 98th Floor

$p_x = (2.935 * 0.85 * 0.946 * (0.8)) - (2.962 \ * 0.85 * -0.18)$

$\boldsymbol{p_x = 2.34\ kN/m^2}$

$p_y = (2.935 * 0.85 * 0.96 * (0.8)) - (2.962 \ * 0.85 * -0.18)$

$\boldsymbol{p_y = 2.37\ kN/m^2}$

99th Floor to 100th Floor

$p_x = (2.962 \ * 0.85 * 0.95 * (0.8)) - (2.973 \ * 0.85 * -0.18)$

$\boldsymbol{p_x = 2.365\ kN/m^2}$

$\mathrm{p_y} = (2.962 \ * 0.85 * 0.963 * (0.8)) - (2.973 \ * 0.85 * -0.18)$

$\boldsymbol{p_y = 2.394\ kN/m^2}$

To find the Leeward Wind Pressure, the following equation is used

$$p = q_h K_d G C_p - q_h K_d (G C_{pi}) \quad \left(\frac{N}{m^2}\right)$$

1st Parking Floor to 5th Parking Floor

$$p_x = q_h K_d G_x C_{px} - q_h K_d (G C_{pi}) \quad \left(\frac{N}{m^2}\right)$$

$$p_y = q_h K_d G_y C_{py} - q_h K_d (G C_{pi}) \quad \left(\frac{N}{m^2}\right)$$

$$p_x = \big(2.962 * 0.85 * 0.92 * (-0.5)\big) - (2.962 * 0.85 * 0.18)$$

$$\boldsymbol{p_x = -1.611 \frac{kN}{m^2}, p_x = 1.611 \frac{kN}{m^2} acting\ outwards\ of\ the\ building}$$

$$p_y = \big(2.962 * 0.85 * 0.935 * (-0.478)\big) - (2.962 * 0.85 * 0.18)$$

$$\boldsymbol{p_y = -1.58 \frac{kN}{m^2}, p_y = 1.58\ kN/m^2 acting\ outwards\ of\ the\ building}$$

6th Floor to 80th Floor

$$p_x = \big(2.962 * 0.85 * 0.943 * (-0.48)\big) - (2.962 * 0.85 * 0.18)$$

$$\boldsymbol{p_x = -1.59\ kN/m^2, p_x = 1.59 \frac{kN}{m^2}\ acting\ outwards\ of\ the\ building.}$$

$$p_y = \big(2.962 * 0.85 * 0.956 * (-0.5)\big) - (2.962 * 0.85 * 0.18)$$

$$\boldsymbol{p_y = -1.657\ kN/m^2, p_y = 1.657 \frac{kN}{m^2}\ acting\ outwards\ of\ the\ building.}$$

81st Floor to 96th Floor

$$p_x = \big(2.962 * 0.85 * 0.944 * (-0.48)\big) - (2.962 * 0.85 * 0.18)$$

$$\boldsymbol{p_x = -1.59\ kN/m^2, p_x = 1.59 \frac{kN}{m^2}\ acting\ outwards\ of\ the\ building.}$$

$$p_y = \big(2.962 * 0.85 * 0.958 * (-0.5)\big) - (2.962 * 0.85 * 0.18)$$

$$\boldsymbol{p_y = -1.659\ kN/m^2, p_y = 1.659 \frac{kN}{m^2}\ acting\ outwards\ of\ the\ building.}$$

97th Floor and 98th Floor

$$p_x = \left(2.962 * 0.85 * 0.946 * (-0.5)\right) - (2.962 * 0.85 * 0.18)$$

$$\boldsymbol{p_x = -1.644\ kN/m^2\ \ ,p_x = 1.644\ \frac{kN}{m^2}\ acting\ outwards\ of\ the\ building.}$$

$$p_y = \left(2.962 * 0.85 * 0.96 * (-0.5)\right) - (2.962 * 0.85 * 0.18)$$

$$\boldsymbol{p_y = -1.66\ kN/m^2, p_y = 1.66\ \frac{kN}{m^2}\ acting\ outwards\ of\ the\ building.}$$

99th Floor and 100th Floor

$$p_x = \left(2.962 * 0.85 * 0.949 * (-0.5)\right) - (2.962 * 0.85 * 0.18)$$

$$\boldsymbol{p_x = -1.648\ kN/m^2\ ,p_x = 1.648\ \frac{kN}{m^2}\ acting\ outwards\ of\ the\ building.}$$

$$p_y = \left(2.962 * 0.85 * 0.963 * (-0.5)\right) - (2.962 * 0.85 * 0.18)$$

$$\boldsymbol{p_y = -1.66\ kN/m^2\ , p_y = 1.66\ \frac{kN}{m^2}\ acting\ outwards\ of\ the\ building.}$$

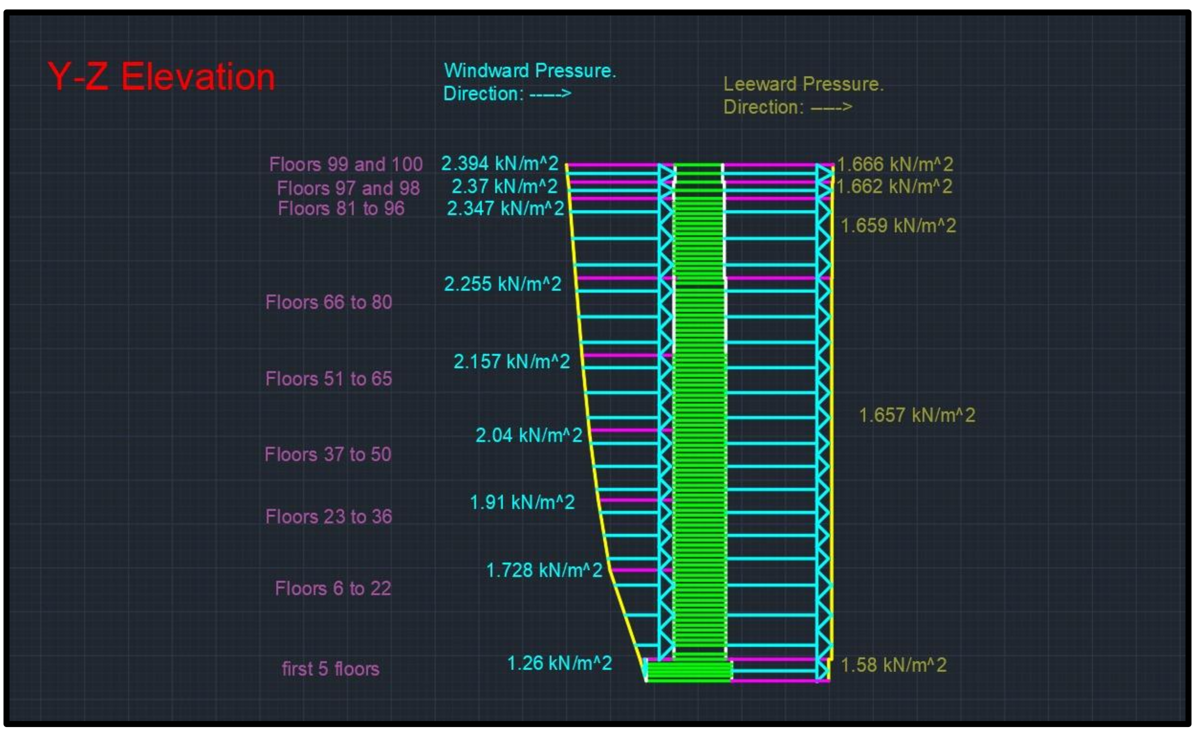


Figure 1.7.3.7.1 – Wind Pressure Distribution in Y-Direction

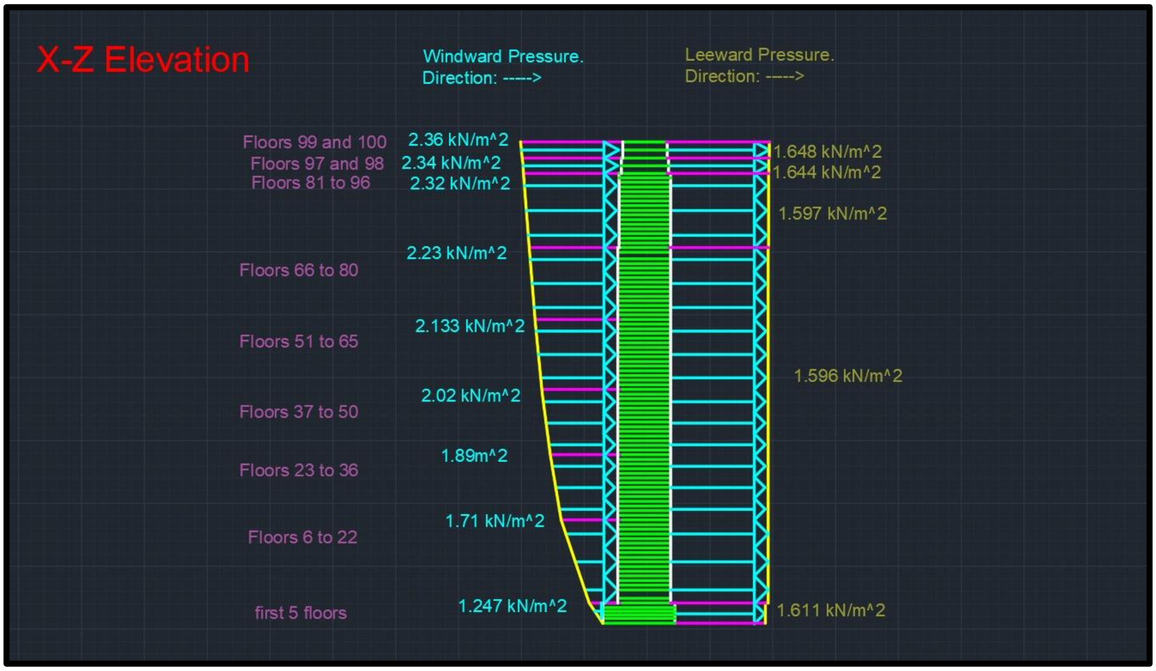


Figure 1.7.3.7.2 – Wind Pressure Distribution in X-Direction

1.8 Geotechnical Investigation

The type and the characteristics of the soil must be known to undergo the design process of the foundation by using appropriate type, and might be needed for the determination of seismic factors. The region specified is **Al Nahda, Dubai, UAE.**

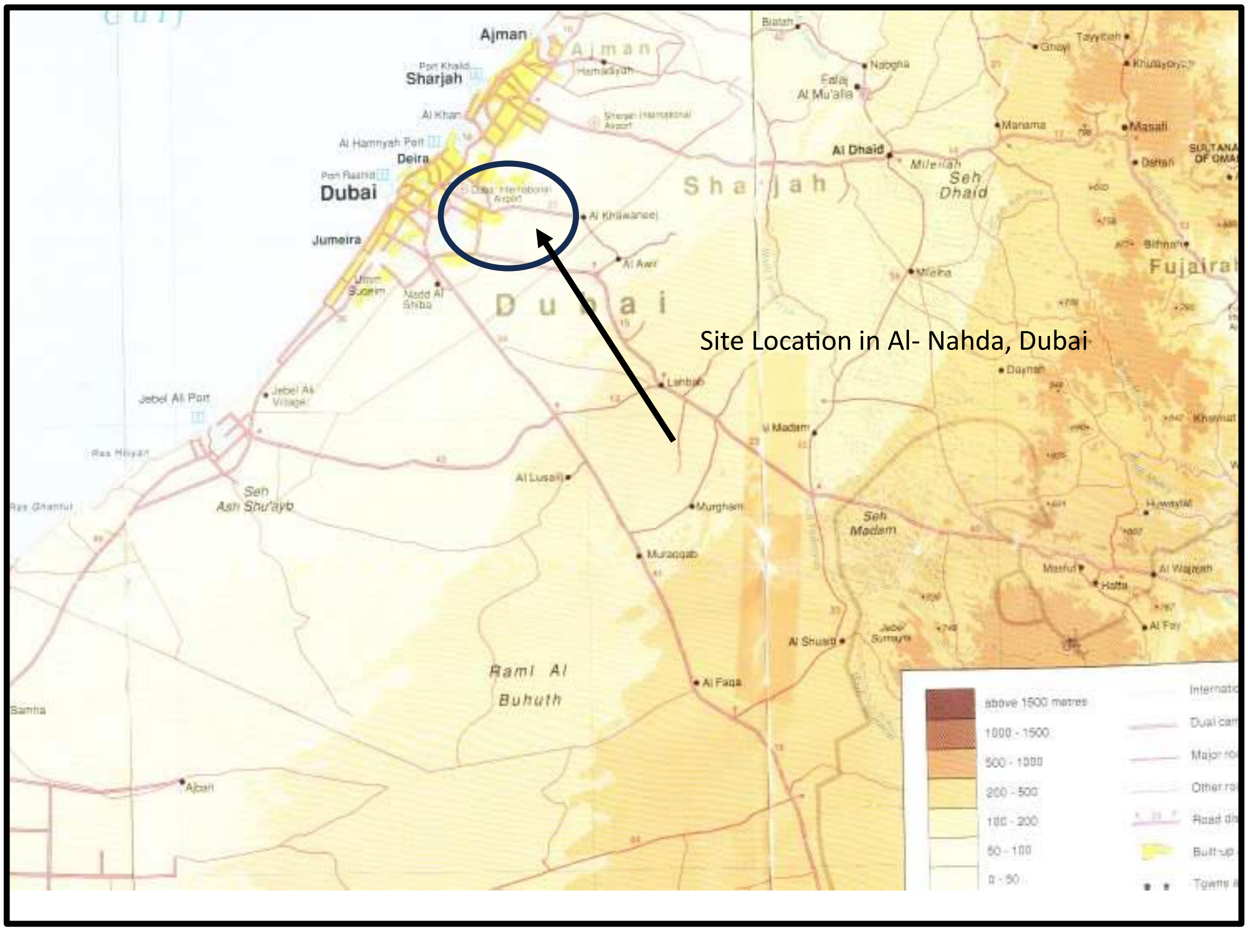


Figure 1.8.1 – Site Location

1.8.1 – Soil Report Results

According to a soil investigation report, the soil type in this region is as follows:

- Up to 8.5 meters of depth: the soil is medium dense to dense, light brown, slightly silty to silty, fine to medium **sand** soil.
- From 8.5 meters to 11.5 meters of depth: the soil is very dense, light brown, slightly silty to silty, fine to medium **sand** soil.
- From 11.5 meters to 17.5 meters of depth: the soil is very weak to weak, partially weathered, light brown, slightly indurated, fine to medium grained **sandstone.**
- **No recovery from 17.5 meters to 18.5 meters.**
- From 18.5 meters to 22 meters of depth: the soil is very dense, light brown, slightly silty to silty, fined to medium grained **cemented sand** soil.

- From 22 meters to 22.5 meters of depth: the soil is very weak to weak, partially weathered, light brown, slightly indurated, fine to medium grained **cemented sand/sandstone.**
- From 22.5 meters to 24 meters of depth: the soil is very dense, light brown, slightly silty to silty, fine to medium grained **cemented sand**.
- From 24 meters to 35 meters of depth: the soil is very weak to weak, partially weathered, light brown, slightly indurated, fine to medium grained **sandstone.**
- From 35 meters to 50 meters of depth: the soil is very weak to weak, partially weathered, pale reddish brown, slightly indurated, fine to medium grained **sandstone.**

1.8.2 - Ground Water

Ground water was encountered at 5.0 meters to 5.2 meters below the existing ground level.

It should be noted that ground water is affected by seasonal and tidal changes as well as dewatering in the vicinity.

BH No.	Depth of BH(m)	Groundwater Depth Below Existing Ground Level (m)
1	40.0	5.0
2	40.0	5.2
3	50.0	5.1
4	40.0	5.1
5	40.0	5.2

Table 17.8.2.1 – Ground Water Depths

1.8.3- Allowable Bearing Capacity and Working Loads.

Based on the results of the Standard penetration test (SPT) that was made, the following values of allowable bearing capacity and working loads are recommended:

Width of foundation (m)	Depth of Foundation below asphalt road level(m)	Allowable Bearing Capacity(kN/m^2)	Modulus of Sub-grade Reaction (kN/m^3)
≥12.0	**1.0 to 2.0**	**200**	**12000**
	2.0 to 4.0	**220**	**13200**

Table 1.8.3.1 – Allowable Bearing Capacities

1.8.4 – Piles Properties and Loads

Piles Allowable Working Loads in Compression (kN).

Pile toe BRL (m)	Pile Length (m)	Pile Diameter (mm)					
		600	700	800	900	1000	1200
-20	18	12698	16572	20950	25832	31214	43490
-21	19	12998	16924	21352	26284	31718	44092
-22	20	13300	17276	21754	26736	32220	44696
-23	21	13602	17628	22158	27188	32722	45300
-24	22	13904	17980	22560	27640	33226	45902
-25	23	14206	18332	22962	28094	33728	46506
-26	24	14506	18684	23364	28546	34230	47108
-27	25	14808	19036	23766	28998	34734	47712
-28	26	24460	32054	40654	50260	60872	85110
-29	27	25062	32758	41458	51166	61876	86316
-30	28	25666	33462	42264	52070	62882	87522
-31	29	26268	34166	43068	52974	63888	88728
-32	30	26872	34870	43872	53880	64892	89936
-33	31	27476	35572	44676	54784	65898	91142
-34	32	28078	36276	45480	55690	66904	92348
-35	33	28682	36980	46284	56594	67908	93554
-37	35	29840	38474	48154	58880	70651	97333
-42	40	50066	64550	80791	98787	118537	163303
-47	45	51067	65841	82407	100763	120907	166569
-52	50	52088	67158	84055	102779	123326	169901
-57	55	53130	68501	85736	104834	125792	173299
-62	60	54193	69871	87451	106931	128308	176765

Table 1.8.4.1 – Piles Allowable Working Loads in Compression

Pile toe BRL (m)	Pile Length (m)	Pile Diameter (mm)					
		600	700	800	900	1000	1200
-20	18	2554	2980	3406	3832	4258	5108
-21	19	2766	3226	3688	4148	4610	5532
-22	20	2976	3472	3968	4466	4962	5954
-23	21	3188	3720	4250	4782	5314	6376
-24	22	3398	3966	4532	5098	5664	6798
-25	23	3610	4212	4814	5416	6016	7220
-26	24	3822	4458	5094	5732	6368	7642
-27	25	4032	4704	5376	6048	6720	8064
-28	26	4454	5196	5940	6682	7424	8910
-29	27	4876	5690	6502	7316	8128	9754
-30	28	5300	6182	7066	7948	8832	10598
-31	29	5722	6674	7628	8582	9536	11442
-32	30	6144	7168	8192	9216	10240	12286
-33	31	6566	7660	8754	9848	10942	13132
-34	32	6988	8152	9318	10482	11646	13976
-35	33	7410	8646	9880	11116	12350	14820
-37	35	29841	38474	48154	58880	70651.	97333
-42	40	50066	64550	80791	98788	118537	163303
-47	45	55277	71269	89200	109070	130874	180300
-52	50	61030	78686	98484	120421	144496	199066
-57	55	67382	86876	108734	132955	159535	219785
-62	60	74395	95918	120051	146793	176139	242660

Table 18.8.4.2 – Piles Allowable Working Loads in Tension

Pile toe BRL (m)	Pile Length (m)	Pile Diameter (mm)					
		600	700	800	900	1000	1200
-17	15	1965386	2216708	2468040	2719360	2970688	3473346
-18	16	2015640	2266972	2518300	2769626	3020952	3523614
-19	17	2065906	2317234	2568570	2819894	3071224	3573874
-20	18	2116174	2367508	2618830	2870160	3121488	3624140
-21	19	2166440	2417772	2669100	2920426	3171752	3674406
-22	20	2216706	2468034	2719360	2970694	3222016	3724674
-23	21	2266974	2518298	2769630	3020952	3272280	3774940
-24	22	2317240	2568572	2819890	3071218	3322552	3825200
-25	23	2367506	2618834	2870160	3121484	3372816	3875466
-26	24	2417774	2669098	2920420	3171752	3423080	3925734
-27	25	2468040	2719360	2970690	3222018	3473344	3976000
-28	26	4076534	4579188	5081840	5584498	6087152	7092460
-29	27	4177066	4679714	5182370	5685022	6187680	7192994
-30	28	4277586	4780252	5282900	5785556	6288208	7293520
-31	29	4378136	4880778	5383430	5886088	6388744	7394054
-32	30	4478654	4981314	5483960	5986622	6489272	7494586
-33	31	4579186	5081840	5584500	6087146	6589808	7595114
-34	32	4679720	5182366	5685030	6187680	6690336	7695646
-35	33	4780254	5282902	5785560	6288214	6790864	7796174
-37	35	4973376	5496331	6019297	6542258	7065215	8111139
-42	40	9212649	10181368	11150107	12118836	13087558	15025021
-47	45	10171509	11241053	12310619	13380174	14449722	16588837
-52	50	16970471	18754933	20539431	22323911	24108379	27677347
-57	55	18667519	20630426	22593374	24556302	26519217	30445082
-62	60	20534270	22693469	24852712	27011932	29171139	33489590

Table 19.8.4.3 – Piles Vertical Stiffness

- Horizontal Stiffness of the pile should be taken as 25% of the vertical stiffness.
- Raft and group of piles allowable settlement taken as maximum of 200 mm.
- Apply factor of safety of 2.5 to the ultimate capacity.
- The analyses made above are for single pile cases without considering group effects, to ensure the pile configuration does not lead to overstressing of the ground, the distance between the piles measured center to center must not be less than 2.5 times the pile diameter.

1.8.5 – Seismic Zone and Soil Profile

Based on the soil strata encountered, and according to ASCE 7-16. Table 20.3-1, site classification, **Site Class "C"** may be adopted.

Site coefficients are as follows:

Site Class	Mapped Spectral acceleration at Short period, S_S	Mapped Spectral acceleration at 1 Second period, S_1	Mapped Spectral Response acceleration at Long term period, TL	Peak ground Acceleration PGA (g)	Site Coefficient Value, F_a for S_S=0.51	Site Coefficient Value, F_v for S_1=0.18
C	0.51	0.18	24	0.15	1.196	1.62

Table 1.8.5.1 – Site Seismic Coefficients

1.8.6 – Lateral Earth Pressure Coefficients and Calculations

The following table shows the lateral earth pressure coefficients for use in design of lateral support.

Depth Below existing ground level(m)	Estimated angle of internal friction(Ø)	Ka (Active)	Kp (Passive)	Ko (at Rest)
0.0 to 2.0	38	0.238	4.204	0.384
2.1 to 6.5	41	0.208	4.815	0.344
6.6 to 10.5	45	0.172	5.828	0.293

Table 20.8.6.1 – Lateral Earth Pressure Coefficients

The Lateral Earth Pressure at Rest is computed.

But to determine the lateral earth pressure, the soil unit weight is needed.

According to the soil report, the soil is dense uniform sand and loose uniform sand.

Table 3.1 Void Ratio, Moisture Content, and Dry Unit Weight for Some Typical Soils in a Natural State

Type of soil	Void ratio, e	Natural moisture content in a saturated state (%)	Dry unit weight, γ_d lb/ft³	Dry unit weight, γ_d kN/m³
Loose uniform sand	0.8	30	92	14.5
Dense uniform sand	0.45	16	115	18
Loose angular-grained silty sand	0.65	25	102	16
Dense angular-grained silty sand	0.4	15	121	19
Stiff clay	0.6	21	108	17
Soft clay	0.9–1.4	30–50	73–93	11.5–14.5
Loess	0.9	25	86	13.5
Soft organic clay	2.5–3.2	90–120	38–51	6–8
Glacial till	0.3	10	134	21

Table 21.8.6.2 – Soil Dry Unit Weights

$The\ Dry\ Unit\ Weight\ (\gamma_d)\ of\ Dense\ Uniform\ Sand\ is\ 18\ kN/m^3$

$The\ Dry\ Unit\ Weight\ (\gamma_d)\ of\ Loose\ Uniform\ Sand\ is\ 14.5\ kN/m^3$

$The\ Saturated\ Unit\ Weight\ (\gamma_{Sat})\ equals\ to{:}\ (\gamma_d) * (1+\omega)$

$Dense\ Uniform\ \gamma_{Sat} = 18 * (1+0.16) = 20.88\ kN/m^3$

$Loose\ Uniform\ \gamma_{Sat} = 14.5 * (1+0.3) = 18.85\ kN/m^3$

$Effective\ Unit\ Weight\ (\gamma') = \gamma_{Sat} - \gamma_{Water}$

$Dense\ Uniform\ \gamma' = 20.88 - 9.81 = 11.07\ kN/m^3$

$Loose\ Uniform\ \gamma' = 18.85 - 9.81 = 9.04\ kN/m^3$

The Soil Pressure Distribution at basement floors is as follows:

$Q_1 = \gamma * h * K_o = 18 * 3.4 * 0.384 = 23.5\ kN/m^2$

$Q_2 = 23.5 + 18 * 1.6 * 0.344 = 33.408\ kN/m^2$

$Q_3 = 33.4 + \gamma_{eff} * h * K_o = 33.4 + (20.88 - 9.81) * 1.45 * 0.344 = 38.93\ kN/m^2$

$Q_4 = 38.93 + \gamma_{eff} * h * K_o = 38.93 + (20.88 - 9.81) * 5.05 * 0.293 = 55.31\ kN/m^2$

$Q_5 = 55.31 + \gamma_{eff} * h * K_o = 55.31 + (18.85 - 9.81) * 7.15 * 0.393 = 74.33\ kN/m^2$

$Q_6 = 74.33 + \gamma_{eff} * h * K_o = 74.33 + (20.88 - 9.81) * 3.05 * 0.293 = 84.22\ kN/m^2$

Finally. the Soil Hysteretic Damping Ratio is given by Table 19.3-3 in ASCE 7-22:

Table 19.3-3. Soil Hysteretic Damping Ratio, β_s.

	Effective Peak Acceleration, $S_{DS}/2.5$[a]			
Site Class	$S_{DS}/2.5=0$	$S_{DS}/2.5=0.1$	$S_{DS}/2.5=0.4$	$S_{DS}/2.5 \geq 0.8$
C	0.01	0.01	0.03	0.05
CD	0.01	0.01	0.05	0.09
D	0.01	0.02	0.07	0.15
DE	0.01	0.03	0.12	0.20
E	0.01	0.05	0.20	[b]
F	[b]	[b]	[b]	[b]

[a] Use straight-line interpolation for intermediate values of $S_{DS}/2.5$.
[b] Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

Table 22.8.6.3 – Soil Hysteric Damping Ratio

Soil Hysteretic Damping Ratio = **0.01508**

1.8.7 Pressure on Basement Floors

For this section, soil, water, and traffic pressure will be calculated as follows.

1.8.7.1 - Effective Soil Pressure

A sample calculation for effective soil pressure for soil which is below water table will be shown below.

$$Effective\ soil\ pressure\ at = (\gamma_s - \gamma_w)(H)(K_o) = (20.88 - 9.81)(1.5)(0.344)$$
$$= \mathbf{5.71212\ KN/m^2}$$

Total effective pressure at this depth (Q3) = 32.4 + 5.71212 = **38.11212 KN/m²**.

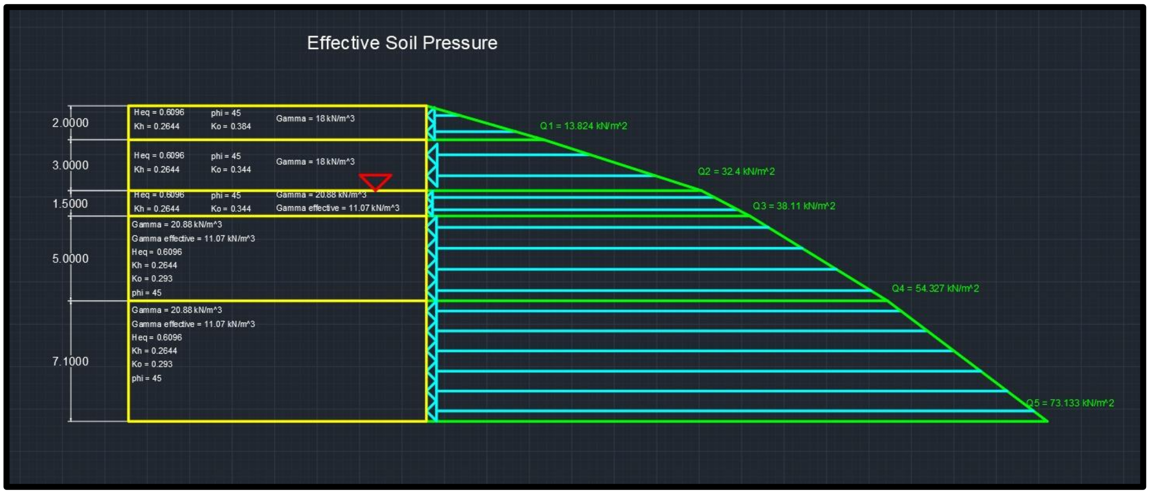


Figure 1.8.7.1.1 Soil Effective Pressure Distribution

1.8.7.2 Water Pressure

$$Total\ Water\ Pressure = \gamma_w * Total\ water\ height = 10\frac{KN}{m^3} * (1.5 + 5 + 7.1) = \mathbf{136\ KN/m^2}$$

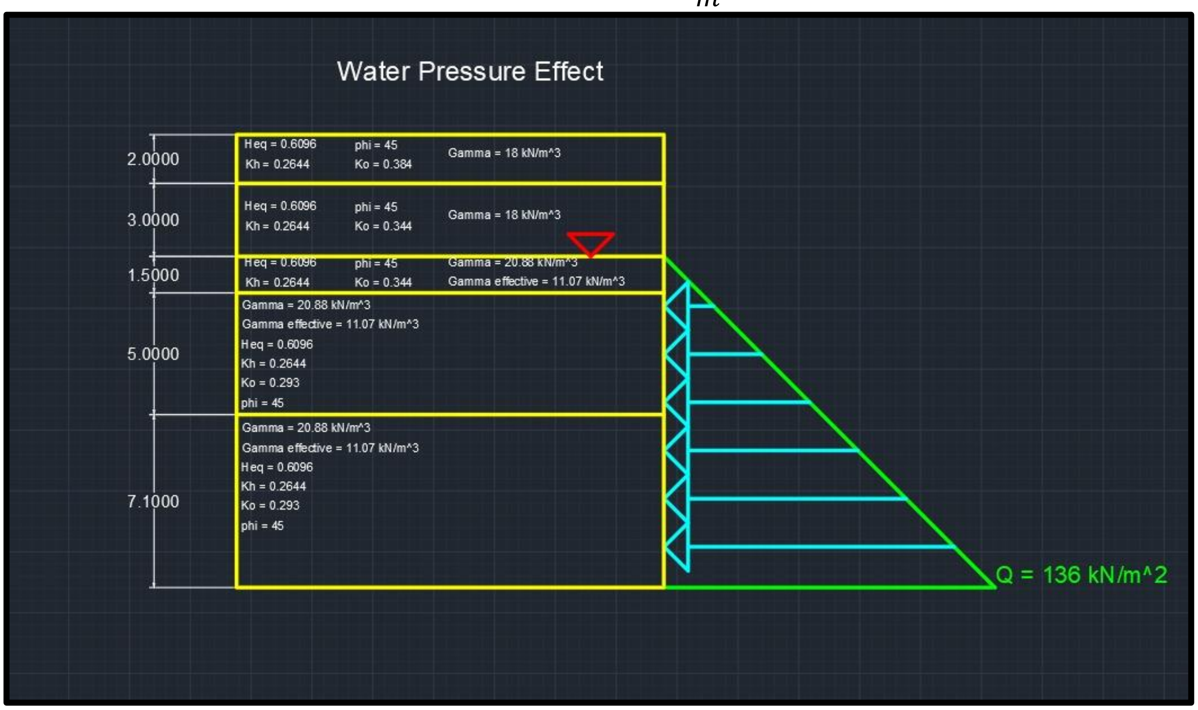


Figure 1.8.7.2.1 Water Pressure Distribution

1.8.7.3 Dynamic Pressure

This pressure is a result from seismic effect on the soil layers. A sample calculation will be shown as below.

$$\Delta p = 0.4 k_h \gamma_t H_{rw} \qquad (8\text{-}30)$$

where Δp = additional earth pressure caused by seismic shaking, which is assumed to be a uniform pressure;
k_h = horizontal seismic coefficient in the soil, which may be assumed equal to $S_{XS}/2.5$;
γ_t = total unit weight of soil;
H_{rw} = height of the retaining wall; and
S_{XS} = spectral response acceleration parameter, as specified in Section 2.4.

Figure 1.8.7.3.1 Soil Dynamic Pressure Parameters

$\Delta p = 0.4 k_h \gamma_t H_{rw}$

$k_h = \frac{S_{xs}}{2.5}$

$S_{xs} = F_a S_s$ (from ASCE 41-13)

$F_a = 1.296$

$S_s = 0.51$

$S_{xs} = 1.296 * 0.51 = 0.661$

$k_h = \frac{0.661}{2.5} = 0.2644$

$Q1 = 0.4 * 0.2644 * 18 * (2+3) = \mathbf{9.5184 \frac{KN}{m^2}}.$

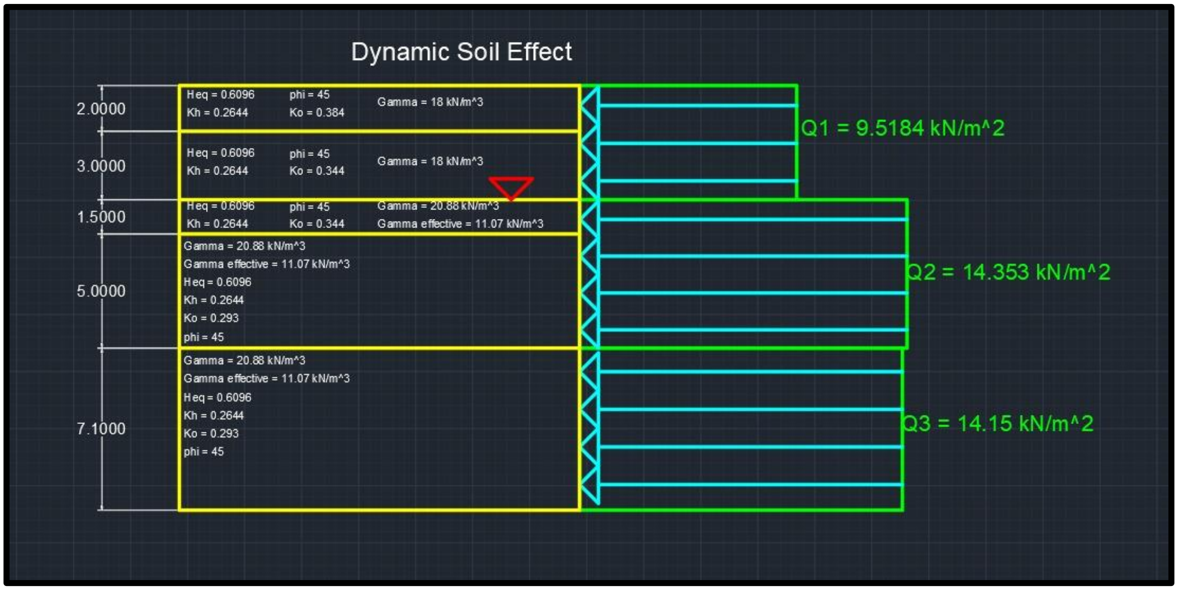


Figure 1.8.7.3.2 Soil Dynamic Pressure Distribution

1.8.7.4 Surcharge Pressure

This pressure is due to the repetitive traffic loading of the nearby roads and streets. Sample calculation will be explained below.

$$\Delta_p = k\gamma_s h_{eq} \qquad (3.11.6.4\text{-}1)$$

where:

Δ_p = constant horizontal earth pressure due to live load surcharge (ksf)
γ_s = total unit weight of soil (kcf)
k = coefficient of lateral earth pressure
h_{eq} = equivalent height of soil for vehicular load (ft)

Figure 1.8.7.4.1 Soil Surcharge Pressure Parameters

Table 3.11.6.4-2—Equivalent Height of Soil for Vehicular Loading on Retaining Walls Parallel to Traffic

Retaining Wall Height (ft)	h_{eq} (ft) Distance from wall backface to edge of traffic	
	0.0 ft	1.0 ft or Further
5.0	5.0	2.0
10.0	3.5	2.0
≥20.0	2.0	2.0

Table 1.8.7.4.2 Equivalent Heights of Soil for Vehicular Loading

Heq = 2ft = 0.6096 m

$$Q1 = k * \gamma_s * H_{eq} = 0.384 * 18 * 0.6096 = \mathbf{4.213}\,\frac{\boldsymbol{KN}}{\boldsymbol{m^2}}.$$

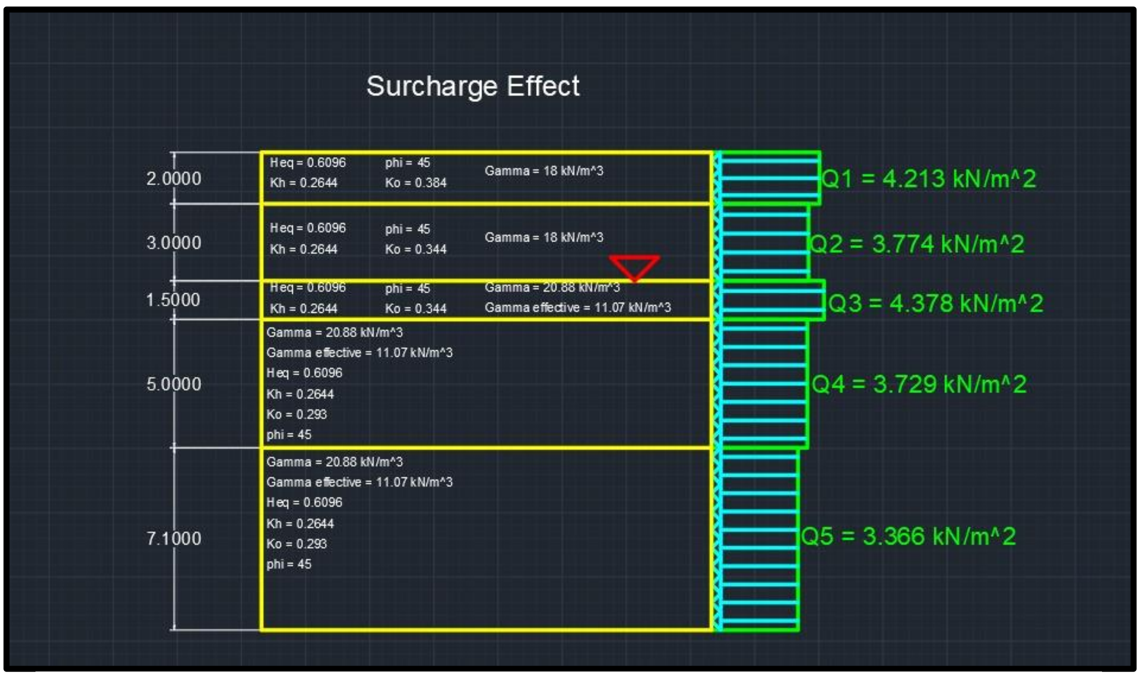


Figure 1.8.7.4.3 Soil Surcharge Pressure Distribution

1.8.7.5 Total Pressure on Basement Floors

This is the total pressure from all the above sectors at each depth.

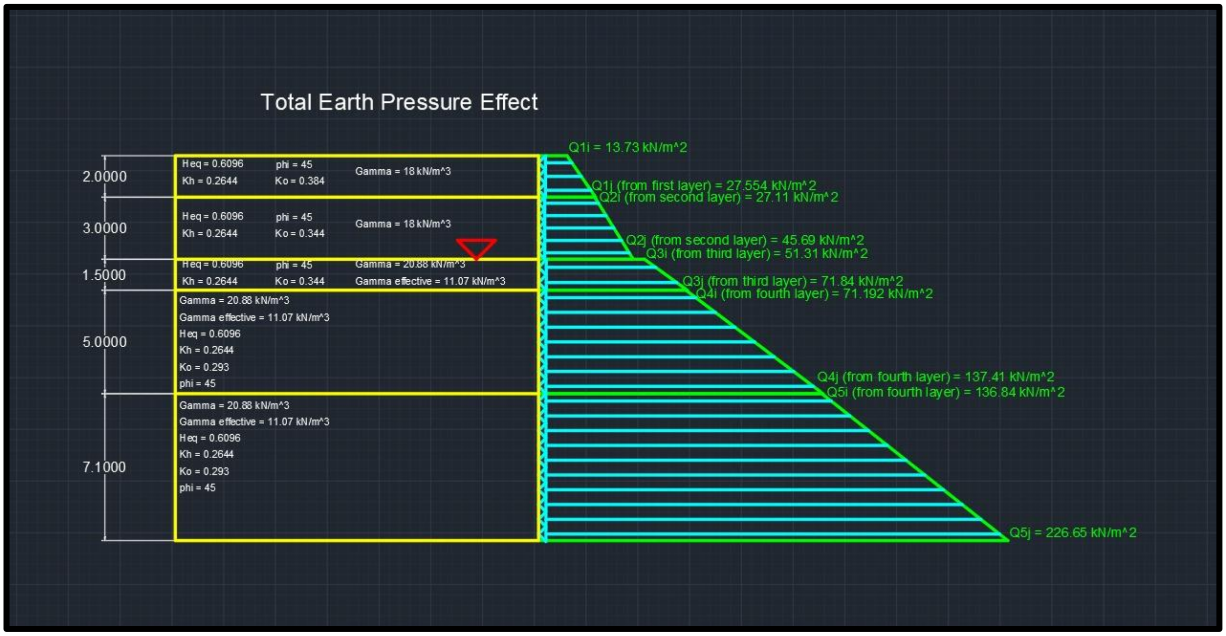


Figure 1.8.7.5.1 Total Earth Pressure Distribution

1.9 Structural systems for high rise buildings

After plans revision is done, modelling process is initiated. Firstly, structural system is chosen. Then, preliminary dimensions are defined for slabs, columns, beams, and shear walls. As a result, two separate structural models are produced on ETABS 20 software, verification checks are done and then conclusions are made up.

1) Braced Frame Structural System.

Braced Frames are basically cantilevered vertical trusses that mainly resist lateral loads, it is primarily composed of diagonal members, in addition to the girders, and they together form the web of the truss, and the columns are considered as the chords of the truss. A braced frame structural system is a type of structural system that uses diagonal braces to resist lateral loads such as wind or seismic forces. It typically consists of a series of vertical columns and horizontal beams that are connected by diagonal braces, forming a rigid frame. The braces are placed in a diagonal pattern to create a triangulated shape, which helps to distribute loads evenly and prevent the frame from buckling or collapsing under stress. Braced frames are commonly used in high-rise buildings, industrial structures, and bridges. They offer a cost-effective and efficient solution for resisting lateral loads, while also allowing for flexible interior layouts and architectural design.

Advantages:

- Stability: Braced frames provide a high level of lateral stability, which is particularly important in regions with high seismic activity or strong winds.
- Efficiency: Braced frames are often more efficient in terms of material usage and cost than other structural systems, such as moment frames or shear walls.
- Flexibility: The location of the braces within the structure can be adjusted to accommodate changes in building use or configuration.
- Aesthetics: The diagonal braces in braced frames can provide an architectural feature that is visually appealing and unique.

Disadvantages:

- Space: Bracing members can take up significant amounts of space within the building, reducing usable floor area.
- Aesthetics: Although the diagonal braces can be visually appealing, they can also be seen as a hindrance to interior design and space layout.
- Cost: Bracing can add to the cost of the structure, depending on the design and material used.
- Maintenance: Over time, the braces can become damaged or corroded, requiring regular inspection and maintenance to ensure their continued effectiveness.

2) Rigid Frame Structural System.

In this structural system, beams and columns are constructed every member on its own and the members are connected by rigid connections to withstand moments and sways imposed due to loads, however, the lateral stiffness of this rigid frame depends of the bending stiffness of the columns, girders, and connections in-plane. A rigid frame structural system is a type of construction method that is commonly used in buildings, bridges, and other large structures. It involves a series of rigid beams or columns that are connected to form a single, stable unit. The joints between the beams or columns are designed to be fixed, meaning that they cannot move relative to one another. This allows the system to resist external loads and forces without deforming or collapsing. Rigid frame structural systems are known for their strength, stability, and durability, and they are widely used in modern construction projects.

Advantages:

- Strength and stability: Rigid frames are designed to provide strong and stable support to the structure. The frames are designed to withstand lateral loads, such as wind or earthquakes.
- Durability: Rigid frames are designed to be durable and long-lasting, which makes them ideal for supporting heavy loads, such as large roofs or walls.
- Ease of construction: Rigid frames are relatively easy to construct and install. They can be pre-fabricated off-site and then assembled on-site, which can save time and reduce labor costs.

- Flexibility: Rigid frames can be designed to accommodate a range of architectural designs and building types. This means they can be used in a variety of construction projects, from large commercial buildings to smaller residential homes.

Disadvantages:

- Cost: Rigid frame structural systems can be more expensive to construct than other structural systems, such as steel or wood framing.

- Maintenance: Over time, rigid frames may require maintenance and repair, which can be costly and time-consuming.
- Limited flexibility: Rigid frames are designed to be strong and stable, which means they may be less flexible than other structural systems. This can make it difficult to make changes to the building's design or layout.
- Seismic concerns: In areas prone to earthquakes, rigid frames may not be the best choice for structural systems. The frames may be susceptible to damage from lateral loads and may not provide adequate seismic resistance.

3) Shear Wall System.

It is a continuous vertical wall constructed along the building’s longitudinal axis, and it is constructed from reinforced concrete or masonry wall, shear walls are also known to be constructed as cores for the building to enhance the building’s stability. Shear wall systems are structural systems used in buildings to resist lateral loads, such as wind and earthquake forces. These systems typically consist of vertical walls made of reinforced concrete, masonry, or wood, which are anchored to the foundation and connected to the building's framing system. Shear walls work by transferring lateral loads from the roof and floors to the foundation, thereby reducing the building's lateral deflection and preventing damage or collapse. In addition to their structural function, shear walls can also serve as partitions, providing privacy and sound insulation between rooms. Overall, shear wall systems are a common and effective way to provide lateral stability to buildings in areas prone to seismic activity or high winds.

Advantages:

1. High stiffness: Shear walls are very stiff, which means they can resist lateral loads better than other types of structural systems.
2. Cost-effective: The use of shear walls can reduce the overall cost of a building as it eliminates the need for additional structural members like columns or beams.
3. Speed of construction: Shear walls can be constructed quickly, especially when precast or prefabricated panels are used.
4. Efficient use of space: Shear walls occupy a small amount of floor space compared to other structural systems, making them ideal for buildings with limited space.
5. Durability: Shear walls are highly durable and can withstand severe weather conditions.

Disadvantages:

1. Limited flexibility: Shear walls provide limited flexibility when it comes to changing the layout of a building or adding additional floors.
2. Limited openings: Shear walls limit the number and size of openings that can be made in the wall for windows or doors, which can affect the overall aesthetic appeal of the building.
3. Structural integrity issues: The structural integrity of a building with shear walls can be compromised if the walls are not properly designed or constructed.
4. High seismic loads: Shear walls are not very effective at resisting high seismic loads.
5. Limited design options: The use of shear walls limits the design options available to architects and engineers, which can make it difficult to achieve a unique or innovative design.

4) Coupled Wall System.

This system is comprised of two or more interconnected shear walls, these shear walls are also connected at the floor levels by a beam or stiff slab, this system is also suitable for buildings up to 40 story heights. A coupled wall structural system is a type of lateral load-resisting system used in buildings, consisting of two or more parallel concrete walls connected by horizontal beams or slabs. The walls are designed to work together to resist lateral forces such as wind or earthquakes, providing increased strength and stiffness compared to individual walls. This system is commonly used in high-rise buildings where lateral loads are significant and where the use of reinforced concrete walls as the primary lateral load resisting system is required.

Advantages:

1. Increased stiffness and strength: Coupled wall structural systems provide higher stiffness and strength compared to individual walls. They can resist lateral loads such as wind and earthquakes more effectively, which is particularly important for tall buildings.
2. Enhanced torsional resistance: Coupled walls can resist torsional forces more effectively than individual walls. This makes them ideal for buildings with irregular shapes or layouts.
3. Reduced floor-to-floor heights: Coupled walls have thinner sections than individual walls, which means that they take up less space. This allows for reduced floor-to-floor heights and can result in higher building efficiency.
4. Better acoustic performance: The rigidity of coupled walls can also help to reduce noise transmission between floors and rooms.

Disadvantages:

1. Limited flexibility: The rigidity of coupled walls can limit the flexibility of building design. This means that any changes to the building layout or design can be difficult and expensive.
2. Increased construction time: Coupled wall systems require more precise construction than other lateral load resisting systems. This can result in increased construction time and costs.
3. Increased weight: Coupled wall systems are typically heavier than other lateral load resisting systems, which can increase the overall weight of the building. This can lead to increased foundation costs and may limit the height of the building.
4. Higher cost: Coupled wall systems are generally more expensive than other lateral load resisting systems due to their increased complexity and construction requirements.

5) Wall frame system (dual system)

A wall frame structural system is a type of construction where the weight of a building is supported by a framework of vertical and horizontal members called studs and plates, respectively, which are made of wood, steel, or concrete. The studs are placed at regular intervals, typically 16 or 24 inches apart, and the plates connect them at the top and bottom. The wall frame structural system is widely used in residential and commercial construction because of its cost-effectiveness, ease of construction, and flexibility in design.

Advantages:

1. Cost-effective: Wall frame structures are relatively affordable and cost-effective to build, especially compared to other types of structural systems such as concrete or steel.
2. Lightweight: Wall frame structures are lightweight, which makes them easier to construct, transport and assemble. They can also be erected quickly, saving time and labor costs.
3. Versatile: Wall frame structures can be designed to suit a range of building styles and shapes, from simple rectangular shapes to more complex and irregular designs.
4. Energy-efficient: Wall frame structures can be constructed to meet high energy efficiency standards. They can be insulated to a high level, which can significantly reduce energy consumption and lower energy costs.

Disadvantages:

1. Susceptible to water damage: Wall frame structures are vulnerable to water damage, which can cause rot and decay in the wooden frames over time. This can weaken the structure and make it unsafe.
2. Prone to fire damage: Wooden wall frame structures are also susceptible to fire damage. Once a fire starts, it can spread rapidly through the wooden frames, causing significant damage and putting occupants at risk.
3. Limited span: Wall frame structures have a limited span, which means that they are not suitable for larger buildings or spaces that require wide open floor plans.
4. Maintenance: Wall frame structures require regular maintenance to ensure that the wooden frames remain structurally sound and free from damage. This can be time-consuming and costly over the lifespan of the building.

6) Core and outrigger system

The core and outrigger structural system is a common design approach used in high-rise buildings to increase their stability and resistance to lateral loads, such as wind and earthquakes.

The core is a vertical structural element located at the center of the building, typically housing elevators, stairwells, and mechanical equipment. The core is designed to resist vertical loads and provide lateral stability to the building.

The outrigger is a horizontal structural element that connects the core to the perimeter columns of the building. The outrigger acts as a cantilever, transferring the lateral loads from the perimeter columns to the core, which then distributes the loads to the foundation.

By combining the strength of the core and outrigger, this system helps to increase the building's stiffness, reduce sway, and mitigate the effects of lateral loads. This design approach allows for taller and more slender buildings to be constructed in areas with high winds or seismic activity, while maintaining structural stability and safety.

Advantages of Core Structural System:

- The core system involves a central vertical column or set of columns that supports the weight of the building, providing greater stability and resistance to lateral forces such as wind or earthquakes.
- This system allows for more flexibility in the placement of interior walls and spaces, as the core provides the necessary structural support.
- The core system can also reduce the number of materials and space required for structural elements, making it a more efficient use of space.

Disadvantages of Core Structural System:

- The core system can limit the amount of natural light and ventilation that can be brought into the building, as it can occupy a significant portion of the building's interior.
- The central column(s) can also create challenges for building design and aesthetics, as it can be difficult to integrate into the building's overall look and feel.

Advantages of Outrigger Structural System:

- The outrigger system involves a series of horizontal trusses or beams that connect the exterior walls of the building to a central core or other support structure, providing increased lateral stability and resistance to forces.
- This system can allow for larger and more open interior spaces, as the external walls do not need to provide as much structural support.
- The outrigger system can also create opportunities for unique and interesting building designs, as the trusses or beams can be visually integrated into the building's exterior.

Disadvantages of Outrigger Structural System:

- The outrigger system can require more materials and space for structural elements, which can increase construction costs and reduce usable floor space.
- The design of the outrigger system can also be more complex and challenging to implement, requiring careful coordination between architects and engineers.

7) Tube structural system

Tube structural system is a design approach used in high-rise building construction, which involves using a framework of closely spaced columns, interconnected by horizontal beams and vertical braces, to form a rigid tube-like structure. The system offers several advantages, including improved resistance to lateral loads, increased stiffness, and strength, and reduced overall weight and material costs. The tube structural system is commonly used in skyscraper designs to achieve greater height, stability, and structural integrity.

There are four types of tube structural system:

- Framed tube structural system
- Braced frame / Trussed tube system
- Bundled tube structural system
- Tube in tube system

Advantages:

1. Structural efficiency: The tube structural system is highly efficient in terms of the amount of material used to construct the building. The tubes are interconnected, creating a very strong and rigid structure that can resist high loads with minimal deflection.
2. Increased flexibility: The tube structural system allows for more flexibility in the design of the building. Because the tubes can be arranged in different patterns and configurations, architects and engineers have more freedom to create unique and innovative structures.
3. Better seismic performance: Tube structures are known for their ability to withstand earthquakes. The interconnected tubes distribute the seismic forces throughout the building, reducing the likelihood of structural damage during an earthquake.
4. Improved energy efficiency: Buildings designed using the tube structural system often have better energy efficiency because of the improved thermal insulation provided by the structural system.

Disadvantages:

1. Cost: The tube structural system is often more expensive to construct than other structural systems because of the complex fabrication and installation process required to create the interconnected network of tubes.
2. Limited availability: The tube structural system is not widely used in the construction industry, so finding experienced engineers and contractors who are familiar with the system can be challenging.
3. Space constraints: The interconnected tubes take up more space than other structural systems, which can limit the amount of usable space within the building.
4. Maintenance: Maintenance of the tube structural system can be challenging because of the complex network of interconnected tubes. Any damage or corrosion to the tubes can be difficult to detect and repair.

8) Hybrid structural system

A hybrid structural system combines two or more different types of structural systems to create a building that is more resilient, efficient, and adaptable to different conditions. The system is designed to take advantage of the strengths of each individual component, while minimizing their weaknesses. For example, a hybrid system might combine a steel frame with a reinforced concrete core, or a timber frame with a steel truss. Hybrid structural systems can be used to create buildings that are better able to withstand earthquakes, hurricanes, and other natural disasters, as well as buildings that are more energy efficient, sustainable, and adaptable to changing needs over time.

Advantages:

1. Enhanced Performance: By combining different structural systems, a hybrid system can provide better performance in terms of structural strength, stability, and resilience.
2. Cost Savings: Depending on the specific materials and systems chosen, a hybrid system can be more cost-effective than a traditional single system.
3. Flexibility: A hybrid system can be tailored to meet the specific needs of a project, allowing for greater flexibility in design.
4. Environmental Sustainability: Some hybrid systems incorporate sustainable materials or construction methods, reducing the environmental impact of a building.

Disadvantages:

1. Complexity: A hybrid system can be more complex to design and construct than a traditional system, requiring additional expertise and coordination among different types of contractors.
2. Maintenance: With multiple systems in place, maintenance and repairs can become more challenging and expensive.
3. Code Compliance: Depending on the specific codes and regulations in place, a hybrid system may require additional approval and testing to meet standards for each system component.
4. Uncertainty: Because hybrid systems are relatively new and not as widely used, there may be uncertainty around their long-term performance and durability.

9) Buttressed core structural system

The buttressed core structural system is a type of building design that uses a central core of reinforced concrete or steel surrounded by external columns and diagonal braces. The core provides the main support for the building and is strengthened by buttresses, which are vertical or inclined structural elements that transfer lateral forces to the foundation. This system is commonly used in tall buildings to increase their stability and resistance to wind and seismic loads. The buttressed core structural system is a highly efficient and cost-effective solution for constructing high-rise buildings.

Advantages:

1. Enhanced stability: The buttresses provide additional support and help distribute the load evenly, making the building more stable and resistant to lateral forces such as earthquakes and strong winds.
2. More open floor plans: The reinforced core allows for fewer columns in the interior of the building, making it possible to create larger, more open floor plans.
3. Increased flexibility: The buttressed core can be designed to accommodate changes in the building's use over time, allowing for greater flexibility in how the space is utilized.
4. Improved aesthetics: The buttresses can add visual interest and texture to the exterior of the building, creating a unique and striking architectural style.

   Disadvantages:

1. Cost: The reinforced concrete core and exterior buttresses can be more expensive to construct than other types of structural systems.
2. Limited design options: The buttresses can limit the design options for the building, as they need to be placed strategically to provide the necessary support.
3. Reduced usable floor space: The buttresses take up additional floor space and can reduce the amount of usable space in the building.
4. Potential maintenance issues: The exterior buttresses may require regular maintenance and repair, which can be costly and time-consuming.

1.10 Optimum and Seismic Force Resisting Systems

1.10.1 Optimum Structural Systems

After studying the above explained structural system, it was decided to choose two separate high rise building systems which are tube in tube system (also called tube system) and outrigger system.

As for tube in tube system, this is the original system which was already installed in the plans for the tower. Also, intensity of shear walls distribution enhances the use of tube in tube system. Moreover, there are several advantages which can be summarized as it is very efficient in terms of structural integrity since putting such system into use gives higher deflection allowance with additional loads resistance. The tube structural system offers increased flexibility in building design, allowing architects and engineers to create unique and innovative structures by arranging the tubes in different patterns and configurations. Tube structures are also known for their ability to withstand earthquakes, with interconnected tubes distributing seismic forces throughout the building and reducing the likelihood of structural damage. Additionally, buildings designed with the tube structural system often have improved energy efficiency due to the better thermal insulation provided by the system.

On the other hand, outrigger is suitable since in the tower's stories breakdown there is several service floors for the residence stories but the service floors are separated from each other. There are many advantages for outrigger system such as it uses horizontal trusses or beams to connect the building's exterior walls to a central support structure, which provides better lateral stability against external forces. It allows for more significant and open interior spaces, as the external walls do not need to provide as much structural support. Additionally, this system can lead to unique and visually appealing building designs as the trusses or beams can be integrated into the building's exterior.

1.10.2 Seismic Force Resisting System

To determine the seismic force resisting system, it is needed to check the percentage of loads that are being held by the columns and walls. Live load case will be taken into consideration.

Base Reactions

File Edit Format-Filter-Sort Select Options

Units: As Noted Hidden Columns: No Sort: None Base Reactions

Filter: None

Output Case	Case Type	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m	X m	Y m
Live	LinStatic	7.815E-07	-0.0001	494404.6432	23137159	-27188817	-0.0035	0	0

Figure 1.10.2.1 Live Load Total Base Reaction

Total live load on the building is 494404.6432 kN.

After checking the base reactions, the columns are bearing a total of 99578 kN of live load.

Columns' percentage = $\frac{Live\ Load\ beared\ by\ columns}{Total\ Live\ Load} = \frac{99578}{494404.6432} * 100\% = \mathbf{20}\%$

So, the walls are bearing 80% of gravity loads.

So, it is deduced that the seismic force resisting system **is Bearing Wall System, Special Reinforced Shear Walls.**

2 PRELEMINARY DIMENSIONS AND THREE-DIMENSIONAL ANALYSIS.

2.1 Preliminary Dimensions

Preliminary dimensions for columns were not modified, same as original structural plans. **C1 has a section of 1200 mm *1500 mm and C2 has a section of 1200 mm *1600 mm.**
Preliminary dimensions will be determined for slabs, beams, and tensile members.

As for slabs, the slab system used is flat plate and there are three slabs with different thicknesses. Slabs are as follows:

1. **Parking slab, h = 500 mm.**
2. **Floor slab, h = 400 mm.**
3. **Circular slab, h = 550 mm.**

To continue, beams thicknesses and widths are determined as shown below:

- **Edge Beam at Parking Level**

  In such cases, worst case must be taken into consideration in the calculations. This means longest span in the parking floor plan and one end continuous beam equation, L/18.5.

  Longest span = 9.25 m, $\boldsymbol{h_{min} = \frac{L}{18.5} = \frac{9250}{18.5} = 500\ mm}$ **.**

  But since parking slab thickness is 500 mm, and this beam is a drop beam, thus drop distance equals 250 mm. As a result, parking edge beam thickness is 750 mm.

Moreover , since it is a one edge continuous beam then width will be calculated using the equation , $\boldsymbol{Width\ of\ beam = \frac{L}{20} = \frac{9250}{20} = 462.50\ mm}$ $\boldsymbol{, then\ take\ beam\ width\ as\ 500\ mm}$ **.**

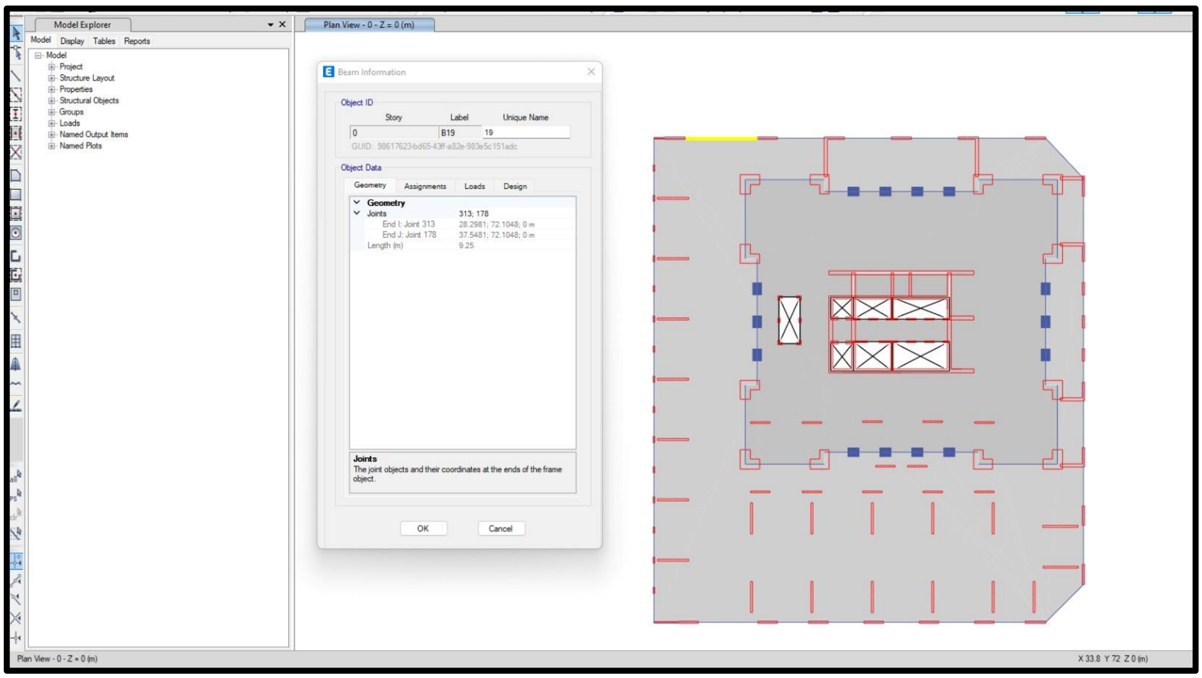


Figure 2.1.1 – Edge Beam at Parking Level Span

- **Edge Beam at Floor Level**

Same as parking edge beam, worst condition must be considered throughout the calculations. For this beam, the worst condition is **simply supported beam**. Longest span is 6.475 m.

$\boldsymbol{h_{min} = \frac{L}{16} = \frac{6457}{16} = 403.57\ mm\ , take\ it\ 450\ mm}$ **. But since it is used as a drop beam in all floors except for parking floors, then largest slab thickness must be used.**

$\boldsymbol{h = Circular\ slab\ thickness + drop\ distance = 550 + 250 = 800\ mm}$ **. Then thickness of floor edge beam is 800 mm.**

As for width, $\boldsymbol{Width} = \frac{\boldsymbol{L}}{\mathbf{20}} = \frac{\mathbf{6457}}{\mathbf{20}} = \mathbf{322.85}\ , \boldsymbol{then\ width\ of\ beam\ is}\ \mathbf{350}\ \boldsymbol{mm}$.

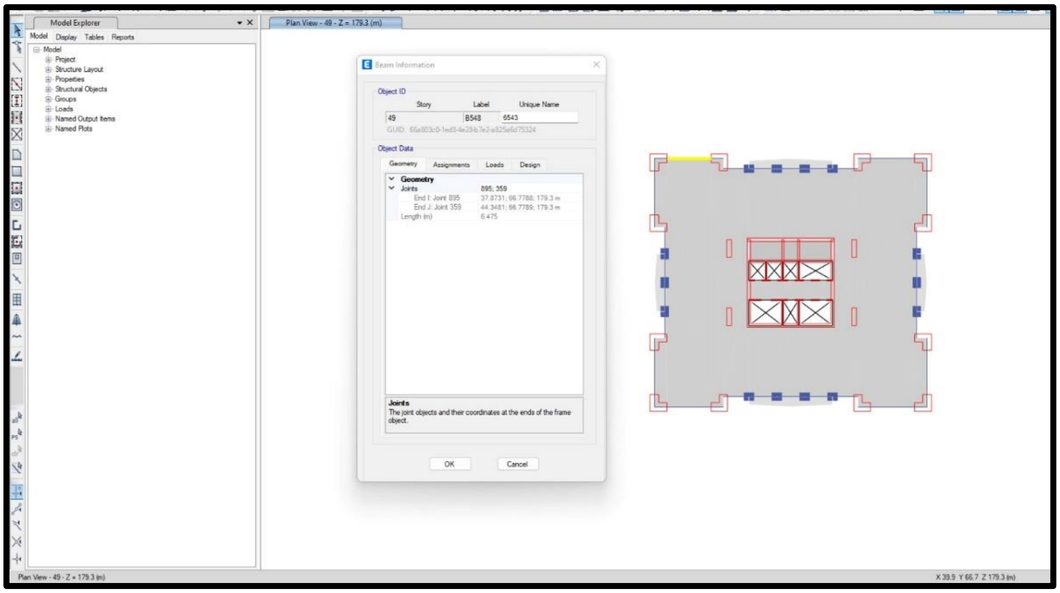


Figure 2.1.2 – Edge Beam at Floor Level Span

Furthermore, as for tensile members preliminary dimensions is that one of these tensile members are diagonal ones which are found in floors 98 and 99 (typical floors).
The following drawing shows a section at floor 99

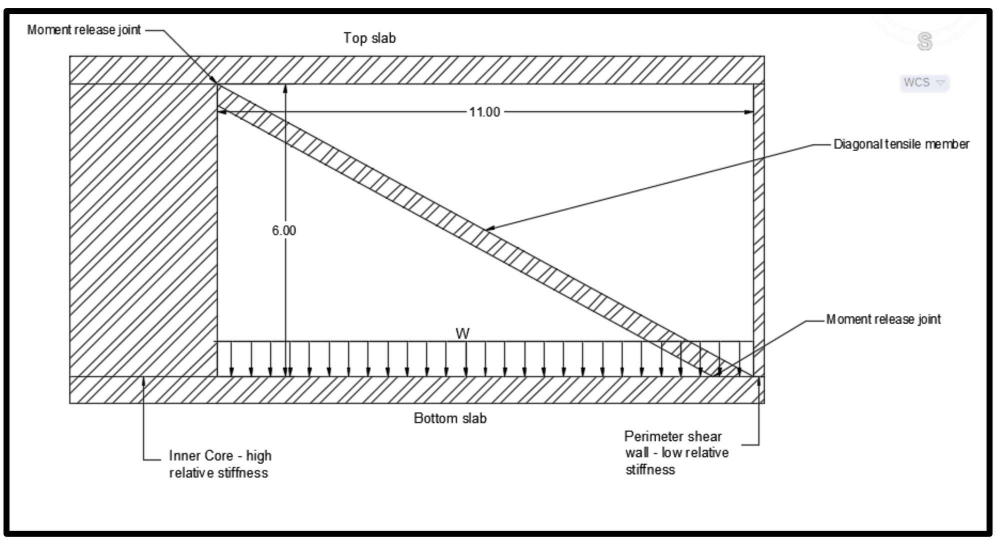


Figure 2.1.3 – Diagonal Tensile Member Longitudinal Section

Radius of circular slabs (R_t) = **17.3 m**

Radius of inner slab to pursue tributary area = $R_t - \frac{1}{2}(largest\ span) = 17.3 - \frac{1}{2}(11) = \mathbf{11.8\ m}$

Area of slab portion that will contribute in the determination of axial force in the tensile member, $A = A = \pi\left({R_t}^2 - {R_{inner}}^2\right) = \pi(17.3^2 - 11.8^2) = \mathbf{502.8\ m^2}$

$W = Live + Dead + SD = 5 + 4.5 + 0.55(25) = \mathbf{23.25\ \frac{KN}{m^2}}$

Total vertical forces on tensile member lower end (for all members combined),

$T_{vt} = A * W = 502.8 * 23.25 = \mathbf{11690\ KN}$

Number of diagonal tensile members in each of floors 98 and 99 = 20.

$$T_v = \frac{11690}{20} = \mathbf{584.50\ KN}$$

$$T = \frac{\sqrt{6^2 + 11^2}}{6}(584.50) = \mathbf{1220.436\ KN}$$

To determine the dimensions of the member section (only steel will withstand axial tensile force):

$As = \dfrac{T}{fy} = \dfrac{122.436 * 10^3}{420} = \mathbf{2905.80\ mm^2}$

Assume As is 1% of Ag, then $Ag = \frac{1586.50}{0.01} = \mathbf{290580\ mm^2}$

For a square section, $S = \sqrt{Ag} = \sqrt{290580} = \mathbf{539.05\ mm}$

Then take **550 mm * 550 mm** for concrete section for diagonal tensile strength members.

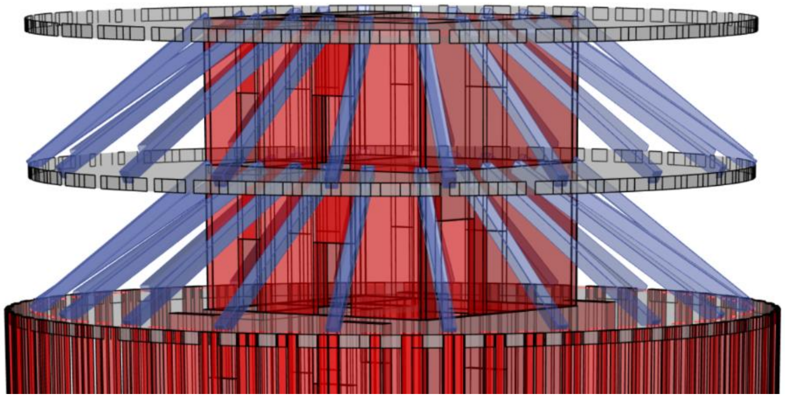


Figure 2.1.4 – Diagonal Tensile Members Side View

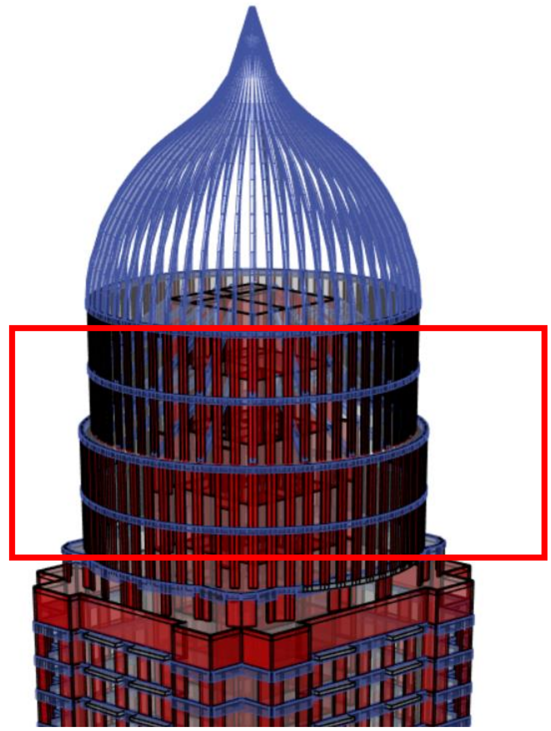


Figure 2.1.5 – Diagonal Tensile Members Outside View

2.2 First Model 3D Structural Analysis

To start with the analysis process, we need to draw a structural model for the tower using ETABS20 software for structural analysis process.

2.2.1 Columns' sections and modifiers

There are two column sections, each of their own rebar reinforcement and modifiers. More details will be shown in the below screenshots

1. C1 section and modifiers

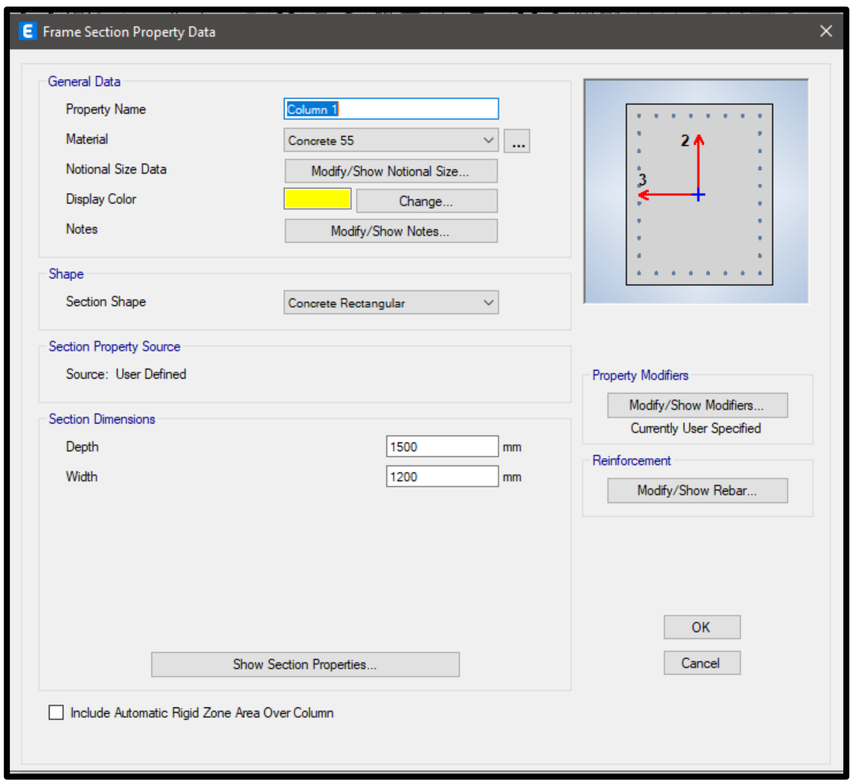


Figure 2.2.1.1 – C1 Section Details

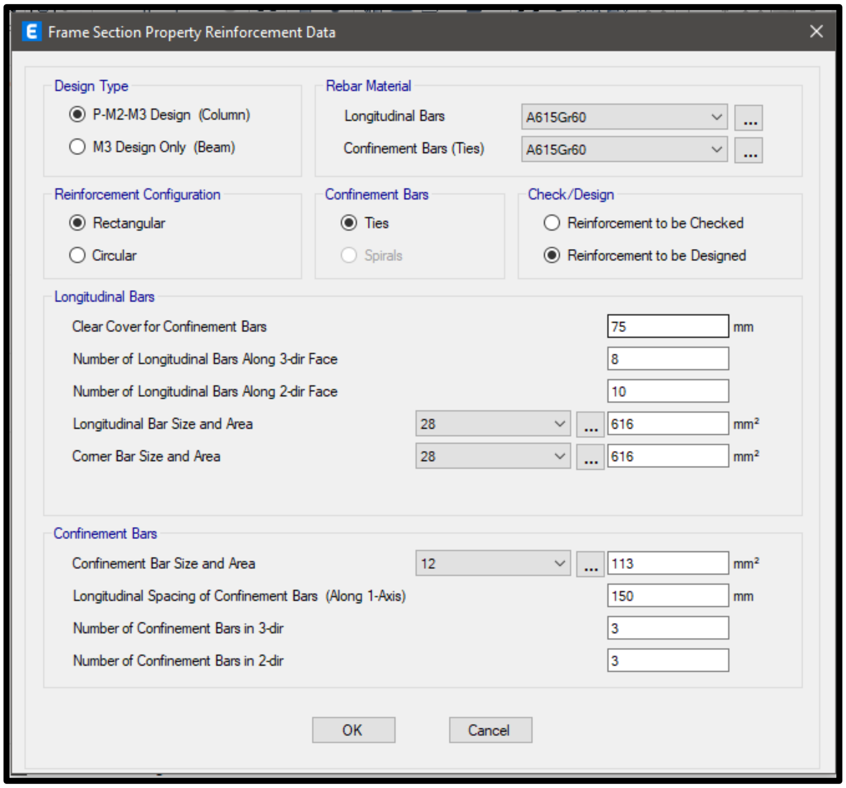


Figure 2.2.1.2 – C1 Reinforcement

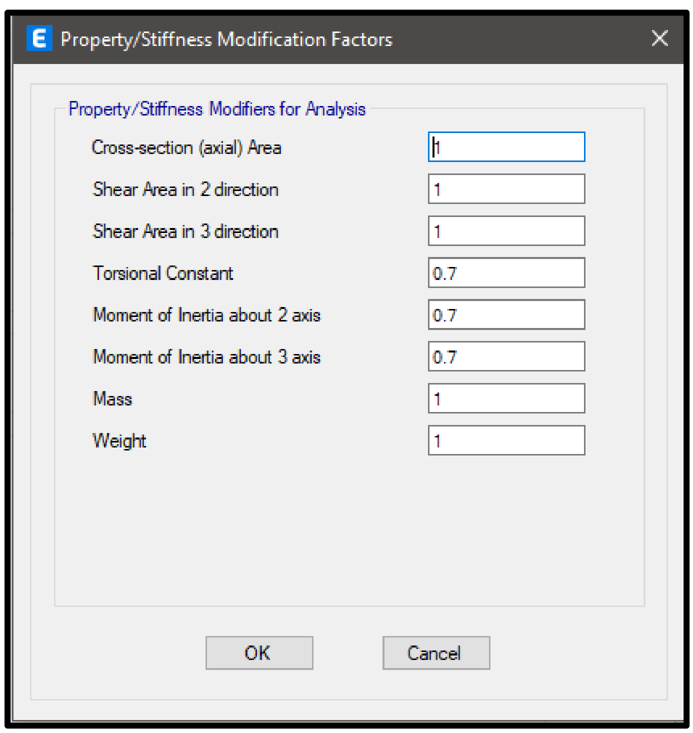


Figure 2.2.1.3 – C1 Stiffness Modifiers

2. C2 sections and modifiers:

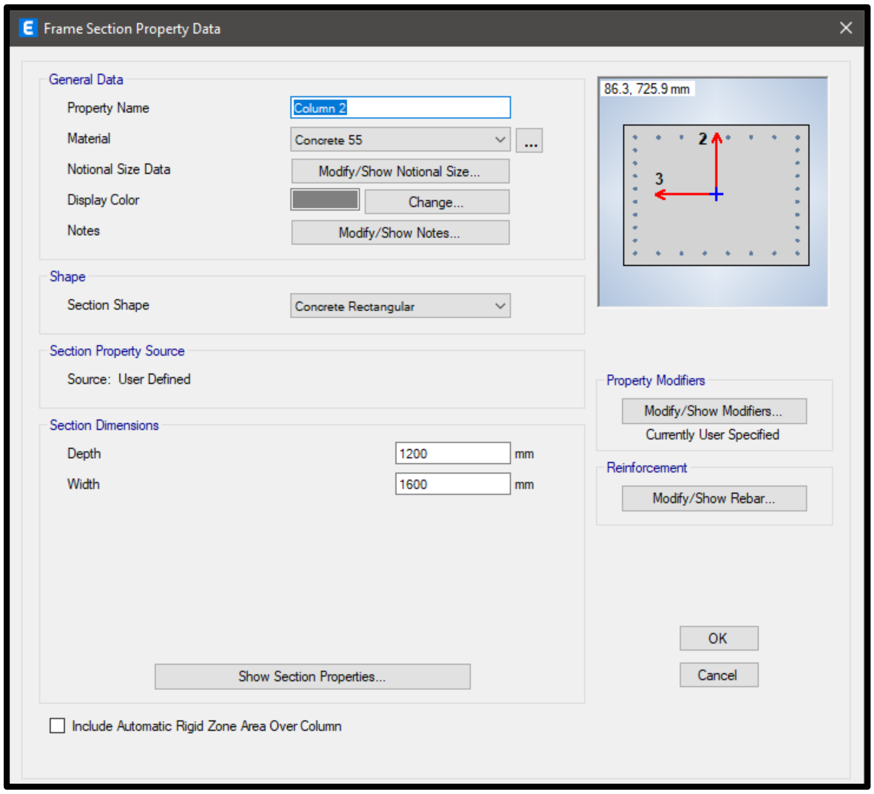


Figure 2.2.1.4 – C2 Section Details

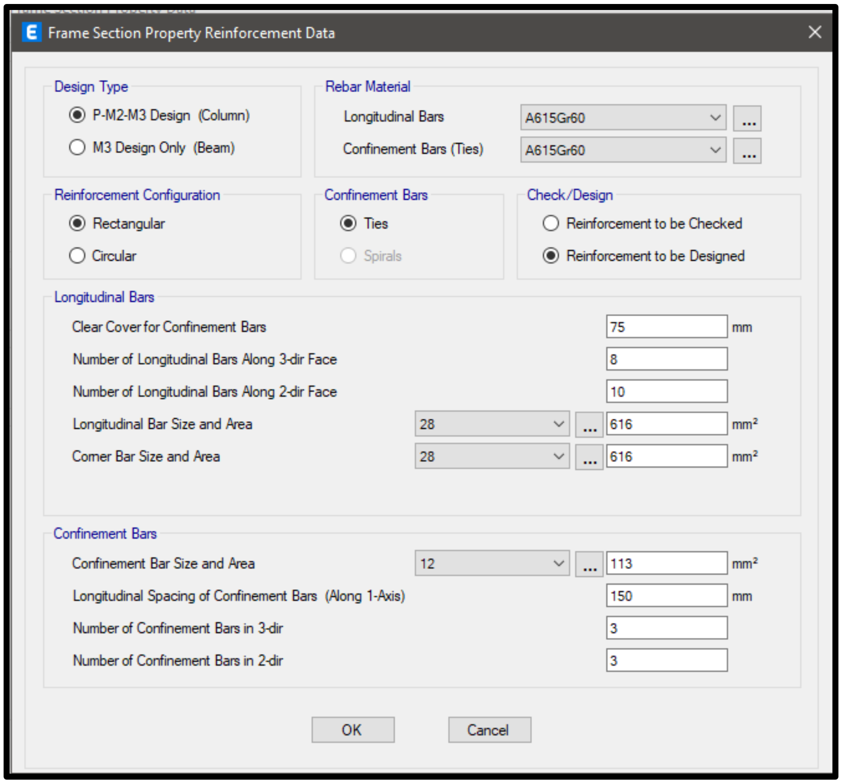


Figure 2.2.1.5 – C2 Reinforcement

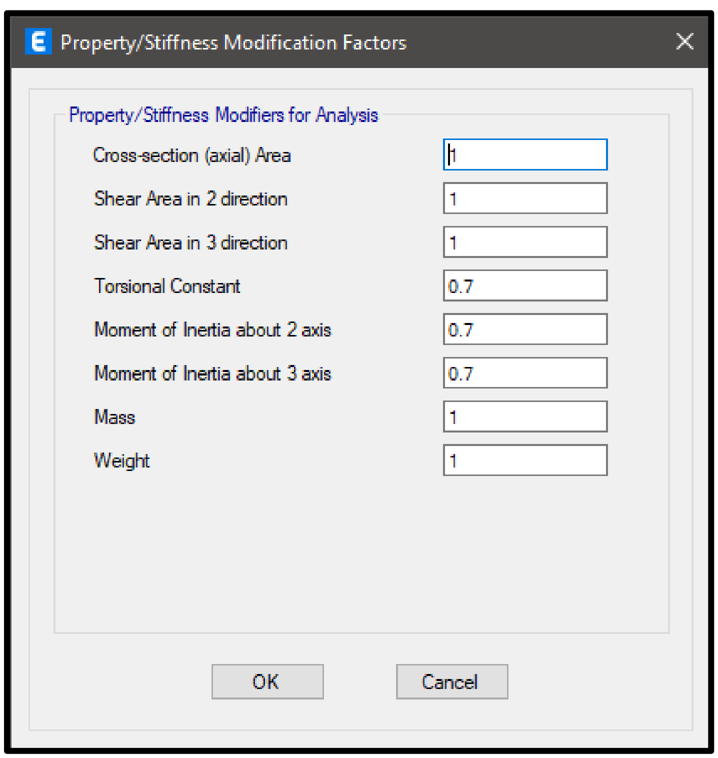


Figure 2.2.1.6 – C2 Stiffness Modifiers

2.2.2 Load cases and combinations

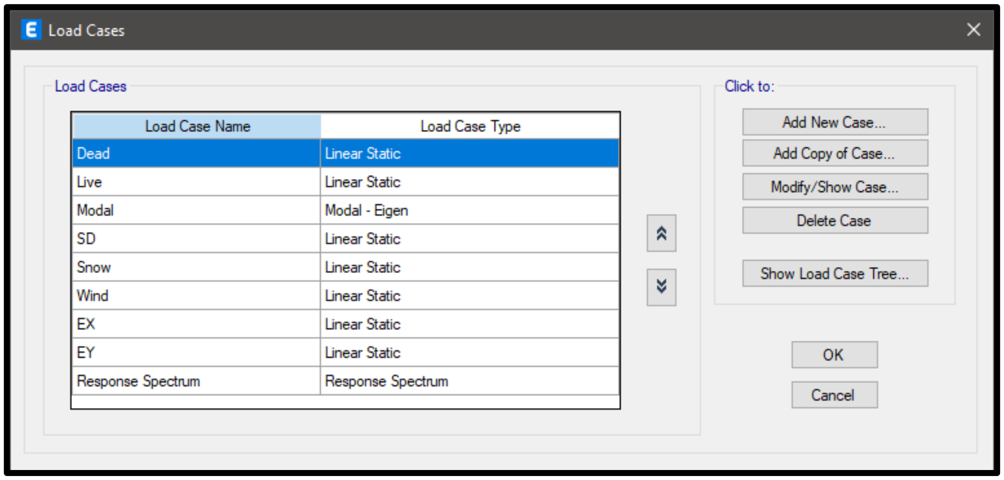


Figure 2.2.2.1 – Load Cases

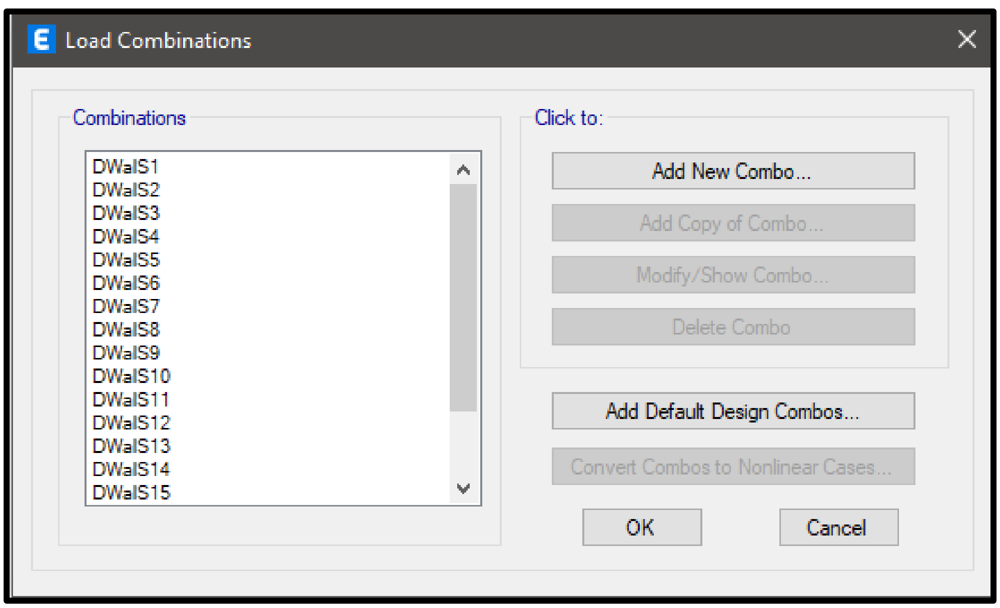


Figure 2.2.2.2- Load Combinations 1

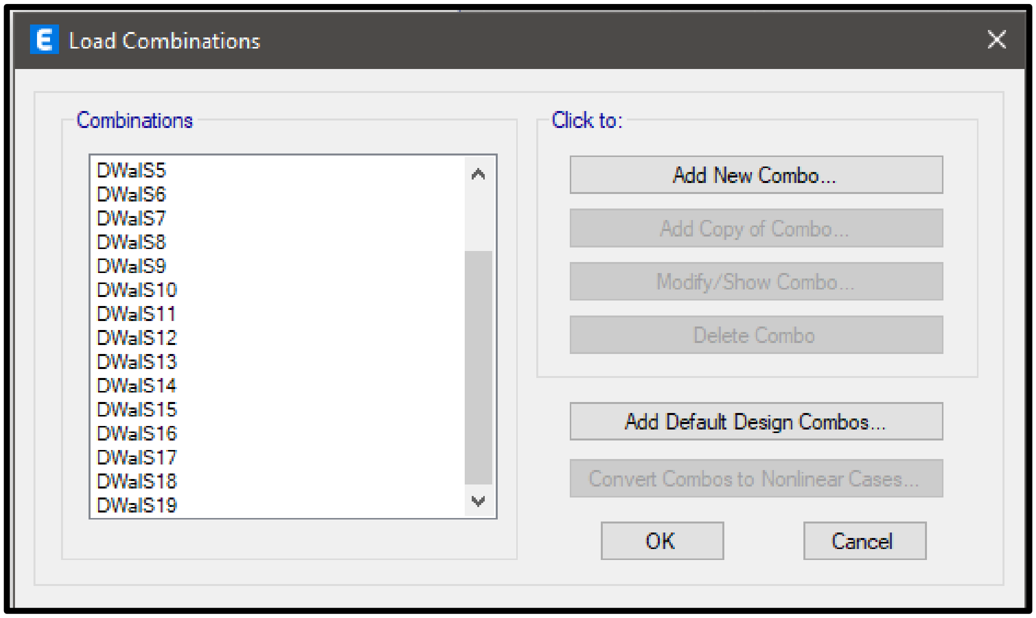


Figure 2.2.2.3 – Load Combinations 2

2.2.3 Load patterns

Finally, load patterns are as follows:

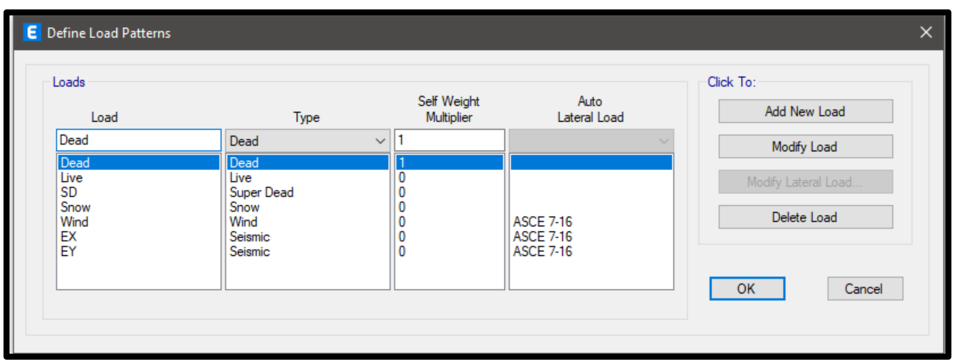


Figure 2.2.3.1 – Load Patterns

2.2.4 Mass source modification

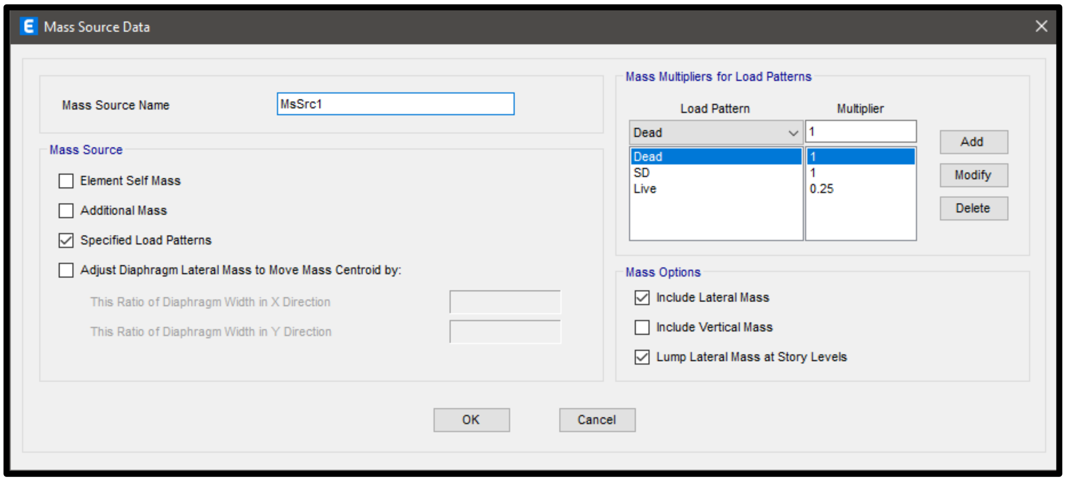


Figure 2.2.4.1 – Mass Source Modification

2.2.5 Design codes and preferences

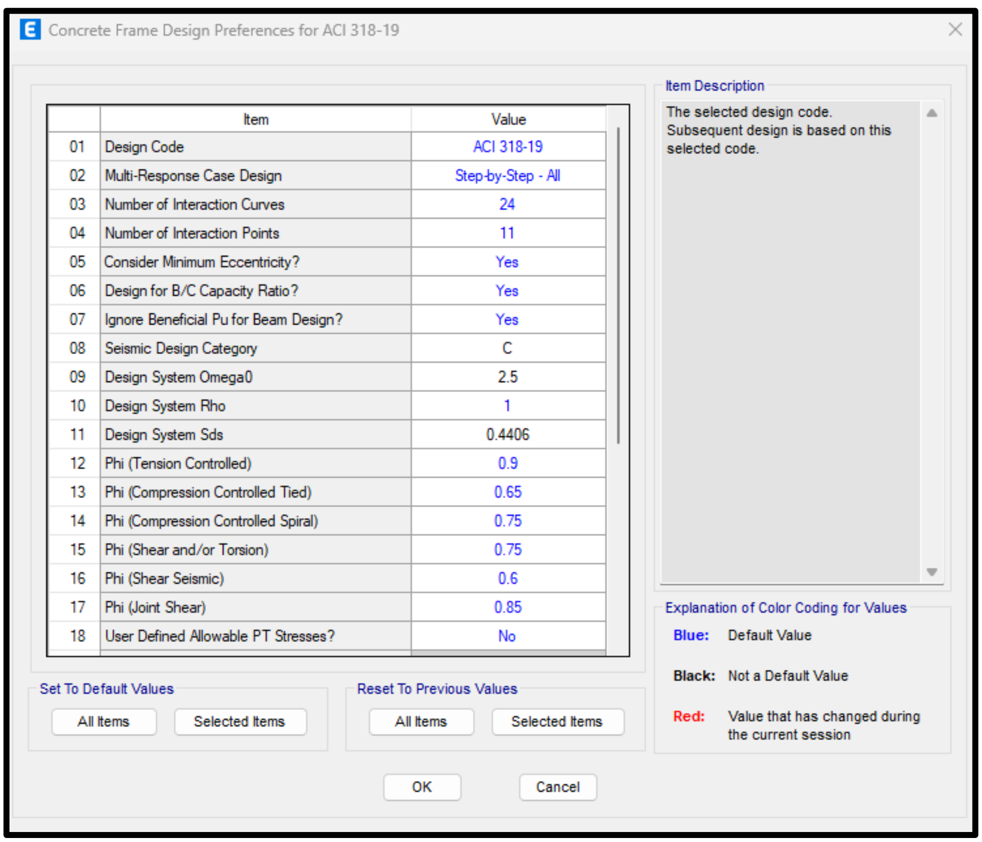


Figure 2.2.5.1 – Concrete Frame Design Preferences 1

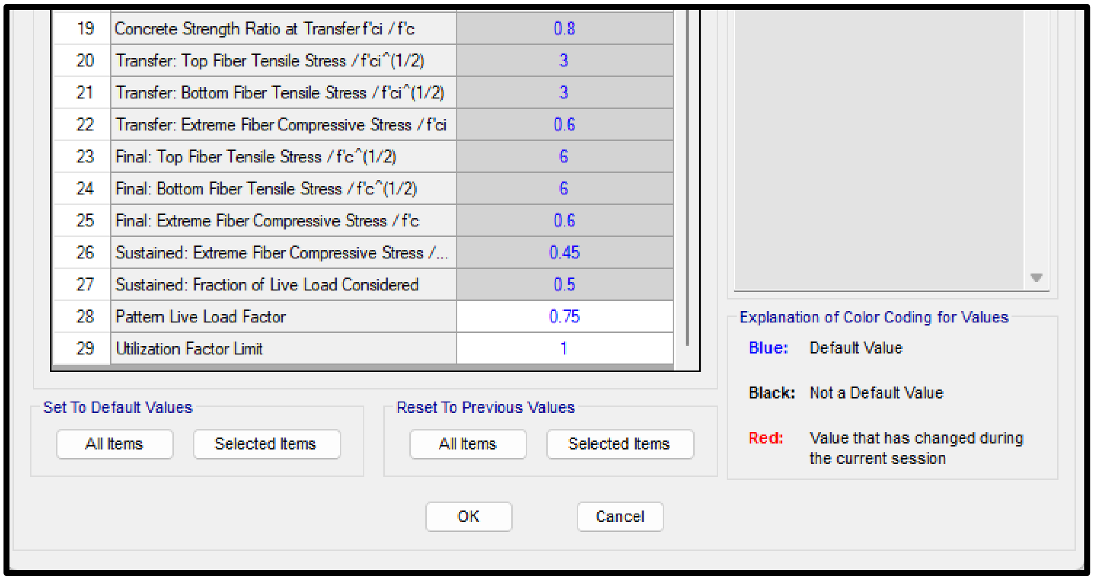


Figure 2.2.5.2 – Concrete Frame Design Preferences 1

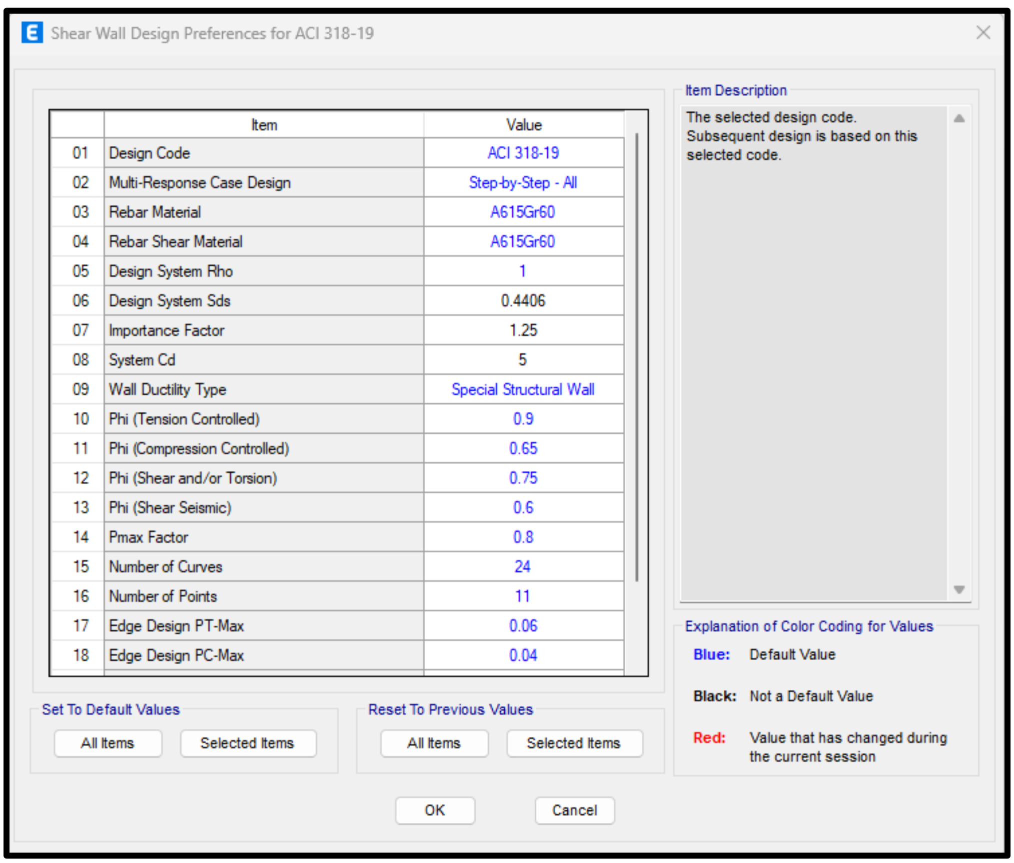


Figure 2.2.5.3 – Wall Design Preferences 1

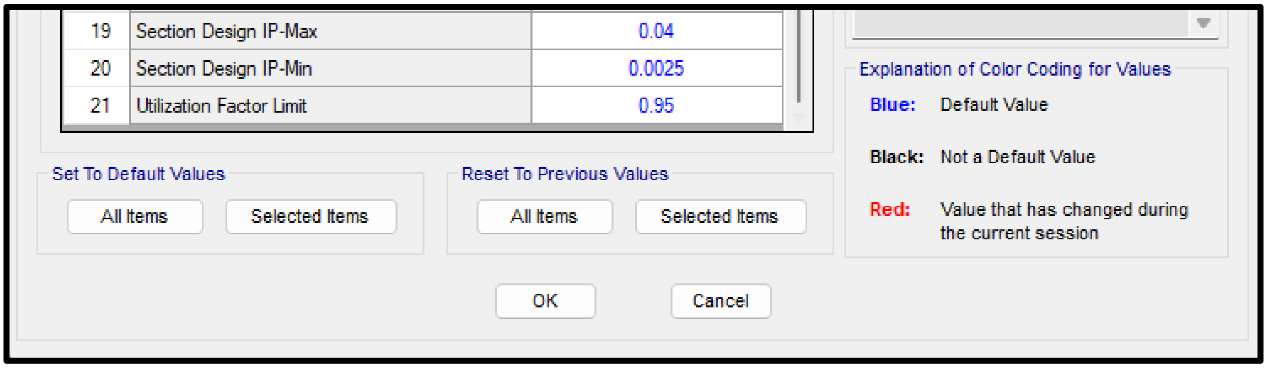


Figure 2.2.5.4 – Wall Design Preferences 2

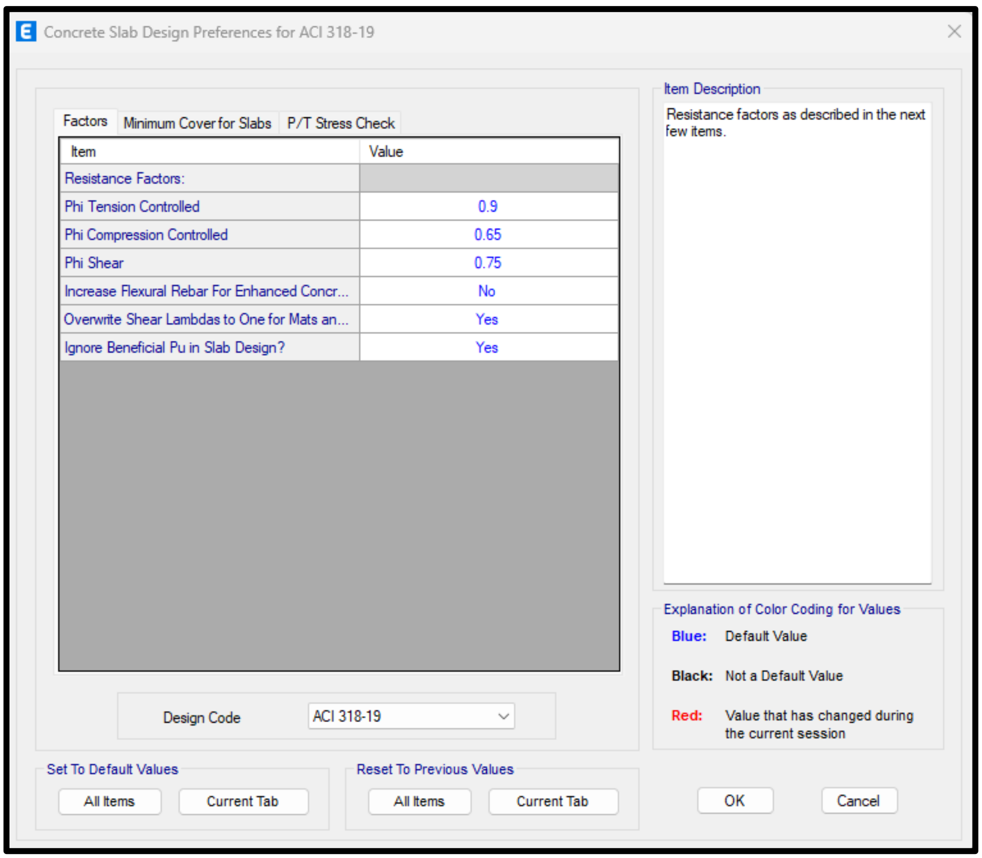


Figure 2.2.5.5 – Slab Design Preferences 1

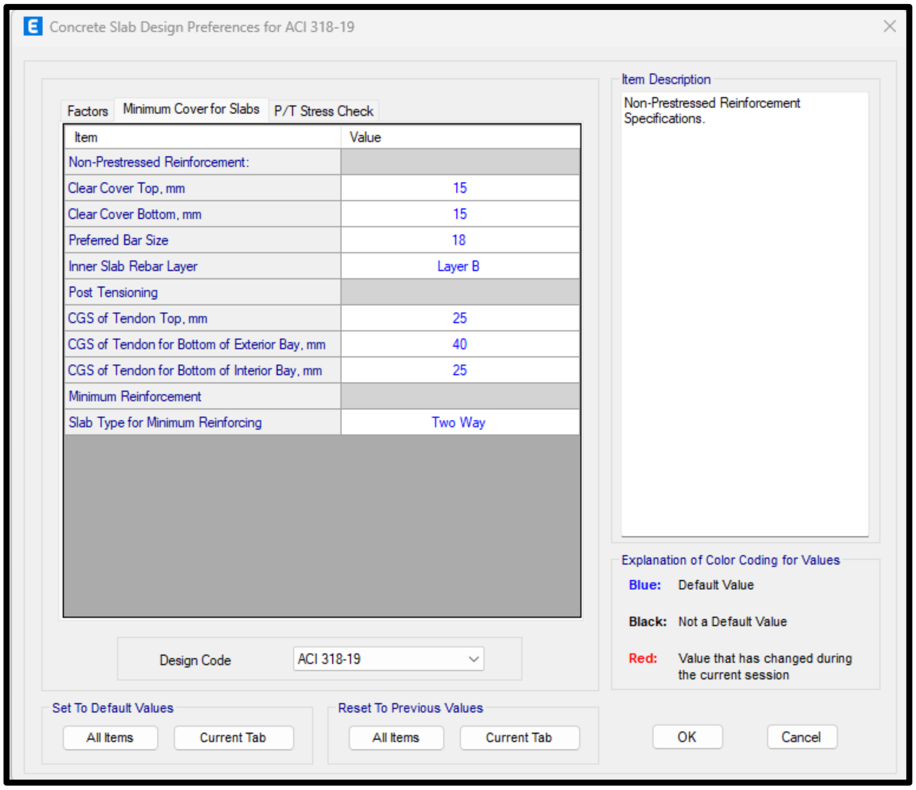


Figure 2.2.5.6 – Slab Design Preferences 2

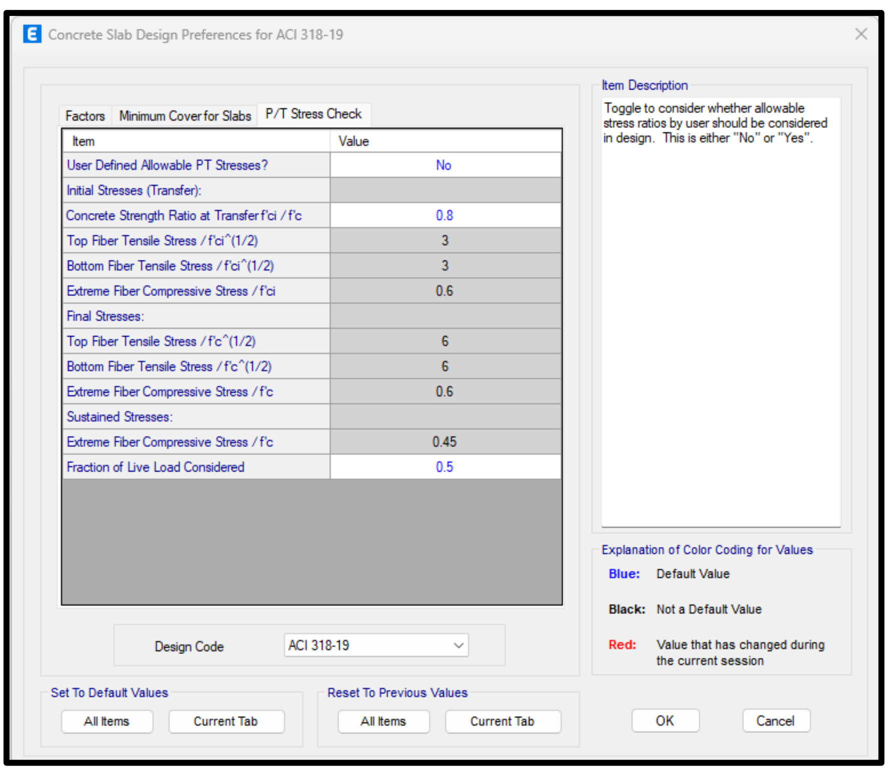


Figure 2.2.5.7 – Slab Design Preferences 3

2.2.6 Verification of Structural Analysis

These checks are very significant to ensure that the structural analysis software analyses properly.

2.2.6.1 Equilibrium Check

Story	# of stories	Net area/m^2	Perimeter/m	Opening area/m^2	Area/m^2
6th Basement Floor to 1st Basement Floor	6	3403.72	******	18.41	3385.31
Ground Floor	1	3403.59	228.86	115.18	3288.41
1st Parking Floor to 3rd parking Floor	3	3403.59	228.86	34.47	3369.12
4th Parking Floor	1	3403.59	228.86	128.1	3275.49
5th Floor-22nd Floor/25th Floor-50th Floor/53rd Floor-79th Floor	71	1349.09	167.88	80.56	1268.53
23rd,24th,51st, and 52nd Floors	4	1349.09	167.88	51.31	1297.78
80th Floor to 94th Floor	15	1291.02	163.39	51.31	1239.71
95th Floor and 97th Floor	2	1290.66	******	51.31	1239.35
96th Floor	1	1117.18	127.99	51.31	1065.87
98th Floor	1	1069.09	******	51.31	1017.78
99th Floor	1	940.02	******	18.56	921.46

Table 2.2.6.1.1 – Floors' Areas

For the Balconies:

5^{th} Floor to 79^{th} Floor:

- 150 Balcony → Area = 16.6 m^2
- 150 Balcony → Area = 11.47 m^2

80^{th} Floor to 94^{th} Floor:

- 60 Balcony → Area = 7.22 m^2
- 30 Balcony → Area = 7.13 m^2
- 30 Balcony → Area = 6.91 m^2

- **Superimposed Load Check**

SD on slab and balcony = 4.5 KN/m^2 and on perimeter walls = 3.5 KN/m^2.

SD on slab = ∑ {# of floors * Area of slab} * SD

= 156319.33* 4.5 = **703436.985 KN**

SD on Balconies = ∑ {# of Balconies * Area of Balcony} * SD

= 5064.9 * 4.5 = **22792.05 KN**

SD on Exterior Beams = ∑ {# of floors * Perimeter} * SD

= 18133.65 * 3.5 = **63467.775 KN**

Total SD by hand= 789696.81 KN

SD by ETABS = 784404.3562 KN

% Error = $\frac{789696.81-784404.3562}{789696.81} * 100\% = \mathbf{0.67\%} < \mathbf{5\%}$**.... ok**

- **Live Load Check**

Live Load on slab = 5 KN/m^2

Live Load on slab = ∑ {# of floors * Area of slab} * Live

= 156319.33* 5 = **781596.65 KN**

Live Load on Balconies =∑ {# of Balconies * Area of Balcony} * Live

= 5064.9 * 5 = **25324.5 KN**

Total LL by hand= 806921.15 KN

Live Load by ETABS = 805505.6745 KN

% Error = $\frac{806921.15-805505.6745}{806921.15} * 100\% = \mathbf{0.18\%}$ **< 5%.... ok**

2.2.6.2 Internal forces check

One of the required analysis checks, is internal forces check. For this project, it will consist of two parts: **Axial on sample column and moment on a sample beam**.

2.2.6.2.1 Axial load check

For this check, axial load by hand will be calculated using tributary area method and then compare it to the axial load value given by ETABS.
Our sample column will be on floor 77 .

$$By\ hand = Service\ Load * Tributary\ Area = (0.4 * 25 + 4.5 + 5) * 24.7976\ m^2$$
$$= \mathbf{483.5532}\ \boldsymbol{KN}$$

From ETABS calculations as follows :

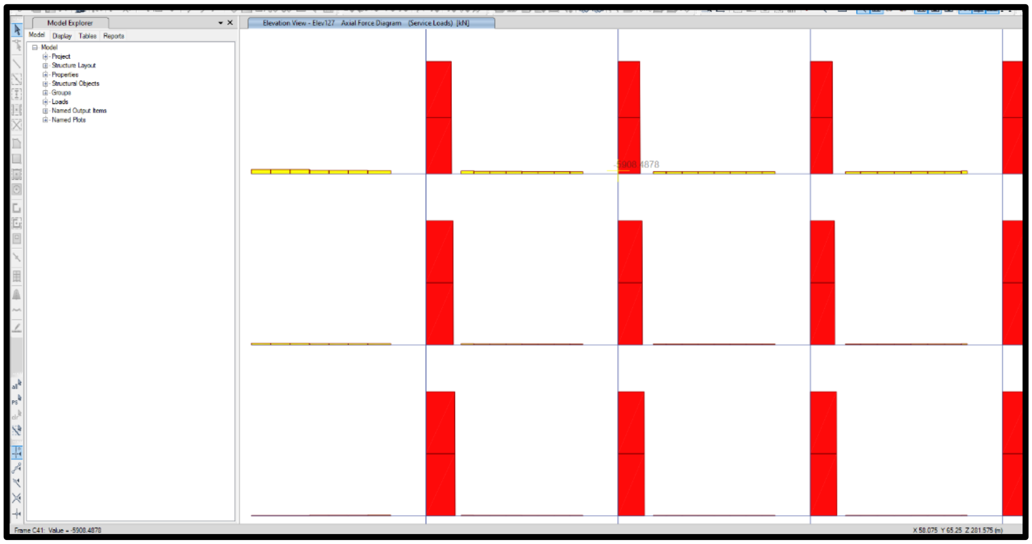


Figure 2.2.6.2.1.1 – Column Axial Load from Above

Axial load first value = 5908.4878 KN

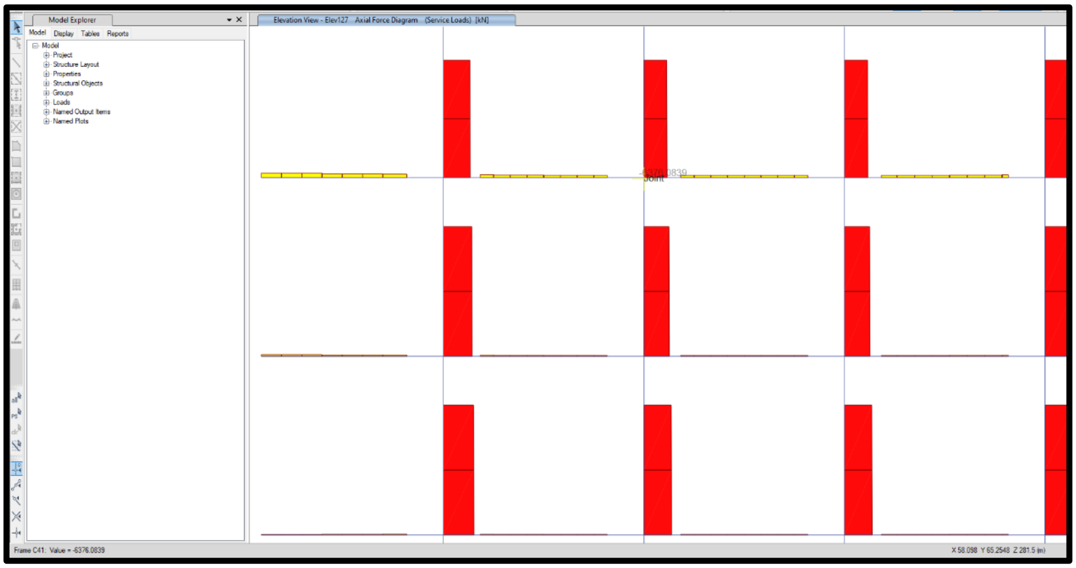


Figure 2.2.6.2.1.2– Column Axial Load from Below

Axial load second value = 6376.089 KN

These two values were taken from the same column on floor 76 and 77. Thus the actual axial load the column withstands is, $\boldsymbol{Axial\ load} = \mathbf{6376.089 - 5908.4878 = 467.6012}\ \boldsymbol{KN}$

$$Error\ percentage = \left|\frac{467.6012 - 483.5532}{467.6012}\right| * 100\% = \mathbf{3.411}\ \% < \mathbf{10}\%\ , \boldsymbol{Acceptable}$$

2.2.6.2.2 Moment on beam check

As usual, hand calculations are done and then compared to software calculations.
For the beam, SD load will be taken into consideration only.

$$Mhand = \frac{wL^2}{24} = \frac{3.5 * 4.25^2}{24} = \mathbf{2.63}\ \boldsymbol{KN.m}$$

Value from software, $Mu = \ \mathbf{3.05}\ \boldsymbol{KN.m}$

$$Error\ percentage = \left|\frac{2.63 - 3.05}{2.63}\right| * 100\ \% = \mathbf{15.8}\ \% < \mathbf{25}\ \%\ , \boldsymbol{Acceptable}$$

2.2.7 First Model Deflection checks

Deflection is a very significant indication of structural analysis and design, if whether it is acceptable for residence use or no, according to specifications mentioned before.

For this project, there are three types of slabs: Parking slab, floor slab, and finally special slab. Each type of these slabs has two deflection checks: long term deflection and immediate deflection.

2.2.7.1 Parking slab deflection check

Our sample panel is from the -4 floor.
First, deflection from **dead + superimposed dead loads** is as shown below in the screenshot.

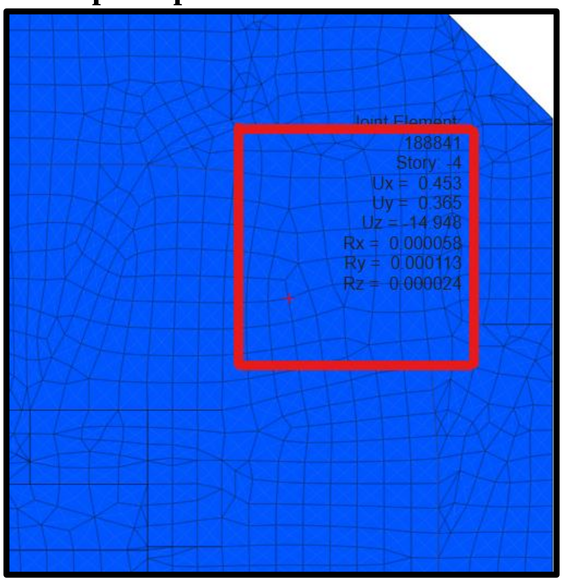


Figure 2.2.7.1.1 – Slab at Parking Level Deflection from Dead and Superimposed Dead Loads

This panel is 9 m * 8.7 m area. Deflection at edges are 5.511 mm, 3.303 mm, 8.878 mm, 5.649 mm, 5.442 mm, and 5.581 mm.
Average deflection at edges = **5.727 mm**

$\Delta_D = \mathbf{14.948(\mathit{deflection\ at\ centre}) - 5.727 = 9.221\ \mathit{mm}}$

Then, deflection must be calculated from **live load (sustained and unsustained).**

Figure 2.2.7.1.2 – Slab at Parking Level Deflection from Total Live Load

Deflection at edges are 1.119 mm, 1.142 mm, 0.604 mm, 1.194 mm, 1.110 mm, and 2.078 mm.
Average deflection at edges = **1.2078 mm**
$\Delta_L = \mathbf{3.868}(\boldsymbol{deflection\ at\ centre}) - \mathbf{1.2078} = \mathbf{2.660}\ \boldsymbol{mm}$

Deflection from **sustained live load** is as follows.

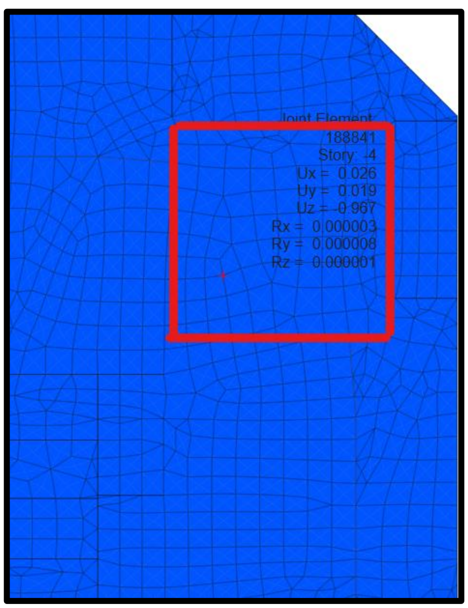


Figure 2.2.7.1.3 – Slab at Parking Level Deflection from Sustained Live Load

Deflection at edges is 0.28 mm, 0.285 mm, 0.151 mm, 0.298 mm, 0.278 mm, 0.52 mm.
Average deflection at edges = **0.302 mm**
$\Delta_{LS} = \mathbf{0.967}(\boldsymbol{deflection\ at\ centre}) - \mathbf{0.302} = \mathbf{0.665}\ \boldsymbol{mm}$

Thus, $\Delta_{Lunsustained} = \Delta_L - \Delta_{LS} = \mathbf{2.660 - 0.967 = 1.693}\ \boldsymbol{mm}$

$$\Delta_{LT}(\boldsymbol{Long\ term\ deflection}) = \ \Delta \boldsymbol{L}_{\boldsymbol{unsustained}} + \boldsymbol{\lambda}_{\infty}\Delta_D + \boldsymbol{\lambda}_t\Delta_{LS}$$
$$= \mathbf{1.693 + 2(9.221) + 2(0.665) = 21.465}\ \boldsymbol{mm}$$

$$\Delta_I(\boldsymbol{Immediate\ deflection}) = \ \Delta_{Lunsustained} + \Delta_D + \Delta_{LS} = \mathbf{1.693 + 9.221 + 0.665}$$
$$= \mathbf{11.579}\ \boldsymbol{mm}$$

Total deflection = 33.044 mm

$$\frac{\boldsymbol{L}}{\mathbf{240}} = \frac{\mathbf{8700}}{\mathbf{240}} = \mathbf{36.25}\ \boldsymbol{mm}\ , \mathbf{33.044}\ \boldsymbol{mm} < \mathbf{36.011}\ \boldsymbol{mm}\ , \boldsymbol{Acceptable}$$

2.2.7.2 Floor slab deflection check

This type of slab is the most used type throughout the structure, except for parking floors and top ones also.

As a result, three panels are going to be taken as samples for deflection checks. Hence, for each load case, deflection values will be calculated for each panel separately.

Deflection from dead + superimposed dead load for first panel is as follows.

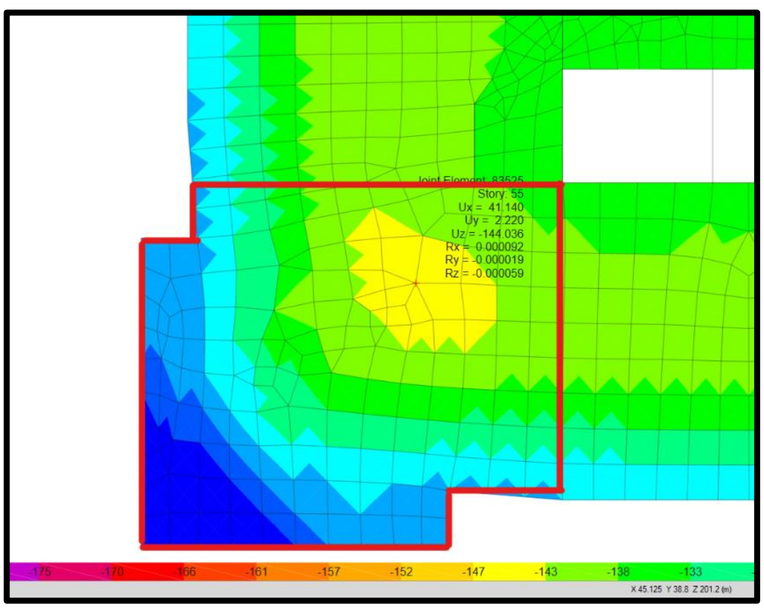


Figure 2.2.7.2.1 – Floor Slab First Panel Deflection from Dead and Superimposed Dead Loads

For this panel, since it is not a symmetrical shape then this means deflection at corners are not at equal distance from the center. Thus, weighted average method will be used to calculate average deflection at corners.

Deflection at far-left corner = 104.732 mm, and distance from center is 12.4 m.
Deflection at far-right corner = 133.909 mm, and distance from center is 5.9 m.

$$Average\ deflection\ at\ edges = \frac{\mathbf{104.732 * 5.9 + 133.909 * 12.4}}{\mathbf{18.3}} = \mathbf{124.502\ mm}$$

$$\Delta_{D1} = \mathbf{144.036(deflection\ at\ center) - 124.502 = 19.534\ mm}$$

Deflection from dead + superimposed dead load for second panel is as explained below.

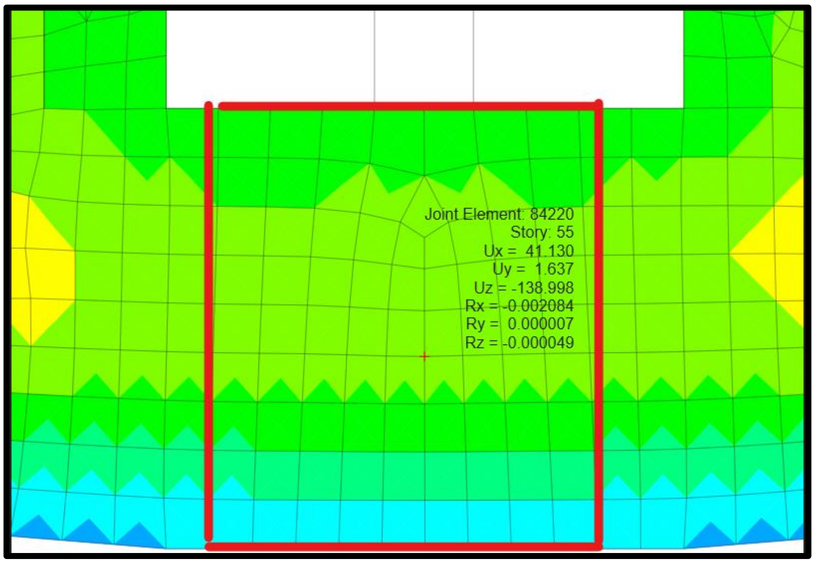


Figure 2.2.7.2.2 – Floor Slab Second Panel Deflection from Dead and Superimposed Dead Loads

Deflection at edges are 133.989 mm and 124.108 mm.

Average deflection at edges = **129.0485 mm.**
$\Delta_{D2} = \mathbf{138.998(deflection\ at\ center) - 129.0485\ mm\ = 9.950\ mm}$

Deflection from dead + superimposed dead load for third panel.

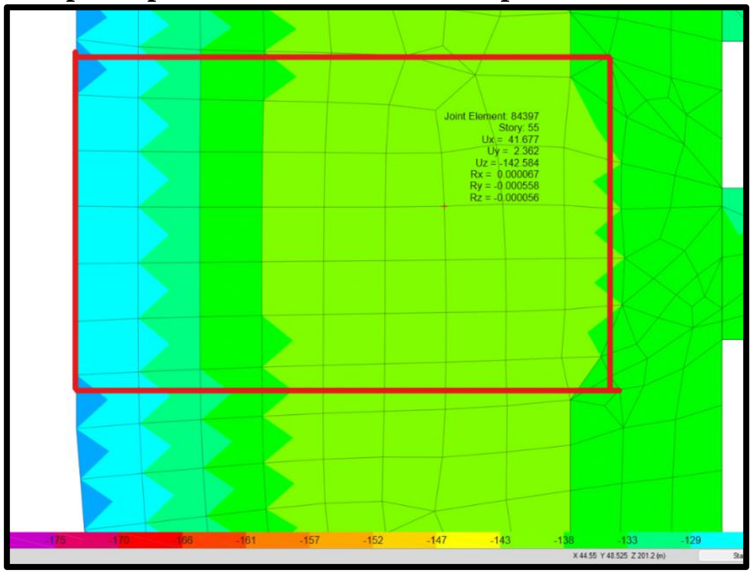


Figure 2.2.7.2.3 – Floor Slab Third Panel Deflection from Dead and Superimposed Dead Loads

Deflection at edges is not at equal distance from center, thus **weighted average method is used.**

Average deflection at edges = $\frac{1}{3}(\mathbf{122.93}) + \frac{2}{3}(\mathbf{136.2845}) = \mathbf{131.833}\ \boldsymbol{mm}$

$\Delta_{D3} = \mathbf{142.584}(\boldsymbol{deflection\ at\ center}) - \mathbf{131.833}\ \boldsymbol{mm} \ = \mathbf{10.571}\ \boldsymbol{mm}$

Deflection from live load (sustained and unsustained) for first panel.

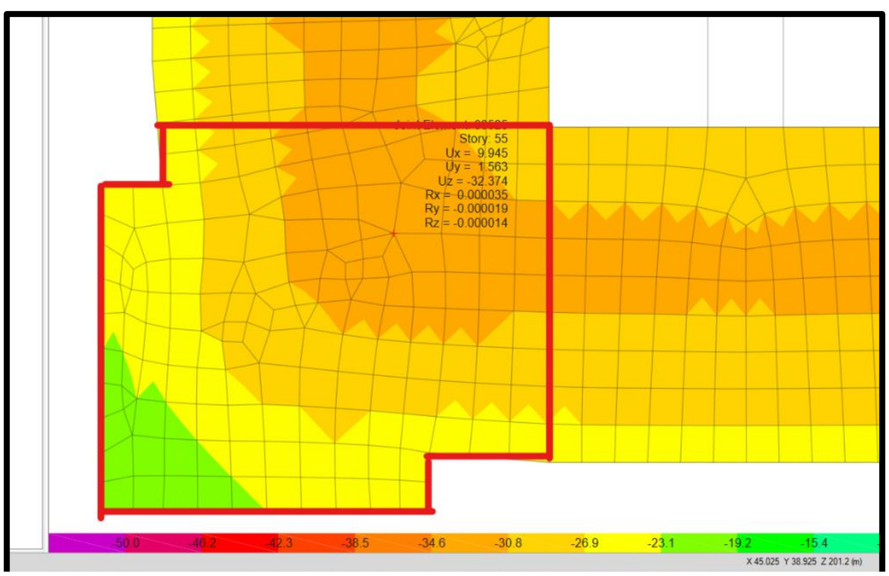


Figure 2.2.7.2.4 – Floor Slab First Panel Deflection from Total Live Load

For this panel, since it is not a symmetrical shape then this means deflection at corners are not at equal distance from the center. Thus, weighted average method will be used to calculate average deflection at corners.

Deflection at far-left corner = 19.607 mm, and distance from center is 12.4 m.
Deflection at far-right corner = 28.814 mm, and distance from center is 5.9 m.

$$Average\ deflection\ at\ edges = \frac{\mathbf{19.607 * 5.9 + 28.814 * 12.4}}{\mathbf{18.3}} = \mathbf{25.846\ mm}$$

$$\Delta_{L1} = \mathbf{32.374(deflection\ at\ center) - 25.846 = 6.528\ mm}$$

Deflection from live load (sustained and unsustained) for second panel.

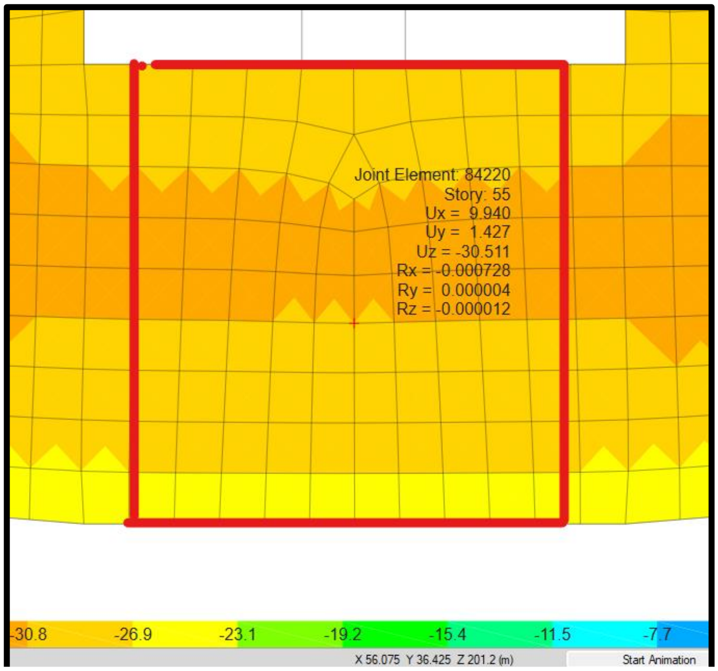


Figure 2.2.7.2.5 – Floor Slab Second Panel Deflection from Total Live Load

Deflection at edges are 28.855 mm and 25.320 mm.
$\mathbf{Average\ deflection\ at\ edges = 27.088\ mm}$

$$\Delta_{L2} = \mathbf{30.511(deflection\ at\ center) - 27.088 = 3.424\ mm}$$

Deflection from live load (sustained+unsustained) for third panel.

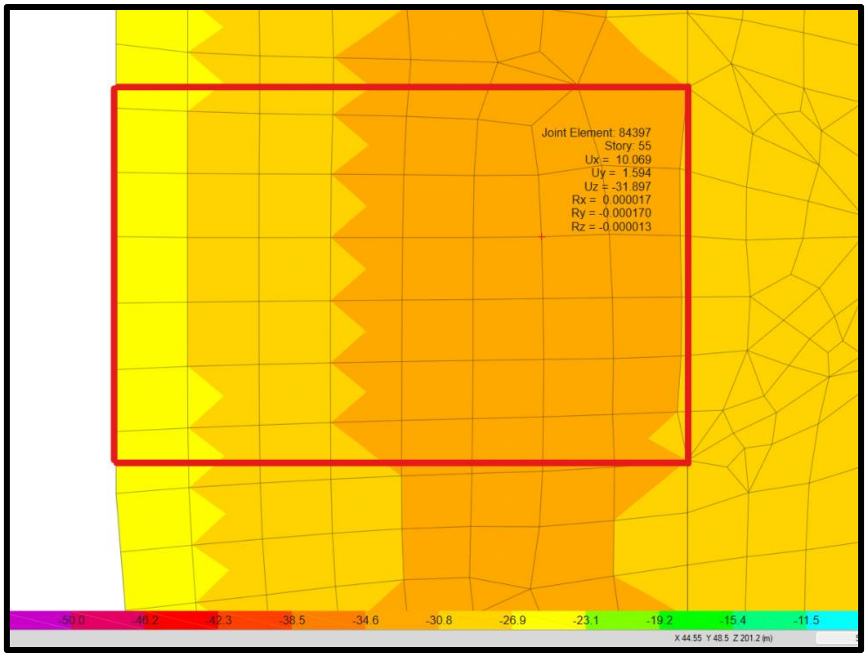


Figure 2.2.7.2.6 – Floor Slab Third Panel Deflection from Total Live Load

Deflection at edges is not at equal distance from center, thus **weighted average method is used.**

Average deflection at edges = $\frac{1}{3}(\mathbf{24.934})+\frac{2}{3}(\mathbf{30.2335}) = \mathbf{28.467}\ \boldsymbol{mm}$

$\Delta_{L3}= \mathbf{31.897}(\boldsymbol{deflection\ at\ center}) - \mathbf{28.467}\ \boldsymbol{mm}\ = \mathbf{3.430}\ \boldsymbol{mm}$

Deflection from sustained live load for first panel.

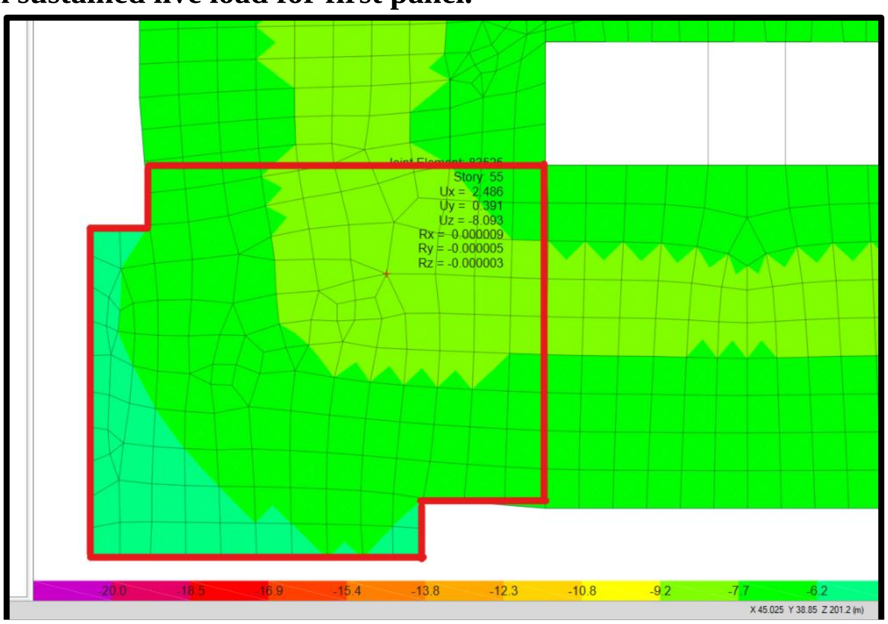


Figure 2.2.7.2.7 – Floor Slab First Panel Deflection from Sustained Live Load

For this panel, since it is not a symmetrical shape then this means deflection at corners are not at equal distance from the center. Thus, weighted average method will be used to calculate average deflection at corners.

Deflection at far-left corner = 4.902 mm, and distance from center is 12.4 m.
Deflection at far-right corner = 7.203 mm, and distance from center is 5.9 m.

$$Average\ defelction\ at\ edges = \frac{\mathbf{4.902 * 5.9 + 7.203 * 12.4}}{\mathbf{18.3}} = \mathbf{6.461\ mm}$$

$$\Delta_{Ls1} = \mathbf{8.093(deflection\ at\ center) - 6.461 = 1.631\ mm}$$

Deflection from sustained live load for second panel.

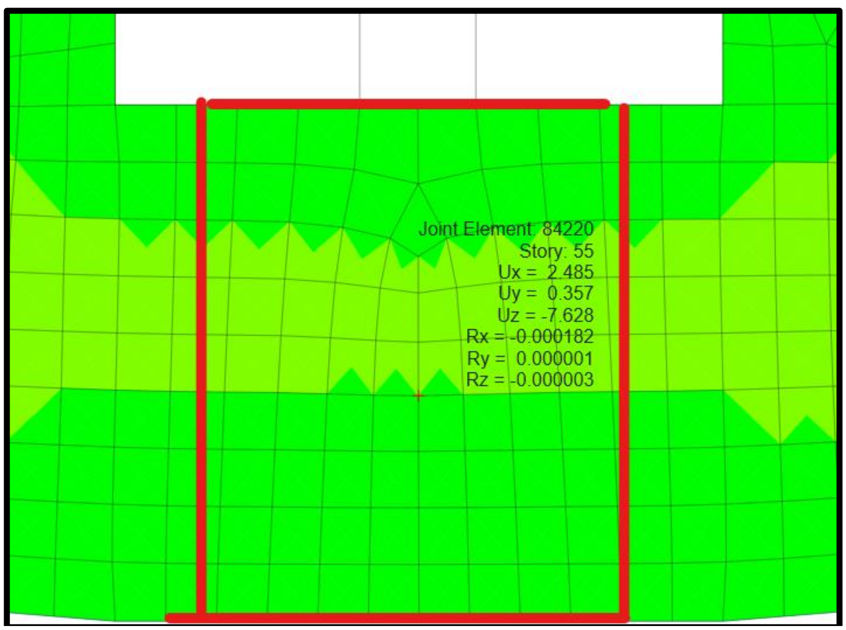


Figure 2.2.7.2.8 – Floor Slab Second Panel Deflection from Sustained Live Load

Deflection at edges are 6.33 mm and 7.214 mm.

$$\boldsymbol{Average\ deflection\ at\ edges = 6.772\ mm}\ .$$

$$\Delta_{Ls2} = \mathbf{7.628(deflection\ at\ center) - 6.772\ mm = 0.856\ mm}\ .$$

Deflection from sustained live load for third panel is as follows.

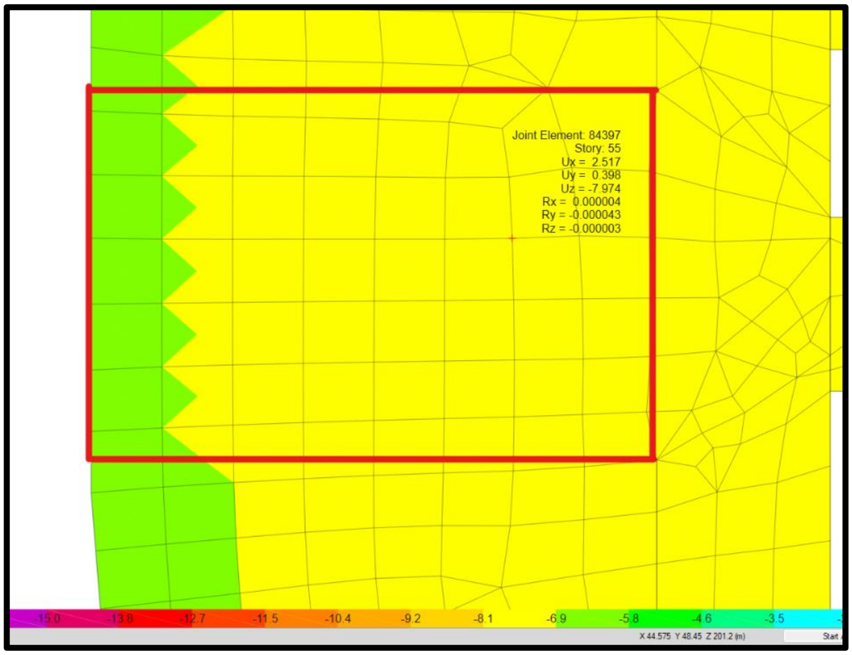


Figure 2.2.7.2.9 – Floor Slab Third Panel Deflection from Sustained Live Load

Deflection at edges is not at equal distance from center, thus **weighted average method is used.**

Average deflection at edges = $\frac{1}{3}(\mathbf{6.233}\ ) + \frac{2}{3}(\mathbf{7.5585}) = \mathbf{7.117}\ \boldsymbol{mm}$

$\Delta_{Ls3} = \mathbf{7.974}(\boldsymbol{deflection\ at\ center}) - \mathbf{7.117}\ \boldsymbol{mm}\ = \mathbf{0.857}\ \boldsymbol{mm}$

Now, for each panel, long term deflection and immediate deflection will be calculated and compared to code deflection specification.

For first panel:

$\Delta \boldsymbol{L_{unsustained}} = \ \Delta \boldsymbol{L_1} - \Delta \boldsymbol{L_{s1}} = \ \mathbf{6.528} - \mathbf{1.631} = \mathbf{4.897}\ \boldsymbol{mm}$

$$\Delta_{LT}(\boldsymbol{Long\ term\ deflection}) = \ \Delta \boldsymbol{L_{unsustained}} + \boldsymbol{\lambda_\infty}\Delta_{\boldsymbol{D}} + \boldsymbol{\lambda_t}\Delta_{\boldsymbol{LS}} \\ = \mathbf{4.897} + \mathbf{2}(\mathbf{19.534}) + \mathbf{2}(\mathbf{1.631}) = \mathbf{47.227}\ \boldsymbol{mm}$$

$$\Delta_{\boldsymbol{I}}(\boldsymbol{Immediate\ deflection}) = \ \Delta_{\boldsymbol{Lunsustained}} + \Delta_{\boldsymbol{D}} + \Delta_{\boldsymbol{LS}} = \mathbf{4.897} + \mathbf{19.534} + \mathbf{1.631} \\ = \mathbf{26.062}\ \boldsymbol{mm}$$

Total deflection = 73.289 mm

$$\frac{\boldsymbol{L}}{\mathbf{240}} = \frac{\mathbf{18300}(\boldsymbol{shortest\ panel\ dimension})}{\mathbf{240}} = \mathbf{76.25}\ \boldsymbol{mm}\ , \mathbf{73.289} \\ < \mathbf{76.25}\ \boldsymbol{mm}, \boldsymbol{Acceptable}$$

For second panel:

$$\Delta L_{unsustained} = \Delta L_2 - \Delta L_{s2} = 3.424 - 0.856 = 2.568\ mm$$

$$\Delta_{LT}(Long\ term\ deflection) = \Delta L_{unsustained} + \lambda_\infty \Delta_D + \lambda_t \Delta_{LS}$$
$$= 2.568 + 2(9.950) + 2(0.856) = 24.18\ mm$$

$$\Delta_I(Immediate\ deflection) = \Delta_{Lunsustained} + \Delta_D + \Delta_{LS} = 2.568 + 9.950 + 0.856$$
$$= 13.374\ mm$$

Total deflection = 37.554 mm

$$\frac{L}{240} = \frac{10450(shortest\ panel\ dimension)}{240} = 43.542\ mm\ , 37.554$$
$$< 43.542\ mm\ , Acceptable$$

For third panel:

$$\Delta L_{unsustained} = \Delta L_3 - \Delta L_{s3} = 3.430 - 0.857 = 2.573\ mm$$

$$\Delta_{LT}(Long\ term\ deflection) = \Delta L_{unsustained} + \lambda_\infty \Delta_D + \lambda_t \Delta_{LS}$$
$$= 2.573 + 2(10.571) + 2(0.857) = 25.429\ mm$$

$$\Delta_I(Immediate\ deflection) = \Delta_{Lunsustained} + \Delta_D + \Delta_{LS} = 2.573 + 10.571 + 0.857$$
$$= 14.00\ mm$$

Total deflection = 39.43 mm

$$\frac{L}{240} = \frac{10550(shortest\ panel\ dimension)}{240} = 43.96\ mm\ , 39.43$$
$$< 43.96\ mm\ , Acceptable\ .$$

2.2.7.3 Circular slab deflection check

The sample panel for this slab deflection check will be from floor 98.

Deflection from dead + superimposed dead loads is as follows.

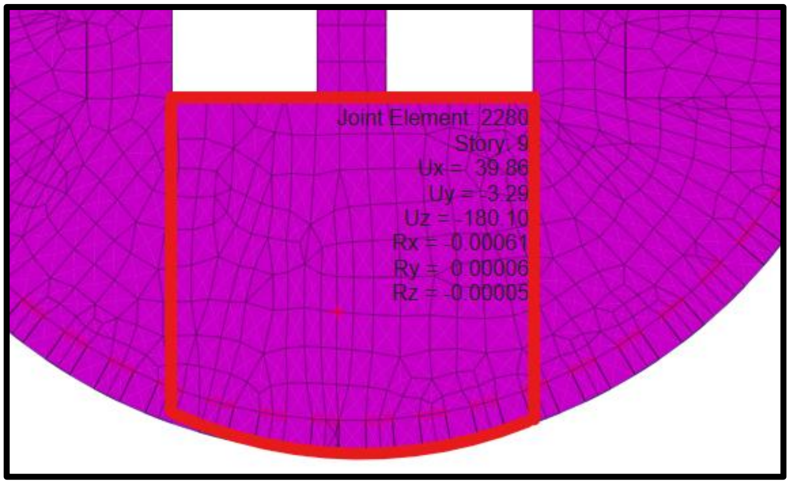


Figure 2.2.7.3.1 – Circular Slab Deflection from Dead and Superimposed Dead Loads

Dimensions of this panel are **12.3 m * 12.1 m**.

Deflection at edges are 166.981 mm, 167.564 mm, 166.723 mm, 166.477 mm, 166.673 mm, 166.761 mm, and 166.651 mm.

$Average\ deflection\ at\ edges = \mathbf{166.824\ mm}$
$\Delta_D = \mathbf{180.10(deflection\ at\ centre) - 166.824 = 13.276\ mm}$

Deflection from live load (sustained and unsustained) is as follows.

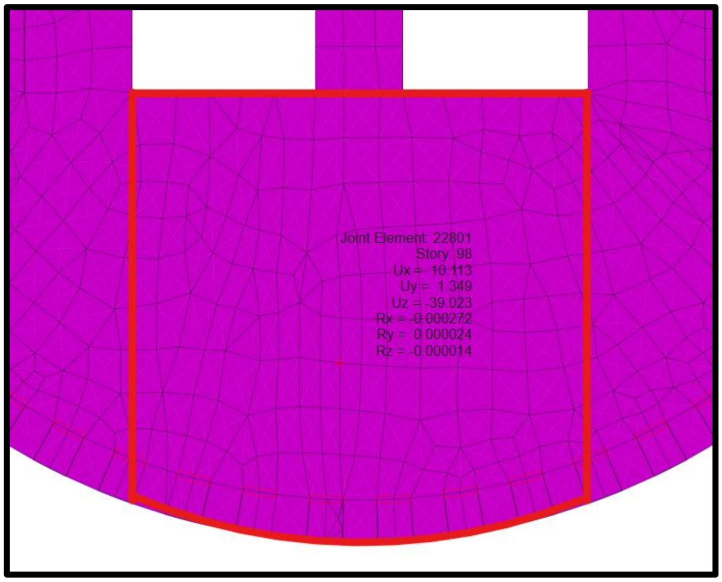


Figure 2.2.7.3.2 – Circular Slab Deflection from Total Live Load

Deflection at edges are 34.569 mm, 34.709 mm, 34.534 mm, 35.592 mm, 35.515 mm, 35.626 mm, and 35.604 mm.

Average deflection at edges = 35.164 mm

$$\Delta_L = \mathbf{39.023}(\boldsymbol{deflection\ at\ centre}) - \mathbf{35.164} = \mathbf{3.859}\ \boldsymbol{mm}$$

Deflection from sustained live load is as shown below.

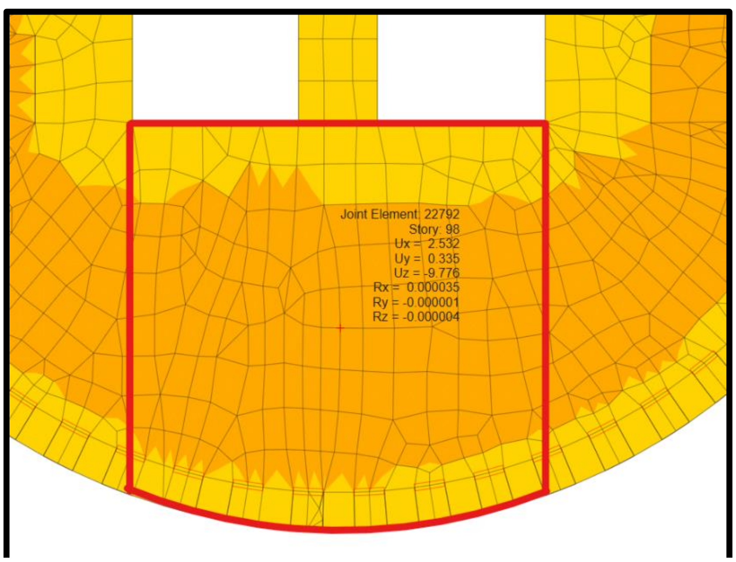


Figure 2.2.7.3.3 – Circular Slab Deflection from Sustained Live

Deflection at edges are 8.642 mm, 8.677 mm, 8.634 mm, 8.898 mm, 8.904 mm, 8.906 mm, and 8.901 mm.

Average deflection at edges = 8.7945 mm

Δ_{LS}= $\mathbf{9.776}(\boldsymbol{deflection\ at\ centre}) - \mathbf{8.7945}\ \boldsymbol{mm} = \mathbf{0.9814}\ \boldsymbol{mm}$

Thus, $\Delta_{Lunsustained}$= $\Delta_L - \Delta_{LS}$= $\mathbf{3.859}\ \boldsymbol{mm} - \mathbf{0.9814}\ \boldsymbol{mm} = \mathbf{2.8776}\ \boldsymbol{mm}$

$$\begin{aligned}\Delta_{LT}(\boldsymbol{Long\ term\ deflection}) &= \ \Delta \boldsymbol{L_{unsustained}} + \boldsymbol{\lambda_\infty} \Delta_{\boldsymbol{D}} + \boldsymbol{\lambda_t}\Delta_{\boldsymbol{LS}} \\ &= \mathbf{2.8776 + 2(13.276) + 2(0.9814) = 31.3924}\ \boldsymbol{mm}\end{aligned}$$

$$\begin{aligned}\Delta_{\boldsymbol{I}}(\boldsymbol{Immediate\ deflection}) &= \ \Delta_{\boldsymbol{Lunsustained}} + \Delta_{\boldsymbol{D}} + \Delta_{\boldsymbol{LS}} = \mathbf{2.8776 + 13.276 + 0.9814} \\ &= \mathbf{17.135}\ \boldsymbol{mm}\end{aligned}$$

Total deflection = 48.5274 mm

$$\frac{\boldsymbol{L}}{\mathbf{240}} = \frac{\mathbf{12100}}{\mathbf{240}} = \mathbf{50.4167}\ \boldsymbol{mm}\ , \mathbf{48.5274}\ \boldsymbol{mm} < \mathbf{50.4167}\ \boldsymbol{mm}\ , \boldsymbol{Acceptable}$$

2.2.8 Wide Beam Shear Check

- **Parking Slab from Story (-6) to Story (+4)**

Thickness = 500 mm

$\emptyset Vc = 0.75(\frac{2}{3})(\lambda)(\lambda_s)(b)(d)(\sqrt{f'c})(\rho^{\frac{1}{3}})$

D = thickness – Cover, take cover = 80 mm→ d = 420 mm

B = 1000 mm

$\boldsymbol{\lambda}$ = 1

$\boldsymbol{\lambda}$ s = $\sqrt{\frac{2}{1+0.004d}} = \sqrt{\frac{2}{1+0.004(420)}} = 0.86$

f’c = 55 MPa

ρ = 0.0018 $\left(\frac{h}{d}\right) = 0.0018\left(\frac{500}{420}\right) = 0.0021$

$$\emptyset Vc = \frac{0.75\left(\frac{2}{3}\right)(1)(0.86)(1000)(420)\left(\sqrt{55}\right)(0.0021^{\frac{1}{3}})}{1000} = \mathbf{169.80}\ \boldsymbol{KN}$$

Vu From ETABS:
Draw a section cut in the X-direction and insert the coordinates of this section cut

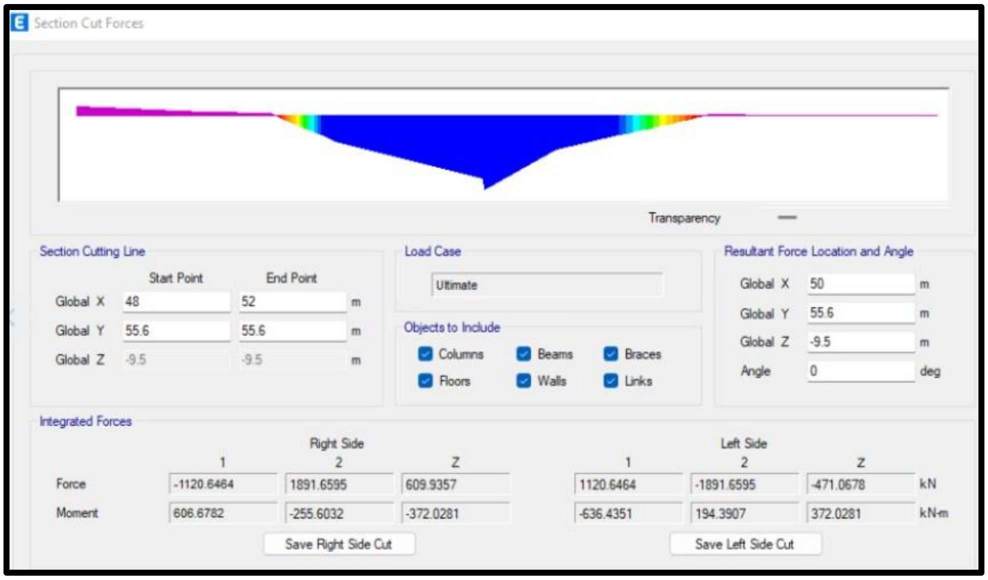


Figure 2.2.8.1 – Parking Slab Section Cut

Vu = 609.9357 / 4 = 152.48 KN, 4 is a section cut length
Then, Vu < ΦVc, slab thickness is adequate

- **Floor Slab from Story (+5) to Story (+94)**

  Thickness = 400mm

  - D = thickness – Cover, take cover = 80 mm→ d = 320 mm
  - B = 1000 mm
  - **λ** = 1
  - **λ**s = $\sqrt{\frac{2}{1+0.004d}} = \sqrt{\frac{2}{1+0.004(320)}} = 0.94$
  - f'c = 55 MPa
  - ρ = 0.0018(h/d) = 0.0018(400/320) = 0.0023

  ΦVc by hand $= \frac{0.75\left(\frac{2}{3}\right)(1)(0.94)(1000)(320)\left(\sqrt{55}\right)\left(0.0023^{\frac{1}{3}}\right)}{1000}$ = **145.76 KN**

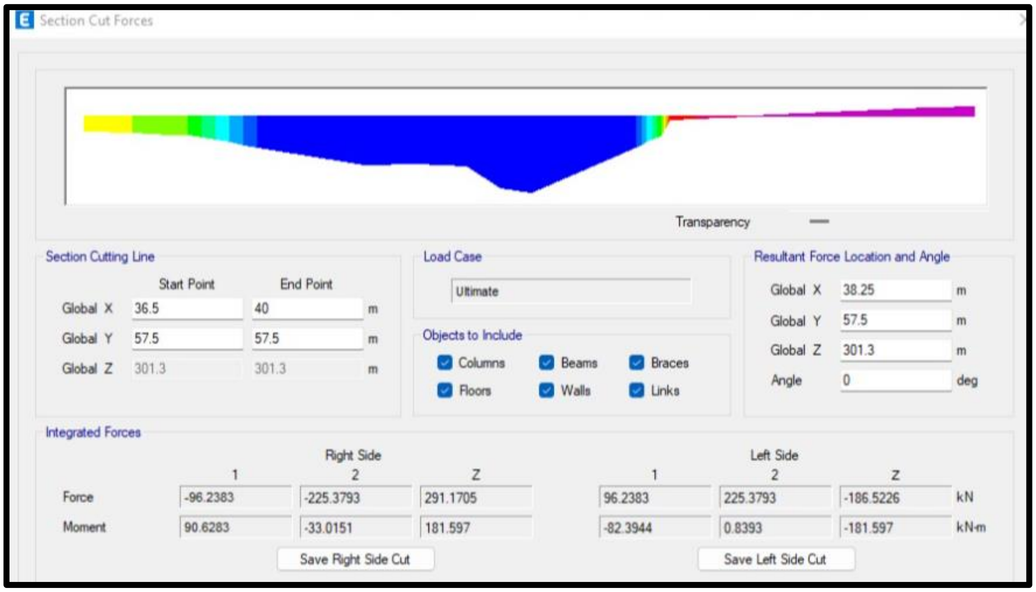


Figure 2.2.8.2 – Floor Slab Section Cut

Vu from ETABS = 291.1705 / 3.5 = 83.192 KN, 3.5 is a section cut length

While Vu < ΦVc → Slab thickness is adequate

- **Circular Slab (with planted shear walls) from Story (+95) to Story (+100)**
  - Thickness = 550 mm
  - D = thickness – Cover, take cover = 80 mm→ d =470 mm
  - B = 1000 mm
  - $\boldsymbol{\lambda}$ = 1
  - $\boldsymbol{\lambda}s = \sqrt{\frac{2}{1+0.004d}} = \sqrt{\frac{2}{1+0.004(470)}} = 0.83$
  - F’c = 55 MPa
  - ρ = 0.0018(h/d) = 0.0018(550/470) = 0.0021

$$\Phi Vc \text{ by hand} = \frac{0.75\left(\frac{2}{3}\right)(1)(0.83)(1000)(470)(\sqrt{55})(0.0021^{\frac{1}{3}})}{1000} = \mathbf{185.24\ KN}$$

From the analysis, it appeared that story 95 has the slab with the maximum Vu compared to the other slabs with planted shear walls.

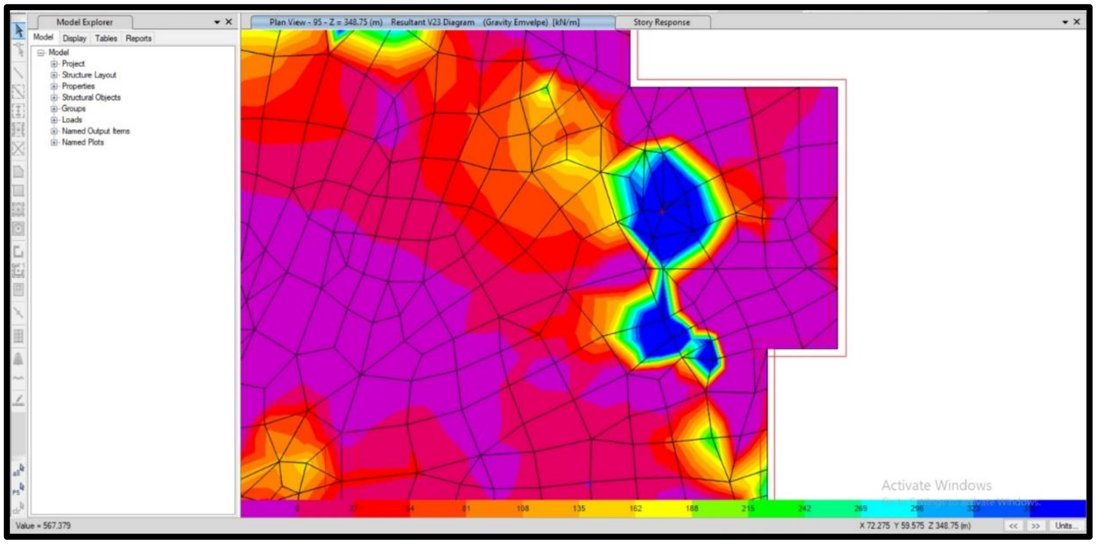


Figure 2.2.8.3 – Circular Slab Shear Force Display

Vu = 567.379 kN, Vu >ΦVc → <u>Slab thickness is not adequate so will be modified in the second model.</u>

2.2.9 Punching Shear Check

- **Basement shear wall – floor 3**

$$Vu,p = \frac{Vu}{b_o * d} + \frac{\gamma_{v1} * M_{u1} * c_1{}'}{J_{c1}} + \frac{\gamma_{v2} * M_{u2} * c_2{}'}{J_{c2}}$$

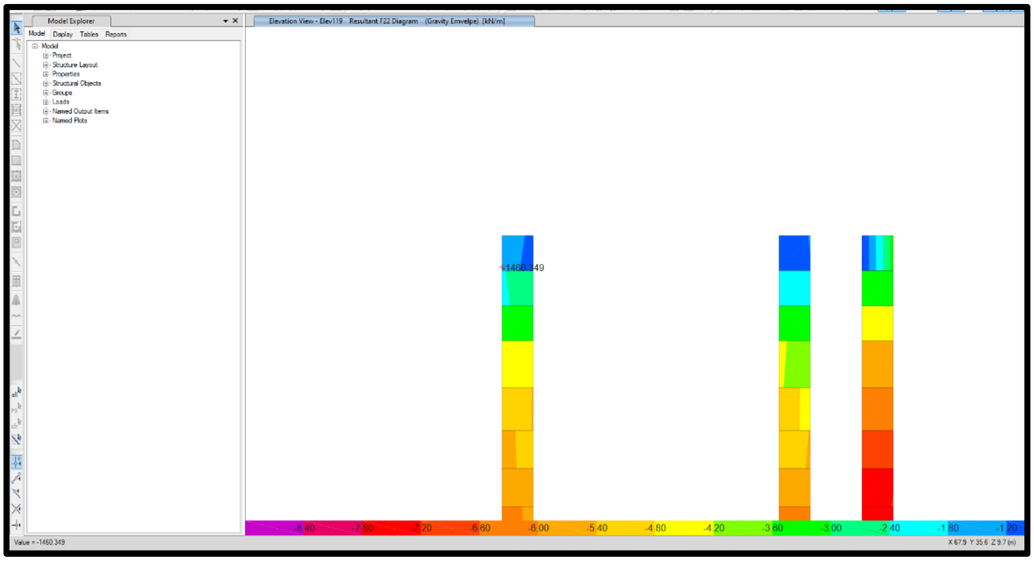


Figure 2.2.9.1 – Basement Shear Wall V1

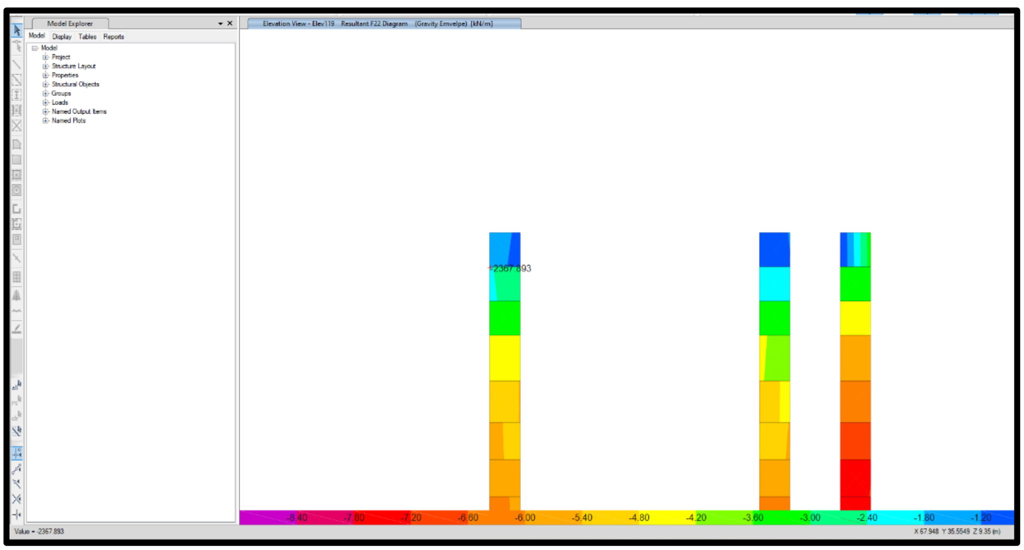


Figure 2.2.9.2 – Basement Shear Wall V2

Vu = 2367.893 – 1460.349 = 907.544 KN

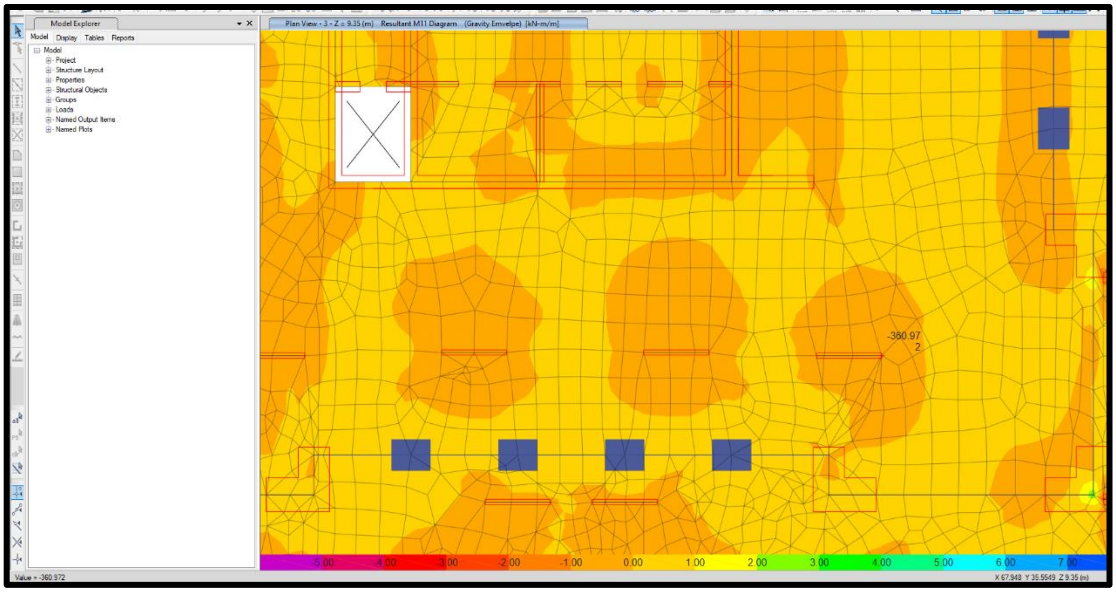


Figure 2.2.9.3 – Basement Shear Wall Mu1

Mu1 = 360.792 KN.m

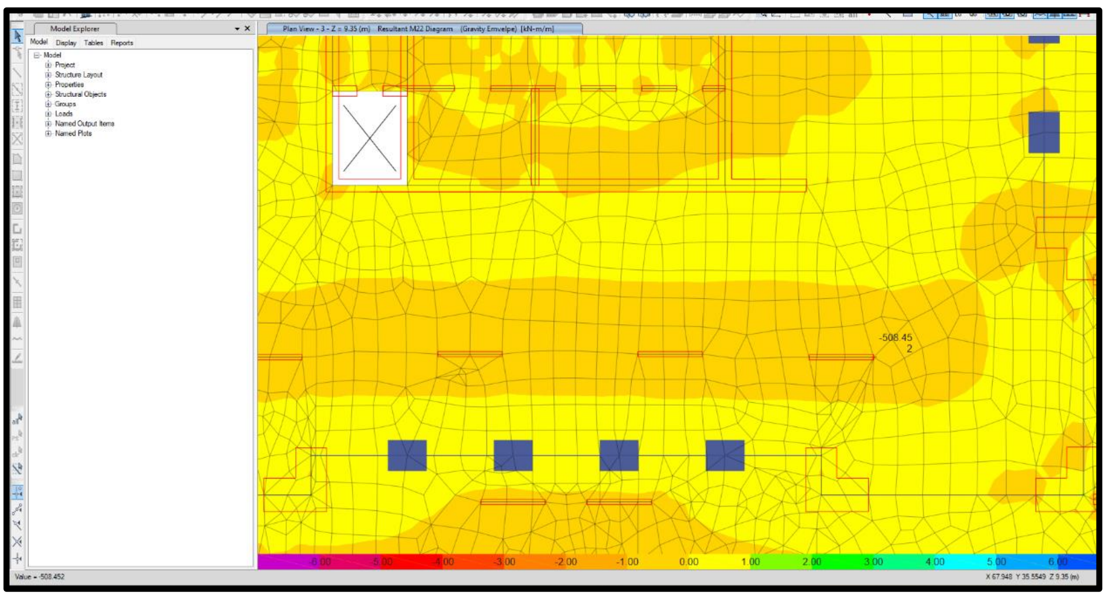


Figure 2.2.9.4 – Basement Shear Wall Mu2

Mu2 = 508.452 KN.m

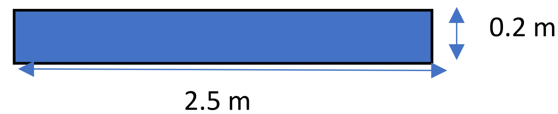


Figure 2.2.9.5 – Basement Shear Wall Section

H= 500 mm then d=420 mm.

$b_1 = 2.5 + d = 2.5 + 0.42 = 2.92$ m

$b_2 = 0.2+0.42 = 0.62$ m

$\beta = \frac{2.92}{0.62} = 4.71$, $\alpha_s = 40$, $b_o = 7.08$ m

$c_1 = \bar{x} = 1.46\, m$, $c_2 = \bar{y} = 0.31\, m$,

$$J_{c1} = 2\left(\frac{b_1 * d^3}{12} + \frac{b_1^3 * d}{12}\right) + 2(b_2 * d * (\bar{x})^2)$$

$$= 2\left(\frac{2.92 * 0.42^3}{12} + \frac{2.92^3 * 0.42}{12}\right) + 2(0.62 * 0.42 * 1.46^2)$$

$$= 2.8889\, m^4$$

$$J_{c2} = 2\left(\frac{b_2 * d^3}{12} + \frac{b_2^3 * d}{12}\right) + 2(b_1 * d * (\bar{y})^2)$$

$$= 2\left(\frac{0.62 * 0.42^3}{12} + \frac{0.62^3 * 0.42}{12}\right) + 2(2.92 * 0.42 * 0.31^2)$$

$$= 0.260\, m^4$$

$$\gamma_{f1} = \frac{1}{1 + 2/3(\sqrt{\frac{b_1}{b_2}})} = 0.4087, \qquad \gamma_{v1} = 1 - 0.4087 = 0.5913$$

$$\gamma_{f2} = \frac{1}{1 + 2/3(\sqrt{\frac{b_2}{b_1}})} = 0.765, \qquad \gamma_{v2} = 1 - 0.765 = 0.235$$

$$\boldsymbol{Vu,p = \frac{907.544}{7.080 * 0.420} + \frac{0.5913 * 360.792 * 1.460}{2.8889} + \frac{0.235 * 508.452 * 0.31}{0.260}}$$

$$\boldsymbol{= 0.55548\, MPa}$$

$\boldsymbol{\lambda s = 0.8638}$

$$Vc,p = 0.33 * \lambda s * \lambda * \sqrt{f'c} = 2.114\, MPa$$

$$Vc,p = 0.17 * \lambda s * \lambda * \sqrt{f'c} * (1 + \frac{2}{\beta}) = 1.55\, MPa$$

$$Vc,p = 0.083 * \lambda s * \lambda * \sqrt{f'c} * (2 + \frac{\alpha_s * d}{b_0}) = 2.3251\, MPa$$

Take minimum of three, Vc, p = 1.55 Mpa

Vu, p < Vc, p, Acceptable

- **Shear wall next to core – 85th floor**

H = 400 mm then d = 320 mm.

$b_1 = 0.7+0.32 = 1.02$ m
$b_2 = 2.5988+0.32 = 2.9188$ m

$b_o = 7.8776$ m

$$\beta = \frac{2.9188}{1.02} = 2.862\ ,\ \alpha_s = 40$$

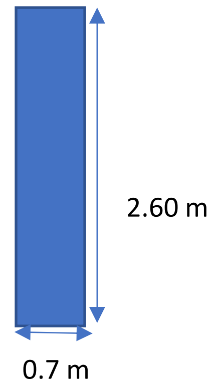


Figure 2.2.9.6 Shear Wall Next to Core Section

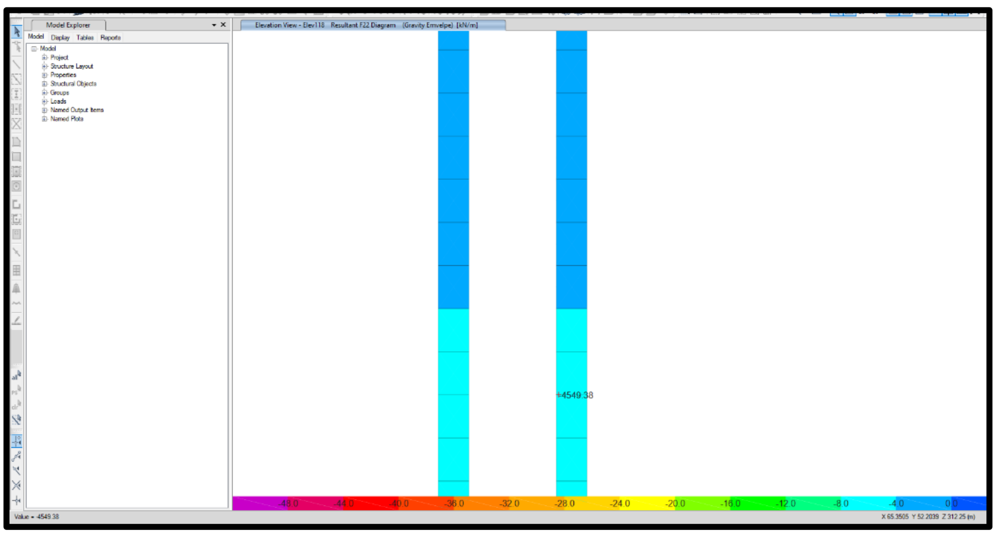


Figure 2.2.9.7 Shear Wall Next to Core V1

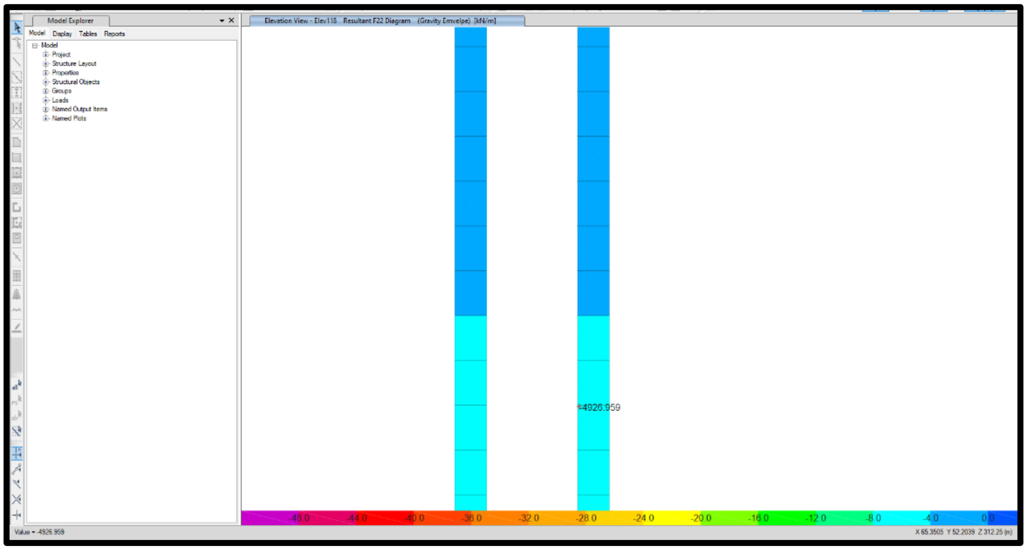
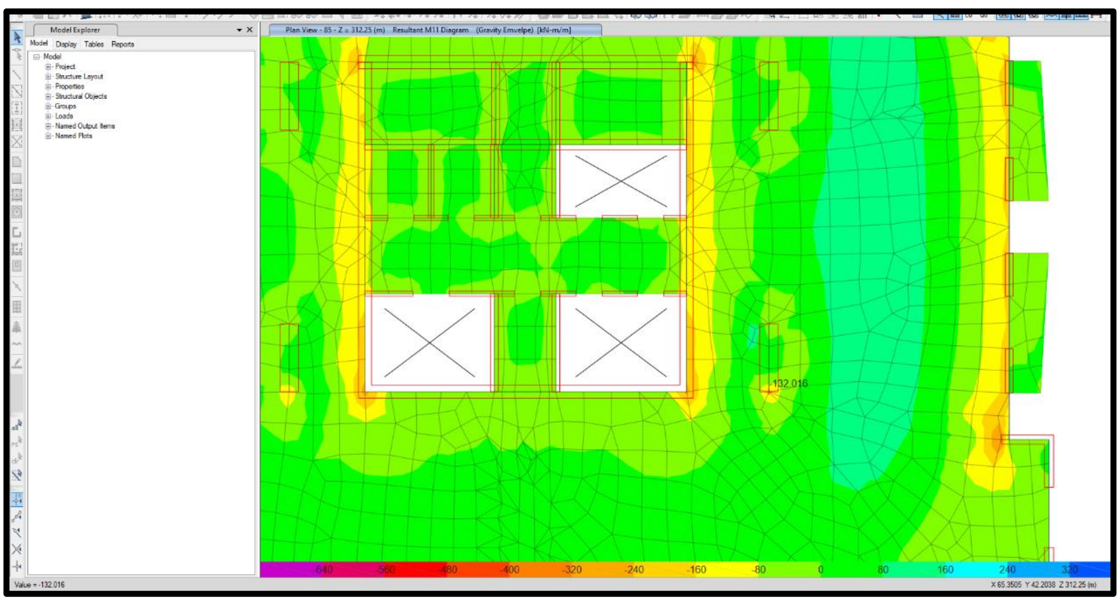


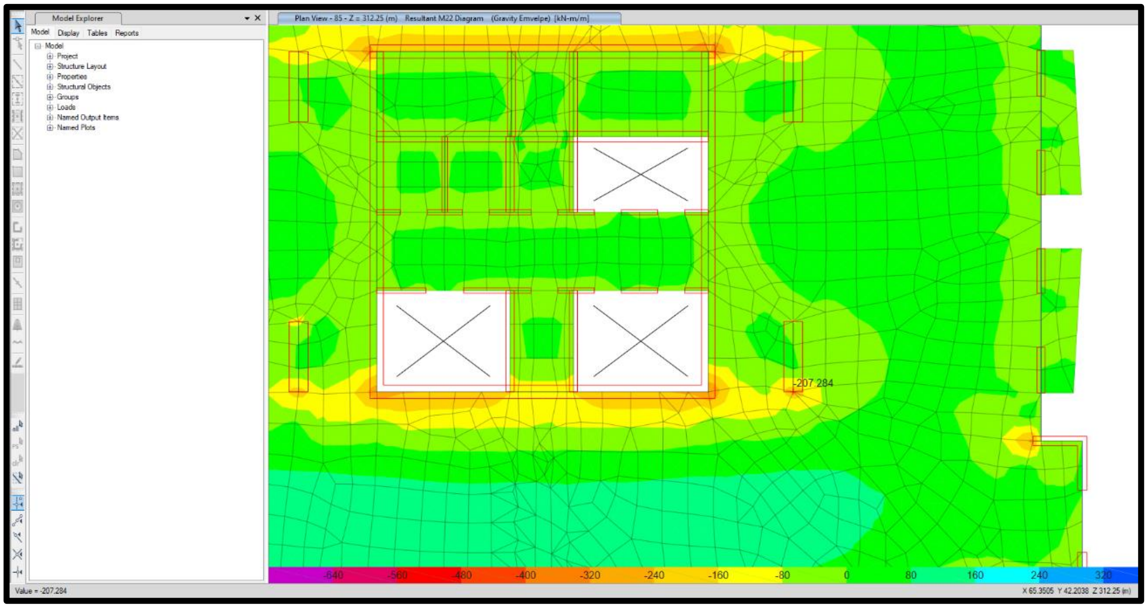
Figure 2.2.9.8 Shear Wall Next to Core V2

Vu = 4926.959 – 4549.38 = 377.579 KN



Mu1 = 102.016 KN.m

Figure 2.2.9.9 Shear Wall Next to Core Mu1



Mu2 = 207.284 KN.m

Figure 2.2.9.10 Shear Wall Next to Core Mu2

$$J_{c1} = 2\left(\frac{b_1 * d^3}{12} + \frac{b_1^3 * d}{12}\right) + 2(b_2 * d * (\bar{x})^2\ ) = 0.548\ m^4$$

$$J_{c2} = 2\left(\frac{b_2 * d^3}{12} + \frac{b_2^3 * d}{12}\right) + 2(b_1 * d * (\bar{y})^2\ ) = 5.32\ m^4$$

$$\gamma_{f1} = \frac{1}{1 + 2/3(\sqrt{\frac{b_1}{b_2}})} = \ 0.7173\ , \qquad \gamma_{v1} = 1 - 0.7173 = 0.2827$$

$$\gamma_{f2} = \frac{1}{1 + 2/3(\sqrt{\frac{b_2}{b_1}})} = \ 0.47, \qquad \gamma_{v2} = 1 - 0.765 = 0.53$$

$$\boldsymbol{Vu,p = 0.2146\ MPa}$$

$$\boldsymbol{\lambda s = 0.877}$$

$$Vc,p = 0.33 * \lambda s * \lambda * \sqrt{f'c} = 2.146\ MPa$$

$$Vc,p = 0.17 * \lambda s * \lambda * \sqrt{f'c} * (1 + \frac{2}{\beta}) = 1.878\ MPa$$

$$Vc,p = 0.083 * \lambda s * \lambda * \sqrt{f'c} * (2 + \frac{\alpha_s * d}{b_0}) = 1.9568\ MPa$$

Take minimum of three, Vc, p = 1.878 Mpa
Vu, p < Vc, p, Acceptable

- **Shear wall at circular slab – 95th floor**

Punching shear force from ETABS (Vu, p) = 2367.456 KN

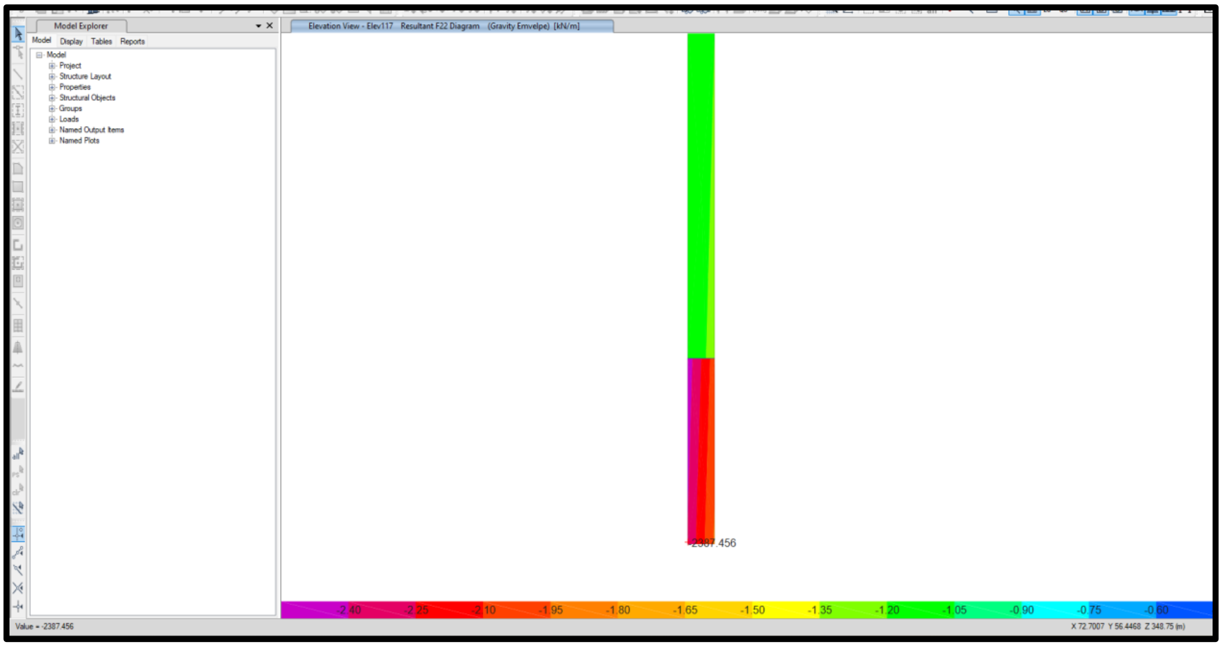


Figure 2.2.9.11 Circular Slab Shear Wall Vu1

The shear wall section and critical section dimensions also are shown below.

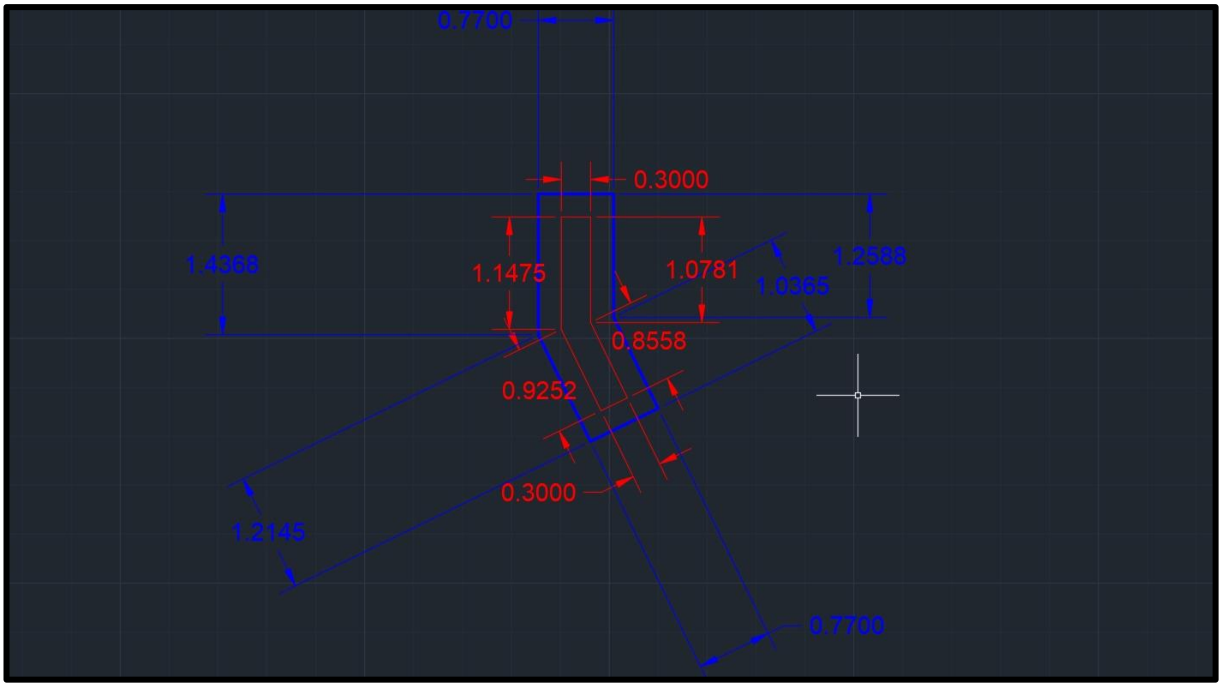


Figure 2.2.9.12 Circular Slab Shear Wall Section

H = 550 mm then d = 470 mm.

$b_1 = (0.952+0.47) + (1.1475+0.47) = 2.6513$ m
$b_2 = 0.3+0.47 = 0.77$ m

$b_o = 6.4866$ m

$\beta = \frac{2.6513}{0.77} = 3.443$, $\alpha_s = 60$

$\lambda_s = 0.833$, $f'c = 55\ MPa$

$$\phi Vc,p = 0.75 * 0.33 * \lambda s * \lambda * \sqrt{f'c} * b_o * d = \mathbf{3801.66\ KN}$$

$$\phi Vc,p = 0.75 * 0.17 * \lambda s * \lambda * \sqrt{f'c} * \left(1 + \frac{2}{\beta}\right) * b_o * d \ = \mathbf{7441.80\ KN}$$

$$\phi Vc,p = 0.75 * 0.083 * \lambda s * \lambda * \sqrt{f'c} * \left(2 + \frac{\alpha_s * d}{b_0}\right) * b_o * d = \mathbf{4661.40\ KN}$$

Take minimum of three, $\phi Vc,p = 3801.66\ KN$.
2367.456 < 3801.66, Vu, p < $\boldsymbol{\phi Vc,p}$ **,** ***Acceptable.***

2.2.10 First Model Design Check

When the model was analyzed using ETABS software, shear failure and moment failure appeared in the beams analyses. As a result, depths of these beams' sections were increased and appropriate dimensions were decided. Upon this change, beams are now behaving in a structural acceptable matter. But, still there appeared an interaction failure (P/M/M failure) in the lower floors' columns as shown in the below screenshots.

Figure 2.2.10.1 Model Design 3D View

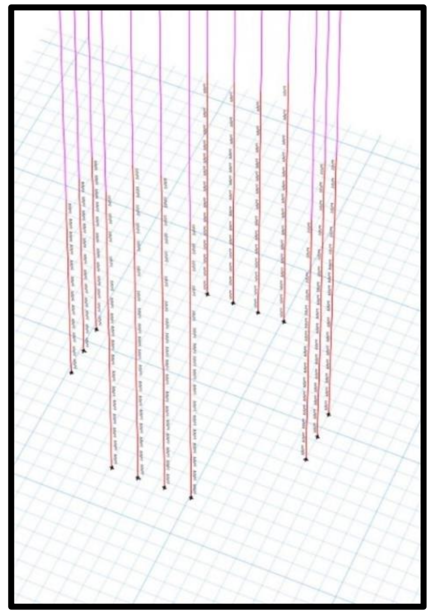


Figure 2.2.10.2 Columns Interaction Failures

2.3 Second Model 3D Structural Analysis

The previously determined failures could be solved by increasing these column dimensions. A more engineering creative solution would be adding a new whole structural system which is called "Outrigger Structural System" to the structure. Such solution will significantly decrease moment values on these columns and ensure that applied gravity loads will be distributed more equally between the columns, it might also decrease shear values in the slabs that were calculated previously which appeared to be greater than slabs' shear capacity, and that is by reducing stories' drifts and displacements. This whole structural system will keep the structure maintaining its safety requirements under gravity loads effect and will result in a better performance in seismic analysis takes place in the second part of this project. In this model, higher strength concrete is used which has allowable stress 70 MPa to increase columns' ability to withstand applied stresses.

2.3.1 Outrigger Structural System

Firstly, high rise building structural system (outrigger system) which will be combined with the original structural system (tube in tube system) to solve the issues that came up throughout first model analysis. Moreover, to ensure that the model is fully prepared and ready for seismic lateral analysis in the second part of this project. As shown below in the following images, the outrigger of members (1000 mm*1000 mm) was added on 3 levels and the dimensions of the outrigger members are shown below.

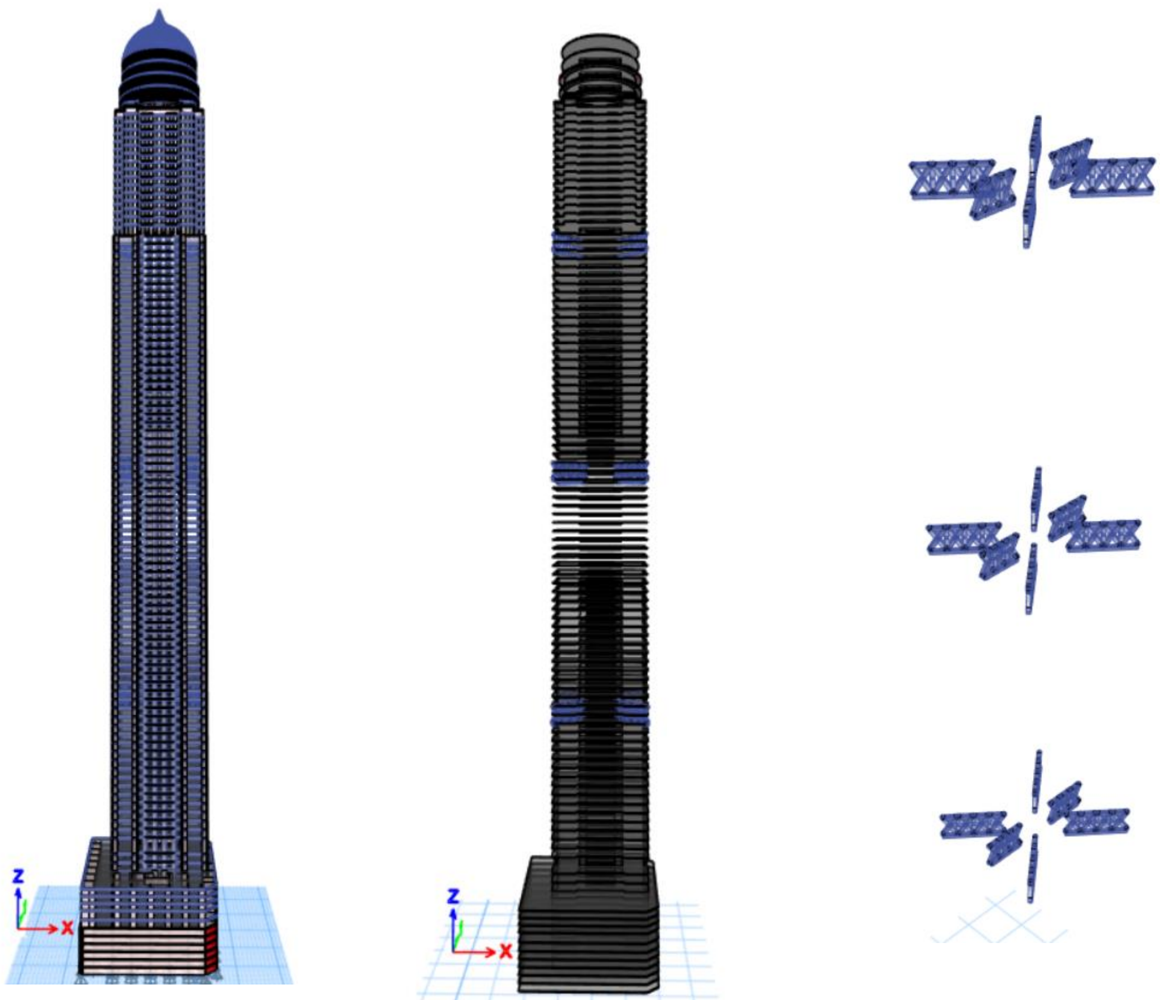


Figure 2.3.1.1 Outrigger Structural System

The outrigger is shown in the below floor plan. It is distributed on 2 floors with a height of 7.3 m. Vertical structural members are connected to the inner core as in the below image.

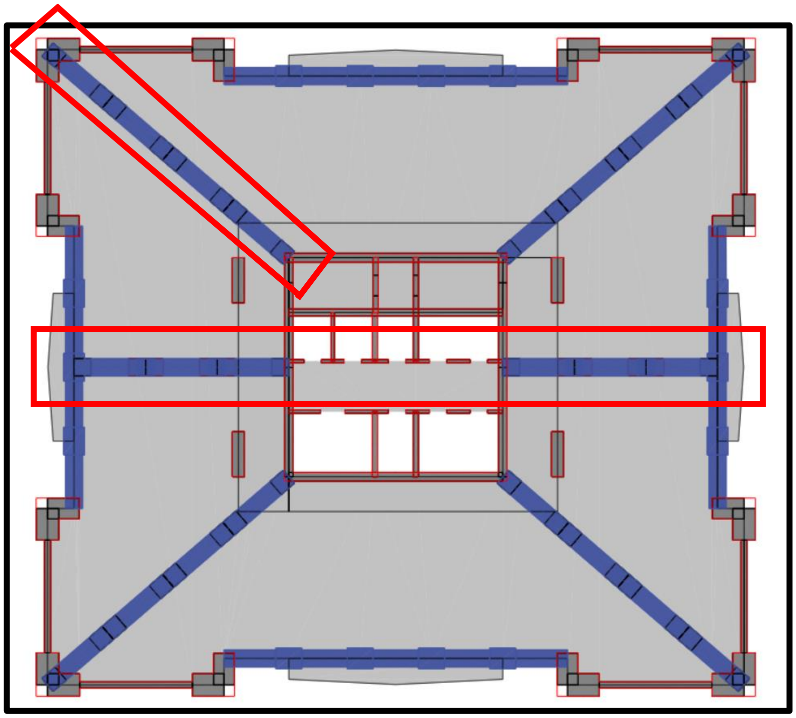


Figure 2.3.1.2 Outrigger Plan Distribution

Moreover, to imagine how a section cut in the horizontal outrigger in the plan view would look like, the following screenshot is an elevation section to enhance understanding the outrigger system.

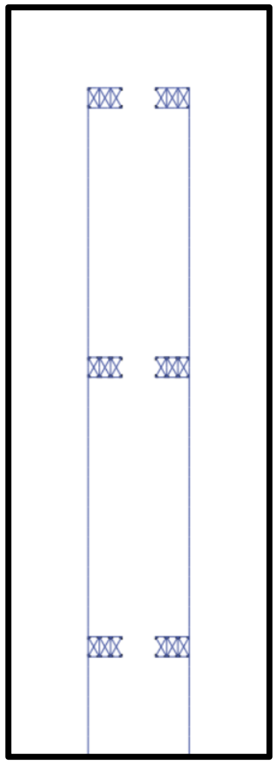


Figure 2.3.1.3 Outrigger Vertical Section

To continue, a zoomed in outrigger from the horizontal section into the elevation section would look like this screenshot, also the dimensions are mentioned below.

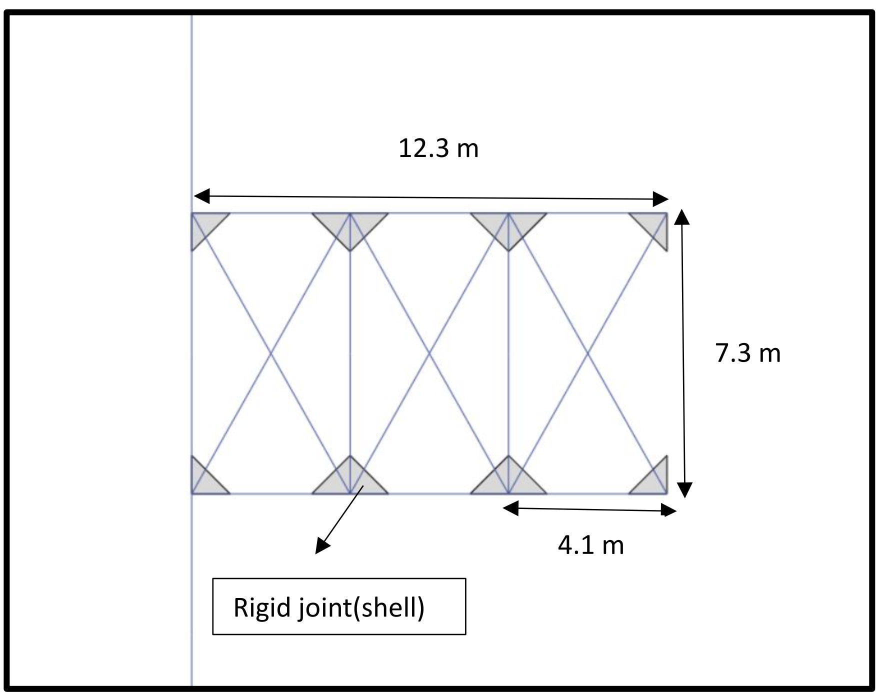


Figure 2.3.1.4 Magnified Outrigger Sample

Another outrigger sample would be the top left in the plan view. An elevation section would look like this.

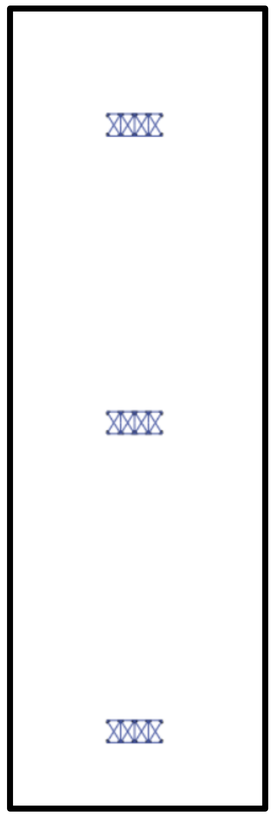


Figure 2.3.1.5 Top Left Outrigger Section

If a closer look is taken onto this section, this is how it would look like.

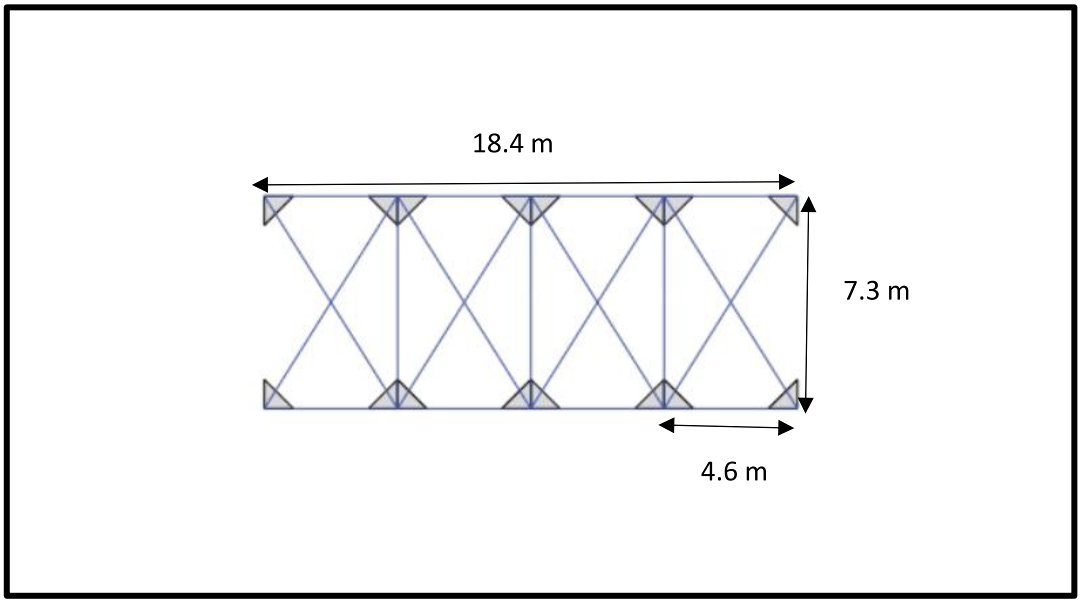


Figure 2.3.1.6 Magnified Top Left Outrigger Section

2.3.2 Drop Panels

After adding the outrigger structural system to the project, drop panels must be used as well to reduce the difference between applied shear and slab shear capacity. Drop panels might not solve the problem completely due to the large difference, but it will for sure help in fixing the shear issue in design phase.

Shown below the drop panels that were planted in the floors of high applied shear values, ring drop panels were added in the floors that faced shear failures only (floors 95,96,97) and around the inner core in all of the floors.

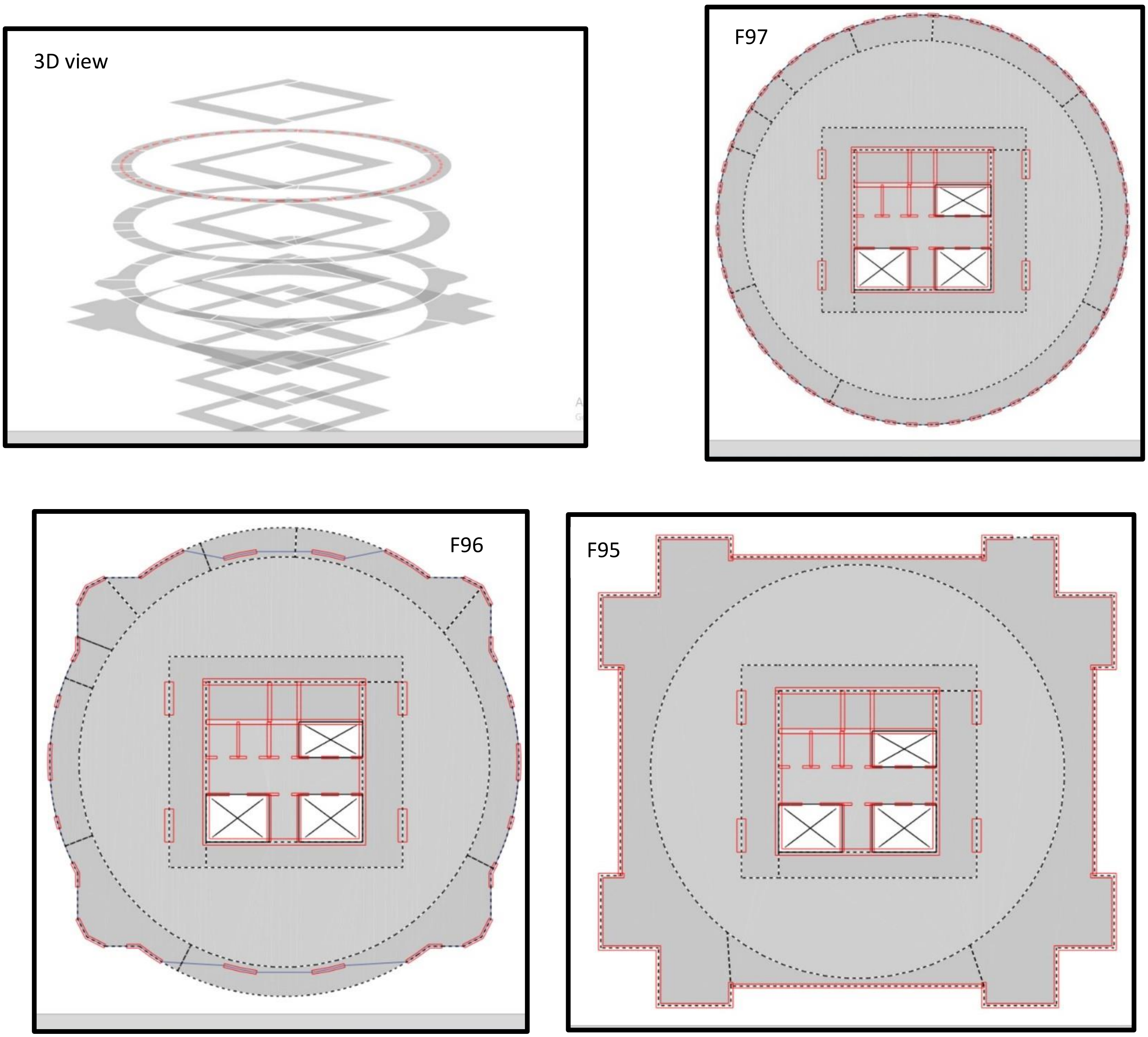


Figure 2.3.2.1 Drop Panels Plan View

2.3.3 Structural Behavior Improvements

- **First model last story displacements**

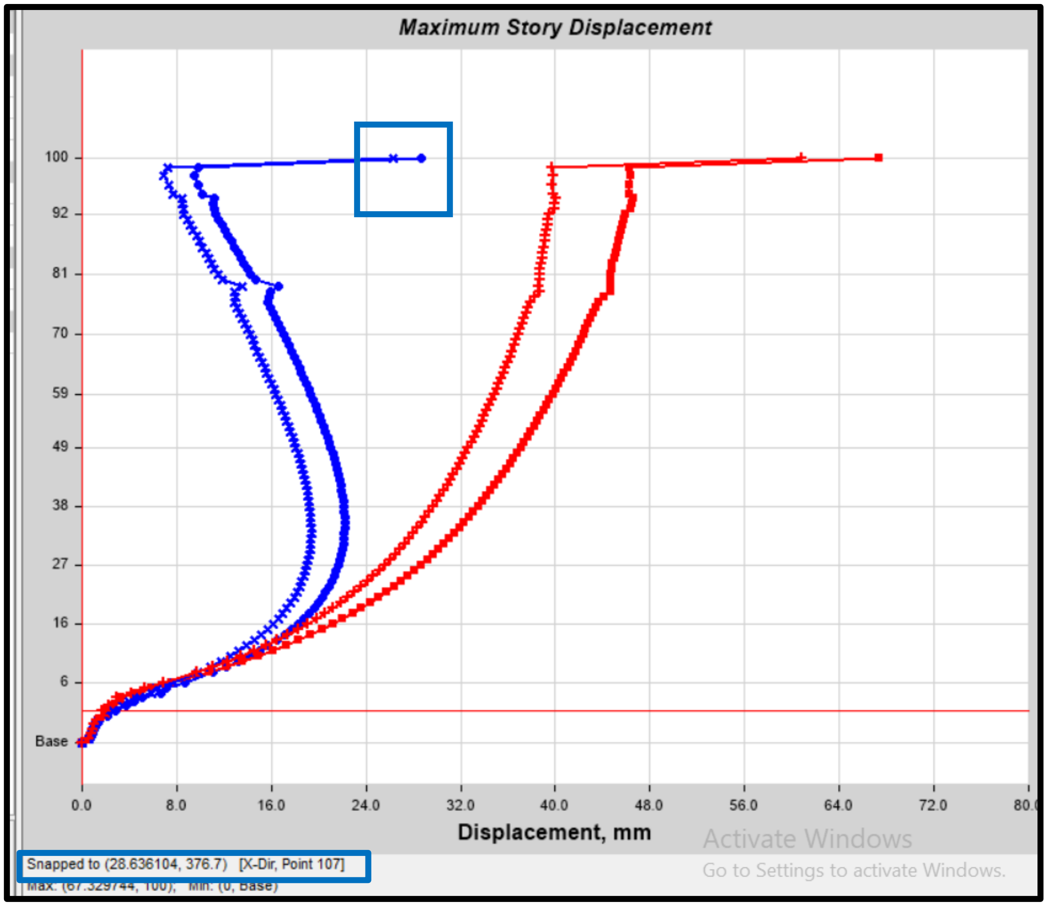


Figure 2.3.3.1 Last Story Displacement in X-Direction

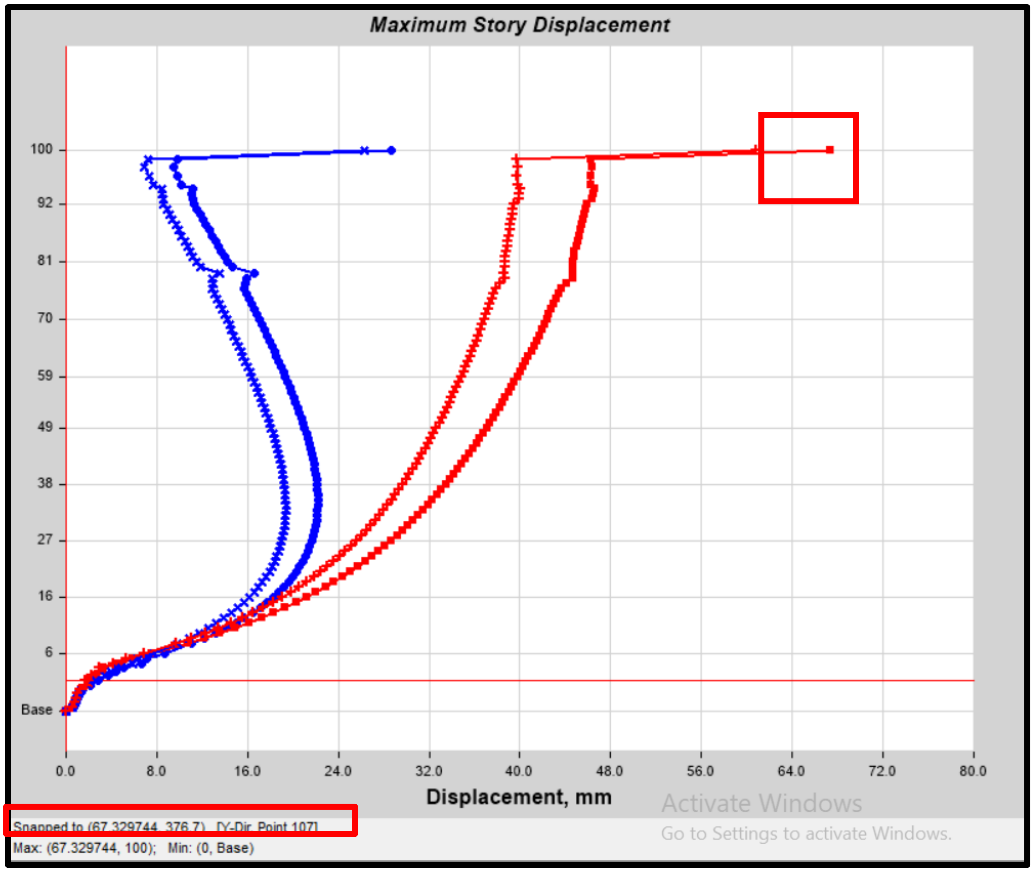


Figure 2.3.3.2 Last Story Displacement in Y-Direction

- **First Model Drifts**

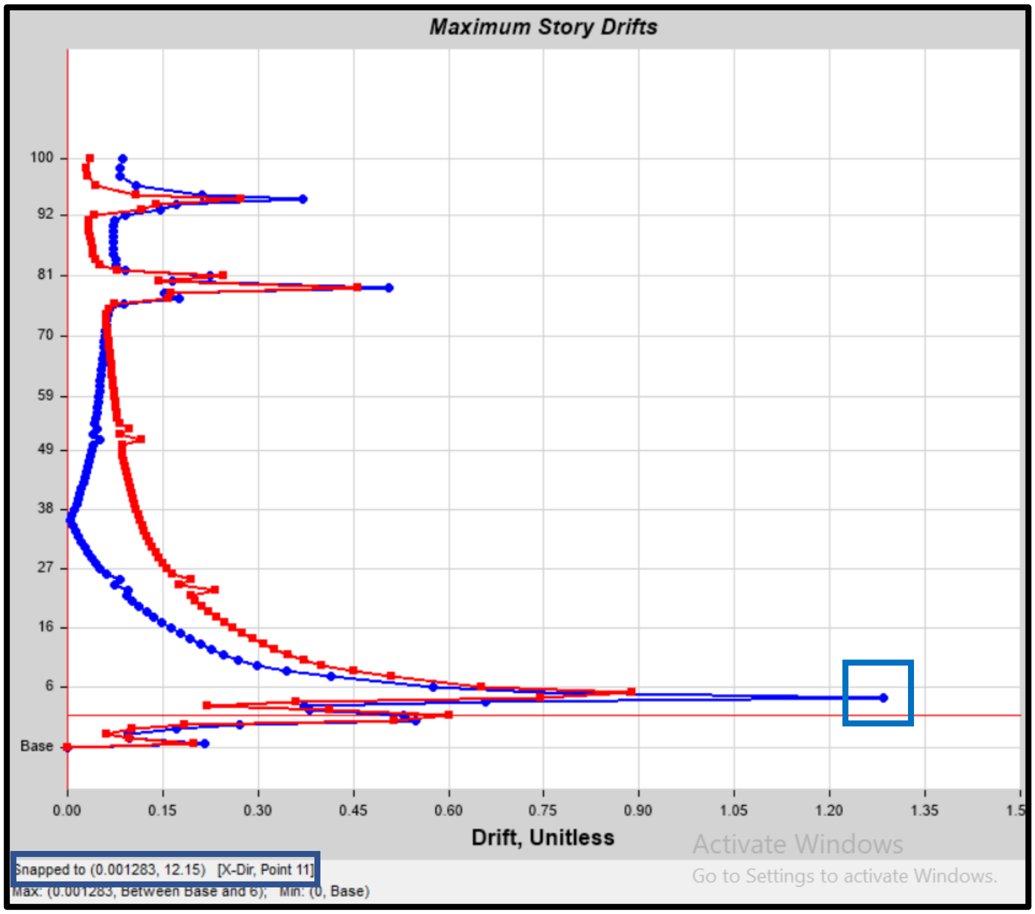


Figure 2.3.3.3 Maximum Story Drift in X-Direction

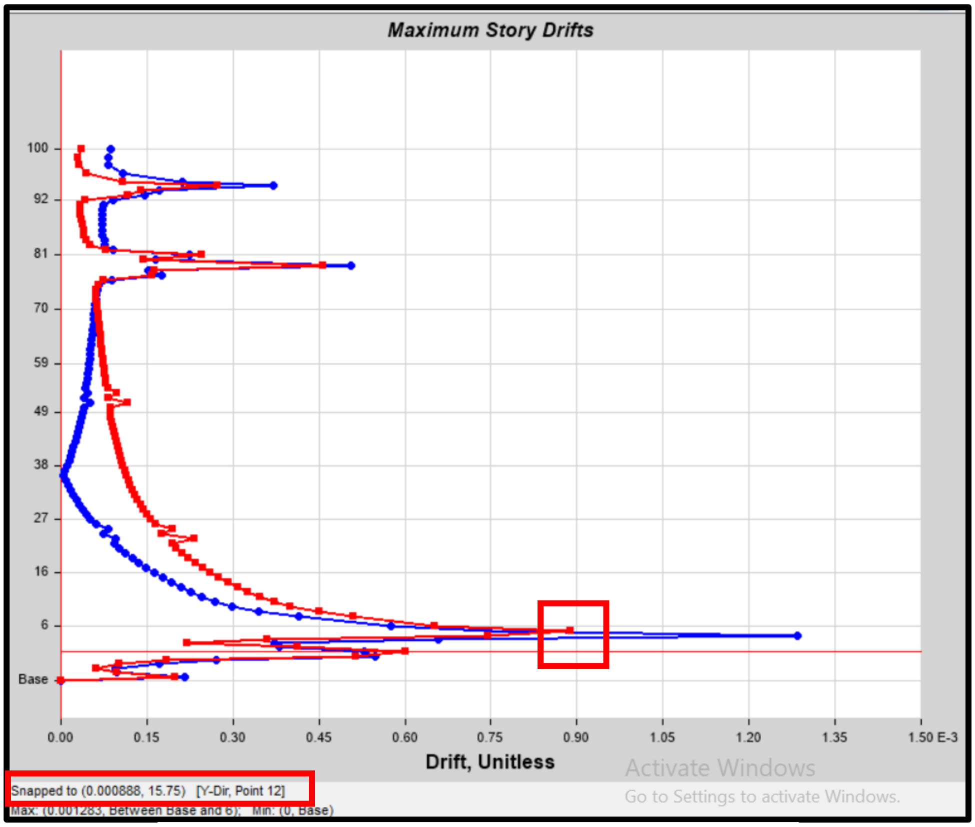


Figure 2.3.3.4 Maximum Story Drift in Y-Direction

- **Second model last story displacements**

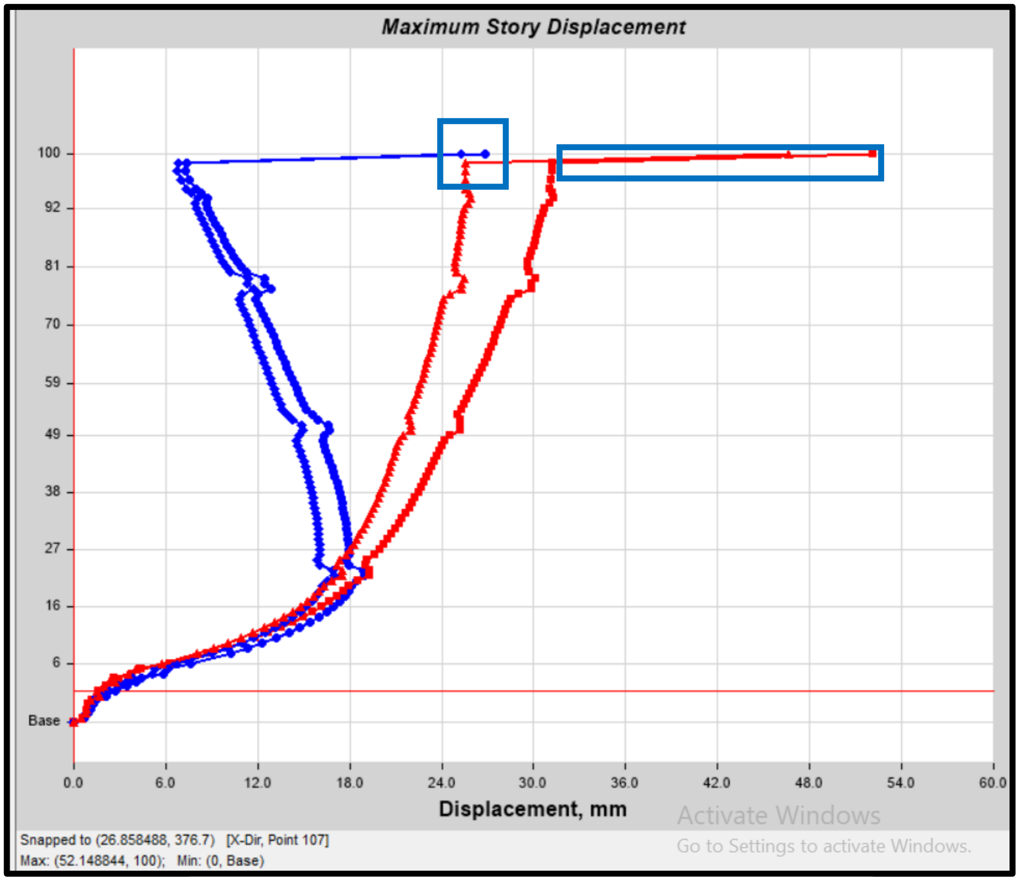


Figure 2.3.3.5 Last Story Displacement in X-Direction

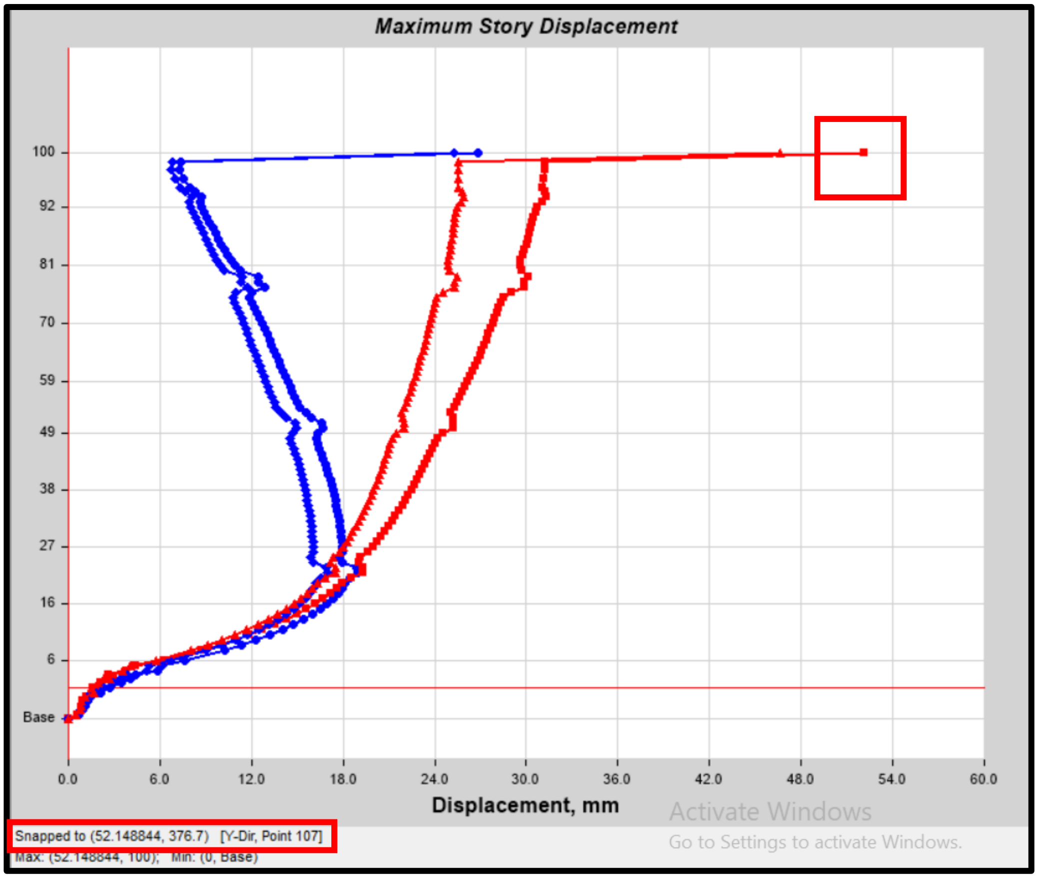


Figure 2.3.3.6 Last Story Displacement in Y-Direction

Second model drifts

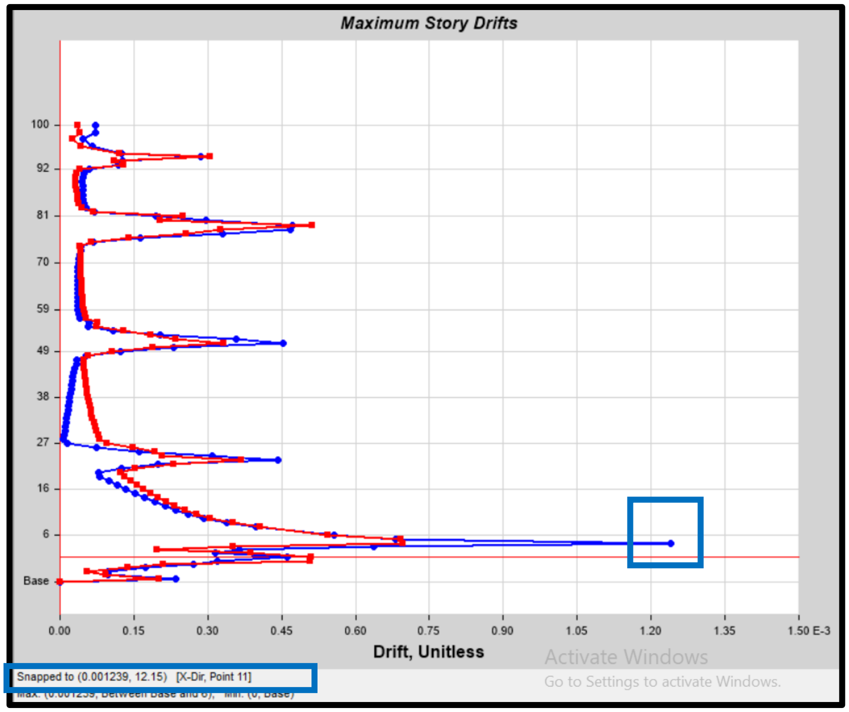


Figure 2.3.3.7 Maximum Story Drift in X-Direction

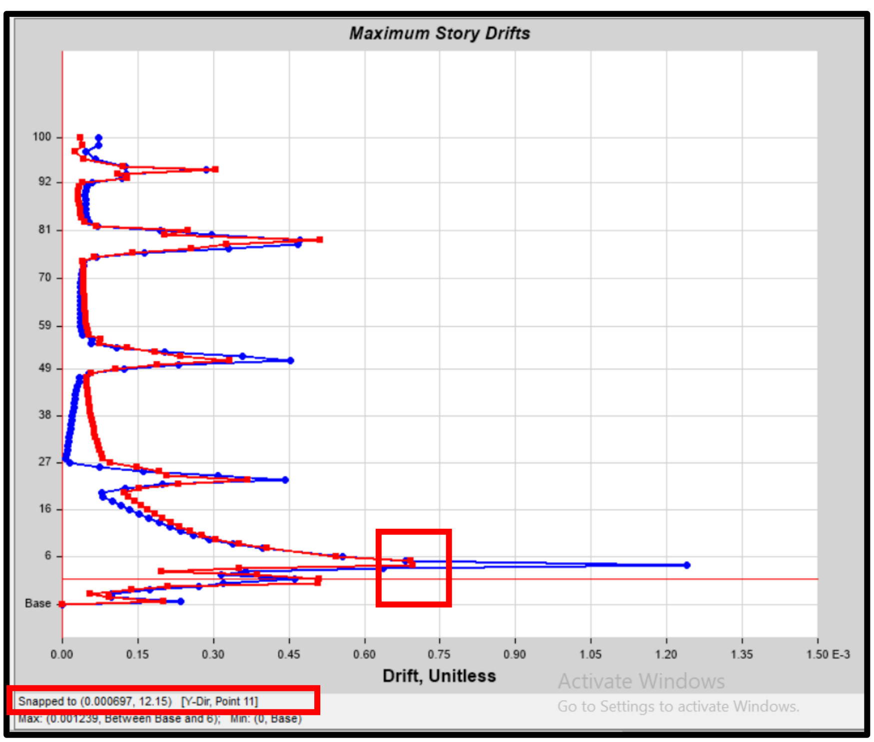


Figure 2.3.3.8 Maximum Story Drift in Y-Direction

- **First model maximum story displacements**

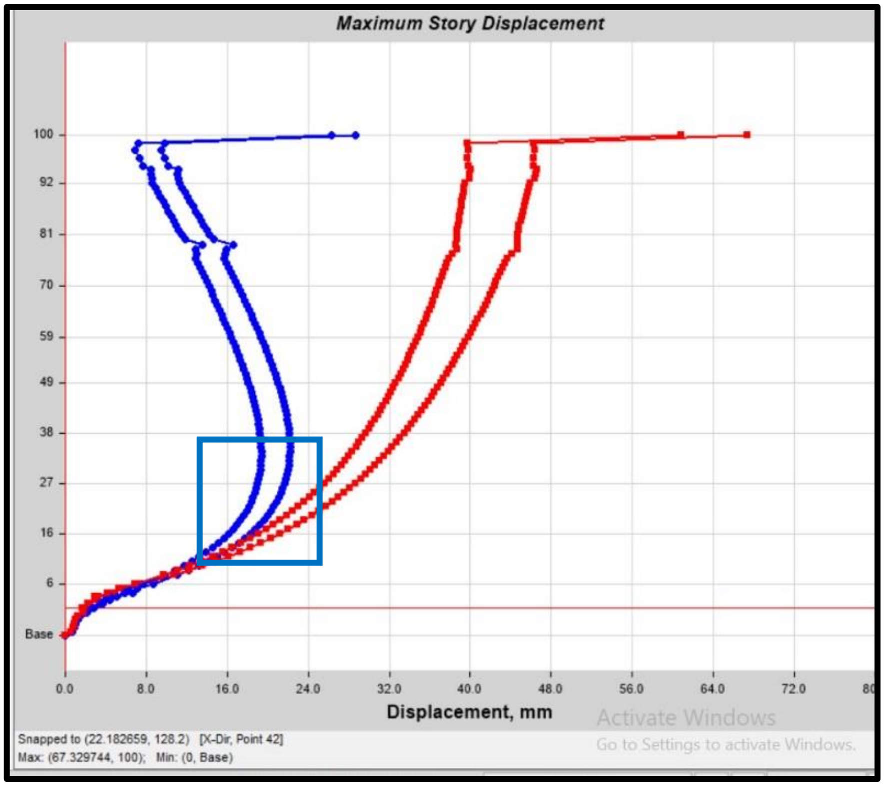


Figure 2.3.3.9 Maximum Below Story Displacement in X-Direction

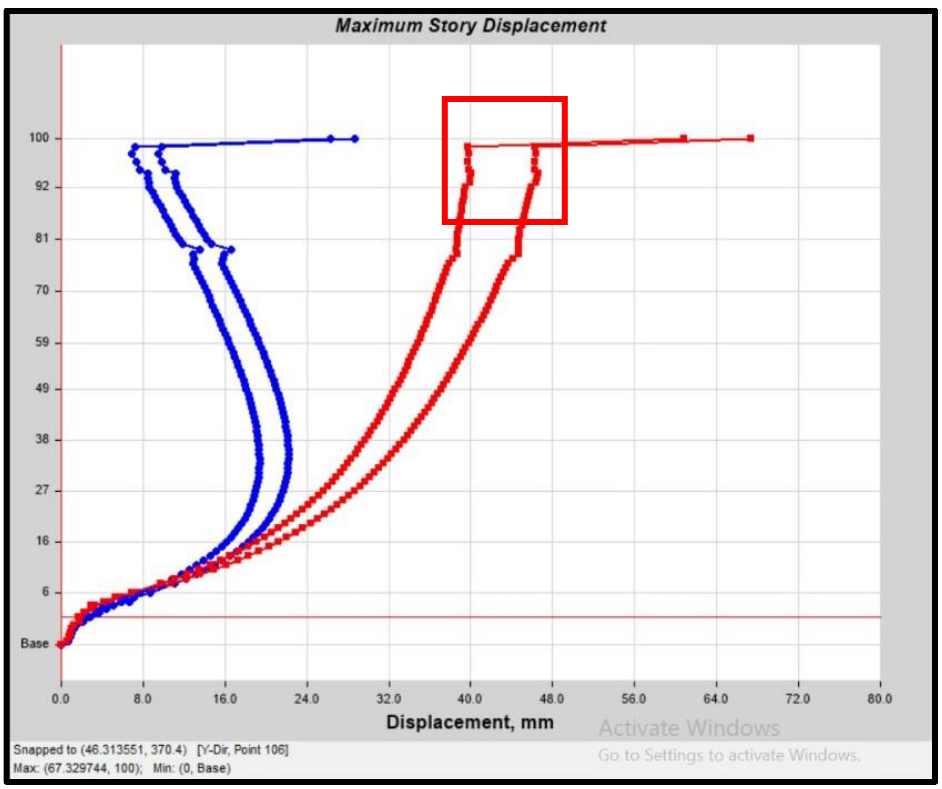


Figure 2.3.3.10 Maximum Below Story Displacement in Y-Direction

- **Second model maximum story displacements**

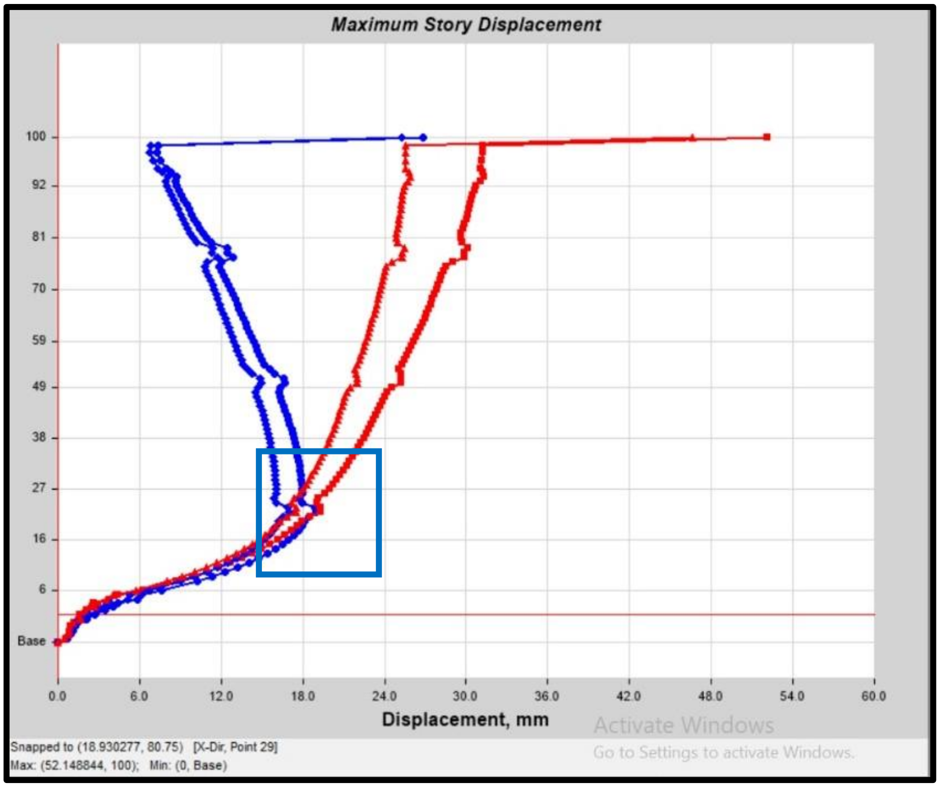


Figure 2.3.3.11 Maximum Below Story Displacement in X-Direction

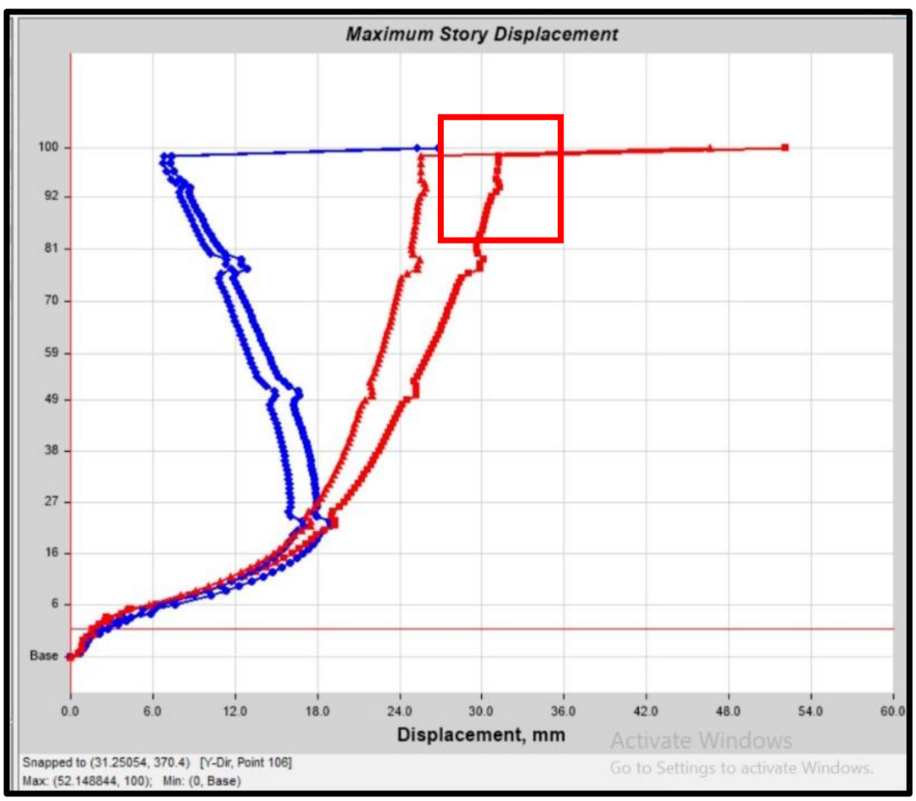


Figure 2.3.3.12 Maximum Below Story Displacement in Y-Direction

2.3.4 Second Model Deflection Check

The load combination for the long-term deflection is:

$$\lambda_\infty(D + SD) + (\lambda_t * Sustained\ Live\ Load) + Unsustained\ live\ load$$

$$\lambda = \frac{\zeta}{1 + 50\rho'}$$

Take $\rho' = 0$

When t is large enough, $\zeta = 2$, $\lambda = 2$

Sustained Live Load = 25% of total live load

Unsustained live load = 75% of total live load

So, the load combination is:

$2 * (D + SD) + (2 * 0.25 * Live\ Load) + 0.75 * Live\ Load$ = 2*D + 2*SD + 1.25*Live Load

- **Floor Slab at Parking Level (3rd Basement Floor)**

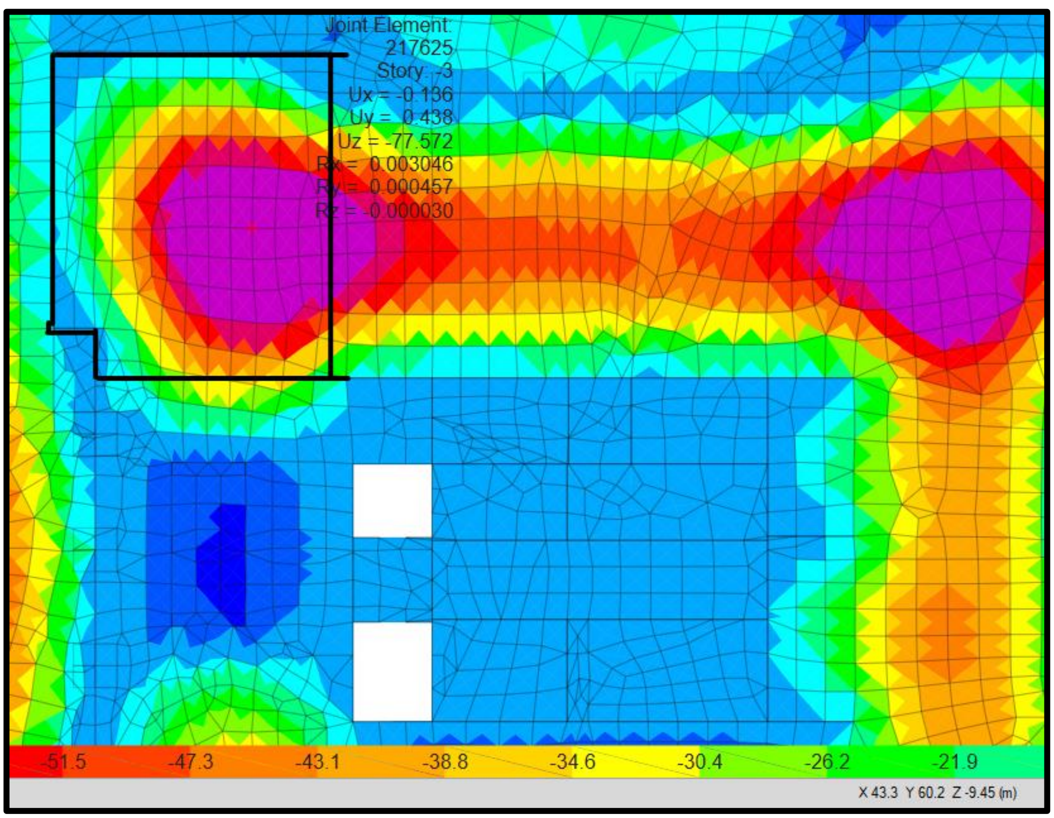


Figure 2.3.4.1 Floor Slab at Parking Level Long Term Deflection

Deflection at middle point = 77.572 mm

Deflection at top left edge = 11.673 mm

Deflection at bottom right edge = 10.291

Average edges deflection = $\frac{6.4882*11.673+9.7431*10.291}{16.2313} = 10.843\ mm$

Relative Deflection = 77.572 – 10.843 = **66.729 mm**

Limit = $\frac{16.2313*1000}{240} = \boldsymbol{67.63\ mm > 66.729\ , Acceptable}$

- **Parking Slab (3rd Basement Floor)**

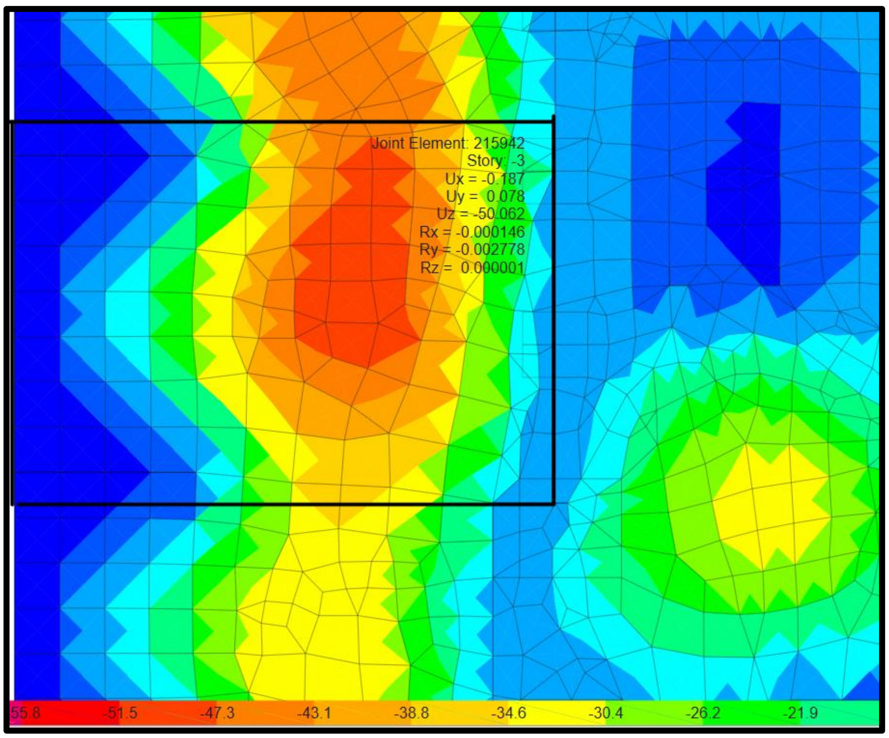


Figure 2.3.4.2 Parking Slab Long Term Deflection

Maximum deflection = 50.062 mm

Deflection at left edge = 1.817 mm

Deflection at right edge = 11.49 mm

Average edges deflection = $\frac{4.5*1.817+8.7*11.49}{13.2} = 8.19\ mm$

Relative Deflection = 50.062 – 8.19 = **41.872 mm**

Limit = $\frac{13.2*1000}{240} = \boldsymbol{55\ mm > 41.872\ mm\ , Acceptable}$

- **Storage Slab (3rd Basement Floor)**

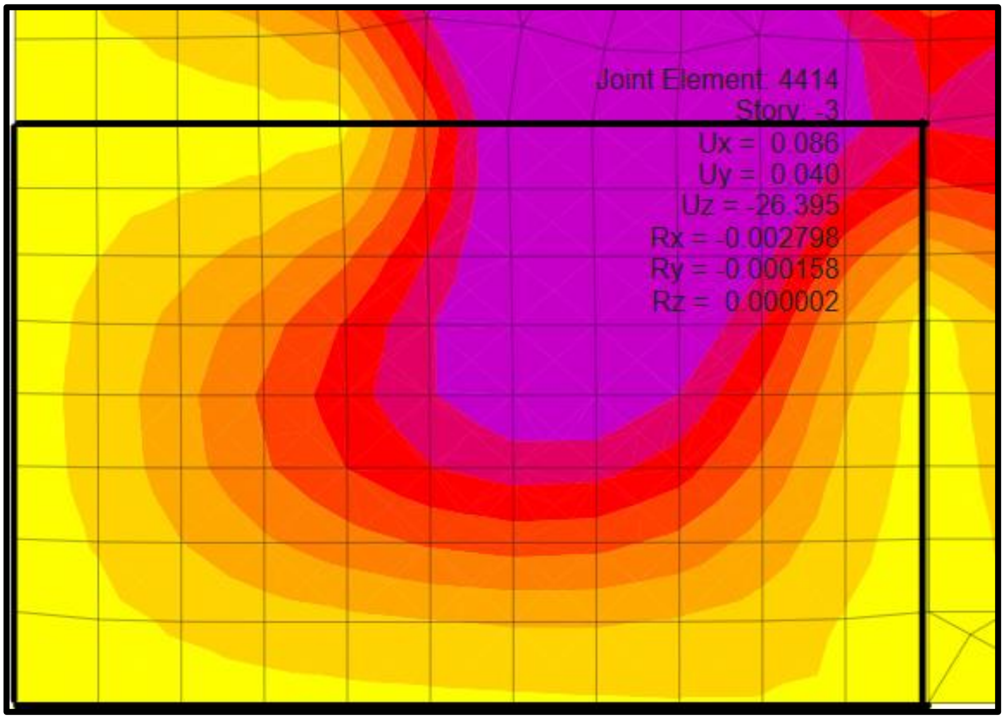


Figure 2.3.4.3 Storage Zone Slab Long Term Deflection

Maximum deflection = 26.395 mm

Top left edge deflection = 0.014 mm

Bottom right edge deflection = 0.857 mm

Average edges deflection = $\frac{6.9202*0.014+8.4073*0.857}{15.3275} = 0.4764\ mm$

Relative deflection = 26.395 – 0.4764 = 25.9186 mm

Limit = $\frac{14.85*1000}{240} = \mathbf{61.875\ mm > 25.9186\ mm\ , Acceptable}$

- **Floor slab (6th Floor)**

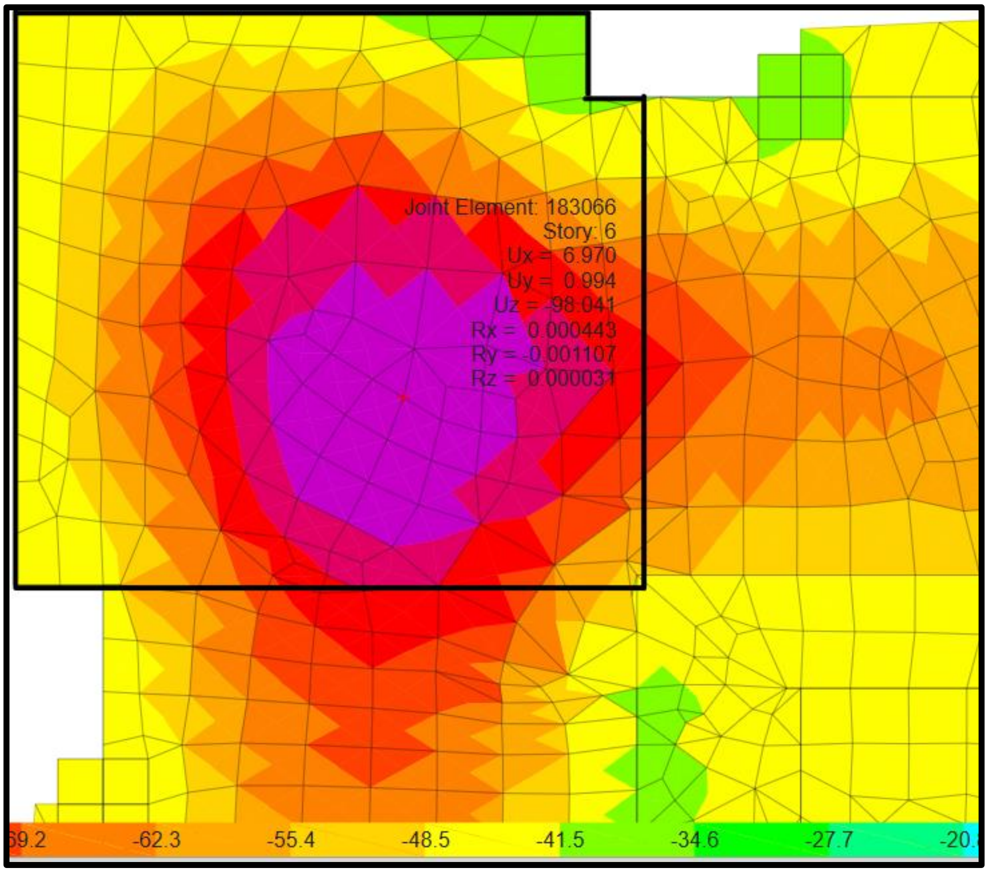


Figure 2.3.4.4 Floor Slab Long Term Deflection

Maximum deflection = 98.041 mm

Top left edge deflection = 42.99 mm

Bottom left edge deflection = 47.645 mm

Average edges deflection = $\frac{5.22*42.99+9.65*47.645}{14.87} = 46\ mm$

Relative deflection = 98.041 – 46 = 52.041 mm

Limit = $\frac{14.87*1000}{240} = \boldsymbol{61.95\ mm > 52.041\ mm\ , Acceptable}$

- **Inner Core Drop Panel Slab (6th Floor)**

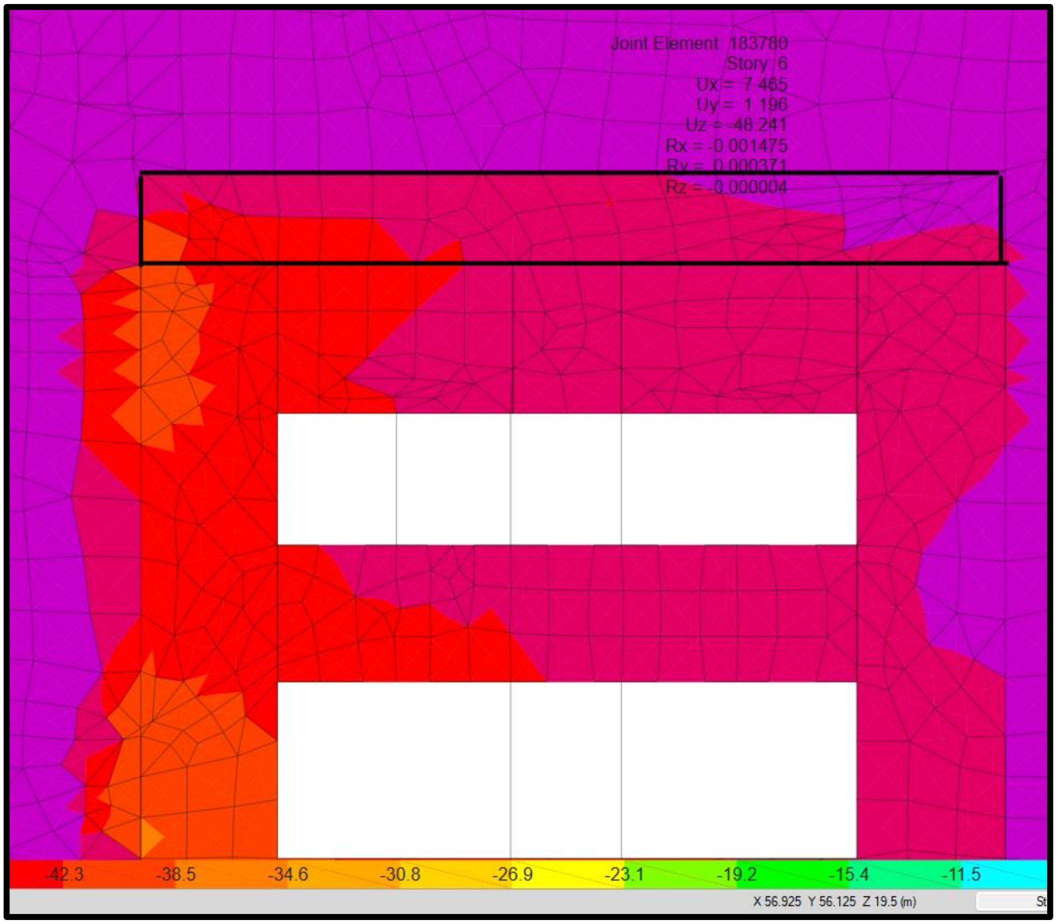


Figure 2.3.4.5 Inner Core Drop Panel Long Term Deflection

Maximum deflection = 48.241 mm

Top edge deflection = 49.288 mm

Bottom edge deflection = 47.317 mm

Average edges deflection = $\frac{1.3*49.288+0.65*47.317}{1.95} = 48.631\ mm$

Relative deflection = 48.631 – 48.241 = 0.39 mm

Limit = $\frac{1.95*1000}{240} = \mathbf{8.125\ mm > 0.39\ mm\ , \mathit{Acceptable}}$

- **Circular Slab (95th Floor)**

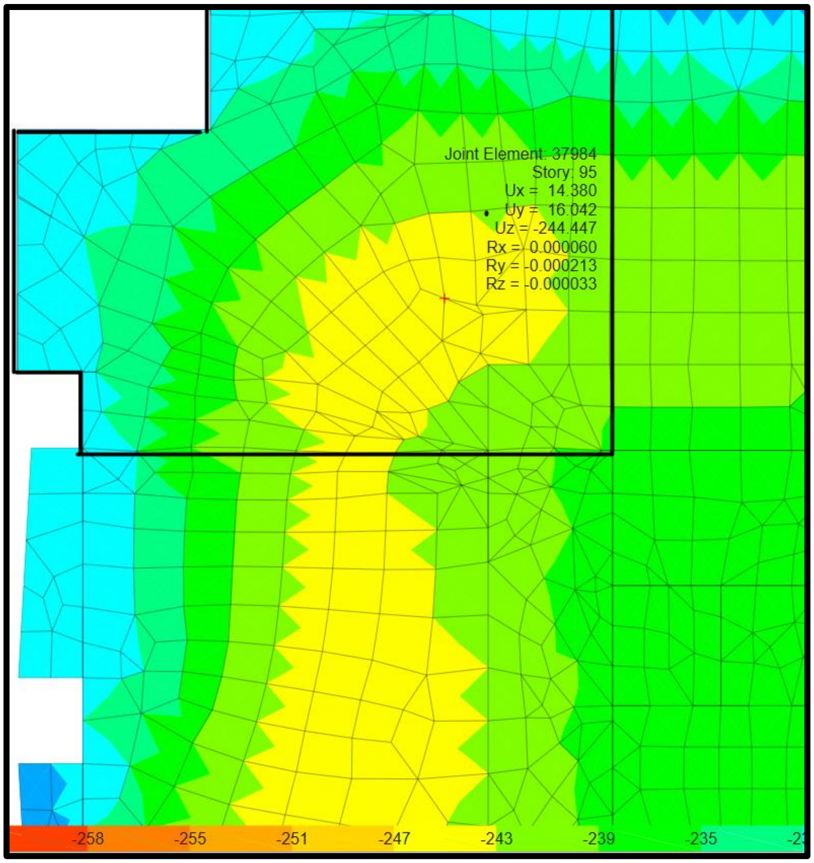


Figure 2.3.4.6 Circular Slab Long Term Deflection

Maximum deflection = 244.447 mm

Top left edge deflection = 228.708 mm

Bottom right edge deflection = 238.057 mm

Average edges deflection = $\frac{5.5*228.708+8.5*238.057}{14} = 234.38\ mm$

Relative deflection = 244.447 – 234.38 = 10.067 mm

Limit = $\frac{14*1000}{240} = \mathbf{58.333\ mm > 10.067\ mm\ ,Acceptable}$

- **Circular Slab (99th Floor)**

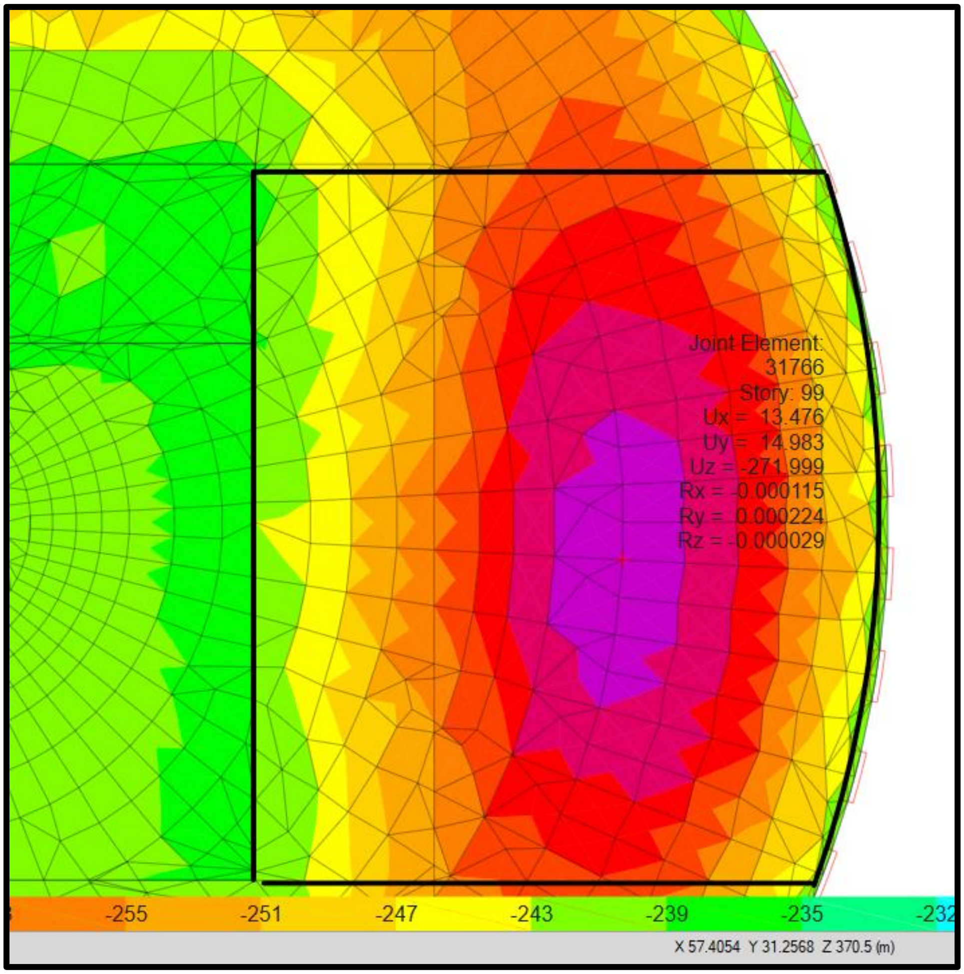


Figure 2.3.4.7 Circular Slab Long Term Deflection

Maximum Deflection = 271.99 mm

Left edge deflection = 238.9 mm

Right edge deflection = 240.634 mm

Average deflection = $\frac{4.6*238.9+6.5*240.634}{11.1} = 239.9\ mm$

Relative deflection = 271.99 – 239.9 = 32.09 mm

Limit = $\frac{11.1*1000}{240} = \mathbf{45.83\ mm > 32.09\ mm, Acceptable}$

- **Ring Drop Panel (96th Floor)**

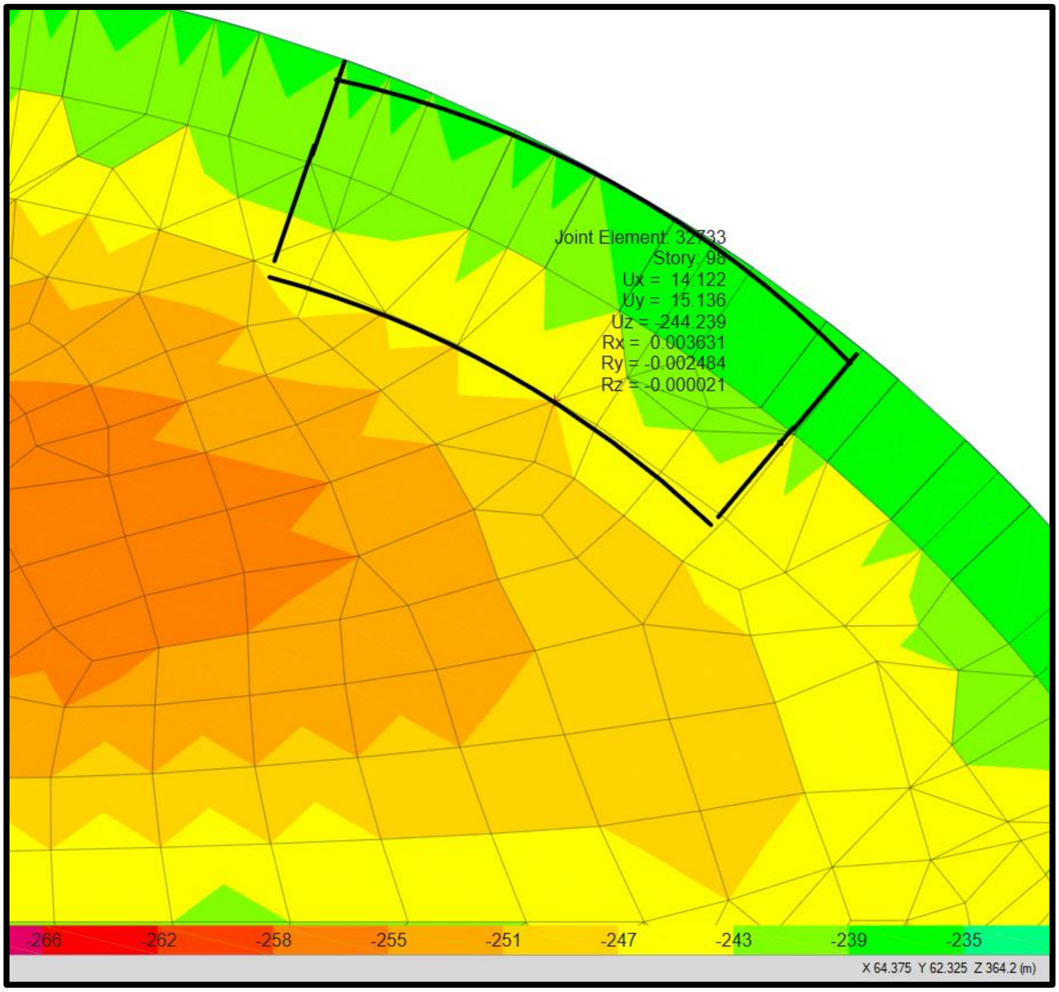


Figure 2.3.4.8 Ring Drop Panel Long Term Deflection

Maximum Deflection = 244.239 mm

Left edge deflection = 244.863 mm

Right edge deflection = 243.203mm

Average edges deflection = $\frac{3.1*244.863+3.4*243.203}{6.5} = 244\ mm$

Relative deflection = 244.239 – 244 = 0.239 mm

Limit = $\frac{6.5*1000}{240}\ =\ \mathbf{27\ mm\ >\ 0.239\ mm\ ,\ Acceptable}$

- **Pump Room Slab (5th Basement Floor)**

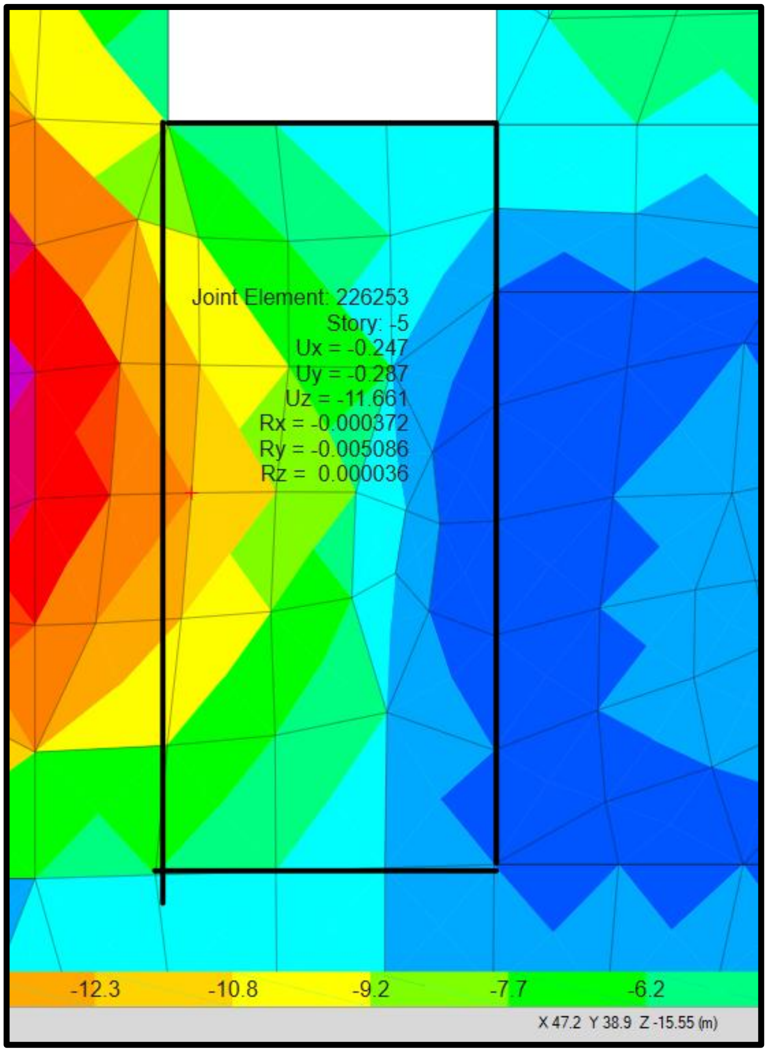


Figure 2.3.4.9 Pump Room Slab Long Term Deflection

Maximum deflection = 11.661 mm

Top edge deflection = 4.223 mm

Bottom edge deflection = 3.809 mm

Average edges deflection = $\frac{3.4*4.223+3.25*3.809}{6.65} = 4.02\ mm$

Relative deflection = 11.661 – 4.02 = 7.641 mm

Limit = $\frac{6.65*1000}{240} = \boldsymbol{27.7\ mm > 7.641\ mm\ , Acceptable}$

- **Water Tank Slab (3rd Basement Floor)**

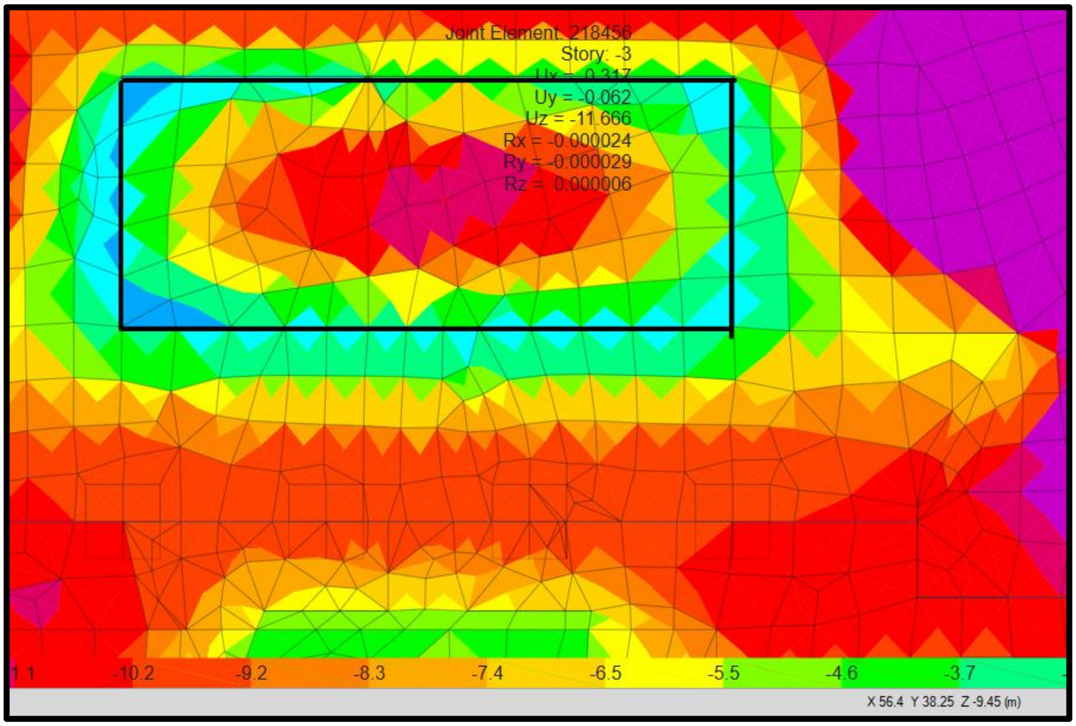


Figure 2.3.4.10 Water Tank Slab Long Term Deflection

Maximum deflection = 11.666 mm

Top edge deflection = 1.965 mm

Bottom edge deflection = 1.166 mm

Average edges deflection = $\frac{2.5*1.965+2.5*1.166}{5} = 1.56\ mm$

Relative deflection = 11.666 – 1.56 = 10.1 mm

Limit = $\frac{5*1000}{240} = \boldsymbol{20.833\ mm > 10.1\ mm\ , Acceptable}$

- **Electrical Room Slab (1st Parking Floor)**

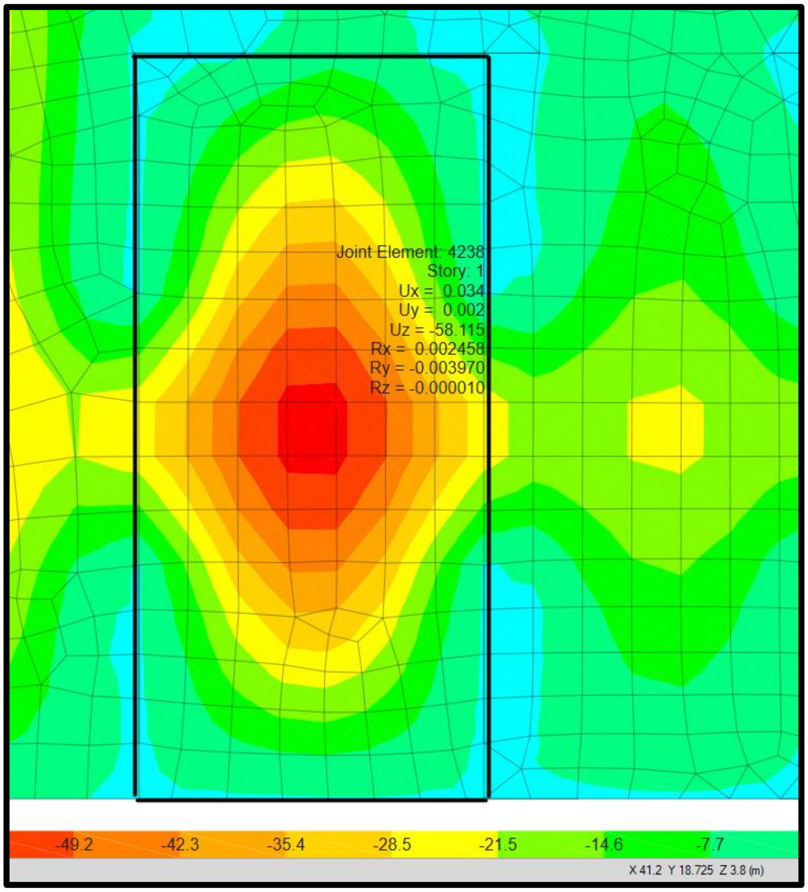


Figure 2.3.4.11 Electrical Room Slab Long Term Deflection

Maximum deflection = 58.115mm

Deflection at bottom left edge = 0.121 mm

Deflection at top right edge = 0.296 mm

Average edges deflection = $\frac{8.525*0.121+9.875*0.296}{18.4} = 0.21492\ mm$

Relative deflection = 58.115 – 0.21492 = 57.9 mm

Limit = $\frac{18.455*1000}{240} = \mathbf{76.895\ mm > 57.9\ mm\ , Acceptable}$

2.3.5 Lateral Loads Definition

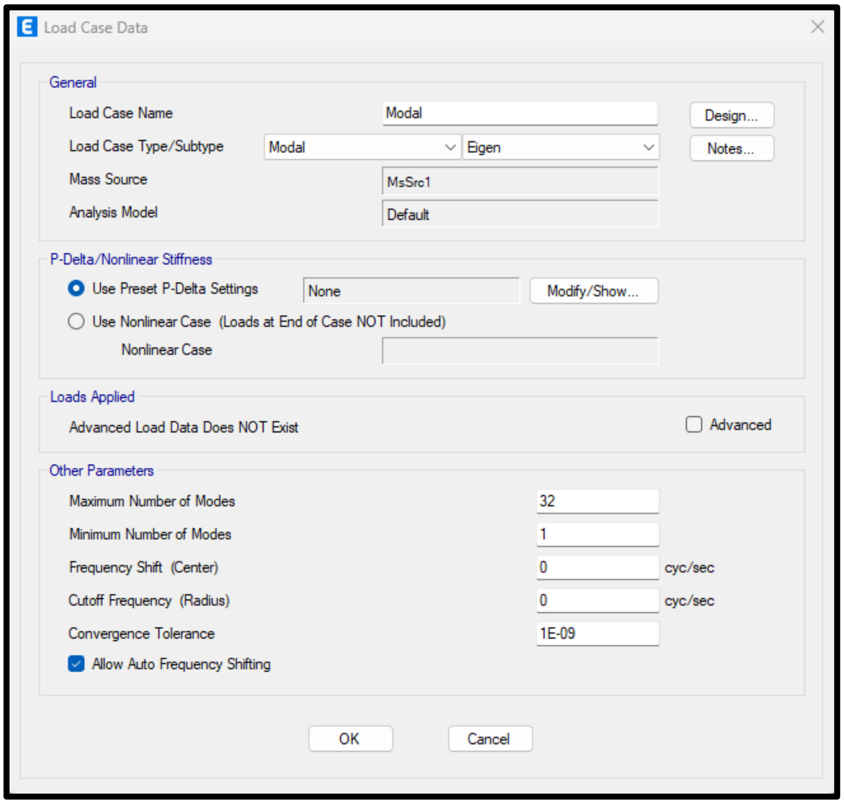


Figure 2.2.5.1 – Eigen Model Load Case Definition

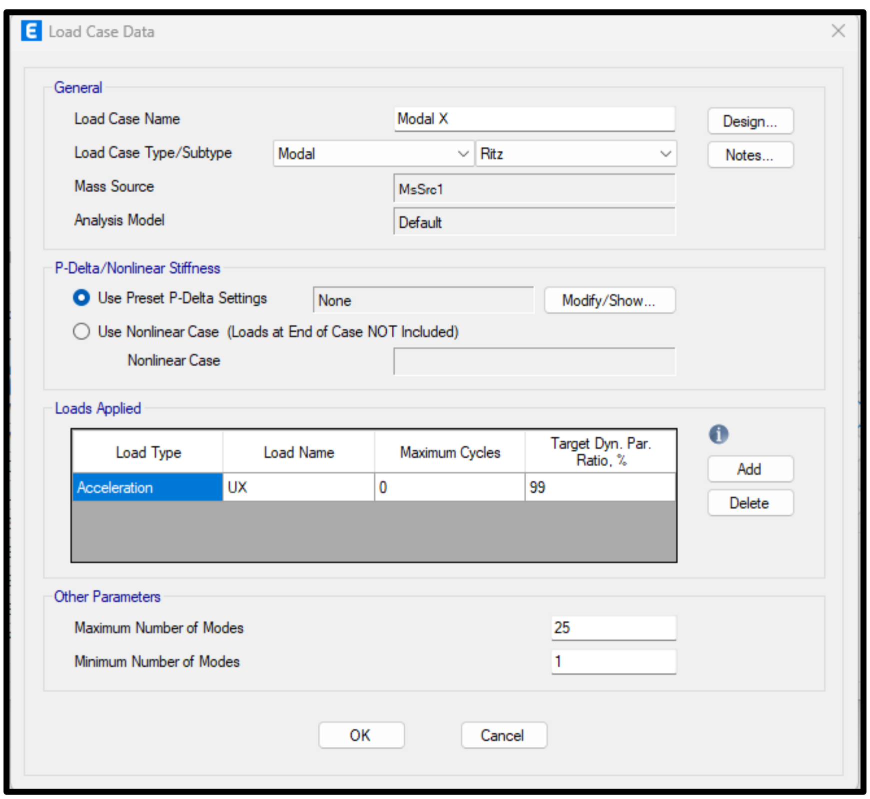


Figure 2.2.5.2 – Modal X

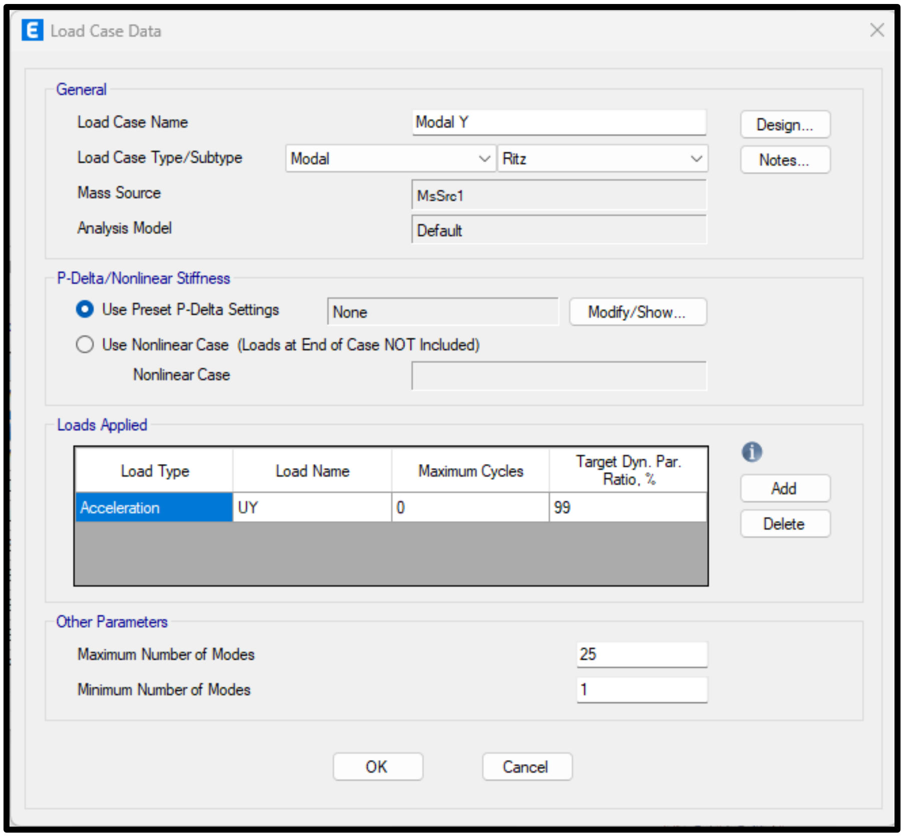


Figure 2.2.5.3 – Modal Y

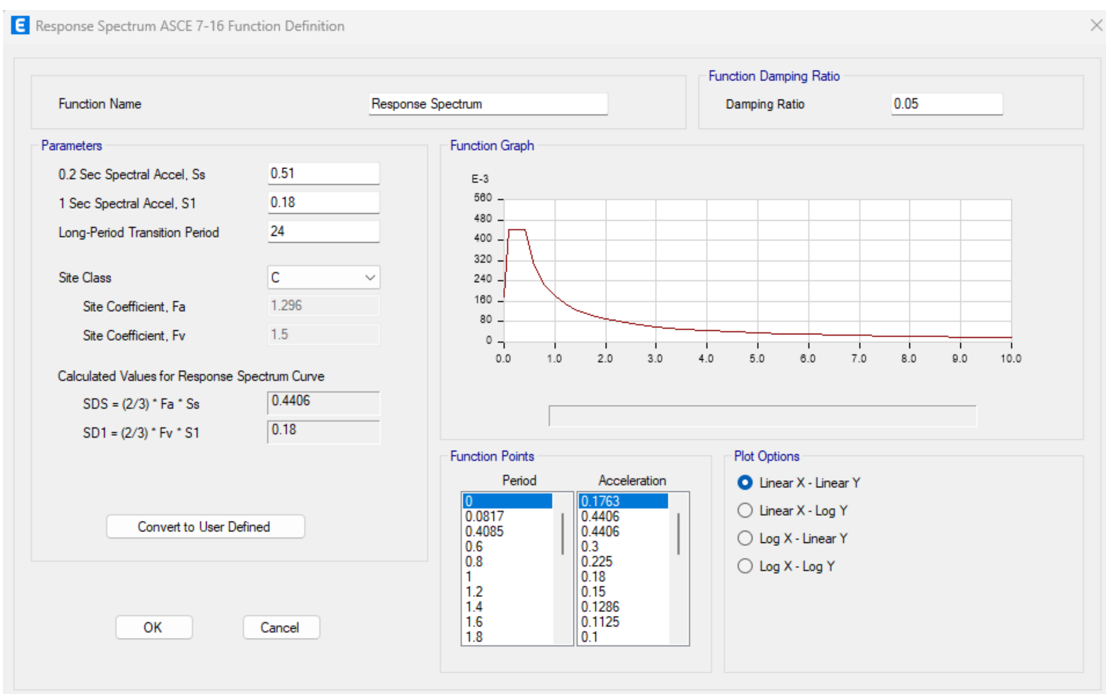


Figure 2.2.5.4 – Response Spectrum Function

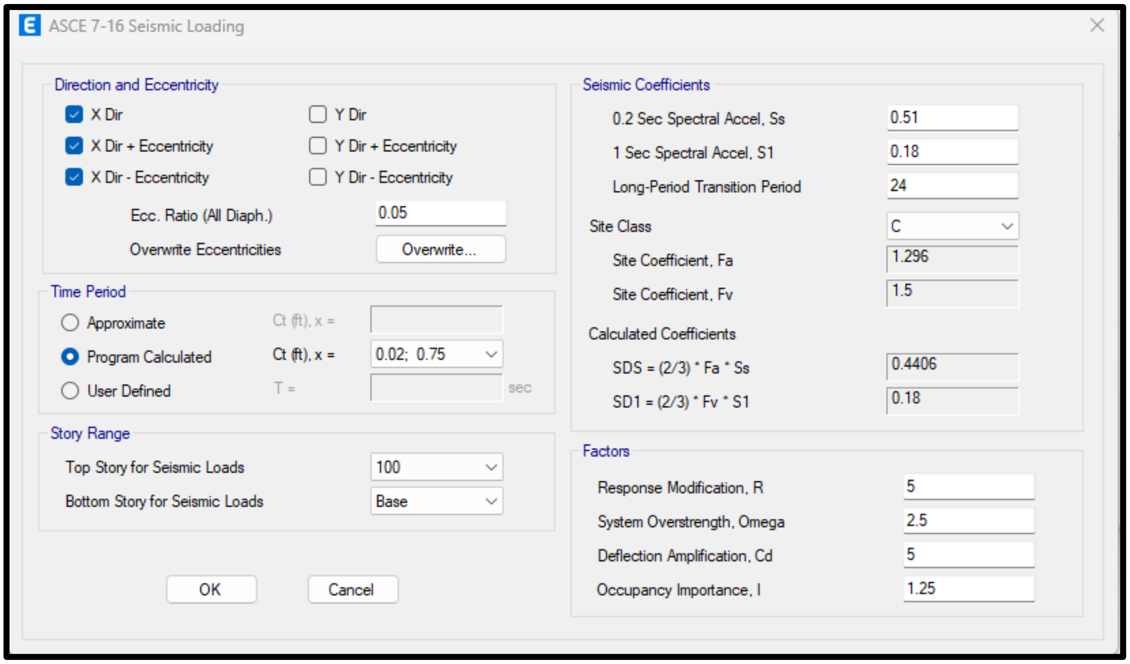


Figure 2.2.5.5 – ELF X Direction Load Pattern Definition

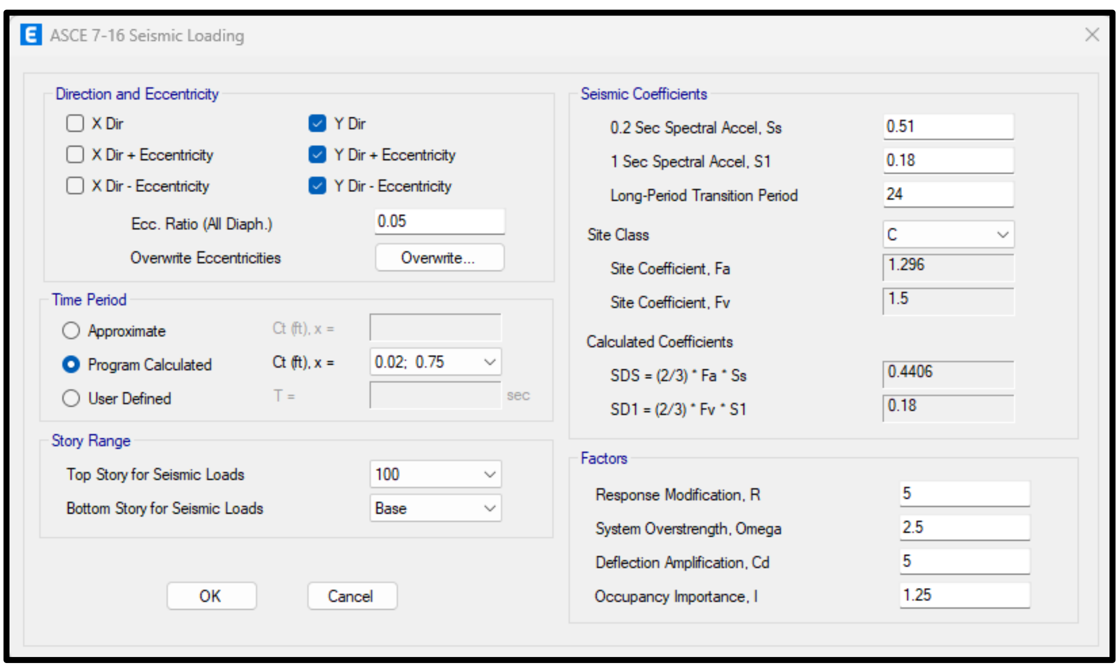


Figure 2.2.5.6 – ELF Y Direction Load Pattern Definition

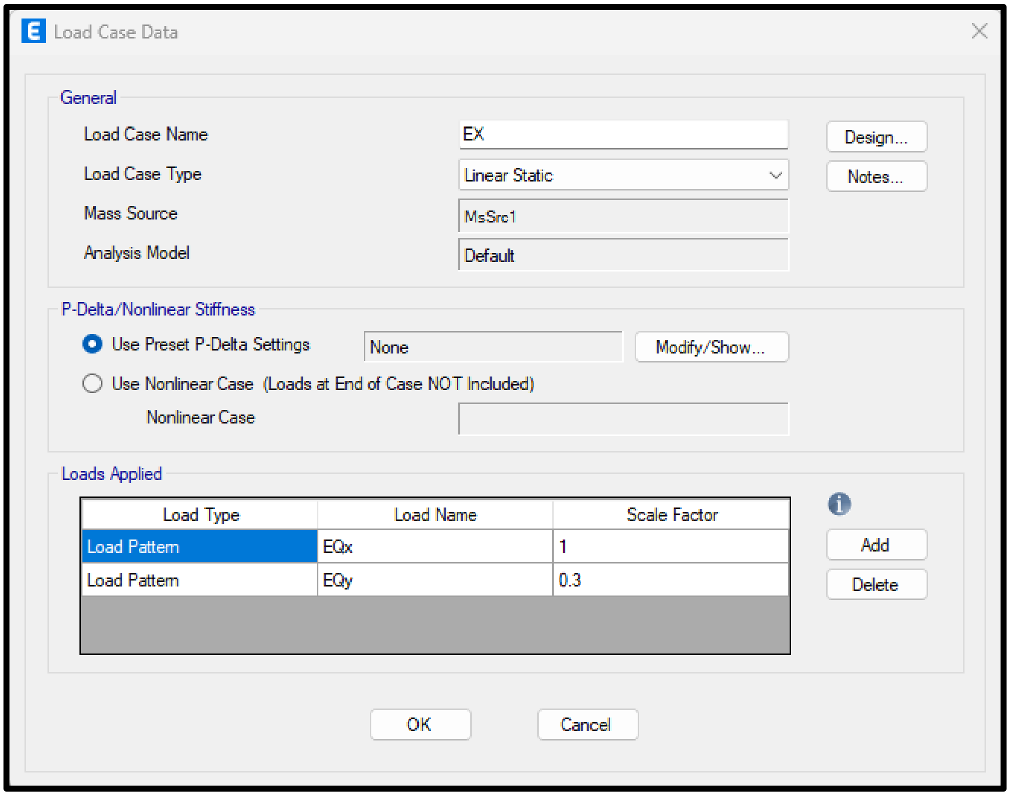


Figure 2.2.5.7 – ELF X Direction Load Case Definition

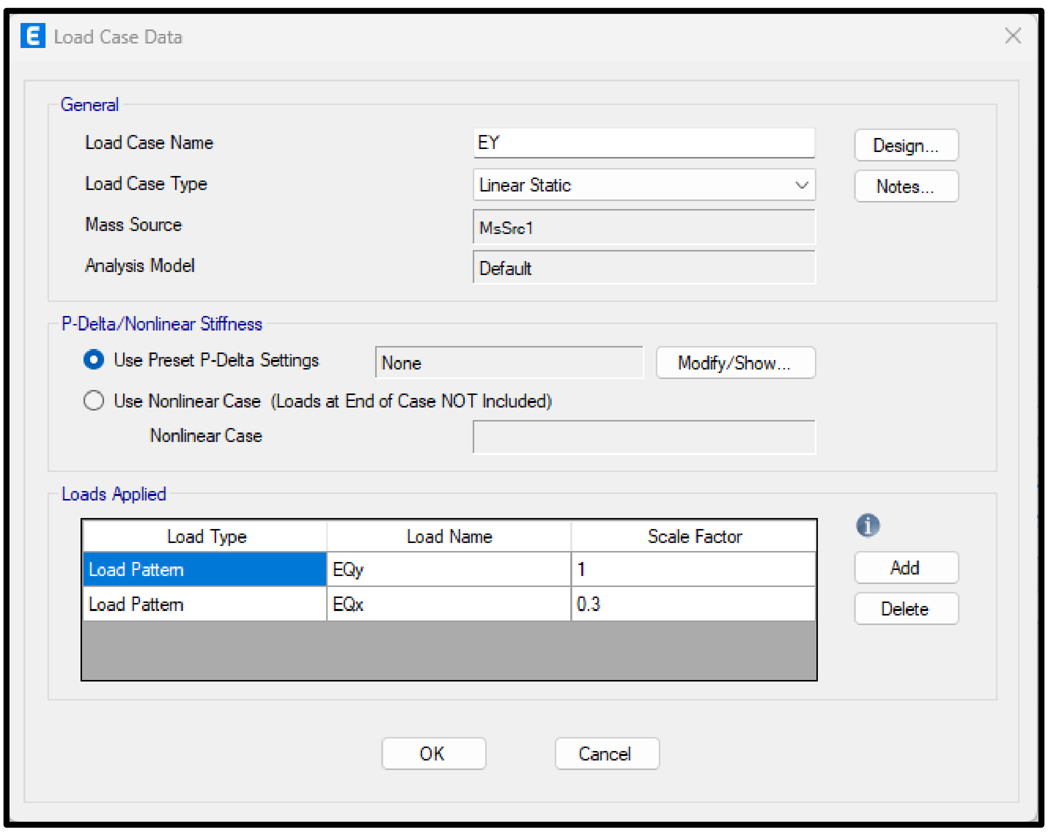


Figure 2.2.5.8 – ELF Y Direction Load Case Definition

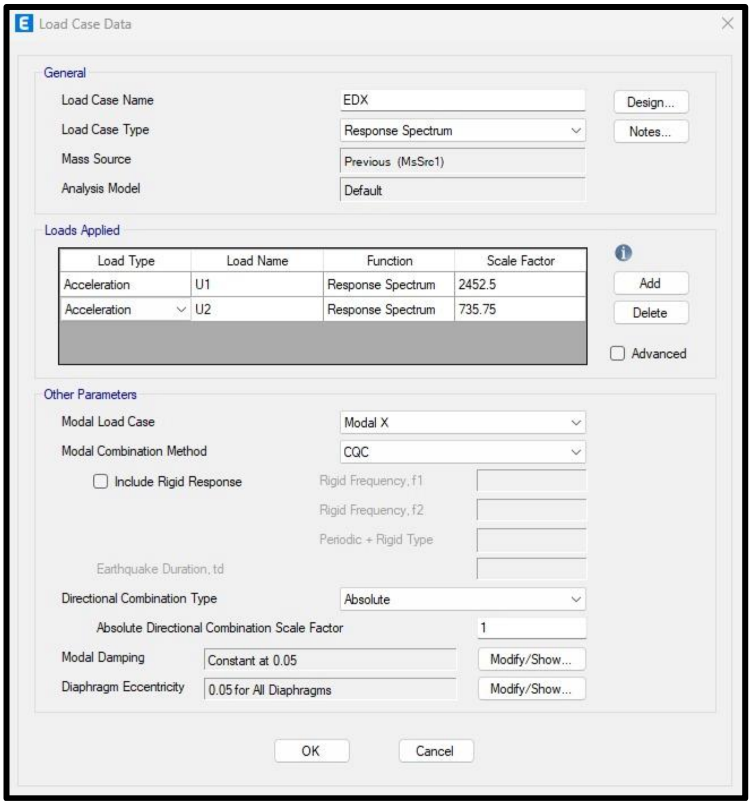


Figure 2.2.5.9 – Response Spectrum X Direction Load Case Before Upscaling Factor

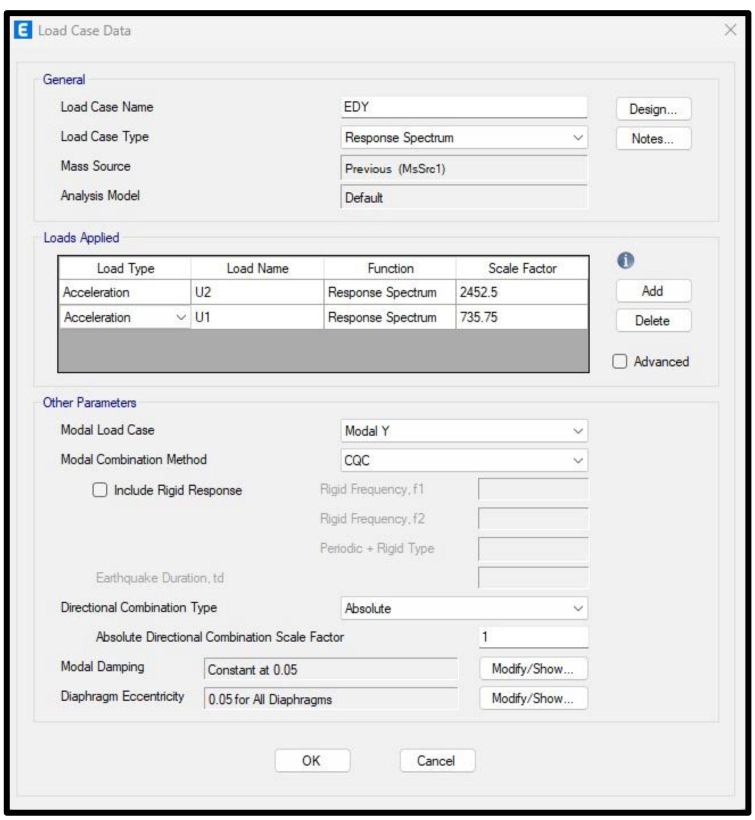


Figure 2.2.5.10 – Response Spectrum Y Direction Load Case Before Upscaling Factor

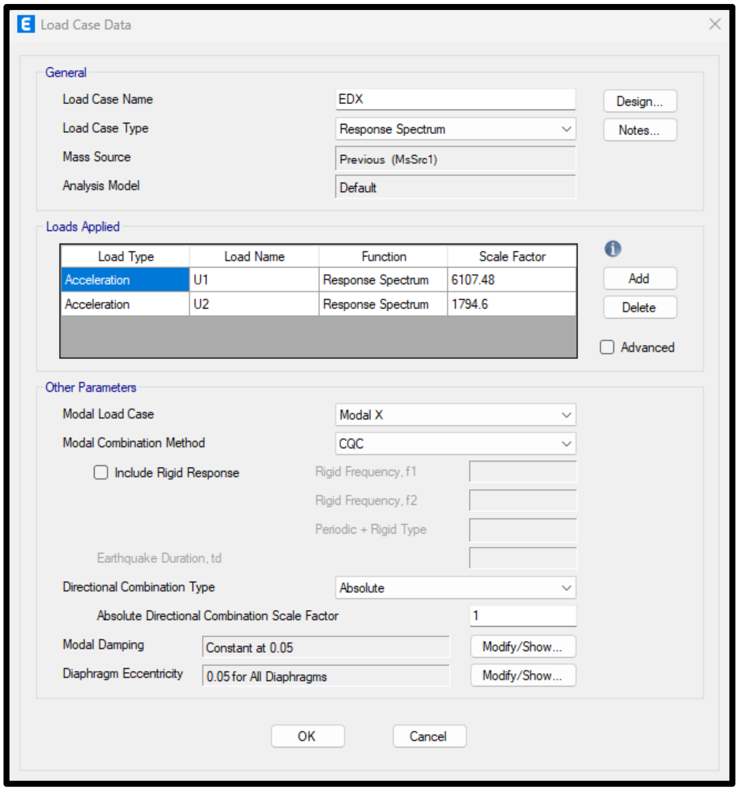


Figure 2.2.5.11 – Response Spectrum X Direction Load Case After Upscaling Factor

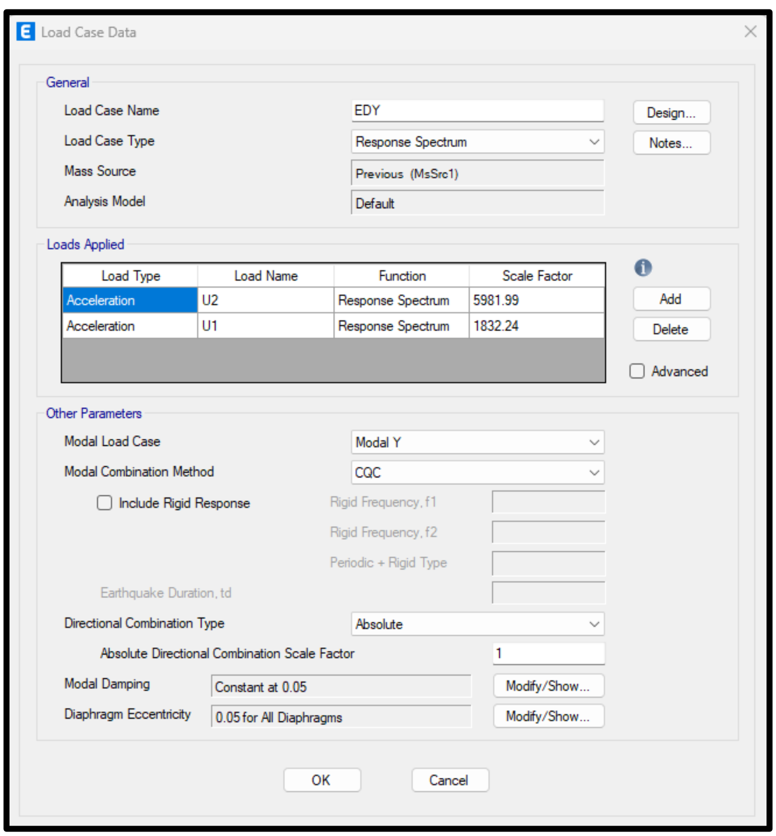


Figure 2.2.5.12 – Response Spectrum Y Direction Load Case After Upscaling Factor

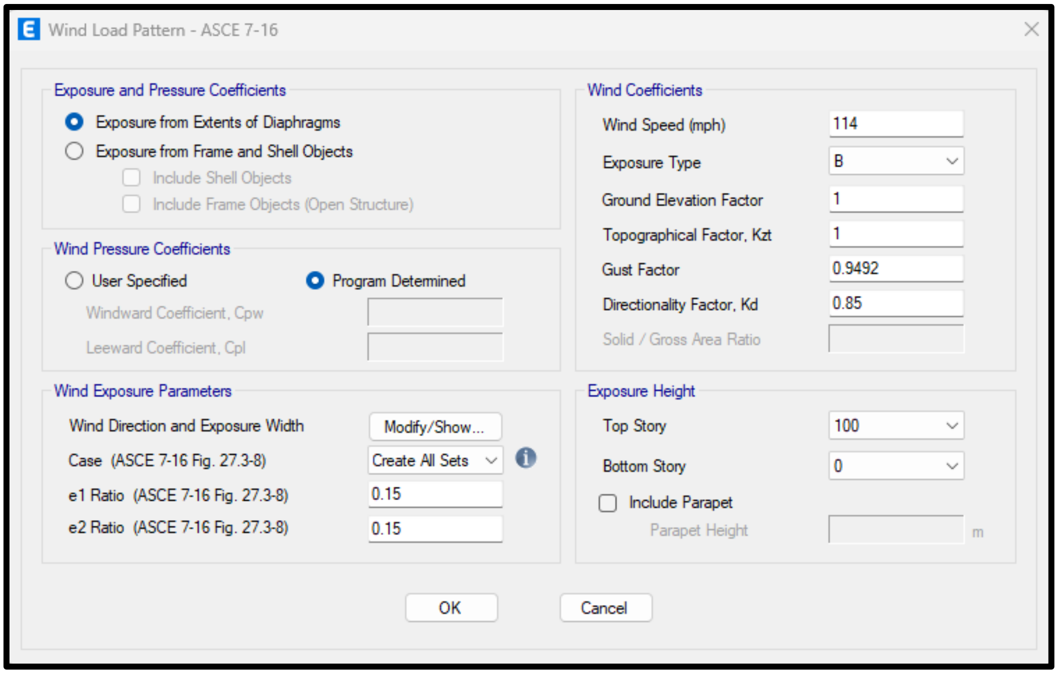


Figure 2.2.5.13 – Wind X Direction Load Pattern

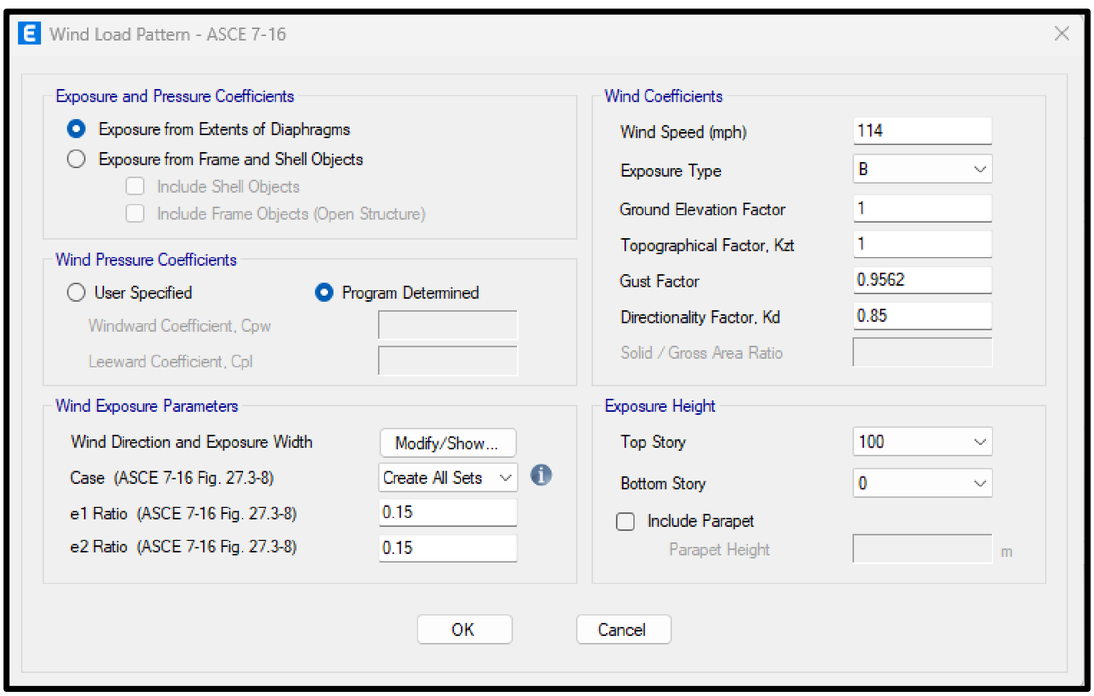


Figure 2.2.5.14 – Wind Y Direction Load Pattern

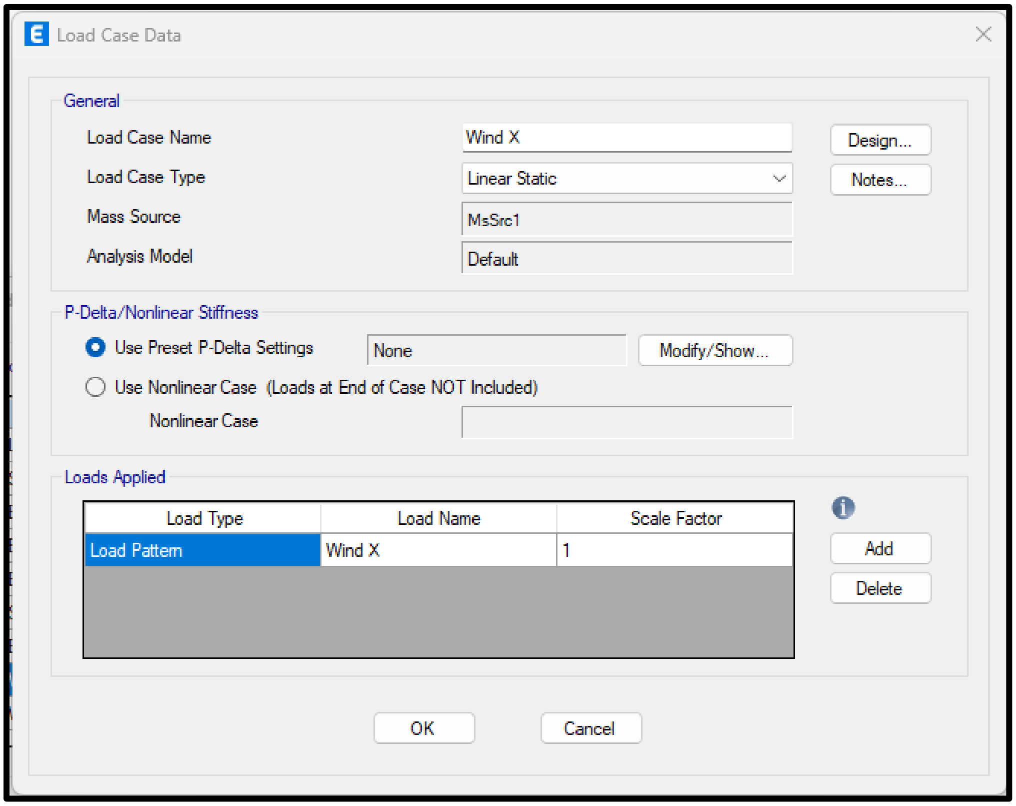


Figure 2.2.5.15 – Wind X Direction Load Case

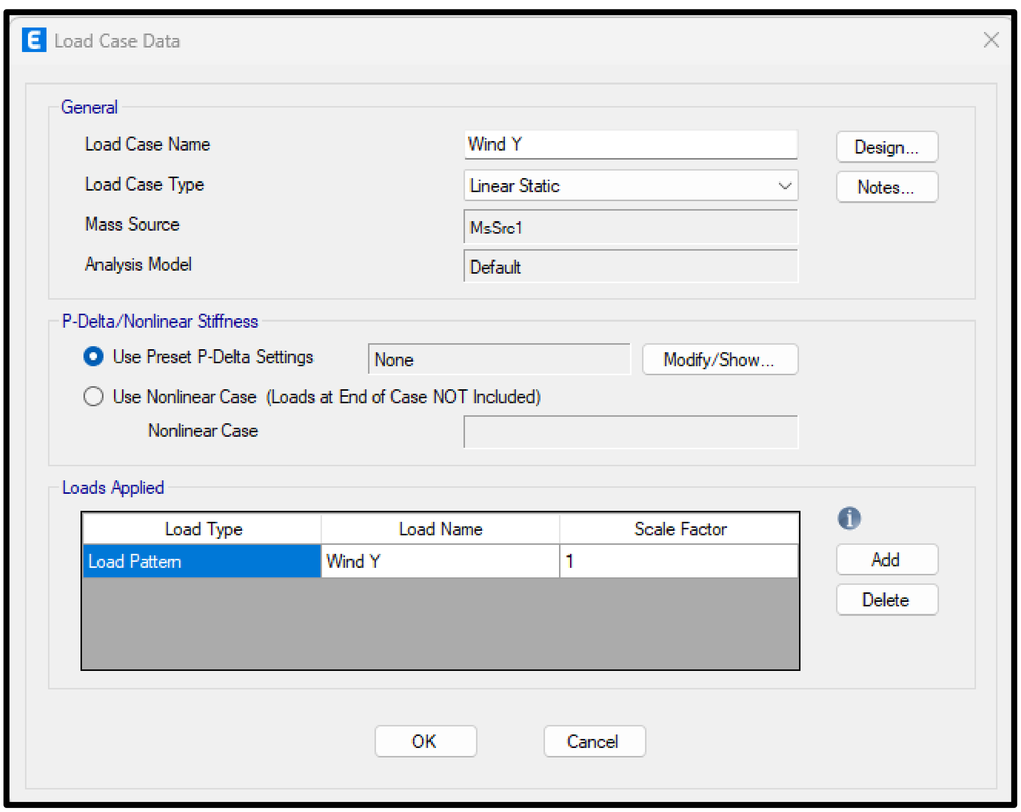


Figure 2.2.5.16 – Wind Y Direction Load Case

2.3.6 Irregularities Checks

Structural irregularities are defined in the code to restrict engineers from designing structures with complicated behavior or conduct them toward using more advanced analysis techniques. Generally regular buildings perform better in earthquakes than do irregular buildings.
There are two types of structural irregularities: horizontal and vertical. Horizontal structural irregularities are primarily of significance when the structure under consideration is assigned to Seismic Design Category (SDC) D or above, or in some cases, SDC C.

Types of irregularities that can exist in the structures:

- Vertical irregularity: This occurs when the stiffness or strength of the building is not uniform along its height.
- Horizontal irregularity: This occurs when the stiffness or strength of the building is not uniform along its length or width.

In a high-rise building, the types of irregularities that can exist are like those in other buildings. These include vertical irregularity, horizontal irregularity, mass irregularity, and torsional irregularity. However, the effects of these irregularities can be more pronounced in a tower due to its height and slender shape. For example, torsional irregularity can cause the tower to twist and sway in the wind. Vertical and horizontal irregularities can cause the tower to vibrate or oscillate.

- **In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity**

In-plane discontinuity in vertical lateral force-resisting element irregularity is defined to exist where there is an in-plane offset of a vertical seismic force-resisting element resulting in overturning demands on supporting structural elements.

In the project, there is an in-plane offset of the seismic force resisting elements resulting in overturning demands on the supporting columns and shear walls.

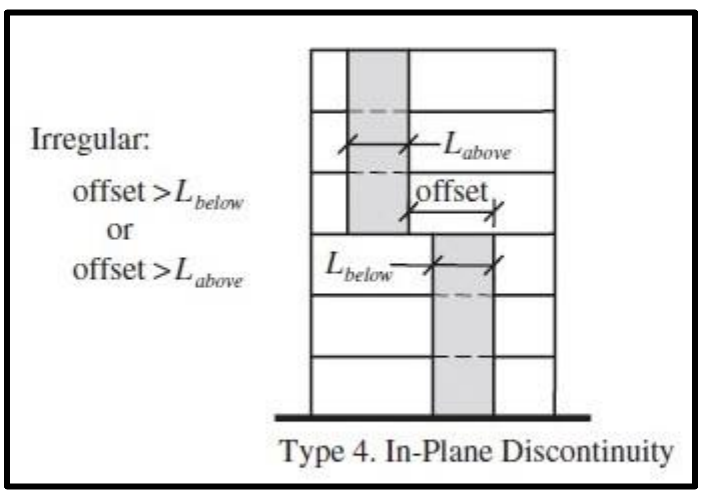


Figure 2.3.6.1 – In-Plane Discontinuity Irregularity Demonstration

Horizontal Structural Irregularities

- **Out of Plane Offset Irregularity**

Out of-plane offset irregularity is defined to exist where there is a discontinuity in a lateral force resistance path, such as an out-of-plane offset of at least one of the vertical elements.

The shear wall is along the entire height of the tower without any discontinuity in any story.

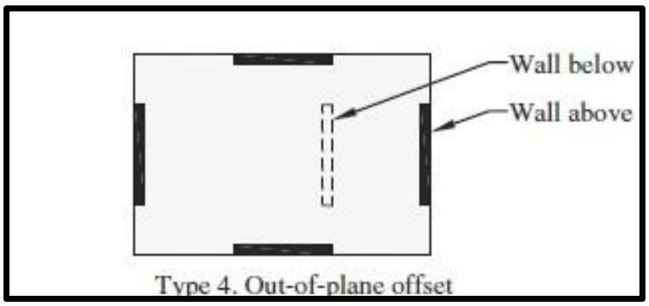


Figure 2.3.6.2 – Out-of-Plane Offset Irregularity Demonstration

- **Nonparallel System Irregularity**

Nonparallel system irregularity is defined to exist where vertical lateral force resisting elements are not parallel to the major orthogonal axes of the seismic force resisting system.

The shear walls are orthogonal on the seismic force line, so we do not have irregularity here.

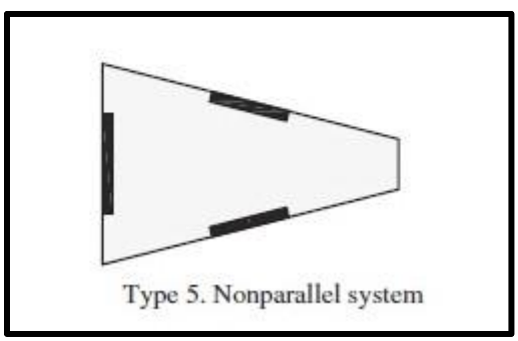


Figure 2.3.6.3 – Nonparallel System Irregularity Demonstration

- **Torsional Irregularity**

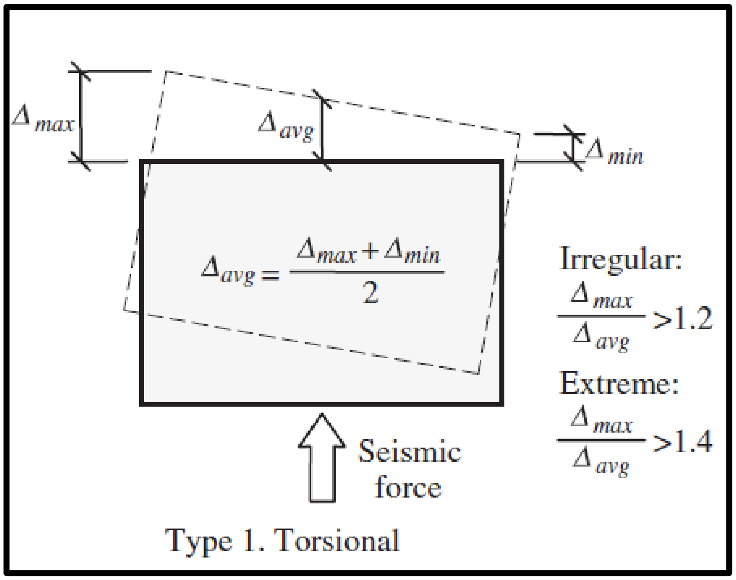


Figure 2.3.6.4 – Torsional Irregularity Demonstration

The Diaphragm max over average drifts values are obtained as follows

Story	Output Case	Max Drift	Avg Drift	Ratio	Check	Output Case	Max Drift	Avg Drift	Ratio	Check
100th Floor	EDX + Soil	0.002655	0.002014	1.319	Needs to be checked	EDY + Soil	0.002858	0.002172	1.316	Needs to be checked
99th Floor	EDX + Soil	0.002654	0.002017	1.315	Needs to be checked	EDY + Soil	0.002856	0.002175	1.313	Needs to be checked
98th Floor	EDX + Soil	0.001433	0.001425	1.006	No need to check	EDY + Soil	0.00154	0.001532	1.005	No need to check
97th Floor	EDX + Soil	0.001435	0.001428	1.005	No need to check	EDY + Soil	0.001529	0.001524	1.003	No need to check
96th Floor	EDX + Soil	0.001377	0.001373	1.004	No need to check	EDY + Soil	0.001481	0.001479	1.001	No need to check
95th Floor	EDX + Soil	0.001446	0.00144	1.004	No need to check	EDY + Soil	0.001544	0.001538	1.004	No need to check
94th Floor	EDX + Soil	0.001472	0.001465	1.005	No need to check	EDY + Soil	0.001575	0.001566	1.006	No need to check
93rd Floor	EDX + Soil	0.001487	0.001478	1.006	No need to check	EDY + Soil	0.001593	0.001582	1.006	No need to check
92nd Floor	EDX + Soil	0.001499	0.001489	1.007	No need to check	EDY + Soil	0.001605	0.001594	1.007	No need to check
91st Floor	EDX + Soil	0.001508	0.001497	1.007	No need to check	EDY + Soil	0.001614	0.001603	1.007	No need to check
90th Floor	EDX + Soil	0.001515	0.001504	1.007	No need to check	EDY + Soil	0.001623	0.001611	1.008	No need to check
89th Floor	EDX + Soil	0.00152	0.001509	1.007	No need to check	EDY + Soil	0.001631	0.001618	1.008	No need to check
88th Floor	EDX + Soil	0.001524	0.001512	1.008	No need to check	EDY + Soil	0.001635	0.001622	1.008	No need to check

87th Floor	EDX + Soil	0.001525	0.001514	1.008	No need to check	EDY + Soil	0.001637	0.001624	1.008	No need to check
86th Floor	EDX + Soil	0.001525	0.001513	1.008	No need to check	EDY + Soil	0.001636	0.001623	1.008	No need to check
85th Floor	EDX + Soil	0.001522	0.00151	1.008	No need to check	EDY + Soil	0.001631	0.001618	1.008	No need to check
84th Floor	EDX + Soil	0.001517	0.001504	1.008	No need to check	EDY + Soil	0.001622	0.001608	1.008	No need to check
83rd Floor	EDX + Soil	0.001508	0.001495	1.008	No need to check	EDY + Soil	0.001605	0.001592	1.008	No need to check
82nd Floor	EDX + Soil	0.001493	0.00148	1.009	No need to check	EDY + Soil	0.001578	0.001566	1.008	No need to check
81st Floor	EDX + Soil	0.001454	0.001443	1.008	No need to check	EDY + Soil	0.001526	0.001516	1.007	No need to check
80th Floor	EDX + Soil	0.001485	0.001384	1.073	No need to check	EDY + Soil	0.001543	0.001449	1.065	No need to check
79th Floor	EDX + Soil	0.001616	0.001449	1.115	No need to check	EDY + Soil	0.001782	0.001573	1.133	No need to check
78th Floor	EDX + Soil	0.001479	0.001465	1.01	No need to check	EDY + Soil	0.001555	0.001542	1.009	No need to check
77th Floor	EDX + Soil	0.001567	0.001548	1.012	No need to check	EDY + Soil	0.00166	0.001641	1.012	No need to check
76th Floor	EDX + Soil	0.001616	0.001595	1.013	No need to check	EDY + Soil	0.001724	0.001701	1.013	No need to check
75th Floor	EDX + Soil	0.001652	0.00163	1.014	No need to check	EDY + Soil	0.001772	0.001748	1.014	No need to check
74th Floor	EDX + Soil	0.001682	0.001659	1.014	No need to check	EDY + Soil	0.001809	0.001784	1.014	No need to check
73rd Floor	EDX + Soil	0.001706	0.001683	1.014	No need to check	EDY + Soil	0.001838	0.001812	1.014	No need to check
72nd Floor	EDX + Soil	0.001725	0.001702	1.013	No need to check	EDY + Soil	0.00186	0.001835	1.014	No need to check
71st Floor	EDX + Soil	0.00174	0.001718	1.013	No need to check	EDY + Soil	0.001877	0.001852	1.014	No need to check
70th Floor	EDX + Soil	0.001752	0.00173	1.013	No need to check	EDY + Soil	0.001889	0.001865	1.013	No need to check
69th Floor	EDX + Soil	0.001761	0.001739	1.013	No need to check	EDY + Soil	0.001897	0.001874	1.012	No need to check
68th Floor	EDX + Soil	0.001767	0.001745	1.013	No need to check	EDY + Soil	0.001901	0.00188	1.011	No need to check
67th Floor	EDX + Soil	0.00177	0.001748	1.013	No need to check	EDY + Soil	0.001902	0.001882	1.011	No need to check
66th Floor	EDX + Soil	0.001771	0.001748	1.013	No need to check	EDY + Soil	0.001901	0.001882	1.01	No need to check
65th Floor	EDX + Soil	0.001769	0.001746	1.013	No need to check	EDY + Soil	0.001898	0.001879	1.01	No need to check
64th Floor	EDX + Soil	0.001765	0.001741	1.014	No need to check	EDY + Soil	0.001893	0.001873	1.011	No need to check
63rd Floor	EDX + Soil	0.001758	0.001733	1.014	No need to check	EDY + Soil	0.001886	0.001865	1.011	No need to check
62nd Floor	EDX + Soil	0.001749	0.001723	1.015	No need to check	EDY + Soil	0.001876	0.001854	1.012	No need to check
61st Floor	EDX + Soil	0.001736	0.001709	1.015	No need to check	EDY + Soil	0.001862	0.001838	1.013	No need to check
60th Floor	EDX + Soil	0.001719	0.001692	1.016	No need to check	EDY + Soil	0.001844	0.001818	1.014	No need to check
59th Floor	EDX + Soil	0.001699	0.001671	1.017	No need to check	EDY + Soil	0.001819	0.001792	1.015	No need to check
58th Floor	EDX + Soil	0.001674	0.001645	1.017	No need to check	EDY + Soil	0.001787	0.001759	1.016	No need to check
57th Floor	EDX + Soil	0.001643	0.001614	1.018	No need to check	EDY + Soil	0.001745	0.001718	1.016	No need to check
56th Floor	EDX + Soil	0.001604	0.001575	1.019	No need to check	EDY + Soil	0.00169	0.001663	1.016	No need to check
55th Floor	EDX + Soil	0.001552	0.001523	1.019	No need to check	EDY + Soil	0.001615	0.001589	1.016	No need to check
54th Floor	EDX + Soil	0.001447	0.001421	1.019	No need to check	EDY + Soil	0.001482	0.001459	1.016	No need to check

53rd Floor	EDX + Soil	0.001193	0.001173	1.017	No need to check	EDY + Soil	0.001227	0.001209	1.015	No need to check
52nd Floor	EDX + Soil	0.001404	0.001273	1.103	No need to check	EDY + Soil	0.001507	0.001345	1.12	No need to check
51st Floor	EDX + Soil	0.001435	0.001408	1.019	No need to check	EDY + Soil	0.001464	0.001444	1.014	No need to check
50th Floor	EDX + Soil	0.001537	0.001507	1.02	No need to check	EDY + Soil	0.00159	0.001568	1.014	No need to check
49th Floor	EDX + Soil	0.001586	0.001555	1.02	No need to check	EDY + Soil	0.00166	0.001636	1.014	No need to check
48th Floor	EDX + Soil	0.00162	0.001588	1.02	No need to check	EDY + Soil	0.00171	0.001686	1.015	No need to check
47th Floor	EDX + Soil	0.001645	0.001614	1.019	No need to check	EDY + Soil	0.001748	0.001723	1.015	No need to check
46th Floor	EDX + Soil	0.001664	0.001633	1.019	No need to check	EDY + Soil	0.001777	0.00175	1.015	No need to check
45th Floor	EDX + Soil	0.001679	0.001648	1.018	No need to check	EDY + Soil	0.001798	0.001771	1.015	No need to check
44th Floor	EDX + Soil	0.001688	0.001659	1.018	No need to check	EDY + Soil	0.001812	0.001786	1.015	No need to check
43rd Floor	EDX + Soil	0.001695	0.001665	1.018	No need to check	EDY + Soil	0.001822	0.001796	1.015	No need to check
42nd Floor	EDX + Soil	0.001697	0.001668	1.017	No need to check	EDY + Soil	0.001827	0.0018	1.015	No need to check
41st Floor	EDX + Soil	0.001697	0.001668	1.017	No need to check	EDY + Soil	0.001827	0.001801	1.015	No need to check
40th Floor	EDX + Soil	0.001694	0.001665	1.018	No need to check	EDY + Soil	0.001823	0.001797	1.014	No need to check
39th Floor	EDX + Soil	0.001689	0.001659	1.018	No need to check	EDY + Soil	0.001814	0.001789	1.014	No need to check
38th Floor	EDX + Soil	0.00168	0.001649	1.019	No need to check	EDY + Soil	0.001802	0.001777	1.014	No need to check
37th Floor	EDX + Soil	0.001667	0.001636	1.019	No need to check	EDY + Soil	0.001786	0.001761	1.014	No need to check
36th Floor	EDX + Soil	0.001651	0.001619	1.02	No need to check	EDY + Soil	0.001766	0.001741	1.014	No need to check
35th Floor	EDX + Soil	0.001632	0.001598	1.021	No need to check	EDY + Soil	0.001743	0.001718	1.015	No need to check
34th Floor	EDX + Soil	0.001608	0.001573	1.022	No need to check	EDY + Soil	0.001715	0.00169	1.015	No need to check
33rd Floor	EDX + Soil	0.00158	0.001545	1.023	No need to check	EDY + Soil	0.001682	0.001657	1.016	No need to check
32nd Floor	EDX + Soil	0.001547	0.001511	1.023	No need to check	EDY + Soil	0.001644	0.001618	1.016	No need to check
31st Floor	EDX + Soil	0.001508	0.001473	1.024	No need to check	EDY + Soil	0.0016	0.001572	1.017	No need to check
30th Floor	EDX + Soil	0.001464	0.001428	1.025	No need to check	EDY + Soil	0.001547	0.001518	1.019	No need to check
29th Floor	EDX + Soil	0.001413	0.001377	1.026	No need to check	EDY + Soil	0.001482	0.001453	1.02	No need to check
28th Floor	EDX + Soil	0.001351	0.001315	1.028	No need to check	EDY + Soil	0.001402	0.001373	1.021	No need to check
27th Floor	EDX + Soil	0.001273	0.001237	1.029	No need to check	EDY + Soil	0.001297	0.001268	1.023	No need to check
26th Floor	EDX + Soil	0.001129	0.001097	1.03	No need to check	EDY + Soil	0.001122	0.001096	1.024	No need to check
25th Floor	EDX + Soil	0.000813	0.000789	1.032	No need to check	EDY + Soil	0.000812	0.000791	1.027	No need to check
24th Floor	EDX + Soil	0.000894	0.00082	1.091	No need to check	EDY + Soil	0.000941	0.000845	1.113	No need to check
23rd Floor	EDX + Soil	0.001089	0.001055	1.032	No need to check	EDY + Soil	0.001075	0.001048	1.026	No need to check
22nd Floor	EDX + Soil	0.001203	0.001164	1.034	No need to check	EDY + Soil	0.001215	0.001185	1.025	No need to check
21st Floor	EDX + Soil	0.001251	0.001209	1.035	No need to check	EDY + Soil	0.001286	0.001254	1.025	No need to check
20th Floor	EDX + Soil	0.00128	0.001237	1.035	No need to check	EDY + Soil	0.001332	0.001299	1.025	No need to check

19th Floor	EDX + Soil	0.001297	0.001253	1.035	No need to check	EDY + Soil	0.001361	0.001328	1.025	No need to check
18th Floor	EDX + Soil	0.001305	0.001261	1.035	No need to check	EDY + Soil	0.001378	0.001345	1.025	No need to check
17th Floor	EDX + Soil	0.001305	0.001261	1.035	No need to check	EDY + Soil	0.001384	0.001351	1.025	No need to check
16th Floor	EDX + Soil	0.001298	0.001255	1.034	No need to check	EDY + Soil	0.001382	0.001349	1.025	No need to check
15th Floor	EDX + Soil	0.001285	0.001242	1.034	No need to check	EDY + Soil	0.001373	0.00134	1.025	No need to check
14th Floor	EDX + Soil	0.001266	0.001224	1.034	No need to check	EDY + Soil	0.001358	0.001325	1.025	No need to check
13th Floor	EDX + Soil	0.001241	0.001201	1.034	No need to check	EDY + Soil	0.001338	0.001304	1.026	No need to check
12th Floor	EDX + Soil	0.001212	0.001171	1.034	No need to check	EDY + Soil	0.001312	0.001278	1.027	No need to check
11th Floor	EDX + Soil	0.001177	0.001137	1.035	No need to check	EDY + Soil	0.001279	0.001244	1.029	No need to check
10th Floor	EDX + Soil	0.001138	0.001097	1.037	No need to check	EDY + Soil	0.001239	0.001203	1.03	No need to check
9th Floor	EDX + Soil	0.001094	0.001051	1.04	No need to check	EDY + Soil	0.001193	0.001156	1.032	No need to check
8th Floor	EDX + Soil	0.001044	0.001	1.044	No need to check	EDY + Soil	0.001142	0.001106	1.033	No need to check
7th Floor	EDX + Soil	0.000958	0.000915	1.047	No need to check	EDY + Soil	0.001081	0.001039	1.041	No need to check
6th Floor	EDX + Soil	0.000793	0.000755	1.049	No need to check	EDY + Soil	0.000884	0.000836	1.057	No need to check
5th Parking Floor	EDX + Soil	0.000661	0.000601	1.1	No need to check	EDY + Soil	0.00072	0.000606	1.188	No need to check
4th Parking Floor	EDX + Soil	0.000617	0.000563	1.095	No need to check	EDY + Soil	0.000682	0.000563	1.212	Needs to be checked
3rd Parking Floor	EDX + Soil	0.000569	0.000521	1.091	No need to check	EDY + Soil	0.000629	0.000513	1.228	Needs to be checked
2nd Parking Floor	EDX + Soil	0.000508	0.000466	1.089	No need to check	EDY + Soil	0.000551	0.000442	1.246	Needs to be checked
1st Parking Floor	EDX + Soil	0.000395	0.000363	1.088	No need to check	EDY + Soil	0.000402	0.000318	1.265	Needs to be checked
Ground Floor	EDX + Soil	0.00016	0.000149	1.072	No need to check	EDY + Soil	0.000113	0.000092	1.224	Needs to be checked
1st Basement Floor	EDX + Soil	0.000125	0.000117	1.067	No need to check	EDY + Soil	0.000081	0.000069	1.189	No need to check
2nd Basement Floor	EDX + Soil	0.000108	0.000102	1.056	No need to check	EDY + Soil	0.000074	0.000063	1.175	No need to check
3rd Basement Floor	EDX + Soil	0.000094	0.00009	1.048	No need to check	EDY + Soil	0.000069	0.000059	1.169	No need to check
4th Basement Floor	EDX + Soil	0.000079	0.000076	1.047	No need to check	EDY + Soil	0.000063	0.000054	1.17	No need to check
5th Basement Floor	EDX + Soil	0.000061	0.000058	1.053	No need to check	EDY + Soil	0.000052	0.000045	1.17	No need to check
6th Basement Floor	EDX + Soil	0.000033	0.000031	1.061	No need to check	EDY + Soil	0.000034	0.000031	1.124	No need to check

Table 2.3.6.1 – Stories Diaphragm Max over Average Drift Ratios Before Modification

As noted from the table above, certain stories have max over average drift ratios larger than 1.2, so According to ASCE 7-16 it is needed to obtain the Torsional Amplification Factor (Ax) in accordance with section 12.8.4.3 for these stories, and this is shown in the table below which shows the story max over average displacements to calculate (Ax).

Story	Output Case	Maximum	Average	Ratio	Torsional Amplification Factor (Ax)	Output Case	Maximum	Average	Ratio	Torsional Amplification Factor (Ax)
100th Floor	EDX + Soil	504.393	497.184	1.014	-	EDY + Soil	540.761	533.994	1.013	-
99th Floor	EDX + Soil	496.013	488.825	1.015	-	EDY + Soil	531.648	524.89	1.013	-
5th Parking Floor	EDX + Soil	10.391	9.556	1.087	-	EDY + Soil	10.481	8.6	1.219	1.031917
4th Parking Floor	EDX + Soil	8.056	7.437	1.083	-	EDY + Soil	7.951	6.47	1.229	1.048917
3rd Parking Floor	EDX + Soil	6.355	5.886	1.08	-	EDY + Soil	6.072	4.922	1.234	1.057469
2nd Parking Floor	EDX + Soil	4.79	4.453	1.076	-	EDY + Soil	4.344	3.518	1.235	1.059184
1st Parking Floor	EDX + Soil	3.399	3.176	1.07	-	EDY + Soil	2.844	2.317	1.227	1.045506
Ground Floor	EDX + Soil	1.948	1.839	1.059	-	EDY + Soil	1.401	1.178	1.189	-

Table 2.3.6.2 – Stories Max over Average Displacement Ratios Before Modification

As noted from the table, a Torsional Amplification Factor is needed in the Y-Direction, so the eccentricity is multiplied by this factor.

$$A_x = \left(\frac{\delta_{max}}{1.2 * \delta_{average}}\right)^2 = \left(\frac{1.235}{1.2}\right)^2 = 1.059184$$

So, the new eccentricity value E = 0.05*1.059184 = 0.0529592

The Diaphragm max over average drifts values after modifications are obtained as follows

Story	Output Case	Max Drift	Avg Drift	Ratio	Check	Output Case	Max Drift	Avg Drift	Ratio	Check
100th Floor	EDX + Soil	0.002515	0.001805	1.393	Needs to be Checked	EDY + Soil	0.002798	0.002027	1.38	Needs to be Checked
99th Floor	EDX + Soil	0.002514	0.001811	1.388	Needs to be Checked	EDY + Soil	0.002796	0.00203	1.377	Needs to be Checked
98th Floor	EDX + Soil	0.001173	0.001163	1.009	No Need to be Checked	EDY + Soil	0.001309	0.001299	1.007	No Need to be Checked
97th Floor	EDX + Soil	0.001171	0.001163	1.007	No Need to be Checked	EDY + Soil	0.001307	0.001299	1.006	No Need to be Checked

96th Floor	EDX + Soil	0.001116	0.00111	1.006	No Need to be Checked	EDY + Soil	0.001262	0.001258	1.003	No Need to be Checked
95th Floor	EDX + Soil	0.001176	0.001168	1.007	No Need to be Checked	EDY + Soil	0.001318	0.001312	1.005	No Need to be Checked
94th Floor	EDX + Soil	0.001202	0.001193	1.008	No Need to be Checked	EDY + Soil	0.001344	0.001336	1.006	No Need to be Checked
93rd Floor	EDX + Soil	0.001218	0.001209	1.008	No Need to be Checked	EDY + Soil	0.001362	0.001353	1.007	No Need to be Checked
92nd Floor	EDX + Soil	0.001232	0.001221	1.009	No Need to be Checked	EDY + Soil	0.001375	0.001365	1.007	No Need to be Checked
91st Floor	EDX + Soil	0.001244	0.001232	1.01	No Need to be Checked	EDY + Soil	0.001386	0.001375	1.008	No Need to be Checked
90th Floor	EDX + Soil	0.001254	0.001241	1.01	No Need to be Checked	EDY + Soil	0.001394	0.001383	1.008	No Need to be Checked
89th Floor	EDX + Soil	0.00126	0.001247	1.011	No Need to be Checked	EDY + Soil	0.001399	0.001388	1.008	No Need to be Checked
88th Floor	EDX + Soil	0.001264	0.001251	1.011	No Need to be Checked	EDY + Soil	0.001402	0.00139	1.009	No Need to be Checked
87th Floor	EDX + Soil	0.001265	0.001252	1.011	No Need to be Checked	EDY + Soil	0.001401	0.001389	1.009	No Need to be Checked
86th Floor	EDX + Soil	0.001264	0.00125	1.011	No Need to be Checked	EDY + Soil	0.001397	0.001385	1.009	No Need to be Checked
85th Floor	EDX + Soil	0.001259	0.001245	1.011	No Need to be Checked	EDY + Soil	0.00139	0.001377	1.009	No Need to be Checked
84th Floor	EDX + Soil	0.00125	0.001236	1.011	No Need to be Checked	EDY + Soil	0.001378	0.001365	1.009	No Need to be Checked
83rd Floor	EDX + Soil	0.001235	0.001223	1.01	No Need to be Checked	EDY + Soil	0.00136	0.001348	1.009	No Need to be Checked
82nd Floor	EDX + Soil	0.001214	0.001202	1.01	No Need to be Checked	EDY + Soil	0.001334	0.001322	1.009	No Need to be Checked
81st Floor	EDX + Soil	0.001173	0.001162	1.009	No Need to be Checked	EDY + Soil	0.00129	0.001278	1.009	No Need to be Checked
80th Floor	EDX + Soil	0.001233	0.001123	1.098	No Need to be Checked	EDY + Soil	0.001317	0.001215	1.084	No Need to be Checked
79th Floor	EDX + Soil	0.001354	0.00119	1.138	No Need to be Checked	EDY + Soil	0.001594	0.001362	1.17	No Need to be Checked
78th Floor	EDX + Soil	0.001174	0.001163	1.009	No Need to be Checked	EDY + Soil	0.001272	0.001262	1.008	No Need to be Checked
77th Floor	EDX + Soil	0.001245	0.001231	1.012	No Need to be Checked	EDY + Soil	0.001352	0.001339	1.01	No Need to be Checked
76th Floor	EDX + Soil	0.001294	0.001277	1.013	No Need to be Checked	EDY + Soil	0.001403	0.001387	1.011	No Need to be Checked
75th Floor	EDX + Soil	0.001332	0.001313	1.015	No Need to be Checked	EDY + Soil	0.001442	0.001424	1.012	No Need to be Checked
74th Floor	EDX + Soil	0.001362	0.001341	1.015	No Need to be Checked	EDY + Soil	0.001473	0.001454	1.013	No Need to be Checked

73rd Floor	EDX + Soil	0.001385	0.001364	1.016	No Need to be Checked	EDY + Soil	0.001498	0.001478	1.014	No Need to be Checked
72nd Floor	EDX + Soil	0.001404	0.001382	1.016	No Need to be Checked	EDY + Soil	0.001518	0.001497	1.014	No Need to be Checked
71st Floor	EDX + Soil	0.001418	0.001396	1.016	No Need to be Checked	EDY + Soil	0.001534	0.001512	1.015	No Need to be Checked
70th Floor	EDX + Soil	0.001428	0.001407	1.016	No Need to be Checked	EDY + Soil	0.001547	0.001524	1.015	No Need to be Checked
69th Floor	EDX + Soil	0.001436	0.001414	1.015	No Need to be Checked	EDY + Soil	0.001555	0.001532	1.015	No Need to be Checked
68th Floor	EDX + Soil	0.001441	0.00142	1.015	No Need to be Checked	EDY + Soil	0.001561	0.001538	1.015	No Need to be Checked
67th Floor	EDX + Soil	0.001445	0.001423	1.015	No Need to be Checked	EDY + Soil	0.001564	0.00154	1.015	No Need to be Checked
66th Floor	EDX + Soil	0.001447	0.001424	1.016	No Need to be Checked	EDY + Soil	0.001564	0.00154	1.016	No Need to be Checked
65th Floor	EDX + Soil	0.001447	0.001424	1.016	No Need to be Checked	EDY + Soil	0.001561	0.001537	1.016	No Need to be Checked
64th Floor	EDX + Soil	0.001444	0.00142	1.017	No Need to be Checked	EDY + Soil	0.001556	0.001531	1.016	No Need to be Checked
63rd Floor	EDX + Soil	0.00144	0.001415	1.018	No Need to be Checked	EDY + Soil	0.001547	0.001522	1.016	No Need to be Checked
62nd Floor	EDX + Soil	0.001432	0.001406	1.018	No Need to be Checked	EDY + Soil	0.001536	0.001511	1.017	No Need to be Checked
61st Floor	EDX + Soil	0.001421	0.001395	1.019	No Need to be Checked	EDY + Soil	0.001521	0.001496	1.017	No Need to be Checked
60th Floor	EDX + Soil	0.001407	0.00138	1.02	No Need to be Checked	EDY + Soil	0.001503	0.001477	1.018	No Need to be Checked
59th Floor	EDX + Soil	0.001389	0.001361	1.02	No Need to be Checked	EDY + Soil	0.00148	0.001454	1.018	No Need to be Checked
58th Floor	EDX + Soil	0.001366	0.001338	1.021	No Need to be Checked	EDY + Soil	0.001453	0.001426	1.019	No Need to be Checked
57th Floor	EDX + Soil	0.001336	0.001309	1.021	No Need to be Checked	EDY + Soil	0.001419	0.001392	1.019	No Need to be Checked
56th Floor	EDX + Soil	0.001299	0.001272	1.021	No Need to be Checked	EDY + Soil	0.001378	0.001351	1.02	No Need to be Checked
55th Floor	EDX + Soil	0.00125	0.001223	1.022	No Need to be Checked	EDY + Soil	0.001326	0.001299	1.021	No Need to be Checked
54th Floor	EDX + Soil	0.001173	0.001147	1.023	No Need to be Checked	EDY + Soil	0.001247	0.00122	1.022	No Need to be Checked
53rd Floor	EDX + Soil	0.000998	0.000973	1.026	No Need to be Checked	EDY + Soil	0.001072	0.001046	1.024	No Need to be Checked
52nd Floor	EDX + Soil	0.001211	0.001077	1.124	No Need to be Checked	EDY + Soil	0.001378	0.001193	1.155	No Need to be Checked
51st Floor	EDX + Soil	0.001169	0.001142	1.024	No Need to be Checked	EDY + Soil	0.001226	0.001199	1.023	No Need to be Checked

50th Floor	EDX + Soil	0.001243	0.001215	1.023	No Need to be Checked	EDY + Soil	0.001311	0.001283	1.022	No Need to be Checked
49th Floor	EDX + Soil	0.001288	0.00126	1.022	No Need to be Checked	EDY + Soil	0.001361	0.001332	1.021	No Need to be Checked
48th Floor	EDX + Soil	0.001323	0.001294	1.022	No Need to be Checked	EDY + Soil	0.001398	0.001369	1.021	No Need to be Checked
47th Floor	EDX + Soil	0.001349	0.00132	1.022	No Need to be Checked	EDY + Soil	0.001427	0.001397	1.021	No Need to be Checked
46th Floor	EDX + Soil	0.00137	0.00134	1.022	No Need to be Checked	EDY + Soil	0.001449	0.001419	1.021	No Need to be Checked
45th Floor	EDX + Soil	0.001385	0.001355	1.022	No Need to be Checked	EDY + Soil	0.001466	0.001436	1.021	No Need to be Checked
44th Floor	EDX + Soil	0.001397	0.001366	1.022	No Need to be Checked	EDY + Soil	0.001479	0.001449	1.021	No Need to be Checked
43rd Floor	EDX + Soil	0.001404	0.001373	1.023	No Need to be Checked	EDY + Soil	0.001488	0.001457	1.021	No Need to be Checked
42nd Floor	EDX + Soil	0.001409	0.001377	1.023	No Need to be Checked	EDY + Soil	0.001492	0.001462	1.021	No Need to be Checked
41st Floor	EDX + Soil	0.00141	0.001378	1.023	No Need to be Checked	EDY + Soil	0.001494	0.001463	1.021	No Need to be Checked
40th Floor	EDX + Soil	0.001408	0.001376	1.024	No Need to be Checked	EDY + Soil	0.001491	0.00146	1.021	No Need to be Checked
39th Floor	EDX + Soil	0.001404	0.001371	1.024	No Need to be Checked	EDY + Soil	0.001486	0.001455	1.021	No Need to be Checked
38th Floor	EDX + Soil	0.001397	0.001364	1.024	No Need to be Checked	EDY + Soil	0.001477	0.001446	1.022	No Need to be Checked
37th Floor	EDX + Soil	0.001386	0.001353	1.024	No Need to be Checked	EDY + Soil	0.001465	0.001434	1.022	No Need to be Checked
36th Floor	EDX + Soil	0.001373	0.00134	1.025	No Need to be Checked	EDY + Soil	0.00145	0.001418	1.022	No Need to be Checked
35th Floor	EDX + Soil	0.001356	0.001324	1.025	No Need to be Checked	EDY + Soil	0.001431	0.001399	1.023	No Need to be Checked
34th Floor	EDX + Soil	0.001336	0.001304	1.025	No Need to be Checked	EDY + Soil	0.001409	0.001376	1.023	No Need to be Checked
33rd Floor	EDX + Soil	0.001313	0.00128	1.026	No Need to be Checked	EDY + Soil	0.001382	0.001349	1.024	No Need to be Checked
32nd Floor	EDX + Soil	0.001285	0.001252	1.026	No Need to be Checked	EDY + Soil	0.00135	0.001317	1.025	No Need to be Checked
31st Floor	EDX + Soil	0.001253	0.00122	1.027	No Need to be Checked	EDY + Soil	0.001313	0.00128	1.026	No Need to be Checked
30th Floor	EDX + Soil	0.001216	0.001183	1.028	No Need to be Checked	EDY + Soil	0.001269	0.001236	1.027	No Need to be Checked
29th Floor	EDX + Soil	0.001172	0.001138	1.03	No Need to be Checked	EDY + Soil	0.001218	0.001184	1.028	No Need to be Checked
28th Floor	EDX + Soil	0.001118	0.001083	1.032	No Need to be Checked	EDY + Soil	0.001157	0.001123	1.03	No Need to be Checked

27th Floor	EDX + Soil	0.001049	0.001014	1.034	No Need to be Checked	EDY + Soil	0.001082	0.001049	1.032	No Need to be Checked
26th Floor	EDX + Soil	0.000945	0.00091	1.039	No Need to be Checked	EDY + Soil	0.000973	0.00094	1.035	No Need to be Checked
25th Floor	EDX + Soil	0.000722	0.000688	1.049	No Need to be Checked	EDY + Soil	0.000748	0.000715	1.045	No Need to be Checked
24th Floor	EDX + Soil	0.000798	0.00072	1.107	No Need to be Checked	EDY + Soil	0.000873	0.000768	1.137	No Need to be Checked
23rd Floor	EDX + Soil	0.000918	0.000881	1.042	No Need to be Checked	EDY + Soil	0.000923	0.000889	1.038	No Need to be Checked
22nd Floor	EDX + Soil	0.001	0.000962	1.039	No Need to be Checked	EDY + Soil	0.001022	0.000987	1.035	No Need to be Checked
21st Floor	EDX + Soil	0.001047	0.001009	1.038	No Need to be Checked	EDY + Soil	0.001076	0.00104	1.034	No Need to be Checked
20th Floor	EDX + Soil	0.00108	0.001041	1.037	No Need to be Checked	EDY + Soil	0.001114	0.001078	1.034	No Need to be Checked
19th Floor	EDX + Soil	0.001101	0.001063	1.036	No Need to be Checked	EDY + Soil	0.00114	0.001104	1.033	No Need to be Checked
18th Floor	EDX + Soil	0.001113	0.001075	1.035	No Need to be Checked	EDY + Soil	0.001158	0.001121	1.033	No Need to be Checked
17th Floor	EDX + Soil	0.001117	0.00108	1.034	No Need to be Checked	EDY + Soil	0.001167	0.00113	1.033	No Need to be Checked
16th Floor	EDX + Soil	0.001116	0.001079	1.033	No Need to be Checked	EDY + Soil	0.00117	0.001133	1.033	No Need to be Checked
15th Floor	EDX + Soil	0.001108	0.001073	1.033	No Need to be Checked	EDY + Soil	0.001167	0.001129	1.033	No Need to be Checked
14th Floor	EDX + Soil	0.001097	0.001062	1.033	No Need to be Checked	EDY + Soil	0.001157	0.00112	1.033	No Need to be Checked
13th Floor	EDX + Soil	0.00108	0.001046	1.033	No Need to be Checked	EDY + Soil	0.001142	0.001105	1.034	No Need to be Checked
12th Floor	EDX + Soil	0.001059	0.001025	1.033	No Need to be Checked	EDY + Soil	0.001121	0.001084	1.034	No Need to be Checked
11th Floor	EDX + Soil	0.001031	0.000998	1.033	No Need to be Checked	EDY + Soil	0.001095	0.001058	1.035	No Need to be Checked
10th Floor	EDX + Soil	0.000998	0.000965	1.034	No Need to be Checked	EDY + Soil	0.001063	0.001025	1.037	No Need to be Checked
9th Floor	EDX + Soil	0.00096	0.000928	1.035	No Need to be Checked	EDY + Soil	0.001025	0.000987	1.038	No Need to be Checked
8th Floor	EDX + Soil	0.000918	0.000885	1.038	No Need to be Checked	EDY + Soil	0.000981	0.000942	1.041	No Need to be Checked
7th Floor	EDX + Soil	0.000849	0.000814	1.043	No Need to be Checked	EDY + Soil	0.000912	0.00087	1.048	No Need to be Checked
6th Floor	EDX + Soil	0.000718	0.000684	1.05	No Need to be Checked	EDY + Soil	0.000781	0.000735	1.062	No Need to be Checked
5th Parking Floor	EDX + Soil	0.000615	0.000558	1.101	No Need to be Checked	EDY + Soil	0.000682	0.000601	1.135	No Need to be Checked

4th Parking Floor	EDX + Soil	0.000567	0.000519	1.094	No Need to be Checked	EDY + Soil	0.000631	0.000548	1.151	No Need to be Checked
3rd Parking Floor	EDX + Soil	0.00052	0.000477	1.09	No Need to be Checked	EDY + Soil	0.000574	0.000493	1.164	No Need to be Checked
2nd Parking Floor	EDX + Soil	0.000464	0.000427	1.086	No Need to be Checked	EDY + Soil	0.000503	0.000426	1.181	No Need to be Checked
1st Parking Floor	EDX + Soil	0.000373	0.000344	1.083	No Need to be Checked	EDY + Soil	0.000383	0.000317	1.209	Needs to be Checked
Ground Floor	EDX + Soil	0.000185	0.000172	1.076	No Need to be Checked	EDY + Soil	0.000149	0.000121	1.232	Needs to be Checked
1st Basement Floor	EDX + Soil	0.000145	0.000136	1.071	No Need to be Checked	EDY + Soil	0.000109	0.00009	1.211	Needs to be Checked
2nd Basement Floor	EDX + Soil	0.000124	0.000117	1.061	No Need to be Checked	EDY + Soil	0.000093	0.000078	1.193	No Need to be Checked
3rd Basement Floor	EDX + Soil	0.000106	0.000101	1.051	No Need to be Checked	EDY + Soil	0.000083	0.00007	1.18	No Need to be Checked
4th Basement Floor	EDX + Soil	8.70E-05	8.30E-05	1.048	No Need to be Checked	EDY + Soil	0.000073	0.000062	1.171	No Need to be Checked
5th Basement Floor	EDX + Soil	6.40E-05	6.10E-05	1.05	No Need to be Checked	EDY + Soil	0.000057	0.000049	1.158	No Need to be Checked
6th Basement Floor	EDX + Soil	3.40E-05	3.20E-05	1.041	No Need to be Checked	EDY + Soil	0.000032	0.000028	1.108	No Need to be Checked

Table 2.3.6.3 – Stories Diaphragm Max over Average Drift Ratios After Modification

Torsional amplification factor for all floors will not be needed except for one floor as shown below.

Story	Output Case	Maximum	Average	Ratio	Torsional Amplification Factor (Ax)	Output Case	Maximum	Average	Ratio	Torsional Amplification Factor (Ax)
1st Parking Floor	**EDX +Soil**	3.602	3.366	1.07	-	**EDY +Soil**	3.155	2.617	1.206	1.010025

Table 2.3.6.4 – Stories Max over Average Displacement Ratios After Modification

- **Diaphragm Discontinuity Irregularity**

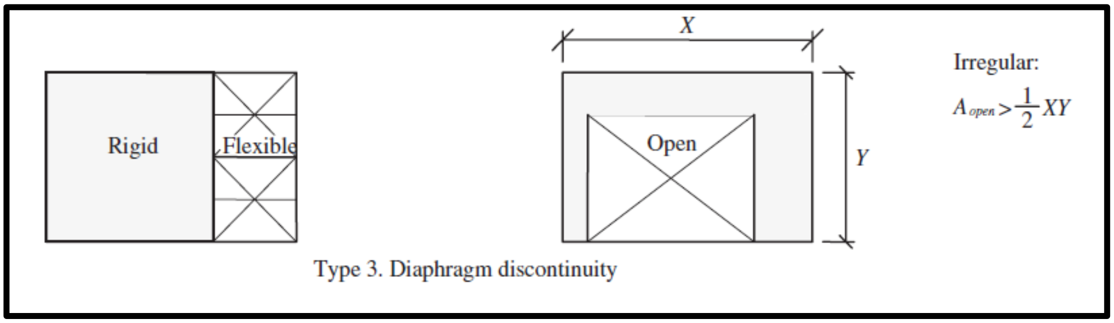


Figure 2.3.6.5 – Diaphragm Discontinuity Irregularity Demonstration

All stories have discontinuities that are less than 50% of the total area of the floor.

2.3.7 Drift Check

To check whether the drift in the building in reasonable, the drift to story height ratio is computed, according to TBI Code for tall buildings, the limit of the drift to story height ratio shall not exceed 0.005

The drift in each story in x-direction is shown in the table below.

Story	Output Case	Elastic drift (mm)	Inelastic drift (mm)	H (m)	Drift Ratio	Check
100th Floor	D + SD + LL + Soil + EDX	10.227	40.908	6.3	0.006493	Not ok
99th Floor	D + SD + LL + Soil + EDX	8.707	34.828	6.3	0.005528	Not ok
98th Floor	D + SD + LL + Soil + EDX	8.55	34.2	6	0.0057	Not ok
97th Floor	D + SD + LL + Soil + EDX	8.354	33.416	6	0.005569	Not ok
96th Floor	D + SD + LL + Soil + EDX	4.885	19.54	3.35	0.005833	Not ok
95th Floor	D + SD + LL + Soil + EDX	5.136	20.544	3.65	0.005628	Not ok
94th Floor	D + SD + LL + Soil + EDX	5.234	20.936	3.65	0.005736	Not ok
93rd Floor	D + SD + LL + Soil + EDX	5.272	21.088	3.65	0.005778	Not ok
92nd Floor	D + SD + LL + Soil + EDX	5.297	21.188	3.65	0.005805	Not ok
91st Floor	D + SD + LL + Soil + EDX	5.317	21.268	3.65	0.005827	Not ok
90th Floor	D + SD + LL + Soil + EDX	5.331	21.324	3.65	0.005842	Not ok
89th Floor	D + SD + LL + Soil + EDX	5.344	21.376	3.65	0.005856	Not ok

88th Floor	D + SD + LL + Soil + EDX	5.351	21.404	3.65	0.005864	Not ok
87th Floor	D + SD + LL + Soil + EDX	5.355	21.42	3.65	0.005868	Not ok
86th Floor	D + SD + LL + Soil + EDX	5.354	21.416	3.65	0.005867	Not ok
85th Floor	D + SD + LL + Soil + EDX	5.35	21.4	3.65	0.005863	Not ok
84th Floor	D + SD + LL + Soil + EDX	5.341	21.364	3.65	0.005853	Not ok
83rd Floor	D + SD + LL + Soil + EDX	5.331	21.324	3.65	0.005842	Not ok
82nd Floor	D + SD + LL + Soil + EDX	5.312	21.248	3.65	0.005821	Not ok
81st Floor	D + SD + LL + Soil + EDX	5.139	20.556	3.65	0.005632	Not ok
80th Floor	D + SD + LL + Soil + EDX	6.607	26.428	5.2	0.005082	Not ok
79th Floor	D + SD + LL + Soil + EDX	4.869	19.476	3.65	0.005336	Not ok
78th Floor	D + SD + LL + Soil + EDX	5.317	21.268	3.65	0.005827	Not ok
77th Floor	D + SD + LL + Soil + EDX	5.558	22.232	3.65	0.006091	Not ok
76th Floor	D + SD + LL + Soil + EDX	5.671	22.684	3.65	0.006215	Not ok
75th Floor	D + SD + LL + Soil + EDX	5.747	22.988	3.65	0.006298	Not ok
74th Floor	D + SD + LL + Soil + EDX	5.819	23.276	3.65	0.006377	Not ok
73rd Floor	D + SD + LL + Soil + EDX	5.888	23.552	3.65	0.006453	Not ok
72nd Floor	D + SD + LL + Soil + EDX	5.949	23.796	3.65	0.006519	Not ok
71st Floor	D + SD + LL + Soil + EDX	5.999	23.996	3.65	0.006574	Not ok
70th Floor	D + SD + LL + Soil + EDX	6.04	24.16	3.65	0.006619	Not ok
69th Floor	D + SD + LL + Soil + EDX	6.073	24.292	3.65	0.006655	Not ok
68th Floor	D + SD + LL + Soil + EDX	6.096	24.384	3.65	0.006681	Not ok
67th Floor	D + SD + LL + Soil + EDX	6.111	24.444	3.65	0.006697	Not ok
66th Floor	D + SD + LL + Soil + EDX	6.117	24.468	3.65	0.006704	Not ok
65th Floor	D + SD + LL + Soil + EDX	6.115	24.46	3.65	0.006701	Not ok
64th Floor	D + SD + LL + Soil + EDX	6.104	24.416	3.65	0.006689	Not ok
63rd Floor	D + SD + LL + Soil + EDX	6.085	24.34	3.65	0.006668	Not ok
62nd Floor	D + SD + LL + Soil + EDX	6.054	24.216	3.65	0.006635	Not ok
61st Floor	D + SD + LL + Soil + EDX	6.016	24.064	3.65	0.006593	Not ok
60th Floor	D + SD + LL + Soil + EDX	5.966	23.864	3.65	0.006538	Not ok
59th Floor	D + SD + LL + Soil + EDX	5.906	23.624	3.65	0.006472	Not ok
58th Floor	D + SD + LL + Soil + EDX	5.838	23.352	3.65	0.006398	Not ok
57th Floor	D + SD + LL + Soil + EDX	5.771	23.084	3.65	0.006324	Not ok
56th Floor	D + SD + LL + Soil + EDX	5.716	22.864	3.65	0.006264	Not ok
55th Floor	D + SD + LL + Soil + EDX	5.602	22.408	3.65	0.006139	Not ok

54th Floor	D + SD + LL + Soil + EDX	5.048	20.192	3.65	0.005532	Not ok
53rd Floor	D + SD + LL + Soil + EDX	4.26	17.04	3.65	0.004668	Ok
52nd Floor	D + SD + LL + Soil + EDX	4.117	16.468	3.65	0.004512	Ok
51st Floor	D + SD + LL + Soil + EDX	5.265	21.06	3.65	0.00577	Not ok
50th Floor	D + SD + LL + Soil + EDX	5.682	22.728	3.65	0.006227	Not ok
49th Floor	D + SD + LL + Soil + EDX	5.758	23.032	3.65	0.00631	Not ok
48th Floor	D + SD + LL + Soil + EDX	5.817	23.268	3.65	0.006375	Not ok
47th Floor	D + SD + LL + Soil + EDX	5.893	23.572	3.65	0.006458	Not ok
46th Floor	D + SD + LL + Soil + EDX	5.964	23.856	3.65	0.006536	Not ok
45th Floor	D + SD + LL + Soil + EDX	6.025	24.1	3.65	0.006603	Not ok
44th Floor	D + SD + LL + Soil + EDX	6.071	24.284	3.65	0.006653	Not ok
43rd Floor	D + SD + LL + Soil + EDX	6.103	24.412	3.65	0.006688	Not ok
42nd Floor	D + SD + LL + Soil + EDX	6.122	24.488	3.65	0.006709	Not ok
41st Floor	D + SD + LL + Soil + EDX	6.13	24.52	3.65	0.006718	Not ok
40th Floor	D + SD + LL + Soil + EDX	6.124	24.496	3.65	0.006711	Not ok
39th Floor	D + SD + LL + Soil + EDX	6.107	24.428	3.65	0.006693	Not ok
38th Floor	D + SD + LL + Soil + EDX	6.078	24.312	3.65	0.006661	Not ok
37th Floor	D + SD + LL + Soil + EDX	6.036	24.144	3.65	0.006615	Not ok
36th Floor	D + SD + LL + Soil + EDX	5.982	23.928	3.65	0.006556	Not ok
35th Floor	D + SD + LL + Soil + EDX	5.915	23.66	3.65	0.006482	Not ok
34th Floor	D + SD + LL + Soil + EDX	5.834	23.336	3.65	0.006393	Not ok
33rd Floor	D + SD + LL + Soil + EDX	5.739	22.956	3.65	0.006289	Not ok
32nd Floor	D + SD + LL + Soil + EDX	5.629	22.516	3.65	0.006169	Not ok
31st Floor	D + SD + LL + Soil + EDX	5.504	22.016	3.65	0.006032	Not ok
30th Floor	D + SD + LL + Soil + EDX	5.368	21.472	3.65	0.005883	Not ok
29th Floor	D + SD + LL + Soil + EDX	5.228	20.912	3.65	0.005729	Not ok
28th Floor	D + SD + LL + Soil + EDX	5.1	20.4	3.65	0.005589	Not ok
27th Floor	D + SD + LL + Soil + EDX	4.886	19.544	3.65	0.005355	Not ok
26th Floor	D + SD + LL + Soil + EDX	4.203	16.812	3.65	0.004606	Ok
25th Floor	D + SD + LL + Soil + EDX	2.911	11.644	3.65	0.00319	Ok
24th Floor	D + SD + LL + Soil + EDX	2.673	10.692	3.65	0.002929	Ok
23rd Floor	D + SD + LL + Soil + EDX	4.242	16.968	3.65	0.004649	Ok
22nd Floor	D + SD + LL + Soil + EDX	4.82	19.28	3.65	0.005282	Not ok
21st Floor	D + SD + LL + Soil + EDX	4.882	19.528	3.65	0.00535	Not ok

20th Floor	D + SD + LL + Soil + EDX	4.932	19.728	3.65	0.005405	Not ok
19th Floor	D + SD + LL + Soil + EDX	5	20	3.65	0.005479	Not ok
18th Floor	D + SD + LL + Soil + EDX	5.058	20.232	3.65	0.005543	Not ok
17th Floor	D + SD + LL + Soil + EDX	5.099	20.396	3.65	0.005588	Not ok
16th Floor	D + SD + LL + Soil + EDX	5.119	20.476	3.65	0.00561	Not ok
15th Floor	D + SD + LL + Soil + EDX	5.118	20.472	3.65	0.005609	Not ok
14th Floor	D + SD + LL + Soil + EDX	5.099	20.396	3.65	0.005588	Not ok
13th Floor	D + SD + LL + Soil + EDX	5.062	20.248	3.65	0.005547	Not ok
12th Floor	D + SD + LL + Soil + EDX	5.005	20.02	3.65	0.005485	Not ok
11th Floor	D + SD + LL + Soil + EDX	4.931	19.724	3.65	0.005404	Not ok
10th Floor	D + SD + LL + Soil + EDX	4.842	19.368	3.65	0.005306	Not ok
9th Floor	D + SD + LL + Soil + EDX	4.743	18.972	3.65	0.005198	Not ok
8th Floor	D + SD + LL + Soil + EDX	4.635	18.54	3.65	0.005079	Not ok
7th Floor	D + SD + LL + Soil + EDX	7.868	31.472	6.6	0.004768	Ok
6th Floor	D + SD + LL + Soil + EDX	3.867	15.468	3.72	0.004158	Ok
5th Parking Floor	D + SD + LL + Soil + EDX	4.066	16.264	3.58	0.004543	Ok
4th Parking Floor	D + SD + LL + Soil + EDX	2.152	8.608	2.8	0.003074	Ok
3rd Parking Floor	D + SD + LL + Soil + EDX	2.183	8.732	2.8	0.003119	Ok
2nd Parking Floor	D + SD + LL + Soil + EDX	2.139	8.556	2.8	0.003056	Ok
1st Parking Floor	D + SD + LL + Soil + EDX	2.135	8.54	3.8	0.002247	Ok
Ground Floor	D + SD + LL + Soil + EDX	1.296	5.184	3.35	0.001547	Ok
1st Basement Floor	D + SD + LL + Soil + EDX	0.905	3.62	3.05	0.001187	Ok
2nd Basement Floor	D + SD + LL + Soil + EDX	0.676	2.704	3.05	0.000887	Ok
3rd Basement Floor	D + SD + LL + Soil + EDX	0.465	1.86	3.05	0.00061	Ok
4th Basement Floor	D + SD + LL + Soil + EDX	0.269	1.076	3.05	0.000353	Ok
5th Basement Floor	D + SD + LL + Soil + EDX	0.061	0.244	3.05	0.00008	Ok
6th Basement Floor	D + SD + LL + Soil + EDX	0.015	0.06	1	0.00006	Ok

Table 2.3.7.1 - Story Drifts in X-Direction Before Modification

- Drift after modifications – X Direction

Story	Output Case	Elastic drift (mm)	Inelastic drift (mm)	H (m)	Drift Ratio	Check
100th Floor	D + SD + LL + Soil + EDX	6.769	27.076	6.3	0.004298	OK
99th Floor	D + SD + LL + Soil + EDX	5.915	23.66	6.3	0.003756	OK
98th Floor	D + SD + LL + Soil + EDX	6.053	24.212	6	0.004035	OK
97th Floor	D + SD + LL + Soil + EDX	5.893	23.572	6	0.003929	OK
96th Floor	D + SD + LL + Soil + EDX	3.504	14.016	3.35	0.004184	OK
95th Floor	D + SD + LL + Soil + EDX	3.654	14.616	3.65	0.004004	OK
94th Floor	D + SD + LL + Soil + EDX	3.69	14.76	3.65	0.004044	OK
93rd Floor	D + SD + LL + Soil + EDX	3.723	14.892	3.65	0.00408	OK
92nd Floor	D + SD + LL + Soil + EDX	3.747	14.988	3.65	0.004106	OK
91st Floor	D + SD + LL + Soil + EDX	3.767	15.068	3.65	0.004128	OK
90th Floor	D + SD + LL + Soil + EDX	3.781	15.124	3.65	0.004144	OK
89th Floor	D + SD + LL + Soil + EDX	3.793	15.172	3.65	0.004157	OK
88th Floor	D + SD + LL + Soil + EDX	3.799	15.196	3.65	0.004163	OK
87th Floor	D + SD + LL + Soil + EDX	3.803	15.212	3.65	0.004168	OK
86th Floor	D + SD + LL + Soil + EDX	3.8	15.2	3.65	0.004164	OK
85th Floor	D + SD + LL + Soil + EDX	3.793	15.172	3.65	0.004157	OK
84th Floor	D + SD + LL + Soil + EDX	3.78	15.12	3.65	0.004142	OK
83rd Floor	D + SD + LL + Soil + EDX	3.76	15.04	3.65	0.004121	OK
82nd Floor	D + SD + LL + Soil + EDX	3.725	14.9	3.65	0.004082	OK
81st Floor	D + SD + LL + Soil + EDX	3.528	14.112	3.65	0.003866	OK
80th Floor	D + SD + LL + Soil + EDX	4.633	18.532	5.2	0.003564	OK
79th Floor	D + SD + LL + Soil + EDX	3.336	13.344	3.65	0.003656	OK
78th Floor	D + SD + LL + Soil + EDX	3.695	14.78	3.65	0.004049	OK
77th Floor	D + SD + LL + Soil + EDX	3.843	15.372	3.65	0.004212	OK
76th Floor	D + SD + LL + Soil + EDX	3.934	15.736	3.65	0.004311	OK
75th Floor	D + SD + LL + Soil + EDX	4.009	16.036	3.65	0.004393	OK
74th Floor	D + SD + LL + Soil + EDX	4.075	16.3	3.65	0.004466	OK
73rd Floor	D + SD + LL + Soil + EDX	4.135	16.54	3.65	0.004532	OK
72nd Floor	D + SD + LL + Soil + EDX	4.183	16.732	3.65	0.004584	OK
71st Floor	D + SD + LL + Soil + EDX	4.224	16.896	3.65	0.004629	OK

70th Floor	D + SD + LL + Soil + EDX	4.256	17.024	3.65	0.004664	OK
69th Floor	D + SD + LL + Soil + EDX	4.28	17.12	3.65	0.00469	OK
68th Floor	D + SD + LL + Soil + EDX	4.298	17.192	3.65	0.00471	OK
67th Floor	D + SD + LL + Soil + EDX	4.309	17.236	3.65	0.004722	OK
66th Floor	D + SD + LL + Soil + EDX	4.315	17.26	3.65	0.004729	OK
65th Floor	D + SD + LL + Soil + EDX	4.313	17.252	3.65	0.004727	OK
64th Floor	D + SD + LL + Soil + EDX	4.307	17.228	3.65	0.00472	OK
63rd Floor	D + SD + LL + Soil + EDX	4.293	17.172	3.65	0.004705	OK
62nd Floor	D + SD + LL + Soil + EDX	4.272	17.088	3.65	0.004682	OK
61st Floor	D + SD + LL + Soil + EDX	4.243	16.972	3.65	0.00465	OK
60th Floor	D + SD + LL + Soil + EDX	4.208	16.832	3.65	0.004612	OK
59th Floor	D + SD + LL + Soil + EDX	4.162	16.648	3.65	0.004561	OK
58th Floor	D + SD + LL + Soil + EDX	4.107	16.428	3.65	0.004501	OK
57th Floor	D + SD + LL + Soil + EDX	4.046	16.184	3.65	0.004434	OK
56th Floor	D + SD + LL + Soil + EDX	3.99	15.96	3.65	0.004373	OK
55th Floor	D + SD + LL + Soil + EDX	3.914	15.656	3.65	0.004289	OK
54th Floor	D + SD + LL + Soil + EDX	3.585	14.34	3.65	0.003929	OK
53rd Floor	D + SD + LL + Soil + EDX	3.044	12.176	3.65	0.003336	OK
52nd Floor	D + SD + LL + Soil + EDX	2.991	11.964	3.65	0.003278	OK
51st Floor	D + SD + LL + Soil + EDX	3.728	14.912	3.65	0.004085	OK
50th Floor	D + SD + LL + Soil + EDX	3.991	15.964	3.65	0.004374	OK
49th Floor	D + SD + LL + Soil + EDX	4.061	16.244	3.65	0.00445	OK
48th Floor	D + SD + LL + Soil + EDX	4.127	16.508	3.65	0.004523	OK
47th Floor	D + SD + LL + Soil + EDX	4.195	16.78	3.65	0.004597	OK
46th Floor	D + SD + LL + Soil + EDX	4.254	17.016	3.65	0.004662	OK
45th Floor	D + SD + LL + Soil + EDX	4.299	17.196	3.65	0.004711	OK
44th Floor	D + SD + LL + Soil + EDX	4.333	17.332	3.65	0.004748	OK
43rd Floor	D + SD + LL + Soil + EDX	4.357	17.428	3.65	0.004775	OK
42nd Floor	D + SD + LL + Soil + EDX	4.371	17.484	3.65	0.00479	OK
41st Floor	D + SD + LL + Soil + EDX	4.376	17.504	3.65	0.004796	OK
40th Floor	D + SD + LL + Soil + EDX	4.373	17.492	3.65	0.004792	OK
39th Floor	D + SD + LL + Soil + EDX	4.362	17.448	3.65	0.00478	OK
38th Floor	D + SD + LL + Soil + EDX	4.34	17.36	3.65	0.004756	OK
37th Floor	D + SD + LL + Soil + EDX	4.312	17.248	3.65	0.004725	OK

36th Floor	D + SD + LL + Soil + EDX	4.273	17.092	3.65	0.004683	OK
35th Floor	D + SD + LL + Soil + EDX	4.225	16.9	3.65	0.00463	OK
34th Floor	D + SD + LL + Soil + EDX	4.168	16.672	3.65	0.004568	OK
33rd Floor	D + SD + LL + Soil + EDX	4.098	16.392	3.65	0.004491	OK
32nd Floor	D + SD + LL + Soil + EDX	4.018	16.072	3.65	0.004403	OK
31st Floor	D + SD + LL + Soil + EDX	3.924	15.696	3.65	0.0043	OK
30th Floor	D + SD + LL + Soil + EDX	3.816	15.264	3.65	0.004182	OK
29th Floor	D + SD + LL + Soil + EDX	3.695	14.78	3.65	0.004049	OK
28th Floor	D + SD + LL + Soil + EDX	3.577	14.308	3.65	0.00392	OK
27th Floor	D + SD + LL + Soil + EDX	3.44	13.76	3.65	0.00377	OK
26th Floor	D + SD + LL + Soil + EDX	3.094	12.376	3.65	0.003391	OK
25th Floor	D + SD + LL + Soil + EDX	2.157	8.628	3.65	0.002364	OK
24th Floor	D + SD + LL + Soil + EDX	2.037	8.148	3.65	0.002232	OK
23rd Floor	D + SD + LL + Soil + EDX	2.968	11.872	3.65	0.003253	OK
22nd Floor	D + SD + LL + Soil + EDX	3.364	13.456	3.65	0.003687	OK
21st Floor	D + SD + LL + Soil + EDX	3.416	13.664	3.65	0.003744	OK
20th Floor	D + SD + LL + Soil + EDX	3.475	13.9	3.65	0.003808	OK
19th Floor	D + SD + LL + Soil + EDX	3.53	14.12	3.65	0.003868	OK
18th Floor	D + SD + LL + Soil + EDX	3.567	14.268	3.65	0.003909	OK
17th Floor	D + SD + LL + Soil + EDX	3.586	14.344	3.65	0.00393	OK
16th Floor	D + SD + LL + Soil + EDX	3.587	14.348	3.65	0.003931	OK
15th Floor	D + SD + LL + Soil + EDX	3.57	14.28	3.65	0.003912	OK
14th Floor	D + SD + LL + Soil + EDX	3.537	14.148	3.65	0.003876	OK
13th Floor	D + SD + LL + Soil + EDX	3.49	13.96	3.65	0.003825	OK
12th Floor	D + SD + LL + Soil + EDX	3.426	13.704	3.65	0.003755	OK
11th Floor	D + SD + LL + Soil + EDX	3.346	13.384	3.65	0.003667	OK
10th Floor	D + SD + LL + Soil + EDX	3.248	12.992	3.65	0.003559	OK
9th Floor	D + SD + LL + Soil + EDX	3.131	12.524	3.65	0.003431	OK
8th Floor	D + SD + LL + Soil + EDX	2.996	11.984	3.65	0.003283	OK
7th Floor	D + SD + LL + Soil + EDX	4.925	19.7	6.6	0.002985	OK
6th Floor	D + SD + LL + Soil + EDX	2.247	8.988	3.72	0.002416	OK
5th Parking Floor	D + SD + LL + Soil + EDX	2.03	8.12	3.58	0.002268	OK
4th Parking Floor	D + SD + LL + Soil + EDX	1.241	4.964	2.8	0.001773	OK

3rd Parking Floor	D + SD + LL + Soil + EDX	1.141	4.564	2.8	0.00163	OK
2nd Parking Floor	D + SD + LL + Soil + EDX	1.046	4.184	2.8	0.001494	OK
1st Parking Floor	D + SD + LL + Soil + EDX	1.193	4.772	3.8	0.001256	OK
Ground Floor	D + SD + LL + Soil + EDX	0.788	3.152	3.35	0.000941	OK
1st Basement Floor	D + SD + LL + Soil + EDX	0.592	2.368	3.05	0.000776	OK
2nd Basement Floor	D + SD + LL + Soil + EDX	0.477	1.908	3.05	0.000626	OK
3rd Basement Floor	D + SD + LL + Soil + EDX	0.361	1.444	3.05	0.000473	OK
4th Basement Floor	D + SD + LL + Soil + EDX	0.24	0.96	3.05	0.000315	OK
5th Basement Floor	D + SD + LL + Soil + EDX	0.157	0.628	3.05	0.000206	OK
6th Basement Floor	D + SD + LL + Soil + EDX	0	0	-18.6	0	OK

Table 2.3.7.2 - Story Drifts in X-Direction After Modification

- Drift after modifications – Y direction

Story	Output Case	Elastic drift (mm)	Inelastic drift (mm)	H (m)	Drift Ratio	Check
100th Floor	D + SD + LL + Soil + EDY	6.322	25.288	6.3	0.004014	OK
99th Floor	D + SD + LL + Soil + EDY	6.09	24.36	6.3	0.003867	OK
98th Floor	D + SD + LL + Soil + EDY	5.811	23.244	6	0.003874	OK
97th Floor	D + SD + LL + Soil + EDY	6.01	24.04	6	0.004007	OK
96th Floor	D + SD + LL + Soil + EDY	3.194	12.776	3.35	0.003814	OK
95th Floor	D + SD + LL + Soil + EDY	3.607	14.428	3.65	0.003953	OK
94th Floor	D + SD + LL + Soil + EDY	3.668	14.672	3.65	0.00402	OK
93rd Floor	D + SD + LL + Soil + EDY	3.697	14.788	3.65	0.004052	OK
92nd Floor	D + SD + LL + Soil + EDY	3.724	14.896	3.65	0.004081	OK
91st Floor	D + SD + LL + Soil + EDY	3.745	14.98	3.65	0.004104	OK
90th Floor	D + SD + LL + Soil + EDY	3.76	15.04	3.65	0.004121	OK
89th Floor	D + SD + LL + Soil + EDY	3.771	15.084	3.65	0.004133	OK
88th Floor	D + SD + LL + Soil + EDY	3.775	15.1	3.65	0.004137	OK
87th Floor	D + SD + LL + Soil + EDY	3.775	15.1	3.65	0.004137	OK
86th Floor	D + SD + LL + Soil + EDY	3.769	15.076	3.65	0.00413	OK
85th Floor	D + SD + LL + Soil + EDY	3.755	15.02	3.65	0.004115	OK
84th Floor	D + SD + LL + Soil + EDY	3.732	14.928	3.65	0.00409	OK
83rd Floor	D + SD + LL + Soil + EDY	3.696	14.784	3.65	0.00405	OK
82nd Floor	D + SD + LL + Soil + EDY	3.642	14.568	3.65	0.003991	OK
81st Floor	D + SD + LL + Soil + EDY	3.545	14.18	3.65	0.003885	OK
80th Floor	D + SD + LL + Soil + EDY	4.68	18.72	5.2	0.0036	OK
79th Floor	D + SD + LL + Soil + EDY	3.273	13.092	3.65	0.003587	OK
78th Floor	D + SD + LL + Soil + EDY	3.452	13.808	3.65	0.003783	OK
77th Floor	D + SD + LL + Soil + EDY	3.667	14.668	3.65	0.004019	OK
76th Floor	D + SD + LL + Soil + EDY	3.821	15.284	3.65	0.004187	OK
75th Floor	D + SD + LL + Soil + EDY	3.926	15.704	3.65	0.004302	OK
74th Floor	D + SD + LL + Soil + EDY	4.004	16.016	3.65	0.004388	OK
73rd Floor	D + SD + LL + Soil + EDY	4.068	16.272	3.65	0.004458	OK
72nd Floor	D + SD + LL + Soil + EDY	4.119	16.476	3.65	0.004514	OK

71st Floor	D + SD + LL + Soil + EDY	4.161	16.644	3.65	0.00456	OK
70th Floor	D + SD + LL + Soil + EDY	4.193	16.772	3.65	0.004595	OK
69th Floor	D + SD + LL + Soil + EDY	4.218	16.872	3.65	0.004622	OK
68th Floor	D + SD + LL + Soil + EDY	4.235	16.94	3.65	0.004641	OK
67th Floor	D + SD + LL + Soil + EDY	4.245	16.98	3.65	0.004652	OK
66th Floor	D + SD + LL + Soil + EDY	4.248	16.992	3.65	0.004655	OK
65th Floor	D + SD + LL + Soil + EDY	4.245	16.98	3.65	0.004652	OK
64th Floor	D + SD + LL + Soil + EDY	4.233	16.932	3.65	0.004639	OK
63rd Floor	D + SD + LL + Soil + EDY	4.215	16.86	3.65	0.004619	OK
62nd Floor	D + SD + LL + Soil + EDY	4.189	16.756	3.65	0.004591	OK
61st Floor	D + SD + LL + Soil + EDY	4.155	16.62	3.65	0.004553	OK
60th Floor	D + SD + LL + Soil + EDY	4.113	16.452	3.65	0.004507	OK
59th Floor	D + SD + LL + Soil + EDY	4.063	16.252	3.65	0.004453	OK
58th Floor	D + SD + LL + Soil + EDY	4.001	16.004	3.65	0.004385	OK
57th Floor	D + SD + LL + Soil + EDY	3.927	15.708	3.65	0.004304	OK
56th Floor	D + SD + LL + Soil + EDY	3.831	15.324	3.65	0.004198	OK
55th Floor	D + SD + LL + Soil + EDY	3.691	14.764	3.65	0.004045	OK
54th Floor	D + SD + LL + Soil + EDY	3.487	13.948	3.65	0.003821	OK
53rd Floor	D + SD + LL + Soil + EDY	3.043	12.172	3.65	0.003335	OK
52nd Floor	D + SD + LL + Soil + EDY	3.026	12.104	3.65	0.003316	OK
51st Floor	D + SD + LL + Soil + EDY	3.331	13.324	3.65	0.00365	OK
50th Floor	D + SD + LL + Soil + EDY	3.661	14.644	3.65	0.004012	OK
49th Floor	D + SD + LL + Soil + EDY	3.868	15.472	3.65	0.004239	OK
48th Floor	D + SD + LL + Soil + EDY	3.994	15.976	3.65	0.004377	OK
47th Floor	D + SD + LL + Soil + EDY	4.082	16.328	3.65	0.004473	OK
46th Floor	D + SD + LL + Soil + EDY	4.147	16.588	3.65	0.004545	OK
45th Floor	D + SD + LL + Soil + EDY	4.2	16.8	3.65	0.004603	OK
44th Floor	D + SD + LL + Soil + EDY	4.24	16.96	3.65	0.004647	OK
43rd Floor	D + SD + LL + Soil + EDY	4.269	17.076	3.65	0.004678	OK
42nd Floor	D + SD + LL + Soil + EDY	4.29	17.16	3.65	0.004701	OK
41st Floor	D + SD + LL + Soil + EDY	4.299	17.196	3.65	0.004711	OK
40th Floor	D + SD + LL + Soil + EDY	4.298	17.192	3.65	0.00471	OK
39th Floor	D + SD + LL + Soil + EDY	4.29	17.16	3.65	0.004701	OK
38th Floor	D + SD + LL + Soil + EDY	4.27	17.08	3.65	0.004679	OK

37th Floor	D + SD + LL + Soil + EDY	4.24	16.96	3.65	0.004647	OK
36th Floor	D + SD + LL + Soil + EDY	4.199	16.796	3.65	0.004602	OK
35th Floor	D + SD + LL + Soil + EDY	4.149	16.596	3.65	0.004547	OK
34th Floor	D + SD + LL + Soil + EDY	4.087	16.348	3.65	0.004479	OK
33rd Floor	D + SD + LL + Soil + EDY	4.012	16.048	3.65	0.004397	OK
32nd Floor	D + SD + LL + Soil + EDY	3.927	15.708	3.65	0.004304	OK
31st Floor	D + SD + LL + Soil + EDY	3.828	15.312	3.65	0.004195	OK
30th Floor	D + SD + LL + Soil + EDY	3.714	14.856	3.65	0.00407	OK
29th Floor	D + SD + LL + Soil + EDY	3.58	14.32	3.65	0.003923	OK
28th Floor	D + SD + LL + Soil + EDY	3.416	13.664	3.65	0.003744	OK
27th Floor	D + SD + LL + Soil + EDY	3.181	12.724	3.65	0.003486	OK
26th Floor	D + SD + LL + Soil + EDY	2.777	11.108	3.65	0.003043	OK
25th Floor	D + SD + LL + Soil + EDY	2.165	8.66	3.65	0.002373	OK
24th Floor	D + SD + LL + Soil + EDY	2.168	8.672	3.65	0.002376	OK
23rd Floor	D + SD + LL + Soil + EDY	2.567	10.268	3.65	0.002813	OK
22nd Floor	D + SD + LL + Soil + EDY	2.985	11.94	3.65	0.003271	OK
21st Floor	D + SD + LL + Soil + EDY	3.239	12.956	3.65	0.00355	OK
20th Floor	D + SD + LL + Soil + EDY	3.385	13.54	3.65	0.00371	OK
19th Floor	D + SD + LL + Soil + EDY	3.474	13.896	3.65	0.003807	OK
18th Floor	D + SD + LL + Soil + EDY	3.531	14.124	3.65	0.00387	OK
17th Floor	D + SD + LL + Soil + EDY	3.565	14.26	3.65	0.003907	OK
16th Floor	D + SD + LL + Soil + EDY	3.584	14.336	3.65	0.003928	OK
15th Floor	D + SD + LL + Soil + EDY	3.585	14.34	3.65	0.003929	OK
14th Floor	D + SD + LL + Soil + EDY	3.57	14.28	3.65	0.003912	OK
13th Floor	D + SD + LL + Soil + EDY	3.539	14.156	3.65	0.003878	OK
12th Floor	D + SD + LL + Soil + EDY	3.491	13.964	3.65	0.003826	OK
11th Floor	D + SD + LL + Soil + EDY	3.42	13.68	3.65	0.003748	OK
10th Floor	D + SD + LL + Soil + EDY	3.328	13.312	3.65	0.003647	OK
9th Floor	D + SD + LL + Soil + EDY	3.206	12.824	3.65	0.003513	OK
8th Floor	D + SD + LL + Soil + EDY	3.057	12.228	3.65	0.00335	OK
7th Floor	D + SD + LL + Soil + EDY	5.153	20.612	6.6	0.003123	OK
6th Floor	D + SD + LL + Soil + EDY	2.84	11.36	3.72	0.003054	OK
5th Parking Floor	D + SD + LL + Soil + EDY	2.082	8.328	3.58	0.002326	OK

4th Parking Floor	D + SD + LL + Soil + EDY	1.382	5.528	2.8	0.001974	OK
3rd Parking Floor	D + SD + LL + Soil + EDY	1.189	4.756	2.8	0.001699	OK
2nd Parking Floor	D + SD + LL + Soil + EDY	1.072	4.288	2.8	0.001531	OK
1st Parking Floor	D + SD + LL + Soil + EDY	1.015	4.06	3.8	0.001068	OK
Ground Floor	D + SD + LL + Soil + EDY	0.712	2.848	3.35	0.00085	OK
1st Basement Floor	D + SD + LL + Soil + EDY	0.441	1.764	3.05	0.000578	OK
2nd Basement Floor	D + SD + LL + Soil + EDY	0.333	1.332	3.05	0.000437	OK
3rd Basement Floor	D + SD + LL + Soil + EDY	0.268	1.072	3.05	0.000351	OK
4th Basement Floor	D + SD + LL + Soil + EDY	0.201	0.804	3.05	0.000264	OK
5th Basement Floor	D + SD + LL + Soil + EDY	-0.084	-0.336	3.05	-0.00011	OK
6th Basement Floor	D + SD + LL + Soil + EDY	0	0	-18.6	0	OK

Table 2.3.7.3 - Story Drifts in Y-Direction After Modification

2.3.8 P Delta Checks

To determine whether the P-Delta analysis is needed, the stability coefficient for each floor is computed by the equation:

$$\theta = \frac{P * \Delta}{V * h}$$

P is the axial force on the floor from the combination D +SD + LL + Soil + EDX

V is the shear force on the floor from the same combination

Δ is the drift in the story

h is the height of the story

The table below shows the stability coefficient values for each story in both directions.

- **P Delta – X direction**

Story	Output Case	P (kN)	Vx (kN)	Drift (mm)	H(m)	Stability Coefficient (θ)	Check
100th Floor	D + SD + LL + Soil + EDX	28785.82	3627.448	10.227	6.3	0.012882056	No P-Delta Needed
99th Floor	D + SD + LL + Soil + EDX	58701.86	5915.643	8.707	6.3	0.013714434	No P-Delta Needed
98th Floor	D + SD + LL + Soil + EDX	89780.15	8250.841	8.55	6	0.015505901	No P-Delta Needed
97th Floor	D + SD + LL + Soil + EDX	124562	9971.17	8.354	6	0.017393331	No P-Delta Needed
96th Floor	D + SD + LL + Soil + EDX	161753.3	11347.53	4.885	3.35	0.020786033	No P-Delta Needed
95th Floor	D + SD + LL + Soil + EDX	191645.8	12215.77	5.136	3.65	0.022075512	No P-Delta Needed
94th Floor	D + SD + LL + Soil + EDX	221538.4	12806.76	5.234	3.65	0.024805637	No P-Delta Needed
93rd Floor	D + SD + LL + Soil + EDX	251430.9	13188.81	5.272	3.65	0.027535661	No P-Delta Needed
92nd Floor	D + SD + LL + Soil + EDX	281323.4	13419.6	5.297	3.65	0.030423091	No P-Delta Needed
91st Floor	D + SD + LL + Soil + EDX	311216	13572.09	5.317	3.65	0.033403257	No P-Delta Needed
90th Floor	D + SD + LL + Soil + EDX	341108.5	13789.83	5.331	3.65	0.036128475	No P-Delta Needed
89th Floor	D + SD + LL + Soil + EDX	371001.1	14114.14	5.344	3.65	0.038485254	No P-Delta Needed
88th Floor	D + SD + LL + Soil + EDX	400893.6	14498.63	5.351	3.65	0.040536321	No P-Delta Needed
87th Floor	D + SD + LL + Soil + EDX	430786.1	14947	5.355	3.65	0.042283823	No P-Delta Needed
86th Floor	D + SD + LL + Soil + EDX	460678.7	15457.13	5.354	3.65	0.043717457	No P-Delta Needed
85th Floor	D + SD + LL + Soil + EDX	490571.2	16015.19	5.35	3.65	0.044898415	No P-Delta Needed
84th Floor	D + SD + LL + Soil + EDX	520463.8	16598.87	5.341	3.65	0.04588193	No P-Delta Needed
83rd Floor	D + SD + LL + Soil + EDX	550356.3	17182.02	5.331	3.65	0.046782723	No P-Delta Needed
82nd Floor	D + SD + LL + Soil + EDX	580248.9	17738.85	5.312	3.65	0.047605157	No P-Delta Needed

81st Floor	D + SD + LL + Soil + EDX	610141.4	18247.05	5.139	3.65	0.047078613	No P-Delta Needed
80th Floor	D + SD + LL + Soil + EDX	684709.4	19203.23	6.607	5.2	0.045303623	No P-Delta Needed
79th Floor	D + SD + LL + Soil + EDX	740993.8	20477.04	4.869	3.65	0.04827189	No P-Delta Needed
78th Floor	D + SD + LL + Soil + EDX	782668.5	21598.82	5.317	3.65	0.052786359	No P-Delta Needed
77th Floor	D + SD + LL + Soil + EDX	816199.5	22177.23	5.558	3.65	0.056042129	No P-Delta Needed
76th Floor	D + SD + LL + Soil + EDX	849730.4	22570.13	5.671	3.65	0.058494355	No P-Delta Needed
75th Floor	D + SD + LL + Soil + EDX	883261.3	22787.75	5.747	3.65	0.061028965	No P-Delta Needed
74th Floor	D + SD + LL + Soil + EDX	916792.2	22823.07	5.819	3.65	0.06404014	No P-Delta Needed
73rd Floor	D + SD + LL + Soil + EDX	950323.1	22673.49	5.888	3.65	0.06761264	No P-Delta Needed
72nd Floor	D + SD + LL + Soil + EDX	983854.1	22367.84	5.949	3.65	0.071689864	No P-Delta Needed
71st Floor	D + SD + LL + Soil + EDX	1017385	21957.94	5.999	3.65	0.076151736	No P-Delta Needed
70th Floor	D + SD + LL + Soil + EDX	1050916	21508.69	6.04	3.65	0.080853377	No P-Delta Needed
69th Floor	D + SD + LL + Soil + EDX	1084447	21088.76	6.073	3.65	0.085559372	No P-Delta Needed
68th Floor	D + SD + LL + Soil + EDX	1117978	20760.2	6.096	3.65	0.089940184	No P-Delta Needed
67th Floor	D + SD + LL + Soil + EDX	1151509	20568.1	6.111	3.65	0.093733006	No P-Delta Needed
66th Floor	D + SD + LL + Soil + EDX	1185040	20533.63	6.117	3.65	0.096719225	No P-Delta Needed
65th Floor	D + SD + LL + Soil + EDX	1218570	20653.94	6.115	3.65	0.098844258	No P-Delta Needed
64th Floor	D + SD + LL + Soil + EDX	1252101	20911.71	6.104	3.65	0.10013172	P-Delta Needed
63rd Floor	D + SD + LL + Soil + EDX	1285632	21291.36	6.085	3.65	0.100665592	P-Delta Needed
62nd Floor	D + SD + LL + Soil + EDX	1319163	21765.21	6.054	3.65	0.100527582	P-Delta Needed
61st Floor	D + SD + LL + Soil + EDX	1352694	22268.87	6.016	3.65	0.100118989	P-Delta Needed
60th Floor	D + SD + LL + Soil + EDX	1386225	22747.15	5.966	3.65	0.09960869	No P-Delta Needed
59th Floor	D + SD + LL + Soil + EDX	1419756	23170.85	5.906	3.65	0.099145331	No P-Delta Needed

58th Floor	D + SD + LL + Soil + EDX	1453287	23528.58	5.838	3.65	0.098793156	No P-Delta Needed
57th Floor	D + SD + LL + Soil + EDX	1486818	23819.95	5.771	3.65	0.098690456	No P-Delta Needed
56th Floor	D + SD + LL + Soil + EDX	1520349	24050.53	5.716	3.65	0.09899605	No P-Delta Needed
55th Floor	D + SD + LL + Soil + EDX	1553880	24228.31	5.602	3.65	0.098433868	No P-Delta Needed
54th Floor	D + SD + LL + Soil + EDX	1587411	24363.69	5.048	3.65	0.090109941	No P-Delta Needed
53rd Floor	D + SD + LL + Soil + EDX	1644103	24600.99	4.26	3.65	0.077999755	No P-Delta Needed
52nd Floor	D + SD + LL + Soil + EDX	1697004	25162.69	4.117	3.65	0.076070074	No P-Delta Needed
51st Floor	D + SD + LL + Soil + EDX	1738679	25988.74	5.265	3.65	0.096502744	No P-Delta Needed
50th Floor	D + SD + LL + Soil + EDX	1772210	26446.72	5.682	3.65	0.104316208	P-Delta Needed
49th Floor	D + SD + LL + Soil + EDX	1805741	26728.13	5.758	3.65	0.106577519	P-Delta Needed
48th Floor	D + SD + LL + Soil + EDX	1839272	26823.19	5.817	3.65	0.109280233	P-Delta Needed
47th Floor	D + SD + LL + Soil + EDX	1872803	26759.27	5.893	3.65	0.112995532	P-Delta Needed
46th Floor	D + SD + LL + Soil + EDX	1906334	26593.2	5.964	3.65	0.117131327	P-Delta Needed
45th Floor	D + SD + LL + Soil + EDX	1939865	26397.55	6.025	3.65	0.121303113	P-Delta Needed
44th Floor	D + SD + LL + Soil + EDX	1973395	26243.3	6.071	3.65	0.125072829	P-Delta Needed
43rd Floor	D + SD + LL + Soil + EDX	2006926	26182.78	6.103	3.65	0.128164033	P-Delta Needed
42nd Floor	D + SD + LL + Soil + EDX	2040457	26238.15	6.122	3.65	0.130435171	P-Delta Needed
41st Floor	D + SD + LL + Soil + EDX	2073988	26399.09	6.13	3.65	0.131942571	P-Delta Needed
40th Floor	D + SD + LL + Soil + EDX	2107519	26629.68	6.124	3.65	0.132784661	P-Delta Needed
39th Floor	D + SD + LL + Soil + EDX	2141050	26883.39	6.107	3.65	0.133253263	P-Delta Needed
38th Floor	D + SD + LL + Soil + EDX	2174581	27122.31	6.078	3.65	0.133510923	P-Delta Needed
37th Floor	D + SD + LL + Soil + EDX	2208112	27331.98	6.036	3.65	0.133599984	P-Delta Needed
36th Floor	D + SD + LL + Soil + EDX	2241643	27525.38	5.982	3.65	0.13347091	P-Delta Needed

35th Floor	D + SD + LL + Soil + EDX	2275174	27735.83	5.915	3.65	0.132933775	P-Delta Needed
34th Floor	D + SD + LL + Soil + EDX	2308705	28002.27	5.834	3.65	0.131779731	P-Delta Needed
33rd Floor	D + SD + LL + Soil + EDX	2342235	28352.9	5.739	3.65	0.129890217	P-Delta Needed
32nd Floor	D + SD + LL + Soil + EDX	2375766	28793.36	5.629	3.65	0.127247659	P-Delta Needed
31st Floor	D + SD + LL + Soil + EDX	2409297	29302.63	5.504	3.65	0.123985076	P-Delta Needed
30th Floor	D + SD + LL + Soil + EDX	2442828	29834.73	5.368	3.65	0.120417728	P-Delta Needed
29th Floor	D + SD + LL + Soil + EDX	2476359	30330.96	5.228	3.65	0.116941899	P-Delta Needed
28th Floor	D + SD + LL + Soil + EDX	2509890	30743.82	5.1	3.65	0.114070737	P-Delta Needed
27th Floor	D + SD + LL + Soil + EDX	2543421	31056.06	4.886	3.65	0.109630782	P-Delta Needed
26th Floor	D + SD + LL + Soil + EDX	2576952	31287.68	4.203	3.65	0.094841742	No P-Delta Needed
25th Floor	D + SD + LL + Soil + EDX	2633935	31706.76	2.911	3.65	0.066252536	No P-Delta Needed
24th Floor	D + SD + LL + Soil + EDX	2686836	32679.28	2.673	3.65	0.060210849	No P-Delta Needed
23rd Floor	D + SD + LL + Soil + EDX	2728511	34079.46	4.242	3.65	0.093048778	No P-Delta Needed
22nd Floor	D + SD + LL + Soil + EDX	2762041	34851.98	4.82	3.65	0.104654261	P-Delta Needed
21st Floor	D + SD + LL + Soil + EDX	2795572	35321.41	4.882	3.65	0.105861409	P-Delta Needed
20th Floor	D + SD + LL + Soil + EDX	2829103	35465.45	4.932	3.65	0.107788767	P-Delta Needed
19th Floor	D + SD + LL + Soil + EDX	2862634	35338.61	5	3.65	0.110966913	P-Delta Needed
18th Floor	D + SD + LL + Soil + EDX	2896165	35060.06	5.058	3.65	0.114471289	P-Delta Needed
17th Floor	D + SD + LL + Soil + EDX	2929696	34782.68	5.099	3.65	0.117666175	P-Delta Needed
16th Floor	D + SD + LL + Soil + EDX	2963227	34632.47	5.119	3.65	0.119997919	P-Delta Needed
15th Floor	D + SD + LL + Soil + EDX	2996758	34652.76	5.118	3.65	0.121261029	P-Delta Needed
14th Floor	D + SD + LL + Soil + EDX	3030289	34815.72	5.099	3.65	0.121590805	P-Delta Needed
13th Floor	D + SD + LL + Soil + EDX	3063820	35064.43	5.062	3.65	0.121178535	P-Delta Needed

12th Floor	D + SD + LL + Soil + EDX	3097351	35359.9	5.005	3.65	0.12011315	P-Delta Needed
11th Floor	D + SD + LL + Soil + EDX	3130881	35717.73	4.931	3.65	0.118419973	P-Delta Needed
10th Floor	D + SD + LL + Soil + EDX	3164412	36209.35	4.842	3.65	0.115932247	P-Delta Needed
9th Floor	D + SD + LL + Soil + EDX	3197943	36919.82	4.743	3.65	0.112556737	P-Delta Needed
8th Floor	D + SD + LL + Soil + EDX	3231474	37868.44	4.635	3.65	0.108362781	P-Delta Needed
7th Floor	D + SD + LL + Soil + EDX	3276583	39231.42	7.868	6.6	0.099565201	No P-Delta Needed
6th Floor	D + SD + LL + Soil + EDX	3310389	40064.45	3.867	3.72	0.08589166	No P-Delta Needed
5th Parking Floor	D + SD + LL + Soil + EDX	3389323	42457.96	4.066	3.58	0.090664692	No P-Delta Needed
4th Parking Floor	D + SD + LL + Soil + EDX	3452893	46265.01	2.152	2.8	0.057360727	No P-Delta Needed
3rd Parking Floor	D + SD + LL + Soil + EDX	3516195	49957.16	2.183	2.8	0.054874538	No P-Delta Needed
2nd Parking Floor	D + SD + LL + Soil + EDX	3579678	53068.9	2.139	2.8	0.05152959	No P-Delta Needed
1st Parking Floor	D + SD + LL + Soil + EDX	3651342	55594.45	2.135	3.8	0.036900759	No P-Delta Needed
Ground Floor	D + SD + LL + Soil + EDX	3732204	57939.08	1.296	3.35	0.024920341	No P-Delta Needed
1st Basement Floor	D + SD + LL + Soil + EDX	3811340	62425.41	0.905	3.05	0.018116113	No P-Delta Needed
2nd Basement Floor	D + SD + LL + Soil + EDX	3890288	68606.12	0.676	3.05	0.012567989	No P-Delta Needed
3rd Basement Floor	D + SD + LL + Soil + EDX	3969237	74586.01	0.465	3.05	0.008113398	No P-Delta Needed
4th Basement Floor	D + SD + LL + Soil + EDX	4048185	77400.57	0.269	3.05	0.004612843	No P-Delta Needed
5th Basement Floor	D + SD + LL + Soil + EDX	4127318	77720.5	0.061	3.05	0.001062093	No P-Delta Needed
6th Basement Floor	D + SD + LL + Soil + EDX	4181121	87516.04	0.015	3.05	0.000234961	No P-Delta Needed

Table 2.3.8.1 - P-Delta Check in X-Direction Before Modification

- P Delta Check – Y direction

Story	Output Case	P (kN)	Vy (kN)	Drift (mm)	H(m)	Stability Coefficient (θ)	Check
100th Floor	D + SD + LL + Soil + EDY	28785.82	2697.47	10.291	6.3	0.017431673	No P-Delta Needed
99th Floor	D + SD + LL + Soil + EDY	58701.86	4477.21	9.945	6.3	0.020697061	No P-Delta Needed
98th Floor	D + SD + LL + Soil + EDY	89780.15	6272.375	9.632	6	0.02297807	No P-Delta Needed
97th Floor	D + SD + LL + Soil + EDY	124562	7529.042	9.775	6	0.026953271	No P-Delta Needed
96th Floor	D + SD + LL + Soil + EDY	161753.3	8495.868	5.167	3.35	0.029365602	No P-Delta Needed
95th Floor	D + SD + LL + Soil + EDY	191645.8	9135.766	5.979	3.65	0.034362921	No P-Delta Needed
94th Floor	D + SD + LL + Soil + EDY	221538.4	9617.352	5.993	3.65	0.037822033	No P-Delta Needed
93rd Floor	D + SD + LL + Soil + EDY	251430.9	10002.6	6.041	3.65	0.041602701	No P-Delta Needed
92nd Floor	D + SD + LL + Soil + EDY	281323.4	10342.74	6.076	3.65	0.045278844	No P-Delta Needed
91st Floor	D + SD + LL + Soil + EDY	311216	10684.72	6.105	3.65	0.048718258	No P-Delta Needed
90th Floor	D + SD + LL + Soil + EDY	341108.5	11059.47	6.126	3.65	0.051765744	No P-Delta Needed
89th Floor	D + SD + LL + Soil + EDY	371001.1	11475.03	6.142	3.65	0.05440495	No P-Delta Needed
88th Floor	D + SD + LL + Soil + EDY	400893.6	11922.32	6.15	3.65	0.056656632	No P-Delta Needed
87th Floor	D + SD + LL + Soil + EDY	430786.1	12385.12	6.153	3.65	0.05863482	No P-Delta Needed
86th Floor	D + SD + LL + Soil + EDY	460678.7	12846.52	6.147	3.65	0.060392491	No P-Delta Needed
85th Floor	D + SD + LL + Soil + EDY	490571.2	13292.77	6.131	3.65	0.06199048	No P-Delta Needed
84th Floor	D + SD + LL + Soil + EDY	520463.8	13715.57	6.102	3.65	0.063438968	No P-Delta Needed
83rd Floor	D + SD + LL + Soil + EDY	550356.3	14112.7	6.055	3.65	0.064692688	No P-Delta Needed

82nd Floor	D + SD + LL + Soil + EDY	580248.9	14487.18	5.974	3.65	0.06555456	No P-Delta Needed
81st Floor	D + SD + LL + Soil + EDY	610141.4	14845.55	5.739	3.65	0.064621587	No P-Delta Needed
80th Floor	D + SD + LL + Soil + EDY	684709.4	15629.21	7.769	5.2	0.065453231	No P-Delta Needed
79th Floor	D + SD + LL + Soil + EDY	740993.8	16792.94	5.489	3.65	0.066357238	No P-Delta Needed
78th Floor	D + SD + LL + Soil + EDY	782668.5	17847.11	5.796	3.65	0.069637873	No P-Delta Needed
77th Floor	D + SD + LL + Soil + EDY	816199.5	18409.71	6.111	3.65	0.074228159	No P-Delta Needed
76th Floor	D + SD + LL + Soil + EDY	849730.4	18781.26	6.357	3.65	0.078798095	No P-Delta Needed
75th Floor	D + SD + LL + Soil + EDY	883261.3	18939.53	6.534	3.65	0.08348459	No P-Delta Needed
74th Floor	D + SD + LL + Soil + EDY	916792.2	18894.65	6.661	3.65	0.088547971	No P-Delta Needed
73rd Floor	D + SD + LL + Soil + EDY	950323.1	18689.62	6.759	3.65	0.094158669	No P-Delta Needed
72nd Floor	D + SD + LL + Soil + EDY	983854.1	18395.46	6.837	3.65	0.100182676	P-Delta Needed
71st Floor	D + SD + LL + Soil + EDY	1017385	18095.99	6.897	3.65	0.106235652	P-Delta Needed
70th Floor	D + SD + LL + Soil + EDY	1050916	17859.36	6.947	3.65	0.111997052	P-Delta Needed
69th Floor	D + SD + LL + Soil + EDY	1084447	17723.34	6.983	3.65	0.117060882	P-Delta Needed
68th Floor	D + SD + LL + Soil + EDY	1117978	17701.08	7.01	3.65	0.121299342	P-Delta Needed
67th Floor	D + SD + LL + Soil + EDY	1151509	17784.35	7.025	3.65	0.124618515	P-Delta Needed
66th Floor	D + SD + LL + Soil + EDY	1185040	17947.36	7.031	3.65	0.127191061	P-Delta Needed
65th Floor	D + SD + LL + Soil + EDY	1218570	18156.19	7.027	3.65	0.12921202	P-Delta Needed
64th Floor	D + SD + LL + Soil + EDY	1252101	18380.43	7.01	3.65	0.130830461	P-Delta Needed
63rd Floor	D + SD + LL + Soil + EDY	1285632	18601.82	6.984	3.65	0.132242989	P-Delta Needed
62nd Floor	D + SD + LL + Soil + EDY	1319163	18817.51	6.944	3.65	0.133368512	P-Delta Needed
61st Floor	D + SD + LL + Soil + EDY	1352694	19037.59	6.892	3.65	0.134165257	P-Delta Needed
60th Floor	D + SD + LL + Soil + EDY	1386225	19277.93	6.825	3.65	0.134456916	P-Delta Needed

59th Floor	D + SD + LL + Soil + EDY	1419756	19549.43	6.738	3.65	0.13406574	P-Delta Needed
58th Floor	D + SD + LL + Soil + EDY	1453287	19847.94	6.629	3.65	0.132981452	P-Delta Needed
57th Floor	D + SD + LL + Soil + EDY	1486818	20153.47	6.484	3.65	0.131056339	P-Delta Needed
56th Floor	D + SD + LL + Soil + EDY	1520349	20439.52	6.285	3.65	0.128081093	P-Delta Needed
55th Floor	D + SD + LL + Soil + EDY	1553880	20684.13	5.988	3.65	0.123244919	P-Delta Needed
54th Floor	D + SD + LL + Soil + EDY	1587411	20878.93	5.547	3.65	0.115543733	P-Delta Needed
53rd Floor	D + SD + LL + Soil + EDY	1644103	21171.82	4.86	3.65	0.103398524	P-Delta Needed
52nd Floor	D + SD + LL + Soil + EDY	1697004	21704.8	4.897	3.65	0.104897318	P-Delta Needed
51st Floor	D + SD + LL + Soil + EDY	1738679	22405.33	5.367	3.65	0.114105534	P-Delta Needed
50th Floor	D + SD + LL + Soil + EDY	1772210	22761.63	5.873	3.65	0.125279166	P-Delta Needed
49th Floor	D + SD + LL + Soil + EDY	1805741	22942.62	6.239	3.65	0.134534779	P-Delta Needed
48th Floor	D + SD + LL + Soil + EDY	1839272	22955.71	6.479	3.65	0.142223166	P-Delta Needed
47th Floor	D + SD + LL + Soil + EDY	1872803	22847.7	6.639	3.65	0.149093747	P-Delta Needed
46th Floor	D + SD + LL + Soil + EDY	1906334	22694.01	6.75	3.65	0.155345446	P-Delta Needed
45th Floor	D + SD + LL + Soil + EDY	1939865	22605.32	6.83	3.65	0.160578902	P-Delta Needed
44th Floor	D + SD + LL + Soil + EDY	1973395	22689.65	6.887	3.65	0.16410563	P-Delta Needed
43rd Floor	D + SD + LL + Soil + EDY	2006926	22884.29	6.925	3.65	0.166387595	P-Delta Needed
42nd Floor	D + SD + LL + Soil + EDY	2040457	23101.55	6.946	3.65	0.168084763	P-Delta Needed
41st Floor	D + SD + LL + Soil + EDY	2073988	23293.53	6.95	3.65	0.169536391	P-Delta Needed
40th Floor	D + SD + LL + Soil + EDY	2107519	23438.45	6.941	3.65	0.170990411	P-Delta Needed
39th Floor	D + SD + LL + Soil + EDY	2141050	23540.7	6.917	3.65	0.172358374	P-Delta Needed
38th Floor	D + SD + LL + Soil + EDY	2174581	23627.55	6.879	3.65	0.173455988	P-Delta Needed
37th Floor	D + SD + LL + Soil + EDY	2208112	23742.39	6.827	3.65	0.173953699	P-Delta Needed

36th Floor	D + SD + LL + Soil + EDY	2241643	23942.36	6.761	3.65	0.173427345	P-Delta Needed
35th Floor	D + SD + LL + Soil + EDY	2275174	24277.64	6.678	3.65	0.171459526	P-Delta Needed
34th Floor	D + SD + LL + Soil + EDY	2308705	24710.65	6.58	3.65	0.168429122	P-Delta Needed
33rd Floor	D + SD + LL + Soil + EDY	2342235	25148.82	6.461	3.65	0.164861717	P-Delta Needed
32nd Floor	D + SD + LL + Soil + EDY	2375766	25530.06	6.32	3.65	0.161129871	P-Delta Needed
31st Floor	D + SD + LL + Soil + EDY	2409297	25831.84	6.153	3.65	0.157227733	P-Delta Needed
30th Floor	D + SD + LL + Soil + EDY	2442828	26062.78	5.953	3.65	0.152867494	P-Delta Needed
29th Floor	D + SD + LL + Soil + EDY	2476359	26253.66	5.705	3.65	0.147430213	P-Delta Needed
28th Floor	D + SD + LL + Soil + EDY	2509890	26452.92	5.383	3.65	0.139930542	P-Delta Needed
27th Floor	D + SD + LL + Soil + EDY	2543421	26721.29	4.925	3.65	0.128432284	P-Delta Needed
26th Floor	D + SD + LL + Soil + EDY	2576952	27076.07	4.225	3.65	0.110167782	P-Delta Needed
25th Floor	D + SD + LL + Soil + EDY	2633935	27774.43	3.317	3.65	0.086181199	No P-Delta Needed
24th Floor	D + SD + LL + Soil + EDY	2686836	28936.74	3.326	3.65	0.084609833	No P-Delta Needed
23rd Floor	D + SD + LL + Soil + EDY	2728511	30321.62	3.879	3.65	0.095631305	No P-Delta Needed
22nd Floor	D + SD + LL + Soil + EDY	2762041	30923.88	4.499	3.65	0.11009291	P-Delta Needed
21st Floor	D + SD + LL + Soil + EDY	2795572	31095.1	4.936	3.65	0.121579708	P-Delta Needed
20th Floor	D + SD + LL + Soil + EDY	2829103	30908.48	5.201	3.65	0.130426297	P-Delta Needed
19th Floor	D + SD + LL + Soil + EDY	2862634	30572.56	5.354	3.65	0.137347118	P-Delta Needed
18th Floor	D + SD + LL + Soil + EDY	2896165	30347.92	5.442	3.65	0.142285298	P-Delta Needed
17th Floor	D + SD + LL + Soil + EDY	2929696	30259.21	5.486	3.65	0.145521768	P-Delta Needed
16th Floor	D + SD + LL + Soil + EDY	2963227	30237.23	5.497	3.65	0.147589593	P-Delta Needed
15th Floor	D + SD + LL + Soil + EDY	2996758	30279.26	5.483	3.65	0.148672908	P-Delta Needed
14th Floor	D + SD + LL + Soil + EDY	3030289	30433.79	5.443	3.65	0.148481883	P-Delta Needed

13th Floor	D + SD + LL + Soil + EDY	3063820	30821.84	5.379	3.65	0.146491787	P-Delta Needed
12th Floor	D + SD + LL + Soil + EDY	3097351	31370.69	5.292	3.65	0.14315063	P-Delta Needed
11th Floor	D + SD + LL + Soil + EDY	3130881	31808.38	5.178	3.65	0.139635008	P-Delta Needed
10th Floor	D + SD + LL + Soil + EDY	3164412	32042.73	5.04	3.65	0.136364466	P-Delta Needed
9th Floor	D + SD + LL + Soil + EDY	3197943	32199.91	4.875	3.65	0.132647147	P-Delta Needed
8th Floor	D + SD + LL + Soil + EDY	3231474	32553.65	4.68	3.65	0.127278176	P-Delta Needed
7th Floor	D + SD + LL + Soil + EDY	3276583	33888.85	7.861	6.6	0.115159075	P-Delta Needed
6th Floor	D + SD + LL + Soil + EDY	3310389	34899.46	4.32	3.72	0.110154209	P-Delta Needed
5th Parking Floor	D + SD + LL + Soil + EDY	3389323	36801.08	2.471	3.58	0.063568514	No P-Delta Needed
4th Parking Floor	D + SD + LL + Soil + EDY	3452893	40024.58	2.249	2.8	0.069292739	No P-Delta Needed
3rd Parking Floor	D + SD + LL + Soil + EDY	3516195	43408.92	1.652	2.8	0.04779098	No P-Delta Needed
2nd Parking Floor	D + SD + LL + Soil + EDY	3579678	46692.22	1.511	2.8	0.041371947	No P-Delta Needed
1st Parking Floor	D + SD + LL + Soil + EDY	3651342	49694	1.355	3.8	0.026200175	No P-Delta Needed
Ground Floor	D + SD + LL + Soil + EDY	3732204	52134.22	0.922	3.35	0.019702831	No P-Delta Needed
1st Basement Floor	D + SD + LL + Soil + EDY	3811340	56040.56	0.5	3.05	0.011149242	No P-Delta Needed
2nd Basement Floor	D + SD + LL + Soil + EDY	3890288	61919.77	0.349	3.05	0.007189157	No P-Delta Needed
3rd Basement Floor	D + SD + LL + Soil + EDY	3969237	68349.57	0.261	3.05	0.004969491	No P-Delta Needed
4th Basement Floor	D + SD + LL + Soil + EDY	4048185	71752.35	0.119	3.05	0.00220126	No P-Delta Needed
5th Basement Floor	D + SD + LL + Soil + EDY	4127318	73629	-0.223	3.05	-0.004098492	No P-Delta Needed
6th Basement Floor	D + SD + LL + Soil + EDY	4181121	89649.46	0.046	3.05	0.000703401	No P-Delta Needed

Table 2.3.8.2 - P-Delta Check in Y-Direction Before Modification

- **P Delta after modifications – X direction**

Story	Output Case	P (kN)	Vx (kN)	Drift (mm)	H(m)	Stability Coefficient (θ)	Check
100th Floor	D + SD + LL + Soil + EDX	32160.4428	4477.559	6.769	6.3	0.007717287	No P-Delta Needed
99th Floor	D + SD + LL + Soil + EDX	65450.2446	7634.9355	5.915	6.3	0.008048595	No P-Delta Needed
98th Floor	D + SD + LL + Soil + EDX	98412.1937	10649.2399	6.053	6	0.009322872	No P-Delta Needed
97th Floor	D + SD + LL + Soil + EDX	135081.6481	12711.1179	5.893	6	0.010437531	No P-Delta Needed
96th Floor	D + SD + LL + Soil + EDX	174313.2501	14249.4784	3.504	3.35	0.012795307	No P-Delta Needed
95th Floor	D + SD + LL + Soil + EDX	207141.1019	15243.6407	3.654	3.65	0.013603581	No P-Delta Needed
94th Floor	D + SD + LL + Soil + EDX	239968.9701	15962.4256	3.69	3.65	0.015198114	No P-Delta Needed
93rd Floor	D + SD + LL + Soil + EDX	272796.8381	16502.9459	3.723	3.65	0.016860794	No P-Delta Needed
92nd Floor	D + SD + LL + Soil + EDX	305624.7057	16987.4647	3.747	3.65	0.018469312	No P-Delta Needed
91st Floor	D + SD + LL + Soil + EDX	338452.5734	17453.7435	3.767	3.65	0.02001299	No P-Delta Needed
90th Floor	D + SD + LL + Soil + EDX	371280.4412	17920.2782	3.781	3.65	0.021462047	No P-Delta Needed
89th Floor	D + SD + LL + Soil + EDX	404108.3095	18421.9184	3.793	3.65	0.022795699	No P-Delta Needed
88th Floor	D + SD + LL + Soil + EDX	436936.1776	18979.6299	3.799	3.65	0.023961097	No P-Delta Needed
87th Floor	D + SD + LL + Soil + EDX	469764.0453	19593.9633	3.803	3.65	0.024979914	No P-Delta Needed
86th Floor	D + SD + LL + Soil + EDX	502591.9129	20247.8895	3.8	3.65	0.02584202	No P-Delta Needed
85th Floor	D + SD + LL + Soil + EDX	535419.7809	20914.0464	3.793	3.65	0.026603962	No P-Delta Needed
84th Floor	D + SD + LL + Soil + EDX	568247.6488	21563.3064	3.78	3.65	0.027291112	No P-Delta Needed
83rd Floor	D + SD + LL + Soil + EDX	601075.5173	22172.3985	3.76	3.65	0.027926168	No P-Delta Needed

82nd Floor	D + SD + LL + Soil + EDX	633903.3854	22729.604	3.725	3.65	0.028461948	No P-Delta Needed
81st Floor	D + SD + LL + Soil + EDX	666731.2533	23237.694	3.528	3.65	0.027732786	No P-Delta Needed
80th Floor	D + SD + LL + Soil + EDX	744448.7462	24245.4161	4.633	5.2	0.027356726	No P-Delta Needed
79th Floor	D + SD + LL + Soil + EDX	804007.6365	25708.8633	3.336	3.65	0.028583176	No P-Delta Needed
78th Floor	D + SD + LL + Soil + EDX	850829.4465	27145.8831	3.695	3.65	0.031729273	No P-Delta Needed
77th Floor	D + SD + LL + Soil + EDX	891146.8821	28011.5585	3.843	3.65	0.033495737	No P-Delta Needed
76th Floor	D + SD + LL + Soil + EDX	931463.2479	28660.0383	3.934	3.65	0.035029219	No P-Delta Needed
75th Floor	D + SD + LL + Soil + EDX	971780.6579	29051.488	4.009	3.65	0.036740331	No P-Delta Needed
74th Floor	D + SD + LL + Soil + EDX	1012098.381	29176.4382	4.075	3.65	0.038728012	No P-Delta Needed
73rd Floor	D + SD + LL + Soil + EDX	1052416.105	29051.5916	4.135	3.65	0.041039325	No P-Delta Needed
72nd Floor	D + SD + LL + Soil + EDX	1092731.636	28721.0028	4.183	3.65	0.043602252	No P-Delta Needed
71st Floor	D + SD + LL + Soil + EDX	1133049.149	28260.6921	4.224	3.65	0.046397759	No P-Delta Needed
70th Floor	D + SD + LL + Soil + EDX	1173365.354	27774.9524	4.256	3.65	0.049259352	No P-Delta Needed
69th Floor	D + SD + LL + Soil + EDX	1213680.91	27335.7103	4.28	3.65	0.052062505	No P-Delta Needed
68th Floor	D + SD + LL + Soil + EDX	1254000.166	26979.3515	4.298	3.65	0.054731788	No P-Delta Needed
67th Floor	D + SD + LL + Soil + EDX	1294315.73	26748.0283	4.309	3.65	0.057125772	No P-Delta Needed
66th Floor	D + SD + LL + Soil + EDX	1334632.898	26676.2016	4.315	3.65	0.059146053	No P-Delta Needed
65th Floor	D + SD + LL + Soil + EDX	1374948.193	26776.0454	4.313	3.65	0.060677334	No P-Delta Needed
64th Floor	D + SD + LL + Soil + EDX	1415265.561	27033.5045	4.307	3.65	0.061775689	No P-Delta Needed
63rd Floor	D + SD + LL + Soil + EDX	1455583.916	27413.1915	4.293	3.65	0.062451902	No P-Delta Needed
62nd Floor	D + SD + LL + Soil + EDX	1495901.044	27870.164	4.272	3.65	0.062820548	No P-Delta Needed
61st Floor	D + SD + LL + Soil + EDX	1536218.312	28367.89	4.243	3.65	0.062951498	No P-Delta Needed
60th Floor	D + SD + LL + Soil + EDX	1576536.391	28895.003	4.208	3.65	0.062901952	No P-Delta Needed

59th Floor	D + SD + LL + Soil + EDX	1616853.392	29427.5171	4.162	3.65	0.062650744	No P-Delta Needed
58th Floor	D + SD + LL + Soil + EDX	1657168.599	29908.7476	4.107	3.65	0.06234481	No P-Delta Needed
57th Floor	D + SD + LL + Soil + EDX	1697485.165	30307.9821	4.046	3.65	0.062084327	No P-Delta Needed
56th Floor	D + SD + LL + Soil + EDX	1737800.553	30624.3673	3.99	3.65	0.062031579	No P-Delta Needed
55th Floor	D + SD + LL + Soil + EDX	1778119.336	30873.9113	3.914	3.65	0.061758567	No P-Delta Needed
54th Floor	D + SD + LL + Soil + EDX	1818437.567	31080.3326	3.585	3.65	0.057465744	No P-Delta Needed
53rd Floor	D + SD + LL + Soil + EDX	1880281.06	31393.5006	3.044	3.65	0.049949921	No P-Delta Needed
52nd Floor	D + SD + LL + Soil + EDX	1938334.189	32018.1961	2.991	3.65	0.049608414	No P-Delta Needed
51st Floor	D + SD + LL + Soil + EDX	1985155.93	32878.5963	3.728	3.65	0.061668644	No P-Delta Needed
50th Floor	D + SD + LL + Soil + EDX	2025473.109	33418.5194	3.991	3.65	0.066271704	No P-Delta Needed
49th Floor	D + SD + LL + Soil + EDX	2065790.348	33821.5357	4.061	3.65	0.06795682	No P-Delta Needed
48th Floor	D + SD + LL + Soil + EDX	2106107.155	34053.5876	4.127	3.65	0.069929302	No P-Delta Needed
47th Floor	D + SD + LL + Soil + EDX	2146422.609	34097.9315	4.195	3.65	0.072347962	No P-Delta Needed
46th Floor	D + SD + LL + Soil + EDX	2186737.794	33979.4735	4.254	3.65	0.075004045	No P-Delta Needed
45th Floor	D + SD + LL + Soil + EDX	2227053.298	33757.9061	4.299	3.65	0.077701551	No P-Delta Needed
44th Floor	D + SD + LL + Soil + EDX	2267368.338	33509.4742	4.333	3.65	0.080324918	No P-Delta Needed
43rd Floor	D + SD + LL + Soil + EDX	2307683.714	33307.3627	4.357	3.65	0.082704804	No P-Delta Needed
42nd Floor	D + SD + LL + Soil + EDX	2348000.99	33205.061	4.371	3.65	0.084680212	No P-Delta Needed
41st Floor	D + SD + LL + Soil + EDX	2388316.401	33229.8223	4.376	3.65	0.086168452	No P-Delta Needed
40th Floor	D + SD + LL + Soil + EDX	2428632.636	33397.0038	4.373	3.65	0.087124627	No P-Delta Needed
39th Floor	D + SD + LL + Soil + EDX	2468945.026	33719.6707	4.362	3.65	0.087502587	No P-Delta Needed
38th Floor	D + SD + LL + Soil + EDX	2509260.929	34132.5352	4.34	3.65	0.087412621	No P-Delta Needed
37th Floor	D + SD + LL + Soil + EDX	2549576.81	34550.8121	4.312	3.65	0.087175757	No P-Delta Needed

36th Floor	D + SD + LL + Soil + EDX	2589892.103	34939.4777	4.273	3.65	0.086777132	No P-Delta Needed
35th Floor	D + SD + LL + Soil + EDX	2630207.829	35300.9918	4.225	3.65	0.086245612	No P-Delta Needed
34th Floor	D + SD + LL + Soil + EDX	2670523.23	35660.2286	4.168	3.65	0.085515942	No P-Delta Needed
33rd Floor	D + SD + LL + Soil + EDX	2710839.03	36048.8089	4.098	3.65	0.084429049	No P-Delta Needed
32nd Floor	D + SD + LL + Soil + EDX	2751155.472	36492.2417	4.018	3.65	0.082991126	No P-Delta Needed
31st Floor	D + SD + LL + Soil + EDX	2791470.622	37014.8021	3.924	3.65	0.081076271	No P-Delta Needed
30th Floor	D + SD + LL + Soil + EDX	2831785.985	37670.8921	3.816	3.65	0.078590495	No P-Delta Needed
29th Floor	D + SD + LL + Soil + EDX	2872102.75	38413.9773	3.695	3.65	0.075688914	No P-Delta Needed
28th Floor	D + SD + LL + Soil + EDX	2912417.946	39116.8432	3.577	3.65	0.072965233	No P-Delta Needed
27th Floor	D + SD + LL + Soil + EDX	2952733.05	39720.5496	3.44	3.65	0.070060707	No P-Delta Needed
26th Floor	D + SD + LL + Soil + EDX	2993048.856	40214.4314	3.094	3.65	0.063089824	No P-Delta Needed
25th Floor	D + SD + LL + Soil + EDX	3054892.312	40842.0474	2.157	3.65	0.044202383	No P-Delta Needed
24th Floor	D + SD + LL + Soil + EDX	3112945.348	41865.7357	2.037	3.65	0.041496449	No P-Delta Needed
23rd Floor	D + SD + LL + Soil + EDX	3159767.013	43171.1156	2.968	3.65	0.059515875	No P-Delta Needed
22nd Floor	D + SD + LL + Soil + EDX	3200083.204	43938.4056	3.364	3.65	0.067124351	No P-Delta Needed
21st Floor	D + SD + LL + Soil + EDX	3240398.141	44469.0805	3.416	3.65	0.068196992	No P-Delta Needed
20th Floor	D + SD + LL + Soil + EDX	3280713.712	44752.2172	3.475	3.65	0.069793619	No P-Delta Needed
19th Floor	D + SD + LL + Soil + EDX	3321028.907	44799.584	3.53	3.65	0.071693621	No P-Delta Needed
18th Floor	D + SD + LL + Soil + EDX	3361344.685	44675.9269	3.567	3.65	0.073527487	No P-Delta Needed
17th Floor	D + SD + LL + Soil + EDX	3401659.629	44483.721	3.586	3.65	0.075128924	No P-Delta Needed
16th Floor	D + SD + LL + Soil + EDX	3441974.771	44325.7078	3.587	3.65	0.076311592	No P-Delta Needed
15th Floor	D + SD + LL + Soil + EDX	3482289.721	44295.0242	3.57	3.65	0.076892735	No P-Delta Needed
14th Floor	D + SD + LL + Soil + EDX	3522604.63	44481.1876	3.537	3.65	0.076741401	No P-Delta Needed

13th Floor	D + SD + LL + Soil + EDX	3562919.676	44834.0258	3.49	3.65	0.075985523	No P-Delta Needed
12th Floor	D + SD + LL + Soil + EDX	3603234.656	45257.7172	3.426	3.65	0.074729899	No P-Delta Needed
11th Floor	D + SD + LL + Soil + EDX	3643549.548	45753.211	3.346	3.65	0.073002238	No P-Delta Needed
10th Floor	D + SD + LL + Soil + EDX	3683864.642	46387.0091	3.248	3.65	0.070669245	No P-Delta Needed
9th Floor	D + SD + LL + Soil + EDX	3724179.825	47236.3265	3.131	3.65	0.067630832	No P-Delta Needed
8th Floor	D + SD + LL + Soil + EDX	3764494.651	48388.6575	2.996	3.65	0.063857521	No P-Delta Needed
7th Floor	D + SD + LL + Soil + EDX	3822151.77	50272.7297	4.925	6.6	0.056733263	No P-Delta Needed
6th Floor	D + SD + LL + Soil + EDX	3862878.245	51528.712	2.247	3.72	0.045281611	No P-Delta Needed
5th Parking Floor	D + SD + LL + Soil + EDX	3948445.067	54008.2784	2.03	3.58	0.041455177	No P-Delta Needed
4th Parking Floor	D + SD + LL + Soil + EDX	4017547.554	57471.2564	1.241	2.8	0.030983044	No P-Delta Needed
3rd Parking Floor	D + SD + LL + Soil + EDX	4086635.707	61098.1678	1.141	2.8	0.027256203	No P-Delta Needed
2nd Parking Floor	D + SD + LL + Soil + EDX	4155931.423	64772.3409	1.046	2.8	0.023969139	No P-Delta Needed
1st Parking Floor	D + SD + LL + Soil + EDX	4235012.693	68094.2576	1.193	3.8	0.019525451	No P-Delta Needed
Ground Floor	D + SD + LL + Soil + EDX	4322248.718	70400.3821	0.788	3.35	0.014441628	No P-Delta Needed
1st Basement Floor	D + SD + LL + Soil + EDX	4407194.641	73742.3963	0.592	3.05	0.011600237	No P-Delta Needed
2nd Basement Floor	D + SD + LL + Soil + EDX	4491953.373	78936.8227	0.477	3.05	0.008899675	No P-Delta Needed
3rd Basement Floor	D + SD + LL + Soil + EDX	4576712.077	84601.856	0.361	3.05	0.006402964	No P-Delta Needed
4th Basement Floor	D + SD + LL + Soil + EDX	4661470.951	87619.376	0.24	3.05	0.004186337	No P-Delta Needed
5th Basement Floor	D + SD + LL + Soil + EDX	4746416.839	88116.6927	0.157	3.05	0.002772729	No P-Delta Needed
6th Basement Floor	D + SD + LL + Soil + EDX	4801948.073	98704.795	0	-18.6	0	No P-Delta Needed

Table 2.3.8.3 - P-Delta Check in X-Direction After Modification

- **P delta after modifications – Y direction**

Story	Output Case	P (kN)	Vy (kN)	Drift (mm)	H(m)	Stability Coefficient (θ)	Check
100th Floor	D + SD + LL + Soil + EDX	32160.44	4290.44	6.322	6.3	0.007522015	No P-Delta Needed
99th Floor	D + SD + LL + Soil + EDX	65450.24	7192.595	6.09	6.3	0.008796349	No P-Delta Needed
98th Floor	D + SD + LL + Soil + EDX	98412.19	9742.045	5.811	6	0.009783594	No P-Delta Needed
97th Floor	D + SD + LL + Soil + EDX	135081.6	11306.67	6.01	6	0.011966993	No P-Delta Needed
96th Floor	D + SD + LL + Soil + EDX	174313.3	12248.25	3.194	3.35	0.013568957	No P-Delta Needed
95th Floor	D + SD + LL + Soil + EDX	207141.1	12872.83	3.607	3.65	0.015901769	No P-Delta Needed
94th Floor	D + SD + LL + Soil + EDX	239969	13359.88	3.668	3.65	0.018050488	No P-Delta Needed
93rd Floor	D + SD + LL + Soil + EDX	272796.8	13841.96	3.697	3.65	0.019961739	No P-Delta Needed
92nd Floor	D + SD + LL + Soil + EDX	305624.7	14358.31	3.724	3.65	0.02171711	No P-Delta Needed
91st Floor	D + SD + LL + Soil + EDX	338452.6	14905.62	3.745	3.65	0.023297355	No P-Delta Needed
90th Floor	D + SD + LL + Soil + EDX	371280.4	15482.19	3.76	3.65	0.024703847	No P-Delta Needed
89th Floor	D + SD + LL + Soil + EDX	404108.3	16076.86	3.771	3.65	0.0259693	No P-Delta Needed
88th Floor	D + SD + LL + Soil + EDX	436936.2	16669.98	3.775	3.65	0.027108603	No P-Delta Needed
87th Floor	D + SD + LL + Soil + EDX	469764	17240.96	3.775	3.65	0.028180087	No P-Delta Needed
86th Floor	D + SD + LL + Soil + EDX	502591.9	17775.76	3.769	3.65	0.029195807	No P-Delta Needed
85th Floor	D + SD + LL + Soil + EDX	535419.8	18271.71	3.755	3.65	0.030146175	No P-Delta Needed
84th Floor	D + SD + LL + Soil + EDX	568247.6	18740	3.732	3.65	0.031003931	No P-Delta Needed

83rd Floor	D + SD + LL + Soil + EDX	601075.5	19206.81	3.696	3.65	0.031689321	No P-Delta Needed
82nd Floor	D + SD + LL + Soil + EDX	633903.4	19707.05	3.642	3.65	0.03209582	No P-Delta Needed
81st Floor	D + SD + LL + Soil + EDX	666731.3	20254.67	3.545	3.65	0.031970466	No P-Delta Needed
80th Floor	D + SD + LL + Soil + EDX	744447.5	21444.91	4.68	5.2	0.03124297	No P-Delta Needed
79th Floor	D + SD + LL + Soil + EDX	804006.3	23009.14	3.273	3.65	0.031333746	No P-Delta Needed
78th Floor	D + SD + LL + Soil + EDX	850828.1	24408.79	3.452	3.65	0.032966546	No P-Delta Needed
77th Floor	D + SD + LL + Soil + EDX	891145.7	25154.84	3.667	3.65	0.035591403	No P-Delta Needed
76th Floor	D + SD + LL + Soil + EDX	931462.1	25590.1	3.821	3.65	0.038104603	No P-Delta Needed
75th Floor	D + SD + LL + Soil + EDX	971779.5	25694.08	3.926	3.65	0.040681044	No P-Delta Needed
74th Floor	D + SD + LL + Soil + EDX	1012097	25494.91	4.004	3.65	0.043548181	No P-Delta Needed
73rd Floor	D + SD + LL + Soil + EDX	1052415	25065.76	4.068	3.65	0.046794439	No P-Delta Needed
72nd Floor	D + SD + LL + Soil + EDX	1092731	24526.17	4.119	3.65	0.050278498	No P-Delta Needed
71st Floor	D + SD + LL + Soil + EDX	1133048	24038.6	4.161	3.65	0.053733365	No P-Delta Needed
70th Floor	D + SD + LL + Soil + EDX	1173365	23717.52	4.193	3.65	0.056832356	No P-Delta Needed
69th Floor	D + SD + LL + Soil + EDX	1213680	23556.29	4.218	3.65	0.059540293	No P-Delta Needed
68th Floor	D + SD + LL + Soil + EDX	1253999	23522.33	4.235	3.65	0.061855401	No P-Delta Needed
67th Floor	D + SD + LL + Soil + EDX	1294315	23582.82	4.245	3.65	0.063830614	No P-Delta Needed
66th Floor	D + SD + LL + Soil + EDX	1334632	23699.56	4.248	3.65	0.065540987	No P-Delta Needed
65th Floor	D + SD + LL + Soil + EDX	1374948	23837.31	4.245	3.65	0.067083205	No P-Delta Needed
64th Floor	D + SD + LL + Soil + EDX	1415265	23975.97	4.233	3.65	0.068456854	No P-Delta Needed
63rd Floor	D + SD + LL + Soil + EDX	1455583	24119.02	4.215	3.65	0.069691878	No P-Delta Needed
62nd Floor	D + SD + LL + Soil + EDX	1495901	24297.69	4.189	3.65	0.070657021	No P-Delta Needed
61st Floor	D + SD + LL + Soil + EDX	1536218	24572.4	4.155	3.65	0.071167791	No P-Delta Needed

60th Floor	D + SD + LL + Soil + EDX	1576536	25005.47	4.113	3.65	0.071045191	No P-Delta Needed
59th Floor	D + SD + LL + Soil + EDX	1616853	25579.94	4.063	3.65	0.070359873	No P-Delta Needed
58th Floor	D + SD + LL + Soil + EDX	1657168	26208.74	4.001	3.65	0.069310048	No P-Delta Needed
57th Floor	D + SD + LL + Soil + EDX	1697485	26806.62	3.927	3.65	0.068128996	No P-Delta Needed
56th Floor	D + SD + LL + Soil + EDX	1737800	27308.28	3.831	3.65	0.066792052	No P-Delta Needed
55th Floor	D + SD + LL + Soil + EDX	1778119	27671.64	3.691	3.65	0.064979618	No P-Delta Needed
54th Floor	D + SD + LL + Soil + EDX	1818438	27881.73	3.487	3.65	0.062307123	No P-Delta Needed
53rd Floor	D + SD + LL + Soil + EDX	1880281	28038.05	3.043	3.65	0.055909289	No P-Delta Needed
52nd Floor	D + SD + LL + Soil + EDX	1938334	28352.07	3.026	3.65	0.056678701	No P-Delta Needed
51st Floor	D + SD + LL + Soil + EDX	1985156	28837.92	3.331	3.65	0.062822097	No P-Delta Needed
50th Floor	D + SD + LL + Soil + EDX	2025473	29118.68	3.661	3.65	0.069768859	No P-Delta Needed
49th Floor	D + SD + LL + Soil + EDX	2065790	29301.11	3.868	3.65	0.074712922	No P-Delta Needed
48th Floor	D + SD + LL + Soil + EDX	2106107	29401.18	3.994	3.65	0.078384634	No P-Delta Needed
47th Floor	D + SD + LL + Soil + EDX	2146423	29430.12	4.082	3.65	0.081564907	No P-Delta Needed
46th Floor	D + SD + LL + Soil + EDX	2186738	29399.01	4.147	3.65	0.08450943	No P-Delta Needed
45th Floor	D + SD + LL + Soil + EDX	2227053	29324.1	4.2	3.65	0.087390126	No P-Delta Needed
44th Floor	D + SD + LL + Soil + EDX	2267368	29226.52	4.24	3.65	0.090119337	No P-Delta Needed
43rd Floor	D + SD + LL + Soil + EDX	2307684	29130.72	4.269	3.65	0.092652775	No P-Delta Needed
42nd Floor	D + SD + LL + Soil + EDX	2348001	29065.12	4.29	3.65	0.094949043	No P-Delta Needed
41st Floor	D + SD + LL + Soil + EDX	2388317	29064.57	4.299	3.65	0.096783782	No P-Delta Needed
40th Floor	D + SD + LL + Soil + EDX	2428633	29168.21	4.298	3.65	0.098045044	No P-Delta Needed
39th Floor	D + SD + LL + Soil + EDX	2468945	29399.54	4.29	3.65	0.098704146	No P-Delta Needed
38th Floor	D + SD + LL + Soil + EDX	2509261	29739.73	4.27	3.65	0.098706054	No P-Delta Needed

37th Floor	D + SD + LL + Soil + EDX	2549577	30138.14	4.24	3.65	0.098270856	No P-Delta Needed
36th Floor	D + SD + LL + Soil + EDX	2589892	30543.97	4.199	3.65	0.097545945	No P-Delta Needed
35th Floor	D + SD + LL + Soil + EDX	2630208	30923.53	4.149	3.65	0.096683334	No P-Delta Needed
34th Floor	D + SD + LL + Soil + EDX	2670523	31266.01	4.087	3.65	0.095639136	No P-Delta Needed
33rd Floor	D + SD + LL + Soil + EDX	2710839	31586.8	4.012	3.65	0.094333562	No P-Delta Needed
32nd Floor	D + SD + LL + Soil + EDX	2751156	31931.75	3.927	3.65	0.092695881	No P-Delta Needed
31st Floor	D + SD + LL + Soil + EDX	2791471	32377.29	3.828	3.65	0.090421493	No P-Delta Needed
30th Floor	D + SD + LL + Soil + EDX	2831786	32978.86	3.714	3.65	0.087372318	No P-Delta Needed
29th Floor	D + SD + LL + Soil + EDX	2872103	33690.58	3.58	3.65	0.083614513	No P-Delta Needed
28th Floor	D + SD + LL + Soil + EDX	2912418	34414.12	3.416	3.65	0.079203087	No P-Delta Needed
27th Floor	D + SD + LL + Soil + EDX	2952733	35071.79	3.181	3.65	0.073373139	No P-Delta Needed
26th Floor	D + SD + LL + Soil + EDX	2993049	35621.65	2.777	3.65	0.063926781	No P-Delta Needed
25th Floor	D + SD + LL + Soil + EDX	3054893	36296.27	2.165	3.65	0.049922801	No P-Delta Needed
24th Floor	D + SD + LL + Soil + EDX	3112946	37296.72	2.168	3.65	0.049575525	No P-Delta Needed
23rd Floor	D + SD + LL + Soil + EDX	3159767	38503.49	2.567	3.65	0.057714914	No P-Delta Needed
22nd Floor	D + SD + LL + Soil + EDX	3200083	39135.01	2.985	3.65	0.066872469	No P-Delta Needed
21st Floor	D + SD + LL + Soil + EDX	3240398	39482.86	3.239	3.65	0.072829591	No P-Delta Needed
20th Floor	D + SD + LL + Soil + EDX	3280714	39593.94	3.385	3.65	0.076843197	No P-Delta Needed
19th Floor	D + SD + LL + Soil + EDX	3321029	39535.86	3.474	3.65	0.079949993	No P-Delta Needed
18th Floor	D + SD + LL + Soil + EDX	3361345	39368.63	3.531	3.65	0.082597649	No P-Delta Needed
17th Floor	D + SD + LL + Soil + EDX	3401660	39165.9	3.565	3.65	0.084829994	No P-Delta Needed
16th Floor	D + SD + LL + Soil + EDX	3441975	39016.89	3.584	3.65	0.086622408	No P-Delta Needed
15th Floor	D + SD + LL + Soil + EDX	3482290	38996.88	3.585	3.65	0.087706411	No P-Delta Needed

14th Floor	D + SD + LL + Soil + EDX	3522605	39147.85	3.57	3.65	0.088009868	No P-Delta Needed
13th Floor	D + SD + LL + Soil + EDX	3562920	39475.04	3.539	3.65	0.087512718	No P-Delta Needed
12th Floor	D + SD + LL + Soil + EDX	3603235	39917.61	3.491	3.65	0.086334624	No P-Delta Needed
11th Floor	D + SD + LL + Soil + EDX	3643550	40383.27	3.42	3.65	0.084538872	No P-Delta Needed
10th Floor	D + SD + LL + Soil + EDX	3683865	40878.61	3.328	3.65	0.082167116	No P-Delta Needed
9th Floor	D + SD + LL + Soil + EDX	3724180	41530.08	3.206	3.65	0.078765955	No P-Delta Needed
8th Floor	D + SD + LL + Soil + EDX	3764495	42482.06	3.057	3.65	0.074217058	No P-Delta Needed
7th Floor	D + SD + LL + Soil + EDX	3822152	44191.4	5.153	6.6	0.067528398	No P-Delta Needed
6th Floor	D + SD + LL + Soil + EDX	3862879	45441.96	2.84	3.72	0.064897718	No P-Delta Needed
5th Parking Floor	D + SD + LL + Soil + EDX	3948445	47439.26	2.082	3.58	0.048404519	No P-Delta Needed
4th Parking Floor	D + SD + LL + Soil + EDX	4017548	50340.4	1.382	2.8	0.03939076	No P-Delta Needed
3rd Parking Floor	D + SD + LL + Soil + EDX	4086636	53581.86	1.189	2.8	0.032387093	No P-Delta Needed
2nd Parking Floor	D + SD + LL + Soil + EDX	4155932	57024.53	1.072	2.8	0.027902521	No P-Delta Needed
1st Parking Floor	D + SD + LL + Soil + EDX	4235013	60565.24	1.015	3.8	0.018677285	No P-Delta Needed
Ground Floor	D + SD + LL + Soil + EDX	4322249	63402.98	0.712	3.35	0.014488895	No P-Delta Needed
1st Basement Floor	D + SD + LL + Soil + EDX	4407195	66715.6	0.441	3.05	0.009551544	No P-Delta Needed
2nd Basement Floor	D + SD + LL + Soil + EDX	4491954	71209.58	0.333	3.05	0.006887177	No P-Delta Needed
3rd Basement Floor	D + SD + LL + Soil + EDX	4576712	76583.19	0.268	3.05	0.005251158	No P-Delta Needed
4th Basement Floor	D + SD + LL + Soil + EDX	4661471	80214.63	0.201	3.05	0.003829708	No P-Delta Needed
5th Basement Floor	D + SD + LL + Soil + EDX	4746417	81527.56	-0.084	3.05	-0.001603396	No P-Delta Needed
6th Basement Floor	D + SD + LL + Soil + EDX	4801948	93029.94	0	-18.6	0	No P-Delta Needed

Table 2.3.8.4 - P-Delta Check in Y-Direction After Modification

2.3.9 Base Shear Check

To find the Base shear resulting from seismic forces, Apply the following equation:

$V = C_s * W$

- W is the effective seismic weight.
- Cs is the seismic response coefficient.

$$C_s = \frac{S_{DS}}{\frac{R}{I_e}}$$

$$C_s = \frac{0.44}{\frac{5}{1.25}} = 0.11$$

$$T_x = T_y = C_t * {h_n}^x$$

- $C_t = 0.0488$
- $h_n = 376.8\ m$
- X = 0.75
- $T_x = T_y = 0.0488 * 376.8^{0.75} = \mathbf{4.173}\ \boldsymbol{Seconds}$

Now, to find upper limit coefficient for the calculated period, use Table 12.8-1 in ASCE 7-16

Table 12.8-1 Coefficient for Upper Limit on Calculated Period

Design Spectral Response Acceleration Parameter at 1 s, S_{D1}	Coefficient C_u
≥0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
≤0.1	1.7

Table 2.3.9.1 - Period Upper Limit

$Sd1 = 0.18$, $Cu = 1.54$

$T_{xu} = T_{yu} = 4.173 * 1.54 = \mathbf{6.426\ \mathit{Seconds}}$

From ETABS, the period is as follows.

Tx = 9.35 Seconds

Ty = 10.1 Seconds

Thus, $\mathbf{T_{xu} < Tx\ ETABS\ , T_{yu} < Ty\ ETABS.}$

So, take $\mathbf{T_x = T_y = 6.426\ \mathit{Seconds}.}$

$\mathbf{T_L = 24\ Seconds.}$

Since T in both directions (Y and X) is smaller than T_L:

$$C_{smax(x)} = C_{smax(y)} = \frac{S_{D1}}{T * \left(\frac{R}{I_e}\right)}$$

$$C_{smax} = \frac{0.18}{6.426 * \left(\frac{5}{1.25}\right)} = \mathbf{0.007 < C_s = 0.11}$$

$C_{s_{min}} = 0.044 * Sds * Ie \geq 0.01 , = 0.044 * 0.44 * 1.25 = 0.0242 > 0.01$, **Acceptable.**

$V = C_s * W$, W is the effective weight of the building

$W = D + SD + 0.25L.L$

Base Reactions

File Edit Format-Filter-Sort Select Options

Units: As Noted Hidden Columns: No Sort: None Base Reactions

Filter: None

Output Case	Case Type	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m	X m	Y m
Effective Wei...	Combination	0.0013	-0.008	4396758.435	209491569	-243432921	-0.3414	0	0

Figure 2.3.9.1 - Building Total Effective Weight

$W = 4396758.435\ kN$

$V = 0.0242 * 4396758.435 = \mathbf{106401.554\ \mathit{kN}}$

Base shear from ETABS

Base Reactions

File Edit Format-Filter-Sort Select Options

Units: As Noted Hidden Columns: No Sort: None Base Reactions

Filter: ([Step Number] = 1)

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
EX	LinStatic	Step By Step	1	-103776.1535	-31132.0866	39.6256	8899642.771	-29660392	3311428.4159
EY	LinStatic	Step By Step	1	-31132.8436	-103773.5988	39.1751	29659630	-8900073	-4299546

Figure 2.3.9.2 - Seismic Base Shear in Both Directions

$$Error\ \% = \frac{Manual\ base\ shear - ETABS\ base\ shear}{Manual\ base\ shear} * 100\%$$

$$Error\ \% = \frac{106401.554 - 103776.1535}{106401.554} * 100\% = \mathbf{2.46\%} < \mathbf{10}\ \%\ , \boldsymbol{Acceptable}$$

Base Shear from Wind Load

*The Wind analysis and calculations were conducted without considering the dome’s existence.

An Excel sheet was used to determine the base shear in both directions.

Area-X (m^2)	Area-Y (m^2)	Px Windward (kN/m^2)	Py Windward (kN/m^2)	Px Leeward (kN/m^2)	Py Leeward (kN/m^2)	Fx (Windward) (kN)	Fx (Leeward) (kN)	Fy (WindWard) (kN)	Fy (Leeward) (kN)
986.25	890.1498	1.247952251	1.261098363	1.611698616	1.580004668	615.3964536	1589.53776	561.2832277	1406.44084
2462.8995	2690.6445	1.710442991	1.728279066	1.596761278	1.657530918	3643.115081	3932.662554	4021.675969	4459.826447
1934.135	2112.985	1.890051133	1.910435218	1.596761278	1.657530918	3481.920851	3088.351875	3844.274351	3502.337966
1934.135	2112.985	2.020010452	2.042238206	1.596761278	1.657530918	3781.293482	3088.351875	4175.969828	3502.337966
2072.2875	2263.9125	2.133548456	2.157386916	1.596761278	1.657530918	4303.684103	3308.948438	4753.791905	3752.504964
2130.955	2328.005	2.230023686	2.255230792	1.596761278	1.657530918	4649.287937	3402.62643	5136.298045	3858.740264
2140.985	2344.335	2.321046759	2.347653815	1.597101254	1.659215416	4871.886778	3419.369828	5395.351743	3889.756772
444	444	2.34167753	2.370271706	1.644351228	1.662386993	1035.124792	730.0919454	1047.379466	738.0998247
438.48	438.48	2.365031638	2.394093133	1.648089524	1.666252958	1031.898918	722.6542944	1044.539347	730.6185971
						X-Direction		Y-Direction	
					Sum	27413.6084	23282.595	29980.56388	25840.66364
					Total	50696.20339		55821.22752	
					Error %	16.87192378		18.12485746	

Table 2.3.9.2 = Wind Base Shear Manual Computation

Base Shear in X-Direction (By Hand) = 50696.20339 kN

Base Shear in Y-Direction (By Hand) = 55821.22752 kN

Base Shear from ETABS

Base Reactions

File Edit Format-Filter-Sort Select Options

Units: As Noted Hidden Columns: No Sort: None Base Reactions

Filter: ([Step Number] = 1)

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
Wind X	LinStatic	Step By Step	1	-42142.7786	-0.0092	7.1058	446.8872	-9089526	2015589.8767
Wind Y	LinStatic	Step By Step	1	0.0023	-45703.7096	6.8854	9905208.4657	-499.1975	-2566247

Figure 2.3.9.3 - Wind Base Shear ETABS Computation

$$\text{Error \% in X-Direction} = \frac{50696.20339-42142.7786}{50696.20339} * 100\% = 16.87\%$$

$$\text{Error \% in Y-Direction} = \frac{55821.22752-45703.7096}{55821.22752} * 100\% = 18.12\%$$

This range of error is acceptable and makes sense due to several factors such as:

- Not considering the dome's existence in the manual calculations.
- The height of the building and number of floors and differences in these floors.

2.3.10 Story Shear Forces Check

The story shear force resulting from seismic forces can be computed from equation 12.8-11 in ASCE 7-16 as follows:

$$F_x = C_{vx} * V$$

- V: Total design base shear
- C_{vx}: Vertical distribution factor and it is found using equation $12.8-12$ as follows.

$$C_{vx} = \frac{W_x * {h_x}^k}{\sum_{i=1}^{n} W_i * {h_i}^k}$$

- $W_x: effective\ weight\ of\ the\ building$
- $W_i: portion\ of\ total\ effective\ weight\ at\ level\ i$
- $h_x: height\ of\ the\ building$
- $h_i: height\ of\ level\ i$
- K: exponent related to the period of the structure
- **The period is larger than 2.5 so, k = 2**
- Story Forces Check for X Direction

Story	W (kN)	Elevation (m)	Cvx	Fx	ETABS Fx	Error %
100th Floor	29986.87	376.8	0.02465938	2559.05527	2635.39	2.982906495
99th Floor	31157.87	370.5	0.02481437	2575.14026	2194.781	14.77042891
98th Floor	30613.62	364.2	0.0235998	2449.09618	2556.861	4.400195412
97th Floor	34226.51	358.2	0.02556644	2653.18655	2533.441	4.513265531
96th Floor	39003.45	352.2	0.02821665	2928.21547	3059.459	4.482017478
95th Floor	29892.72	348.85	0.02123764	2203.96024	2282.461	3.561790951
94th Floor	29892.73	345.2	0.02081895	2160.51093	2171.223	0.495821063
93rd Floor	29892.73	341.55	0.02040443	2117.49331	2127.993	0.495849909
92nd Floor	29892.73	337.9	0.01999408	2074.90828	2085.191	0.495589089
91st Floor	29892.73	334.25	0.01958789	2032.75581	2042.833	0.495750364
90th Floor	29892.73	330.6	0.01918587	1991.03591	2000.911	0.495952204
89th Floor	29892.73	326.95	0.01878802	1949.74858	1959.416	0.495813781
88th Floor	29892.73	323.3	0.01839434	1908.89386	1918.36	0.495912463
87th Floor	29892.73	319.65	0.01800483	1868.47167	1877.735	0.495754111
86th Floor	29892.73	316	0.01761948	1828.48206	1837.548	0.495801587
85th Floor	29892.73	312.35	0.01723831	1788.92509	1797.792	0.495677867
84th Floor	29892.73	308.7	0.0168613	1749.80066	1758.478	0.495875986
83rd Floor	29892.73	305.05	0.01648846	1711.10879	1719.593	0.495813718
82nd Floor	29892.73	301.4	0.01611979	1672.84949	1681.149	0.496099999
81st Floor	29892.73	297.75	0.01575528	1635.0228	1643.129	0.495779145
80th Floor	74532.05	294.1	0.03838449	3983.3944	2973.71	25.34732692
79th Floor	56364.72	288.9	0.02807381	2913.39184	3245.004	11.38234315
78th Floor	43627.65	285.25	0.02121865	2201.99031	2709.572	23.05104559
77th Floor	37199.4	281.6	0.01766158	1832.85058	1828.875	0.216934074
76th Floor	37201.16	277.95	0.01723694	1788.78295	1794.476	0.318241644
75th Floor	37202.43	274.3	0.01681721	1745.22588	1750.766	0.317438678
74th Floor	37202.47	270.65	0.01640211	1702.14839	1707.541	0.316782451
73rd Floor	37201.2	267	0.01599164	1659.55053	1664.864	0.320144857
72nd Floor	37200.72	263.35	0.01558671	1617.52874	1622.711	0.32036917
71st Floor	37198.52	259.7	0.01518627	1575.97297	1581.068	0.323268722
70th Floor	37201.05	256.05	0.01479295	1535.15574	1540.021	0.316948729
69th Floor	37197.79	252.4	0.01440252	1494.63839	1499.521	0.326641722
68th Floor	37199.44	248.75	0.0140192	1454.85853	1459.588	0.325067546
67th Floor	37200.74	245.1	0.0136409	1415.60059	1420.098	0.317682379
66th Floor	37201.08	241.45	0.01326742	1376.84221	1381.234	0.318975661
65th Floor	37197.91	237.8	0.01289791	1338.49515	1342.877	0.327400924
64th Floor	37202.29	234.15	0.01253619	1300.95717	1305.084	0.31719981

63rd Floor	37202.43	230.5	0.01217818	1263.8044	1267.776	0.314233643
62nd Floor	37203.08	226.85	0.01182552	1227.20678	1231.014	0.310218374
61st Floor	37200.87	223.2	0.01147715	1191.05461	1194.855	0.319060715
60th Floor	37197.99	219.55	0.01113381	1155.42418	1159.187	0.325691404
59th Floor	37202.94	215.9	0.01079799	1120.57345	1124.121	0.31655654
58th Floor	37200.92	212.25	0.0104653	1086.04831	1089.498	0.31759066
57th Floor	37199.47	208.6	0.01013799	1052.08124	1055.499	0.324876298
56th Floor	37202.32	204.95	0.00981702	1018.77285	1022.014	0.318142832
55th Floor	37202.49	201.3	0.00950051	985.926794	989.0611	0.317904518
54th Floor	37200.83	197.65	0.00918874	953.571789	956.6557	0.323406322
53rd Floor	58570.54	194	0.01398509	1451.31908	1242.046	14.41950875
52nd Floor	54793.15	190.35	0.01263984	1311.71438	1307.321	0.334956721
51st Floor	43627.6	186.7	0.00971725	1008.41905	1240.856	23.04965925
50th Floor	37200.69	183.05	0.00799517	829.708336	827.873	0.221202531
49th Floor	37200.47	179.4	0.00770971	800.084431	802.6439	0.319899901
48th Floor	37199.61	175.75	0.00742932	770.985964	773.4258	0.316456649
47th Floor	37200.91	172.1	0.006681661	742.470163	744.8283	0.317606998
46th Floor	37201.43	168.45	0.0064248	714.475733	716.7637	0.320230166
45th Floor	37200.74	164.8	0.006172973	686.996648	689.1869	0.318815506
44th Floor	37202.12	161.15	0.00592618	660.093561	662.1811	0.316248939
43rd Floor	37200.71	157.5	0.005684423	633.679329	635.71	0.320457169
42nd Floor	37199.59	153.85	0.005447699	607.810263	609.7503	0.319184575
41st Floor	37200.66	150.2	0.00521601	582.51522	584.4024	0.323970858
40th Floor	37200.53	146.55	0.004989356	557.739069	559.5276	0.320675195
39th Floor	37198.29	142.9	0.004767736	533.471088	535.2378	0.33117288
38th Floor	37200.66	139.25	0.00455115	509.807641	511.4425	0.320681525
37th Floor	37201.83	135.6	0.004339599	486.663755	488.1887	0.313346685
36th Floor	37200.69	131.95	0.004133083	464.027868	465.5271	0.323090922
35th Floor	37200.61	128.3	0.0039316	441.944387	443.3719	0.323007472
34th Floor	37201.01	124.65	0.003735153	420.404748	421.7331	0.315969859
33rd Floor	37200.67	121	0.00354374	399.395011	400.6841	0.322760392
32nd Floor	37199.65	117.35	0.003357361	378.917047	380.1276	0.319476988
31st Floor	37200.59	113.7	0.003176017	358.997437	360.139	0.317986498
30th Floor	37200.61	110.05	0.002999707	339.606737	340.6849	0.317474117
29th Floor	37201.74	106.4	0.002828432	320.763927	321.7698	0.31358657
28th Floor	37200.62	102.75	0.002662191	302.440064	303.4079	0.320009067
27th Floor	37200.54	99.1	0.002500984	284.663597	285.5775	0.321046556
26th Floor	37200.71	95.45	0.002344812	267.427282	268.2811	0.319270987
25th Floor	58570.53	91.8	0.003875705	394.7578	337.8153	14.42466748
24th Floor	54793.08	88.15	0.003342685	345.494802	344.3515	0.330917396

23rd Floor	43627.53	84.5	0.002412955	256.769442	315.9675	23.05494653
22nd Floor	37200.49	80.85	0.001770469	203.858946	203.4119	0.219291976
21st Floor	37200.7	77.2	0.00163947	189.314161	189.9243	0.322289387
20th Floor	37199.83	73.55	0.001513505	175.302425	175.8726	0.325252222
19th Floor	37200.51	69.9	0.001392574	161.836381	162.3434	0.313291015
18th Floor	37201.14	66.25	0.001276678	148.908023	149.3891	0.323070147
17th Floor	37200.75	62.6	0.001165816	136.51381	136.9445	0.315492105
16th Floor	37200.45	58.95	0.001059989	124.658469	125.0494	0.313601994
15th Floor	37200.41	55.3	0.000959196	113.342449	113.7085	0.322959969
14th Floor	37200.45	51.65	0.000863438	102.564992	102.8922	0.319025028
13th Floor	37201.19	48	0.000772714	92.327578	92.6227	0.31964667
12th Floor	37200.86	44.35	0.000687025	82.6257217	82.8832	0.311620062
11th Floor	37200.39	40.7	0.00060637	73.4620937	73.6971	0.319901404
10th Floor	37200.75	37.05	0.000530749	64.8384761	65.0461	0.320217194
9th Floor	37200.31	33.4	0.000460163	56.7517849	56.9373	0.326888606
8th Floor	37200.84	29.75	0.000394612	49.2048893	49.3658	0.327021752
7th Floor	54542.48	26.1	0.000398817	61.8654936	52.4582	15.20604306
6th Floor	37611.86	19.5	0.000238648	31.2291019	38.5146	23.32919512
5th Parking Floor	71706.29	15.78	0.000411197	48.7477022	41.5886	14.68603011
4th Parking Floor	60053.66	12.2	0.000286051	32.9818451	35.1084	6.447652873
3rd Parking Floor	60030.06	9.4	0.000232328	27.4186465	26.6898	2.658214622
2nd Parking Floor	60237.59	6.6	0.000185941	22.4569811	21.8285	2.798600091
1st Parking Floor	69791.91	3.8	0.00014911	20.7547744	18.9411	8.738589047
Ground Floor	77709.1	0	0.000117307	16.213069	14.9388	7.859517613
1st Basement Floor	75424.97	-3.35	7.55299E-05	10.8169076	10.9227	0.978028241
2nd Basement Floor	75238.05	-6.4	4.45386E-05	7.11978063	6.9834	1.9155173
3rd Basement Floor	75238.04	-9.45	2.17932E-05	4.20969629	4.1291	1.914539224
4th Basement Floor	75238.08	-12.5	7.09112E-06	2.05985018	2.0205	1.910341919
5th Basement Floor	75284.81	-15.55	4.326E-07	0.67065411	0.6576	1.946475043
6th Basement Floor	45921.09	-18.6	0.000105766	0.02493983	0.0318	27.50690698

Table 2.3.10.1 - Story Shear Forces Check in X-Direction

Story Forces Check for Y direction

Story	W (kN)	Elevation (m)	Cvx	Fx	ETABS Fx	Error %
100th Floor	29986.87	376.8	0.02465938	2558.99228	2635.39	2.985457361
99th Floor	31157.87	370.5	0.02481437	2575.07687	2194.792	14.76792297
98th Floor	30613.62	364.2	0.0235998	2449.03589	2556.87	4.403104447
97th Floor	34226.51	358.2	0.02556644	2653.12124	2533.441	4.510918607
96th Floor	39003.45	352.2	0.02821665	2928.14339	3059.461	4.484678411
95th Floor	29892.72	348.85	0.02123764	2203.90599	2282.468	3.56464898
94th Floor	29892.73	345.2	0.02081895	2160.45775	2171.223	0.498304328
93rd Floor	29892.73	341.55	0.02040443	2117.44118	2127.996	0.498484488
92nd Floor	29892.73	337.9	0.01999408	2074.8572	2085.198	0.498400464
91st Floor	29892.73	334.25	0.01958789	2032.70576	2042.836	0.498376876
90th Floor	29892.73	330.6	0.01918587	1990.9869	2000.909	0.498335807
89th Floor	29892.73	326.95	0.01878802	1949.70058	1959.416	0.498292917
88th Floor	29892.73	323.3	0.01839434	1908.84687	1918.358	0.498250265
87th Floor	29892.73	319.65	0.01800483	1868.42568	1877.738	0.498399384
86th Floor	29892.73	316	0.01761948	1828.43704	1837.549	0.498363101
85th Floor	29892.73	312.35	0.01723831	1788.88106	1797.796	0.498347524
84th Floor	29892.73	308.7	0.0168613	1749.75758	1758.479	0.498407145
83rd Floor	29892.73	305.05	0.01648846	1711.06666	1719.594	0.49833448
82nd Floor	29892.73	301.4	0.01611979	1672.80831	1681.146	0.498406631
81st Floor	29892.73	297.75	0.01575528	1634.98255	1643.129	0.498283733
80th Floor	74532.05	294.1	0.03838449	3983.29634	2973.65	25.347018
79th Floor	56364.72	288.9	0.02807381	2913.32012	3245.134	11.38955086
78th Floor	43627.65	285.25	0.02121865	2201.9361	2709.523	23.05183592
77th Floor	37199.4	281.6	0.01766158	1832.80546	1828.871	0.214668577
76th Floor	37201.16	277.95	0.01723694	1788.73891	1794.411	0.317088616
75th Floor	37202.43	274.3	0.01681721	1745.18292	1750.716	0.317037535
74th Floor	37202.47	270.65	0.01640211	1702.10649	1707.451	0.313999735
73rd Floor	37201.2	267	0.01599164	1659.50968	1664.7	0.312744154
72nd Floor	37200.72	263.35	0.01558671	1617.48892	1622.596	0.31572289
71st Floor	37198.52	259.7	0.01518627	1575.93418	1581.075	0.326208044
70th Floor	37201.05	256.05	0.01479295	1535.11795	1539.996	0.317757223
69th Floor	37197.79	252.4	0.01440252	1494.60159	1499.525	0.329385885
68th Floor	37199.44	248.75	0.0140192	1454.82271	1459.44	0.317398657
67th Floor	37200.74	245.1	0.0136409	1415.56574	1419.99	0.312557864
66th Floor	37201.08	241.45	0.01326742	1376.80831	1381.256	0.323050477
65th Floor	37197.91	237.8	0.01289791	1338.4622	1342.786	0.323034582

64th Floor	37202.29	234.15	0.01253619	1300.92514	1305.055	0.317486359
63rd Floor	37202.43	230.5	0.01217818	1263.77329	1267.718	0.312169139
62nd Floor	37203.08	226.85	0.01182552	1227.17657	1230.987	0.31048764
61st Floor	37200.87	223.2	0.01147715	1191.02529	1194.771	0.314519605
60th Floor	37197.99	219.55	0.01113381	1155.39574	1159.104	0.320968882
59th Floor	37202.94	215.9	0.01079799	1120.54587	1124.09	0.316295322
58th Floor	37200.92	212.25	0.0104653	1086.02158	1089.383	0.309489587
57th Floor	37199.47	208.6	0.01013799	1052.05534	1055.422	0.319960555
56th Floor	37202.32	204.95	0.00981702	1018.74777	1021.908	0.31021735
55th Floor	37202.49	201.3	0.00950051	985.902523	988.9822	0.312371326
54th Floor	37200.83	197.65	0.00918874	953.548314	956.5404	0.313784406
53rd Floor	58570.54	194	0.01398509	1451.28335	1242	14.42058532
52nd Floor	54793.15	190.35	0.01263984	1311.68208	1307.287	0.335064762
51st Floor	43627.6	186.7	0.00971725	1008.39422	1240.815	23.0485532
50th Floor	37200.69	183.05	0.00799517	829.687911	827.8695	0.21916802
49th Floor	37200.47	179.4	0.00770971	800.064735	802.5813	0.314545211
48th Floor	37199.61	175.75	0.00742932	770.966984	773.3748	0.312311171
47th Floor	37200.91	172.1	0.00715454	742.451885	744.807	0.317207745
46th Floor	37201.43	168.45	0.00688478	714.458145	716.6706	0.309668997
45th Floor	37200.74	164.8	0.00661999	686.979736	689.1369	0.314006921
44th Floor	37202.12	161.15	0.00636074	660.077311	662.1271	0.310537666
43rd Floor	37200.71	157.5	0.00610621	633.66373	635.6628	0.315478114
42nd Floor	37199.59	153.85	0.00585694	607.795301	609.7547	0.322378162
41st Floor	37200.66	150.2	0.00561319	582.50088	584.2831	0.30595998
40th Floor	37200.53	146.55	0.00537444	557.725339	559.4804	0.314681944
39th Floor	37198.29	142.9	0.00514059	533.457956	535.1701	0.32095205
38th Floor	37200.66	139.25	0.00491257	509.795091	511.3987	0.314559534
37th Floor	37201.83	135.6	0.00468955	486.651775	488.1812	0.314275058
36th Floor	37200.69	131.95	0.00447143	464.016445	465.4898	0.317522169
35th Floor	37200.61	128.3	0.00425863	441.933507	443.3356	0.31726332
34th Floor	37201.01	124.65	0.00405107	420.394398	421.7507	0.322625984
33rd Floor	37200.67	121	0.00384862	399.385179	400.6664	0.320798328
32nd Floor	37199.65	117.35	0.00365129	378.907719	380.1194	0.319782539
31st Floor	37200.59	113.7	0.00345934	358.988599	360.1393	0.320539696
30th Floor	37200.61	110.05	0.00327249	339.598376	340.6712	0.315909557
29th Floor	37201.74	106.4	0.00309092	320.756031	321.7744	0.317490203
28th Floor	37200.62	102.75	0.00291435	302.432619	303.3871	0.315601181
27th Floor	37200.54	99.1	0.00274305	284.65659	285.5845	0.325975365
26th Floor	37200.71	95.45	0.00257696	267.420699	268.2721	0.318375164
25th Floor	58570.53	91.8	0.00380394	394.748082	337.8234	14.42050884

24th Floor	54793.08	88.15	0.00332923	345.486297	344.3274	0.335439416
23rd Floor	43627.53	84.5	0.00247426	256.763121	315.9614	23.05560017
22nd Floor	37200.49	80.85	0.00196441	203.853928	203.4104	0.217571394
21st Floor	37200.7	77.2	0.00182425	189.3095	189.9113	0.317892061
20th Floor	37199.83	73.55	0.00168924	175.298109	175.8633	0.322416782
19th Floor	37200.51	69.9	0.00155948	161.832397	162.3478	0.318479391
18th Floor	37201.14	66.25	0.0014349	148.904357	149.3854	0.323055085
17th Floor	37200.75	62.6	0.00131546	136.510449	136.9503	0.322210432
16th Floor	37200.45	58.95	0.00120122	124.6554	125.0658	0.329227785
15th Floor	37200.41	55.3	0.00109218	113.339659	113.709	0.325870873
14th Floor	37200.45	51.65	0.00098833	102.562467	102.8781	0.307746964
13th Floor	37201.19	48	0.00088968	92.3253051	92.6146	0.313343015
12th Floor	37200.86	44.35	0.00079619	82.6236876	82.8881	0.320020038
11th Floor	37200.39	40.7	0.00070789	73.4602853	73.6802	0.299365453
10th Floor	37200.75	37.05	0.00062479	64.8368799	65.0362	0.307417791
9th Floor	37200.31	33.4	0.00054687	56.7503878	56.9233	0.30468902
8th Floor	37200.84	29.75	0.00047414	49.203678	49.3597	0.317094157
7th Floor	54542.48	26.1	0.00059614	61.8639706	52.4584	15.20363231
6th Floor	37611.86	19.5	0.00030093	31.2283331	38.5146	23.33223124
5th Parking Floor	71706.29	15.78	0.00046974	48.7465022	41.5857	14.68987899
4th Parking Floor	60053.66	12.2	0.00031782	32.9810332	35.1067	6.445118925
3rd Parking Floor	60030.06	9.4	0.00026421	27.4179715	26.6884	2.6609244
2nd Parking Floor	60237.59	6.6	0.0002164	22.4564283	21.8276	2.800214947
1st Parking Floor	69791.91	3.8	0.0002	20.7542635	18.9408	8.737787858
Ground Floor	77709.1	0	0.00015623	16.2126699	14.9386	7.8584829
1st Basement Floor	75424.97	-3.35	0.00010423	10.8166413	10.9227	0.980514119
2nd Basement Floor	75238.05	-6.4	6.8607E-05	7.11960536	6.9834	1.913102655
3rd Basement Floor	75238.04	-9.45	4.0565E-05	4.20959265	4.1292	1.909749028
4th Basement Floor	75238.08	-12.5	1.9849E-05	2.05979947	2.0203	1.917636829
5th Basement Floor	75284.81	-15.55	6.4625E-06	0.67063761	0.6576	1.94406116
6th Basement Floor	45921.09	-18.6	2.4032E-07	0.02493921	0.0319	27.91102095

Table 2.3.10.2 - Story Shear Forces Check in Y-Direction

2.3.11 Period Check

To calculate the period manually, the following equation must be used.

$$T = 2 * \pi * \sqrt{\frac{\sum W * Delta^2}{9810 * \sum F * Delta^2}}$$

- W is the weight of each floor
- Delta is the displacement of each floor
- F is the force applied to each floor

- Period Check for X Direction

Story	W (kN)	Delta (UX) mm	W*Delta^2	F (KN)	F * Delta
100th Floor	29986.87	1489.026	66486844371	2635.39	3924163
99th Floor	31157.87	1465.043	66875726895	2194.781	3215449
98th Floor	30613.62	1441.489	63611750889	2556.861	3685687
97th Floor	34226.51	1418.257	68845020857	2533.441	3593071
96th Floor	39003.45	1394.947	75895924958	3059.459	4267783
95th Floor	29892.72	1381.87	57082080706	2282.461	3154064
94th Floor	29892.73	1367.54	55904358562	2171.223	2969235
93rd Floor	29892.73	1353.046	54725623211	2127.993	2879272
92nd Floor	29892.73	1338.404	53547604526	2085.191	2790828
91st Floor	29892.73	1323.636	52372431212	2042.833	2703968
90th Floor	29892.73	1308.763	51202080862	2000.911	2618718
89th Floor	29892.73	1293.802	50038147842	1959.416	2535096
88th Floor	29892.73	1278.768	48882016560	1918.36	2453138
87th Floor	29892.73	1263.678	47735166207	1877.735	2372852
86th Floor	29892.73	1248.548	46598943350	1837.548	2294267
85th Floor	29892.73	1233.399	45475007353	1797.792	2217395
84th Floor	29892.73	1218.257	44365300242	1758.478	2142278
83rd Floor	29892.73	1203.155	43272177663	1719.593	2068937
82nd Floor	29892.73	1188.129	42198088715	1681.149	1997421
81st Floor	29892.73	1173.168	41142057603	1643.129	1927666
80th Floor	74532.05	1158.386	1.00011E+11	2973.71	3444704
79th Floor	56364.72	1139.728	73216637968	3245.004	3698422
78th Floor	43627.65	1126.588	55372232081	2709.572	3052571
77th Floor	37199.4	1111.667	45971146702	1828.875	2033099
76th Floor	37201.16	1096.079	44693074858	1794.476	1966887
75th Floor	37202.43	1080.173	43406812019	1750.766	1891130
74th Floor	37202.47	1063.943	42112252257	1707.541	1816726
73rd Floor	37201.2	1047.395	40811061018	1664.864	1743770
72nd Floor	37200.72	1030.564	39509481214	1622.711	1672307
71st Floor	37198.52	1013.489	38208831497	1581.068	1602395
70th Floor	37201.05	996.206	36919306799	1540.021	1534179
69th Floor	37197.79	978.749	35633605747	1499.521	1467654
68th Floor	37199.44	961.145	34364827673	1459.588	1402876
67th Floor	37200.74	943.424	33110479897	1420.098	1339754
66th Floor	37201.08	925.612	31872307928	1381.234	1278487

65th Floor	37197.91	907.735	30650436524	1342.877	1218977
64th Floor	37202.29	889.822	29456148310	1305.084	1161292
63rd Floor	37202.43	871.9	28281641913	1267.776	1105374
62nd Floor	37203.08	853.998	27132673678	1231.014	1051283
61st Floor	37200.87	836.148	26008745493	1194.855	999075.5
60th Floor	37197.99	818.383	24913380730	1159.187	948659.2
59th Floor	37202.94	800.741	23854011372	1124.121	900129.5
58th Floor	37200.92	783.264	22822859703	1089.498	853364.2
57th Floor	37199.47	765.999	21826956579	1055.499	808511.3
56th Floor	37202.32	748.982	20869535723	1022.014	765470.1
55th Floor	37202.49	732.193	19944500615	989.0611	724183.6
54th Floor	37200.83	715.644	19052267761	956.6557	684624.9
53rd Floor	58570.54	700.206	28716456476	1242.046	869688.1
52nd Floor	54793.15	687.48	25896820142	1307.321	898756.8
51st Floor	43627.6	674.988	19877120539	1240.856	837563
50th Floor	37200.69	659.332	16171836040	827.873	545843.2
49th Floor	37200.47	642.696	15365958085	802.6439	515856
48th Floor	37199.61	625.883	14572186983	773.4258	484074.1
47th Floor	37200.91	608.857	13790632031	744.8283	453493.9
46th Floor	37201.43	591.596	13019971957	716.7637	424034.5
45th Floor	37200.74	574.138	12262644914	689.1869	395688.4
44th Floor	37202.12	556.533	11522574127	662.1811	368525.6
43rd Floor	37200.71	538.828	10800690296	635.71	342538.3
42nd Floor	37199.59	521.065	10100014789	609.7503	317719.5
41st Floor	37200.66	503.285	9422771571	584.4024	294121
40th Floor	37200.53	485.523	8769377706	559.5276	271663.5
39th Floor	37198.29	467.815	8140877630	535.2378	250392.3
38th Floor	37200.66	450.195	7539664490	511.4425	230248.9
37th Floor	37201.83	432.699	6965240810	488.1887	211238.8
36th Floor	37200.69	415.361	6418039695	465.5271	193361.8
35th Floor	37200.61	398.217	5899152866	443.3719	176558.2
34th Floor	37201.01	381.302	5408700312	421.7331	160807.7
33rd Floor	37200.67	364.657	4946749148	400.6841	146112.3
32nd Floor	37199.65	348.32	4513315341	380.1276	132406
31st Floor	37200.59	332.338	4108751129	360.139	119687.9
30th Floor	37200.61	316.761	3732617163	340.6849	107915.7
29th Floor	37201.74	301.644	3384952652	321.7698	97059.93
28th Floor	37200.62	287.031	3064839712	303.4079	87087.47
27th Floor	37200.54	272.909	2770671231	285.5775	77936.67
26th Floor	37200.71	259.306	2501361035	268.2811	69566.9

25th Floor	58570.53	247.158	3577902211	337.8153	83493.75
24th Floor	54793.08	238.175	3108265386	344.3515	82015.92
23rd Floor	43627.53	229.541	2298694207	315.9675	72527.5
22nd Floor	37200.49	217.864	1765711001	203.4119	44316.13
21st Floor	37200.7	205.357	1568809432	189.9243	39002.28
20th Floor	37199.83	192.896	1384163628	175.8726	33925.12
19th Floor	37200.51	180.455	1211397607	162.3434	29295.68
18th Floor	37201.14	168.017	1050177558	149.3891	25099.91
17th Floor	37200.75	155.629	901016593.5	136.9445	21312.54
16th Floor	37200.45	143.351	764450987.5	125.0494	17925.96
15th Floor	37200.41	131.24	640737562.9	113.7085	14923.1
14th Floor	37200.45	119.348	529881171.9	102.8922	12279.98
13th Floor	37201.19	107.724	431699730.2	92.6227	9977.688
12th Floor	37200.86	96.417	345828021.9	82.8832	7991.349
11th Floor	37200.39	85.474	271778763.5	73.6971	6299.186
10th Floor	37200.75	74.944	208941877.2	65.0461	4874.815
9th Floor	37200.31	64.88	156591524.4	56.9373	3694.092
8th Floor	37200.84	55.337	113915796.4	49.3658	2731.755
7th Floor	54542.48	46.363	117240571.3	52.4582	2432.12
6th Floor	37611.86	31.727	37860189.27	38.5146	1221.953
5th Parking Floor	71706.29	24.746	43910389.9	41.5886	1029.151
4th Parking Floor	60053.66	19.469	22762857.62	35.1084	683.5254
3rd Parking Floor	60030.06	15.826	15035266.23	26.6898	422.3928
2nd Parking Floor	60237.59	12.54	9472457.109	21.8285	273.7294
1st Parking Floor	69791.91	9.612	6448112.578	18.9411	182.0619
Ground Floor	77709.1	6.259	3044260.24	14.9388	93.50195
1st Basement Floor	75424.97	4.119	1279672.262	10.9227	44.9906
2nd Basement Floor	75238.05	2.604	510175.3766	6.9834	18.18477
3rd Basement Floor	75238.04	1.445	157098.9049	4.1291	5.96655
4th Basement Floor	75238.08	0.622	29108.41043	2.0205	1.256751
5th Basement Floor	75284.81	0.122	1120.539041	0.6576	0.080227
6th Basement Floor	45921.09	-0.05	114.8027263	0.0318	-0.00159

Table 2.3.11.1 - Natural Period Check in X-Direction

$\sum W*Delta^2 = 2.36655*10^{12}$.

$\sum F*Delta^2 = 106575233.9$

Period (T) = 9.45312698 seconds

Period (T) by Etabs = 9.35 seconds

Error percentage = 1.090929809 %

- Period Check for Y Direction

Story	W (kN)	Delta (UY) mm	W*Delta^2	F (KN)	F * Delta
100th Floor	29986.87	1767.126	93641032179	2635.39	4657066
99th Floor	31157.87	1736.978	94006180565	2194.792	3812305
98th Floor	30613.62	1706.784	89180891651	2556.87	4364024
97th Floor	34226.51	1677.678	96334064284	2533.441	4250298
96th Floor	39003.45	1648.347	1.05974E+11	3059.461	5043054
95th Floor	29892.72	1631.978	79614838422	2282.468	3724937
94th Floor	29892.73	1613.989	77869382996	2171.223	3504331
93rd Floor	29892.73	1595.846	76128548831	2127.996	3395954
92nd Floor	29892.73	1577.563	74394189919	2085.198	3289532
91st Floor	29892.73	1559.158	72668441954	2042.836	3185105
90th Floor	29892.73	1540.651	70953550825	2000.909	3082702
89th Floor	29892.73	1522.064	69251856088	1959.416	2982356
88th Floor	29892.73	1503.418	67565514042	1918.358	2884093
87th Floor	29892.73	1484.733	65896494552	1877.738	2787939
86th Floor	29892.73	1466.033	64247032799	1837.549	2693908
85th Floor	29892.73	1447.345	62619518560	1797.796	2602031
84th Floor	29892.73	1428.702	61016725798	1758.479	2512342
83rd Floor	29892.73	1410.15	59442383401	1719.594	2424885
82nd Floor	29892.73	1391.77	57902928613	1681.146	2339768
81st Floor	29892.73	1373.7	56409128175	1643.129	2257167
80th Floor	74532.05	1356.263	1.37098E+11	2973.65	4033051
79th Floor	56364.72	1332.954	1.00147E+11	3245.134	4325615
78th Floor	43627.65	1316.479	75611814482	2709.523	3567030
77th Floor	37199.4	1298.906	62761216836	1828.871	2375532
76th Floor	37201.16	1280.446	60992868151	1794.411	2297646

75th Floor	37202.43	1261.344	59188638075	1750.716	2208255
74th Floor	37202.47	1241.772	57366119319	1707.451	2120265
73rd Floor	37201.2	1221.825	55536039761	1664.7	2033972
72nd Floor	37200.72	1201.563	53708680796	1622.596	1949651
71st Floor	37198.52	1181.031	51885770692	1581.075	1867299
70th Floor	37201.05	1160.268	50080869581	1539.996	1786808
69th Floor	37197.79	1139.31	48283742656	1499.525	1708423
68th Floor	37199.44	1118.194	46512605013	1459.44	1631937
67th Floor	37200.74	1096.953	44763875115	1419.99	1557663
66th Floor	37201.08	1075.622	43040262629	1381.256	1485709
65th Floor	37197.91	1054.236	41342257415	1342.786	1415613
64th Floor	37202.29	1032.83	39685089854	1305.055	1347900
63rd Floor	37202.43	1011.44	38058486527	1267.718	1282221
62nd Floor	37203.08	990.104	36470398988	1230.987	1218805
61st Floor	37200.87	968.862	34920217467	1194.771	1157569
60th Floor	37197.99	947.755	33412704889	1159.104	1098547
59th Floor	37202.94	926.825	31957497934	1124.09	1041835
58th Floor	37200.92	906.119	30543879822	1089.383	987110.4
57th Floor	37199.47	885.691	29181072013	1055.422	934777.3
56th Floor	37202.32	865.613	27875172700	1021.908	884576.9
55th Floor	37202.49	846.005	26626732854	988.9822	836683.9
54th Floor	37200.83	827.104	25449124702	956.5404	791158.4
53rd Floor	58570.54	809.38	38369322414	1242	1005250
52nd Floor	54793.15	793.592	34508085567	1307.287	1037453
51st Floor	43627.6	777.839	26396162680	1240.815	965153.9
50th Floor	37200.69	760.417	21510705570	827.8695	629526
49th Floor	37200.47	741.692	20464240901	802.5813	595268.1
48th Floor	37199.61	722.198	19402200442	773.3748	558529.7
47th Floor	37200.91	702.235	18345033374	744.807	523029.5
46th Floor	37201.43	681.951	17300789901	716.6706	488734.2
45th Floor	37200.74	661.431	16274987350	689.1369	455816.5
44th Floor	37202.12	640.738	15273150486	662.1271	424250
43rd Floor	37200.71	619.92	14296261968	635.6628	394060.1
42nd Floor	37199.59	599.028	13348499523	609.7547	365260.1
41st Floor	37200.66	578.108	12432791427	584.2831	337778.7
40th Floor	37200.53	557.205	11549925150	559.4804	311745.3
39th Floor	37198.29	536.367	10701558771	535.1701	287047.6
38th Floor	37200.66	515.637	9890968799	511.3987	263696.1
37th Floor	37201.83	495.062	9117663016	488.1812	241680
36th Floor	37200.69	474.688	8382382455	465.4898	220962.4

35th Floor	37200.61	454.559	7686534481	443.3356	201522.2
34th Floor	37201.01	434.725	7030463929	421.7507	183345.6
33rd Floor	37200.67	415.234	6414112783	400.6664	166370.3
32nd Floor	37199.65	396.137	5837537339	380.1194	150579.4
31st Floor	37200.59	377.486	5300923433	360.1393	135947.5
30th Floor	37200.61	359.337	4803457067	340.6712	122415.8
29th Floor	37201.74	341.755	4345031909	321.7744	109968
28th Floor	37200.62	324.828	3925157379	303.3871	98548.62
27th Floor	37200.54	308.694	3544913609	285.5845	88158.22
26th Floor	37200.71	293.622	3207217653	268.2721	78770.59
25th Floor	58570.53	280.129	4596161324	337.8234	94634.13
24th Floor	54793.08	269.044	3966179512	344.3274	92639.22
23rd Floor	43627.53	258.187	2908234387	315.9614	81577.13
22nd Floor	37200.49	245.785	2247291589	203.4104	49995.23
21st Floor	37200.7	232.229	2006245489	189.9113	44102.91
20th Floor	37199.83	218.127	1769945288	175.8633	38360.53
19th Floor	37200.51	203.809	1545238741	162.3478	33087.94
18th Floor	37201.14	189.438	1335028329	149.3854	28299.27
17th Floor	37200.75	175.113	1140744838	136.9503	23981.78
16th Floor	37200.45	160.907	963159178.9	125.0658	20123.96
15th Floor	37200.41	146.883	802584577.3	113.709	16701.92
14th Floor	37200.45	133.101	659038572.1	102.8781	13693.18
13th Floor	37201.19	119.621	532318662.2	92.6146	11078.65
12th Floor	37200.86	106.506	421988971.1	82.8881	8828.08
11th Floor	37200.39	93.818	327431006	73.6802	6912.529
10th Floor	37200.75	81.624	247849188.6	65.0362	5308.515
9th Floor	37200.31	69.995	182255484.1	56.9233	3984.346
8th Floor	37200.84	59.005	129518068.2	49.3597	2912.469
7th Floor	54542.48	48.732	129527903.7	52.4584	2556.403
6th Floor	37611.86	32.239	39091999.82	38.5146	1241.672
5th Parking Floor	71706.29	24.377	42610613.87	41.5857	1013.735
4th Parking Floor	60053.66	18.292	20093790.83	35.1067	642.1718
3rd Parking Floor	60030.06	14.19	12087419.39	26.6884	378.7084
2nd Parking Floor	60237.59	10.632	6809222.262	21.8276	232.071
1st Parking Floor	69791.91	7.619	4051361.849	18.9408	144.31
Ground Floor	77709.1	4.426	1522280.526	14.9386	66.11824

1st Basement Floor	75424.97	2.615	515772.9469	10.9227	28.56286
2nd Basement Floor	75238.05	1.493	167709.3012	6.9834	10.42622
3rd Basement Floor	75238.04	0.751	42434.32884	4.1292	3.101029
4th Basement Floor	75238.08	0.302	6862.014104	2.0203	0.610131
5th Basement Floor	75284.81	0.083	518.637023	0.6576	0.054581
6th Basement Floor	45921.09	-0.006	1.653159258	0.0319	-0.00019

Table 2.3.11.2 - Natural Period Check in Y-Direction

$\sum$W*Delta^2 = 3.23331*10^{12}

$\sum$ F*Delta^2 = 124762854.2

Period (T) = 10.2123739 seconds

Period (T) by ETABS = 10.1 seconds

Error percentage = 1.100369978 %

3 STRUCTURAL DESIGN

Now, after analyses have been done and results are obtained, design phase will be initiated.

3.1 Structural Design Verification

3.1.1 Verification of beams design

Design combination (1.2+0.2Sds) (D+Sd) +LL+EDx+Soil)

1. Check for Moment Design

- **Edge Beam at 65th Floor (B1XG)**

H=1000mm

B=800mm

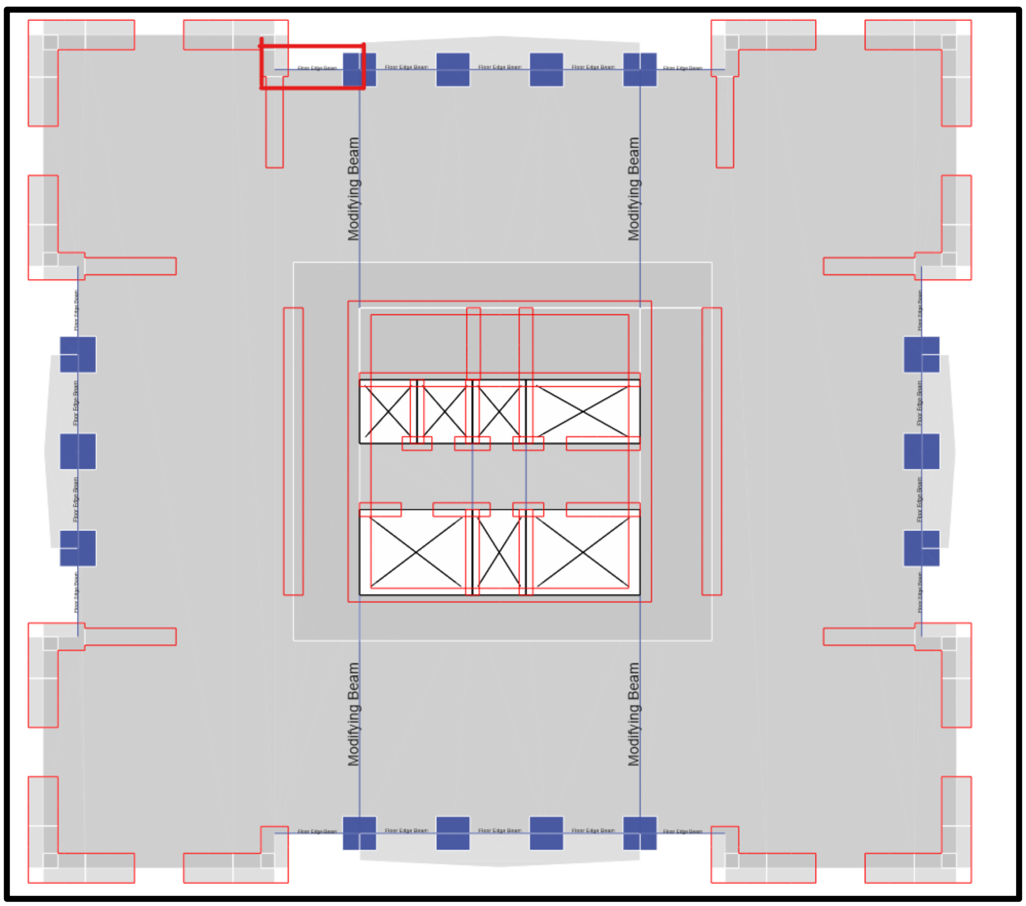


Figure 3.1.1.1 – Location of Edge Beam to Check

Max negative moment at left = 2905.7 KN.m

Figure 3.1.1.2 – Internal Forces in Edge Beam

$$\rho = \frac{0.85(70)}{420}\left(1 - \sqrt{1 - \frac{2.61(931.5*10^6)}{800(1000-75)^2(70)}}\right) = 0.012$$

$$As = 0.012(800)(1000 - 75) = 8880\ mm^2$$

9048	6626	6296
6691	4954	8395

Figure 3.1.1.3 – Edge Beam Reinforcement

As by ETABS = 9048 mm².

$$\text{Error percentage} = \frac{9048 - 8880}{9048} * 100\% = \mathbf{1.9}\ \% \ < \mathbf{25\%}\ \ , \boldsymbol{Acceptable}$$

- **Interior Beam at 4th Basement Floor (B2A)**

H =1000 mm

B=1000 mm

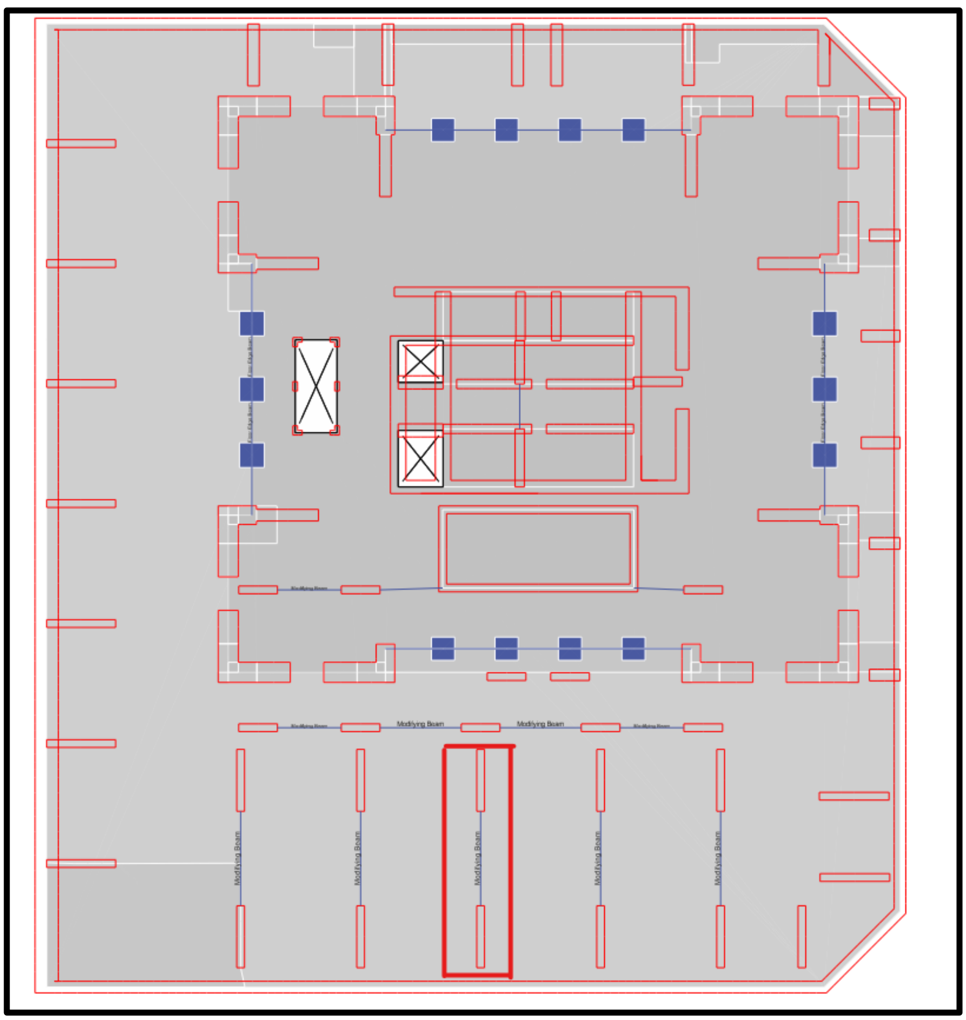


Figure 3.1.1.4 – Location of Interior Beam to Check

Max negative moment = 213.91 KN.m

Figure 3.1.1.5 – Internal Forces in Interior Beam

$$\rho = \frac{0.85(70)}{420}\left(1-\sqrt{1-\frac{2.61(213.91*10^6)}{1000(1000-75)^2(70)}}\right) = 0.001$$

$$As \ = \ 0.001(1000)(1000-75) = 925\ mm^2$$

As by ETABS = **1190 mm²**

$$\%\ \text{Error} = \frac{1190-925}{1190} * 100\% = \mathbf{22.3\ \%}\ < \mathbf{25\%}\ \ ,\boldsymbol{Acceptable}$$

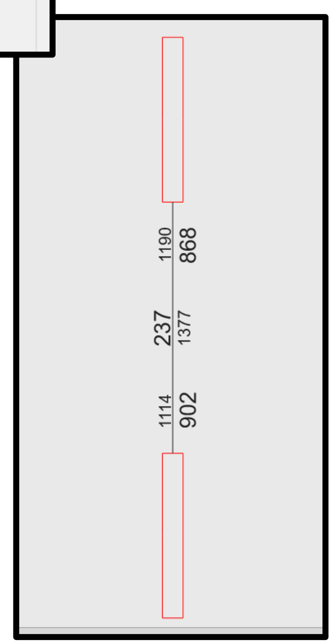


Figure 3.1.1.6 – Interior Beam Reinforcement

2. **Shear Design Check**

- **Edge Beam at 65th Floor (B1XG)**

H = 1000 mm

B = 800 mm

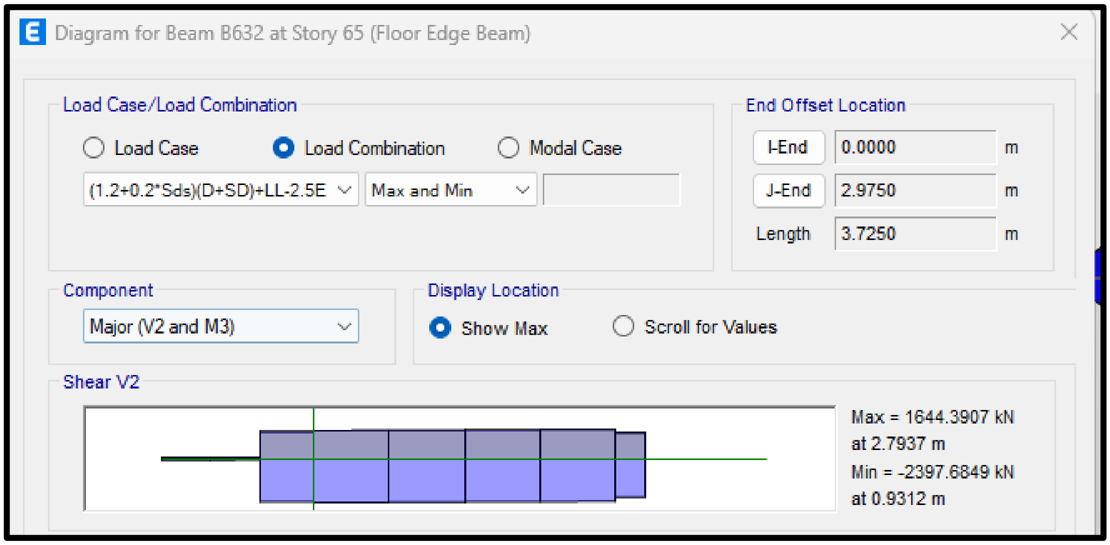


Figure 3.1.1.6 – Shear in Edge Beam

Vu on this beam = 2397.7 KN

$$\frac{\text{Vu}}{\text{Ø}} = \frac{2397.7}{0.75} = 3196.9 \text{ KN}$$

$$\text{Vc} = \frac{1}{6}\lambda\sqrt{fc}bw * d = 652.62 \text{ KN}$$

$\text{Vc} < \frac{\text{Vu}}{\text{Ø}}$, **shear reinforcement is needed.**

$$\text{Vs} = \frac{\text{Vu}}{\text{Ø}} - \text{Vc} = 3196.9 - 652.62 = 2544.28 \text{ KN}$$

$$\frac{\text{Av}}{\text{S}} = \frac{\text{Vs}}{\text{fy d}} = \frac{2544.28 * 1000}{420(1000 - 75)} = 6.55 \text{ mm}^2/\text{mm}$$

Beam Element Details (Part 1 of 2)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)
65	B632	3796	Floor Edge Beam	(1.2+0.2*Sds)(D+SD)+LL-2.5EDX + Soil	600	3725

Beam Element Details (Part 2 of 2)

LLRF	Type
1	Sway Intermediate

Section Properties

b (mm)	h (mm)	b_f (mm)	d_s (mm)	d_{ct} (mm)	d_{cb} (mm)
800	1000	800	0	75	75

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
39323	70	1	413.69	413.69

Design Code Parameters

ϕ_T	ϕ_{CTied}	$\phi_{CSpiral}$	ϕ_{Vns}	ϕ_{Vs}	ϕ_{Vjoint}
0.9	0.65	0.75	0.75	0.6	0.85

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

	Design Moment kN-m	Design P_u kN	-Moment Rebar mm²	+Moment Rebar mm²	Minimum Rebar mm²	Required Rebar mm²
Top (+2 Axis)	-2905.6737	-183.7508	9048	0	3728	9048
Bottom (-2 Axis)	1957.0832	-183.7508	0	6097	3728	6097

Shear Force and Reinforcement for Shear, V_{u2}

Shear V_{u2} kN	Shear ϕV_c kN	Shear ϕV_s kN	Shear V_p kN	Rebar A_v /s mm²/m
2349.2615	765.3181	1583.9434	1108.7523	5519.08

$\frac{\mathbf{Av}}{\mathbf{s}}\ \mathbf{min} = \ \mathbf{0.67\ mm^2/mm}$ **, take** $\frac{\mathbf{Av}}{\mathbf{s}} = \ \mathbf{6.55\ \ mm^2/mm}$

From ETABS

$\frac{\mathbf{Av}}{\mathbf{s}} = \mathbf{5.52\ \ mm^2/mm}$

$$\text{Error percentage} = \frac{6.55 - 5.52}{6.55} * 100\% = \mathbf{15.7\ \%}\ < \mathbf{25\%}\ , \boldsymbol{Acceptable}\ .$$

- **Interior Beam at 4th Basement Floor (B2A)**

H = 1000 mm

B = 1000 mm

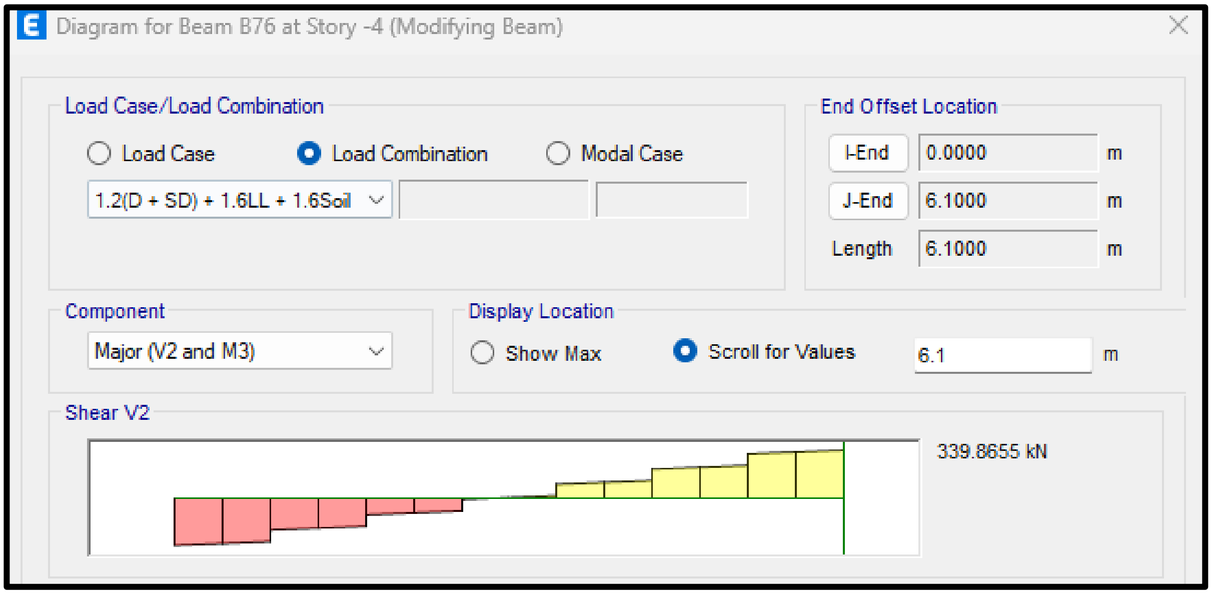


Figure 3.1.1.7 – Shear in Interior Beam

Vu on this beam = 339.9 KN

$$\frac{\text{Vu}}{\emptyset} = \frac{339.9}{0.75} = 453.2 \text{ KN}$$

$\text{Vc} = \frac{1}{6}\lambda\sqrt{fc}bw * d = 815.77 \text{ KN, Vc} > \frac{\text{Vu}}{\emptyset}$, **no need for shear reinforcement**.

Beam Element Details (Part 1 of 2)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	LLRF
-4	B76	592	Modifying Beam	1.2(D + SD) + 1.6LL + 1.6Soil	3050	6100	0.764

Beam Element Details (Part 2 of 2)

Type
Sway Intermediate

Section Properties

b (mm)	h (mm)	b_f (mm)	d_s (mm)	d_{ct} (mm)	d_{cb} (mm)
1000	1000	1000	0	75	75

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
39323	70	1	413.69	413.69

Design Code Parameters

ϕ_T	ϕ_{CTied}	$\phi_{CSpiral}$	ϕ_{Vns}	ϕ_{Vs}	ϕ_{Vjoint}
0.9	0.65	0.75	0.75	0.6	0.85

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

	Design Moment kN-m	Design P_u kN	-Moment Rebar mm²	+Moment Rebar mm²	Minimum Rebar mm²	Required Rebar mm²
Top (+2 Axis)	-40.1358	0	117	0	155	155
Bottom (-2 Axis)	354.2255	0	0	1033	1377	1377

Shear Force and Reinforcement for Shear, V_{u2}

Shear V_{u2} kN	Shear ϕV_c kN	Shear ϕV_s kN	Shear V_p kN	Rebar A_v /s mm²/m
4.6418	286.7856	0	97.5514	0

From ETABS

$\frac{\mathbf{Av}}{\mathbf{S}} = \mathbf{0}\ \frac{\boldsymbol{mm^2}}{\mathbf{mm}}$ **Error percentage = 0 %.**

3. Check for Torsion Design

- **Edge Beam at 65th Floor (B1XG)**

H = 1000mm

B = 800mm

Torsion Force and Torsion Reinforcement for Torsion, T_u

T_u kN-m	ϕT_{th} kN-m	ϕT_{cr} kN-m	Area A_o cm²	Perimeter, p_h mm	Rebar A_t /s mm²/m	Rebar A_l mm²
4.4689	90.5823	362.3293	5507	3244.4	0	0

Tu on this beam = 4.5 KN.m

$$ØTth = Ø * \lambda * \left(\frac{1}{12}\right)\sqrt{fc'} * \frac{Acp^2}{Pcp}$$

$$Acp = 1000 * 800 = 800000\ mm^2$$

$$Pcp = 2(1000 + 800) = 3600\ mm$$

$$ØTth = \frac{0.75 * 1 * \left(\frac{1}{12}\right)\sqrt{70} * \frac{800000^2}{3600}}{10^6} = 58.79\ KN.m$$

$$ØTth = 58.79 > Tu = 4.5\ , \boldsymbol{no\ need\ to\ design\ for\ torsion.}$$

$$\boldsymbol{\frac{At}{s} = 0\ \frac{mm^2}{mm}}$$

$\boldsymbol{From\ ETABS}$

$$\boldsymbol{\frac{At}{s} = 0\ \frac{mm^2}{mm}}\ , \boldsymbol{Error\ Percentage\ =\ 0}\ \%\ .$$

- **Interior Beam at 4th Basement Floor (B2A)**

H = 1000mm

B = 1000mm

Torsion Force and Torsion Reinforcement for Torsion, T_u						
T_u kN-m	ϕT_{th} kN-m	ϕT_{cr} kN-m	Area A_o cm²	Perimeter, p_h mm	Rebar A_t/s mm²/m	Rebar A_l mm²
0.235	169.6435	678.574	7055.9	3644.4	0	0

Tu on this beam = 0.235 KN.m

$$ØTth = Ø\lambda(1/12)\sqrt{fc'}\frac{Acp^2}{Pcp}$$

$$Acp = 1000 * 1000 = 1 * 10\text{^}6\, mm2$$

$$Pcp = 2(1000 + 1000) = 4000\, mm$$

$$ØTth = \frac{0.75 * 1 * \left(\frac{1}{12}\right)\sqrt{70} * \frac{10^{6^2}}{4000}}{10^6} = 130.73\, KN.m$$

$$ØTth = 130.73 > Tu = 0.235\ , \boldsymbol{no\ need\ to\ design\ for\ Torsion}$$

$$\boldsymbol{\frac{At}{s} = 0\ \frac{mm^2}{mm}}$$

From ETABS

$$\boldsymbol{\frac{At}{s} = 0\ \frac{mm^2}{mm}\ , Error\ percentage = 0}\,\%$$

3.1.2 Check Design for Columns

Take a column between the 4th and 3rd Basement Floors

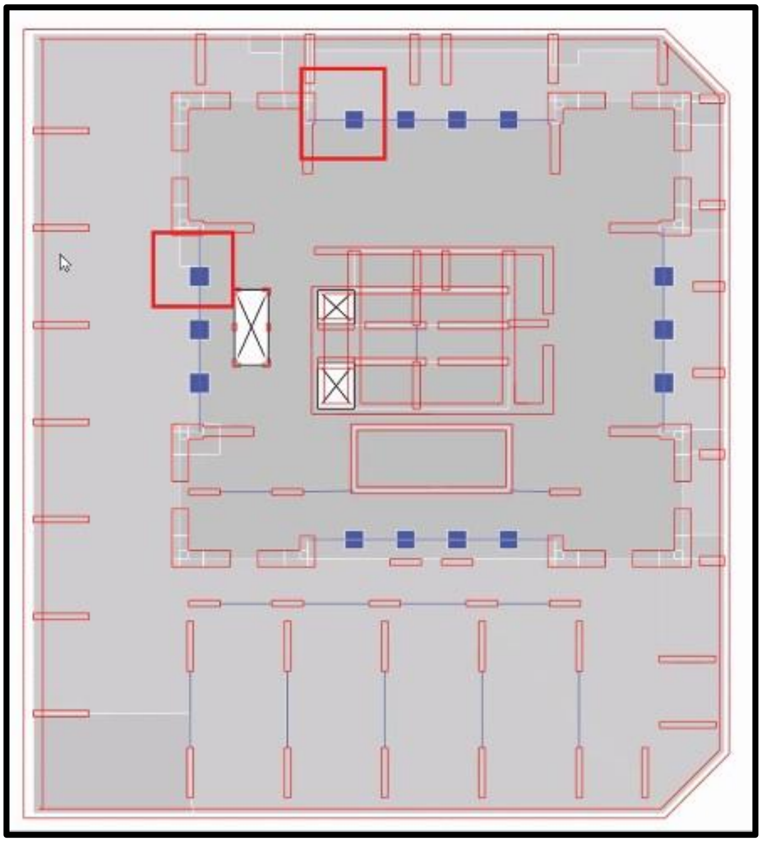
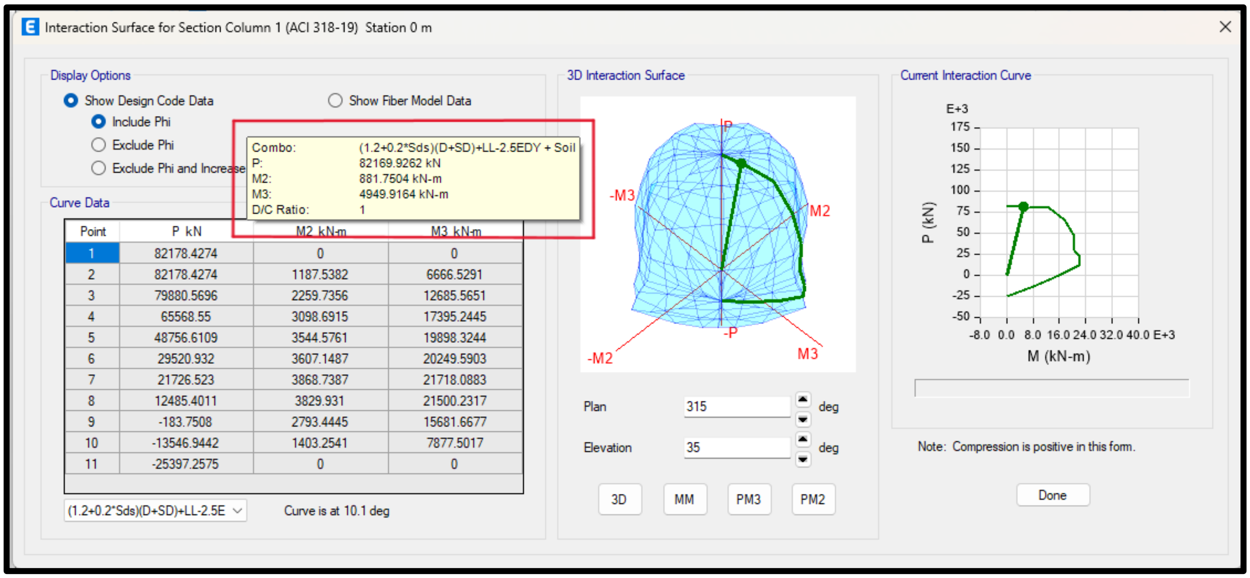


Figure 3.1.2.1 – Location of Columns to Check

1. Column 1 (C1A)

1500 mm * 1500 mm



Point	P kN	M2 kN-m	M3 kN-m
1	82178.4274	0	0
2	82178.4274	1187.5382	6666.5291
3	79880.5696	2259.7356	12685.5651
4	65568.55	3098.6915	17395.2445
5	48756.6109	3544.5761	19898.3244
6	29520.932	3607.1487	20249.5903
7	21726.523	3868.7387	21718.0883
8	12485.4011	3829.931	21500.2317
9	-183.7508	2793.4445	15681.6677
10	-13546.9442	1403.2541	7877.5017
11	-25397.2575	0	0

Figure 3.1.2.2 – Column 1 P-M2-M3 Interaction Surface

Interaction diagram for column 1 (P-M2)

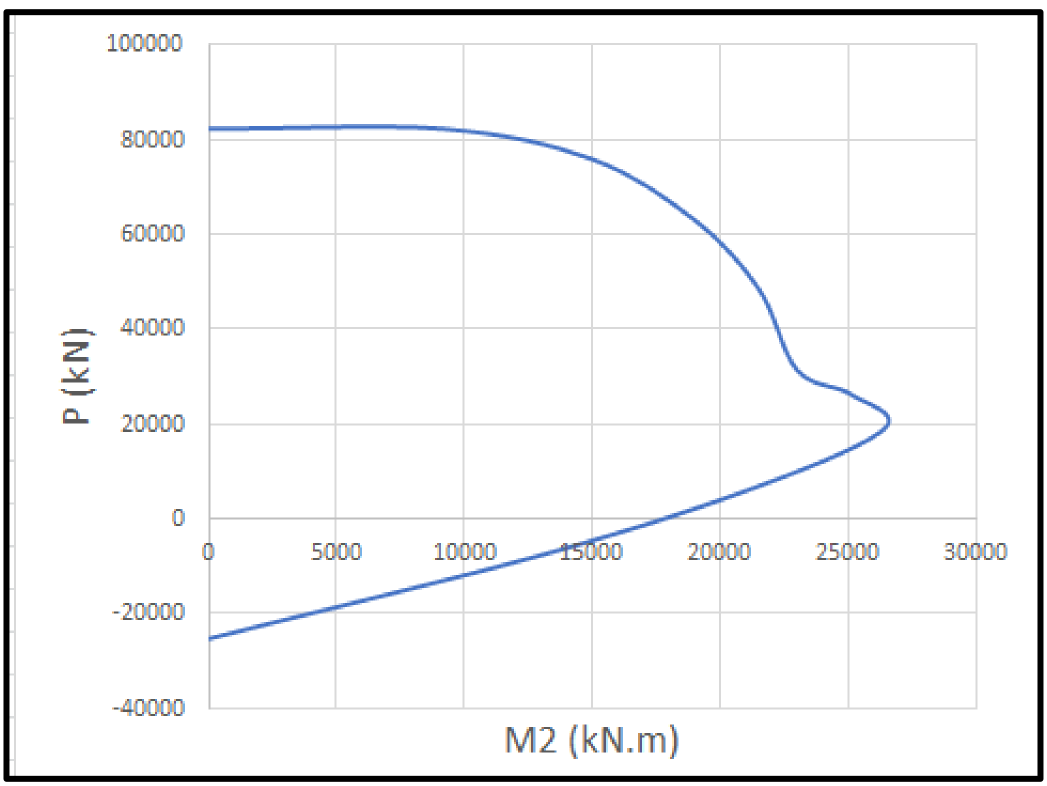


Figure 3.1.2.3 – Column 1 P-M2 Interaction Diagram

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 M2 kN-m	Minimum M3 kN-m	Rebar Area mm²	Rebar % %
82169.9262	881.7504	4949.9164	4949.9164	4949.9164	68214	3.03

The axial load on the column = 82169.93 KN and Mu_2 = 881.7504 KN.m and this value is under the curve.

Interaction diagram for column 1 (P-M3)

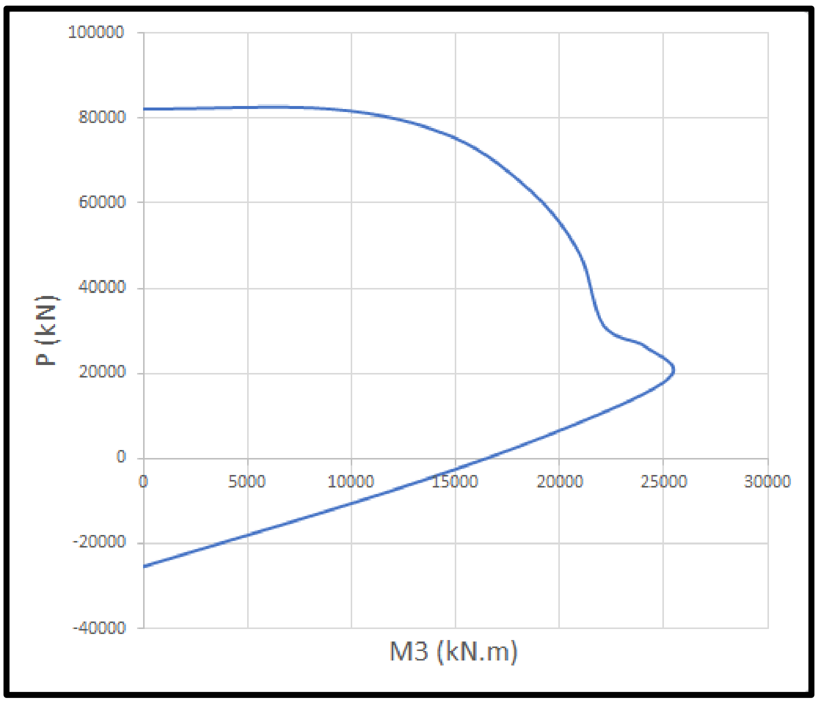


Figure 3.1.2.4 – Column 1 P-M3 Interaction Diagram

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 M2 kN-m	Minimum M3 kN-m	Rebar Area mm²	Rebar % %
82169.9262	881.7504	4949.9164	4949.9164	4949.9164	68214	3.03

The axial load on the column = 82169.93 KN and Mu_3 = 4949.9164 KN.m and this value is under the curve.

- Check Shear

Vu on column 1 = 581.37 KN

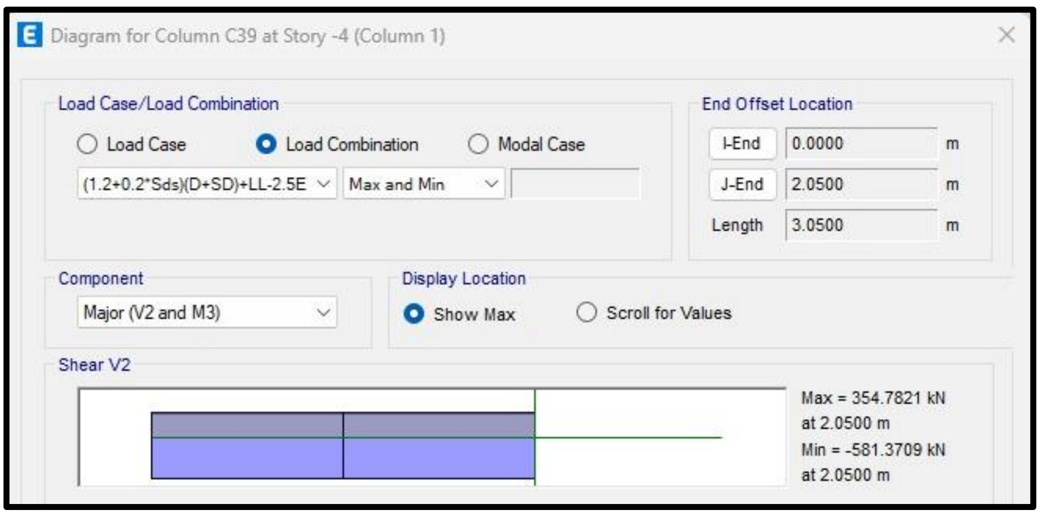


Figure 3.1.2.5 – Column 1 Shear Force

$$\frac{Vu}{\emptyset} = \frac{581.37}{0.75} = 775.16\ \text{KN}$$

$$Vc = \frac{1}{6}\lambda\sqrt{fc}bw * d = 2980.6\ \text{KN}$$

Vc > $\frac{Vu}{\emptyset}$, no need shear reinforcement

$$\frac{\mathbf{Av}}{\boldsymbol{s}} = \mathbf{0}\ \frac{\boldsymbol{mm^2}}{\boldsymbol{mm}}$$

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

	Shear V_u kN	Shear ϕV_c kN	Shear ϕV_s kN	Shear ϕV_p kN	Rebar A_v /s mm²/m
Major, V_{u2}	552.2833	5425.7409	0	519.7279	0
Minor, V_{u3}	232.8297	5425.7409	0	0	0

From ETABS

$$\frac{\mathbf{Av}}{\mathbf{S}} = \mathbf{0}\ \frac{\boldsymbol{mm^2}}{\boldsymbol{mm}}\ , \boldsymbol{error\ percentage} = \mathbf{0}\ \%\ .$$

2. Column 2

1600 mm * 1600 mm

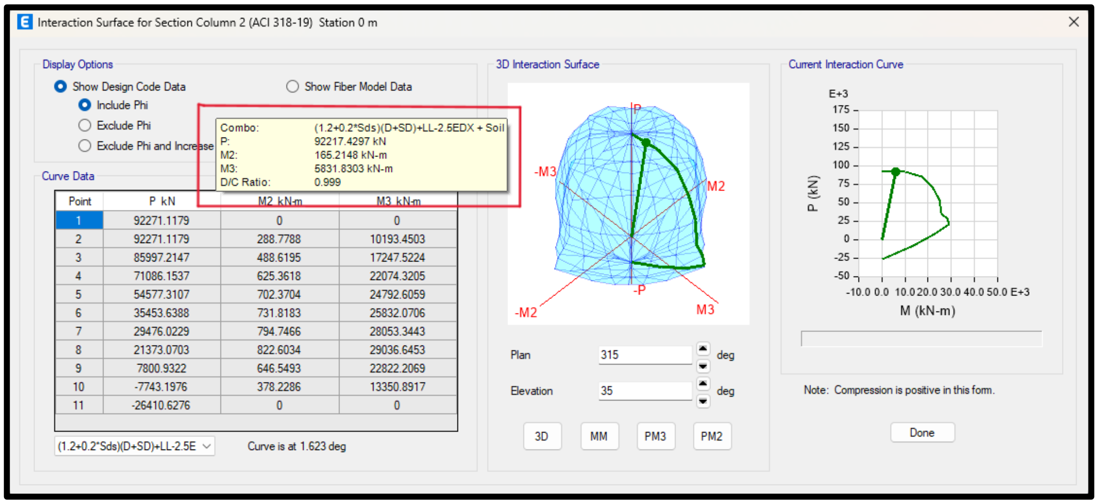


Figure 3.1.2.6 – Column 2 P-M2-M3 Interaction Surface

Interaction diagram for column 2 (P-Mu2)

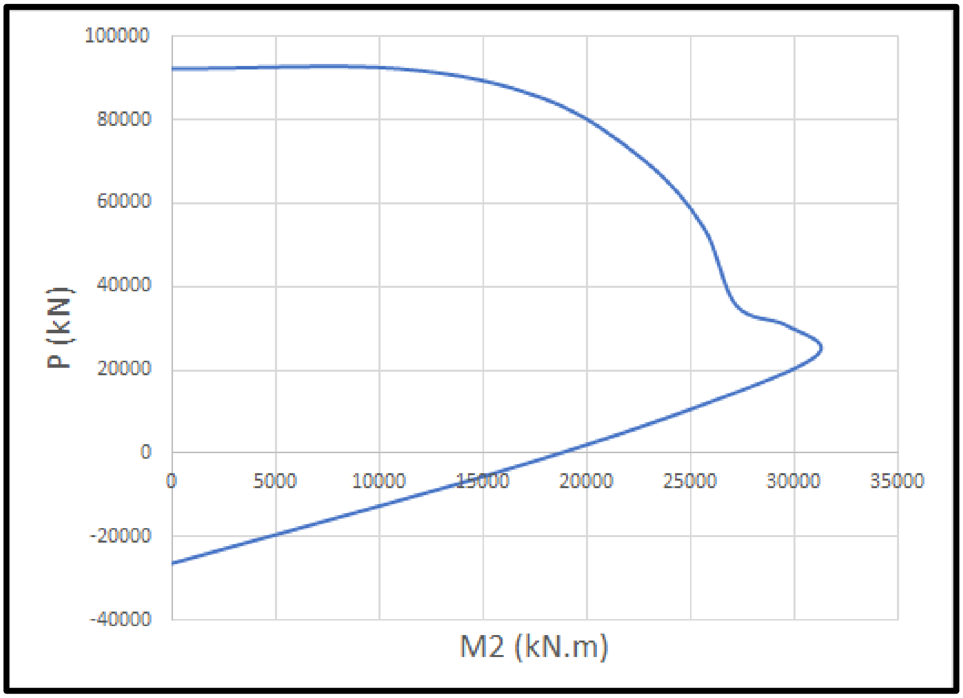


Figure 3.1.2.7 – Column 2 P-M2 Interaction Diagram

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 M2 kN-m	Minimum M3 kN-m	Rebar Area mm²	Rebar % %
92217.4297	165.2148	5831.8303	5831.8303	5831.8303	70936	2.77

The axial load on the column = 92217.43 KN and Mu2 = 165.2148 KN.m and this value is under the curve.

Interaction diagram for column 2 (P-Mu3)

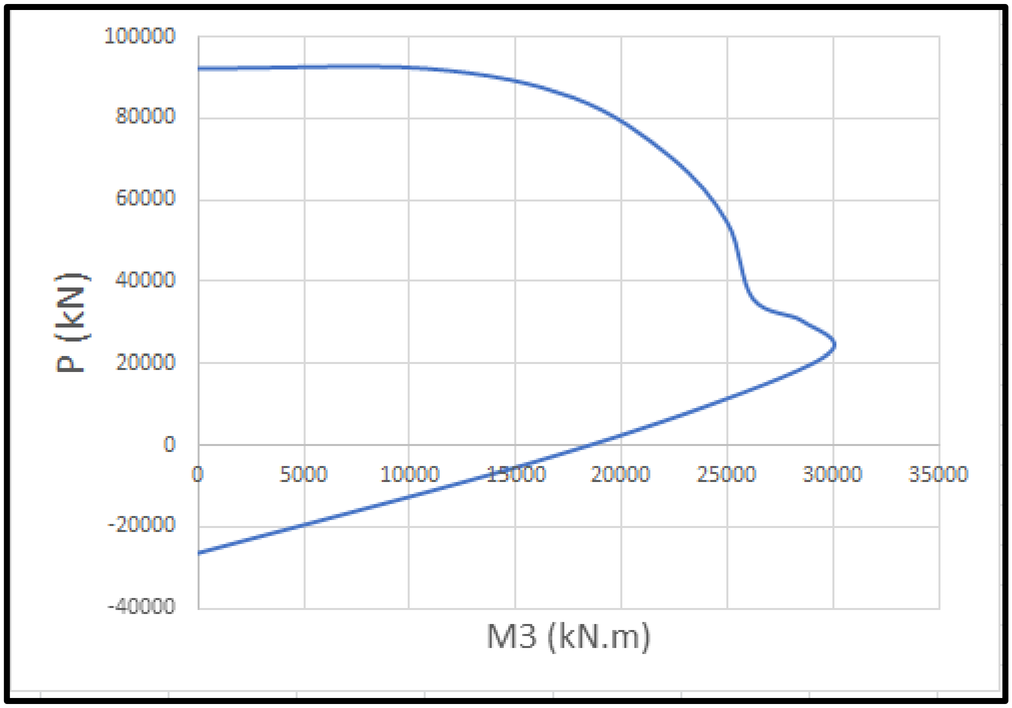


Figure 3.1.2.8 – Column 2 P-M3 Interaction Diagram

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 M2 kN-m	Minimum M3 kN-m	Rebar Area mm²	Rebar % %
92217.4297	165.2148	5831.8303	5831.8303	5831.8303	70936	2.77

The axial load on the column = 92217.43 KN and Mu3= 5831.8303 KN.m and this value is under the curve.

- Check Shear

Vu on column 1 = 542.79 KN

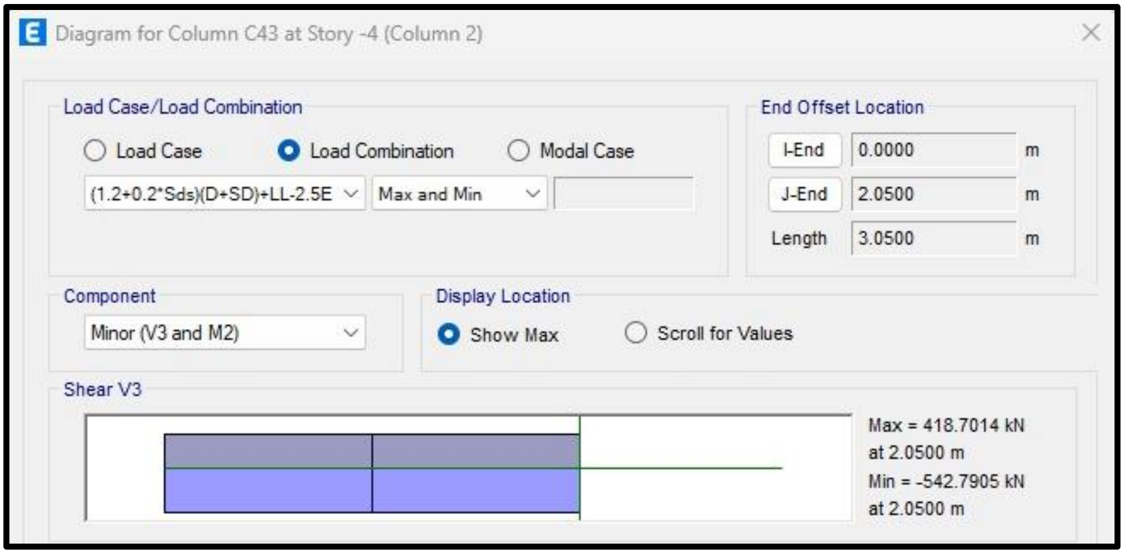


Figure 3.1.2.9 – Column 2 Shear Force

$$\frac{\text{Vu}}{\emptyset} = \frac{542.79}{0.75} = 723.72 \text{ KN}$$

$$\text{Vc} = \frac{1}{6}\lambda\sqrt{fc}bw * d = 3402.42 \text{ KN}$$

$\text{Vc} > \frac{\text{Vu}}{\emptyset}$ **, no need for shear reinforcement**.

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

	Shear V_u kN	Shear ϕV_c kN	Shear ϕV_s kN	Shear ϕV_p kN	Rebar A_v /s mm²/m
Major, V_{u2}	248.3681	6201.1424	0	0	0
Minor, V_{u3}	320.0074	6201.1424	0	257.9628	0

$$\frac{\text{Av}}{\text{s}} = \mathbf{0}\ \frac{\boldsymbol{mm^2}}{\boldsymbol{mm}}$$

From ETABS

$\frac{\textbf{Av}}{\textbf{S}} = \mathbf{0}\ \frac{\boldsymbol{mm^2}}{\boldsymbol{mm}}$ **so, error percentage = 0 %.**

3.1.3 Check Design for Slabs

1. Floor Slab Floor (50): Direction 1 Top Rebar: Load Combo = (1.2+0.2SDS)(D+SD) + LL+ EDX+ Soil.

- The max value of top rebar intensity from ETABS is 3200 $\frac{mm^2}{m}$ according to the critical section.
- M11 = 100 kN.m, Cover = 15 mm, h = 250 mm, Fy = 413.69 MPa, f'c = 70 MPa, b = 1000 mm, spacing=250 mm, bar size: 18.
- As= 0.25 x 3200 = 800 mm^2.

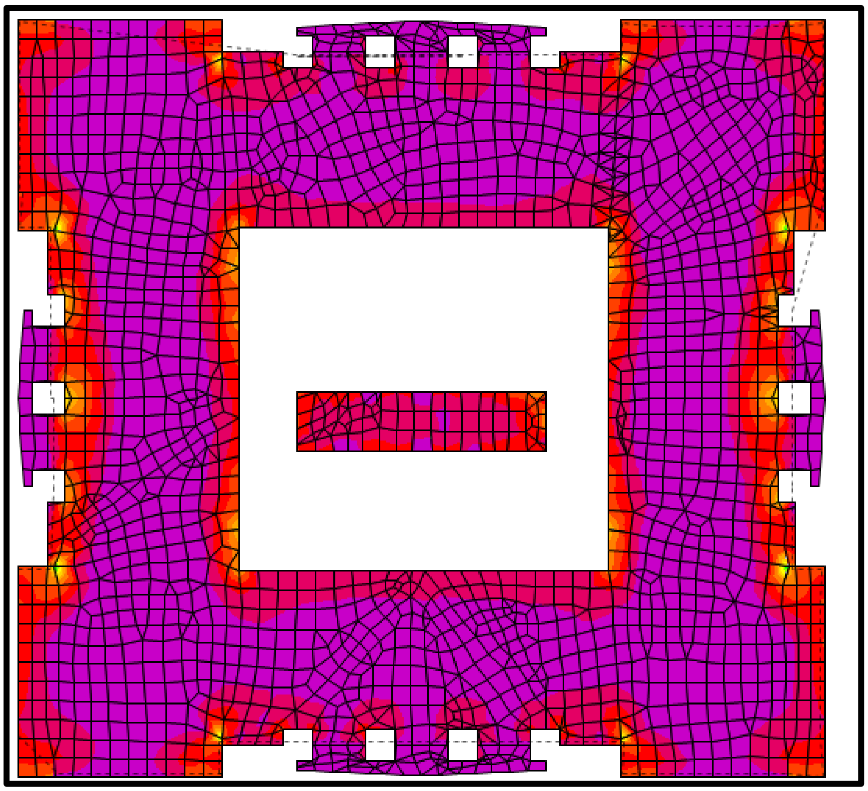


Figure 3.1.3.1 – Floor Slab Top Rebar Direction-1

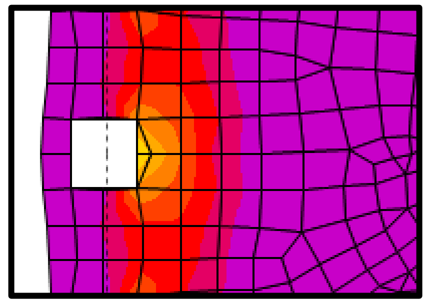


Figure 3.1.3.2 – Floor Slab Critical Section Direction-1

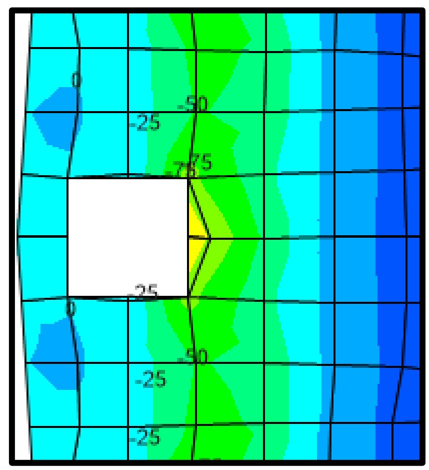


Figure 3.1.3.3 – Floor Slab M11 at Critical Section

$$\beta_1 = 0.85 - 0.05\left(\frac{70-28}{7}\right) = 0.55 < 0.65\ we\ take\ \beta_1 = 0.65$$

$$\rho = \frac{0.65(70)}{413.69}\left(1 - \sqrt{1 - \frac{2.61 \times 100 \times 10^6}{1000 \times (250-15)^2 \times 70}}\right) = 0.00377251$$

$$A_s = \rho \times b \times d = 0.00377251 \times 1000 \times (250-15) = \mathbf{886.53985\ mm^2}$$

$$Error\ percentage = \left(\frac{800}{800-886.53985}\right) \times 100 = \mathbf{10.82}\% < \mathbf{25}\%\ , \boldsymbol{Acceptable}$$

2. Floor Slab: Direction 2 Top Rebar: Load Combo = (1.2+0.2SDS) (D+SD) + LL+ EDX+ Soil.

- The max value of top rebar intensity from ETABS is 4800 $\frac{mm^2}{m}$ according to the critical section.
- M22 = 150 kN.m, Cover = 15 mm, h = 250 mm, Fy = 413.69 MPa, f'c = 70 MPa, b = 1000 mm, spacing=250 mm, bar size: 18.

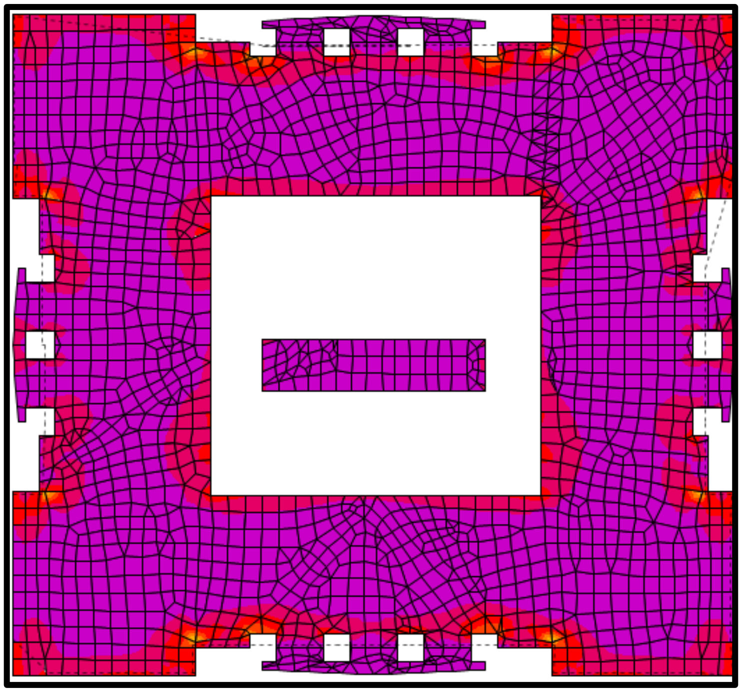


Figure 3.1.3.4 – Floor Slab Top Rebar Direction-2

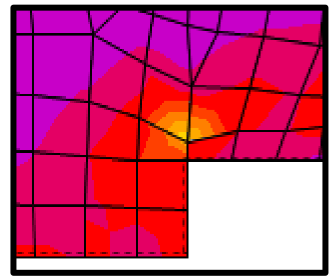


Figure 3.1.3.5 – Floor Slab Critical Section Direction-1

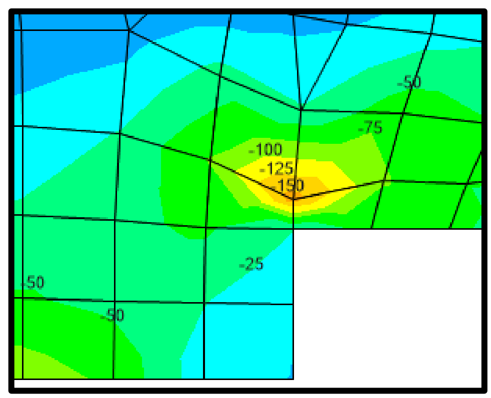


Figure 3.1.3.6 – Floor Slab M22 at Critical Section

$$As = 0.25\ x\ 4800 = 1200\ mm^2.$$

$$\rho = \frac{0.65(70)}{413.69}\left(1 - \sqrt{1 - \frac{2.61 \times 150 \times 10^6}{1000 \times (250 - 15)^2 \times 70}}\right) = 0.0057193$$

$$A_s = \rho \times b \times d = 0.0057193 \times 1000 \times (250 - 15) = 1344.0355\ mm^2$$

$$Error\ Percentage = \left(\frac{1200}{1200 - 1344.0355}\right) \times 100 = \mathbf{12\%} < \mathbf{25\%}, \boldsymbol{Acceptable}$$

3.1.4 Check Design for Wall

Around the core (50th Floor)

a. **Piers - Pier 47**

ETABS Shear Wall Design

ACI 318-19 Pier Design

Pier Details

Story ID	Pier ID	Centroid X (mm)	Centroid Y (mm)	Length (mm)	Thickness (mm)	LLRF
50	P47	49898.1	44253.9	4100.1	850	0.4

Material Properties

E_c (MPa)	f'_c (MPa)	Lt. Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
39323	70	1	413.69	413.69

Design Code Parameters

ϕ_T	ϕ_C	ϕ_V	ϕ_V (Seismic)	IP_{MAX}	IP_{MIN}	P_{MAX}
0.9	0.65	0.75	0.6	0.04	0.0025	0.8

Pier Leg Location, Length and Thickness

Station Location	ID	Left X_1 mm	Left Y_1 mm	Right X_2 mm	Right Y_2 mm	Length mm	Thickness mm
Top	Leg 1	49898.1	42203.9	49898.1	46303.9	4100.1	850
Bottom	Leg 1	49898.1	42203.9	49898.1	46303.9	4100.1	850

Flexural Design for P_u, M_{u2} and M_{u3}

Station Location	Required Rebar Area (mm²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P_u kN	M_{u2} kN-m	M_{u3} kN-m	Pier A_g mm²
Top	8713	0.0025	0.0074	(0.9-0.2Sds)(D+SD)-EDY	35580.5414	148.4359	-7433.7508	3485041
Bottom	8713	0.0025	0.0074	(0.9-0.2Sds)(D+SD)-EDY	35093.6884	-123.5882	-2017.4797	3485041

Shear Design

Station Location	ID	Rebar mm²/m	Shear Combo	P_u kN	M_u kN-m	V_u kN	ϕV_c kN	ϕV_n kN
Top	Leg 1	2125	1.2(D + SD) + LL - Wind Y + Soil	47672.2891	-8638.3385	2721.3417	3604.2786	6307.4878
Bottom	Leg 1	2125	1.2(D + SD) + LL - Wind X + Soil	43495.3013	792.1278	2871.1354	3604.2786	6307.4878

Boundary Element Check (ACI 18.10.6.3, 18.10.6.4) (Part 1 of 2)

Station Location	ID	Edge Length (mm)	Governing Combo	P_u kN	M_u kN-m	Stress Comp MPa
Top–Left	Leg 1	2155.8	(1.2+0.2*Sds) (D+SD) +LL+EDX + Soil	55842.03	-299.8829	16.15
Top–Right	Leg 1	2155.8	(1.2+0.2*Sds) (D+SD) +LL+EDX + Soil	55842.03	-299.8829	15.9
Bottom–Left	Leg 1	2126.7	(1.2+0.2*Sds) (D+SD) +LL+EDX + Soil	55149.3887	1813.7721	15.06
Bottom–Right	Leg 1	2126.7	(1.2+0.2*Sds) (D+SD) +LL+EDX + Soil	55149.3887	1813.7721	16.59

Boundary Element Check (ACI 18.10.6.3, 18.10.6.4) (Part 2 of 2)

Stress Limit MPa	C Depth mm	C Limit mm
14	2565.8	911.1
14	2565.8	911.1
14	2536.7	911.1
14	2536.7	911.1

- **Shear Check (Bottom)**

Length L_w= 4100.1 mm

Thickness b_w= 850 mm.

Height of Wall h_w = 183.05 - 179.4 = 3.65 m = 3650 mm.

Vu = 2871.1354 kN.

ΦVc = 3604.2786 kN, ΦVn = 6307.4878 kN. (From ETABS).

Transverse Steel, Av = 2125 $\frac{mm^2}{m}$.

f'c = 70 MPa, Fy = 413.69 MPa.

Spacing = 120 mm.

$$\frac{h_w}{l_w} = \frac{3650}{4100.1} = 0.89$$

$$\alpha_c = 0.25 \, for \, \frac{h_w}{l_w} \leq 1.5$$

$$\rho_t = \frac{A_{v,horizontal}}{bs_2} = \frac{2125 \times 0.12}{850 \times 120} = 0.0025$$

$$V_n = \left(\alpha_c \lambda \sqrt{f'_c} + \rho_t f_{yt}\right) A_{cv} = \frac{\left(0.25(1)\sqrt{70} + 0.0025 \times 413.69\right) \times (850 \times 4100.1)}{1000}$$

$$= 10893.94031 \, kN.$$

$$\Phi V_n = (0.6 \times 10893.94031) = 6536.3642 \, kN.$$

$$\%difference = \left(\left(\frac{6307.4878}{6536.3642}\right)\right) \times 100 = \mathbf{3.6\%} < \mathbf{25\%}\, , \boldsymbol{Acceptable}$$

$$\Phi V_c = \Phi \alpha_c \lambda \sqrt{f'_c} \, A_{cv} = \frac{0.6(0.25)(1)\sqrt{70}\,(850 \times 4100.1)}{1000} = 4373.7470 \, kN.$$

$$Error\ percentage = \left(\left(\frac{3604.2786}{4373.7470}\right)\right) \times 100 = \mathbf{21.35\%} < \mathbf{25\%}\, , \boldsymbol{Acceptable}.$$

- **Check stress for the boundary elements (bottom left side of wall)**

  - Mu = 1813.7721 kN.m.
  - Pu = Nu = 55149.3887 =kN.
  - Stress = 15.06 MPa. (From ETABS)
  - Stress Limit = 14 MPa. (From ETABS)
  - C depth = 2536.7 mm. (From ETABS)
  - C limit = 911.1 mm. (From ETABS)
  - Edge Length = 2126.7 mm.
  - y = 4100.1/2= 2050.05 mm.

$$\sigma = \frac{N_u}{A} \pm \frac{M_u\, y}{I} = \frac{55149.3887(1000)}{(850 \times 4100.1)} - \frac{1813.7721x10^6(2050.05)}{\frac{1}{12}(850)(4100.1)^3} = 15.82 - 0.76$$

$$= 15.06\ MPa.$$

$$Error\ percentage = \left(\left(\frac{15.06}{15.06}\right) - 1\right) \times 100 = \mathbf{0\%} < \mathbf{25\%}\ , \boldsymbol{Acceptable}$$

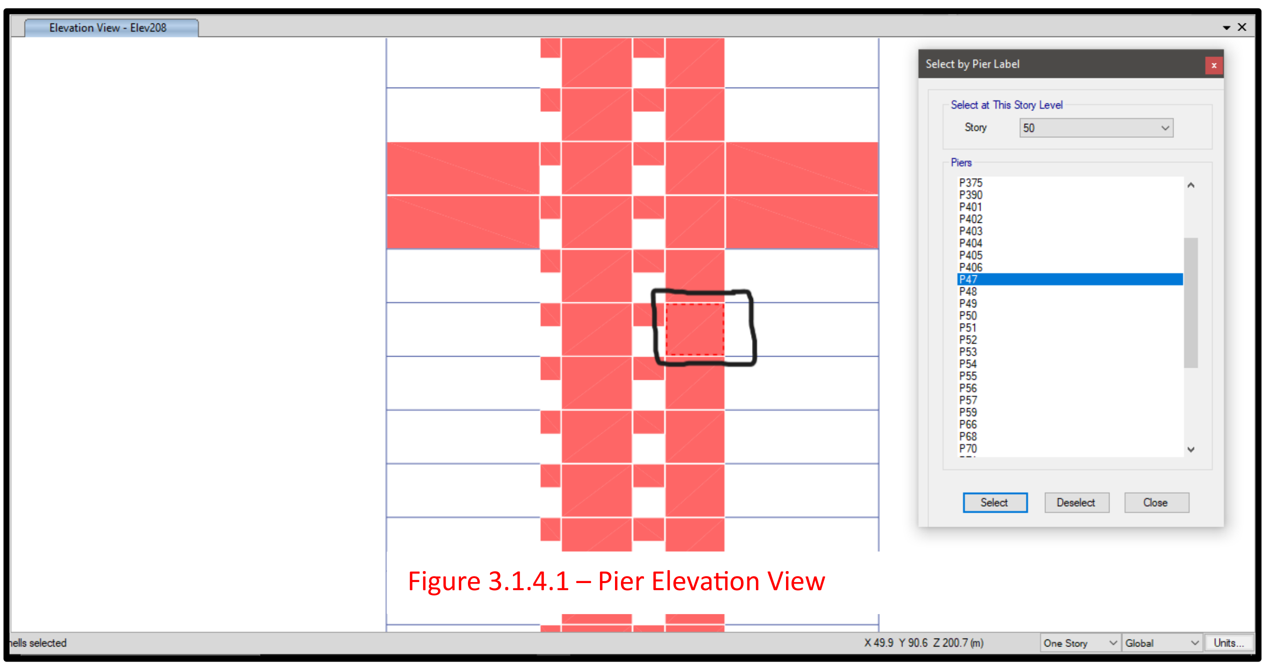


Figure 3.1.4.1 – Pier Elevation View

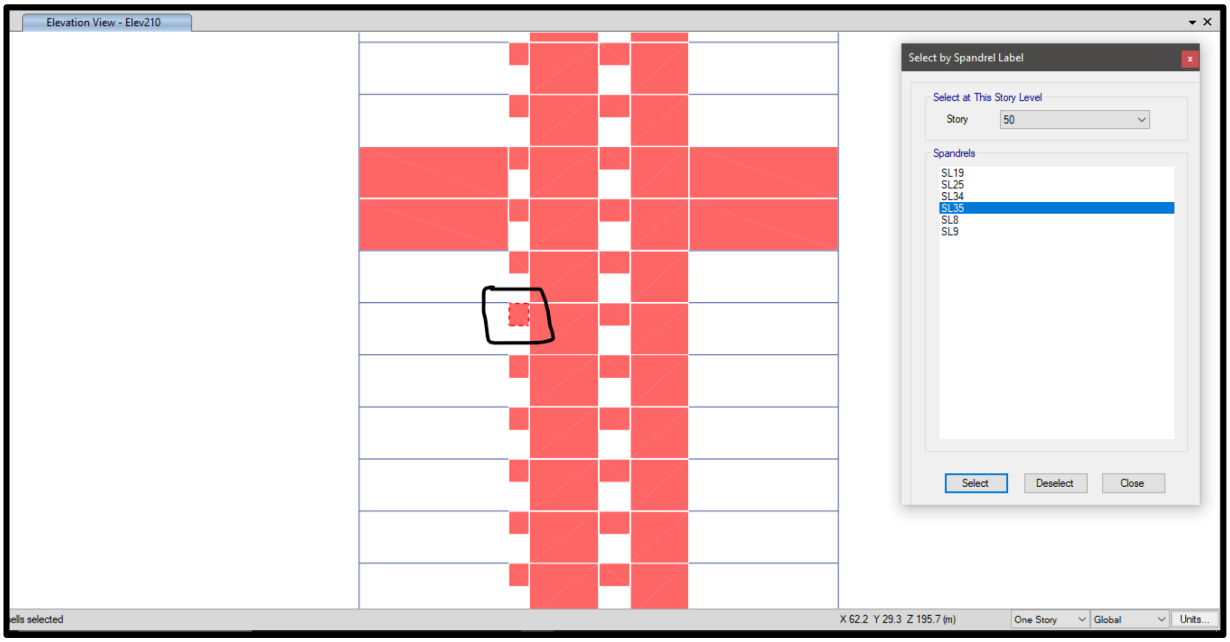


Figure 3.1.4.2 – Spandrel Elevation View

- **Flexural Check**

Ultimate moment at the bottom of the wall = 123.6 KN.m

$$\rho = \frac{0.85(28)}{420}\left(1 - \sqrt{1 - \frac{2.61(123.6*10^6)}{1000(850-75)^2(28)}}\right) = 0.00055$$

In the shear walls the value of $\rho \min = 0.0025 \; so \; take \;\; \rho = 0.0025$

$$As \; = \; 0.0025(4100)(850 - 75) \; = \; 7943.75 \; mm^2$$

$$Error\ percentage \; = \frac{8713 - 7943.75}{7943.75} = \; \mathbf{9.68}\,\% \; < \; \mathbf{25}\,\%\,, \boldsymbol{Acceptable}$$

Ultimate moment at the top of the wall = 148.4 KN.m

$$\rho = \frac{0.85(28)}{420}\left(1 - \sqrt{1 - \frac{2.61(148.4 * 10^6)}{1000(850 - 75)^2(28)}}\right) = 0.00066$$

In the shear walls the value of $\rho \min = 0.0025 \; so \; take \;\; \rho = 0.0025$

$$As \; = \; 0.0025(4100)(850 - 75) = \; \mathbf{7943.75 \; mm^2}.$$

$$Error\ percentage \; = \frac{8713 - 7943.75}{7943.75} = \; \mathbf{9.68}\,\% \; < \; \mathbf{25}\,\%\,, \boldsymbol{Acceptable}$$

b. Spandrel 35 Design

ETABS Shear Wall Design

ACI 318-19 Spandrel Design

Spandrel Details

Story ID	Spandrel ID	Centroid X (mm)	Centroid Y (mm)	Depth (mm)	Width (mm)	LLRF
50	SL35	62198	53354.1	1650	850	1

Material Properties

E_c (MPa)	f'_c (MPa)	Lt. Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
39323	70	1	413.69	413.69

Design Code Parameters

ϕ_T	ϕ_C	ϕ_V	ϕ_V (Seismic)	ϕ_d (diagonal)
0.9	0.65	0.75	0.6	0.85

Spandrel Flexural Design—Top Reinforcement

Station Location	Reinf Area mm²	Reinf Percentage	Reinf Combo	Moment, M_u kN-m
Left	1452	0.1	(0.9-0.2Sds)(D+SD)-EDY	-674.7566
Right	214	0.02	0.9(D+SD) - Wind Y	-152.9445

Spandrel Flexural Design—Bottom Reinforcement

Station Location	Reinf Area mm²	Reinf Percentage	Reinf Combo	Moment, M_u kN-m
Left	1409	0.1	0.9(D+SD) - Wind Y	472.4915
Right	904	0.06	(0.9-0.2Sds)(D+SD)-EDY	510.3027

Spandrel Shear Design

Station Location	A_{vert} mm²/m	A_{horiz} mm²/m	ShearCombo	V_u kN	ϕV_c kN	ϕV_s kN	ϕV_n kN
Left	2125	2125	1.2(D + SD) + LL + Wind X + Soil	1650.762	1467.6622	979.0772	2446.7393
Right	2125	2125	1.2(D + SD) + LL + Wind X + Soil	1137.3518	1418.0473	979.0772	2397.1245

Spandrel Shear Design—Diagonal Reinforcement

Station Location	A_{diag} mm²	Shear Combo	V_u kN	V_{uLimit} kN	L/H Ratio	Seismic Design	Diag Reinf Mandatory
Left	3389	(1.2+0.2*Sds) (D+SD) +LL+EDY + Soil	1604.7266	3481.1633	0.879	Yes	No
Right	2076	(1.2+0.2*Sds) (D+SD) +LL+EDY + Soil	982.835	3481.1633	0.879	Yes	No

- Beam Section Width (b) = 850 mm.
- Beam Section Thickness (h) = 1650 mm.
- Beam Section Effective Depth (d) = 0.9 (1650) = 1485 mm.
- Beam Section Length (L)=Ln= 1449.8 mm.
- $\Phi_s = 0.85$

- **Shear Check**

$$\frac{L_n}{h} = \frac{1449.8}{1650} = 0.87 < 4\ .$$

$$\frac{h_w}{L_w} = \frac{1650}{1449.8} = 1.13 < 1.5\ so\ \alpha_c = 0.25$$

$$V_n = \left(\alpha_c \lambda \sqrt{f'_c} + \rho_t f_{yt}\right) A_{cv} = \frac{\left(0.25(1)\sqrt{70} + 0.0025 \times 413.69\right) \times (850 \times 1650)}{1000}$$
$$= 4384.0398\ kN.$$

$$\Phi V_n = (0.6 \times 4384.0398) = 2630.42388\ kN.$$

$$Error\ percentage = \left(\left(\frac{2446.7393}{2630.42388}\right) - 1\right) \times 100 = \mathbf{6.98}\ \% < \mathbf{25}\%\ , \boldsymbol{Acceptable}$$

$$\Phi V_c = \Phi \alpha_c \lambda \sqrt{f'_c}\, A_{cv} = \frac{0.6(0.25)(1)\sqrt{70}\ (1650 \times 850)}{1000} = 1760.1235\ kN.$$

$$Error\ percentage\ = \left(\left(\frac{1467.6622}{1760.1235}\right) - 1\right) \times 100 = \mathbf{16.6}\ \% < \mathbf{25}\%\ , \boldsymbol{Acceptable}$$

- **Diagonal Shear Reinforcement Check**

$$A_{vd} = \frac{V_u}{2\Phi_s f_y sin\alpha}$$

$$sin\alpha = \frac{0.8h_s}{sqrt((L_s)^2 + (0.8h_s)^2)} = \frac{0.8(1650)}{sqrt((1449.8)^2 + (0.8 \times 1650)^2)} = 0.67$$

$$A_{vd} = \frac{V_u}{2\Phi_s f_y sin\alpha} = \frac{1604.7266(1000)}{2(0.85)(413.69)(0.67)} = 3405.6680\ mm^2$$

$$A_{vd}\ from\ ETABS = 3389\ mm^2$$

$$Error\ percentage = \left(\left(\frac{3389}{3405.6680}\right) - 1\right) \times 100 = \mathbf{0.489}\ \% < \mathbf{25}\%\ , \boldsymbol{Acceptable}$$

- **Flexural Check**

Ultimate moment at the bottom of the wall = 510.3027 KN.m

$$\rho = \frac{0.85(28)}{420}\left(1 - \sqrt{1 - \frac{2.61(510.3027*10^6)}{850(1650-75)^2(28)}}\right) = 0.00064$$

$$As\ =\ 0.00064(850)(1650 - 75)\ =\ 856.8\ mm^2$$

$$Error\ percentage\ = \frac{904 - 856.8}{856.8} =\ \mathbf{5.5}\ \%\ <\ \mathbf{25}\ \%, \boldsymbol{Acceptable}$$

Ultimate moment at the top of the wall = 674.8 KN.m

$$\rho = \frac{0.85(28)}{420}\left(1 - \sqrt{1 - \frac{2.61(674.8*10^6)}{850(1650-75)^2(28)}}\right) = 0.00085$$

$$As\ =\ 0.00085(850)(1650 - 75)\ =\ 1137.94\ mm^2$$

$$Error\ = \frac{1452 - 1137.94}{1452} =\ \mathbf{21.63}\ \%\ <\ \mathbf{25}\ \%\ , \boldsymbol{Acceptable}$$

3.2 Design of Beams

Minimum Shear Reinforcement

Table 9.6.3.4—Required $A_{v,min}$

Beam type	$A_{v,min}/s$		
Nonprestressed and prestressed with $A_{ps}f_{se} < 0.4(A_{ps}f_{pu} + A_s f_y)$	Greater of:	$0.062\sqrt{f'_c}\dfrac{b_w}{f_{yt}}$	(a)
		$0.35\dfrac{b_w}{f_{yt}}$	(b)

Table 3.2.1 – Required Minimum Shear Reinforcement for Beams

f'_c = 70 MPa

f_{yt} = 420 MPa

Take the greater of:

- $0.001235b_w$
- $0.0008333b_w$

Take $0.001235b_w$ for all cases

Notes:

- For west-south ends, take west end if the beam is on the x-axis and take south end if the beam is on the y-axis.
- For east-north spans, take east end if the beam is on the x-axis and take north end if the beam is on the y-axis.
- For Parking Edge Beam (B3) and Modifying Beam (B2), reinforcement is the same in both directions, x, and y, thus no need for any subcategories per direction.
- For beam label references, use "Beam Labels" sheet in "Beams Excel File."
- For any detailed reinforcement requirement, check "Reinforcement" sheet in "Beams Excel File."
- For development lengths, splicing lengths, and confinement spacings details for each beam, check their sheets respectively in "Beams Excel File."

1. **Interior Beams at 5th Basement Floor to 5th Parking Floor (B1XA)**

Thickness = 1000 mm

Width = 800 mm

Cover = 75 mm

Design load combination:

(1.2 + 0.2Sds) (D + SD) + LL – 2.5EDX + Soil

Longitudinal reinforcement.

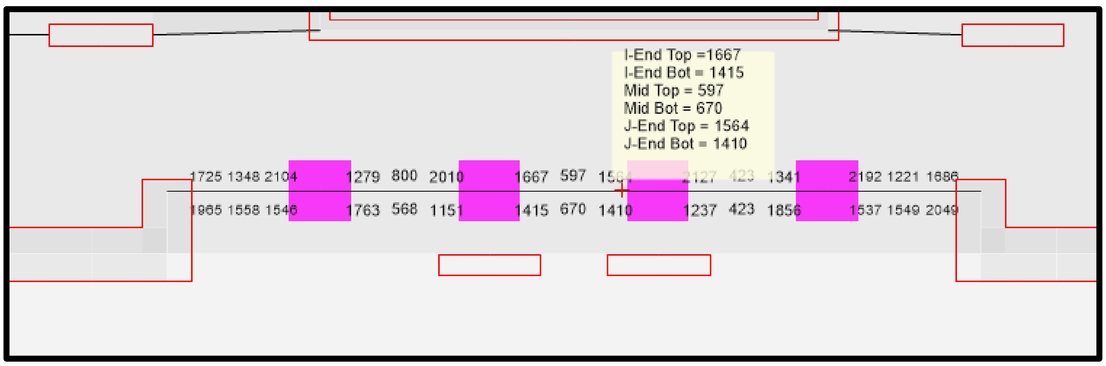


Figure 3.2.1 – Interior Beam Flexural Longitudinal

Top west end steel = 1667 mm^2

Top east end steel = 1564 mm^2

Top middle steel = 597 mm^2

Bottom west end steel = 1415 mm^2

Bottom east end steel = 1410 mm^2

Bottom middle steel = 670 mm^2

Use $6\emptyset 20$ at top west end with spacing of 130 mm $= 1885\ mm^2\ (excess\ reinforcement)$

Use $6\emptyset 20$ at top east end with spacing of 130 mm $= 1885\ mm^2\ (excess\ reinforcement)$

Use $6\emptyset 12$ at top middle with spacing of 130 mm $= 679\ mm^2\ (excess\ reinforcement)$

Use $6\emptyset 18$ at bottom west end with spacing of 130 mm $=$
$1527\ mm^2\ (excess\ reinforcement)$

Use $6\emptyset 18$ at bottom east end with spacing of 130 mm $= 1527\ mm^2\ (excess\ reinforcement)$

Use $6\emptyset 12$ at bottom middle with spacing of 130 mm $= 679\ mm^2\ (excess\ reinforcement)$

Use additional $4\emptyset 16$ at top and bottom

Development Length (ι_{dt}):

West end and East end:

$$\iota_{dt} = \frac{0.48 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b < 20\ \text{mm}$$

$$\iota_{dt} = \frac{0.59 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b \geq 20\ \text{mm}$$

$$\iota_{dt} \geq 300\ \text{mm}$$

$$d_b = 20\ \text{mm}$$

$$\iota_{dt} = \frac{0.59 * 420}{\sqrt{70}} * 20 = 592.35\ \text{mm}$$

Since there is excess reinforcement, use modification factor that equals: $\frac{A_{s,required}}{A_{s,provided}}$

$\frac{1667}{1885} = 0.884$, $\iota_{dt} = 0.884 * 592.35 = 524\ mm$, $\boldsymbol{take\ it\ equal\ 525\ mm}$.

Development of Hooks

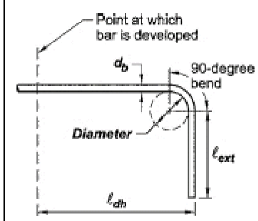
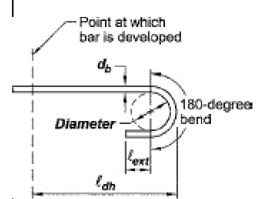
Type of standard hook	Bar size	Minimum inside bend diameter, mm	Straight extension[1] ℓ_{ext}, mm	Type of standard hook
90-degree hook	No. 10 through No. 25	6*d* *b*	$12d_b$	
	No. 29 through No. 36	8*d* *b*		
	No. 43 and No. 57	10		
180-degree hook	No. 10 through No. 25	6*d* *b*	Greater of $4d_b$ and 65 mm	
	No. 29 through No. 36	8*d* *b*		
	No. 43 and No. 57	10 *db*		

Table 3.2.2 – Development of Hooks Requirements

$$\ell_{dh} = \frac{f_y * \varphi_e * \varphi_r * \varphi_o * \varphi_c}{23\lambda * \sqrt{f'_c}} * {d_b}^{1.5} \geq 8d_b \geq 150\ mm$$

$\varphi_e = 1.0$

$\varphi_r = 1.6$

$\varphi_o = 1.25$

$\varphi_c = 1.0$

$\lambda = 1.0$

$$\ell_{dh} = \frac{0.087 f_y}{\lambda * \sqrt{f'_c}} * {d_b}^{1.5}$$

$$\ell_{dh} = \frac{0.087*420}{1*\sqrt{70}} * 20^{1.5} = 390.63\ mm$$

$8d_b = 8 * 20 = 160\ mm$

Take ℓ_{dh} = 395 mm

$\ell_{ext} = 12d_b = 12 * 20 = 240\ mm$, ***take it as 160 mm.***

Minimum inside bend diameter = $6d_b = 6 * 20 = \mathbf{120\ mm}$

Lap splicing:

$l_{st} = 1.3 l_{dt} = 1.3 * 525 = 582.5\ mm > 300\ mm$, **take it = 680 mm.**

Development Length (L_{dh})

Middle:

$$\iota_{dt} = \frac{0.48 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b < 20 \text{ mm}$$

$$\iota_{dt} = \frac{0.59 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b \geq 20 \text{ mm}$$

$$\iota_{dt} \geq 300 \text{ mm}$$

$$d_b = 12 \text{ mm}$$

$$\iota_{dt} = \frac{0.48 * 420}{\sqrt{70}} * 12 = 289.15 \text{ mm}$$

Take ι_{dt} = 300 mm

Since there is excess reinforcement, use modification factor that equals: $\frac{A_{s,required}}{A_{s,provided}}$

But, since the tension development length is already at the minimum value, there is no need to reduce it using the modification factor.

Development of Hooks

$$\iota_{dh} = \frac{f_y * \varphi_e * \varphi_r * \varphi_o * \varphi_c}{23\lambda * \sqrt{f'_c}} * d_b{}^{1.5} \geq 8d_b \geq 150\ mm$$

$\varphi_e = 1.0$

$\varphi_r = 1.6$

$\varphi_o = 1.25$

$\varphi_c = 1.0$

$\lambda = 1.0$

$$\iota_{dh} = \frac{0.087 f_y}{\lambda * \sqrt{f'_c}} * d_b{}^{1.5}$$

$$\iota_{dh} = \frac{0.087 * 420}{1 * \sqrt{70}} * 12^{1.5} = 181.54\ mm$$

$8d_b = 8 * 12 = 96\ mm$

Take ι_{dh} = 185 mm

$\iota_{ext} = 12d_b = 12 * 12 = 144\ mm$, **take it 145 mm**

Minimum inside bend diameter = $6d_b = 6 * 12 = \mathbf{72\ mm}$

Lap splicing:

$l_{st} = 1.3 l_{dt} = 1.3 * 300 = 390\ mm > 300\ mm$

Shear Reinforcement

Shear Force and Reinforcement for Shear, V_{u2}

Shear V_{u2} kN	Shear ϕV_c kN	Shear ϕV_s kN	Shear V_p kN	Rebar A_v /s mm²/m
497.6817	765.3181	286.9943	449.561	1000

Av/s = 1 mm^2/mm

Use four legs stirrups of diameter 8 mm with spacing of 150 mm.

Torsion Reinforcement

Torsion Force and Torsion Reinforcement for Torsion, T_u

T_u kN-m	ϕT_{th} kN-m	ϕT_{cr} kN-m	Area A_o cm^2	Perimeter, p_h mm	Rebar A_t/s mm^2/m	Rebar A_l mm^2
7.792	103.528	414.1118	5507	3244.4	0	0

No torsion reinforcement needed.

Design Check for Beam

The positive moment strength at the face of the joint shall be at least one-third the negative moment strength provided at that face of the joint. Neither the negative nor the positive moment strength at any section along the length of the beam shall be less than one-fifth the maximum moment strength provided at the face of either joint.

Moment strength $\text{Ø}Mn = 0.9 * A_s * f_y * \left(d - \frac{A_s * f_y}{1.7f'_c b}\right)$

At east end joint:

Area of steel at bottom = 1527 mm^2

Area of steel at top = 1885 mm^2

Positive ØMn = 530.027 kN.m

Negative ØMn = 653.16 kN.m

$\frac{653.16}{3} = 217.72$ kN. m < Positive ØMn, Acceptable

At west end joint:

Area of steel at bottom = 1527 mm^2

Area of steel at top = 1885 mm^2

Positive ØMn = 530.027 kN.m

Negative ØMn = 653.16 kN.m

$\frac{653.16}{3} = 217.72$ kN. m < Positive ØMn, Acceptable

Maximum moment strength is for A_s = 1885 mm^2

ØMn = 653.16 kN.m

$$\frac{653.16}{5} = 130.63 \text{ kN.m}$$

Minimum moment strength along the span is for A_s = 679 mm^2

ØMn = 236.64 kN.m > 130.63 kN.m

ϕVn shall be at least the lesser of (a) and (b):

a) The sum of the shear associated with development of nominal moment strengths of the beam at each restrained end of the clear span due to reverse curvature bending and the shear calculated for factored gravity and vertical earthquake loads.
b) The maximum shear obtained from design load combinations that include *E*, with *E* taken as twice that prescribed by the general building code.

$ØV_s = 0.75*f_{yt}*d*Av/s$

Av/s = 1.34 mm^2/mm

V_s = 0.75*420*925*1.34 = 390442 N = 390.442 kN

$ØV_n = ØV_c + ØVs$ = 765.3181 + 390.442 = 1155.76 kN

V_u from design load combination = 497.68 kN

$ØV_n > V_u$, Acceptable

2. Edge Beam at 1st Typical Floor (33rd to 43rd Floors) (B1XD)

Thickness = 1000 mm

Width = 800 mm

Cover = 75 mm

Design load combination

(1.2 + 0.2Sds) (D + SD) + LL – 2.5EDX + Soil

Longitudinal Reinforcement

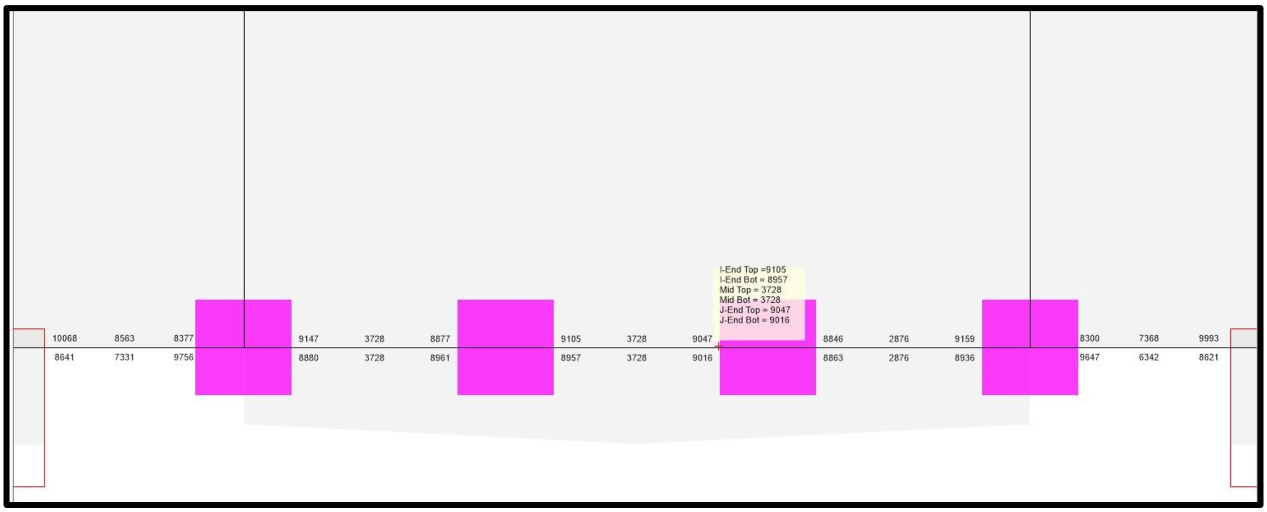


Figure 3.2.2 – Edge Beam Flexural Longitudinal Reinforcement

Top west end steel = 9105 mm^2

Top east end steel = 9047 mm^2

Top middle steel = 3728 mm^2

Bottom west end steel = 8957 mm^2

Bottom east end steel = 9018 mm^2

Bottom middle steel = 3728 mm^2

Use 14$\emptyset$30 at top west end with spacing of 50 mm $= 9896\ mm^2\ (excess\ reinforcement)$

Use 14$\emptyset$30 at top east end with spacing of 50 mm $= 9896\ mm^2\ (excess\ reinforcement)$

Use 6Ø30 at top middle with spacing of 130 mm $= 4241\ mm^2\ (excess\ reinforcement)$

Use 14Ø30 at bottom west end with spacing of 50 mm =
$9896\ mm^2\ (excess\ reinforcement)$

Use 14Ø30 at bottom east end with spacing of 50 mm =
$9896\ mm^2\ (excess\ reinforcement)$

Use 6Ø30 at bottom middle with spacing of 130 mm $= 4241\ mm^2\ (excess\ reinforcement)$

Use additional 4Ø20 at top and bottom

Development Length (ι_{dt})

West end and East end

$$\iota_{dt} = \frac{0.48 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b < 20 \text{ mm}$$

$$\iota_{dt} = \frac{0.59 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b \geq 20 \text{ mm}$$

$\iota_{dt} \geq 300$ mm

$d_b = 30$ mm

$$\iota_{dt} = \frac{0.59 * 420}{\sqrt{70}} * 30 = 888.53 \text{ mm}$$

Since there is excess reinforcement, use modification factor that equals: $\frac{A_{s,required}}{A_{s,provided}}$

Take largest as provided thus $\frac{9105}{9896} = 0.92$

$l_{dt} = 888.53 * 0.92 = 817.44\ mm$, ***take 820 mm.***

Development of Hooks

$$\iota_{\mathrm{dh}} = \frac{f_y * \varphi_e * \varphi_r * \varphi_o * \varphi_c}{23\lambda * \sqrt{f'_c}} * {d_b}^{1.5} \geq 8d_b \geq 150\ mm$$

$\varphi_e = 1.0$

$\varphi_r = 1.6$

$\varphi_o = 1.25$

$\varphi_c = 1.0$

$\lambda = 1.0$

$$\iota_{\mathrm{dh}} = \frac{0.087 f_y}{\lambda * \sqrt{f'_c}} * {d_b}^{1.5}$$

$\iota_{\mathrm{dh}} = \frac{0.087*420}{1*\sqrt{70}} * 30^{1.5} = \mathbf{717.63\ mm}$

$8d_b = 8 * 30 = 240\ mm$

Take ι_{dh} = **720 mm**

$\iota_{\mathrm{ext}} = 12d_b = 12 * 30 = \mathbf{360\ mm}$

Minimum inside bend diameter = $8d_b = 8 * 30 = \mathbf{240\ mm}$

Lap splicing

$l_{st} = 1.3 l_{dt} = 1.3 * 820 = \mathbf{1066\ mm} > 300\ mm$

Development Length (L_{dt})

Middle

$$\iota_{\mathrm{dt}} = \frac{0.48 * \mathrm{f_y}}{\sqrt{\mathrm{f'_c}}} * \mathrm{d_b} \quad \text{for } \mathrm{d_b} < 20\ \mathrm{mm}$$

$$\iota_{\mathrm{dt}} = \frac{0.59 * \mathrm{f_y}}{\sqrt{\mathrm{f'_c}}} * \mathrm{d_b} \quad \text{for } \mathrm{d_b} \geq 20\ \mathrm{mm}$$

$\iota_{\mathrm{dt}} \geq 300\ \mathrm{mm}$

$d_b = 30$ mm

$$\iota_{dt} = \frac{0.59 * 420}{\sqrt{70}} * 30 = 888.53 \text{ mm}$$

Since there is excess reinforcement, use modification factor that equals: $\frac{A_{s,required}}{A_{s,provided}}$

$\frac{3728}{4241} = 0.879\ , l_{dt} = 0.879 * 888.53 = 781.05\ mm\ , \boldsymbol{take\ it\ as\ 780\ mm}$.

Development of Hooks

$$\iota_{dh} = \frac{f_y * \varphi_e * \varphi_r * \varphi_o * \varphi_c}{23\lambda * \sqrt{f'_c}} * {d_b}^{1.5} \geq 8d_b \geq 150\ mm$$

$\varphi_e = 1.0$

$\varphi_r = 1.6$

$\varphi_o = 1.25$

$\varphi_c = 1.0$

$\lambda = 1.0$

$$\iota_{dh} = \frac{0.087 f_y}{\lambda * \sqrt{f'_c}} * {d_b}^{1.5}$$

$\iota_{dh} = \frac{0.087*420}{1*\sqrt{70}} * 30^{1.5} = 717.63\ mm\ , \boldsymbol{take\ it\ as\ 710\ mm}$.

$8d_b = 8 * 30 = 240\ mm$

Take ι_{dh} = 150 mm

$\iota_{ext} = 12d_b = 12 * 30 = \boldsymbol{360\ mm}$

Minimum inside bend diameter = $8d_b = 8 * 30 = \boldsymbol{240\ mm}$

Lap splicing

$l_{st} = 1.3 l_{dt} = 1.3 * 710 = 932.92\ mm > 300\ mm\ , take\ it\ \mathbf{930\ mm}$.

Shear Reinforcement

Shear Force and Reinforcement for Shear, V_{u2}

Shear V_{u2} kN	Shear ϕV_c kN	Shear ϕV_s kN	Shear V_p kN	Rebar A_v /s mm²/m
1852.7391	765.3181	1087.421	2553.5802	3789

$$\frac{A_v}{s} = 3.789\ mm^2/mm$$

Use stirrups with six legs of diameter 10 mm with spacing of 100 mm.

Torsion Reinforcement

Torsion Force and Torsion Reinforcement for Torsion, T_u

T_u kN-m	ϕT_{th} kN-m	ϕT_{cr} kN-m	Area A_o cm²	Perimeter, p_h mm	Rebar A_t /s mm²/m	Rebar A_l mm²
14.3032	86.7734	347.0938	5507	3244.4	0	0

No Torsion Reinforcement needed.

Design Checks for Beam

The positive moment strength at the face of the joint shall be at least one-third the negative moment strength provided at that face of the joint. Neither the negative nor the positive moment strength at any section along the length of the beam shall be less than one-fifth the maximum moment strength provided at the face of either joint.

Moment strength $\text{ØMn} = 0.9 * A_s * f_y * \left(d - \frac{A_s * f_y}{1.7f'_c b}\right)$

At east end joint:

Area of steel at bottom = 9896 mm^2

Area of steel at top = 9896 mm^2

Positive ØMn = 3296.8 kN.m

Negative ØMn = 3296.8 kN.m

$\frac{3296.8}{3} = 1098.9$ kN.m < Positive ØMn, Acceptable

At west end joint:

Area of steel at bottom = 9896 mm^2

Area of steel at top = 9896 mm^2

Positive ØMn = 3296.8 kN.m

Negative ØMn = 3296.8 kN.m

$\frac{3296.8}{3} = 1098.9$ kN.m < Positive ØMn, Acceptable

Maximum moment strength is for A_s = 9896 mm^2

ØMn = 3296.8 kN.m

$$\frac{3296.8}{5} = 659.3 \text{ kN.m}$$

Minimum moment strength along the span is for A_s = 4241 mm^2

ØMn = 1452.8 kN.m > 659.3 kN.m   Acceptable.

ϕVn shall be at least the lesser of (a) and (b):

a) The sum of the shear associated with development of nominal moment strengths of the beam at each restrained end of the clear span due to reverse curvature bending and the shear calculated for factored gravity and vertical earthquake loads.
b) The maximum shear obtained from design load combinations that include *E*, with *E* taken as twice that prescribed by the general building code.

$ØV_s = 0.75*f_{yt}*d*Av/s$

$Av/s = 4.7123\ mm^2/mm$

$V_s = 0.75*420*925*4.7123 = 1373072$ N = 1373.072 kN

$ØV_n = ØV_c + ØVs = 765.3181 + 1373.072 = 2138.39$ kN

V_u from design load combination = 1852.74 kN

$ØV_n > V_u$, Acceptable

3. **Interior Beam at 6th Basement Floor to 5th Parking Floor (B2A)**

Thickness = 1000 mm

Width = 1000 mm

Cover = 75 mm

Design Load Combination:

1.2(D + SD) + 1.6LL + 1.6Soil

Longitudinal Reinforcement

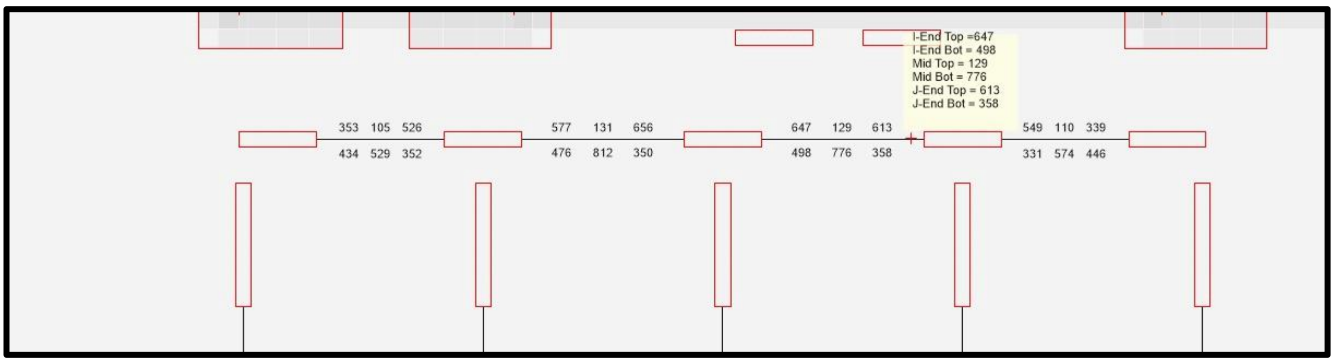


Figure 3.2.3 – Interior Beam Flexural Longitudinal Reinforcement

Top west end steel = 647 mm^2

Top east end steel = 613 mm^2

Top middle steel = 129 mm^2

Bottom west end steel = 498 mm^2

Bottom east end steel = 358 mm^2

Bottom middle steel = 776 mm^2

Use 6Ø12 at top west end with spacing of 170 mm $= \ 679\ mm^2\ (excess\ reinforcement)$

Use 6Ø12 at top east end with spacing of 170 mm $= \ 679\ mm^2\ (excess\ reinforcement)$

Use 2Ø10 at top middle with spacing of 850 mm $= \ 157\ mm^2\ (excess\ reinforcement)$

Use 6∅12 at bottom west end with spacing of 170 mm = 679 mm^2 (*excess reinforcement*)

Use 6∅10 at bottom east end with spacing of 170 mm = 471 mm^2 (*excess reinforcement*)

Use 6∅14 at bottom middle with spacing of 170 mm = 923 mm^2 (*excess reinforcement*)

Use additional 4∅10 at top and bottom

Development Length (ι_{dt})

West end and East end

$$\iota_{dt} = \frac{0.48 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b < 20 \text{ mm}$$

$$\iota_{dt} = \frac{0.59 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b \geq 20 \text{ mm}$$

$\iota_{dt} \geq 300$ mm

$d_b = 12$ mm

$$\iota_{dt} = \frac{0.48 * 420}{\sqrt{70}} * 12 = 289.15 \text{ mm}$$

Since there is excess reinforcement, use modification factor that equals: $\frac{A_{s,required}}{A_{s,provided}}$

Take largest As provided thus $\frac{647}{679} = 0.95$, $l_{dt} = 289.15 * 0.95 =$ 276 mm , *take it as* **270 mm**

Development of Hooks

$$\iota_{\mathrm{dh}} = \frac{f_y * \varphi_e * \varphi_r * \varphi_o * \varphi_c}{23\lambda * \sqrt{f'_c}} * d_b{}^{1.5} \geq 8d_b \geq 150\ mm$$

$\varphi_e = 1.0$

$\varphi_r = 1.6$

$\varphi_o = 1.25$

$\varphi_c = 1.0$

$\lambda = 1.0$

$$\iota_{\mathrm{dh}} = \frac{0.087 f_y}{\lambda * \sqrt{f'_c}} * d_b{}^{1.5}$$

$$\iota_{\mathrm{dh}} = \frac{0.087*420}{1*\sqrt{70}} * 12^{1.5} = 181.55\ mm$$

$8d_b = 8 * 12 = 96\ mm$

Take $\iota_{\mathbf{dh}}$ = 185 mm

$\iota_{\mathrm{ext}} = 12d_b = 12 * 12 = 144\ mm$, $\boldsymbol{take\ 145\ mm}$.

Minimum inside bend diameter = $6 * d_b = 6 * 12 = 72\ \ mm$, $\boldsymbol{take\ 70\ mm}$.

Lap Splicing

$l_{st} = 1.3 l_{dt} = 1.3 * 276 = 358.18\ mm > 300\ mm$, $\boldsymbol{take\ 350\ mm}$.

Development Length (L_{dt})

Top Middle

$$\iota_{\mathrm{dt}} = \frac{0.48 * \mathrm{f_y}}{\sqrt{\mathrm{f'_c}}} * \mathrm{d_b} \quad \text{for } \mathrm{d_b} < 20\ \mathrm{mm}$$

$$\iota_{\mathrm{dt}} = \frac{0.59 * \mathrm{f_y}}{\sqrt{\mathrm{f'_c}}} * \mathrm{d_b} \quad \text{for } \mathrm{d_b} \geq 20\ \mathrm{mm}$$

$\iota_{\mathrm{dt}} \geq 300\ \mathrm{mm}$

$\mathrm{d_b} = 10\ \mathrm{mm}$

$$\iota_{dt} = \frac{0.48 * 420}{\sqrt{70}} * 10 = 240.96 \text{ mm}$$

Since there is excess reinforcement, use modification factor that equals: $\frac{A_{s,required}}{A_{s,provided}}$

$$\frac{129}{157} = 0.823\ , l_{dt} = 0.823 * 240.96 = \ 197.98\ mm\ , \boldsymbol{take\ l_{dt} = 300\ mm}\ .$$

Development of Hooks

$$\iota_{dh} = \frac{f_y * \varphi_e * \varphi_r * \varphi_o * \varphi_c}{23\lambda * \sqrt{f'_c}} * {d_b}^{1.5} \geq 8d_b \geq 150\ mm$$

$\varphi_e = 1.0$

$\varphi_r = 1.6$

$\varphi_o = 1.25$

$\varphi_c = 1.0$

$\lambda = 1.0$

$$\iota_{dh} = \frac{0.087 f_y}{\lambda * \sqrt{f'_c}} * {d_b}^{1.5}$$

$$\iota_{dh} = \frac{0.087*420}{1*\sqrt{70}} * 10^{1.5} = 138.10\ mm$$

$8d_b = 8 * 10 = 80\ mm$

Take ι_{dh} = 130 mm

$\iota_{ext} = 12d_b = 12 * 10 = \boldsymbol{120\ mm}$

Minimum inside bend diameter = $6d_b = 6 * 10 = \boldsymbol{60\ mm}$

Lap Splicing

$l_{st} = 1.3 l_{dt} = 1.3 * 300 = 390\ mm > 300\ mm\ , \boldsymbol{take\ it\ 350\ mm}.$

Bottom Middle

$$\iota_{dt} = \frac{0.48 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b < 20 \text{ mm}$$

$$\iota_{dt} = \frac{0.59 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b \geq 20 \text{ mm}$$

$\iota_{dt} \geq 300$ mm

$d_b = 14$ mm

$$\iota_{dt} = \frac{0.48 * 420}{\sqrt{70}} * 14 = 337.34 \text{ mm}$$

Since there is excess reinforcement, use modification factor that equals: $\frac{A_{s,required}}{A_{s,provided}}$

$\frac{776}{923} = 0.841\ , l_{dt} = 0.841 * 337.34 = \ 283.61\ mm\ , take\ \boldsymbol{l_{dt} = 300\ mm}$.

Development of Hooks

$$\iota_{dh} = \frac{f_y * \varphi_e * \varphi_r * \varphi_o * \varphi_c}{23\lambda * \sqrt{f'_c}} * {d_b}^{1.5} \geq 8d_b \geq 150\ mm$$

$\varphi_e = 1.0$

$\varphi_r = 1.6$

$\varphi_o = 1.25$

$\varphi_c = 1.0$

$\lambda = 1.0$

$$\iota_{dh} = \frac{0.087 f_y}{\lambda * \sqrt{f'_c}} * {d_b}^{1.5}$$

$\iota_{dh} = \frac{0.087*420}{1*\sqrt{70}} * 14^{1.5} = 228.78\ mm$

$8d_b = 8 * 14 = 112\ mm$

Take ι_{dh} = 150 mm

$l_{ext} = 12d_b = 12 * 14 = 168\ mm$, ***take it as* 160 *mm***.

Minimum inside bend diameter = $6d_b = 6 * 14 = 84\ mm$, ***take it as* 90 *mm***.

Lap Splicing

$l_{st} = 1.3l_{dt} = 1.3 * 300 = 390\ mm > 300\ mm$, ***take it* 350 *mm***.

Shear Reinforcement

Shear Force and Reinforcement for Shear, V_{u2}				
Shear V_{u2} kN	Shear ϕV_c kN	Shear ϕV_s kN	Shear V_p kN	Rebar A_v /s mm²/m
23.6765	236.8716	0	61.8852	0

Use minimum shear reinforcement

$Av_{min}/s = 0.001235b_w = 0.001235 * 1000 =$ **1.235 mm²/mm**

Use four legs stirrups of diameter 8 mm with spacing of 150 mm.

Torsion Reinforcement

Torsion Force and Torsion Reinforcement for Torsion, T_u						
T_u kN-m	ϕT_{th} kN-m	ϕT_{cr} kN-m	Area A_o cm²	Perimeter, p_h mm	Rebar A_t /s mm²/m	Rebar A_l mm²
10.0824	136.5192	546.0767	7055.9	3644.4	0	0

No torsion reinforcement.

Design Checks for Beam

The positive moment strength at the face of the joint shall be at least one-third the negative moment strength provided at that face of the joint. Neither the negative nor the positive moment strength at any section along the length of the beam shall be less than one-fifth the maximum moment strength provided at the face of either joint.

Moment strength $\emptyset Mn = 0.9 * A_s * f_y * \left(d - \frac{A_s * f_y}{1.7f'_c b}\right)$

At east end joint:

Area of steel at bottom = 471 mm^2

Area of steel at top = 679 mm^2

Positive ØMn = 164.389 kN.m

Negative ØMn = 236.79 kN.m

$\frac{236.79}{3} = 78.9\ \text{kN.m}$ < Positive ØMn, Acceptable.

At west end joint:

Area of steel at bottom = 679 mm^2

Area of steel at top = 679 mm^2

Positive ØMn = 236.79 kN.m

Negative ØMn = 236.79 kN.m

$\frac{236.79}{3} = 78.9\ \text{kN.m}$ < Positive ØMn, Acceptable.

Maximum moment strength is for A_s = 679 mm^2

ØMn = 236.79 kN.m

$$\frac{236.79}{5} = 47.36\ \text{kN.m}$$

Minimum moment strength along the span is for A_s = 157 mm^2

ØMn = 54.86 kN.m > 47.36 kN.m, Acceptable.

ϕVn shall be at least the lesser of (a) and (b)

a) The sum of the shear associated with development of nominal moment strengths of the beam at each restrained end of the clear span due to reverse curvature bending and the shear calculated for factored gravity and vertical earthquake loads.
b) The maximum shear obtained from design load combinations that include *E*, with *E* taken as twice that prescribed by the general building code.

$ØV_s = 0.75*f_{yt}*d*Av/s$

$Av/s = 1.34\ mm^2/mm$

$V_s = 0.75*420*925*1.34 = 390442\ N = 390.442\ kN$

$ØV_n = ØV_c + ØVs = 236.87 + 390.442 = 627.312\ kN$

V_u from design load combination = 23.67 kN

$ØV_n > V_u$ Acceptable.

4. **Edge Beam at Parking Levels (B3)**

Thickness = 750 mm

Width = 550 mm

Cover = 75 mm

Design load combination:

(1.2 + 0.2Sds) (D + SD) + LL – 2.5EDY + Soil

Longitudinal Reinforcement

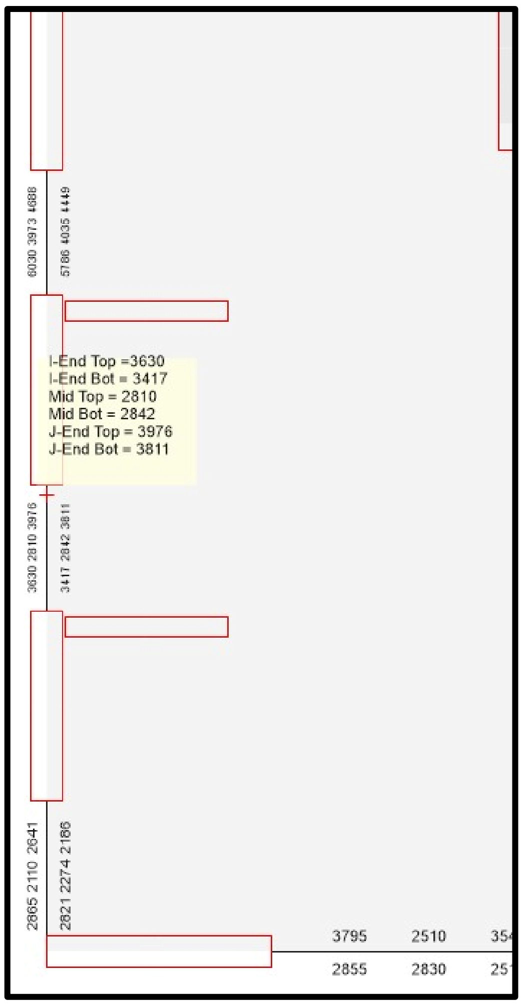


Figure 3.2.4 – Edge Beam at Parking Flexural Longitudinal Reinforcement

Top west end steel = 3630 mm^2

Top east end steel = 3976 mm^2

Top middle steel = 2810 mm^2

Bottom west end steel = 3417 mm^2

Bottom east end steel = 3811 mm^2

Bottom middle steel = 2842 mm^2

Use 6Ø28 at top west end with spacing of 80 mm = $3649\ mm^2$ $(excess\ reinforcement)$

Use 6Ø30 at top east end with spacing of 80 mm = $4241\ mm^2$ $(excess\ reinforcement)$

Use 6Ø25 at top middle with spacing of 80 mm = $2945\ mm^2$ $(excess\ reinforcement)$

Use 6Ø28 at bottom west end with spacing of 80 mm =
$3649\ mm^2$ $(excess\ reinforcement)$

Use 6Ø30 at bottom east end with spacing of 80 mm = $4241\ mm^2$ $(excess\ reinforcement)$

Use 6Ø25 at bottom middle with spacing of 80 mm = $2945\ mm^2$ $(excess\ reinforcement)$

Use additional 4Ø26 at top and bottom

Development Length (ι_{dt})

West end and East end

$$\iota_{dt} = \frac{0.48 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b < 20\text{ mm}$$

$$\iota_{dt} = \frac{0.59 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b \geq 20\text{ mm}$$

$$\iota_{dt} \geq 300\text{ mm}$$

$$d_b = 30\text{ mm}$$

$$\iota_{dt} = \frac{0.59 * 420}{\sqrt{70}} * 30 = 888.53\text{ mm}$$

Since there is excess reinforcement, use modification factor that equals: $\frac{A_{s,required}}{A_{s,provided}}$

Take largest As provided thus $\frac{3976}{4241} = 0.9375\ l_{dt} = 888.53 * 0.9375 = 833\ mm$, $\boldsymbol{take\ it\ as\ 635\ mm}$.

Development of Hooks

$$\iota_{dh} = \frac{f_y * \varphi_e * \varphi_r * \varphi_o * \varphi_c}{23\lambda * \sqrt{f'_c}} * {d_b}^{1.5} \geq 8d_b \geq 150\ mm$$

$\varphi_e = 1.0$

$\varphi_r = 1.6$

$\varphi_o = 1.25$

$\varphi_c = 1.0$

$\lambda = 1.0$

$$\iota_{dh} = \frac{0.087 f_y}{\lambda * \sqrt{f'_c}} * {d_b}^{1.5}$$

$\iota_{dh} = \frac{0.087*420}{1*\sqrt{70}} * 30^{1.5} = 717.6\ mm$

$8d_b = 8 * 30 = 240\ mm$

Take ι_{dh} = 720 mm

$\iota_{ext} = 12d_b = 12 * 30 = \mathbf{360\ mm}$.

Minimum inside bend diameter = $8 * d_b = 8 * 30 = \mathbf{240\ \ mm}$.

Lap Splicing

$l_{st} = 1.3 l_{dt} = 1.3 * 720 = 936\ mm > 300\ mm$, $\boldsymbol{take\ 940\ mm}$.

Development Length (L_{dt})

Middle

$$l_{dt} = \frac{0.48 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b < 20\text{ mm}$$

$$l_{dt} = \frac{0.59 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b \geq 20\text{ mm}$$

$$l_{dt} \geq 300\text{ mm}$$

$$d_b = 25\text{ mm}$$

$$l_{dt} = \frac{0.59 * 420}{\sqrt{70}} * 25 = 740.44\text{ mm}$$

Since there is excess reinforcement, use modification factor that equals: $\frac{A_{s,required}}{A_{s,provided}}$

$\frac{2842}{2945} = 0.965$, $l_{dt} = 0.965 * 740.44 = 714.5\ mm$, $\boldsymbol{take\ l_{dt} = 715\ mm}$.

Development of Hooks

$$l_{dh} = \frac{f_y * \varphi_e * \varphi_r * \varphi_o * \varphi_c}{23\lambda * \sqrt{f'_c}} * {d_b}^{1.5} \geq 8d_b \geq 150\ mm$$

$\varphi_e = 1.0$

$\varphi_r = 1.6$

$\varphi_o = 1.25$

$\varphi_c = 1.0$

$\lambda = 1.0$

$$l_{dh} = \frac{0.087 f_y}{\lambda * \sqrt{f'_c}} * {d_b}^{1.5}$$

$$l_{dh} = \frac{0.087*420}{1*\sqrt{70}} * 25^{1.5} = 546\ mm$$

$8d_b = 8 * 25 = 200\ mm$

Take ι_{dh} = 550 mm

$\iota_{ext} = 12d_b = 12 * 25 = \mathbf{300\ mm}$

Minimum inside bend diameter = $6d_b = 6 * 25 = \mathbf{150\ mm}$

Lap Splicing

$l_{st} = 1.3l_{dt} = 1.3 * 715 = 929.5\ mm > 300\ mm$, ***take it* 930 *mm*.**

Shear Reinforcement

Shear Force and Reinforcement for Shear, V_{u2}				
Shear V_{u2} kN	Shear ϕV_c kN	Shear ϕV_s kN	Shear V_p kN	Rebar A_v /s mm²/m
689.4203	375.4196	314.0007	632.2553	1533.4

$$\frac{\mathbf{A_v}}{\mathbf{s}} = \mathbf{1.5334\ mm^2/mm}$$

Use six legs stirrups of diameter 8 mm with spacing of 140 mm.

Torsion Reinforcement

Torsion Force and Torsion Reinforcement for Torsion, T_u						
T_u kN-m	ϕT_{th} kN-m	ϕT_{cr} kN-m	Area A_o cm²	Perimeter, p_h mm	Rebar A_t /s mm²/m	Rebar A_l mm²
28.9627	41.0668	164.2671	2591.1	2244.4	0	0

No torsion reinforcement needed.

Design Checks for Beam

The positive moment strength at the face of the joint shall be at least one-third the negative moment strength provided at that face of the joint. Neither the negative nor the positive moment strength at any section along the length of the beam shall be less than one-fifth the maximum moment strength provided at the face of either joint.

Moment strength $\text{ØMn} = 0.9 * A_s * f_y * \left(d - \frac{A_s * f_y}{1.7f'_c b}\right)$

At east end joint:

Area of steel at bottom = 4241 mm^2

Area of steel at top = 4241 mm^2

Positive ØMn = 1038.46 kN.m

Negative ØMn = 1038.46 kN.m

$\frac{1038.46}{3} = 346.15\ \text{kN.m}$ < Positive ØMn. Acceptable.

At west end joint:

Area of steel at bottom = 3649 mm^2

Area of steel at top = 3649 mm^2

Positive ØMn = 898.74 kN.m

Negative ØMn = 898.74 kN.m

$\frac{898.74}{3} = 299.58\ \text{kN.m}$ < Positive ØMn. Acceptable.

Maximum moment strength is for A_s = 4241 mm^2

ØMn = 1038.46 kN.m

$$\frac{1038.46}{5} = 207.69\ \text{kN.m}$$

Minimum moment strength along the span is for A_s = 2945 mm^2

ØMn = 730.3 kN.m > 207.69 kN.m Acceptable.

ϕVn shall be at least the lesser of (a) and (b):

a) The sum of the shear associated with development of nominal moment strengths of the beam at each restrained end of the clear span due to reverse curvature bending and the shear calculated for factored gravity and vertical earthquake loads.
b) The maximum shear obtained from design load combinations that include *E*, with *E* taken as twice that prescribed by the general building code.

$ØV_s = 0.75*f_{yt}*d*Av/s$

$Av/s = 2.154\ mm^2/mm$

$V_s = 0.75*420*675*2.154 = 457994\ N = 457.994\ kN$

$ØV_n = ØV_c + ØVs = 375.42 + 457.994 = 833.414\ kN$

V_u from design load combination = 689.42 kN

$ØV_n > V_{u,\ Acceptable}$.

5. Edge Beams at 79th Floor and 80th Floor (B4)

Thickness = 1500 mm

Width = 1500 mm

Cover = 75 mm

Design Load Combination:

(1.2 + 0.2Sds) (D + SD) + LL – 2.5EDX + Soil

Longitudinal Reinforcement

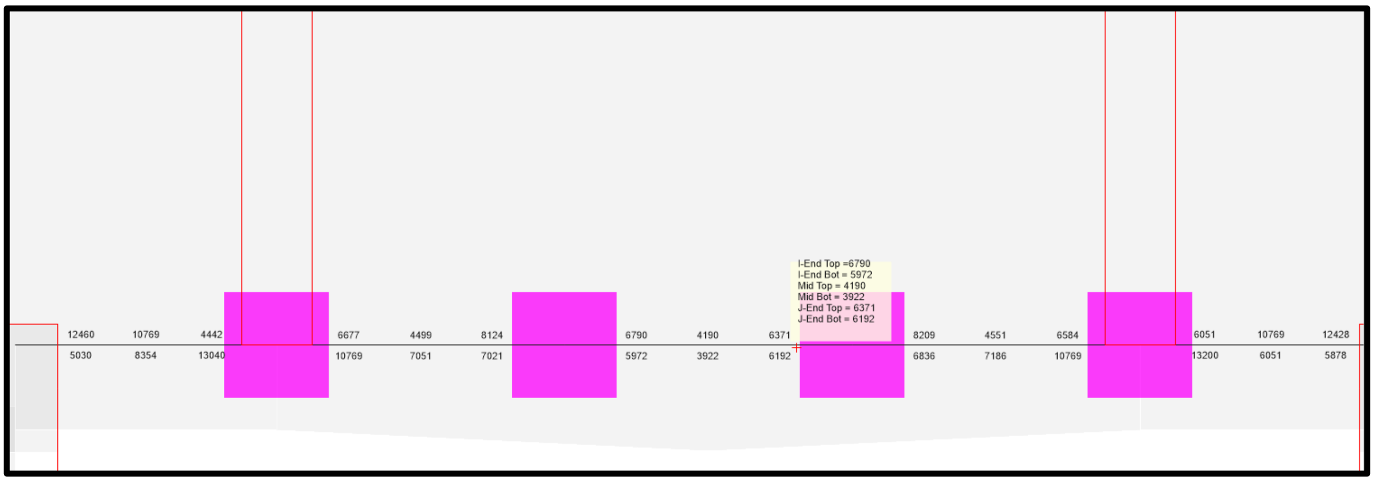


Figure 3.2.5 – Edge Beam Flexural Longitudinal Reinforcement

Top West end Steel = 6970 mm^2

Top East end Steel = 6371 mm^2

Top Middle Steel = 4190 mm^2

Bottom West end Steel = 5972 mm^2

Bottom East end Steel = 6192 mm^2

Bottom Middle Steel = 3922 mm^2

Use 10$\emptyset$30 at top west end with spacing of 150 mm $= \ 7068\ mm^2\ (excess\ reinforcement)$

Use 10$\emptyset$30 at top east end with spacing of 150 mm $= \ 7068\ mm^2\ (excess\ reinforcement)$

Use 10$\emptyset$24 at top middle with spacing of 150 mm $= \ 4524\ mm^2\ (excess\ reinforcement)$

Use 10$\emptyset$28 at bottom west end with spacing of 150 mm = 6157 mm^2 $(excess\ reinforcement)$

Use 10$\emptyset$28 at bottom east end with spacing of 150 mm = 6157 mm^2 $(excess\ reinforcement)$

Use 10$\emptyset$25 at bottom mid with spacing of 150 mm = 4908 mm^2 $(excess\ reinforcement)$

Use additional 4$\emptyset$25 at top and bottom

Development Length (ι_{dt})

West end and east end

$$\iota_{dt} = \frac{0.48 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b < 20 \text{ mm}$$

$$\iota_{dt} = \frac{0.59 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b \geq 20 \text{ mm}$$

$$\iota_{dt} \geq 300 \text{ mm}$$

$$d_b = 28 \text{ mm}$$

$$\iota_{dt} = \frac{0.59 * 420}{\sqrt{70}} * 28 = 829.3 \text{ mm}$$

Since there is excess reinforcement, use modification factor that equals: $\frac{A_{s,required}}{A_{s,provided}}$

Modification factor = $\frac{5972}{6157} = 0.97$

New ι_{dt} = 0.97*829.3 = 804.4 mm

Take ι_{dt} = 805 mm

Development of Hooks

$$\iota_{dh} = \frac{f_y * \varphi_e * \varphi_r * \varphi_o * \varphi_c}{23\lambda * \sqrt{f'_c}} * {d_b}^{1.5} \geq 8d_b \geq 150\,mm$$

$\varphi_e = 1.0$

$\varphi_r = 1.6$

$\varphi_o = 1.25$

$\varphi_c = 1.0$

$\lambda = 1.0$

$$\iota_{dh} = \frac{0.087 f_y}{\lambda * \sqrt{f'_c}} * {d_b}^{1.5}$$

$\iota_{dh} = \frac{0.087*420}{1*\sqrt{70}} * 28^{1.5} = \mathbf{647\,mm}$

$8d_b = 8 * 28 = 224\,mm$

Tale L_{dh} = 650 mm

$\iota_{ext} = 12d_b = 12 * 28 = \mathbf{336\,mm}$

Minimum inside bend diameter = $8d_b = 8 * 28 = \mathbf{224\,mm}$

Lap Splicing

$l_{st} = 1.3 l_{dt} = 1.3 * 805 = 1047\,mm > 300\,mm$, **take it 1050 mm**.

Development Length (L_{dt})

Middle

$$\iota_{dt} = \frac{0.48 * \mathrm{f_y}}{\sqrt{\mathrm{f'_c}}} * \mathrm{d_b} \quad \text{for } \mathrm{d_b} < 20\text{ mm}$$

$$\iota_{dt} = \frac{0.59 * \mathrm{f_y}}{\sqrt{\mathrm{f'_c}}} * \mathrm{d_b} \quad \text{for } \mathrm{d_b} \geq 20\text{ mm}$$

$\iota_{dt} \geq 300\text{ mm}$

$\mathrm{d_b} = 25\text{ mm}$

$\iota_{dt} = \frac{0.59*420}{\sqrt{70}} * 25 = 740.44$ mm.

Since there is excess reinforcement, use modification factor that equals: $\frac{A_{s,required}}{A_{s,provided}}$

Modification factor = $\frac{3922}{4908} = 0.799$

New ι_{dt} = 0.799*740.44 = 591.688 mm

Take ι_{dt} = 595 mm.

Development of Hooks

$$\iota_{dh} = \frac{f_y * \varphi_e * \varphi_r * \varphi_o * \varphi_c}{23\lambda * \sqrt{f'_c}} * {d_b}^{1.5} \geq 8d_b \geq 150\ mm$$

$\varphi_e = 1.0$

$\varphi_r = 1.6$

$\varphi_o = 1.25$

$\varphi_c = 1.0$

$\lambda = 1.0$

$$\iota_{dh} = \frac{0.087f_y}{\lambda * \sqrt{f'_c}} * {d_b}^{1.5}$$

$\iota_{dh} = \frac{0.087*420}{1*\sqrt{70}} * 25^{1.5} = 545.9\ mm$, **take it 550 mm**

$8d_b = 8 * 25 = 200\ \boldsymbol{mm}$.

$\iota_{ext} = 12d_b = 12 * 25 = 300\ \boldsymbol{mm}$.

Minimum inside bend diameter = $8d_b = 8 * 25 = 200\ mm$, **take it 200 mm.**

Lap Splicing

$l_{st} = 1.3l_{dt} = 1.3 * 617 = 802\ mm > 300\ mm$, **take it 800 mm.**

Shear Reinforcement

Shear Force and Reinforcement for Shear, V_{u2}

Shear V_{u2} kN	Shear ϕV_c kN	Shear ϕV_s kN	Shear V_p kN	Rebar A_v /s mm²/m
1876.1413	2210.6317	828.9869	2912.3952	1875

$$\frac{A_v}{s} = 1.875\ mm^2/mm$$

Use six legs stirrups of diameter 8 mm with spacing of 150 mm.

Torsion Reinforcement

Torsion Force and Torsion Reinforcement for Torsion, T_u

T_u kN-m	ϕT_{th} kN-m	ϕT_{cr} kN-m	Area A_o cm²	Perimeter, p_h mm	Rebar A_t /s mm²/m	Rebar A_l mm²
89.437	403.0641	1612.2563	16925.2	5644.4	0	0

No torsion reinforcement needed.

Design Checks for Beam

The positive moment strength at the face of the joint shall be at least one-third the negative moment strength provided at that face of the joint. Neither the negative nor the positive moment strength at any section along the length of the beam shall be less than one-fifth the maximum moment strength provided at the face of either joint.

Moment strength $ØMn = 0.9 * A_s * f_y * \left(d - \frac{A_s * f_y}{1.7f'_c b}\right)$

At east end joint:

Area of steel at bottom = 6157 mm^2

Area of steel at top = 7068 mm^2

Positive ØMn = 2089.577 kN.m

Negative ØMn = 2388 kN.m

$\frac{2388}{3} = 796\ \mathrm{kN.m}$ < Positive ØMn, Acceptable

At west end joint:

Area of steel at bottom = 6157 mm^2

Area of steel at top = 7068 mm^2

Positive ØMn = 2089.577 kN.m

Negative ØMn = 2388 kN.m

$\frac{2388}{3} = 796\ \mathrm{kN.m}$ < Positive ØMn, Acceptable

Maximum moment strength is for A_s = 7068 mm^2

ØMn = 2388 kN.m

$$\frac{2388}{5} = 477.6\ \mathrm{kN.m}$$

Minimum moment strength along the span is for A_s = 4908 mm^2

ØMn = 1675.9 kN.m > 477.6 kN.m, Acceptable

ϕVn shall be at least the lesser of (a) and (b):

a) The sum of the shear associated with development of nominal moment strengths of the beam at each restrained end of the clear span due to reverse curvature bending and the shear calculated for factored gravity and vertical earthquake loads.
b) The maximum shear obtained from design load combinations that include *E*, with *E* taken as twice that prescribed by the general building code.

$ØV_s = 0.75*f_{yt}*d*Av/s$

$Av/s = 2\ mm^2/mm$

$V_s = 0.75*420*925*2 = 582750\ N = 582.75\ kN$

$ØV_n = ØV_c + ØVs = 2210.63 + 582.75 = 2793.38\ kN$

V_u from design load combination = 1876.14 kN

$ØV_n > V_u$, Acceptable

6. Interior Beam at 96th Floor (B5)

Thickness = 1100 mm

Width = 1100 mm

Cover = 75 mm

Design Load Combination:

(1.2 + 0.2Sds) (D + SD) + LL – 2.5EDY + Soil

Longitudinal Reinforcement

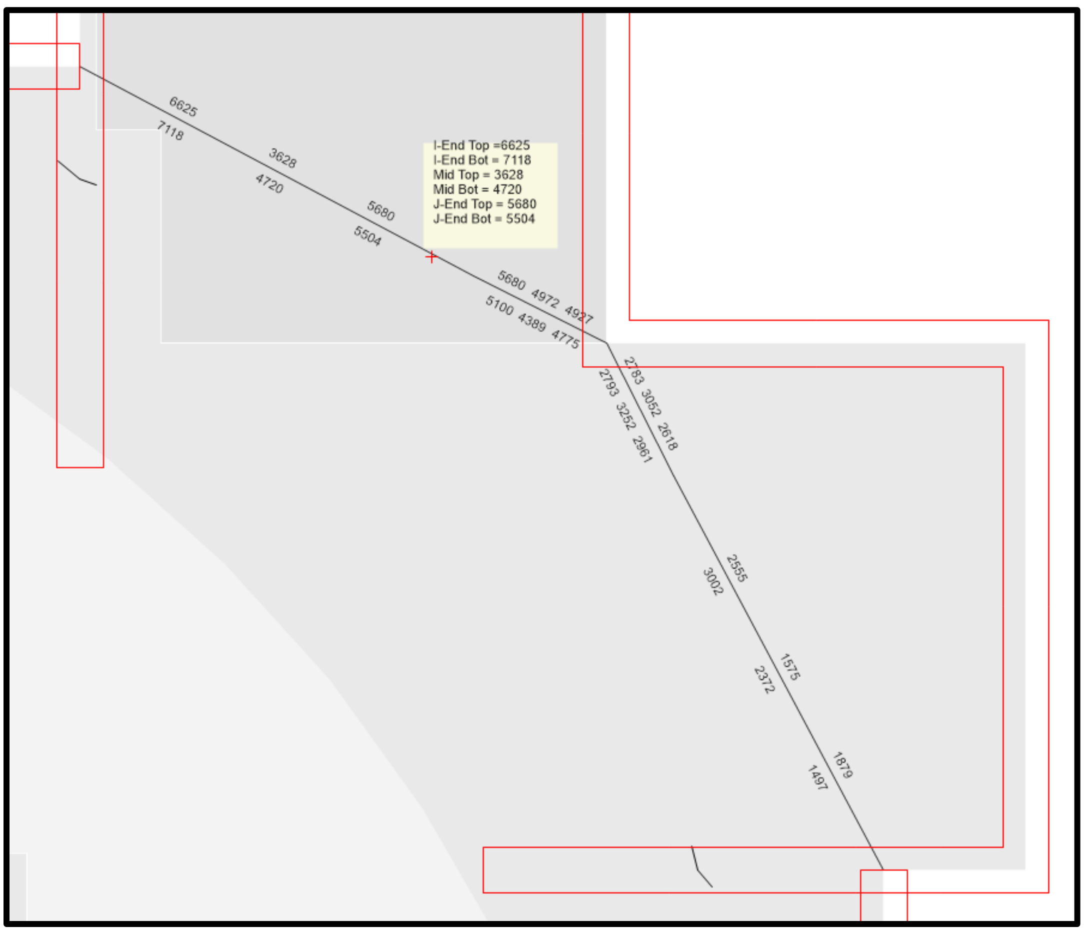


Figure 3.2.6 – Interior Beam Flexural Longitudinal Reinforcement

Top West end Steel = 6625 mm^2

Top East end Steel = 5680 mm^2

Top Middle Steel = 3628 mm^2

Bottom West end Steel = 7118 mm^2

Bottom East end Steel = 5504 mm^2

Bottom Middle Steel = 4720 mm^2

Use 11Ø28 at top west end with spacing of 95 mm $= 6773\ mm^2\ (excess\ reinforcement)$

Use 11Ø28 at top east end with spacing of 95 mm $= 6773\ mm^2\ (excess\ reinforcement)$

Use 6Ø28 at top middle with spacing of 190 mm $= 3694\ mm^2\ (excess\ reinforcement)$

Use 11Ø30 at bottom west end with spacing of 95 mm $=$
$7775\ mm^2\ (excess\ reinforcement)$

Use 11Ø26 at bottom east end with spacing of 95 mm $=$
$5840\ mm^2\ (excess\ reinforcement)$

Use 6Ø32 at bottom mid with spacing of 190 mm $= 4825\ mm^2\ (excess\ reinforcement)$

Use additional 4Ø20 at top and bottom

Development Length (ι_{dt})

West end

$$\iota_{dt} = \frac{0.48 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b < 20 \text{ mm}$$

$$\iota_{dt} = \frac{0.59 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b \geq 20 \text{ mm}$$

$$\iota_{dt} \geq 300 \text{ mm}$$

$$d_b = 30 \text{ mm}$$

$$\iota_{dt} = \frac{0.59 * 420}{\sqrt{70}} * 30 = 888 \text{ mm}$$

Since there is excess reinforcement, use modification factor that equals: $\frac{A_{s,required}}{A_{s,provided}}$

Modification factor = $\frac{7118}{7775} = 0.915$

New ι_{dt} = 0.915*888 = 812.52 mm

Take ι_{dt} = 815 mm

Development of Hooks

$$\iota_{dh} = \frac{f_y * \varphi_e * \varphi_r * \varphi_o * \varphi_c}{23\lambda * \sqrt{f'_c}} * {d_b}^{1.5} \geq 8d_b \geq 150\, mm$$

$\varphi_e = 1.0$

$\varphi_r = 1.6$

$\varphi_o = 1.25$

$\varphi_c = 1.0$

$\lambda = 1.0$

$$\iota_{dh} = \frac{0.087 f_y}{\lambda * \sqrt{f'_c}} * {d_b}^{1.5}$$

$\iota_{dh} = \frac{0.087*420}{1*\sqrt{70}} * 30^{1.5} = \boldsymbol{718\, mm}$

$8d_b = 8 * 30 = \boldsymbol{240\, mm}$

$\iota_{ext} = 12d_b = 12 * 30 = \boldsymbol{360\, mm}$

Minimum inside bend diameter = $8d_b = 8 * 30 = \boldsymbol{240\, mm}$

Lap Splicing

$l_{st} = 1.3 l_{dt} = 1.3 * 815 = 1059.5\, mm > 300\, mm$, **take it 1060 mm.**

East end

$$\iota_{dt} = \frac{0.48 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b < 20 \text{ mm}$$

$$\iota_{dt} = \frac{0.59 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b \geq 20 \text{ mm}$$

$$\iota_{dt} \geq 300 \text{ mm}$$

$$d_b = 26 \text{ mm}$$

$$\iota_{dt} = \frac{0.59 * 420}{\sqrt{70}} * 26 = 770 \text{ mm}$$

Since there is excess reinforcement, use modification factor that equals: $\frac{A_{s,required}}{A_{s,provided}}$

Modification factor = $\frac{5504}{5840} = 0.94$

New ι_{dt} = 0.94*770 = 723.8 mm

Take ι_{dt} = 725 mm

Development of Hooks

$$\iota_{dh} = \frac{f_y * \varphi_e * \varphi_r * \varphi_o * \varphi_c}{23\lambda * \sqrt{f'_c}} * {d_b}^{1.5} \geq 8d_b \geq 150\ mm$$

$\varphi_e = 1.0$

$\varphi_r = 1.6$

$\varphi_o = 1.25$

$\varphi_c = 1.0$

$\lambda = 1.0$

$$\iota_{dh} = \frac{0.087 f_y}{\lambda * \sqrt{f'_c}} * {d_b}^{1.5}$$

$\iota_{dh} = \frac{0.087*420}{1*\sqrt{70}} * 26^{1.5} = \mathbf{579\ mm}$

$8d_b = 8 * 26 = \mathbf{208\ mm}$

$\iota_{ext} = 12d_b = 12 * 26 = \mathbf{312\ mm}$

Minimum inside bend diameter = $8d_b = 8 * 26 = \mathbf{208\ mm}$

Lap Splicing

$l_{st} = 1.3 l_{dt} = 1.3 * 725 = 942.5\ mm > 300\ mm,$ ***take it 940 mm***.

Development Length (L_{dt})

Middle

$$\iota_{dt} = \frac{0.48 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b < 20 \text{ mm}$$

$$\iota_{dt} = \frac{0.59 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b \geq 20 \text{ mm}$$

$\iota_{dt} \geq 300$ mm

$d_b = 24$ mm

$$\iota_{dt} = \frac{0.59 * 420}{\sqrt{70}} * 24 = 711 \text{ mm}$$

Since there is excess reinforcement, use modification factor that equals: $\frac{A_{s,required}}{A_{s,provided}}$

Modification factor = $\frac{4720}{4976} = 0.95$

New ι_{dt} = 0.95*711 = 675.45 mm

Take ι_{dt} = 680 mm

Development of Hooks

$$\iota_{dh} = \frac{f_y * \varphi_e * \varphi_r * \varphi_o * \varphi_c}{23\lambda * \sqrt{f'_c}} * d_b{}^{1.5} \geq 8d_b \geq 150\,mm$$

$\varphi_e = 1.0$

$\varphi_r = 1.6$

$\varphi_o = 1.25$

$\varphi_c = 1.0$

$\lambda = 1.0$

$$\iota_{dh} = \frac{0.087 f_y}{\lambda * \sqrt{f'_c}} * d_b{}^{1.5}$$

$\iota_{dh} = \frac{0.087*420}{1*\sqrt{70}} * 24^{1.5} = \mathbf{513\ mm}$

$8d_b = 8 * 24 = \mathbf{192\ mm}$

$\iota_{ext} = 12d_b = 12 * 24 = \mathbf{288\ mm}$

Minimum inside bend diameter = $8d_b = 8 * 24 = \mathbf{192\ mm}$

Lap Splicing

$l_{st} = 1.3 l_{dt} = 1.3 * 680 = 884\,mm > 300\,mm$, **take it 885 mm.**

Shear Reinforcement

Shear Force and Reinforcement for Shear, V_{u2}

Shear V_{u2} kN	Shear ϕV_c kN	Shear ϕV_s kN	Shear V_p kN	Rebar A_v/s mm²/m
962.4936	1166.0759	437.2785	704.134	1375

$$\frac{A_v}{s} = \mathbf{1.375\ mm^2/mm}$$

Use six legs stirrups of diameter 8 mm with spacing of 150 mm

Torsion Reinforcement

Torsion Force and Torsion Reinforcement for Torsion, T_u						
T_u kN-m	ϕT_{th} kN-m	ϕT_{cr} kN-m	Area A_o cm^2	Perimeter, p_h mm	Rebar A_t /s mm^2/m	Rebar A_l mm^2
62.9205	204.413	817.6522	8689.7	4044.4	0	0

No torsion reinforcement needed.

Design Checks for Beam

The positive moment strength at the face of the joint shall be at least one-third the negative moment strength provided at that face of the joint. Neither the negative nor the positive moment strength at any section along the length of the beam shall be less than one-fifth the maximum moment strength provided at the face of either joint.

Moment strength $\emptyset Mn = 0.9 * A_s * f_y * \left(d - \frac{A_s * f_y}{1.7f'_c b}\right)$

At east end joint:

Area of steel at bottom = 5840 mm^2

Area of steel at top = 6773 mm^2

Positive ØMn = 2221.34 kN.m

Negative ØMn = 2568.56 kN.m

$\frac{2568.56}{3} = 856.18\ \mathrm{kN.m} <$ Positive ØMn. Acceptable.

At west end joint:

Area of steel at bottom = 6773 mm^2

Area of steel at top = 7775 mm^2

Positive ØMn = 2939 kN.m

Negative ØMn = 2568.56 kN.m

$\frac{2568.56}{3} = 856.18\ \mathrm{kN.m} <$ Positive ØMn. Acceptable.

Maximum moment strength is for $A_s = 7775\ mm^2$

$ØMn = 2388$ kN.m

$$\frac{7775}{5} = 587.8\ \mathrm{kN.m}$$

Minimum moment strength along the span is for $A_s = 3694\ mm^2$

$ØMn = 1414.69$ kN.m > 587.8 kN.m   Acceptable.

ϕVn shall be at least the lesser of (a) and (b):

a) The sum of the shear associated with development of nominal moment strengths of the beam at each restrained end of the clear span due to reverse curvature bending and the shear calculated for factored gravity and vertical earthquake loads.
b) The maximum shear obtained from design load combinations that include *E*, with *E* taken as twice that prescribed by the general building code.

$ØV_s = 0.75*f_{yt}*d*Av/s$

$Av/s = 2\ mm^2/mm$

$V_s = 0.75*420*1025*2 = 645750$ N = 645.75 kN

$ØV_n = ØV_c + ØVs = 1166 + 645.75 = 1811.75$ kN

V_u from design load combination = 962.5 kN

$ØV_n > V_u$ Acceptable.

7. **Edge Beam at 79th Floor to 100th Floor (B6)**

Thickness = 900 mm

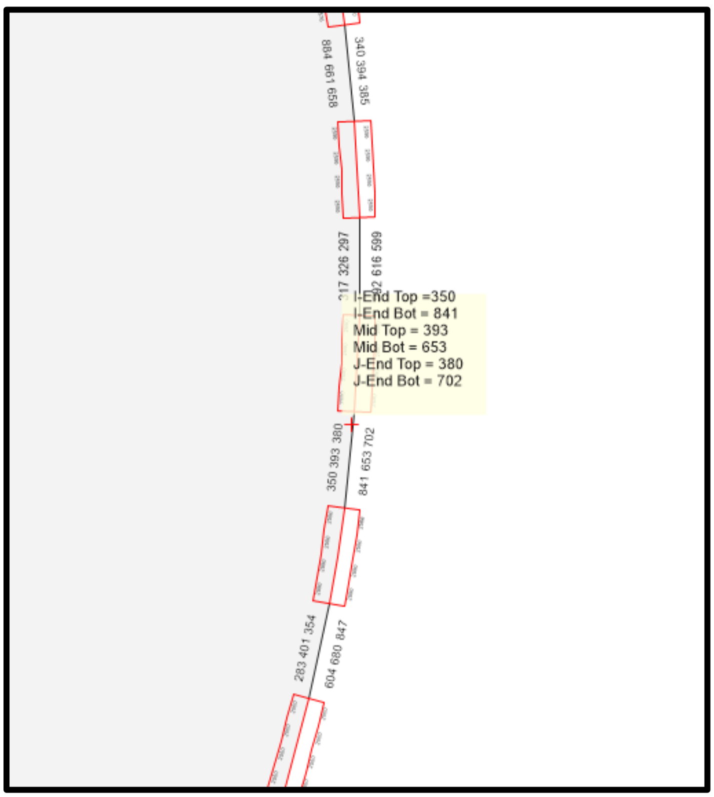
Width = 750 mm

Cover = 75 mm

Design Load Combination:

(1.2 + 0.2Sds) (D + SD) + LL – 2.5EDX + Soil

Longitudinal Reinforcement



Top West end Steel = 350 mm^2

Top East end Steel = 380 mm^2

Top Middle Steel = 393 mm^2

Bottom West end Steel = 841 mm^2

Bottom East end Steel = 702 mm^2

Bottom Middle Steel = 653 mm^2

Use 5Ø10 at top west end with spacing of 150 mm $= 393\ mm^2\ (excess\ reinforcement)$

Use 5Ø10 at top east end with spacing of 150 mm $= 393\ mm^2\ (excess\ reinforcement)$

Use 6Ø10 at top middle with spacing of 150 mm $= 471\ mm^2\ (excess\ reinforcement)$

Use 6Ø14 at bottom west end with spacing of 120 mm =
$924\ mm^2\ (excess\ reinforcement)$

Use 7Ø12 at bottom east end with spacing of 100 mm $= 792\ mm^2\ (excess\ reinforcement)$

Use 6Ø12 at bottom middle with spacing of 100 mm $= 679\ mm^2\ (excess\ reinforcement)$

Use additional 4Ø8 at top and bottom

Development Length (ι_{dt})

West end

$$\iota_{dt} = \frac{0.48 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b < 20 \text{ mm}$$

$$\iota_{dt} = \frac{0.59 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b \geq 20 \text{ mm}$$

$$\iota_{dt} \geq 300 \text{ mm}$$

$$d_b = 14 \text{ mm}$$

$$\iota_{dt} = \frac{0.48 * 420}{\sqrt{70}} * 14 = 337.34 \text{ mm}$$

ι_{dt} = 338 mm

Since there is excess reinforcement, use modification factor that equals: $\frac{A_{s,required}}{A_{s,provided}}$

Modification factor = $\frac{841}{924} = 0.91$

New ι_{dt} = 0.91*338 = 308 mm

Development of Hooks

$$\iota_{\text{dh}} = \frac{f_y * \varphi_e * \varphi_r * \varphi_o * \varphi_c}{23\lambda * \sqrt{f'_c}} * {d_b}^{1.5} \geq 8d_b \geq 150\ mm$$

$\varphi_e = 1.0$

$\varphi_r = 1.6$

$\varphi_o = 1.25$

$\varphi_c = 1.0$

$\lambda = 1.0$

$$\iota_{\text{dh}} = \frac{0.087 f_y}{\lambda * \sqrt{f'_c}} * {d_b}^{1.5}$$

$\iota_{\text{dh}} = \frac{0.087*420}{1*\sqrt{70}} * 14^{1.5} = 229\ mm$

$8d_b = 8 * 14 = 112\ mm$

ι_{dh} = 229 mm

$\iota_{\text{ext}} = 12d_b = 12 * 14 = \mathbf{168\ mm}$

Minimum inside bend diameter = $6d_b = 6 * 14 = \mathbf{84\ mm}$

Lap Splicing

$l_{st} = 1.3 l_{dt} = 1.3 * 338 = \mathbf{440\ mm} > 300\ mm$

Shear Reinforcement

Shear Force and Reinforcement for Shear, V_{u2}				
Shear V_{u2} kN	Shear ϕV_c kN	Shear ϕV_s kN	Shear V_p kN	Rebar A_v /s mm²/m
224.738	194.5629	0	458.5027	0

Use minimum shear reinforcement.

$Av_{min}/s = 0.001235 b_w = 0.001235 * 750 = 0.92625\ mm^2/mm$.

Use four legs stirrups of diameter 8 mm with spacing of 150 mm

Torsion Reinforcement

Torsion Force and Torsion Reinforcement for Torsion, T_u						
T_u kN-m	ϕT_{th} kN-m	ϕT_{cr} kN-m	Area A_o cm^2	Perimeter, p_h mm	Rebar A_t /s mm^2/m	Rebar A_l mm^2
12.144	73.0993	292.3973	4557.9	2944.4	0	0

No torsion Reinforcement needed.

Design Checks for Beam

The positive moment strength at the face of the joint shall be at least one-third the negative moment strength provided at that face of the joint. Neither the negative nor the positive moment strength at any section along the length of the beam shall be less than one-fifth the maximum moment strength provided at the face of either joint.

Moment strength $\emptyset\mathrm{Mn} = 0.9 * \mathrm{A_s} * \mathrm{f_y} * \left(\mathrm{d} - \frac{\mathrm{A_s} * \mathrm{f_y}}{1.7\mathrm{f'_c b}}\right)$

At east end joint:

Area of steel at bottom = 792 mm^2

Area of steel at top = 393 mm^2

Positive ØMn = 245.86 kN.m

Negative ØMn = 122.28 kN.m

$\frac{122.28}{3} = 40.76\ \mathrm{kN.m}$ < Positive ØMn, Acceptable.

At west end joint:

Area of steel at bottom = 924 mm^2

Area of steel at top = 393 mm^2

Positive ØMn = 286.63 kN.m

Negative ØMn = 122.28 kN.m

$\frac{122.28}{3} = 40.76\ \mathrm{kN.m}$ < Positive ØMn, Acceptable.

Maximum moment strength is for $A_s = 924\ mm^2$

$ØMn = 2388$ kN.m

$$\frac{286.63}{5} = 57.3 \text{ kN. m}$$

Minimum moment strength along the span is for $A_s = 393\ mm^2$

$ØMn = 122.28$ kN.m > 57.3 kN.m   Acceptable.

ϕVn shall be at least the lesser of (a) and (b):

a) The sum of the shear associated with development of nominal moment strengths of the beam at each restrained end of the clear span due to reverse curvature bending and the shear calculated for factored gravity and vertical earthquake loads.
b) The maximum shear obtained from design load combinations that include *E*, with *E* taken as twice that prescribed by the general building code.

$ØV_s = 0.75*f_{yt}*d*Av/s$

$Av/s = 1.34\ mm^2/mm$

$V_s = 0.75*420*825*1.34 = 348232$ N = 348.232 kN

$ØV_n = ØV_c + ØVs = 149.563 + 348.232 = 497.795$ kN

V_u from design load combination = 224.738 kN

$ØV_n > V_u$, Acceptable.

3.3 Design of Vertical Columns

1. Columns at 15th Floor to 79th Floor (C1B)

Section 1500 mm*1500 mm

Cover: 75 mm

Longitudinal reinforcement

Column Element Details (Part 1 of 2)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)
26	C37	2853	Column 1	(1.2+0.2*Sds)(D+SD)+LL-2.5EDY + Soil	0	3650

Column Element Details (Part 2 of 2)

LLRF	Type
0.4	Sway Intermediate

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
1500	1500	101	62.3

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
39323	70	1	413.69	413.69

Design Code Parameters

ϕ_T	ϕ_{CTied}	$\phi_{CSpiral}$	ϕ_{Vns}	ϕ_{Vs}	ϕ_{Vjoint}	Ω_0
0.9	0.65	0.75	0.75	0.6	0.85	2.5

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 M2 kN-m	Minimum M3 kN-m	Rebar Area mm²	Rebar % %
62304.9251	-4204.8817	2527.0479	3753.2487	3753.2487	22500	1

Axial Force and Biaxial Moment Factors

	C_m Factor Unitless	δ_{ns} Factor Unitless	δ_s Factor Unitless	K Factor Unitless	Length mm
Major Bend(M3)	0.487076	1	1	1	2650
Minor Bend(M2)	0.453027	1	1	1	2650

Rebar area = **22500 mm²**

Use $\mathbf{20\emptyset 40}\ \boldsymbol{longitidual\ reinforcement}$, $As = 25132\ mm^2\ with\ spacing\ of\ 270\ mm.$

Development Length

$$\iota_{dt} = \frac{0.48 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b < 20 \text{ mm}$$

$$\iota_{dt} = \frac{0.59 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b \geq 20 \text{ mm}$$

$$\iota_{dt} \geq 300 \text{ mm}$$

$$d_b = 40 \text{ mm}$$

$$\iota_{dt} = \frac{0.59 * 420}{\sqrt{70}} * 40 = 1184.7 \text{ mm}$$

Since there is excess reinforcement, use modification factor that equals: $\frac{A_{s,required}}{A_{s,provided}}$

$\frac{22500}{25132} = 0.895$, $\iota_{dt} = 0.895 * 1184.7 = 1060.6\,mm$, $take\ it\ equal$ $\mathbf{1065\ mm}$.

Lap splicing:

$l_{st} = 1.3 l_{dt} = 1.3 * 1065\,mm = \mathbf{1380\ mm} > 300\,mm$

Confinement spacing (So) and confinement length (lo) are to be determined.

So shall not exceed the smallest of:

- The smaller of $8d_b$ of the smallest longitudinal bar enclosed and 200 mm.
- One-half the smallest dimension of the column's cross section.

$8d_b = 8 * 40 = 320$ mm

0.5*1500 = 750 mm

Thus, take Confinement spacing (So) = **200 mm**

Lo shall not be less than the largest of:

- One-sixth the clear span of the column
- Maximum cross-sectional dimension of the column
- 450 mm

Lo = 1500 mm

Shear Reinforcement

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

	Shear V_u kN	Shear ϕV_c kN	Shear ϕV_s kN	Shear ϕV_p kN	Rebar A_v /s mm²/m
Major, V_{u2}	1218.9379	4295.1146	0	1091.1781	0
Minor, V_{u3}	1185.5602	4295.1146	0	0	0

Joint Shear Check/Design

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

(6/5) Beam/Column Capacity Ratio

Major Ratio	Minor Ratio
N/N	N/N

Notes:

N/A: Not Applicable

N/C: Not Calculated

N/N: Not Needed

Minimum shear reinforcement

Table 9.6.3.4—Required $A_{v,min}$			
Beam type	$A_{v,min}/s$		
Nonprestressed and prestressed with $A_{ps}f_{se} < 0.4(A_{ps}f_{pu} + A_s f_y)$	Greater of:	$0.062\sqrt{f'_c}\dfrac{b_w}{f_{yt}}$	(a)
		$0.35\dfrac{b_w}{f_{yt}}$	(b)

Table 3.3.1 – Requirements for Minimum Shear Reinforcement

f'_c = 70 MPa

f_{yt} = 420 MPa

Take the greater of:

- $0.001235b_w$
- $0.0008333b_w$

Take $0.001235b_w$ for all cases

$A_v/s_{min} = 0.001235*1500 =$ **1.8525 mm²/mm**

Use **four legs stirrups with diameter 16 mm with spacing of 400 mm.**

Design Checks for Column

ϕVn shall be at least the lesser of (a) and (b):

a) The shear associated with development of nominal moment strengths of the column at each restrained end of the unsupported length due to reverse curvature bending. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the highest flexural strength.
b) The maximum shear obtained from factored load combinations that include *E*, with $\Omega_0 E$ substituted for *E*.

$ØV_s = 0.75*f_{yt}*d*Av/s$

$Av/s = 2\ mm^2/mm$

$V_s = 0.75*420*1425*2 = 897750\ N = 897.75\ kN$

$ØV_n = ØV_c + ØVs = 4295.1146 + 897.75 = 5192.86\ kN$

V_u from design load combination = 1218.94 kN

$ØV_n > V_{u,}$ Acceptable.

2. Column at 11th Floor to 79th Floor (C2C)

Section 1600 mm * 1600 mm

Cover: 75 mm

Longitudinal reinforcement

Column Element Details (Part 1 of 2)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)
68	C44	3578	Column 2	(1.2+0.2*Sds)(D+SD)+LL-2.5EDX + Soil	0	3650

Column Element Details (Part 2 of 2)

LLRF	Type
0.4	Sway Intermediate

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
1600	1600	101	62.3

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
39323	70	1	413.69	413.69

Design Code Parameters

ϕ_T	ϕ_{CTied}	$\phi_{CSpiral}$	ϕ_{Vns}	ϕ_{Vs}	ϕ_{Vjoint}	Ω_0
0.9	0.65	0.75	0.75	0.6	0.85	2.5

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 M2 kN-m	Minimum M3 kN-m	Rebar Area mm²	Rebar % %
31326.1567	1501.3126	1981.0662	1981.0662	1981.0662	25600	1

Axial Force and Biaxial Moment Factors

	C_m Factor Unitless	δ_{ns} Factor Unitless	δ_s Factor Unitless	K Factor Unitless	Length mm
Major Bend(M3)	0.473089	1	1	1	2650
Minor Bend(M2)	0.41668	1	1	1	2650

Rebar area = $\mathbf{25600\ mm2}$

Use $\mathbf{40\emptyset 30\ longitidual\ reinforcement}$, $As = 28273\ mm^2\ with\ spacing\ of\ 145\ mm.$

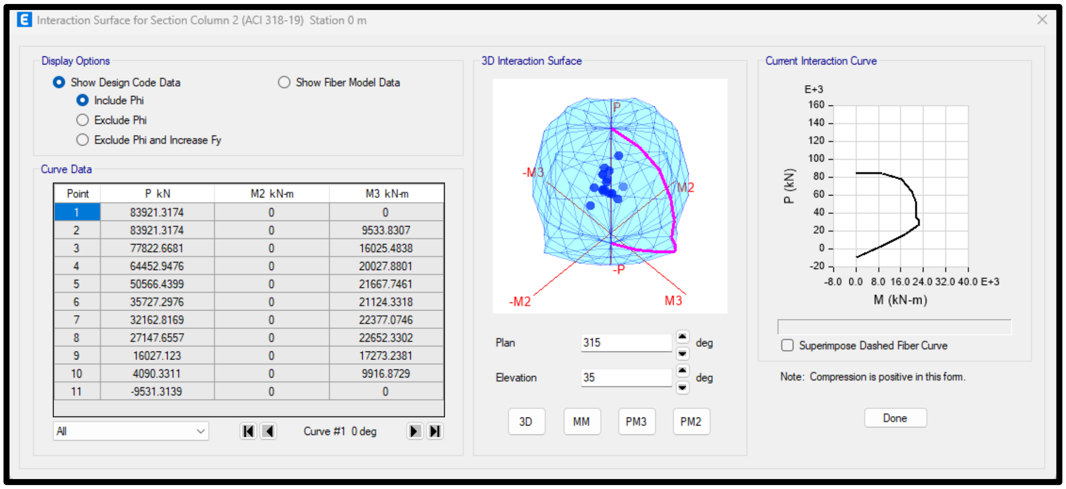


Figure 3.3.1 – Column C2C P-M3 Interaction Diagram

Development Length

$$\iota_{dt} = \frac{0.48 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b < 20 \text{ mm}$$

$$\iota_{dt} = \frac{0.59 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b \geq 20 \text{ mm}$$

$$\iota_{dt} \geq 300 \text{ mm}$$

$$d_b = 30 \text{ mm}$$

$$\iota_{dt} = \frac{0.59 * 420}{\sqrt{70}} * 30 = 888.53 \text{ mm}$$

Since there is excess reinforcement, use modification factor that equals: $\frac{A_{s,required}}{A_{s,provided}}$

$\frac{25600}{28273} = 0.905$, $\iota_{dt} = 0.905 * 888.53 = 804.5\ mm$, ***take it equal* $\boldsymbol{805\ mm}$** .

Lap splicing

$l_{st} = 1.3 l_{dt} = 1.3 * 805\ mm = \mathbf{1045\ mm > 300\ mm}.$

Confinement spacing (So) and confinement length (Lo) are to be determined.

So shall not exceed the smallest of:

- The smaller of $8d_b$ of the smallest longitudinal bar enclosed and 200 mm.
- One-half the smallest dimension of the column's cross section.

8db = 8 * 40 = 320 mm

0.5*1600 = 800 mm

Take Confinement spacing (So) = 200 mm.

Lo shall not be less than the largest of:

- One-sixth the clear span of the column
- Maximum cross-sectional dimension of the column
- 450 mm

Lo = 1600 mm

Shear Reinforcement

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

	Shear V_u kN	Shear ϕV_c kN	Shear ϕV_s kN	Shear ϕV_p kN	Rebar A_v/s mm²/m
Major, V_{u2}	177.3379	2498.5422	0	0	0
Minor, V_{u3}	1270.409	4045.611	930.1714	1268.032	2000

Joint Shear Check/Design

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

(6/5) Beam/Column Capacity Ratio

Major Ratio	Minor Ratio
N/N	N/N

Notes:

N/A: Not Applicable

N/C: Not Calculated

N/N: Not Needed

Av/s = 2 mm²/mm

Use nine legs stirrups with diameter 12 mm with spacing of 400 mm.

Design Checks for Column

ϕVn shall be at least the lesser of (a) and (b):

a) The shear associated with development of nominal moment strengths of the column at each restrained end of the unsupported length due to reverse curvature bending. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the highest flexural strength.
b) The maximum shear obtained from factored load combinations that include *E*, with $\Omega_0 E$ substituted for *E*.

$ØV_s = 0.75*f_{yt}*d*Av/s$

$Av/s = 2.54\ mm^2/mm$

$V_s = 0.75*420*1525*2.54 = 1220152\ N = 1220.152\ kN$

$ØV_n = ØV_c + ØVs = 4045.611 + 1220.152 = 5265.763\ kN$

V_u from design load combination = 1270.409 kN

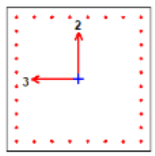
$ØV_n > V_{u,}$ Acceptable.

3. Column at 6th Basement Floor to 14th Floor (C1A)

Parking Columns

Section 1500 mm * 1500 mm

Cover: 75 mm



Column Element Details (Part 1 of 2)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)
-5	C35	5823	Column 1	(1.2+0.2*Sds)(D+SD)+LL-2.5EDY + Soil	0	3050

Column Element Details (Part 2 of 2)

LLRF	Type
0.4	Sway Intermediate

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
1500	1500	101	62.3

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
39323	70	1	413.69	413.69

Design Code Parameters

Φ_T	Φ_{CTied}	$\Phi_{CSpiral}$	Φ_{Vns}	Φ_{Vs}	Φ_{Vjoint}	Ω_0
0.9	0.65	0.75	0.75	0.6	0.85	2.5

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 M2 kN-m	Minimum M3 kN-m	Rebar Area mm^2	Rebar % %
83980.248	-295.172	5058.9701	5058.9701	5058.9701	78481	3.49

Axial Force and Biaxial Moment Factors

	C_m Factor Unitless	δ_{ns} Factor Unitless	δ_s Factor Unitless	K Factor Unitless	Length mm
Major Bend(M3)	0.381958	1	1	1	2050
Minor Bend(M2)	0.284017	1	1	1	2050

Rebar area = **78481 mm^2**

Use $\boldsymbol{40\emptyset 50\ longitidual\ reinforcement}$, $As = 78537\ mm^2\ with\ spacing\ of\ 135\ mm.$

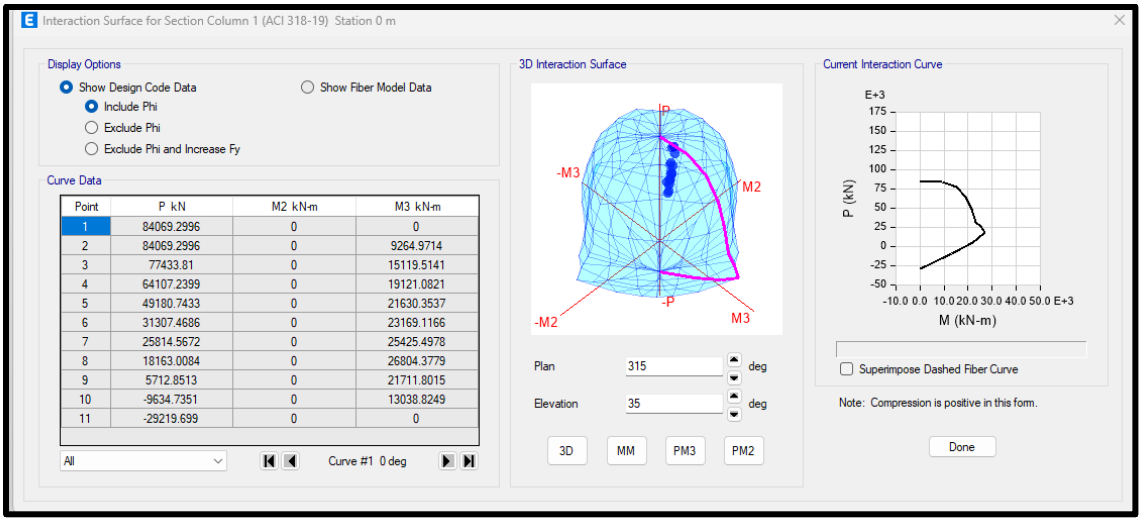


Figure 3.3.2 – Column C1A P-M3 Interaction Diagram

Development Length

$$l_{dt} = \frac{0.48 * f_y}{\sqrt{f'_c}} * d_b \qquad \text{for } d_b < 20 \text{ mm}$$

$$l_{dt} = \frac{0.59 * f_y}{\sqrt{f'_c}} * d_b \qquad \text{for } d_b \geq 20 \text{ mm}$$

$$l_{dt} \geq 300 \text{ mm}$$

$$d_b = 50 \text{ mm}$$

$$l_{dt} = \frac{0.59 * 420}{\sqrt{70}} * 50 = 1480.888 \text{ mm}$$

Since there is excess reinforcement, use modification factor that equals: $\frac{A_{s,required}}{A_{s,provided}}$

$\frac{78481}{78537} = 0.999$, $l_{dt} = 0.999 * 1480.888 = \mathbf{1479.83\ mm}$, ***take it equal* 1480 *mm*** .

Lap splicing

$$l_{st} = 1.3 l_{dt} = 1.3 * 1480\ mm = \mathbf{1920\ mm} > 300\ mm$$

Confinement spacing (So) and Confinement length (lo) are to be determined.

So shall not exceed the smallest of:

The smaller of $8d_b$ of the smallest longitudinal bar enclosed and 200 mm.

One-half the smallest dimension of the column's cross section.

$8d_b$ = 8 * 40 = 320 mm

0.5*1500 = 750 mm

Confinement spacing (So) = 200 mm

Lo shall not be less than the largest of:

- One-sixth the clear span of the column
- Maximum cross-sectional dimension of the column
- 450 mm

Lo = 1500 m

Shear Reinforcement

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

	Shear V_u kN	Shear ϕV_c kN	Shear ϕV_s kN	Shear ϕV_p kN	Rebar A_v /s mm²/m
Major, V_{u2}	362.0842	5425.7409	0	230.4665	0
Minor, V_{u3}	348.7023	5425.7409	0	0	0

Joint Shear Check/Design

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

(6/5) Beam/Column Capacity Ratio

Major Ratio	Minor Ratio
N/N	N/N

Notes:

N/A: Not Applicable

N/C: Not Calculated

N/N: Not Needed

Minimum Shear Reinforcement

Table 9.6.3.4—Required $A_{v,min}$

Beam type	$A_{v,min}/s$		
Nonprestressed and prestressed with $A_{ps}f_{se} < 0.4(A_{ps}f_{pu} + A_s f_y)$	Greater of:	$0.062\sqrt{f'_c}\dfrac{b_w}{f_{yt}}$	(a)
		$0.35\dfrac{b_w}{f_{yt}}$	(b)

f'_c = 70 MPa

f_{yt} = 420 MPa

Take the greater of:

- $0.001235b_w$
- $0.0008333b_w$

Take $0.001235b_w$ for all cases

A_v/s_{min} = 0.001235*1500 = 1.8525 mm^2/mm.

Use nine legs stirrups with diameter 12 mm with spacing of 400 mm.

Design Checks for Column

ϕVn shall be at least the lesser of (a) and (b):

a) The shear associated with development of nominal moment strengths of the column at each restrained end of the unsupported length due to reverse curvature bending. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the highest flexural strength.
b) The maximum shear obtained from factored load combinations that include *E*, with $\Omega_0 E$ substituted for *E*.

$ØV_s = 0.75*f_{yt}*d*Av/s$

$Av/s = 2.54\ mm^2/mm$

$V_s = 0.75*420*1425*2.54 = 1140142\ N = 1140.142\ kN$

$ØV_n = ØV_c + ØVs = 5425.7409 + 1140.142 = 6562.883\ kN$

V_u from design load combination = 362.0842 kN

$ØV_n > V_u$ Acceptable.

4. Column at 6th Basement Floor to 1st Parking floor (C2A)

Section 1600 mm * 1600 mm

Cover: 75 mm

Column Element Details (Part 1 of 2)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)
-5	C43	5819	Column 2	(1.2+0.2*Sds)(D+SD)+LL-2.5EDX + Soil	0	3050

Column Element Details (Part 2 of 2)

LLRF	Type
0.4	Sway Intermediate

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
1600	1600	101	62.3

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
39323	70	1	413.69	413.69

Design Code Parameters

Φ_T	Φ_{CTied}	$\Phi_{CSpiral}$	Φ_{Vns}	Φ_{Vs}	Φ_{Vjoint}	Ω_0
0.9	0.65	0.75	0.75	0.6	0.85	2.5

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 M2 kN-m	Minimum M3 kN-m	Rebar Area mm^2	Rebar % %
92871.5613	-5873.1975	1359.7779	5873.1975	5873.1975	74337	2.9

Axial Force and Biaxial Moment Factors

	C_m Factor Unitless	δ_{ns} Factor Unitless	δ_s Factor Unitless	K Factor Unitless	Length mm
Major Bend(M3)	0.392547	1	1	1	2050
Minor Bend(M2)	0.654981	1	1	1	2050

Rebar area = $\mathbf{74337\ mm^2}$

Use $\mathbf{40\emptyset 50\ longitidual\ reinforcement}$ $, As = 78537\ mm^2\ with\ spacing\ of\ 135\ mm.$

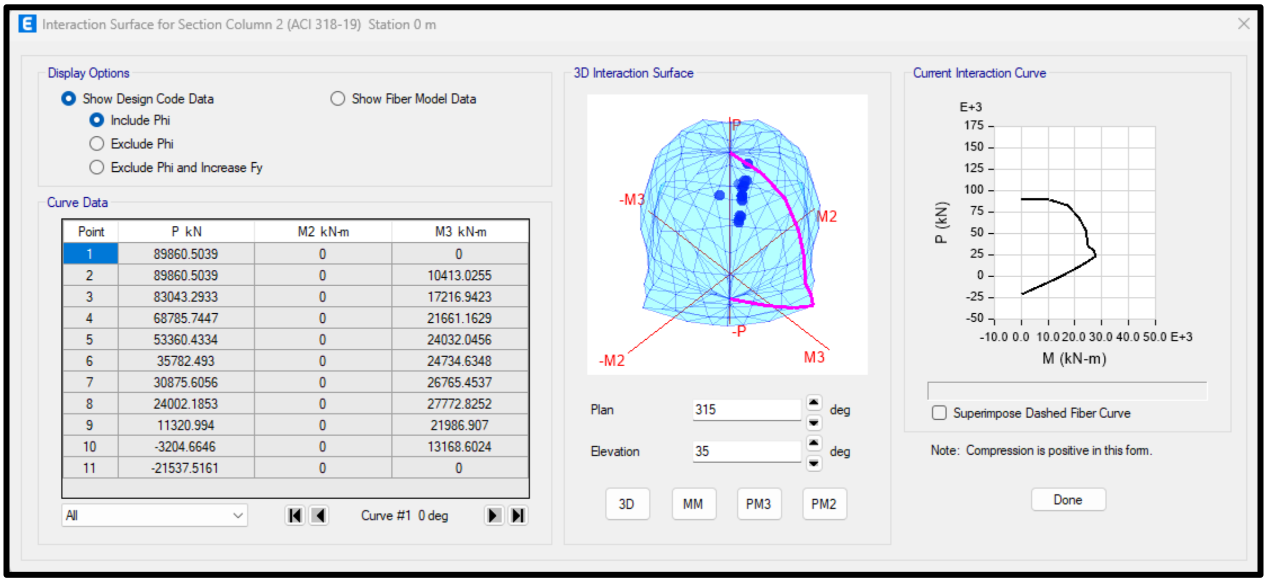


Figure 3.3.3 – Column C2A P-M3 Interaction Diagram

Development Length

$$\iota_{dt} = \frac{0.48 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b < 20\text{ mm}$$

$$\iota_{dt} = \frac{0.59 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b \geq 20\text{ mm}$$

$$\iota_{dt} \geq 300\text{ mm}$$

$$d_b = 50\text{ mm}$$

$$\iota_{dt} = \frac{0.59 * 420}{\sqrt{70}} * 50 = 1480.888\ \text{ mm}$$

Since there is excess reinforcement, use modification factor that equals: $\frac{A_{s,required}}{A_{s,provided}}$

$\frac{74337}{78537} = 0.94$, $\iota_{dt} = 0.94 * 1480.888 = \mathbf{1401.69}\ \boldsymbol{mm}$, $\boldsymbol{take\ it\ equal}\ \mathbf{1405}\ \boldsymbol{mm}$.

Lap splicing

$l_{st} = 1.3 l_{dt} = 1.3 * 1405\ mm = \mathbf{1825}\ \boldsymbol{mm} > \mathbf{300}\ \boldsymbol{mm}$.

Confinement spacing (So) and Confinement Length (lo) are to be determined.

So shall not exceed the smallest of:

- The smaller of $8d_b$ of the smallest longitudinal bar enclosed and 200 mm.
- One-half the smallest dimension of the column's cross section.

$8d_b$ = 8 * 40 = 320 mm

0.5*1600 = 800 mm

Confinement Spacing (So) = 200 mm.

Lo shall not be less than the largest of:

- One-sixth the clear span of the column
- Maximum cross-sectional dimension of the column
- 450 mm

Lo = 1600 mm

Shear Reinforcement

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

	Shear V_u kN	Shear ϕV_c kN	Shear ϕV_s kN	Shear ϕV_p kN	Rebar A_v /s mm²/m
Major, V_{u2}	486.7445	6201.1424	0	0	0
Minor, V_{u3}	366.4837	6201.1424	0	1143.7472	0

Joint Shear Check/Design

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

(6/5) Beam/Column Capacity Ratio

Major Ratio	Minor Ratio
N/N	N/N

Notes:

N/A: Not Applicable

N/C: Not Calculated

N/N: Not Needed

Minimum Shear Reinforcement

Table 9.6.3.4—Required $A_{v,min}$

Beam type	$A_{v,min}/s$		
Nonprestressed and prestressed with $A_{ps}f_{se} < 0.4(A_{ps}f_{pu} + A_s f_y)$	Greater of:	$0.062\sqrt{f'_c}\dfrac{b_w}{f_{yt}}$	(a)
		$0.35\dfrac{b_w}{f_{yt}}$	(b)

$f'_c = 70$ MPa

$f_{yt} = 420$ MPa

Take the greater of:

- $0.001235b_w$
- $0.0008333b_w$

Take $0.001235b_w$ for all cases

$A_v/s_{min} = 0.001235*1600 =$ **1.976 mm²/mm**

Use nine legs stirrups with diameter 12 mm with spacing of 400 mm.

Design Checks for Columns

ϕVn shall be at least the lesser of (a) and (b):

a) The shear associated with development of nominal moment strengths of the column at each restrained end of the unsupported length due to reverse curvature bending. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the highest flexural strength.
b) The maximum shear obtained from factored load combinations that include *E*, with $\Omega_0 E$ substituted for *E*.

$ØV_s = 0.75*f_{yt}*d*Av/s$

$Av/s = 2.54\ mm^2/mm$

$V_s = 0.75*420*1525*2.54 = 1220152\ N = 1220.152\ kN$

$ØV_n = ØV_c + ØV_s = 6201.14 + 1220.152 = 7421.3\ kN$

V_u from design load combination = 486.74 kN

$ØV_n > V_u$, Acceptable

5. Column at 1st Parking Floor to 10th Parking Floor (C2B)

Section: 1600 mm * 1600 mm

Cover: 75 mm

Column Element Details (Part 1 of 2)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)
3	C43	5371	Column 2	(1.2+0.2*Sds)(D+SD)+LL-2.5EDX + Soil	0	2800

Column Element Details (Part 2 of 2)

LLRF	Type
0.4	Sway Intermediate

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
1600	1600	101	62.3

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
39323	70	1	413.69	413.69

Design Code Parameters

ϕ_T	ϕ_{CTied}	$\phi_{CSpiral}$	ϕ_{Vns}	ϕ_{Vs}	ϕ_{Vjoint}	Ω_0
0.9	0.65	0.75	0.75	0.6	0.85	2.5

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 M2 kN-m	Minimum M3 kN-m	Rebar Area mm^2	Rebar % %
88379.502	-1338.0725	5589.1197	5589.1197	5589.1197	49968	1.95

Axial Force and Biaxial Moment Factors

	C_m Factor Unitless	δ_{ns} Factor Unitless	δ_s Factor Unitless	K Factor Unitless	Length mm
Major Bend(M3)	0.345022	1	1	1	1800
Minor Bend(M2)	0.5097	1	1	1	1800

Rebar area = **49958 mm²**

Use $\mathbf{40\emptyset40}$ $\boldsymbol{longitidual\ reinforcement}$ $, As = 50264\ mm^2\ with\ spacing\ of\ 135\ mm.$

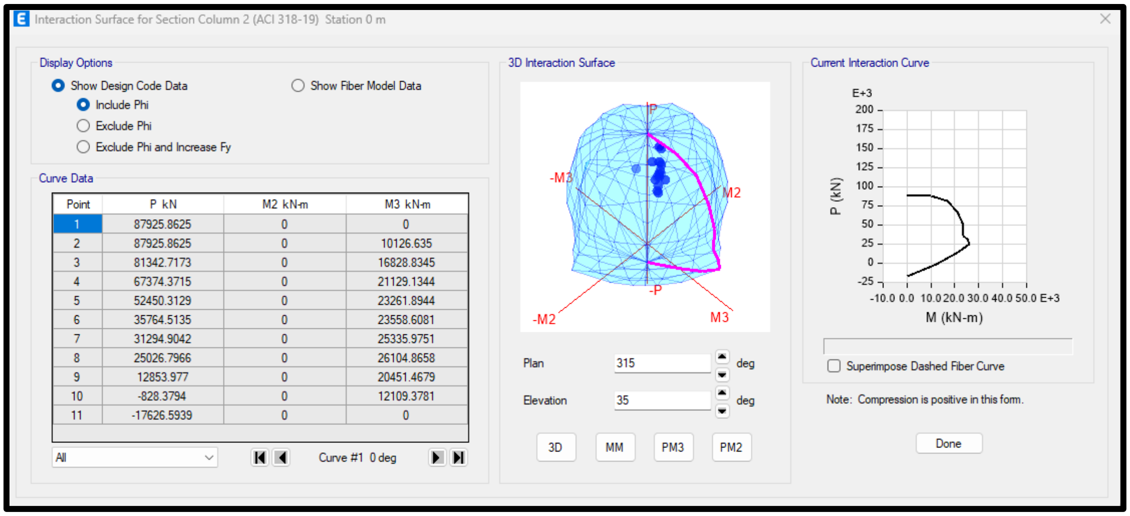


Figure 3.3.4 – Column C2B P-M3 Interaction Diagram

Development Length

$$\iota_{dt} = \frac{0.48 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b < 20 \text{ mm}$$

$$\iota_{dt} = \frac{0.59 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b \geq 20 \text{ mm}$$

$$\iota_{dt} \geq 300 \text{ mm}$$

$$d_b = 40 \text{ mm}$$

$$\iota_{dt} = \frac{0.59 * 420}{\sqrt{70}} * 40 = 1184.7 \text{ mm}$$

Since there is excess reinforcement, use modification factor that equals: $\frac{A_{s,required}}{A_{s,provided}}$

$$\frac{49958}{50264} = 0.994 \text{ , } \iota_{dt} = 0.994 * 1184.7 = 1177.6\, mm \text{ , } \boldsymbol{take\ it\ equal\ 1180\ mm} \text{ .}$$

Lap splicing

$l_{st} = 1.3 l_{dt} = 1.3 * 1180\ mm = \mathbf{1530\ mm > 300\ mm}.$

Confinement spacing (So) and Confinement Length (lo) are to be determined.

So shall not exceed the smallest of:

- The smaller of $8d_b$ of the smallest longitudinal bar enclosed and 200 mm.
- One-half the smallest dimension of the column’s cross section.

$8d_b = 8 * 40 = 320$ mm

0.5*1600 = 800 mm

Confinement Spacing (So) = 200 mm

Lo shall not be less than the largest of:

- One-sixth the clear span of the column
- Maximum cross-sectional dimension of the column
- 450 mm

Lo = 1600 mm

Shear Reinforcement

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

	Shear V_u kN	Shear ϕV_c kN	Shear ϕV_s kN	Shear ϕV_p kN	Rebar A_v /s mm²/m
Major, V_{u2}	430.2944	6201.1424	0	0	0
Minor, V_{u3}	1425.3406	6201.1424	0	1344.9746	0

Joint Shear Check/Design

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

(6/5) Beam/Column Capacity Ratio

Major Ratio	Minor Ratio
N/N	N/N

Minimum Shear Reinforcement

Table 9.6.3.4—Required $A_{v,min}$

Beam type	$A_{v,min}/s$		
Nonprestressed and prestressed with $A_{ps}f_{se} < 0.4(A_{ps}f_{pu} + A_s f_y)$	Greater of:	$0.062\sqrt{f'_c}\frac{b_w}{f_{yt}}$	(a)
		$0.35\frac{b_w}{f_{yt}}$	(b)

A_v/s_{min} = 0.001235*1600 = 1.976 mm^2/mm.

Use nine legs stirrups with diameter 12 mm with spacing of 400 m

Design Checks for Columns

ϕVn shall be at least the lesser of (a) and (b):

a) The shear associated with development of nominal moment strengths of the column at each restrained end of the unsupported length due to reverse curvature bending. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the highest flexural strength.
b) The maximum shear obtained from factored load combinations that include *E*, with $\Omega_0 E$ substituted for *E*.

$ØV_s = 0.75*f_{yt}*d*Av/s$

Av/s = 2.54 mm^2/mm

V_s = 0.75*420*1525*2.54 = 1220152 N = 1220.152 kN

$ØV_n = ØV_c + ØV_s$ = 6201.14 + 1220.152 = 7421.3 kN

V_u from design load combination = 1425.34 kN

$ØV_n > V_u$, Acceptable

- **Special Cases as per code**

18.4.3.1 ϕV_n shall be at least the lesser of (a) and (b):

(a) The shear associated with development of nominal moment strengths of the column at each restrained end of

the unsupported length due to reverse curvature bending. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the highest flexural strength

(b) The maximum shear obtained from factored load combinations that include E, with $\Omega_o E$ substituted for E

Figure 3.3.5 – Code Recommendations for Column Strength

As seen from the above screenshots for shear reinforcements, **ØVn > Vu from the load combination including overstrength factor.**

18.4.3.6 Columns supporting reactions from discontinuous stiff members, such as walls, shall be provided with transverse reinforcement at the spacing s_o in accordance with 18.4.3.3 over the full height beneath the level at which the discontinuity occurs if the portion of factored axial compressive force in these members related to earthquake effects exceeds $A_g f_c'/10$. If design forces have been magnified to account for the overstrength of the vertical elements of the seismic-force-resisting system, the limit of $A_g f_c'/10$ shall be increased to $A_g f_c'/4$. Transverse reinforcement shall extend above and below the column in accordance with 18.7.5.6(b).

Figure 3.3.6 – Code Recommendation for Critical Columns

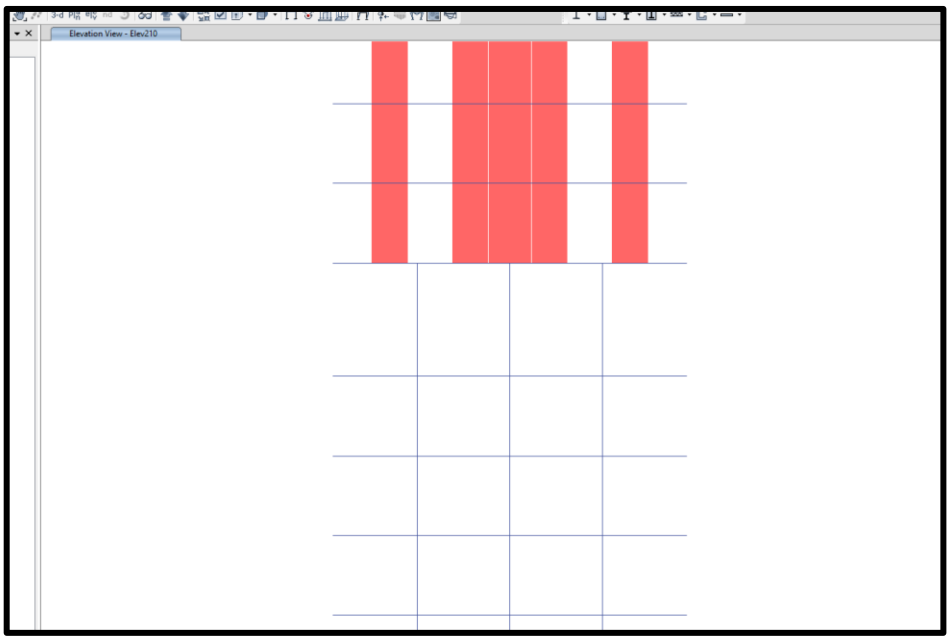


Figure 3.3.7 – Critical Columns Elevation View

As seen, this case exists in story 79 columns.

6. Walls Special Case

Column Element Details (Part 1 of 2)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)
80	C44	2	Column 2	(1.2+0.2*Sds)(D+SD)+LL-2.5EDX + Soil	0	5200

Column Element Details (Part 2 of 2)

LLRF	Type
0.5	Sway Intermediate

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
1600	1600	101	62.3

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
39323	70	1	413.69	413.69

Design Code Parameters

ϕ_T	ϕ_{CTied}	$\phi_{CSpiral}$	ϕ_{Vns}	ϕ_{Vs}	ϕ_{Vjoint}	Ω_0
0.9	0.65	0.75	0.75	0.6	0.85	2.5

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 M2 kN-m	Minimum M3 kN-m	Rebar Area mm²	Rebar % %
17004.6323	-794.3934	-1469.1095	1075.3729	1075.3729	25600	1

Axial Force and Biaxial Moment Factors

The walls are not designed using combinations including overstrength factor, so check the value $A_g f'_c/10$.

Ag = 22500 mm^2

f'c = 70 MPa

$A_g f'_c/10$ = 157700 N = 157.7 kN

Thus, design the column in accordance with 18.7.5.6(b).

(b) Transverse reinforcement shall extend into the discontinued member at least ℓ_d of the largest longitudinal column bar, where ℓ_d is in accordance with 18.8.5. Where the lower end of the column terminates on a wall, the required transverse reinforcement shall extend into the wall at least ℓ_d of the largest longitudinal column bar at the point of termination. Where the column terminates on a footing or mat, the required transverse reinforcement shall extend at least 300 mm into the footing or mat.

Figure 3.3.8 – Code Recommendation for Walls Supported by Critical Columns

CODE

18.8.5.3 For bar sizes No. 10 through No. 36, ℓ_d, the development length in tension for a straight bar, shall be at least the greater of (a) and (b):

(a) 2.5 times the length in accordance with 18.8.5.1 if the depth of the concrete cast in one lift beneath the bar does not exceed 300 mm.

(b) 3.25 times the length in accordance with 18.8.5.1 if the depth of the concrete cast in one lift beneath the bar exceeds 300 mm.

COMMENTARY

R18.8.5.3 Minimum development length in tension for straight bars is a multiple of the length indicated by 18.8.5.1. Section 18.8.5.3(b) refers to top bars. Lack of reference to No. 43 and No. 57 bars in 18.8.5 is due to the paucity of information on anchorage of such bars subjected to load reversals simulating earthquake effects.

Figure 3.3.9 – Code Recommendation for Stirrups Extension Inside Walls

18.8.5.1 For bar sizes No. 10 through No. 36 terminating in a standard hook, ℓ_{dh} shall be calculated by Eq. (18.8.5.1), but ℓ_{dh} shall be at least the greater of $8d_b$ and 150 mm for normalweight concrete and at least the greater of $10d_b$ and 190 mm for lightweight concrete.

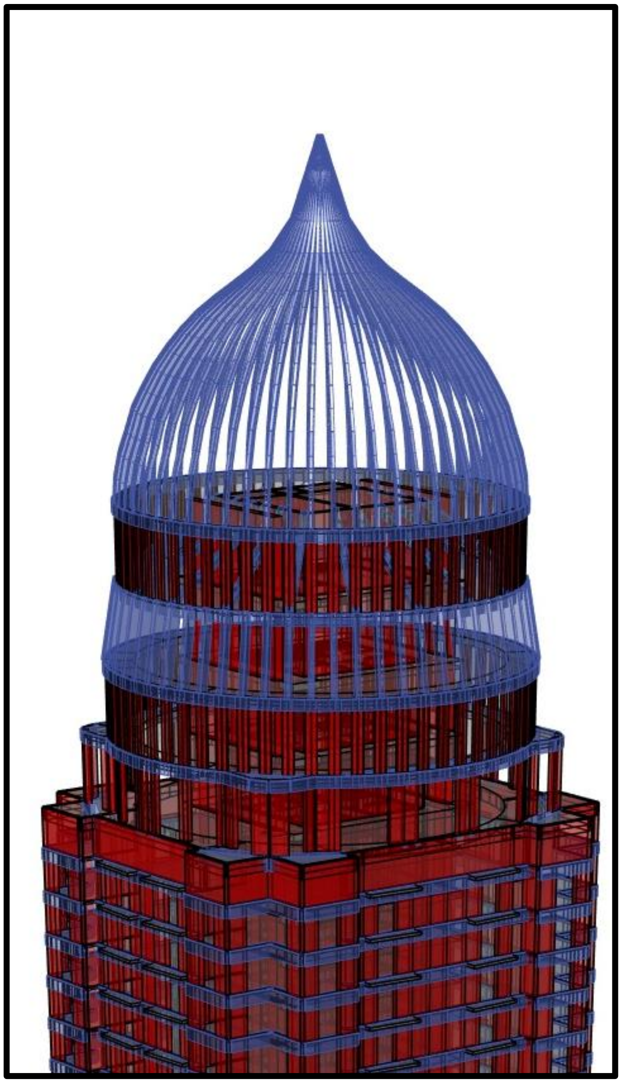
$$\ell_{dh} = f_y d_b / (5.4\lambda\sqrt{f'_c}) \qquad (18.8.5.1)$$

Figure 3.3.10 – Code Recommendation for Critical Columns Development Length

$$L_d = 2.5 * L_{dh} = 2.5 * 420 * \frac{40}{5.4 * 1 * \sqrt{70}} = 929.6\, mm, \boldsymbol{take\ it\ 930\ mm.}$$

Stirrups should extend to inside the walls up to 930 mm.

3.4 Design of Inclined Columns



1. **Inclined Columns (Story 99)**

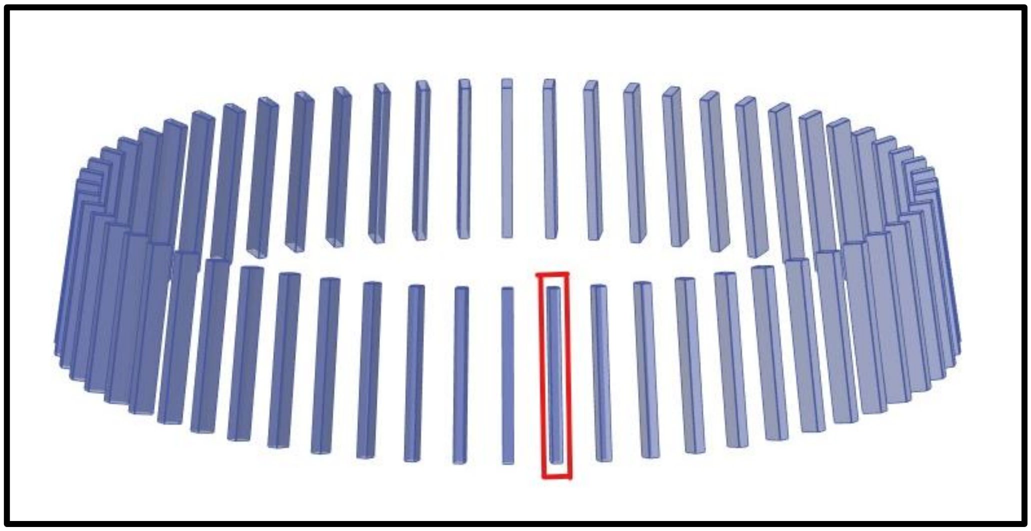


Figure 3.4.1 – Location of Inclined Column to Check

Rectangular Cross Section, B=400 mm H=900 mm.

Cover: 40 mm

Longitudinal reinforcement

Column Element Details (Part 1 of 2)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)
99	D232	8206	Transfer Member	(1.2+0.2*Sds) (D+SD) +LL-EDY + Soil	0	6404

Column Element Details (Part 2 of 2)

LLRF	Type
1	Sway Intermediate

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
400	900	60	27.3

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 M2 kN-m	Minimum M3 kN-m	Rebar Area mm²	Rebar % %
206.3228	-89.774	317.6772	5.6202	8.7151	3600	1

Axial Force and Biaxial Moment Factors

	C_m Factor Unitless	δ_{ns} Factor Unitless	δ_s Factor Unitless	K Factor Unitless	Length mm
Major Bend(M3)	1	1.009553	1	1	6404
Minor Bend(M2)	0.204289	1	1	1	6404

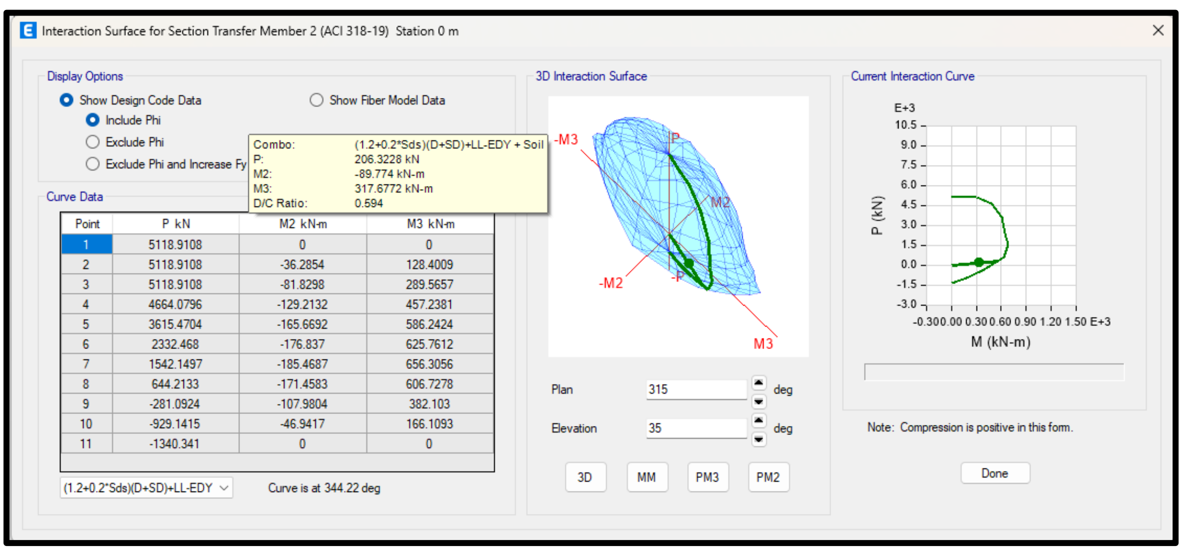


Figure 3.4.2 – Inclined Column P-M2 Interaction

Interaction diagram for Inclined Columns (P-M2)

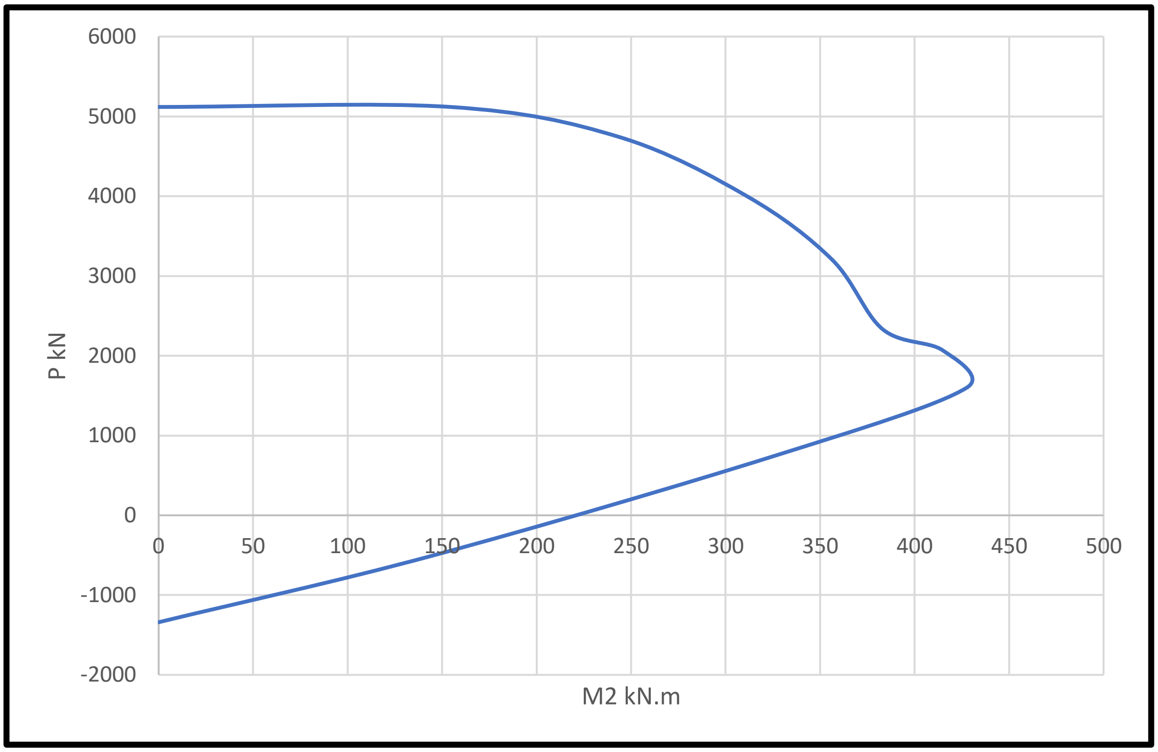


Figure 3.4.3 – Inclined Column P-M2 Interaction Diagram

The axial load on the Inclined Columns Pu = 206.3228 KN and Mu_2 = -89.774 KN.m and this value is under the curve.

Interaction diagram for Inclined Columns (P-M3)

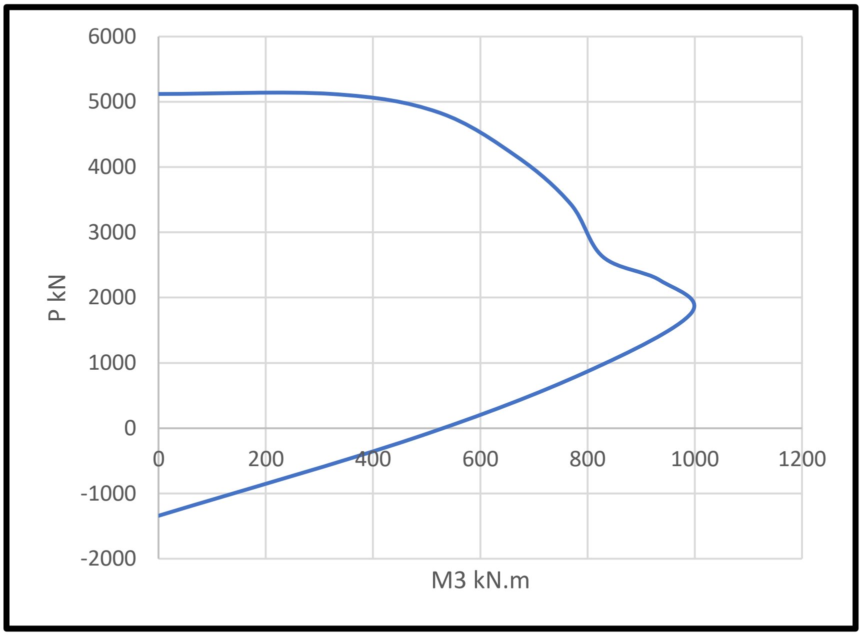


Figure 3.4.5 – Inclined Column P-M3 Interaction Diagram

The axial load on the Inclined Columns Pu = 206.3228 KN and Mu3 = 317.6772 KN.m and this value is under the curve.

Rebar area = 3600 mm²

Use 12 $\emptyset$ 20 $\boldsymbol{longitidual\ reinforcement}$

Development Length

$$\iota_{dt} = \frac{0.59 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b \geq 20 \text{ mm}$$

$$\iota_{dt} \geq 300 \text{ mm}$$

$$d_b = 25 \text{ mm}$$

$\iota_{dt} = \frac{0.59*420}{\sqrt{70}} * 25 = 740.44$ mm. take $\iota_{dt} = 750$ mm

Lap splicing:

$l_{st} = 1.3 l_{dt} = 1.3 * 750\, mm = \mathbf{975\, mm} > 300\, mm$

Confinement spacing (So) and confinement length (lo) are to be determined.

So shall not exceed the smallest of:

- The smaller of $8d_b$ of the smallest longitudinal bar enclosed and 200 mm.
- One-half the smallest dimension of the column's cross section.

$8d_b = 8 * 25 = 200$ mm

0.5*400 = 200 mm

Thus, take Confinement spacing (So) **= 200 mm**

Lo shall not be less than the largest of:

- One-sixth the clear span of the column
- Maximum cross-sectional dimension of the column
- 450 mm

Lo = 900 mm

Shear Reinforcement

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

	Shear V_u kN	**Shear ϕV_c kN**	**Shear ϕV_s kN**	**Shear ϕV_p kN**	**Rebar A_v /s mm^2/m**
Major, V_{u2}	64.1939	128.8902	0	0	0
Minor, V_{u3}	27.8855	155.5748	0	0	0

Minimum shear reinforcement

A_v/s_{min} = 0.001235*400 = **0.494 mm²/mm**

Use **two stirrups (two legs stirrups) with diameter 10 mm with spacing of 300 mm.**

Along one Horizontal cross tie with diameter 10 mm with spacing of 300 mm.

Design Checks for Inclined Member

ϕVn shall be at least the lesser of (a) and (b):

a) The shear associated with development of nominal moment strengths of the column at each restrained end of the unsupported length due to reverse curvature bending. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the highest flexural strength.
b) The maximum shear obtained from factored load combinations that include *E*, with $\Omega_0 E$ substituted for *E*.

$ØV_s = 0.75*f_{yt}*d*Av/s$

$Av/s = 0.52\ mm^2/mm$

V_s = 0.75*420*825*0.52 = 135135 N = 135.135 kN

$ØV_n = ØV_c + ØV_s$ = 128.89 + 135.135 = 264.025 kN

V_u from design load combination = 64.194 kN

$ØV_n > V_u$ Acceptable.

3.5 Design of Walls

1. Pier - Story 65 (Pier 49)

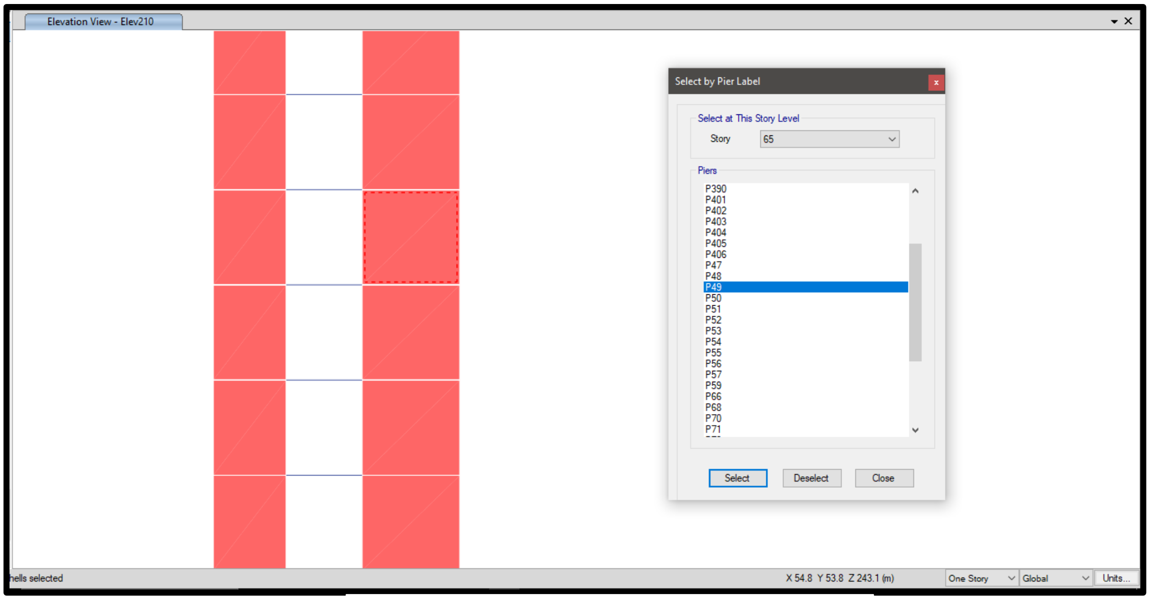


Figure 3.5.1 – Pier 49 Elevation View

Story ID	Pier ID	Centroid X (mm)	Centroid Y (mm)	Length (mm)	Thickness (mm)	LLRF
65	P49	54848	44078.8	3749.9	600	0.4

Station Location	Required Rebar Area (mm²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P_u kN	M_{u2} kN-m	M_{u3} kN-m	Pier A_g mm²
Top	5625	0.0025	0.0101	(0.9-0.2Sds)(D+SD)-EDY	10773.4704	-4.7164	-702.6253	2249922
Bottom	5625	0.0025	0.0101	(0.9-0.2Sds)(D+SD)-EDY	11213.9245	-5.466	-701.4266	2249922

Station Location	ID	Rebar mm²/m	Shear Combo	P_u kN	M_u kN-m	V_u kN	ϕV_c kN	ϕV_n kN
Top	Leg 1	1500	(1.2+0.2*Sds) (D+SD) +LL-EDY + Soil	13826.3236	646.7319	226.1054	1861.5201	3257.6603
Bottom	Leg 1	1500	(1.2+0.2*Sds) (D+SD) +LL-EDY + Soil	14324.1359	660.0815	239.8634	1861.5201	3257.6603

Station Location	ID	Edge Length (mm)	Governing Combo	P_u kN	M_u kN-m	Stress Comp MPa
Top–Left	Leg 1	Not Required	(1.2+0.2*Sds) (D+SD) +LL+EDY + Soil	16749.0679	646.7319	6.98
Top–Right	Leg 1	Not Required	(1.2+0.2*Sds) (D+SD) +LL+EDY + Soil	16749.0679	646.7319	7.9
Bottom–Left	Leg 1	Not Required	(1.2+0.2*Sds) (D+SD) +LL+EDY + Soil	17423.1854	660.0815	7.27
Bottom–Right	Leg 1	Not Required	(1.2+0.2*Sds) (D+SD) +LL+EDY + Soil	17423.1854	660.0815	8.21

- **Vertical Longitudinal Reinforcement**

$5625\ /\ (2 * 3.75)\ =\ 750\ mm^2/m$

Use Ø 14 / 200 mm

- **Shear Reinforcement**

As/s = 1500 mm2 /m

Use Ø 12/ 150 mm

Length of the vertical bars = Height of the wall + 0.5 the height of the above wall

So, the length of vertical bars = 3.65 + (3.65/2) = 5.5 m

Ldt = 3.65/2 = 1.825 m

Splicing length: Lst = 150 mm, 1.3 lst = 195 mm or 300 mm which is larger.

Take lst = 300mm

2. Pier - Story 85 (Pier 50)

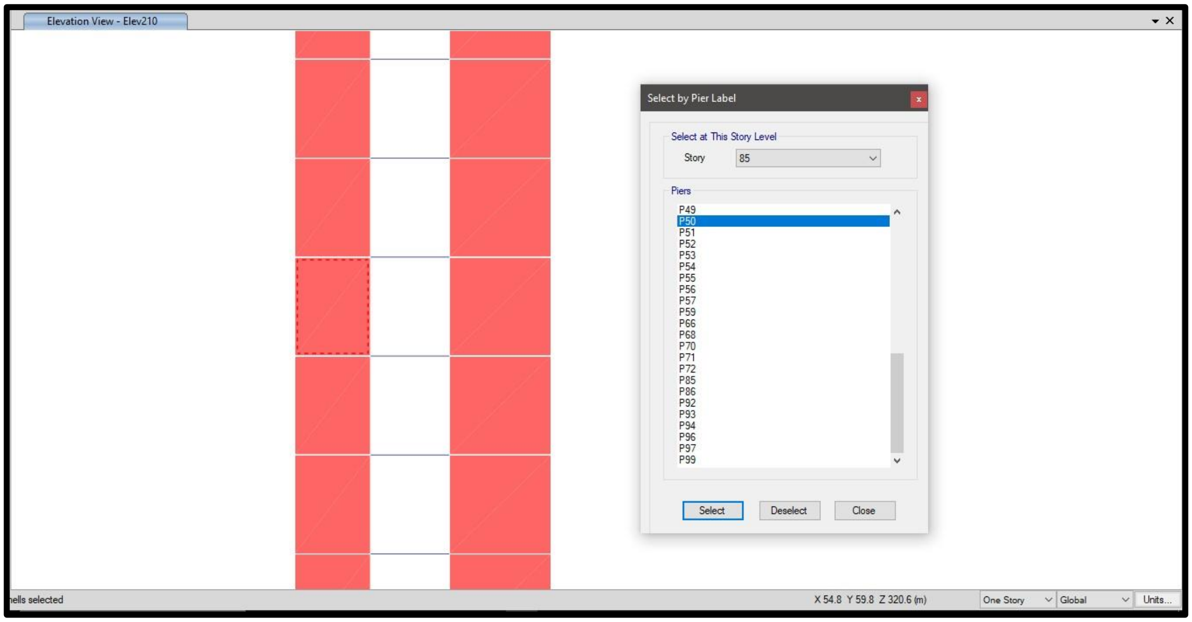


Figure 3.5.2 – Pier 50 Elevation View

Story ID	Pier ID	Centroid X (mm)	Centroid Y (mm)	Length (mm)	Thickness (mm)	LLRF
85	P50	54848	50253.7	2800.1	600	0.491

Station Location	Required Rebar Area (mm²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P_u kN	M_{u2} kN-m	M_{u3} kN-m	Pier A_g mm²
Top	4200	0.0025	0.0105	(0.9-0.2Sds)(D+SD)-EDY	3952.8195	-3.8017	-254.2641	1680066
Bottom	4200	0.0025	0.0105	(0.9-0.2Sds)(D+SD)-EDY	4103.9328	-4.6771	-385.1063	1680066

Station Location	ID	Rebar mm²/m	Shear Combo	P_u kN	M_u kN-m	V_u kN	ϕV_c kN	ϕV_n kN
Top	Leg 1	1500	(1.2+0.2*Sds) (D+SD) +LL-EDY + Soil	5821.0891	148.7719	345.5034	1390.0378	2432.5662
Bottom	Leg 1	1500	(1.2+0.2*Sds) (D+SD) +LL-EDY + Soil	5993.5592	257.0855	480.5516	1390.0378	2432.5662

Station Location	ID	Edge Length (mm)	Governing Combo	P_u kN	M_u kN-m	Stress Comp MPa
Top–Left	Leg 1	Not Required	(1.2+0.2*Sds) (D+SD) +LL+EDX + Soil	6410.4333	1.5203	3.81
Top–Right	Leg 1	Not Required	(1.2+0.2*Sds) (D+SD) +LL+EDX + Soil	6410.4333	1.5203	3.82
Bottom–Left	Leg 1	Not Required	(1.2+0.2*Sds) (D+SD) +LL+EDX + Soil	6632.0337	30.0062	3.91
Bottom–Right	Leg 1	Not Required	(1.2+0.2*Sds) (D+SD) +LL+EDX + Soil	6632.0337	30.0062	3.99

- **Vertical Longitudinal Reinforcement**

4200 / (2*2.8) = 750 mm^2/m.

Use Ø 14 / 200 mm

- **Shear Reinforcement**

Av/s = 1500 mm^2 /m

Use Ø 12/ 150 mm

Length of the vertical bars = Height of the wall + 0.5 the height of the above wall

So, the length of vertical bars = 3.65 + (3.65/2) = 5.5 m

Ldt = 3.65/2 = 1.825 m

Splicing length: Lst = 150 mm, 1.3 lst = 195 mm or 300 mm which is larger

Take lst = 300mm

3. Spandrel - Story 65 (SL 25)

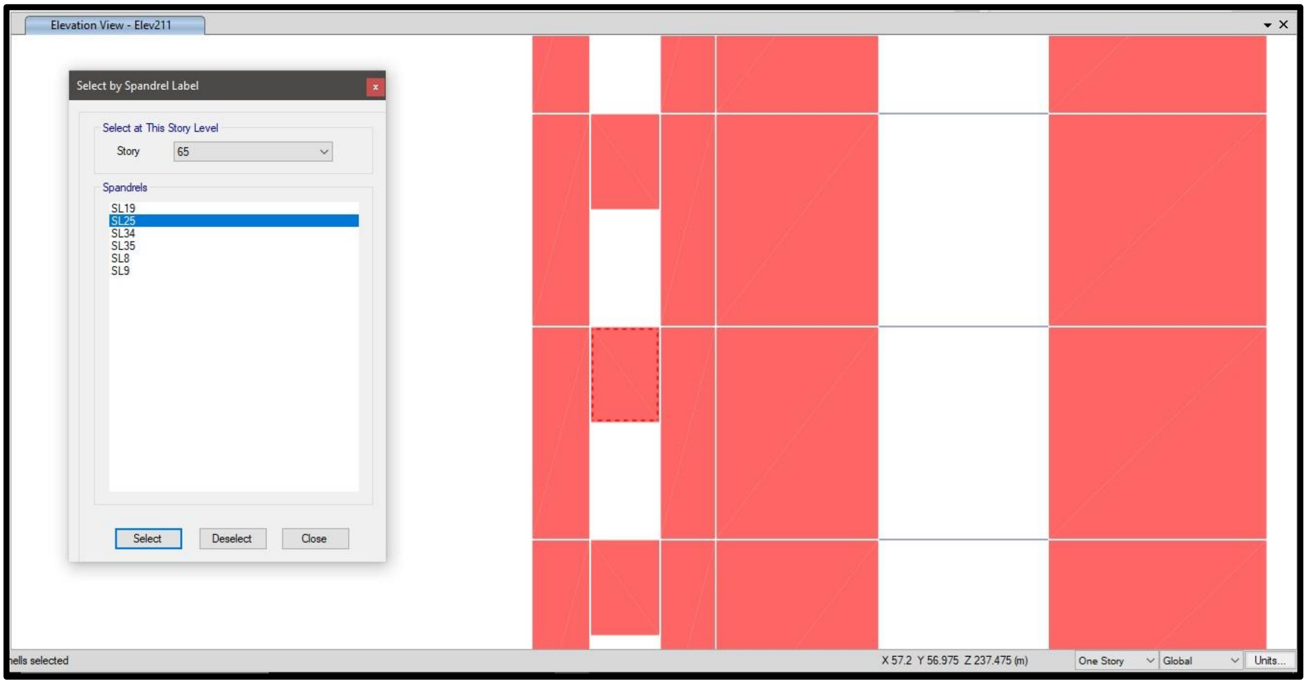


Figure 3.5.3 – Spandrel 25 Elevation View

Story ID	Spandrel ID	Centroid X (mm)	Centroid Y (mm)	Depth (mm)	Width (mm)	LLRF
65	SL25	57198	52603.9	1650	600	1

Spandrel Flexural Design—Top Reinforcement

Station Location	Reinf Area mm²	Reinf Percentage	Reinf Combo	Moment, M_u kN-m
Left	340	0.03	(0.9-0.2Sds)(D+SD)-EDY	-320.976
Right	0	0	(0.9-0.2Sds)(D+SD)-EDY	0

Spandrel Flexural Design—Bottom Reinforcement

Station Location	Reinf Area mm²	Reinf Percentage	Reinf Combo	Moment, M_u kN-m
Left	0	0	(0.9-0.2Sds)(D+SD)-EDY	0

Station Location	Reinf Area mm²	Reinf Percentage	Reinf Combo	Moment, M_u kN-m
Right	209	0.02	(0.9-0.2Sds)(D+SD)-EDY	273.6423

Spandrel Shear Design

Station Location	A_{vert} mm²/m	A_{horiz} mm²/m	ShearCombo	V_u kN	ϕV_c kN	ϕV_s kN	ϕV_n kN
Left	1500	1500	(1.2+0.2*Sds) (D+SD) +LL+EDY + Soil	773.2172	777.253	552.8907	1330.1437
Right	1500	1500	(1.2+0.2*Sds) (D+SD) +LL+EDY + Soil	608.168	776.2631	552.8907	1329.1537

Spandrel Shear Design—Diagonal Reinforcement

Station Location	A_{diag} mm²	Shear Combo	V_u kN	V_{uLimit} kN	L/H Ratio	Seismic Design	Diag Reinf Mandatory
Left	1486	(1.2+0.2*Sds) (D+SD) +LL+EDY + Soil	773.2172	2457.2917	0.727	Yes	No
Right	1169	(1.2+0.2*Sds) (D+SD) +LL+EDY + Soil	608.168	2457.2917	0.727	Yes	No

- **Top Steel:**

As = 340 mm^2, **use 4 Ø12, length = 2.4 m**

- **Bottom steel:**

As = 209 mm^2, **use 4 Ø 10, length = 2.4 m**

- **Horizontal steel between the bottom and the top steel** = 0.0025(1000) (600)

= 1500 mm^2, use **Ø 20 / 200 mm, length = 1.55 m**

- **Shear design:**

Av/s = 1500 mm^2/m = 1.5 mm^2/mm

Use stirrup. **12:**

Spacing = (113*2)/ (1.5) = 150 mm → use Ø 12 / 150 mm

- **The diagonal shear reinforcement required = 1486 mm^2**

Use 4 Ø 25

Use hoops. 16 / 150 mm, length = Ldt = $\frac{0.59 * fy * db}{\sqrt{f'c}}(1.25)$

$= \frac{0.59(420)(25)}{\sqrt{70}}(1.25) = 925.56$ mm **take Ldt = 1000 mm**

<u>**So, the length of diagonal bars = 4040 mm takes it 4.1 m**</u>

$$\sin \alpha = \frac{0.8 \text{ hs}}{\sqrt{\text{Ls}^2 + (0.8 \text{ hs})^2}} = \frac{0.8(1650)}{\sqrt{1000^2 + \left(0.8(1650)\right)^2}} = 0.79$$

So, the angle between diagonal bars and the longitudinal axis, <u>**α = 52 degree**</u>

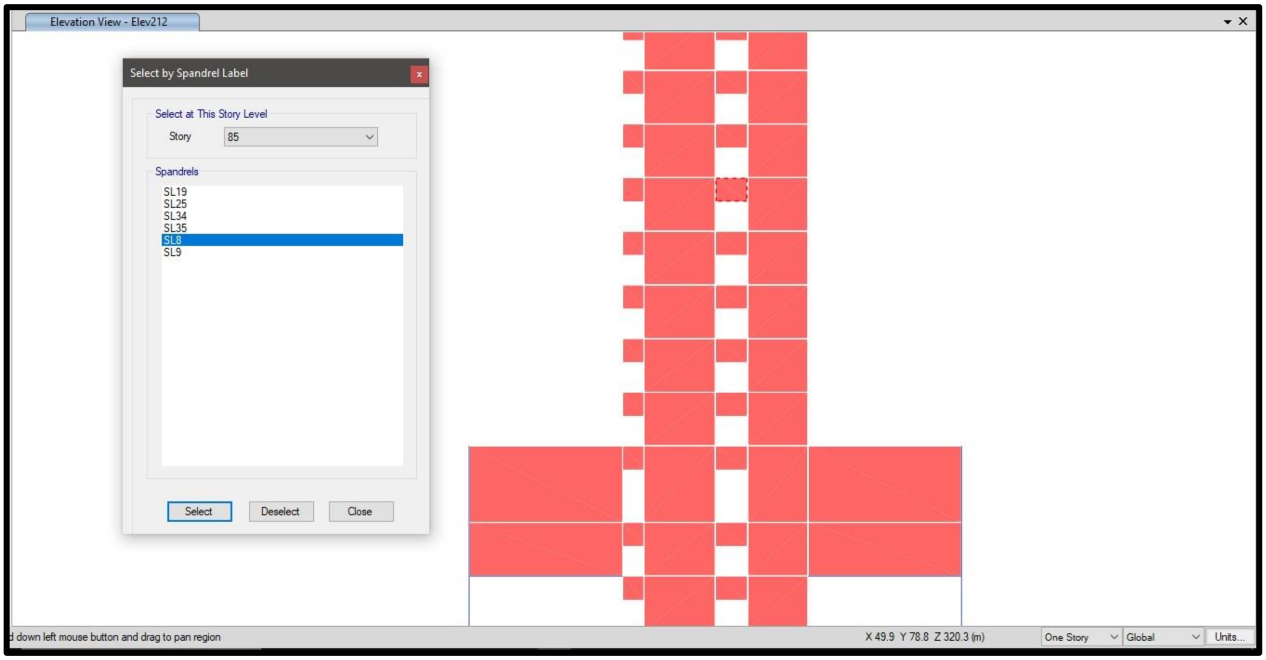


Figure 3.5.4 – Spandrel 8 Elevation View

4. Spandrel - Story 85(SL 8)

Story ID	Spandrel ID	Centroid X (mm)	Centroid Y (mm)	Depth (mm)	Width (mm)	LLRF
85	SL8	49898.1	46303.9	1650	850	1

Spandrel Flexural Design—Top Reinforcement

Station Location	Reinf Area mm²	Reinf Percentage	Reinf Combo	Moment, M_u kN-m
Left	2781	0.2	(0.9-0.2Sds)(D+SD)-EDY	-894.1755
Right	2606	0.19	(0.9-0.2Sds)(D+SD)-EDY	-821.499

Spandrel Flexural Design—Bottom Reinforcement

Station Location	**Reinf Area mm²**	**Reinf Percentage**	**Reinf Combo**	**Moment, M_u kN-m**
Left	2334	0.17	(0.9-0.2Sds) (D+SD)-EDY	709.9329
Right	2433	0.17	(0.9-0.2Sds) (D+SD)-EDY	750.3649

Spandrel Shear Design

Station Location	**A_{vert} mm²/m**	**A_{horiz} mm²/m**	**ShearCombo**	**V_u kN**	**ϕV_c kN**	**ϕV_s kN**	**ϕV_n kN**
Left	2125	2125	(1.2+0.2*Sds) (D+SD) +LL+EDY + Soil	1288.1994	1027.2669	783.2618	1810.5287
Right	2125	2125	(1.2+0.2*Sds) (D+SD) +LL+EDY + Soil	1125.6997	1024.69	783.2618	1807.9518

Spandrel Shear Design—Diagonal Reinforcement

Station Location	**A_{diag} mm²**	**Shear Combo**	**V_u kN**	**V_{uLimit} kN**	**L/H Ratio**	**Seismic Design**	**Diag Reinf Mandatory**
Left	3560	(1.2+0.2*Sds) (D+SD) +LL+EDY + Soil	1288.1994	3481.1633	1.333	Yes	No
Right	3111	(1.2+0.2*Sds) (D+SD) +LL+EDY + Soil	1125.6997	3481.1633	1.333	Yes	No

- **Top Steel:**

As = 2781 mm^2, **use 6 Ø 25, length = 4.5 m**

- **Bottom steel:**

As = 2433 mm^2, **use 5 Ø 25, length = 4.5 m**

- **Horizontal steel between the bottom and the top steel** = 0.0025(1000) (850)

= 2125 mm^2 **use Ø 20 / 150 mm, length = 1.55 m**

- **Shear design:**

Av/s = 2125 mm^2/m = 2.125 mm^2/mm

Use stirrup Ø 12

Spacing = (113*2)/ (2.125) = 100 mm, use Ø 12 / 100 mm

- **The diagonal shear reinforcement required = 3560 mm^2**

Use 4 Ø 35

Use hoops. 16 / 100 mm, length = Ldt = $\frac{0.59*fy*db}{\sqrt{f'c}}(1.25)$

$= \frac{0.59(420)(35)}{\sqrt{70}}(1.25) = 1295.78$ mm , **take Ldt = 1300 mm**

<u>So, the length of diagonal bars = 5350 mm takes it 5.35 m</u>

$$\sin\alpha = \frac{0.8\text{ hs}}{\sqrt{\text{Ls}^2 + (0.8\text{ hs})^2}} = \frac{0.8(1650)}{\sqrt{1300^2 + \left(0.8(1650)\right)^2}} = 0.71$$

So, the angle between diagonal bars and the longitudinal axis, **<u>α = 45.44 degree</u>**

3.6 – Design of Staircase

3.6.1 - Steps for stair case design

- Specify material data
- Specify dimensions for architectural plans and sections.
- Assume or calculate initial waist slab thickness for deflection control.
- Loads calculation
- Take unit strip of 1m of stairs slab.
- Calculate load per length
- Calculate shear and moment values and then check depth.
- Reinforcement steel determined and checked.

3.6.2 – Stairs' dimensions

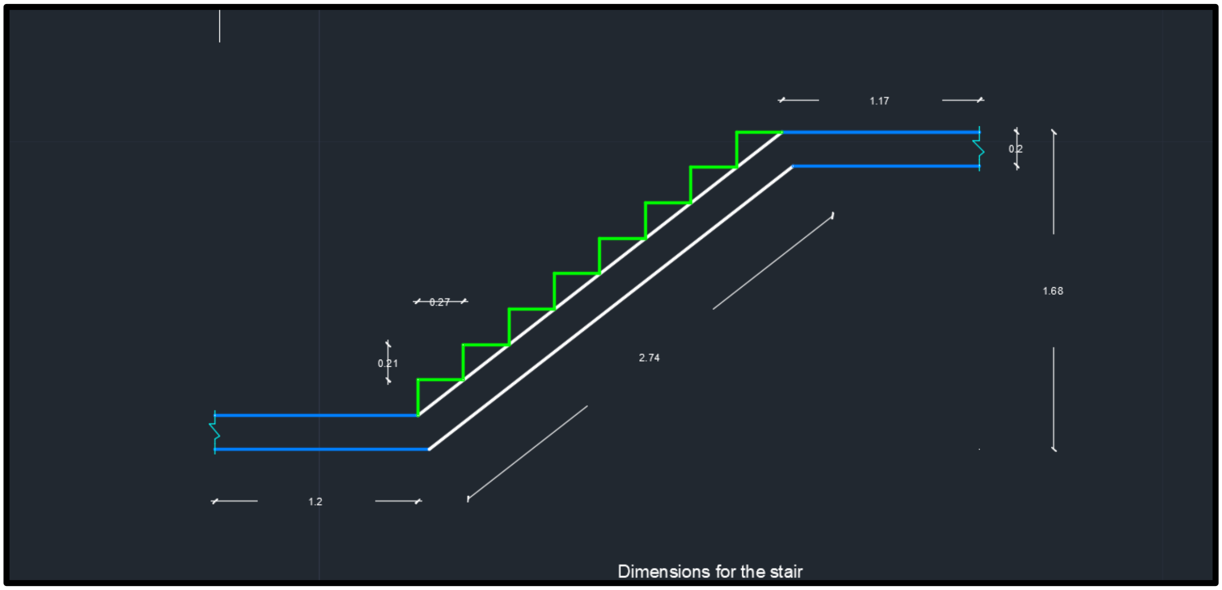


Figure 3.6.2.1 – Staircase Longitudinal Section

- Starting landing length = 1.2 m
- Ending landing length = 1.17 m
- Length of each step (Trade) = 0.27 m.
- Height of each step = 0.21 m.
- # of trades = 8
- # of heights = 8

Total horizontal length = 8*0.27 = 2.16 m
Total height = 8*0.21 = 1.68 m

$$\emptyset = tan^{-1}\left(\frac{1.68}{2.16}\right) = 37.88\ degrees$$

Stair inclined length = $\frac{2.16}{\cos 37.88} = 2.74\ m$

Effective span length = 1.2+2.16+1.17 = 4.53 m

One-way solid slab thickness is assumed to be L/20 to control deflection. Since flight does not have uniform thickness as normal slab, flight of stair is stiffer (for trade + step height) than slab of waist slab thickness, so required slab depth is reduced by 15%.

$$Min\ waist\ slab\ thickness = 0.85\left(\frac{L}{20}\right) = 0.85\left(\frac{4.53}{20}\right)$$
$$= 0.192\ m\ , take\ thickness\ as\ 0.20\ m\ .$$

$$Clear\ cover = 25\ mm\ , effective\ depth = 200 - 25(cover) - 6(half\ bar\ diameter)$$
$$= 169\ mm = 0.169\ m$$

3.6.3 – Staircase Loads

LL = 5 kN/m^2, SD = 4.5 kN/m^2

- Load calculation for flight:

$$Self\ weight\ of\ waist\ slab = 0.2 * 25 = 5\ kN/m^2$$

$$Self\ weight\ \ of\ \ steps = \frac{8 * \frac{1}{2} * (0.21 * 0.27) * 25}{2.74} = \ 2.07\ kN/m^2$$

$$Total\ dead\ load = 5 + 2.07 + 4.5(SD) = 11.57\ kN/m^2$$

$$Ultimate\ load\ on\ inclined\ length = 1.2D + 1.6L = 1.2(11.57) + 1.6(5) = 21.884\ kN/m^2$$

$$Ultimate\ load\ on\ horizontal\ length = \frac{21.884}{\cos 37.88} = 27.72\ kN/m^2$$

- Load calculation for landing

  $$Total\ dead\ load = 5 + 4.5(SD) = 9.5\ kN/m^2$$

  $$Ultimate\ load\ on\ landing = 1.2(9.5) + 1.6(5) = 19.4\ kN/m^2$$

  Then, SAP software was used to model the stair case, as shown below.

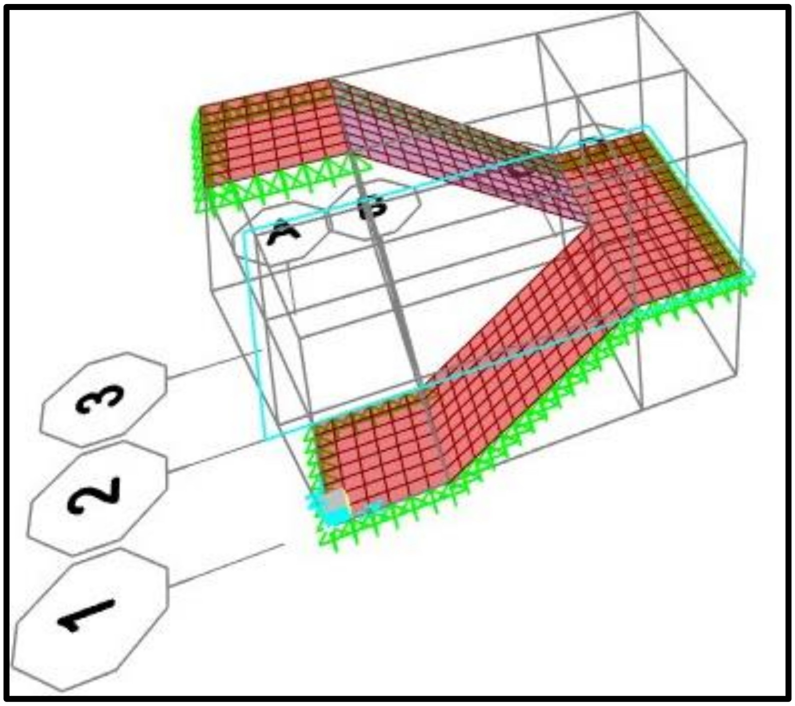


Figure 3.6.3.1 – Staircase 3D View

3.6.4 – Shear reinforcement

From SAP, maximum shear value Vu = 75.31 kN.

$$\lambda_s = \sqrt{\frac{2}{1+0.004*d}} = \sqrt{\frac{2}{1+0.004*169}}$$

$$= 1.09\ since\ greater\ than\ 1\ , take\ it\ as\ 1.$$

$$Mu\ from\ SAP = 11.41\ kN.m$$

$$\rho_w = \frac{0.85*f'c}{fy}(1-\sqrt{1-\frac{2.61*Mu}{bd^2*f'c}}) = \frac{0.85*70}{420}\left(1-\sqrt{1-\frac{2.61*11.41*10^6}{200*169^2*70}}\right)$$

$$= 0.0054$$

$$\emptyset Vc = 1.1(\emptyset)\left(0.66*\lambda*\lambda_s*\rho_w{}^{\frac{1}{3}}*\ \sqrt{f'c}\right)*b_w*d$$

$$= 1.1(0.75)(0.66)(1)(1)\left(0.0054^{\frac{1}{3}}*\sqrt{70}*200*169\right) = 27.01\ kN.m$$

$$< Vu\ , shear\ reinformcement\ shall\ be\ used.$$

$$Vc = \frac{27.01}{0.75} = 36.02\ kN\ , \frac{Vu}{\phi} = \frac{75.31}{0.75} = 100.41\ kN\ , 100.41 - 36.02 = 64.39\ kN(Vs)$$

$$\frac{Av}{s} = \frac{Vs}{fy * d} = \frac{64.9 * 10^3}{420 * 169} = 0.91\ mm^2/mm$$

$$\frac{Av}{s}\ minimum = 0.062 * \sqrt{f'c} * \frac{b_w}{d}\ or\ 0.35 * \frac{b_w}{fy} = 0.247 < 0.91\ , take \frac{Av}{s} = 0.91\ mm^2/mm$$

$$\frac{1}{3} * \sqrt{f'c} * b * d = \frac{1}{3} * \sqrt{70} * 200 * 169 = 94.26\ kN > Vu\ , adequate\ section.$$

$$Smax = \min\left(\frac{d}{2}, 100\right)\ , take\ 100\ mm$$

$$Smax' = \min(d, 600)\ , d = 169\ mm\ , take\ it\ as\ 170\ mm.$$

$$Min\ \#\ of\ legs\ in\ stirrup = \frac{200 - 80}{169} + 1 = 2\ legs$$

Use stirrups of $\phi 12$, Av = 2*113 = 226 mm^2.

$$S = \frac{226}{0.91} = 248.35\ mm\ but\ Smax = 100\ mm\ , take\ spacing\ 100\ mm\ .$$

3.6.5 – Longitudinal Reinforcement for the flights

$$Mu\ in\ x - direction = 4.54\ kN.m$$

$$\rho_w = \frac{0.85 * f'c}{fy}\left(1 - \sqrt{1 - \frac{2.61 * Mu}{bd^2 * f'c}}\right) = \frac{0.85 * 70}{420}\left(1 - \sqrt{1 - \frac{2.61 * 4.54 * 10^6}{200 * 169^2 * 70}}\right) = 0.0021$$

$$As = \rho bd = 0.0021 * 200 * 169 = 71.48\ mm^2$$

$As_{min} = \rho_{min} * b * d = 0.0033 * 200 * 169 = 112.67\ mm^2\ , take\ As = 112.67\ mm^2$. $\boldsymbol{Use\ 2\phi 12}$

$$Spacing = \frac{1000}{112.67/113.1} = 1003.8\ mm$$

$$Smax = \min(5 * h_f, 450) = 5 * 200 = 1000\ mm\ , take\ 450\ mm.$$

$\boldsymbol{Use\ \phi 12\ with\ spacing\ 200\ mm\ , in\ x - direction.}$

Mu in y-direction = 9.81 kN.m

$$\rho = \frac{0.85 * f'c}{fy}\left(1 - \sqrt{1 - \frac{2.61 * Mu}{bd^2 * f'c}}\right) = \frac{0.85 * 70}{420}\left(1 - \sqrt{1 - \frac{2.61 * 9.81 * 10^6}{200 * 169^2 * 70}}\right) = 0.0046$$

As = 0.0046*200*169 = 155.48 mm^2 > As_{min}, take As = 155.48 mm^2.

Use $\boldsymbol{\phi 12\ with\ spacing\ of\ 200\ mm\ in\ y-direction}$.

3.6.6 – Longitudinal Reinforcement for Landings

$Mu = 11.50\ kN.m$, x-direction

$$\rho_w = \frac{0.85 * f'c}{fy}(1 - \sqrt{1 - \frac{2.61 * Mu}{bd^2 * f'c}}) = \frac{0.85 * 70}{420}\left(1 - \sqrt{1 - \frac{2.61 * 11.50 * 10^6}{200 * 169^2 * 70}}\right)$$
$$= 0.0029$$

As = 0.0029*169*200 = 99.51 mm^2 < 112.67 mm^2

$\boldsymbol{\phi 12\ with\ spacing\ 200\ mm\ , in\ x-direction}$.

Mu = 9.81 kN.m, y-direction

$$\rho = \frac{0.85 * f'c}{fy}(1 - \sqrt{1 - \frac{2.61 * Mu}{bd^2 * f'c}}) = \frac{0.85 * 70}{420}\left(1 - \sqrt{1 - \frac{2.61 * 9.81 * 10^6}{200 * 169^2 * 70}}\right)$$
$$= 0.0046$$

As = 0.0046*200*169 = 155.48 mm^2 > As_{min}, take As = 155.48 mm^2.

Use $\boldsymbol{\phi 12\ with\ spacing\ of\ 200\ mm\ in\ y-direction}$.

Finally, shrinkage steel for the steps as follows:

As = 0.0018*200*1000 = 360 mm^2

$$Spacing = \frac{1000}{360/50} = 136\ mm\ , take\ spacing = 150\ mm\ .$$

$\boldsymbol{Use\ \phi 8\ with\ spacing\ of\ 150\ mm}$.

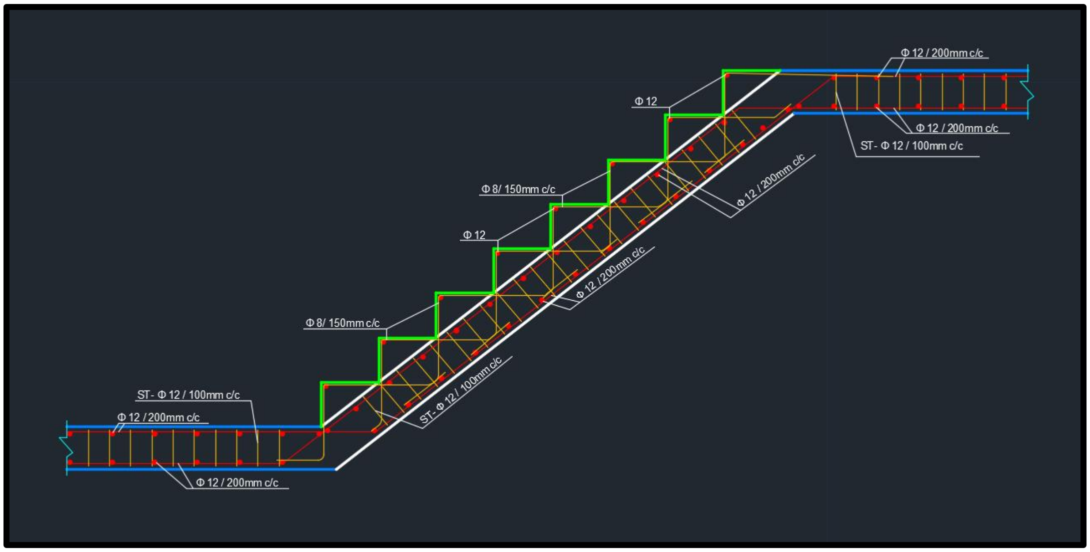


Figure 3.6.6.1 – Staircase Longitudinal Section for Reinforcement

3.7 Design of Diagonal Tensile Members

Diagonal Tensile Members (Story 100)

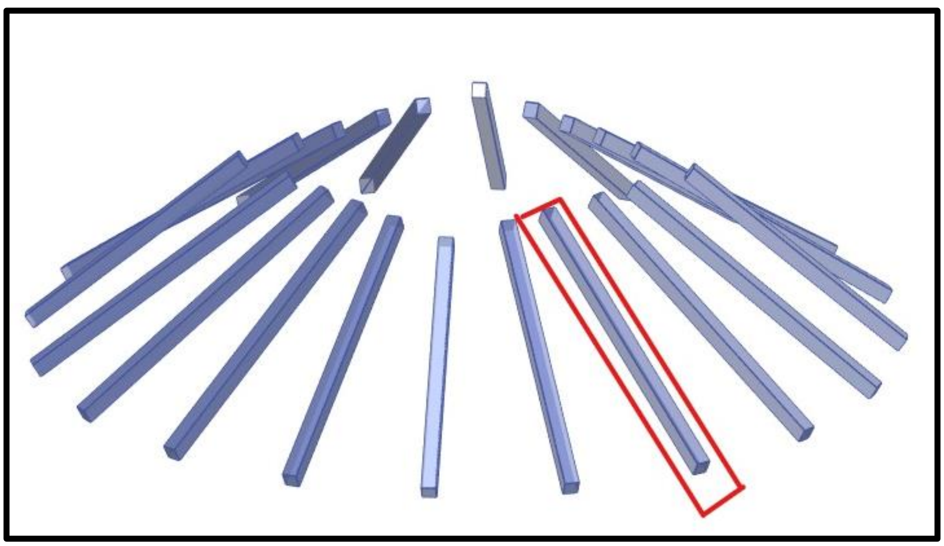


Figure 3.7.1 – Location of Diagonal Tensile Member to Check

Rectangular Cross Section, B=550 mm H=550 mm.

Cover: 50 mm

Longitudinal reinforcement

Column Element Details (Part 1 of 2)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)
100	D24	8461	Tensile Member	(1.2+0.2*Sds) (D+SD) +LL-EDY + Soil	6291.4	12582.8

Column Element Details (Part 2 of 2)

LLRF	Type
1	Sway Intermediate

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
550	550	69	37.3

Material Properties

E_c (MPa)	f'_c (MPa)	Lt. Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
39323	70	1	413.69	413.69

Design Code Parameters

ϕ_T	ϕ_{CTied}	$\phi_{CSpiral}$	ϕ_{Vns}	ϕ_{Vs}	ϕ_{Vjoint}	Ω_0
0.9	0.65	0.75	0.75	0.6	0.85	2.5

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 M2 kN-m	Minimum M3 kN-m	Rebar Area mm²	Rebar % %
-2598.5843	82.4791	166.8816	82.4791	82.4791	8973	2.97

Axial Force and Biaxial Moment Factors

	C_m Factor Unitless	δ_{ns} Factor Unitless	δ_s Factor Unitless	K Factor Unitless	Length mm
Major Bend(M3)	1	1	1	1	12582.8

	C_m Factor Unitless	δ_{ns} Factor Unitless	δ_s Factor Unitless	K Factor Unitless	Length mm
Minor Bend(M2)	1	1	1	1	12582.8

Interaction diagram for Diagonal Tensile Members (P-M2)

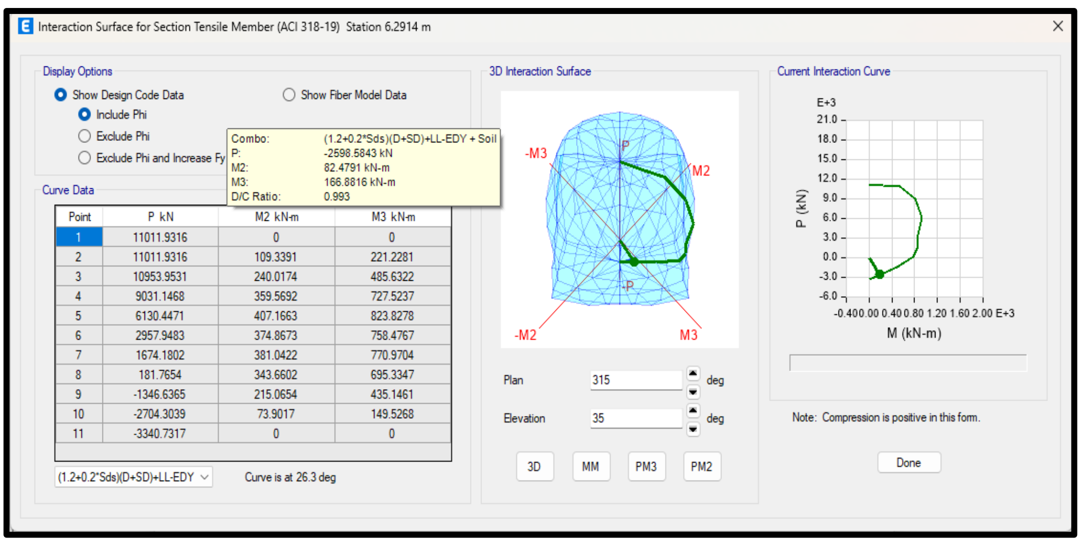


Figure 3.7.2 – Diagonal Tensile Member P-M2-M3 Interaction Surface

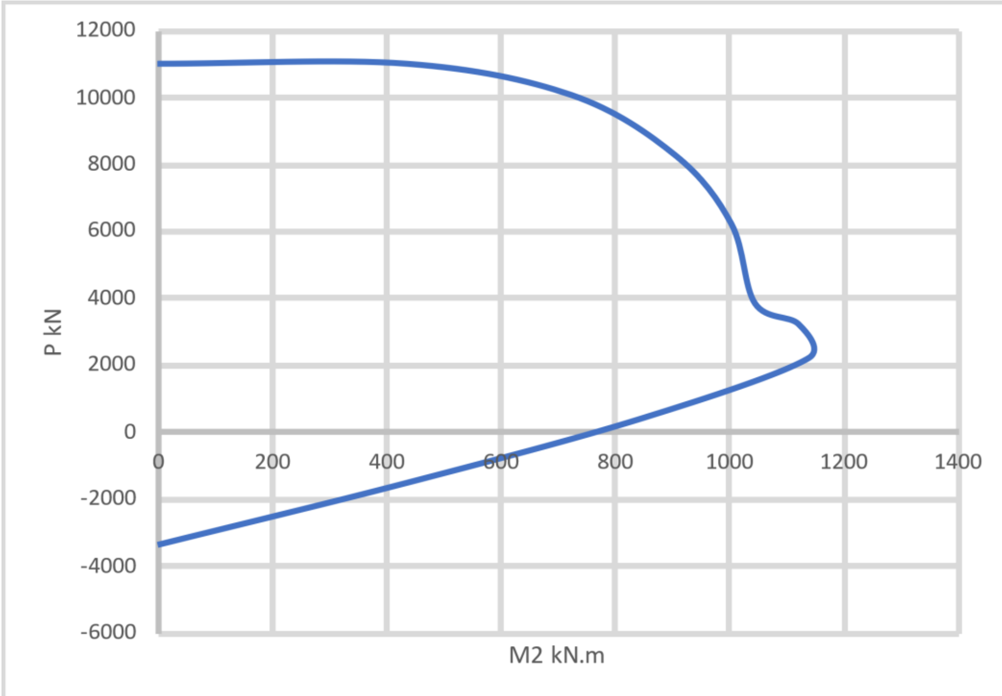


Figure 3.7.3 – Diagonal Tensile Member P-M2 Interaction Diagram

The axial load on the Diagonal Tensile Members Pu = -2598.5843 and Mu_2 = 82.4791 KN.m, and this value is under the curve.

Interaction diagram for Diagonal Tensile Members (P-M3)

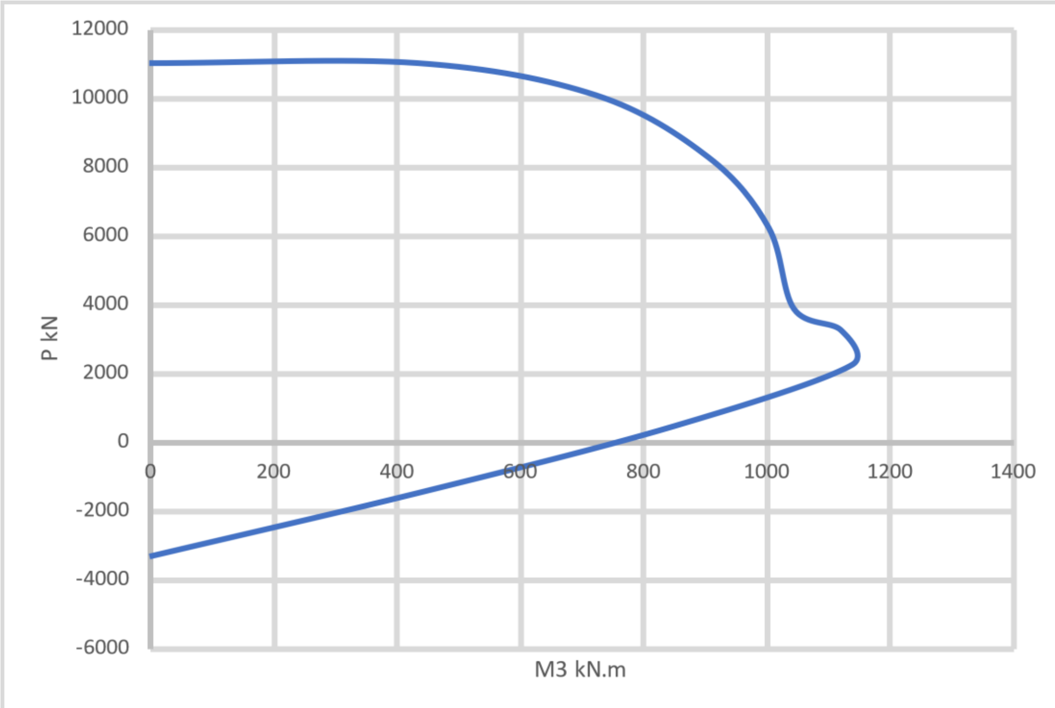


Figure 3.7.4 – Diagonal Tensile Member P-M3 Interaction Diagram

The axial load on the Diagonal Tensile Members Pu = -2598.5843 and Mu3 = 166.8816 KN.m, and this value is under the curve.

Rebar area = 8973 mm^2

- **Use 12 ∅ 32 $\boldsymbol{longitidual\ reinforcement}$**

Development Length

$$\iota_{dt} = \frac{0.59 * f_y}{\sqrt{f'_c}} * d_b \quad \text{for } d_b \geq 20 \text{ mm}$$

$\iota_{dt} \geq 300$ mm

$d_b = 36$ mm

$\iota_{dt} = \frac{0.59*420}{\sqrt{70}} * 36 = 1066.24$ mm. take $\iota_{dt} = 1100$ mm

Lap splicing:

$l_{st} = 1.3 l_{dt} = 1.3 * 1070\ mm = \mathbf{1430\ mm} > 300\ mm$

Confinement spacing (So) and confinement length (lo) are to be determined.

So shall not exceed the smallest of:

- The smaller of $8d_b$ of the smallest longitudinal bar enclosed and 200 mm.
- One-half the smallest dimension of the column's cross section.

$8d_b$ = 8 * 36 = 288 mm, 200 mm.

0.5*550 = 275 mm

Thus, take Confinement spacing (So) = **200 mm**

Lo shall not be less than the largest of:

- One-sixth the clear span of the column
- Maximum cross-sectional dimension of the column
- 450 mm

Lo = 550 mm

- **Shear Reinforcement**

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

	Shear V_u kN	**Shear ϕV_c kN**	**Shear ϕV_s kN**	**Shear ϕV_p kN**	**Rebar A_v /s mm²/m**
Major, V_{u2}	0	0	102.6009	0	687.5
Minor, V_{u3}	0	0	0	0	0

- **Minimum shear reinforcement**

A_v/s_{min} = 0.001235*550 = **0.67925 mm²/mm = 679.25 mm²/m.**

From ETABS, Av/s_{min} = **687.5 mm²/m, so we take value from ETABS.**

A_v/s_{min} = **0.6875 mm²/mm.**

Use one stirrup (two legs stirrups) with diameter 10 mm with spacing of 200 mm.

Design Checks for Diagonal Tensile Members

ϕVn shall be at least the lesser of (a) and (b):

a) The shear associated with development of nominal moment strengths of the column at each restrained end of the unsupported length due to reverse curvature bending. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the highest flexural strength.
b) The maximum shear obtained from factored load combinations that include *E*, with $\Omega_0 E$ substituted for *E*.

$ØV_s = 0.75*f_{yt}*d*Av/s$

$Av/s = 0.78$ mm²/mm

$V_s = 0.75*420*475*0.78 = 116707$ N = 116.707 kN

$ØV_n = ØV_c + ØV_s = 0 + 116.707 = 116.707$ kN

V_u from design load combination = 0 kN

$ØV_n > V_u$ Acceptable.

3.8 Design of Outrigger Members

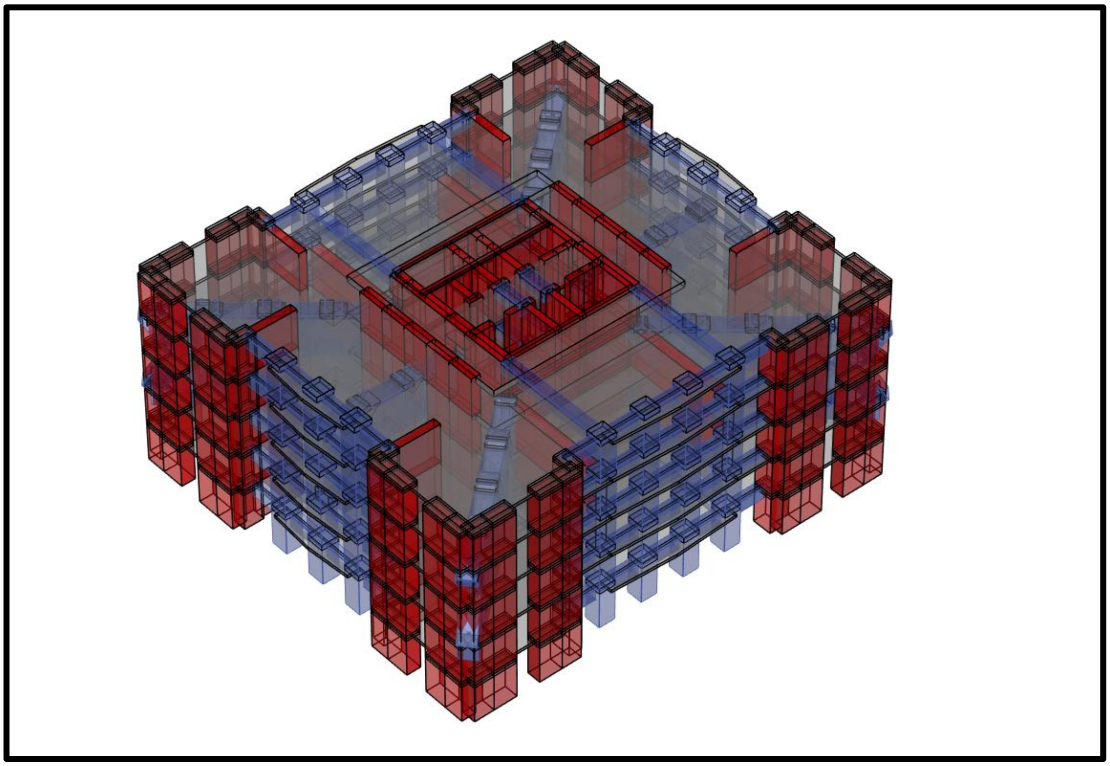


Figure 3.8.1 – Outrigger Region Outside 3D View

Notes for Outrigger Design

- **All longitudinal reinforcement was divided into two layers for each member. In the reinforcement tables along with structural detailed drawings, the total rebar area is used as a sum of the two layers for each member.**

- **All members designed as samples in the report, are in Floors 23 – 25 and the rest are in the reinforcement table.**
- **For diagonals direction, upper is meant by the diagonals that starts from the upper floor on the right-hand side and then continues to the lower floor. The lower diagonals are the ones which start from the lower floor on the right-hand side and then continues to the upper floor.**

1. Design of Joints

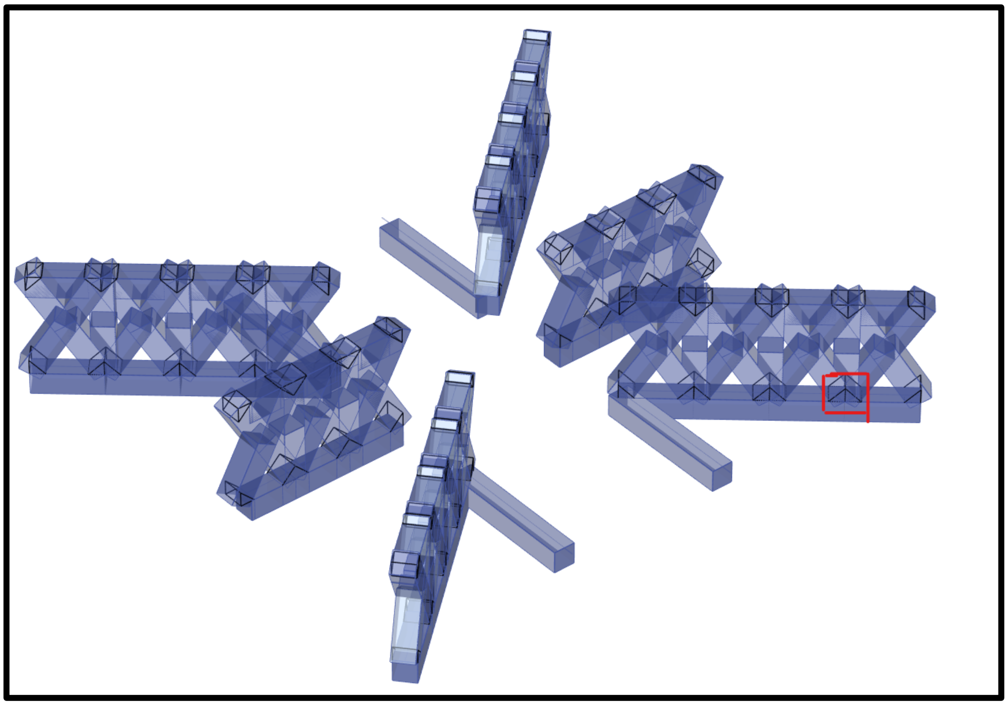


Figure 3.8.2 – Location of Outrigger Joint

The joints were designed as joints of special moment frames, according to ACI 318-19 code.

According to section 18.8.2.2 in ACI 318-19 Code:

Longitudinal reinforcement terminated in a joint shall extend to the far face of the joint core and shall be developed in tension in accordance with the following:

For bar sizes No. 10 through No. 36 terminating in a standard hook, ℓ_{dh} shall be calculated by Eq. (18.8.5.1), but ℓ_{dh} shall be at least the greater of $8d_b$ and 150 mm for normal weight concrete.

$$\ell_{dh} = \frac{1.25 f_y d_b}{5.4\lambda\sqrt{f'_c}} \qquad (18.8.5.1)$$

So, this equation shall be used when calculating the development length of longitudinal reinforcement for the members of the outrigger.

For bar sizes No. 10 through No. 36, $\boldsymbol{\ell d}$, the development length in tension for a straight bar, shall be 2.5 times ℓ_{dh} calculated in the equation above.

Straight bars terminated at a joint shall pass through the confined core of a column or a boundary element. Any portion of ℓd not within the confined core shall be increased by a factor of 1.6.

2. Design of Outrigger Beams

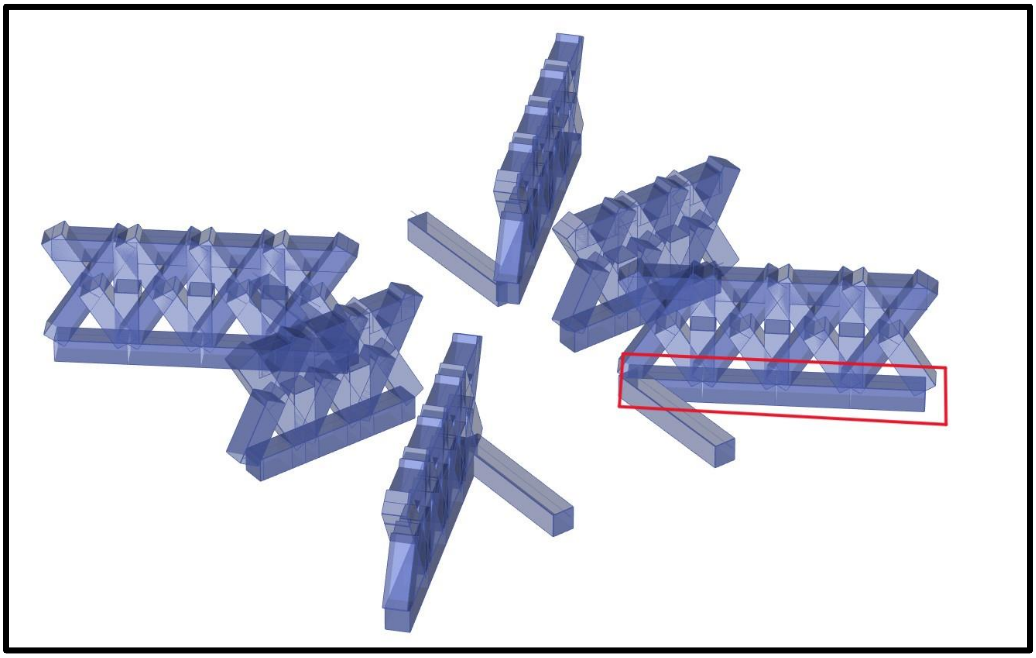


Figure 3.8.2 – Location of Outrigger Beam

- **Flexural Reinforcement for Outrigger Beams**

Outrigger Beam at 23rd Floor (BO1AA)

Thickness = 1500 mm

Width = 1500 mm

Cover = 75 mm

Design load combination:

(1.2 + 0.2Sds) (D + SD) + LL – 2.5EDX + Soil

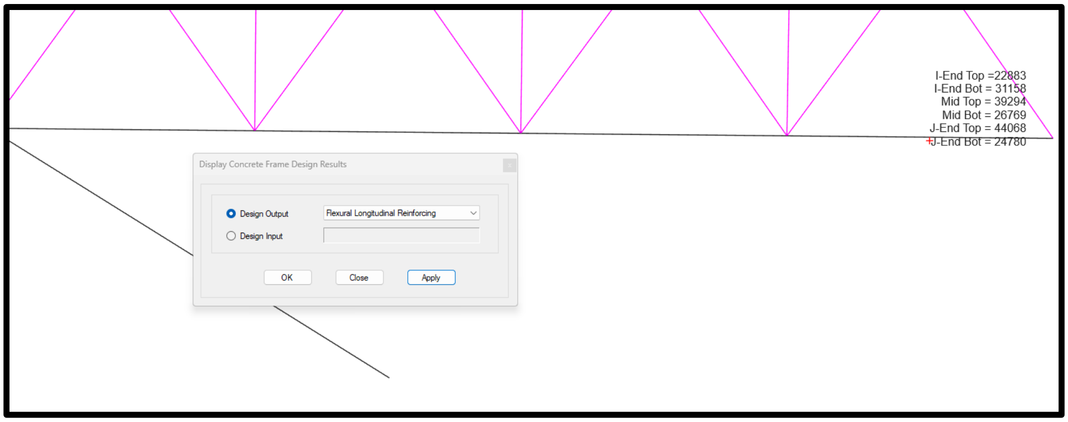


Figure 3.8.3 – Flexural Reinforcement for Outrigger Beam

Top west end steel = 22883 mm^2

Top east end steel = 44068 mm^2

Top middle steel = 39294 mm^2

Bottom west end steel = 31158 mm^2

Bottom east end steel = 24780 mm^2

Bottom middle steel = 26769 mm^2

For longitudinal flexural reinforcement, 2 layers of steel bars will be used. Thus, each steel reinforcement area will be divided by 2 to calculate reinforcement distribution for each layer.

$$\textbf{Top west end steel} = \frac{\mathbf{22883}}{\mathbf{2}} = \mathbf{11441.5\ mm^2\ per\ layer}$$

$$\text{Top east end steel} = \frac{44068}{2} = 22034\ \text{mm}^2\ \text{per layer}$$

$$\text{Top middle steel} = \frac{39294}{2} = 19647\ \text{mm}^2\ \text{per layer}$$

$$\text{Bottom west end steel} = \frac{31158}{2} = 15579\ \text{mm}^2\ \text{per layer}$$

$$\text{Bottom east end steel} = \frac{24780}{2} = 12390\ \text{mm}^2\ \text{per layer}$$

$$\text{Bottom middle steel} = \frac{26769}{2} = 13384.5\ \text{mm}^2\ \text{per layer}$$

$$10\emptyset 40 \text{ per layer at top west end with spacing of 150 mm} = 25133\ \text{mm}^2\ (\text{excess reinforcement})$$

12Ø50 per layer at top east end with spacing of 120 mm

$$= 47124 \text{ mm}^2 \text{ (excess reinforcement)}$$

11Ø50 per layer at top middle with spacing of 135 mm

$$= 43195 \text{ mm}^2 \text{ (excess reinforcement)}$$

10Ø50 per layer at bottom west end with spacing of 150 mm

$$= 39269 \text{ mm}^2 \text{ (excess reinforcement)}$$

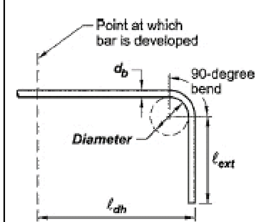
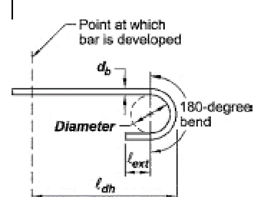
11Ø40 per layer at bottom east end with spacing of 135 mm

$$= 27645 \text{ mm}^2 \text{ (excess reinforcement)}$$

11Ø40 per layer at bottom middle with spacing of 135 mm

$$= 27645 \text{ mm}^2 \text{ (excess reinforcement)}$$

Development Lengths

Type of standard hook	Bar size	Minimum inside bend diameter, mm	Straight extension[1] ℓ_{ext}, mm	Type of standard hook
90-degree hook	No. 10 through No. 25	$6d_b$	$12d_b$	
	No. 29 through No. 36	$8d_b$		
	No. 43 and No. 57	10		
180-degree hook	No. 10 through No. 25	$6d_b$	Greater of $4d_b$ and 65 mm	
	No. 29 through No. 36	$8d_b$		
	No. 43 and No. 57	$10 d_b$		

$$\ell_{dh} = \frac{1.25 f_y d_b}{5.4 \lambda \sqrt{f'_c}}$$

$$\ell_{dh} = \frac{1.25 * 420 * 50}{5.4 * \sqrt{70}} = 585\ mm$$

$8d_b = 8 * 50 = 400\ \text{mm}$

$\iota_{ext} = 12d_b = 12 * 50 = 600\ \text{mm}$

Minimum inside bend diameter = $10d_b = 10 * 50 = \mathbf{500\ mm}$

$l_d = 2.5 * 465 = 1462.5\ mm, take\ it\ as\ 1465\ mm$

Lap splicing:

$l_{st} = 1.3l_d = 1.3 * 1465 = 1900\ \text{mm} > 300\ \text{mm}$

- **Shear and Torsion Reinforcement**

Shear Force and Reinforcement for Shear, V_{u2}

Shear V_{u2} kN	Shear ϕV_c kN	Shear ϕV_s kN	Shear V_p kN	Rebar A_v /s mm²/m
8159.6933	2210.6317	5949.0616	5158.6913	13455.57

$$\frac{Av}{s} = 13455.57 \frac{mm^2}{m} = 13.45557 \frac{mm^2}{mm}$$

Torsion Force and Torsion Reinforcement for Torsion, T_u

T_u kN-m	ϕT_{th} kN-m	ϕT_{cr} kN-m	Area A_o cm²	Perimeter, p_h mm	Rebar A_t /s mm²/m	Rebar A_l mm²
218.8036	0	0	16925.2	5644.4	208.33	15222

$$\frac{At}{s} = 208.33 \frac{mm^2}{m} * 2 = 416.66 \frac{mm^2}{m} = 0.41666 \frac{mm^2}{mm}$$

$$\frac{Av + t}{s} = 0.41666 + 13.45557 = \mathbf{13.87223} \frac{\mathbf{mm^2}}{\mathbf{mm}}$$

Use six legs stirrups of diameter 18 mm with spacing of 100 mm.

- **Torsion Longitudinal Reinforcement**

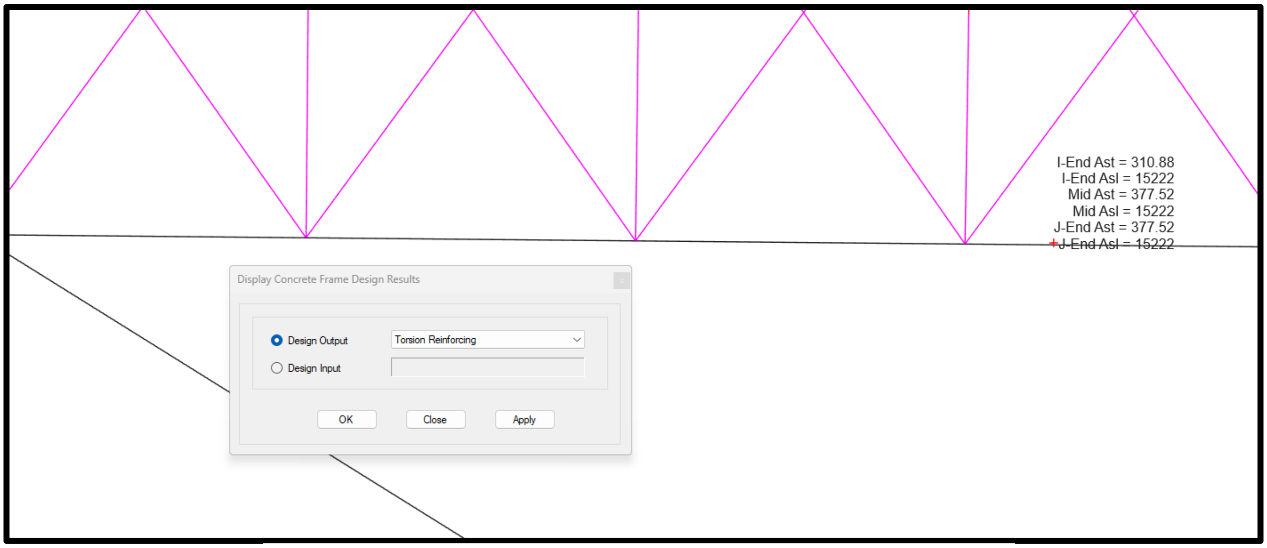


Figure 3.8.4 – Torsional Reinforcement for Outrigger Beam

Al for torsional reinforcement = 15222 mm^2. It must be divided into two layers, each of steel area = 7611 mm^2.

Use 10∅32 per layer spacing of 150 mm .

$$\ell_{dh} = \frac{1.25 f_y d_b}{5.4\lambda\sqrt{f'_c}}$$

$$\ell_{dh} = \frac{1.25 * 420 * 32}{5.4 * \sqrt{70}} = 375\ mm$$

$8d_b = 8 * 32 = 256$ mm

$\iota_{ext} = 12d_b = 12 * 32 = 385$ mm

Minimum inside bend diameter = $6d_b = 6 * 32 =$ **192 mm**

$l_d = 2.5 * 375 = 940\ mm$

Lap splicing:

$l_{st} = 1.3 l_d = 1.3 * 940 = 1220\ \text{mm} > 300\ \text{mm}$

Design Checks for Outrigger Beams

The positive moment strength at the face of the joint shall be at least one-third the negative moment strength provided at that face of the joint. Neither the negative nor the positive moment strength at any section along the length of the beam shall be less than one-fifth the maximum moment strength provided at the face of either joint.

Moment strength $\emptyset\mathrm{Mn} = 0.9 * \mathrm{A_s} * \mathrm{f_y} * \left(\mathrm{d} - \frac{\mathrm{A_s} * \mathrm{f_y}}{1.7\mathrm{f'}_\mathrm{c}\mathrm{b}}\right)$

At east end joint:

Area of steel at bottom = 27645 mm^2

Area of steel at top = 47124 mm^2

Positive ØMn = 14211.25 kN.m

Negative ØMn = 23408.25 kN.m

$\frac{23408.25}{3} = 7802.75\ \mathrm{kN.m}$ < Positive ØMn, Acceptable

At west end joint:

Area of steel at bottom = 39269 mm^2

Area of steel at top = 25133 mm^2

Positive ØMn = 19780.73 kN.m

Negative ØMn = 12976.08 kN.m

$\frac{12976.08}{3} = 4325.359\ \mathrm{kN.m}$ < Positive ØMn, Acceptable

Maximum moment strength is for A_s = 47124 mm^2

ØMn = 23408.25 kN.m

$$\frac{23408.25}{5} = 4681.65\ \mathrm{kN.m}$$

Minimum moment strength along the span is for A_s = 25133 mm^2

ØMn = 12976.08 kN.m > 4681.65 kN.m   Acceptable.

ϕVn shall be at least the lesser of (a) and (b):

a) The sum of the shear associated with development of nominal moment strengths of the beam at each restrained end of the clear span due to reverse curvature bending and the shear calculated for factored gravity and vertical earthquake loads.
b) The maximum shear obtained from design load combinations that include *E*, with *E* taken as twice that prescribed by the general building code.

$ØV_s = 0.75*f_{yt}*d*Av/s$

$Av/s = 15.268\ mm^2/mm$

$V_s = 0.75*420*1425*15.268 = 6853423\ N = 6853.423\ kN$

$ØV_n = ØV_c + ØV_s = 2210.63 + 6853.423 = 9064.053\ kN$

V_u from design load combination = 8159.6933 kN

$ØV_n > V_u$, Acceptable

3. Outrigger Columns Design

Outrigger Column at 23rd Floor CO1AA

Section: 1500 mm * 1500 mm

Cover: 75 mm

Design load combination:

(1.2 + 0.2Sds) (D + SD) + LL – 2.5EDX + Soil

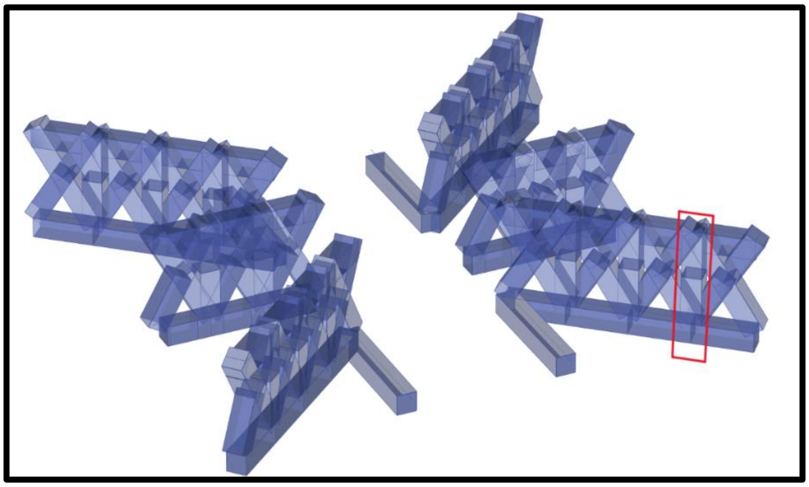
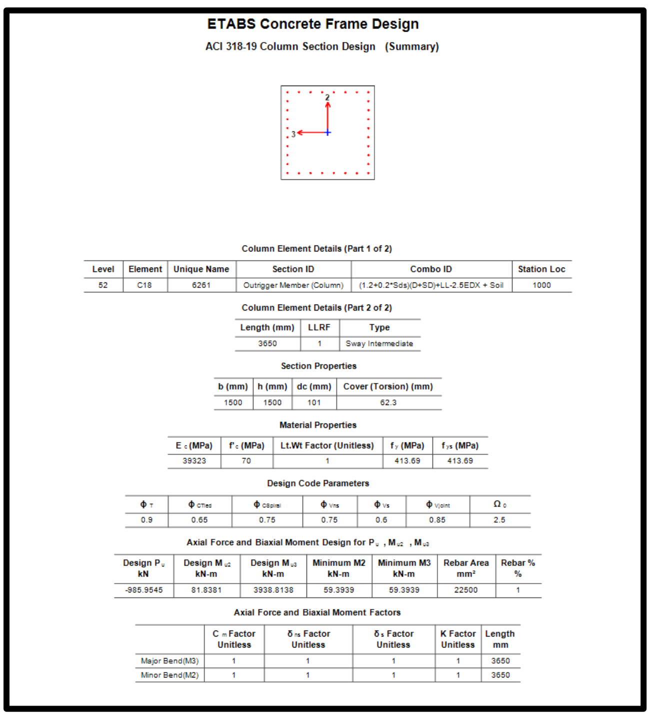


Figure 3.8.5 – Location of Outrigger Column



ETABS Concrete Frame Design

ACI 318-19 Column Section Design (Summary)

Column Element Details (Part 1 of 2)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc
52	C18	6261	Outrigger Member (Column)	(1.2+0.2*Sds)(D+SD)+LL-2.5EDX + Soil	1000

Column Element Details (Part 2 of 2)

Length (mm)	LLRF	Type
3650	1	Sway Intermediate

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
1500	1500	101	62.3

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
39323	70	1	413.69	413.69

Design Code Parameters

Φ_T	Φ_{CTied}	$\Phi_{CSpiral}$	Φ_{Vns}	Φ_{Vs}	Φ_{VJoint}	Ω_0
0.9	0.65	0.75	0.75	0.6	0.85	2.5

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 M2 kN-m	Minimum M3 kN-m	Rebar Area mm²	Rebar % %
-985.9545	81.8381	3938.8138	59.3939	59.3939	22500	1

Axial Force and Biaxial Moment Factors

	C_m Factor Unitless	δ_{ns} Factor Unitless	δ_s Factor Unitless	K Factor Unitless	Length mm
Major Bend(M3)	1	1	1	1	3650
Minor Bend(M2)	1	1	1	1	3650

Rebar area = 22500 mm^2

- Use **36Ø32 longitidual reinforcement** , As = 28953 mm^2 with spacing of 150 mm.

Interaction diagram (P – M2) used for outrigger column design is as follows.

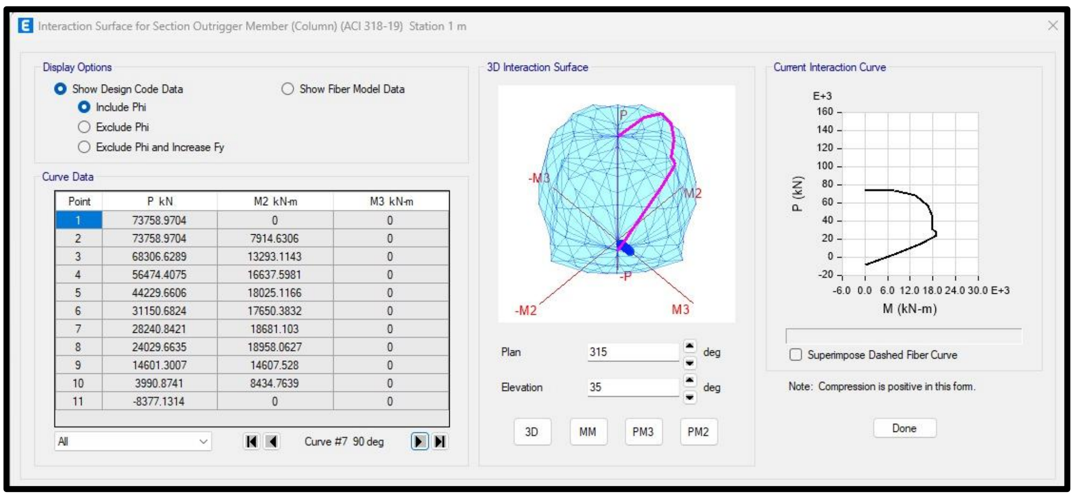


Figure 3.8.6 – Outrigger Column P-M2 Interaction Diagram

Interaction diagram (P-M3) used for outrigger column design is as follows.

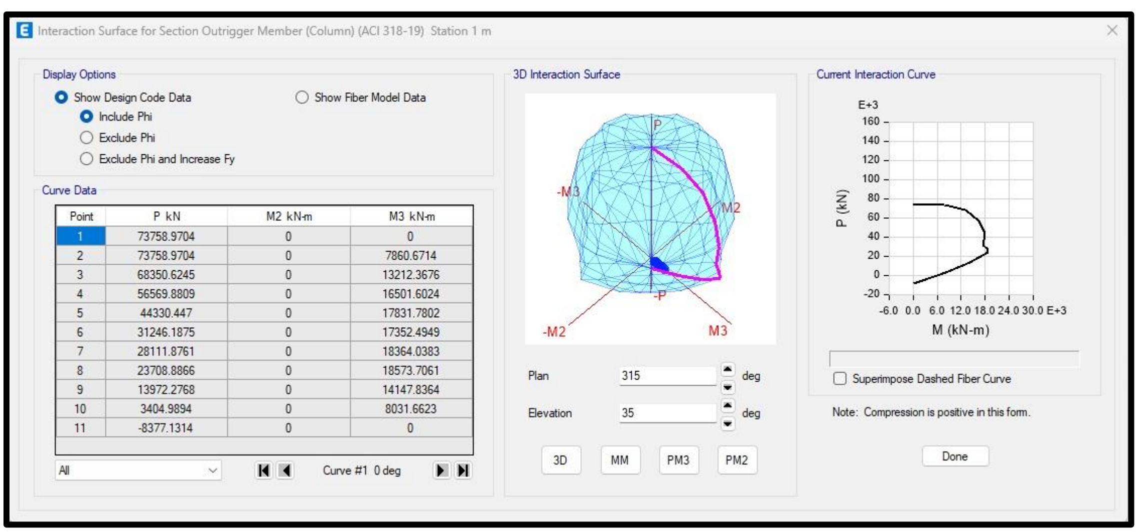


Figure 3.8.7 – Outrigger Column P-M3 Interaction Diagram

Development Lengths

$$\ell_{dh} = \frac{1.25 f_y d_b}{5.4 \lambda \sqrt{f'_c}}$$

$$\ell_{dh} = \frac{1.25 * 420 * 32}{5.4 * \sqrt{70}} = 375\ mm$$

$8d_b = 8 * 32 = 256$ mm

$\iota_{ext} = 12d_b = 12 * 32 = 384$ mm, take it as 385 mm

Minimum inside bend diameter = $8d_b = 8 * 32 = \mathbf{256\ mm}$

$l_d = 2.5 * 375 = 940\ mm$

Lap splicing:

$l_{st} = 1.3l_d = 1.3 * 940 = 1220$ mm > 300 mm

Confinement spacing (So) and confinement length (lo) are to be determined.

So shall not exceed the smallest of:

- The smaller of $8d_b$ of the smallest longitudinal bar enclosed and 200 mm.
- One-half the smallest dimension of the column's cross section.

$8d_b = 8 * 40 = 320$ mm

0.5*1500 = 750 mm

Thus, take Confinement spacing (So) **= 200 mm**

Lo shall not be less than the largest of:

- One-sixth the clear span of the column
- Maximum cross-sectional dimension of the column
- 450 mm

Lo = 1500 mm

- **Shear Reinforcement for Outrigger Column**

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

	Shear V_u kN	Shear ϕV_c kN	Shear ϕV_s kN	Shear ϕV_p kN	Rebar A_v/s mm²/m
Major, V_{u2}	1441.928	2055.3505	813.8611	0	1875
Minor, V_{u3}	83.2771	727.333	0	0	0

Joint Shear Check/Design

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

(6/5) Beam/Column Capacity Ratio

Major Ratio	Minor Ratio
N/N	N/N

Notes:

N/A: Not Applicable

N/C: Not Calculated

N/N: Not Needed

Av/s = 1875 mm²/m = 1.875 mm²/mm

Av/s min is as follows:

f'c = 70 MPa

fyt = 420 MPa

Take the greater of:

- 0.001235bw
- 0.0008333bw

Take 0.001235bw for all cases

$$Av/smin = 0.001235 * 1500 = \mathbf{1.8525\ mm^2/mm}$$

Use six legs stirrups with diameter 10 mm with spacing of 250 mm.

Design Checks for Outrigger Column

ϕVn shall be at least the lesser of (a) and (b):

a) The shear associated with development of nominal moment strengths of the column at each restrained end of the unsupported length due to reverse curvature bending. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the highest flexural strength.
b) The maximum shear obtained from factored load combinations that include *E*, with $\Omega_0 E$ substituted for *E*.

$ØV_s = 0.75*f_{yt}*d*Av/s$

$Av/s = 1.885\ mm^2/mm$

$V_s = 0.75*420*1425*1.885 = 846129\ N = 846.129\ kN$

$ØV_n = ØV_c + ØV_s = 2055.35 + 846.129 = 2901.479\ kN$

V_u from design load combination = 1441.928 kN

$ØV_n > V_u$, Acceptable

4. Diagonal Outrigger Members Design

Diagonal Outrigger Members at 23rd Floor (DO1AA and DO1AB)

Section: 1500 mm * 1500 mm

Cover: 75 mm

Design load combination:

(1.2 + 0.2Sds) (D + SD) + LL – 2.5EDX + Soil

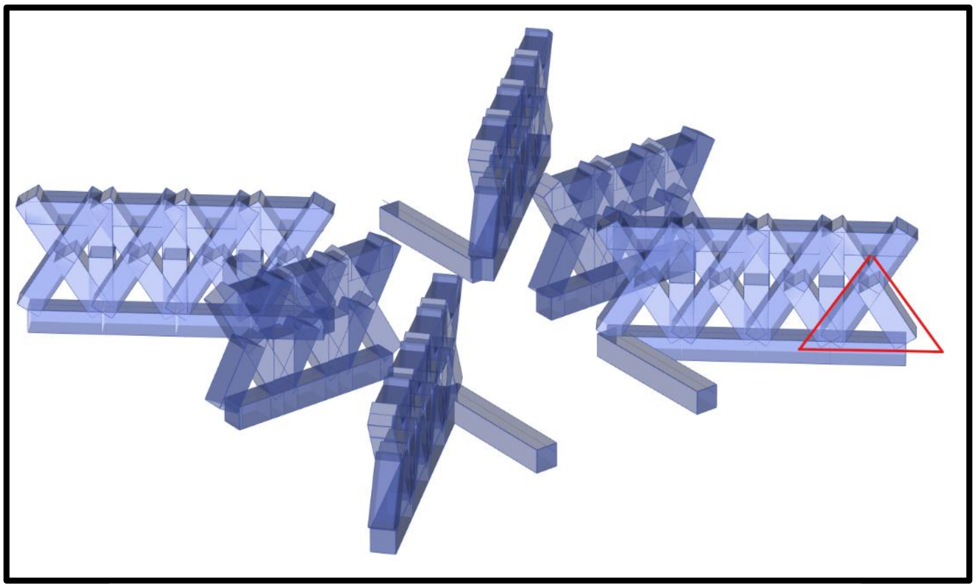


Figure 3.8.8 – Location of Outrigger Diagonal Member

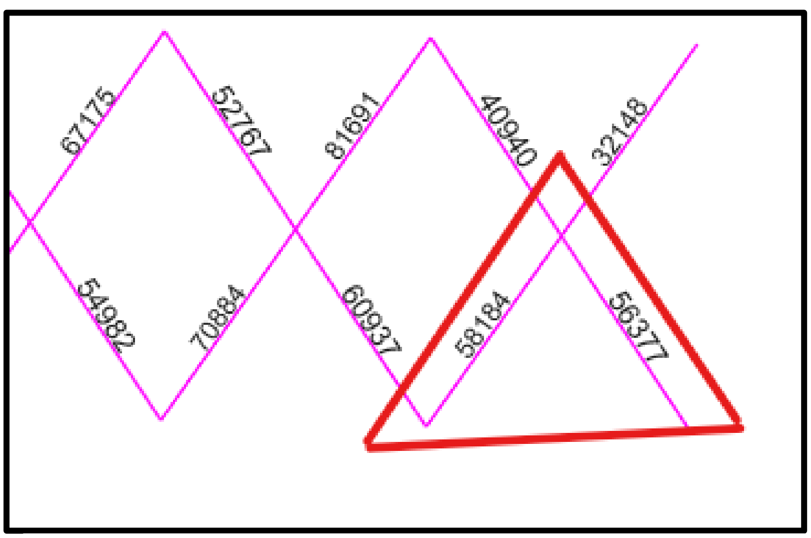
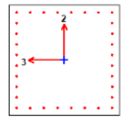


Figure 3.8.9 – Reinforcement of Outrigger Diagonal Member

ACI 318-19 Column Section Design (Summary)



Column Element Details (Part 1 of 2)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)
24	D62	6265	Outrigger Member (Diadonal)	(0.9-0.2Sds)(D+SD)-2.5EDX	725.4	4309.3

Column Element Details (Part 2 of 2)

LLRF	Type
1	Sway Special

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
1500	1500	101	62.3

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
39323	70	1	413.69	413.69

Design Code Parameters

Φ_T	Φ_{CTied}	$\Phi_{CSpiral}$	Φ_{Vns}	Φ_{Vs}	Φ_{Vjoint}	Ω_0
0.9	0.65	0.75	0.75	0.6	0.85	2.5

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 M2 kN-m	Minimum M3 kN-m	Rebar Area mm²	Rebar % %
-7996.325	-481.6986	-7910.2707	481.6986	481.6986	56377	2.51

Axial Force and Biaxial Moment Factors

	C_m Factor Unitless	δ_{ns} Factor Unitless	δ_s Factor Unitless	K Factor Unitless	Length mm
Major Bend(M3)	1	1	1	1	4309.3
Minor Bend(M2)	1	1	1	1	4309.3

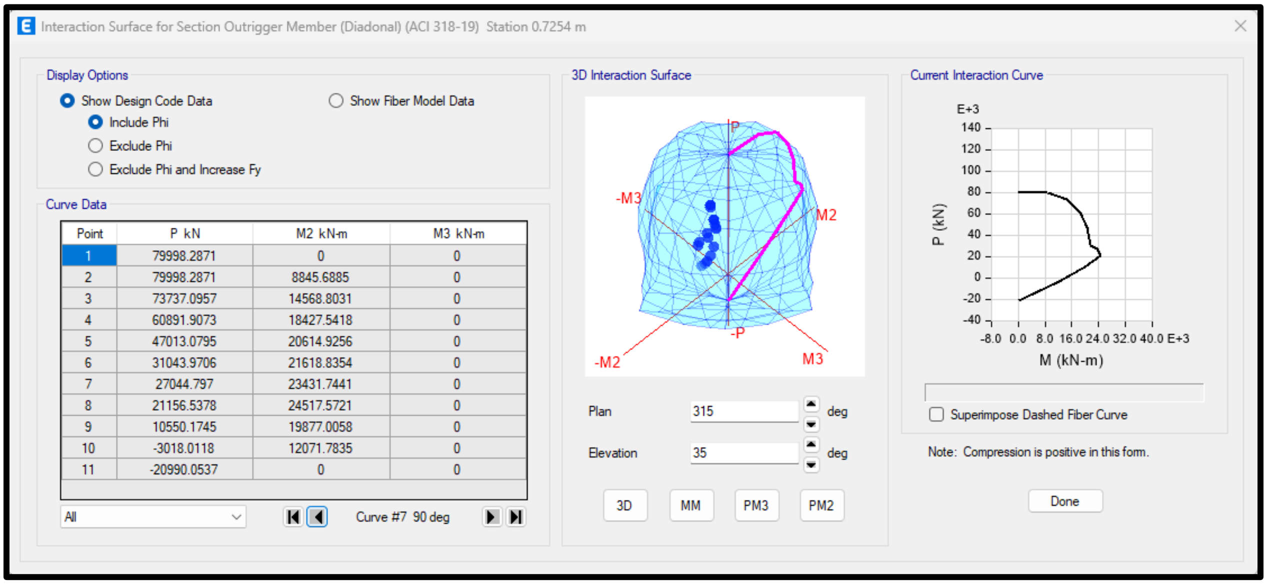
Upper diagonal member area of steel = 56377 mm^2

- **Use 36 Ø 50, with area of steel = 70684 mm^2, with spacing of 150 mm.**

Lower diagonal member area of steel = 58184 mm^2

- **Use 36 Ø 50, with area of steel = 70684 mm^2, with spacing of 150 mm.**

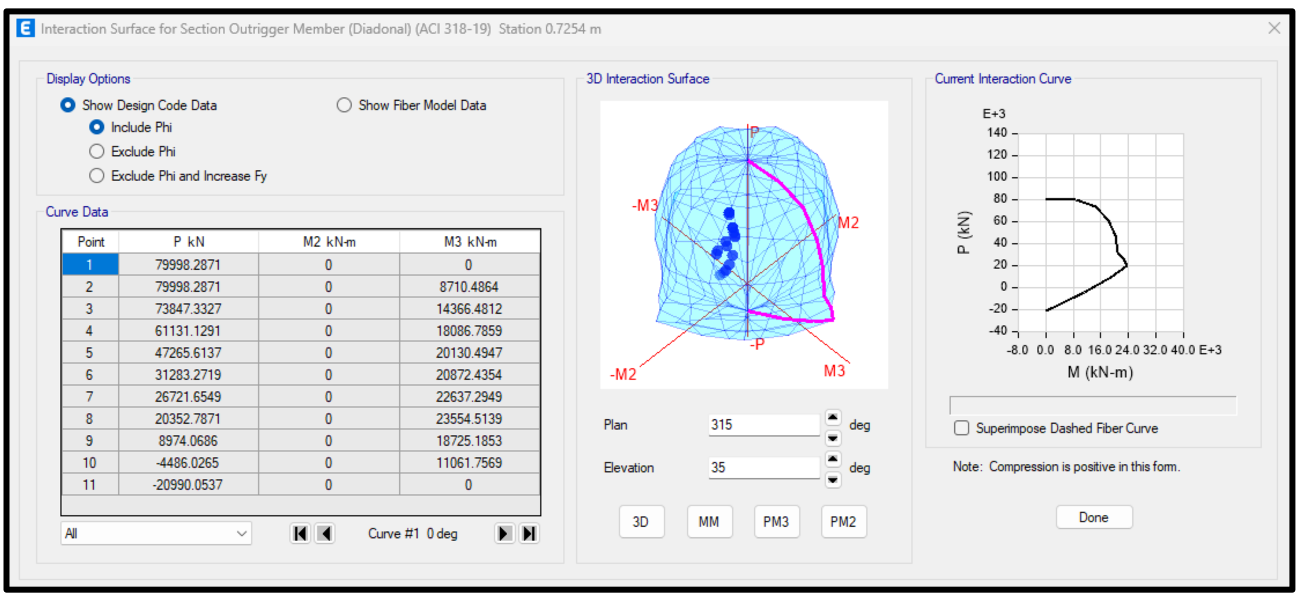
Lower diagonal has the below interaction diagram (P-M2)



Point	P kN	M2 kN-m	M3 kN-m
1	79998.2871	0	0
2	79998.2871	8845.6885	0
3	73737.0957	14568.8031	0
4	60891.9073	18427.5418	0
5	47013.0795	20614.9256	0
6	31043.9706	21618.8354	0
7	27044.797	23431.7441	0
8	21156.5378	24517.5721	0
9	10550.1745	19877.0058	0
10	-3018.0118	12071.7835	0
11	-20990.0537	0	0

Figure 3.8.10 – Outrigger Diagonal Member P-M2 Interaction Diagram

Upper diagonal has the below interaction diagram (P-M3).



Point	P kN	M2 kN-m	M3 kN-m
1	79998.2871	0	0
2	79998.2871	0	8710.4864
3	73847.3327	0	14366.4812
4	61131.1291	0	18086.7859
5	47265.6137	0	20130.4947
6	31283.2719	0	20872.4354
7	26721.6549	0	22637.2949
8	20352.7871	0	23554.5139
9	8974.0686	0	18725.1853
10	-4486.0265	0	11061.7569
11	-20990.0537	0	0

Figure 3.8.11 – Outrigger Diagonal Member P-M3 Interaction Diagram

Development of Hooks

It will be calculated for both members.

$$\ell_{dh} = \frac{1.25 * f_y d_b}{5.4\lambda\sqrt{f'_c}}$$

$$\ell_{dh} = \frac{420 * 50}{5.4 * \sqrt{70}} = 585\ mm$$

$8d_b = 8 * 50 = 400$ mm

$\iota_{ext} = 12d_b = 12 * 50 = 600$ mm

Minimum inside bend diameter = $10d_b = 10 * 50 = \mathbf{500\ mm}$

$l_d = 2.5 * 585 = 1462.5\ mm, take\ it\ as\ 1465\ mm$

Lap splicing:

$l_{st} = 1.3l_d = 1.3 * 1465\ = 1900\ \ mm > 300\ mm$

$L_d = 47 * d_b = 47 * 50 = \mathbf{2350\ mm}$

$Lu = 4309.3 - 1500 = \mathbf{2809.3\ mm}$

1.25 *L_d = 1.25*2350 = 2937.50 mm

Lu/2 = 2809.3/2 = 1404.65 mm

1.25 L_d > Lu/2

Pu from the above diagram = -7995.325 KN

0.3Agf'c = $0.3 * \frac{1500*1500*70}{1000} = \mathbf{47250\ kN > 7995.325\ kN}$

- **Shear Reinforcement for Diagonals**

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

	Shear V_u kN	Shear ϕV_c kN	Shear ϕV_s kN	Shear ϕV_p kN	Rebar A_v /s mm²/m
Major, V_{u2}	3997.7847	990.4469	3007.3378	0	8660.5
Minor, V_{u3}	75.2089	169.4186	0	0	0

Joint Shear Check/Design

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

(6/5) Beam/Column Capacity Ratio

Major Ratio	Minor Ratio
N/N	N/N

L_o of hoops is the greater of

- Depth of columns = 1500 mm
- One sixth the clear height of the column
- 450 mm

L_o = 1500 mm

So is the smallest of:

- One-fourth the minimum column dimension = 1500/4 = 375 mm
- $6d_b$ = 6*50 = 300 mm
- $S_o = 100 + \left(\frac{350-\text{hx}}{3}\right) = 100 + \left(\frac{350-150}{3}\right) = 166.66\text{ mm} > 156\text{ mm}$

So, the spacing is 100 mm

Table 18.7.5.4—Transverse reinforcement for columns of special moment frames

Transverse reinforcement	Conditions	Applicable expressions	
A_{sh}/sb_c for rectilinear hoop	$P_u \le 0.3A_g f_c'$ and $f_c' \le 70$ MPa	Greater of (a) and (b)	$0.3\left(\frac{A_g}{A_{ch}}-1\right)\frac{f_c'}{f_{yt}}$ (a)
	$P_u > 0.3A_g f_c'$ or $f_c' > 70$ MPa	Greatest of (a), (b), and (c)	$0.09\frac{f_c'}{f_{yt}}$ (b)
			$0.2k_f k_n \frac{P_u}{f_{yt}A_{ch}}$ (c)
ρ_s for spiral or circular hoop	$P_u \le 0.3A_g f_c'$ and $f_c' \le 70$ MPa	Greater of (d) and (e)	$0.45\left(\frac{A_g}{A_{ch}}-1\right)\frac{f_c'}{f_{yt}}$ (d)
	$P_u > 0.3A_g f_c'$ or $f_c' > 70$ MPa	Greatest of (d), (e), and (f)	$0.12\frac{f_c'}{f_{yt}}$ (e)
			$0.35k_f \frac{P_u}{f_{yt}A_{ch}}$ (f)

$$0.3\left(\frac{\mathrm{Ag}}{\mathrm{Ach}}-1\right)*\frac{\mathrm{f'c}}{\mathrm{fyt}}=0.3*\left(\frac{1500*1500}{1350*1350}-1\right)*\frac{70}{420}=0.04$$

$$0.09*\frac{70}{420}=0.015$$

A_{sh}/S_{bc} = 0.04

$$A_{sh} = 100*1350*0.04 = 5400\ mm^2$$

Use 10 legs, 5400/10 = 540 mm^2

Use Ø28 stirrups with spacing of 100 mm

Design Checks for Diagonal Outrigger Members

Dimensional Limits:

Columns shall satisfy (a) and (b):

a) The shortest cross-sectional dimension, measured on a straight line passing through the geometric centroid, shall be at least 300 mm.
b) The ratio of the shortest cross-sectional dimension to the perpendicular dimension shall be at least 0.4

The diagonal outrigger members have square cross sections of dimensions 1500 mm*1500 mm, so both conditions mentioned above are satisfied.

ACI 318-19 Section 18.7.6.1.1:

The design shear force V_e shall be calculated from considering the maximum forces that can be generated at the faces of the joints at each end of the column. These joint forces shall be calculated using the maximum probable flexural strengths, M_{pr}, at each end of the column associated with the range of factored axial forces, P_u, acting on the column. The column shears need not exceed those calculated from joint strengths based on M_{pr} of the beams framing into the joint. In no case shall V_e be less than the factored shear calculated by analysis of the structure.

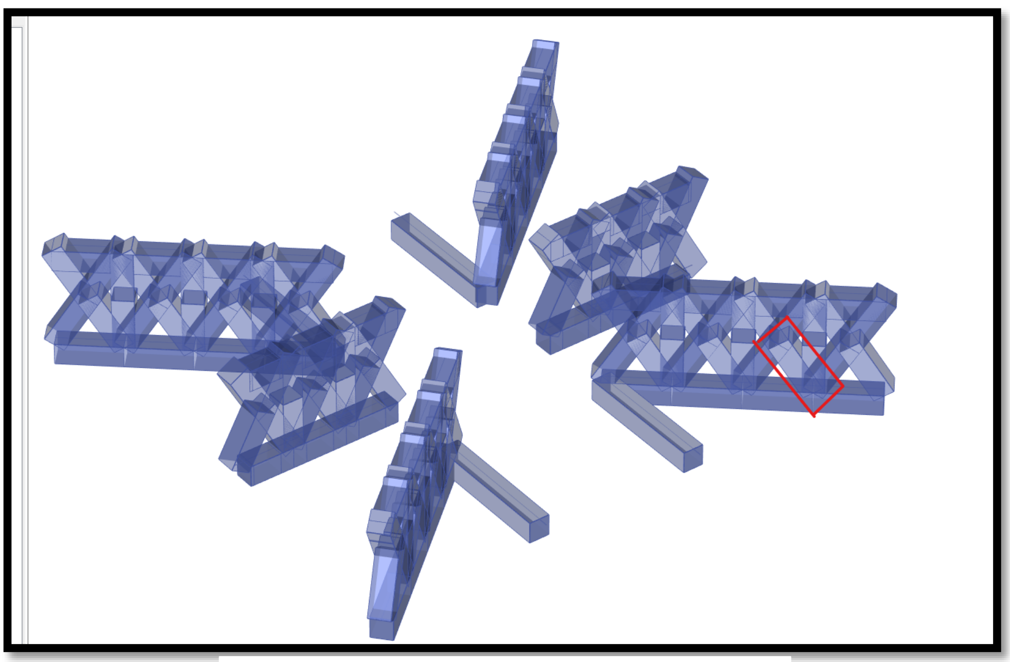


Figure 3.8.12 – Diagonal Member to Check

Checking at the joint shown in the figure below

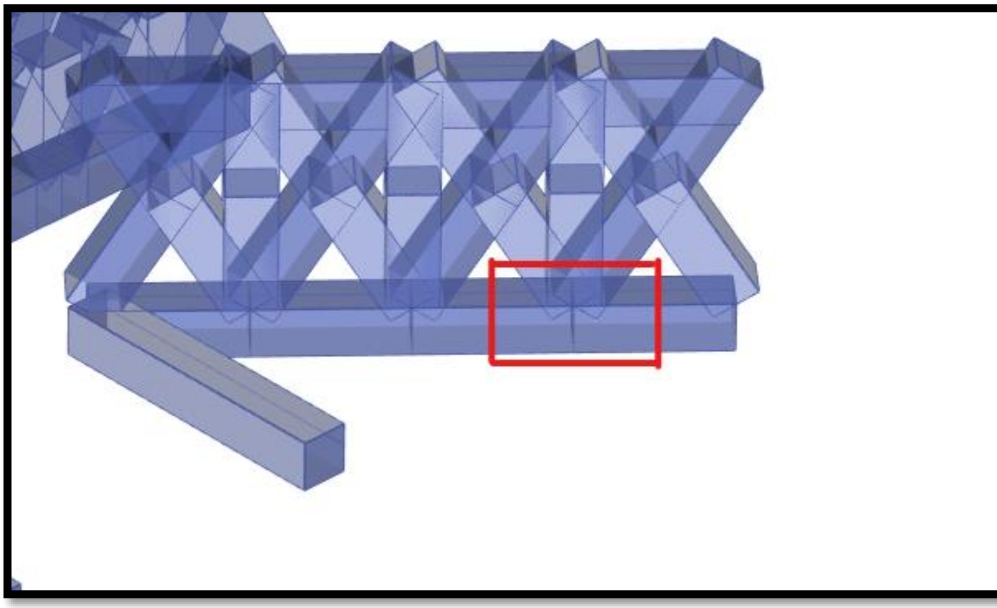


Figure 3.8.13 – Joint to Check Magnified

$$M_{pr} = A_s * 1.25f_y * \left(d - \frac{A_s * 1.25f_y}{1.7f'_c b}\right)$$

For the first span of the beam:

Top Steel = 25133 mm^2

M_{pr} = 14417.8 kN.m

Bottom Steel = 39269 mm^2

M_{pr} = 21978.6 kN.m

For the second span of the beam:

Top Steel = 10618 mm^2

M_{pr} = 6243.45 kN.m

Bottom Steel = 10618 mm^2

M_{pr} = 6243.45 kN.m

Direction 1, sum of beam moments M = 14417.8 + 6243.45 = 20661.32 kN.m

Direction 2, sum of beam moments M = 21978.6 + 6243.45 = 28222 kN.m

$$V_e = \frac{\sum M_{pr,beams}}{h_{diagonal}} = \frac{28222}{4.3094} = 6549 \text{ kN}$$

ACI 318-19 Section 18.7.6.2.1:

Transverse reinforcement over the lengths ℓ_o, given in 18.7.5.1, shall be designed to resist shear assuming $V_c = 0$ when both (a) and (b) occur:

a) The earthquake-induced shear force, calculated in accordance with 18.7.6.1, is at least one-half of the maximum required shear strength within ℓ_o.

b) The factored axial compressive force P_u including earthquake effects is less than $A_g f_c'/20$.

$V_e = 6549$ kN

$V_u = 1107.7781$ kN

So, V_e is larger than $V_u/2$ so point (a) is achieved

$A_g f'_c/20 = 1500*1500*70/20 = 7875000$ N $= 7875$ kN

The factored axial compressive force P_u including earthquake effects = 17291.97368 kN

$P_u > A_g f'_c/20$

So, point (b) is not achieved

So, V_c should not be considered zero

ACI 318-19 Section 21.2.4.1:

For any member designed to resist *E*, ϕ for shear shall be 0.60 if the nominal shear strength of the member is less than the shear corresponding to the development of the nominal moment strength of the member. The nominal moment strength shall be the maximum value calculated considering factored axial loads from load combinations that include *E*.

$V_n = V_c + V_s$

$V_c = 123.4952kN$

$V_s = f_{yt}*d*Av/s$

$Av/s = 3.078\ mm^2/mm$

$V_s = 420*1425*3.078/1000 = 1842.18\ kN$

$V_n = 123.4952 + 1842.18 = 1965.67\ kN$

The interaction diagram of this diagonal member is shown below

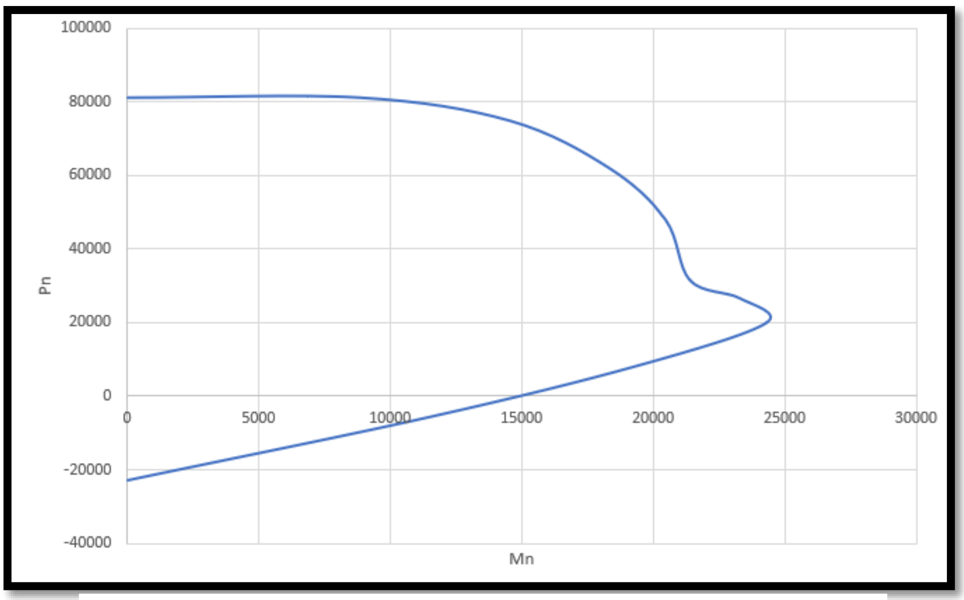


Figure 3.8.13 – Outrigger Diagonal Member Interaction Diagram

$P_n = 17291.97368\ kN$

$M_n = 23500\ kN.m$

$$V_e = \frac{M_{n,\text{bottom considering axial force}} + M_{n,\text{top considering axial force}}}{h_{\text{diagonal}}}$$

$$V_e = \frac{23500 + 23500}{4.3094} = 10906.4\ kN$$

$V_e > V_n$

So, Ø is considered to be 0.6

3.9 Design of Slabs

3.9.1 General Notes for Slabs Reinforcement

1) Minimum spacing of reinforcement bars shall be the greatest of:
   - 25 mm
   - d_b

2) Minimum steel reinforcement shall be determined by the following equation:

   $$A_{s\,min} = 0.0018 \times b \times h$$

   Where:

   b is the section width
   h is the slab thickness

3) Maximum reinforcement steel ratio shall be determined by the following equation:

   $$\rho_{max} = 0.375\beta_1 \frac{0.85f'_c}{f_y}$$

   $f'_c = 70\ MPa$
   $f_y = 420\ MPa$
   $\beta_1 = 0.65\ for\ f'_c \geq 55\ MPa$

   $$\rho_{max} = 0.375 * 0.65 \frac{0.85 * 70}{420} = 0.0345$$

3.9.2 Slabs Reinforcement Details

1. Parking Slab at 6th Basement Floor to 1st Basement Floor

X-Direction:

Mid-Span

For Top Reinforcement, use Φ10/250 mm

For Bottom Reinforcement, use Φ16/150 mm

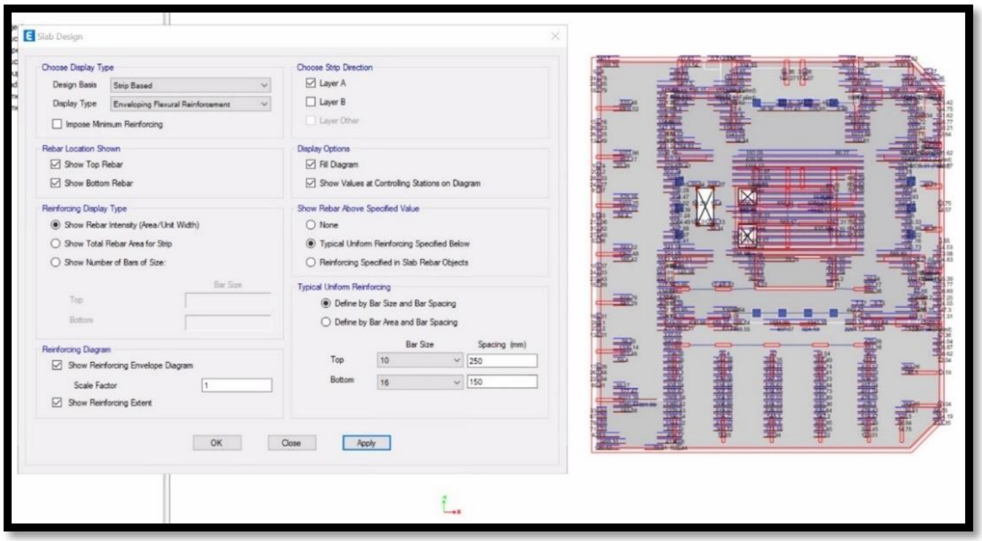


Figure 3.9.2.1 – Mid-Span X-Direction Reinforcement for 4th Basement Floor

End-Spans

For Top Reinforcement, use Φ16/150 mm

For Bottom Reinforcement, use Φ10/250 mm

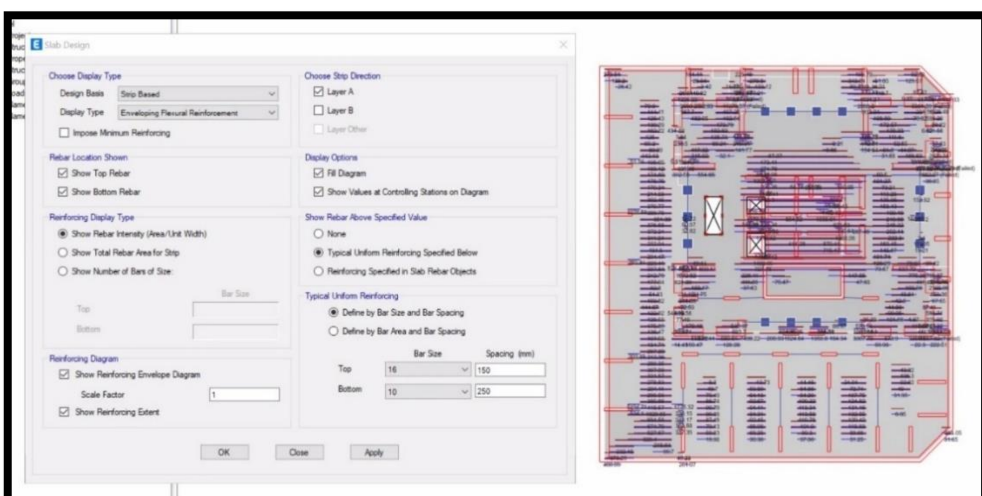


Figure 3.9.2.2 – End-Spans X-Direction Reinforcement for 4th Basement Floor

Shear Reinforcement

No need for shear reinforcement

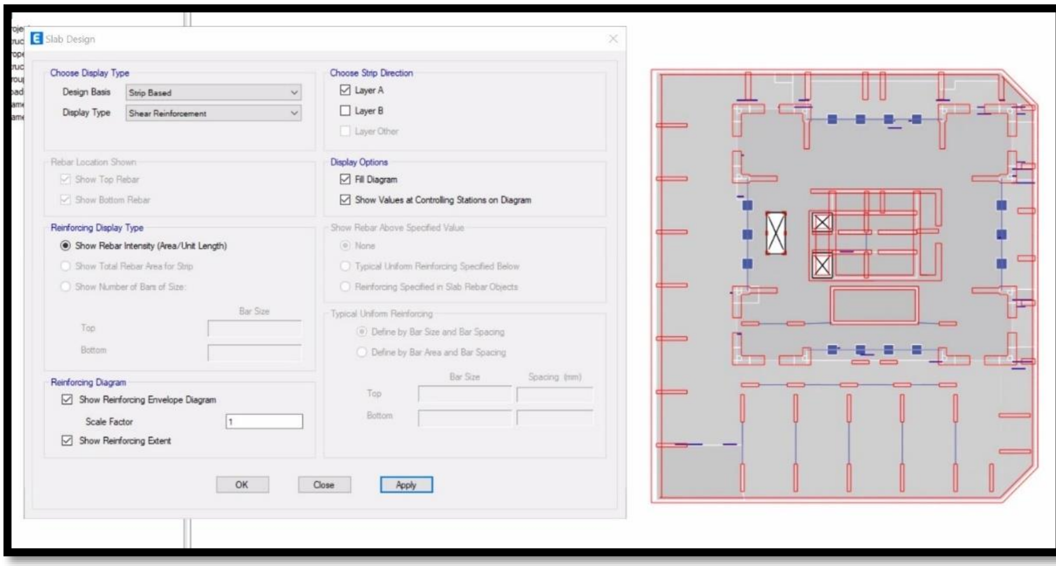


Figure 3.9.2.3 – Shear Reinforcement in X-Direction for 4th Basement Floor

Y-Direction

Mid-Span

For Top Reinforcement, use Φ10/250 mm

For Bottom Reinforcement, use Φ16/150 mm.

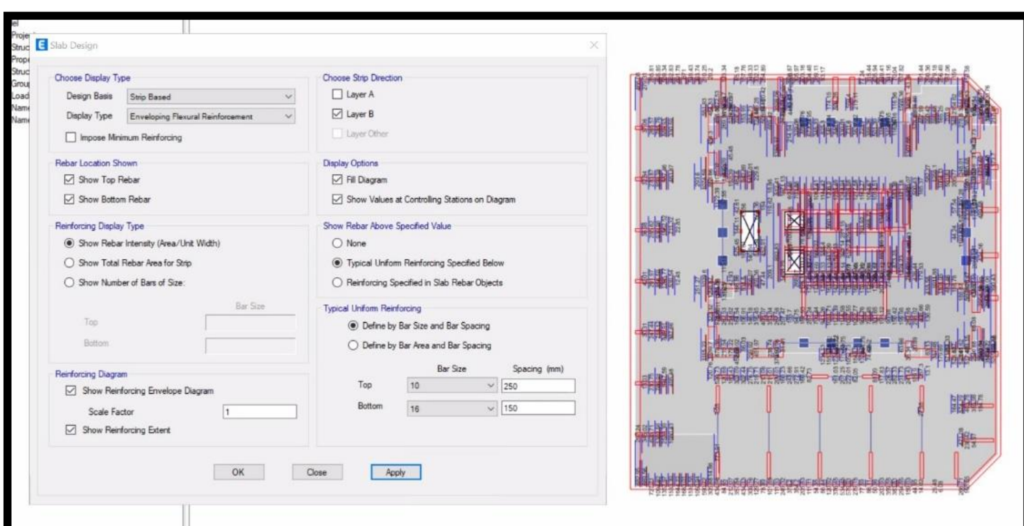


Figure 3.9.2.4 – Mid-Span Y-Direction Reinforcement for 4th Basement Floor

End-Spans

For Top Reinforcement, use Φ16/150 mm

For Bottom Reinforcement, use Φ10/250 mm.

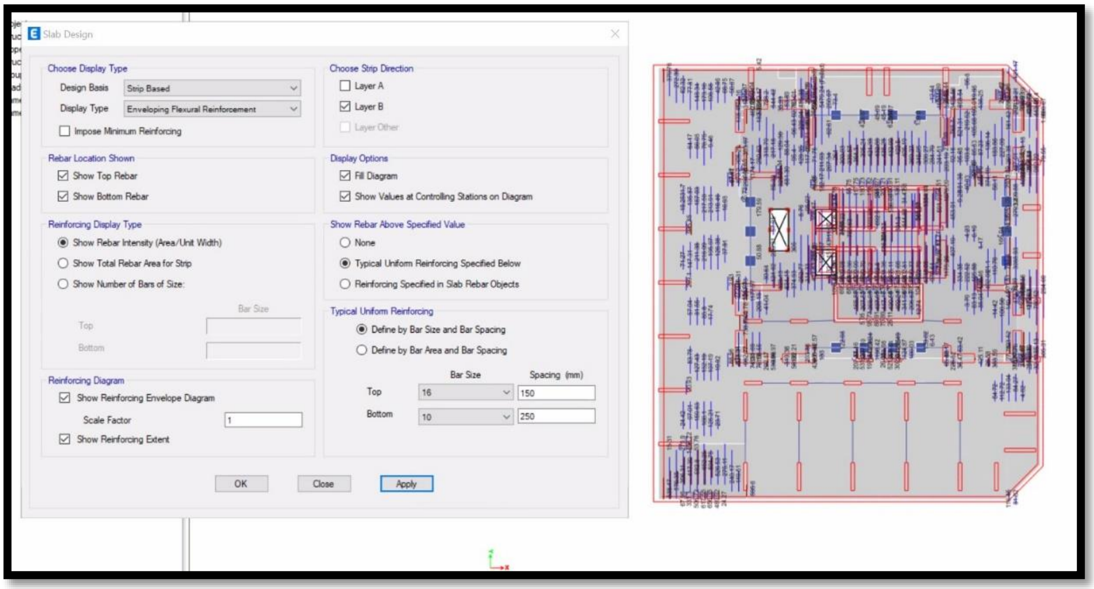


Figure 3.9.2.5 – End-Spans Y-Direction Reinforcement for 4th Basement Floor

Shear Reinforcement

No need for shear reinforcement

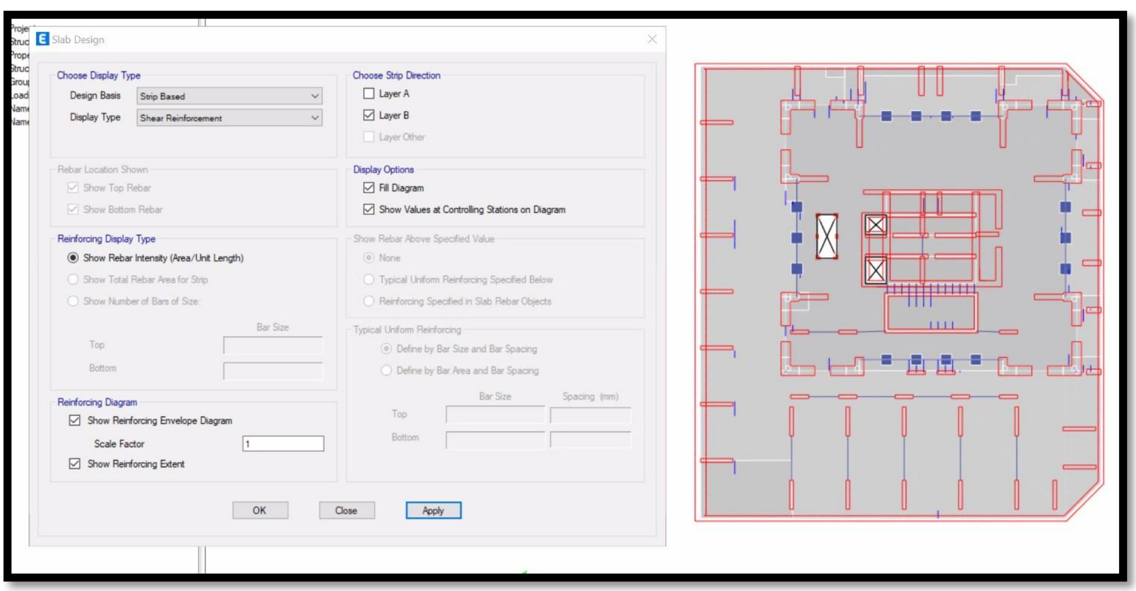


Figure 3.9.2.6 – Shear Reinforcement in Y-Direction for 4th Basement Floor

Core Reinforcement for both directions

For Top Reinforcement, use Φ20/150 mm

For Bottom Reinforcement, use Φ20/150 mm.

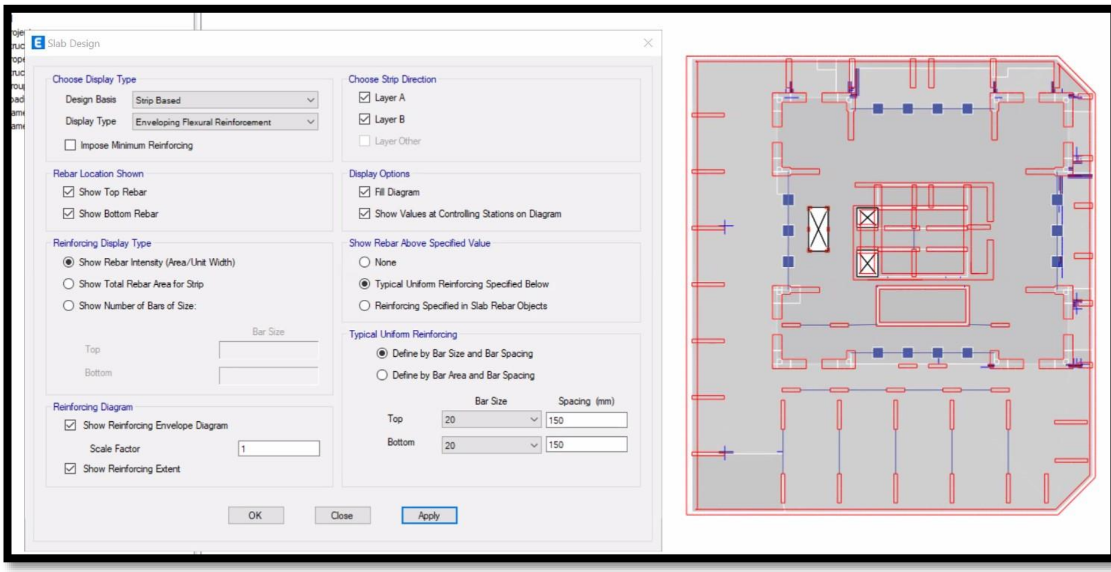


Figure 3.9.2.7 – Core Reinforcement for Both Directions for 4th Basement Floor

2. Parking Slab at Ground Floor to 5th Floor

X-Direction:

Bottom reinforcement is required for the entirety slab and top reinforcement is required for a significant portion of the slab due to different moment signs from different combinations

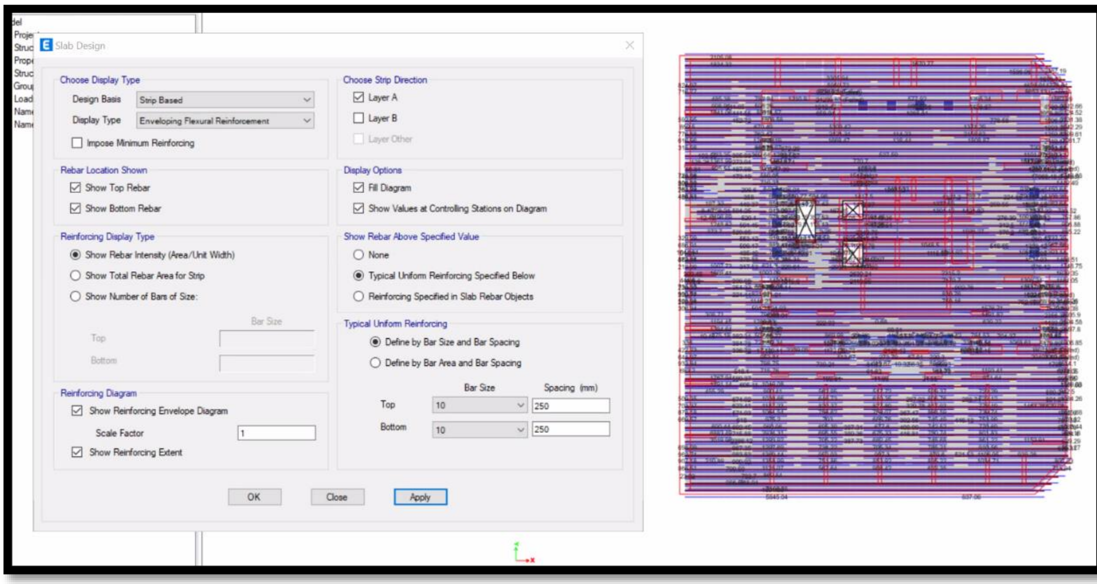


Figure 3.9.2.8 – Reinforcement Regions in X-Direction for Ground Floor

Bottom Reinforcement for the whole slab in X direction

Use Φ20/150 mm

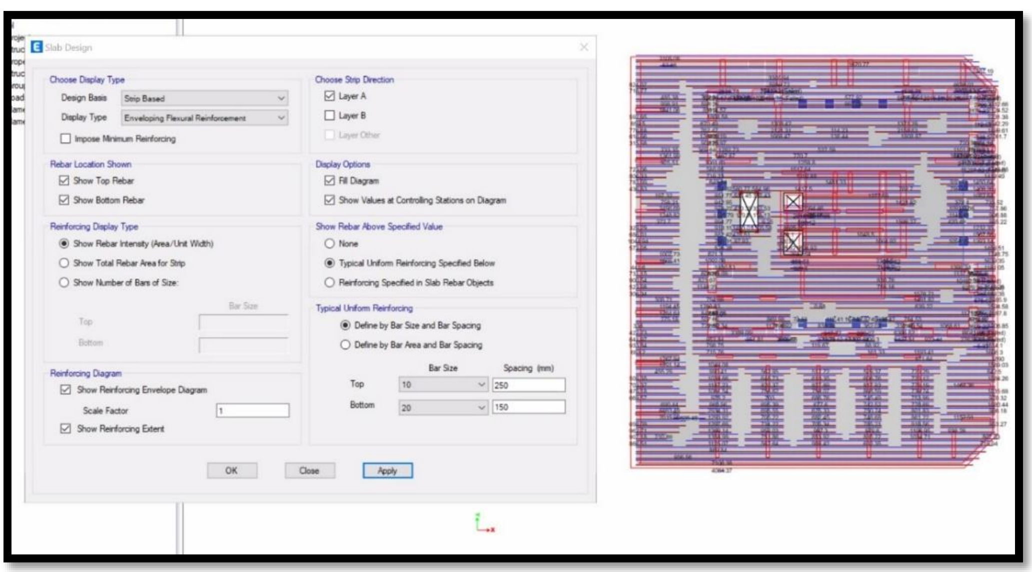


Figure 3.9.2.9 – Bottom Reinforcement in X-Direction for Ground Floor

Top Reinforcement for the whole slab in X direction

Use Φ16/150 mm

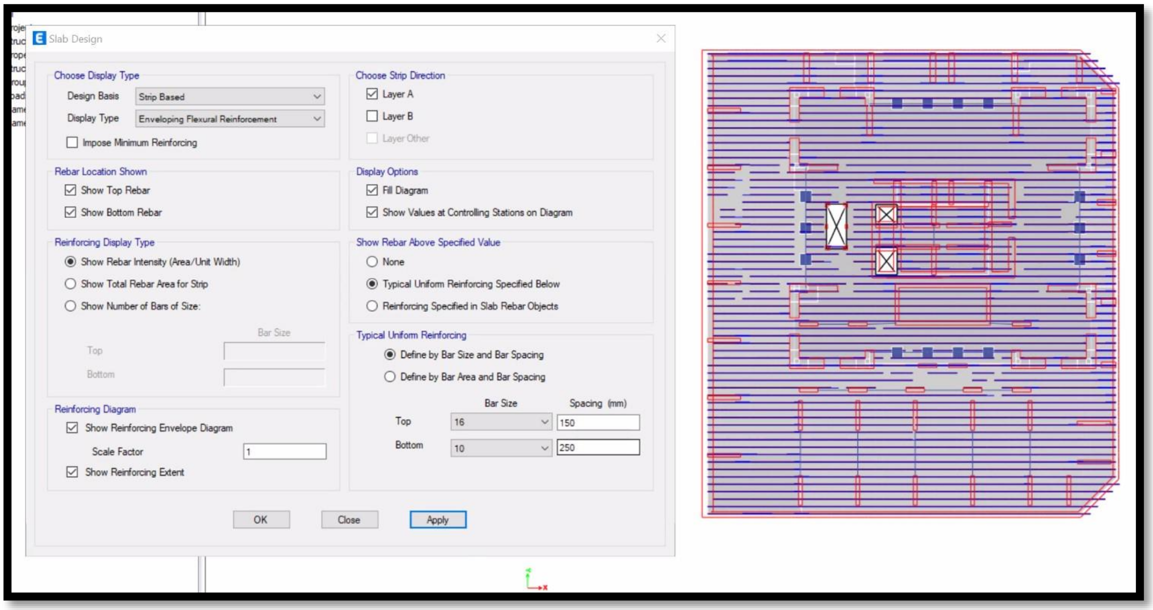


Figure 3.9.2.10 – Top Reinforcement in X-Direction for Ground Floor

The below figure shows that some zones need to be reinforced more than the others

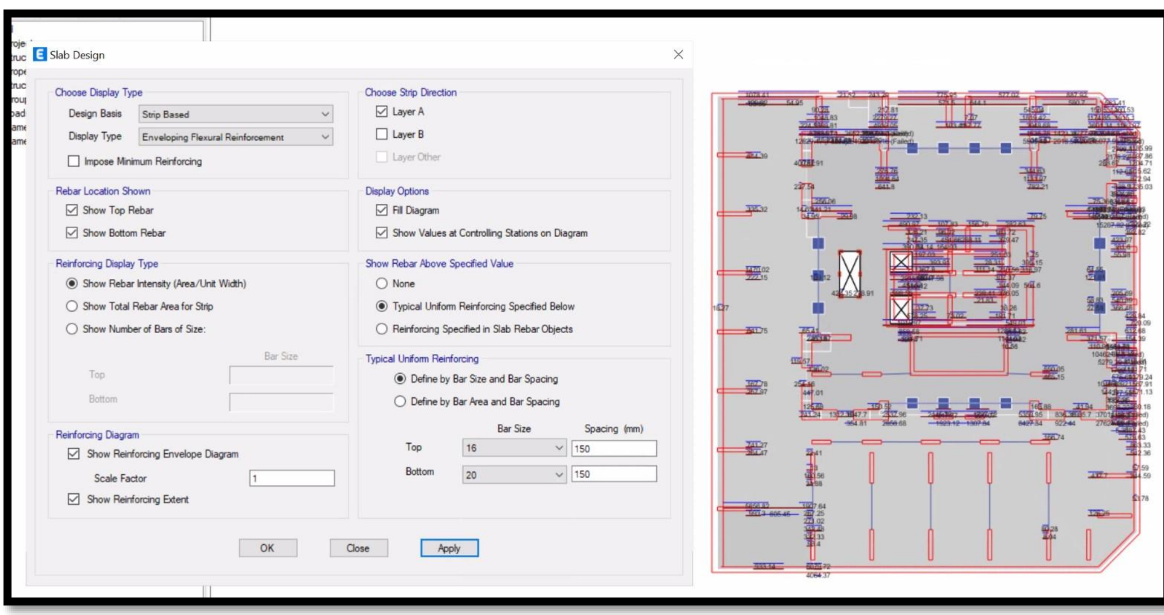


Figure 3.9.2.11 – Demonstration for Odd Zones in X-Direction in Ground Floor

For the zones mentioned above

For Top, use Φ25/150 mm

For Bottom, use Φ25/150 mm

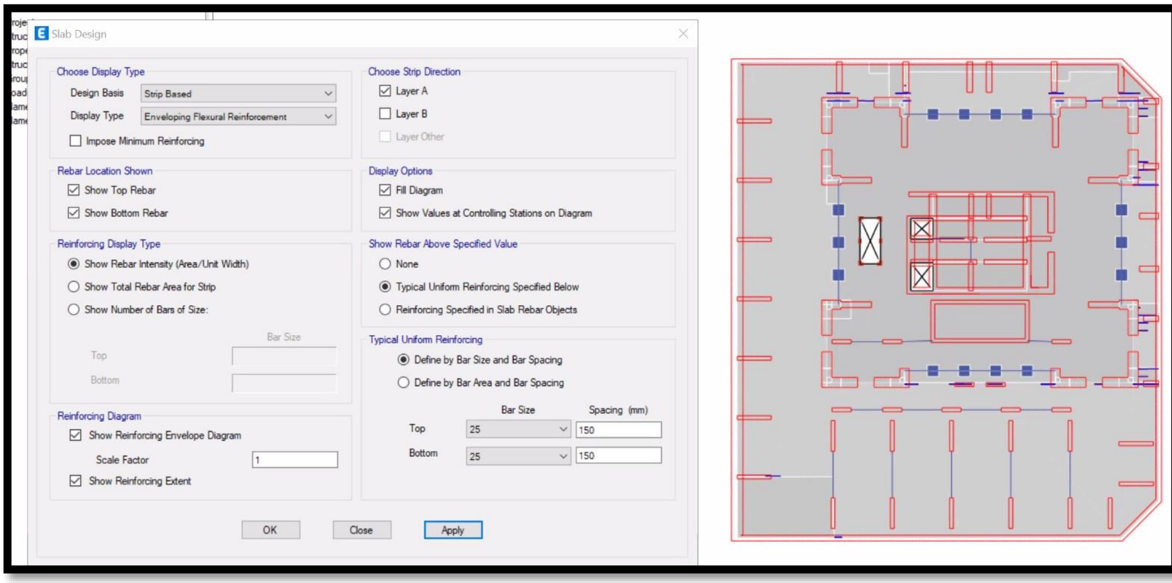


Figure 3.9.2.12 – Reinforcement in X-Direction for Odd Zones in Ground Floor

Shear Reinforcement

No need for shear reinforcement

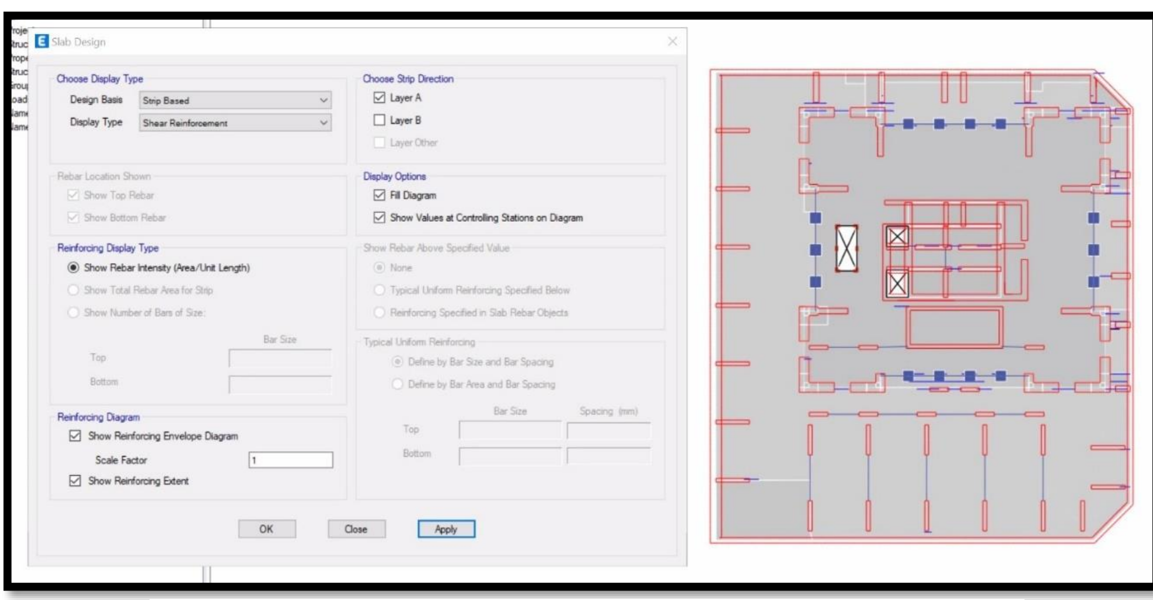


Figure 3.9.2.13 – Shear Reinforcement in X-Direction for Ground Floor

Y-Direction

Bottom reinforcement is required for the entirety of the slab and top reinforcement is required for a significant portion of the slab due to different moment signs from different combinations

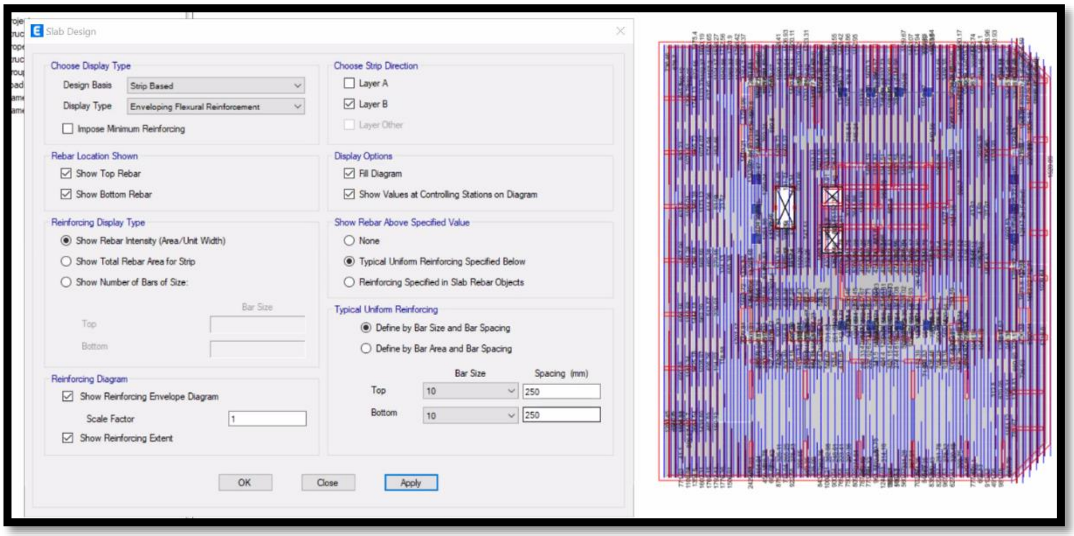


Figure 3.9.2.14 – Reinforcement Regions in Y-Direction for Ground Floor

Bottom reinforcement for the whole slab:

Use Φ18/150 mm

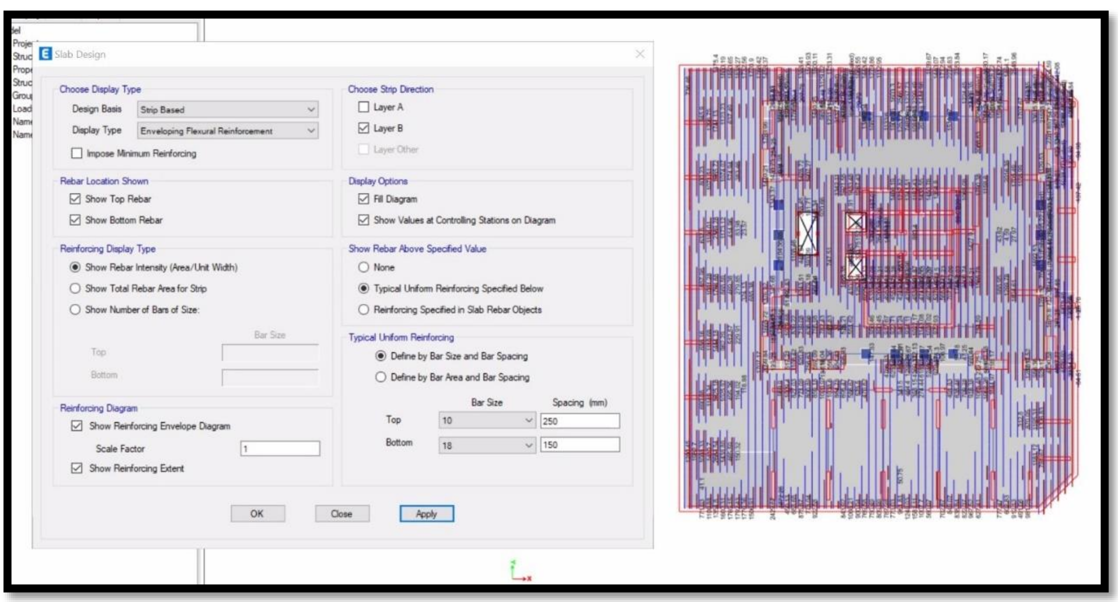


Figure 3.9.2.15 – Bottom Reinforcement in Y-Direction for Ground Floor

Top reinforcement for the whole slab in Y direction

Use Φ18/120 mm

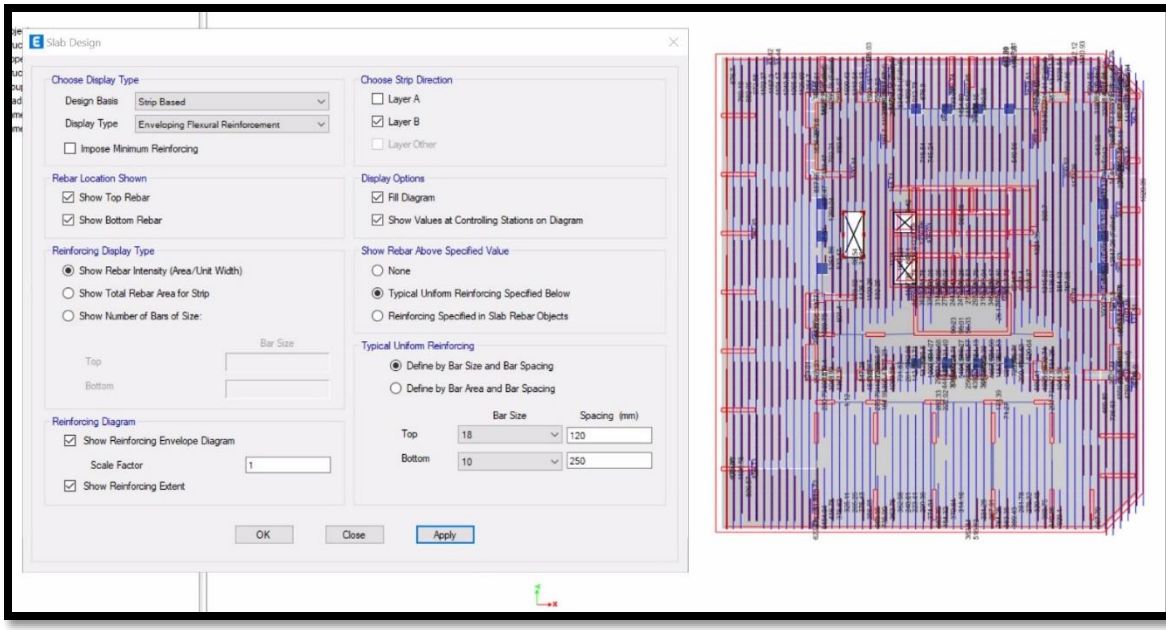


Figure 3.9.2.16 – Top Reinforcement in Y-Direction for Ground Floor

Some zones need to have steel reinforcement more than the others

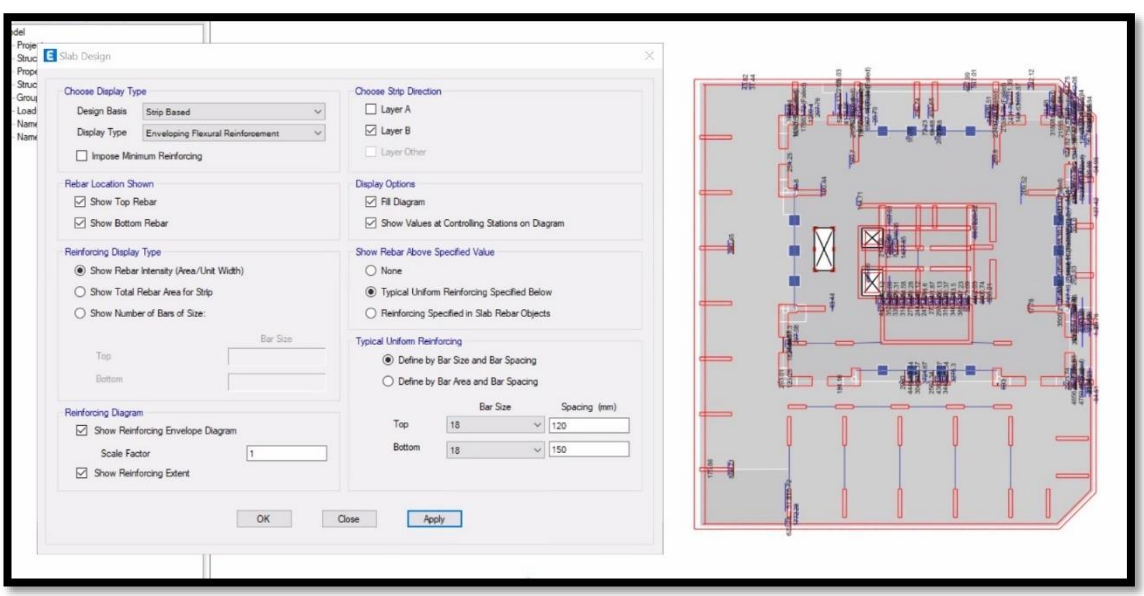


Figure 3.9.2.17 – Demonstration for Odd Zones in Y-Direction in Ground Floor

Reinforcement of the zones mentioned above

For Top, use Φ28/120 mm

For Bottom, use Φ28/120 mm

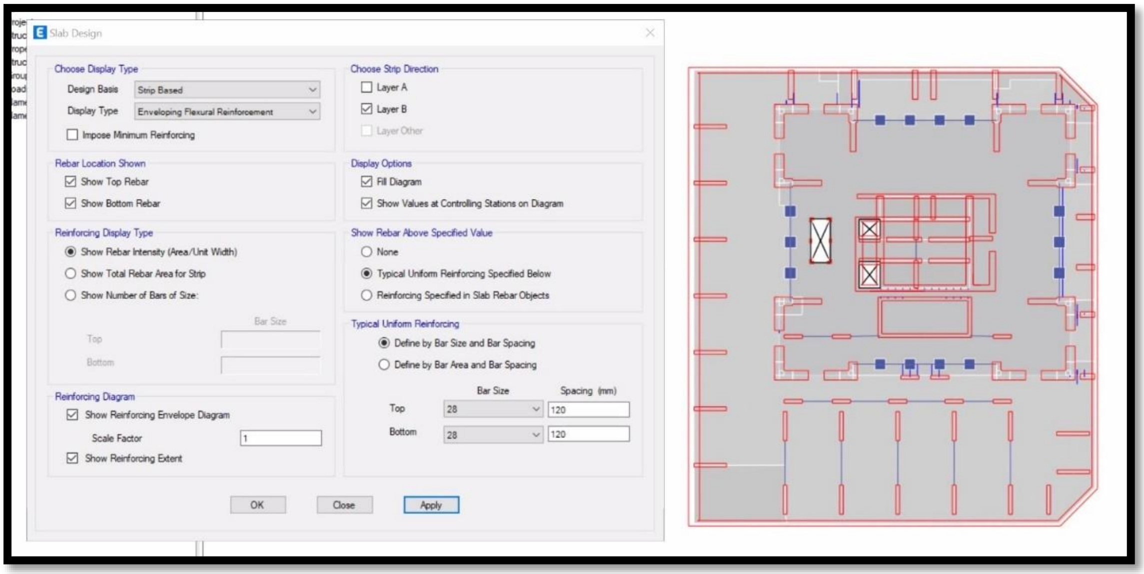


Figure 3.9.2.18 – Odd Zones Reinforcement in Y-Direction for Ground Floor

Shear Reinforcement

No need for shear reinforcement

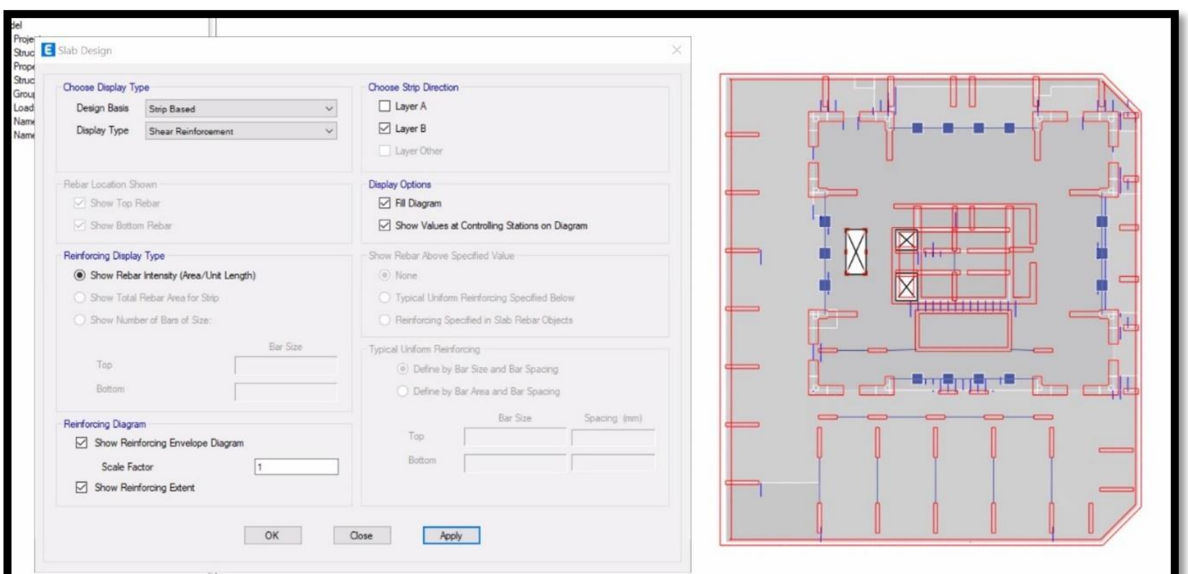


Figure 3.9.2.19 –Shear Reinforcement in Y-Direction for Ground Floor

3. Floor slab at 6th Floor to 22nd Floor, 26th Floor to 50th Floor, 54th Floor to 77th Floor.

X-Direction

Bottom reinforcement is required for the whole slab and top reinforcement for significant portion of the slab due to different moment signs from different combinations

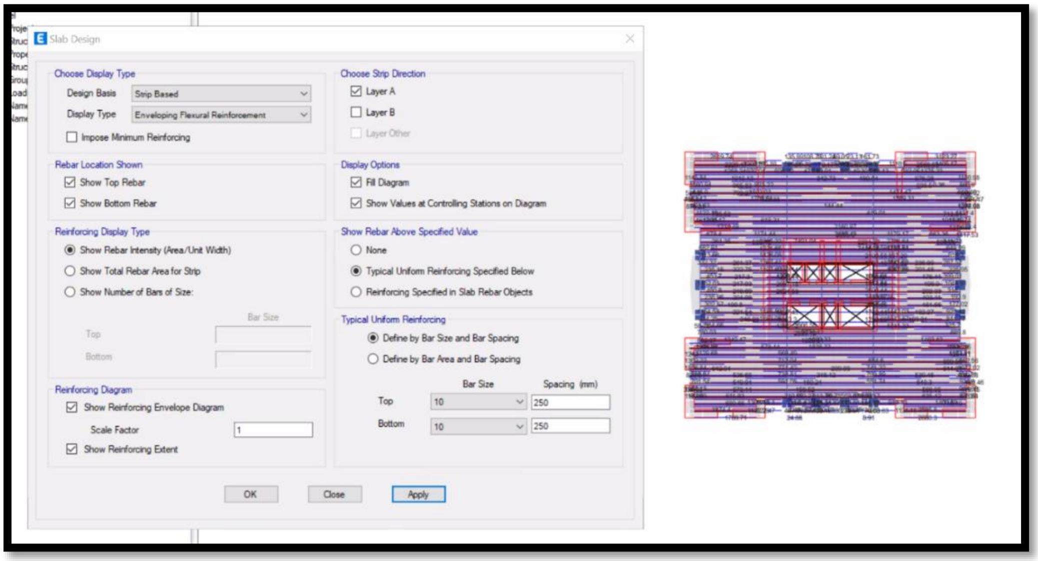


Figure 3.9.2.20 – Reinforcement Regions in X-Direction for 8th Floor

Use Φ18/250 mm for Bottom reinforcement for the whole slab (except for the shown zones)

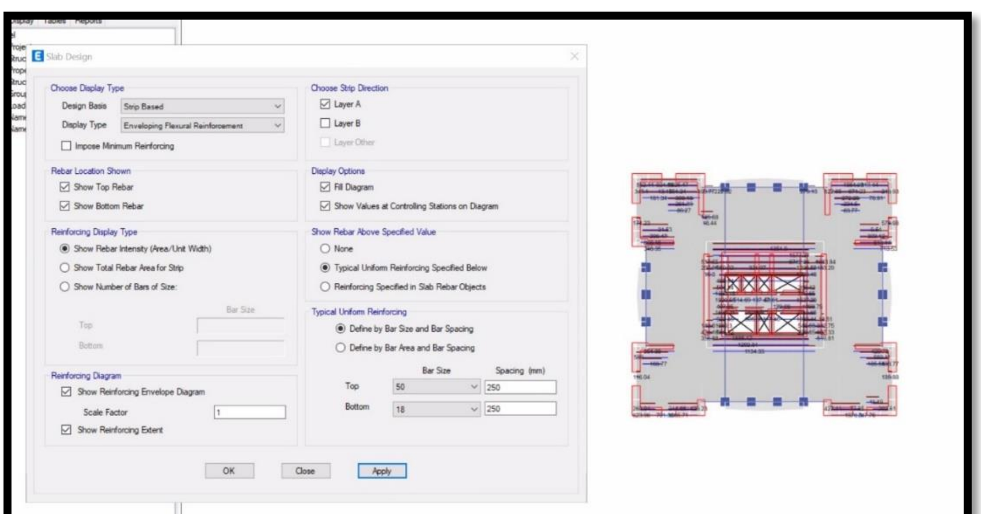


Figure 3.9.2.21 – Bottom Reinforcement in X-Direction for 8th Floor

Use Φ22/120 mm for Bottom reinforcement for the zones mentioned above

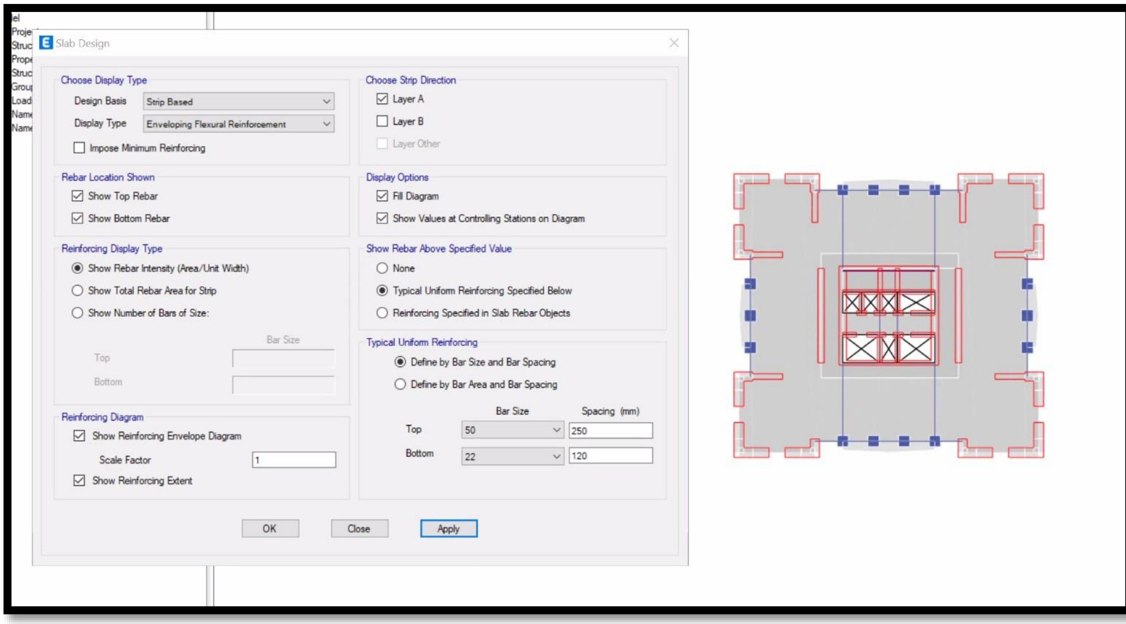


Figure 3.9.2.22 – Bottom Reinforcement in X-Direction for Odd Zones in 8th Floor

Use Φ16/250 mm for top reinforcement for the whole slab (except for the shown zones)

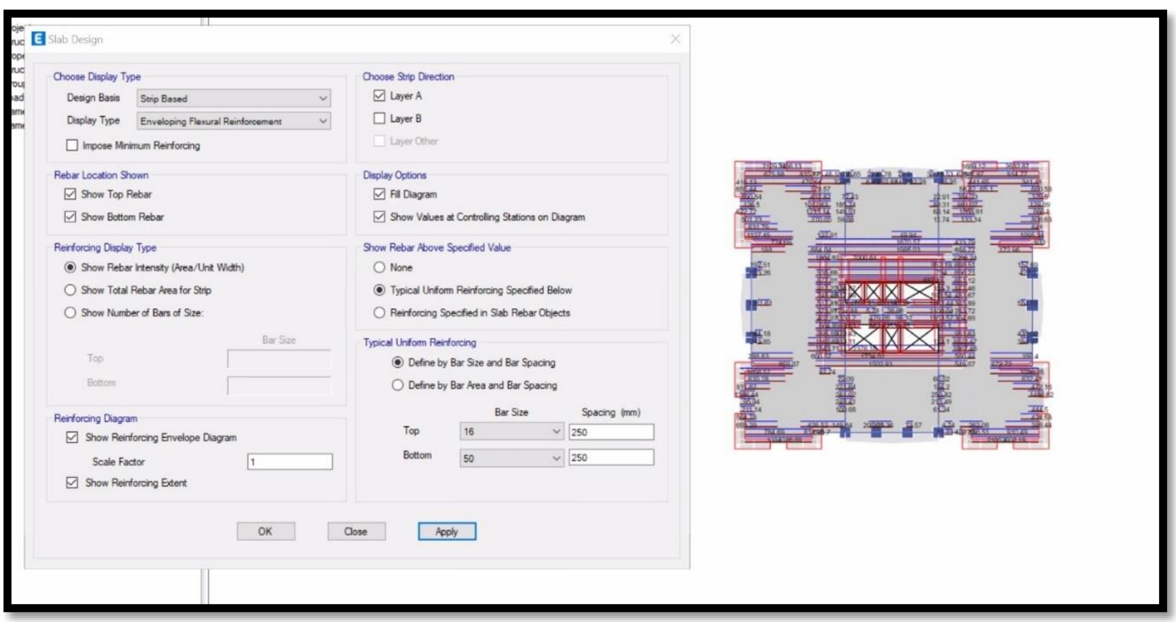


Figure 3.9.2.23 – Top Reinforcement in X-Direction for 8th Floor

Use Φ20/120 mm for top reinforcement for the zones mentioned above

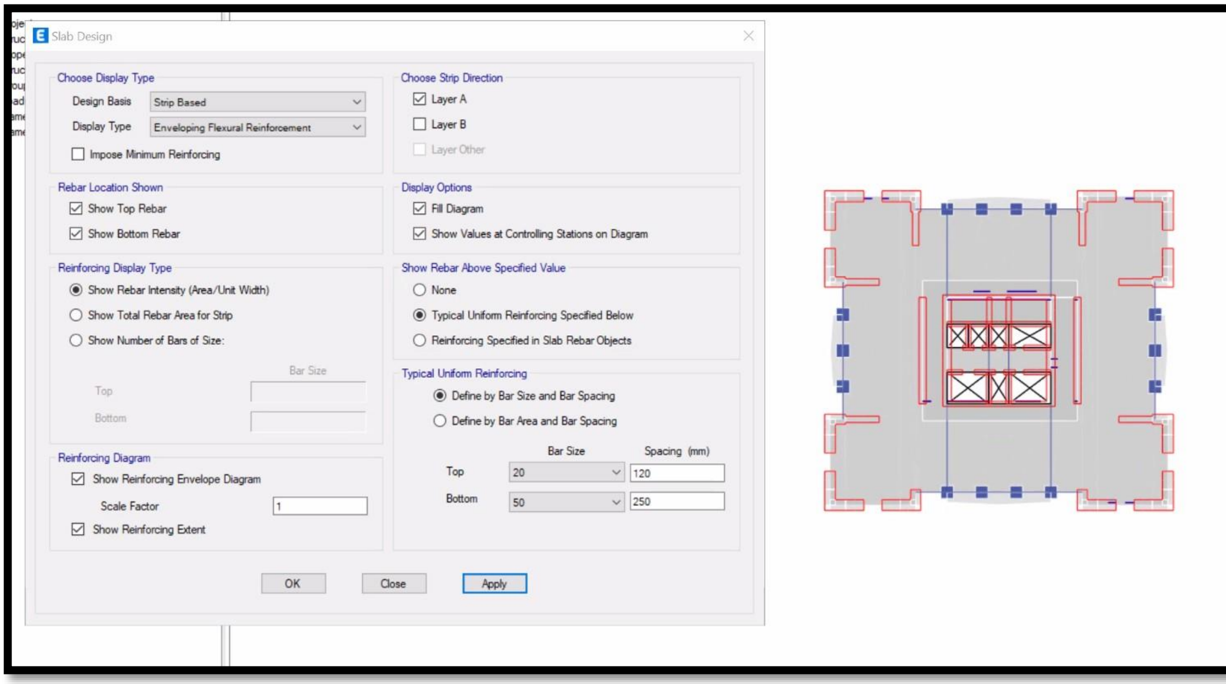


Figure 3.9.2.24 –Top Reinforcement in X-Direction for Odd Zones in 8th Floor

Shear Reinforcement

No need for shear reinforcement

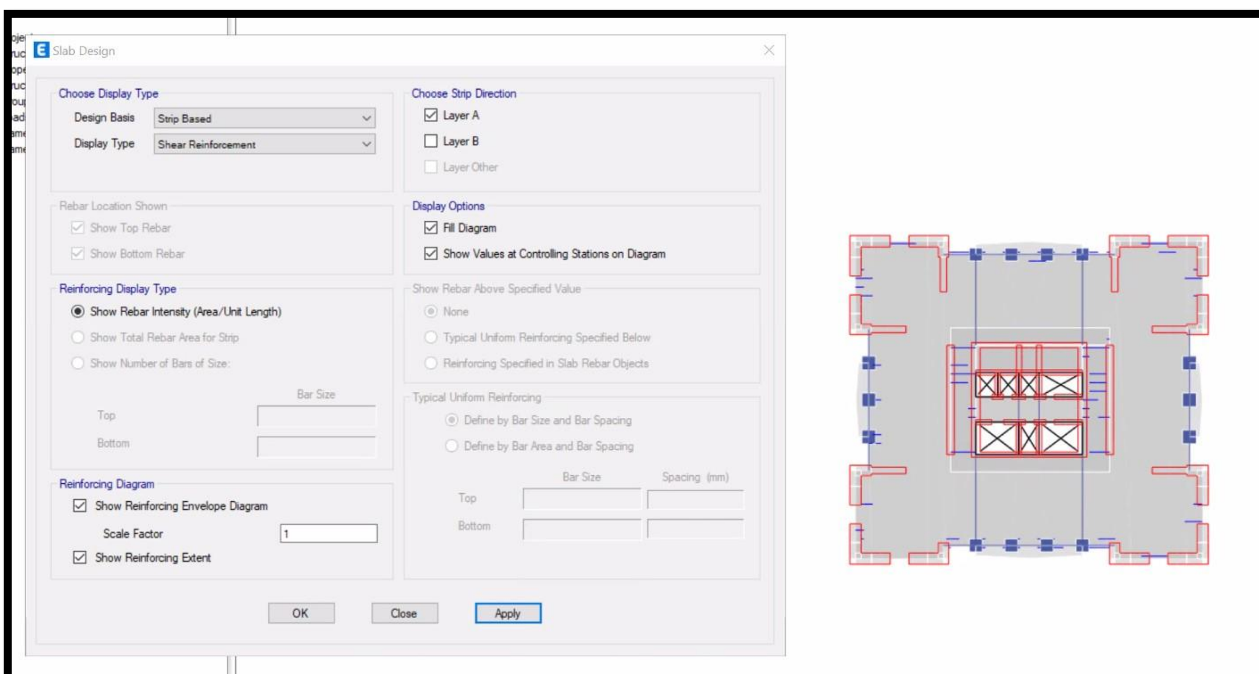


Figure 3.9.2.25 – Shear Reinforcement in X-Direction for 8th Floor

Y-Direction

Bottom reinforcement is required in the whole slab and top reinforcement is required for significant portions of the slab due to different moment signs from different combinations

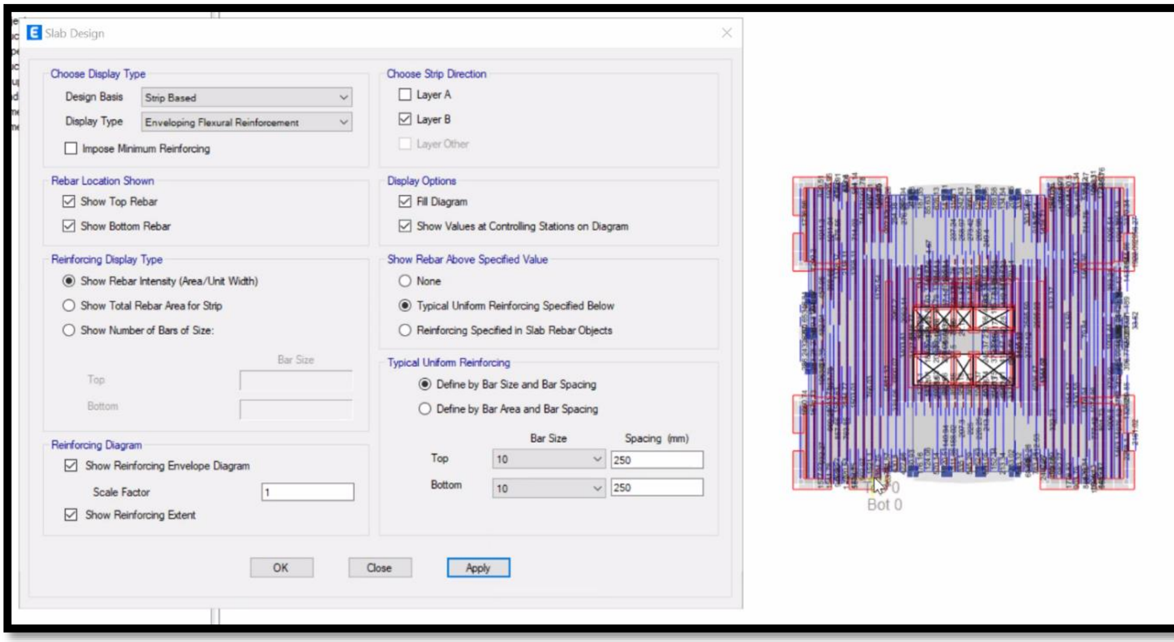


Figure 3.9.2.26 – Reinforcement Regions in Y-Direction for 8th Floor

Use Φ16/120 mm for bottom reinforcement for the whole slab (except for the zones beside the core as shown)

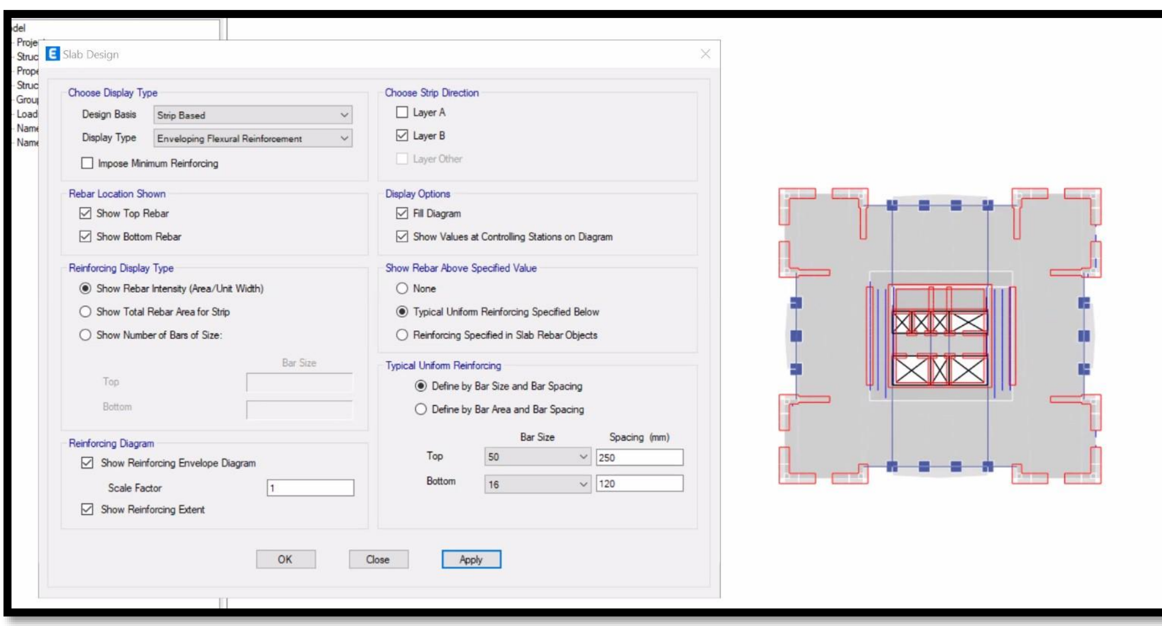


Figure 3.9.2.27 – Bottom Reinforcement in Y-Direction for 8th Floor

Use Φ26/120 mm for bottom reinforcement for the zones beside the core

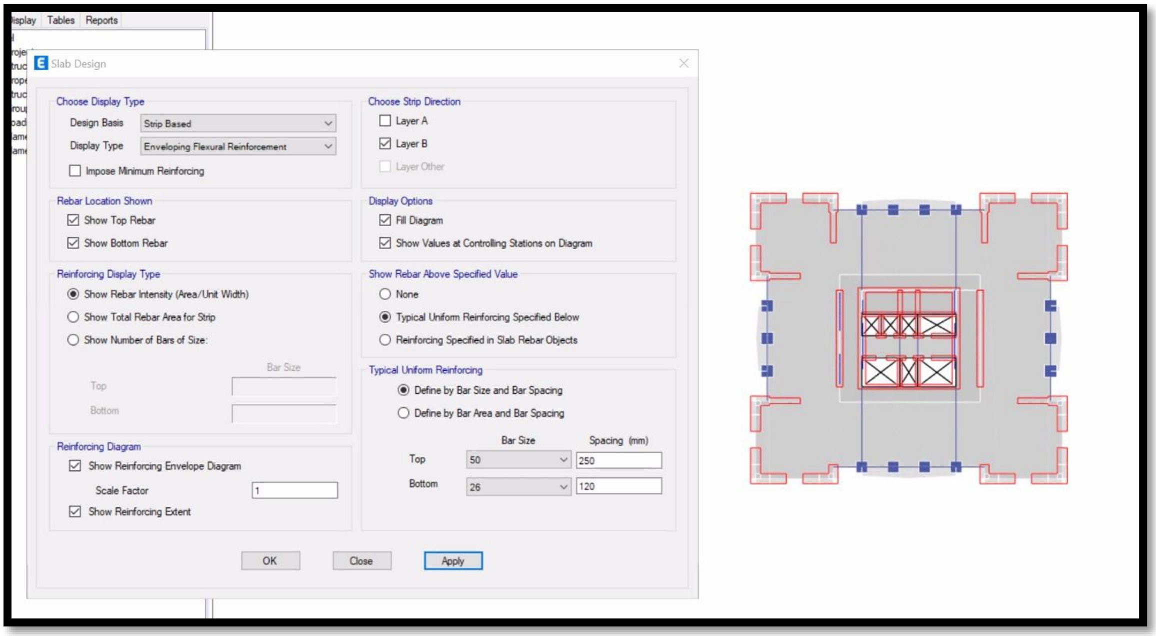


Figure 3.9.2.28 – Bottom Reinforcement in Y-Direction besides the core for 8th Floor

Use Φ16/250 mm for top reinforcement for the whole slab (except for the zones shown)

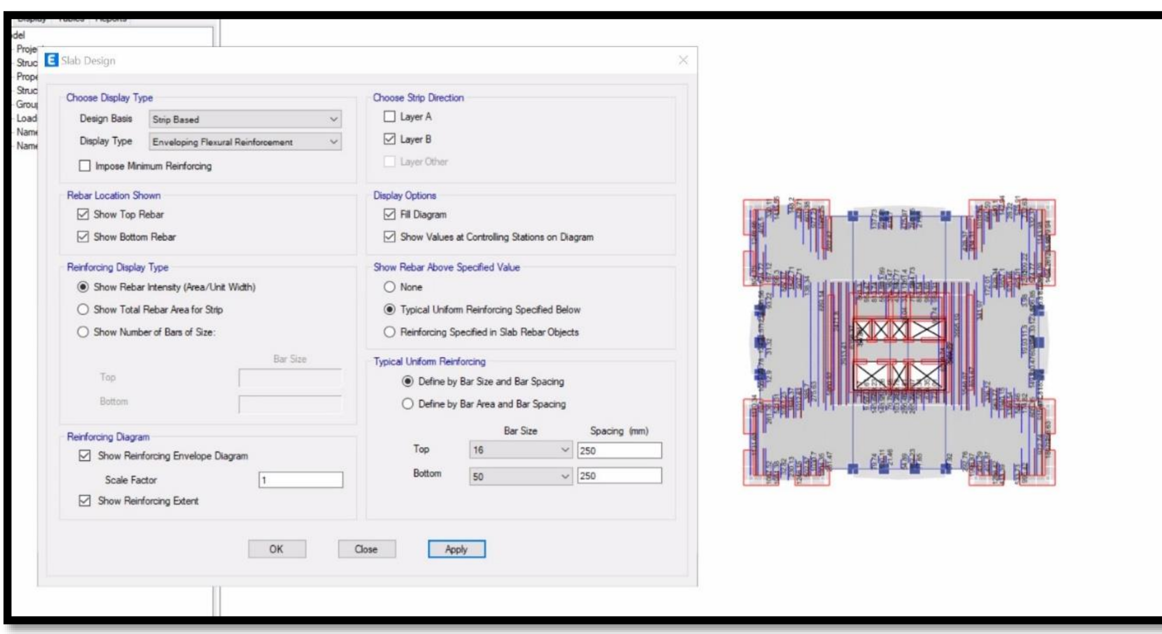


Figure 3.9.2.29 – Top Reinforcement in Y-Direction for 8th Floor

Use Φ25/120 mm for top reinforcement for the zones shown above

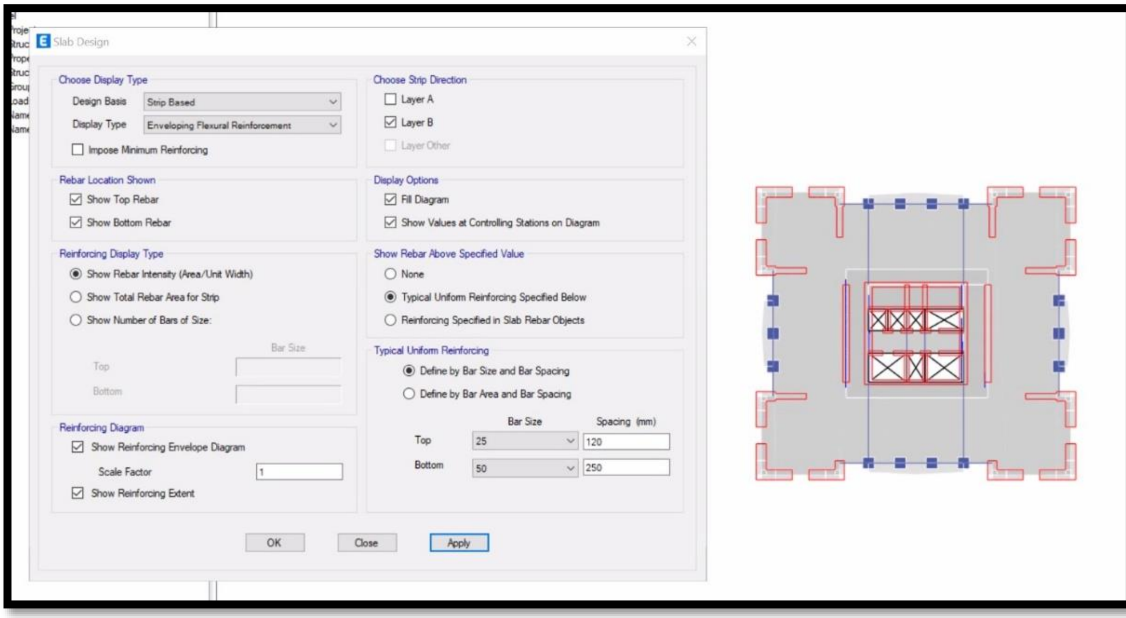


Figure 3.9.2.30 – Top Reinforcement in Y-Direction for Odd Zones for 8th Floor

Shear Reinforcement

No need for shear reinforcement

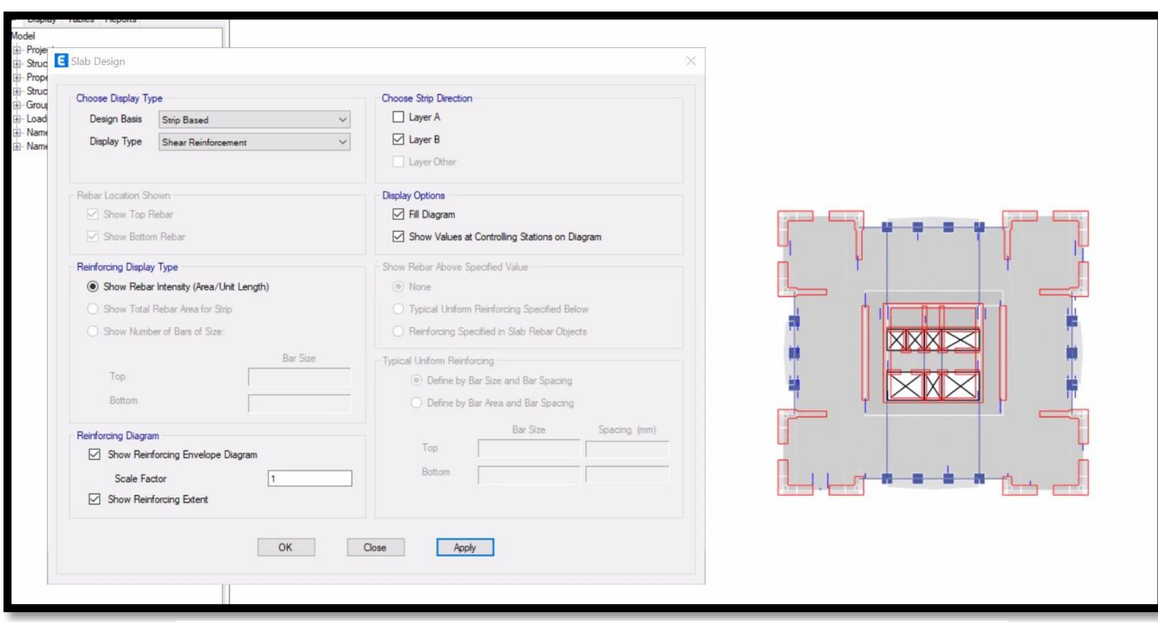


Figure 3.9.2.31 – Shear Reinforcement in Y-Direction for 8th Floor

4. Floor Slab at Service Floors (23-25, 51-53, 78-80)

X-Direction:
Bottom and top reinforcement are required for the whole slab due to different moment signs from different combinations

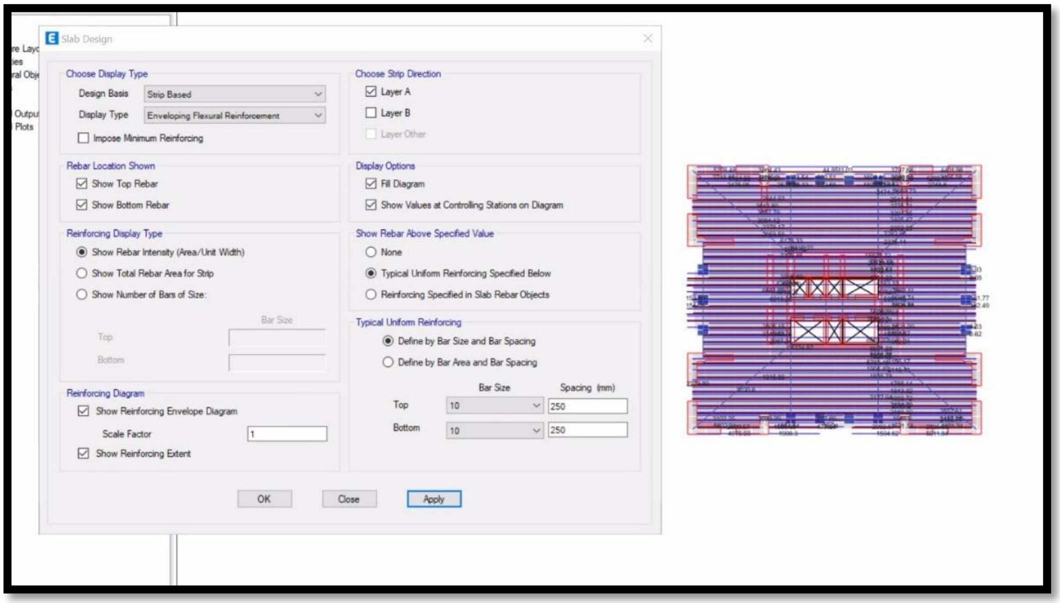


Figure 3.9.2.32 – Reinforcement Regions in X-Direction for 23rd Floor

Use Φ22/120 mm for bottom reinforcement for the whole slab (except for the shown zones)

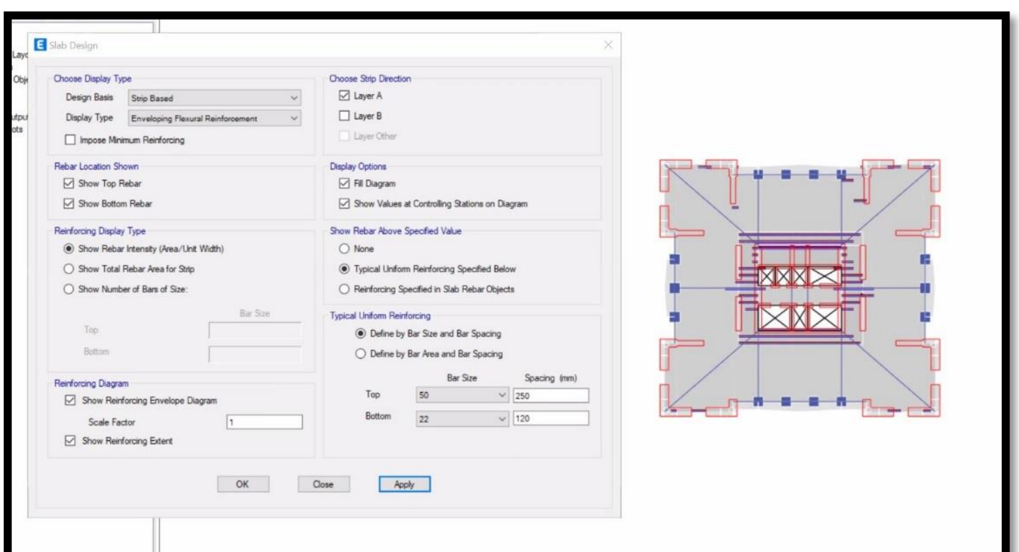


Figure 3.9.2.33 – Bottom Reinforcement in X-Direction for 23rd Floor

Use Φ22/120 mm for top reinforcement for the whole slab (except for the shown zones)

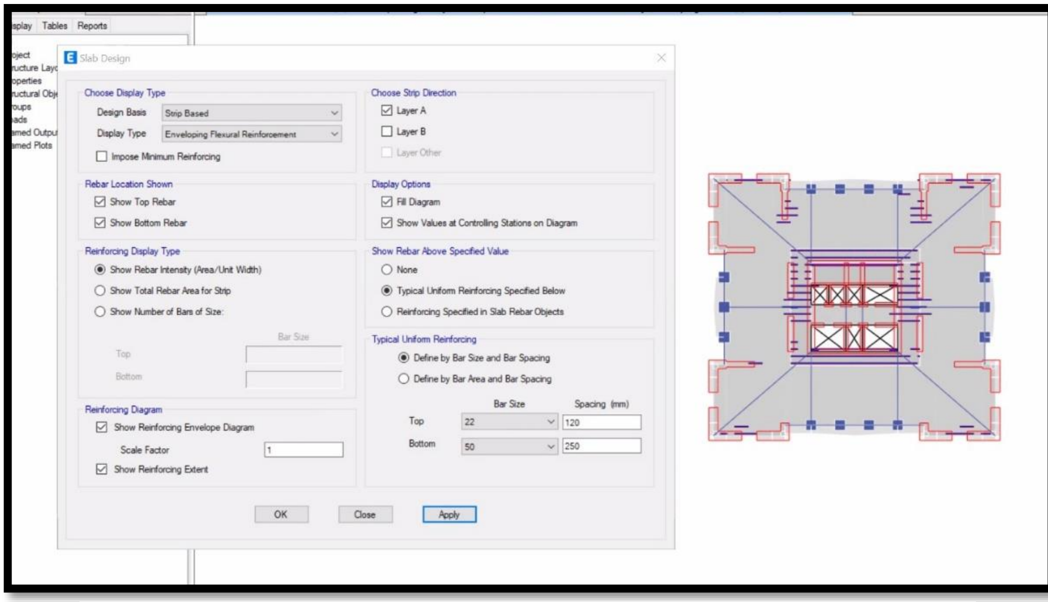


Figure 3.9.2.34 – Top Reinforcement in X-Direction for 23rd Floor

For the zones mentioned above:
Use Φ32/120 mm for top reinforcement
Use Φ32/120 mm for bottom reinforcement

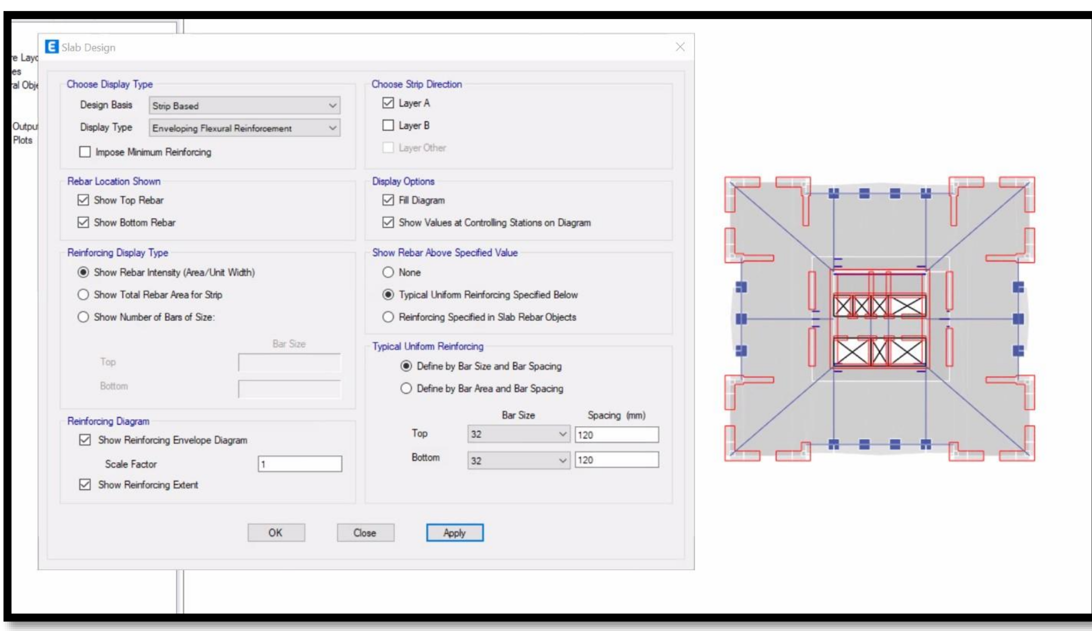


Figure 3.9.2.35 – Reinforcement in X-Direction for Odd Zones in 23rd Floor

Shear Reinforcement

No need for shear reinforcement

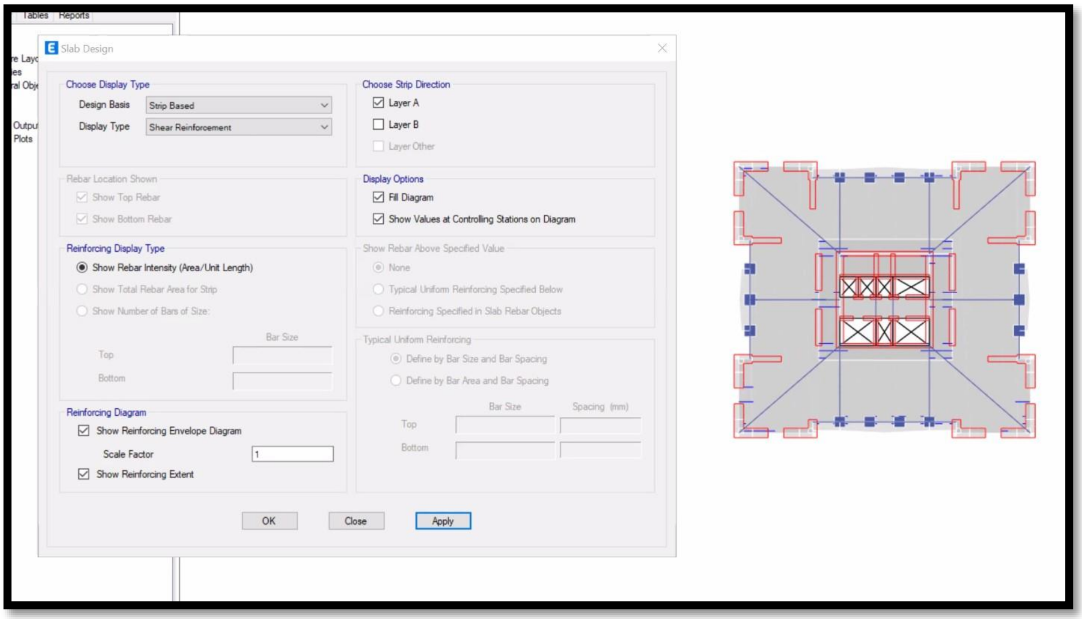


Figure 3.9.2.36 – Shear Reinforcement in X-Direction for 23rd Floor

Y-Direction

Bottom and top reinforcement is required for the whole slab due to different moment signs from different load combinations

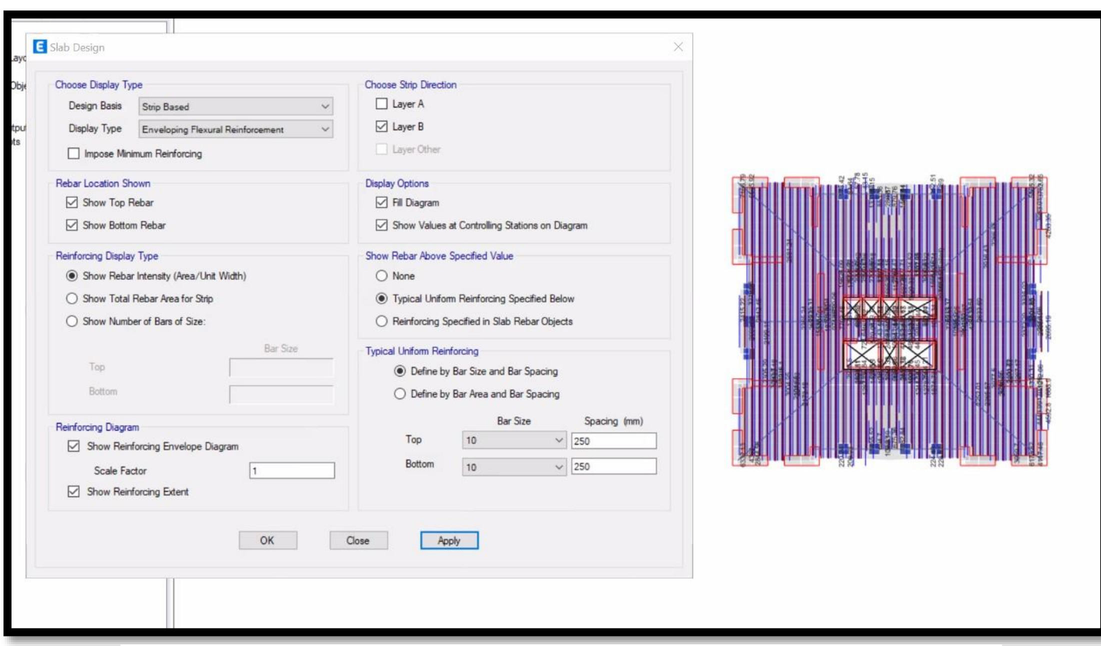


Figure 3.9.2.37 – Reinforcement Regions in Y-Direction for 23rd Floor

Use Φ22/120 mm for bottom reinforcement for the whole slab (except for the zones shown)

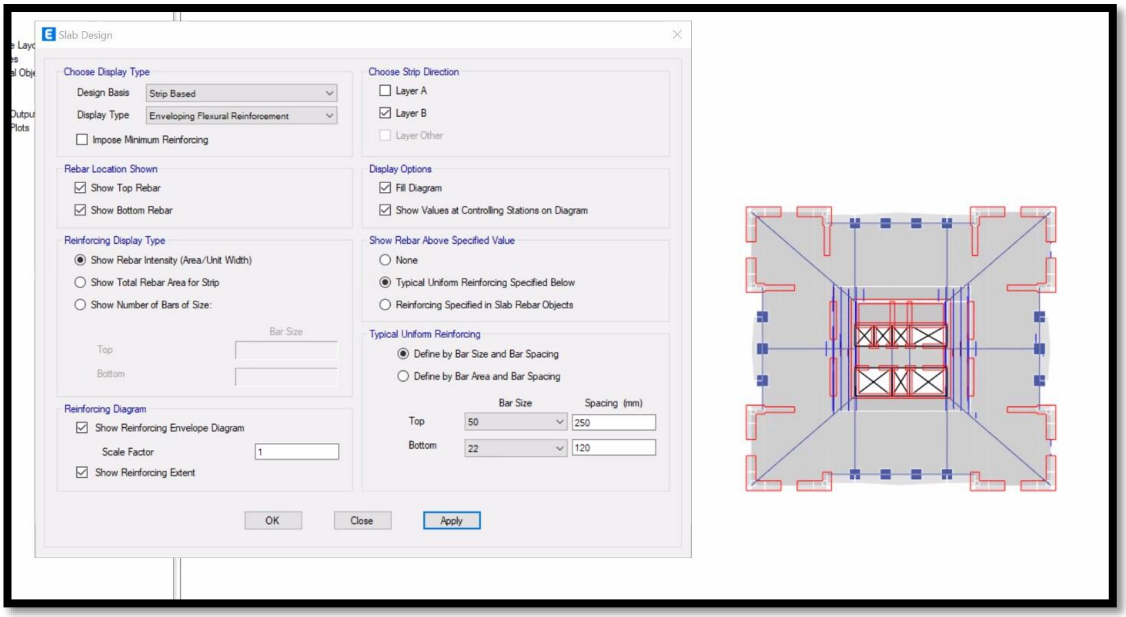


Figure 3.9.2.38 – Bottom Reinforcement in Y-Direction for 23rd Floor

Use Φ22/120 mm for top reinforcement (except for the zones shown)

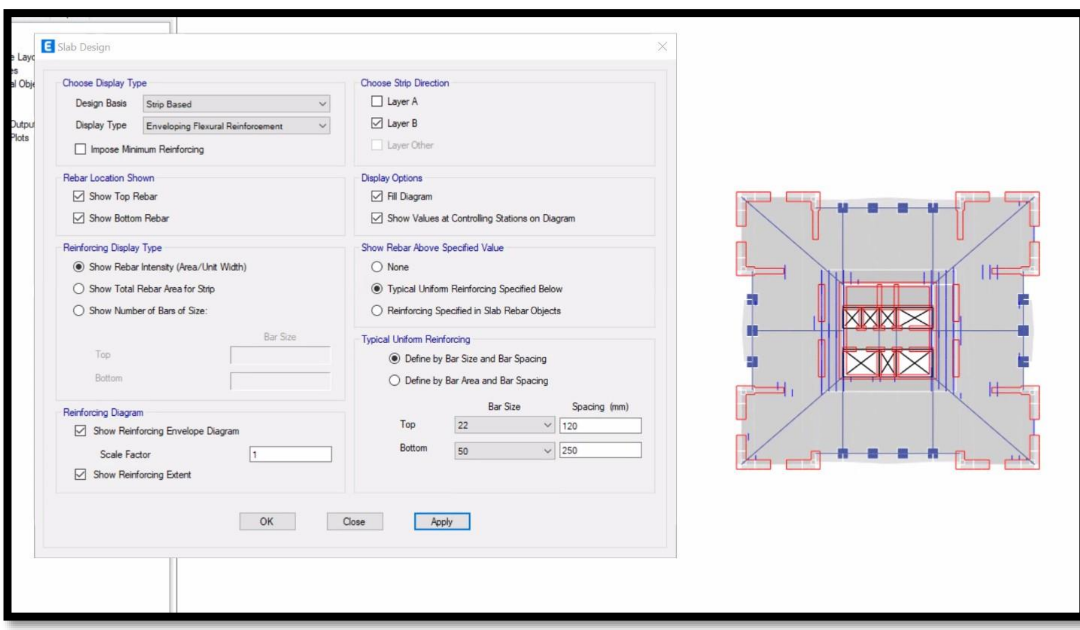


Figure 3.9.2.39 – Top Reinforcement in Y-Direction for 23rd Floor

For the zones mentioned above:

Use Φ36/100 mm for top reinforcement

Use Φ36/100 mm for bottom reinforcement

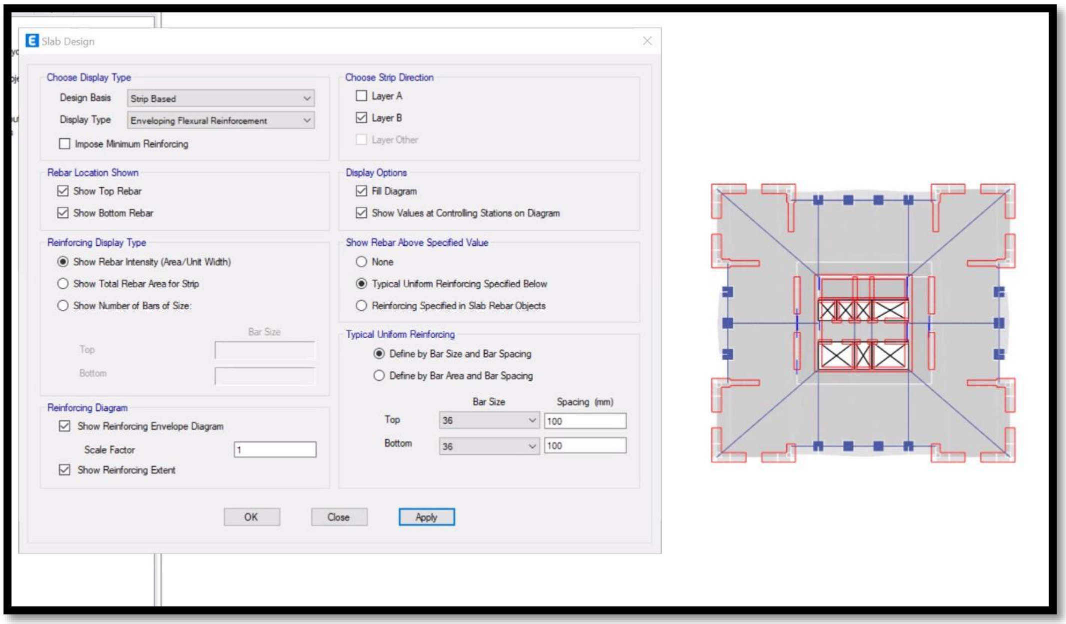


Figure 3.9.2.40 – Reinforcement in Y-Direction for Odd Zones in 23rd Floor

Shear Reinforcement

No need for shear reinforcement

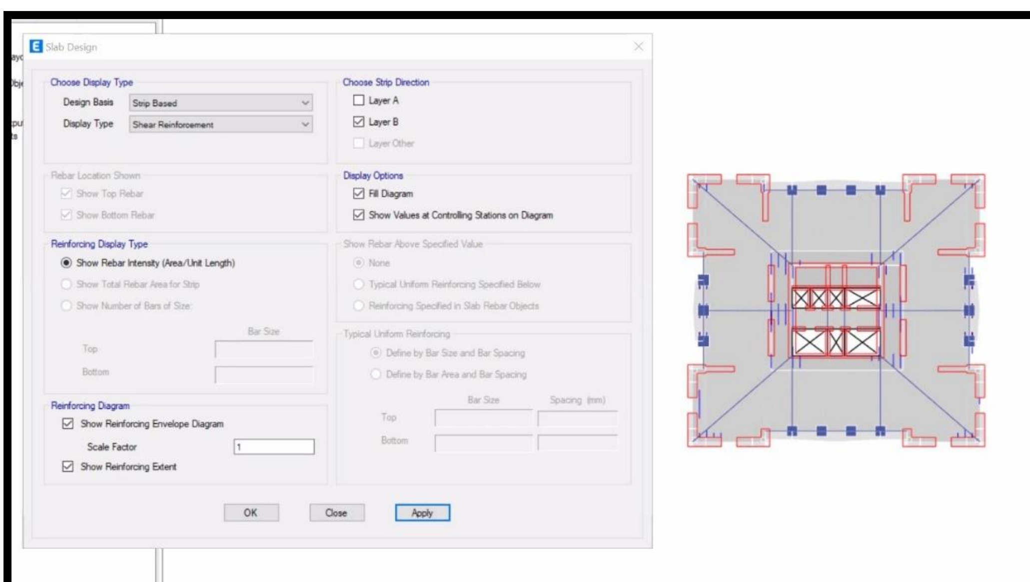


Figure 3.9.2.41– Shear Reinforcement in Y-Direction for 23rd Floor

5. Floor slab at 81st Floor to 95th Floor

X-Direction

For the whole slab (except for the shown zones)

Use Φ14/200 mm for top reinforcement

Use Φ14/200 mm for bottom reinforcement

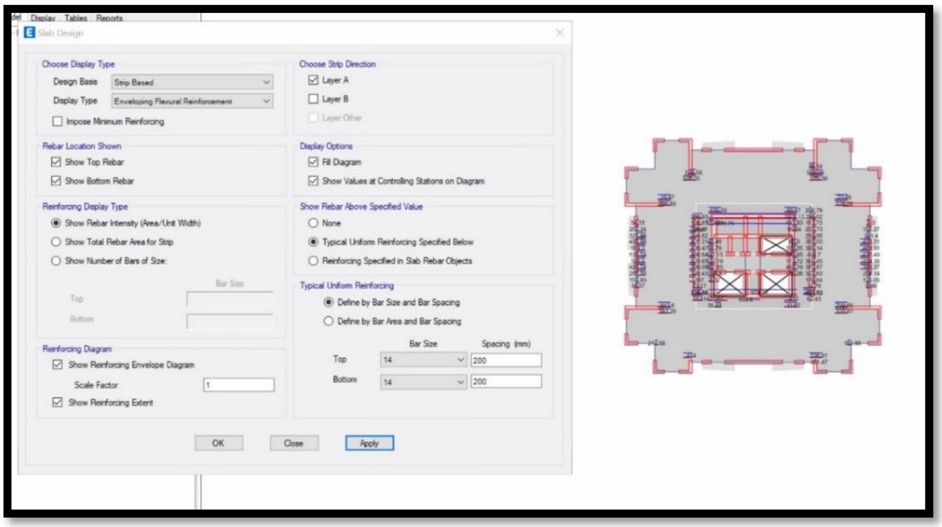


Figure 3.9.2.42 – Reinforcement in X-Direction for 81st Floor

For the zones mentioned above:

Use Φ18/120 mm for top reinforcement

Use Φ18/120 mm for bottom reinforcement

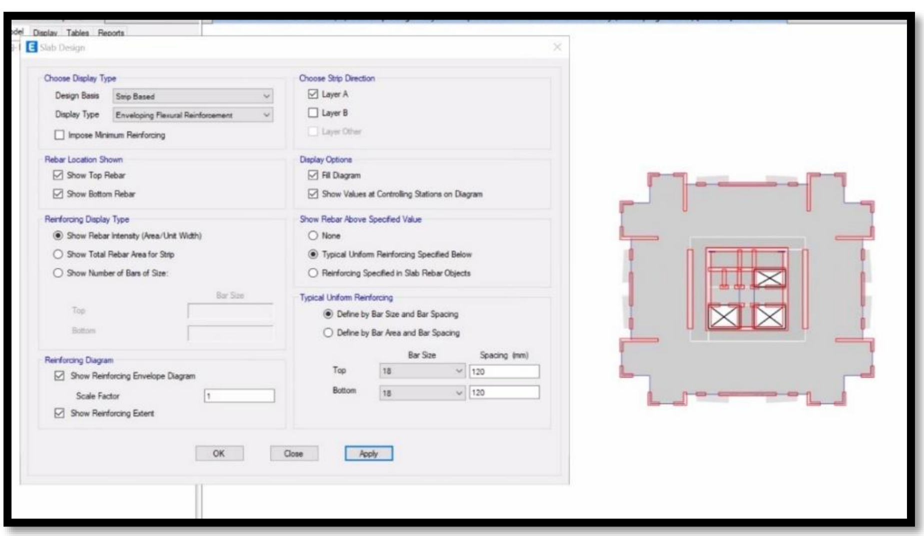


Figure 3.9.2.43 – Reinforcement in X-Direction for Odd Zones in 81st Floor

Shear Reinforcement

No need for shear reinforcement

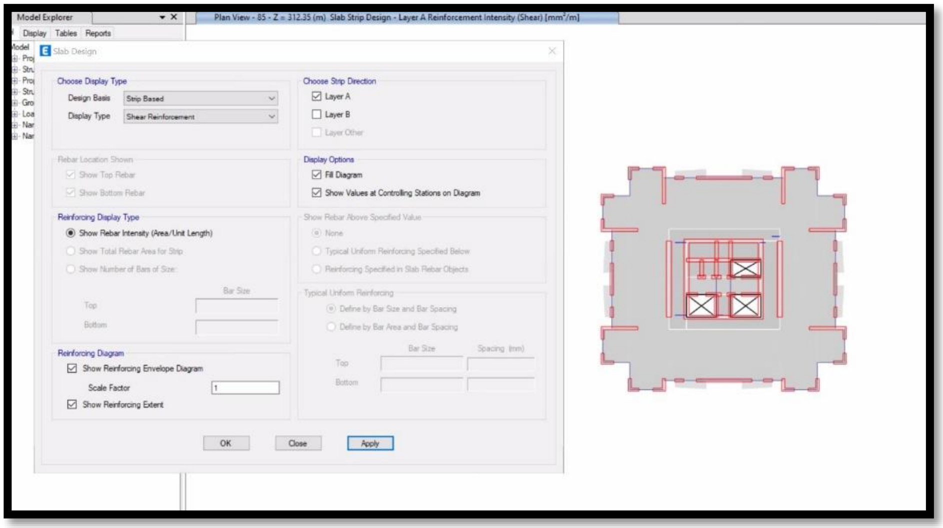


Figure 3.9.2.44 – Shear Reinforcement in X-Direction for 81st Floor

Y-Direction

For the whole slab (except the shown zones)

Use Φ14/200 mm for top reinforcement

Use Φ14/200 mm for bottom reinforcement

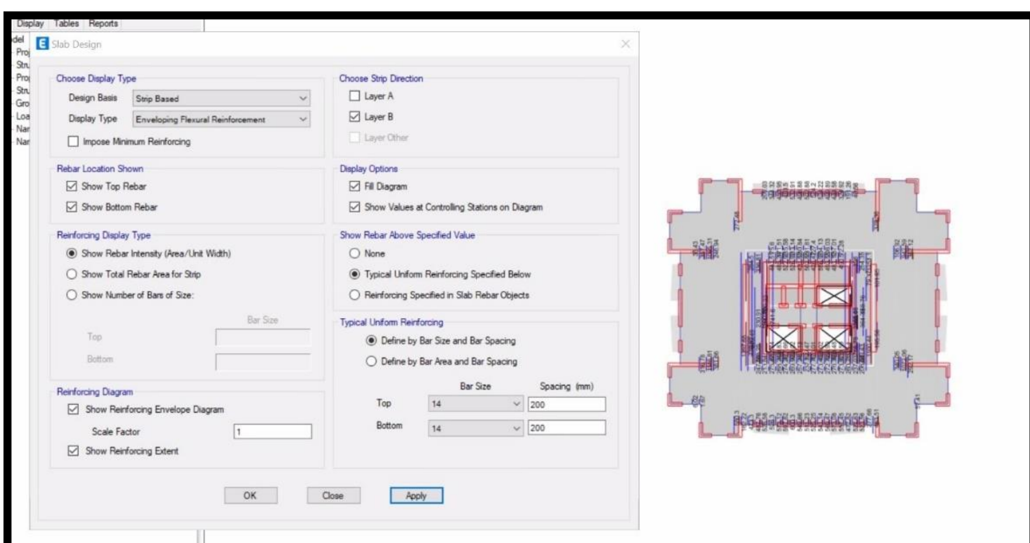


Figure 3.9.2.45 – Reinforcement in Y-Direction for 81st Floor

For the zones mentioned above

Use Φ18/120 mm for top reinforcement

Use Φ18/120 mm for bottom reinforcement

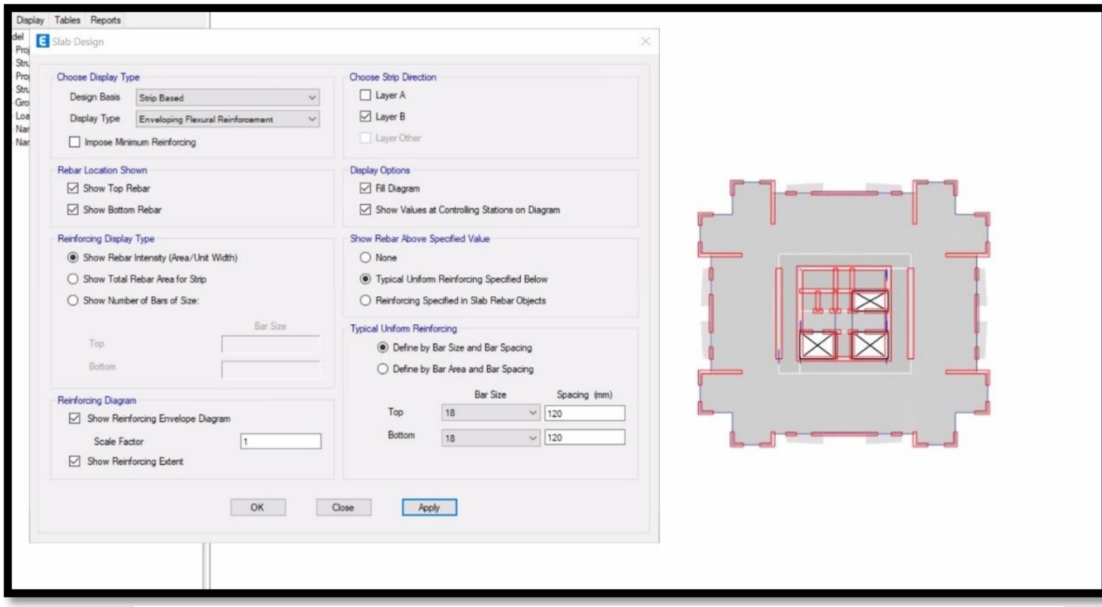


Figure 3.9.2.46 – Reinforcement in Y-Direction for Odd Zones in 81st Floor

Shear Reinforcement

No need for shear reinforcement

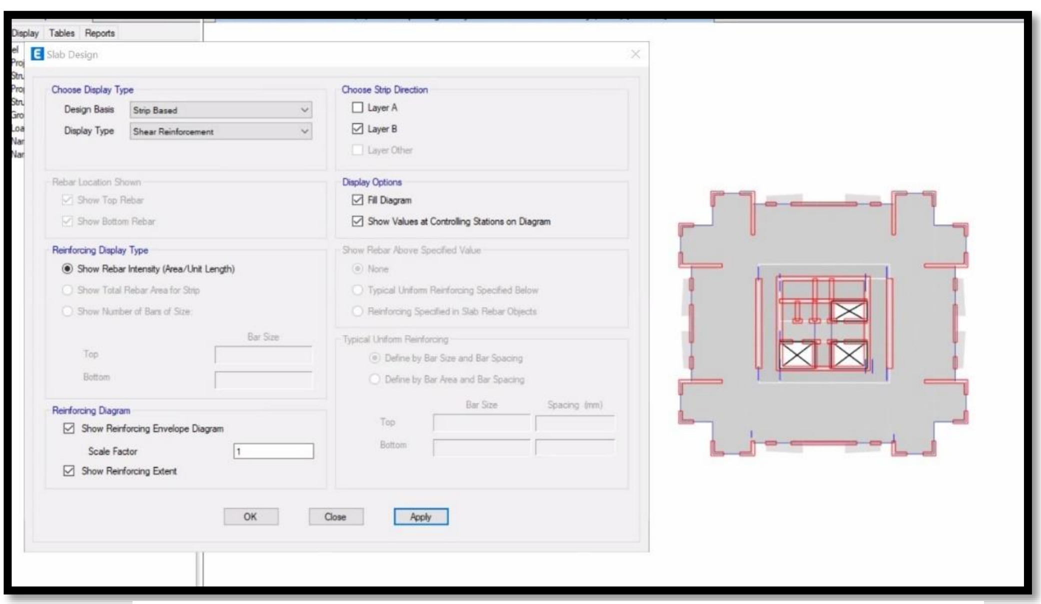


Figure 3.9.2.47 – Shear Reinforcement in Y-Direction for 81st Floor

6. Floor slab at 96th Floor

X-Direction

Bottom reinforcement is required for the whole slab and top reinforcement for a significant portion of the slab due to different moment signs from different combinations

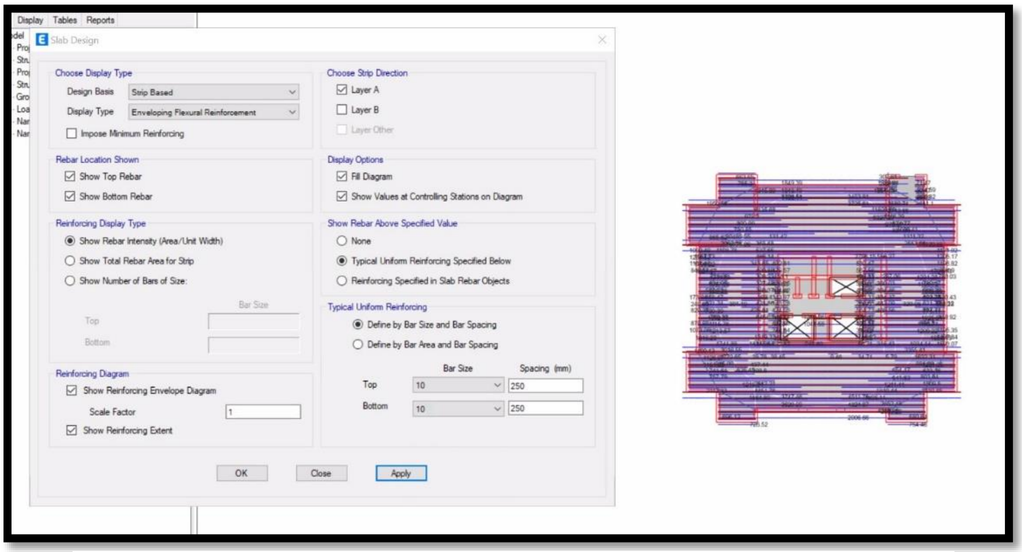


Figure 3.9.2.48 – Reinforcement Regions in X-Direction for 96th Floor

Use Φ16/250 mm for bottom reinforcement for Non-Drop panels regions

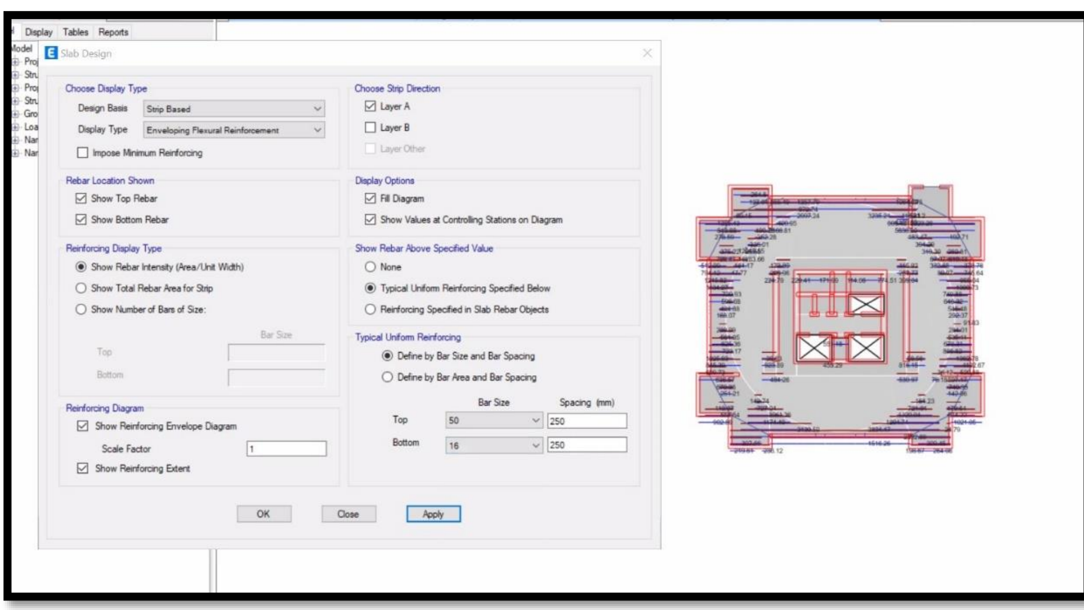


Figure 3.9.2.49 – Bottom Reinforcement in X-Direction for Non-Drop Panels Regions in 96th Floor

Use Φ16/250 mm for top reinforcement for Non-Drop panels regions

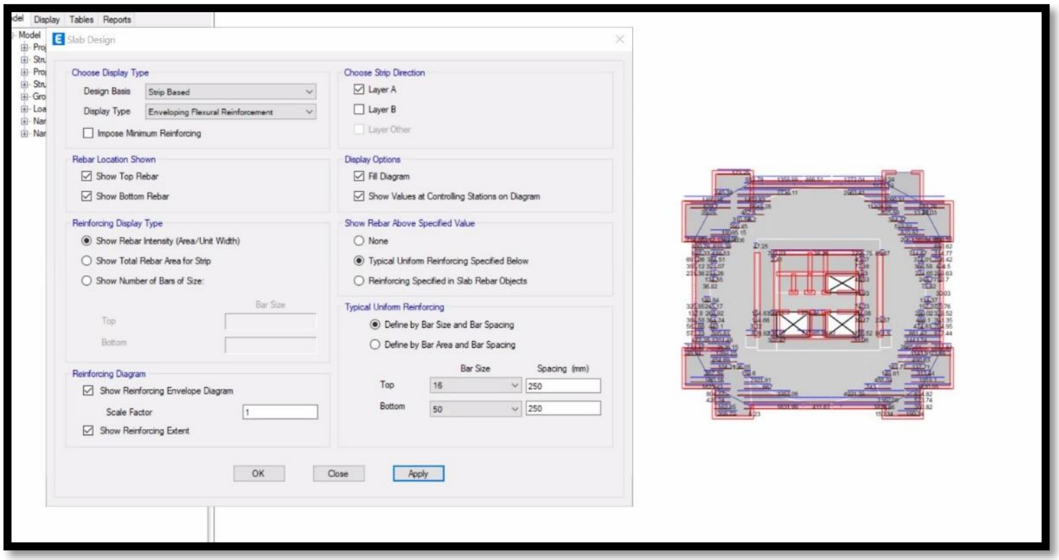


Figure 3.9.2.50 – Top Reinforcement in X-Direction for Non-Drop Panels Regions in 96th Floor

Use Φ25/120 mm for bottom reinforcement for drop panels regions

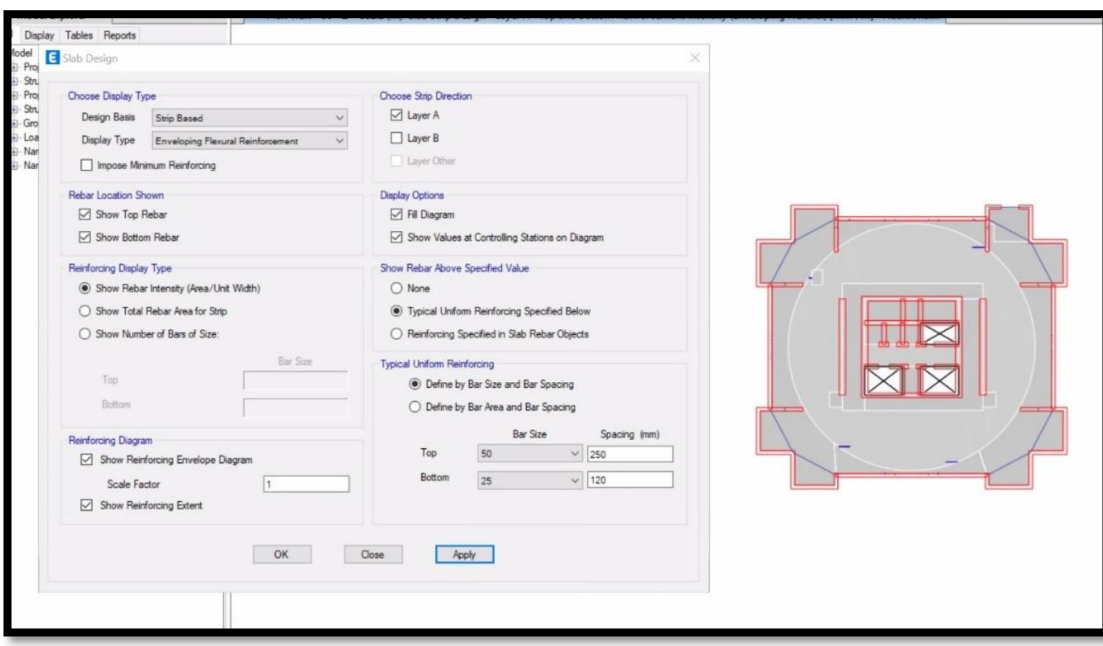


Figure 3.9.2.51 – Bottom Reinforcement in X-Direction for Drop Panels Regions in 96th Floor

Use Φ25/120 mm for top reinforcement for drop panels regions

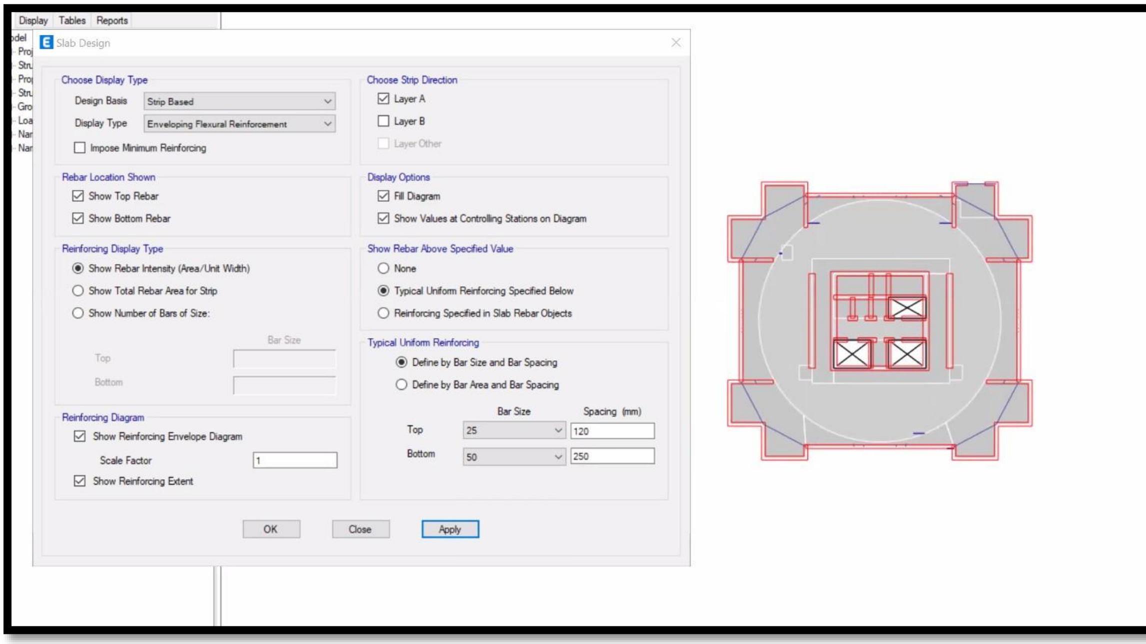


Figure 3.9.2.52 – Top Reinforcement in X-Direction for Drop Panels Regions in 96th Floor

Shear Reinforcement

Shear reinforcement is needed (stud rails)

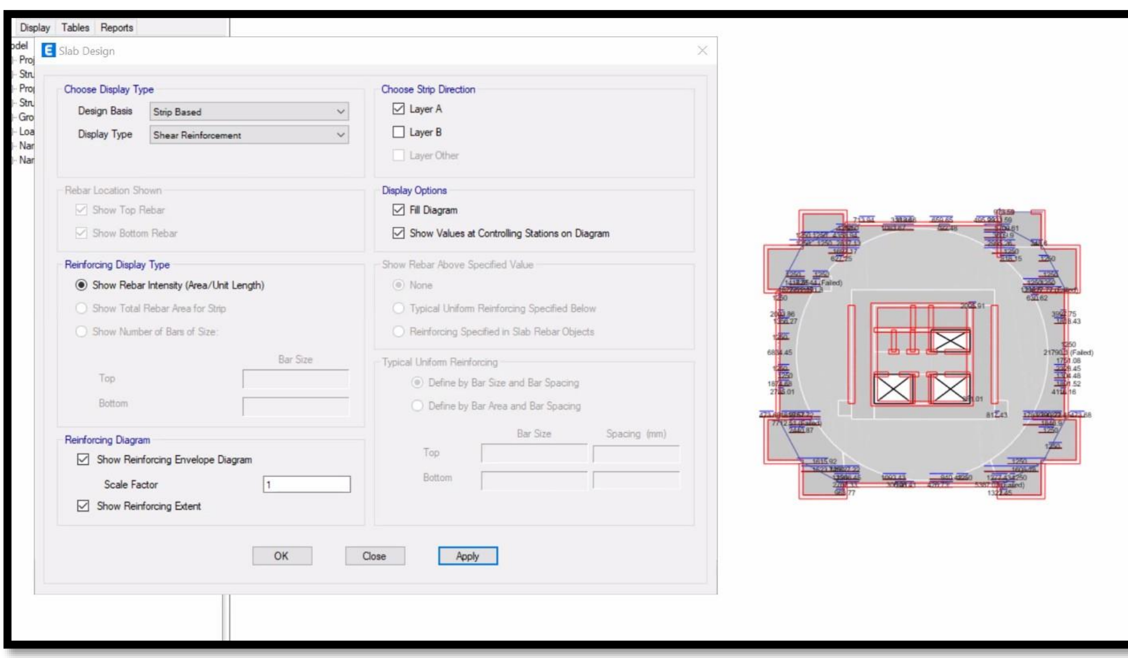


Figure 3.9.2.53 – Shear reinforcement in X-Direction for 96th Floor

Y-Direction

Bottom reinforcement is required for the whole slab and top reinforcement for a significant portion of the slab due to different moment signs from different combinations

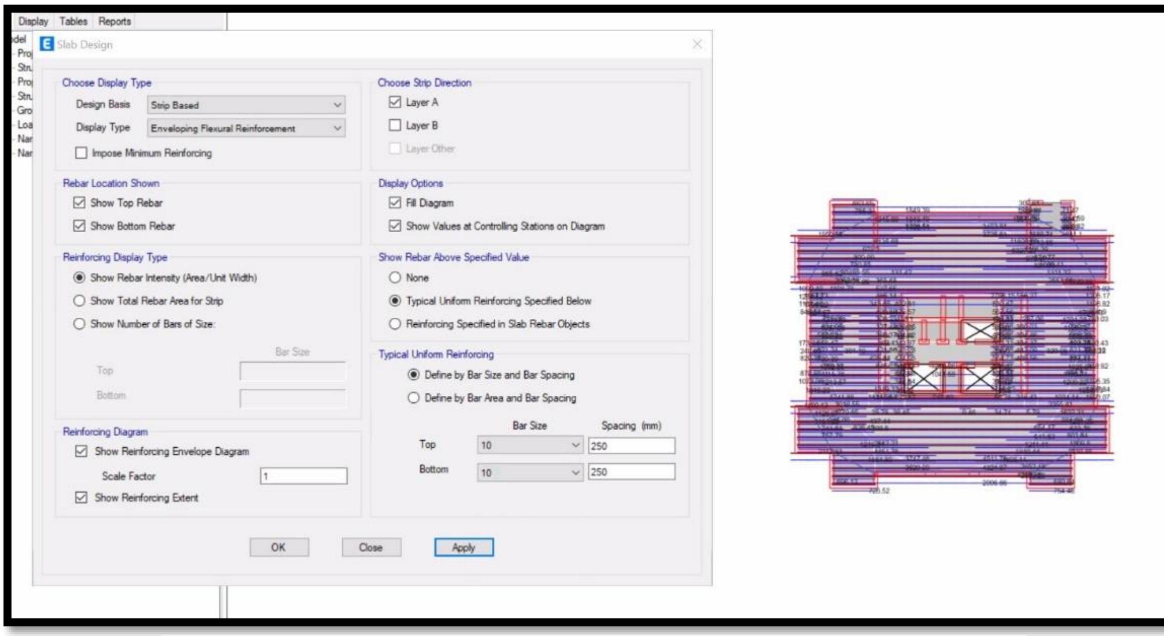


Figure 3.9.2.54 – Reinforcement Regions in Y-Direction for 96th Floor

Use Φ16/150 mm for bottom reinforcement for Non-Drop panels regions

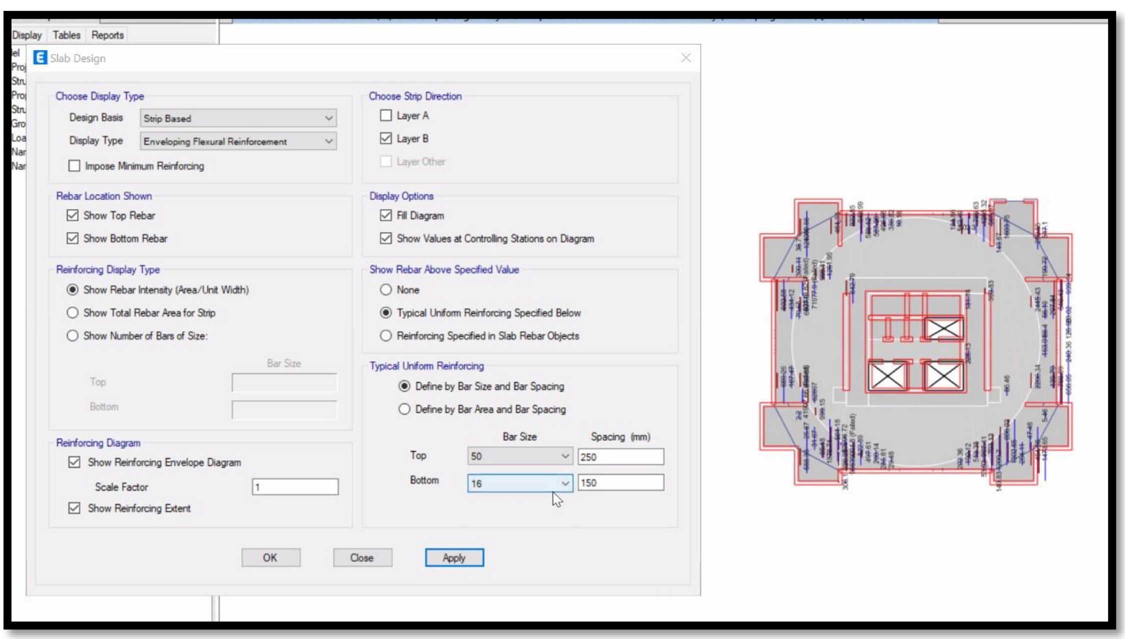


Figure 3.9.2.55 – Bottom Reinforcement in Y-Direction for Non-Drop Panels Regions in 96th Floor

Use Φ16/250 mm for top reinforcement for Non-Drop panels regions

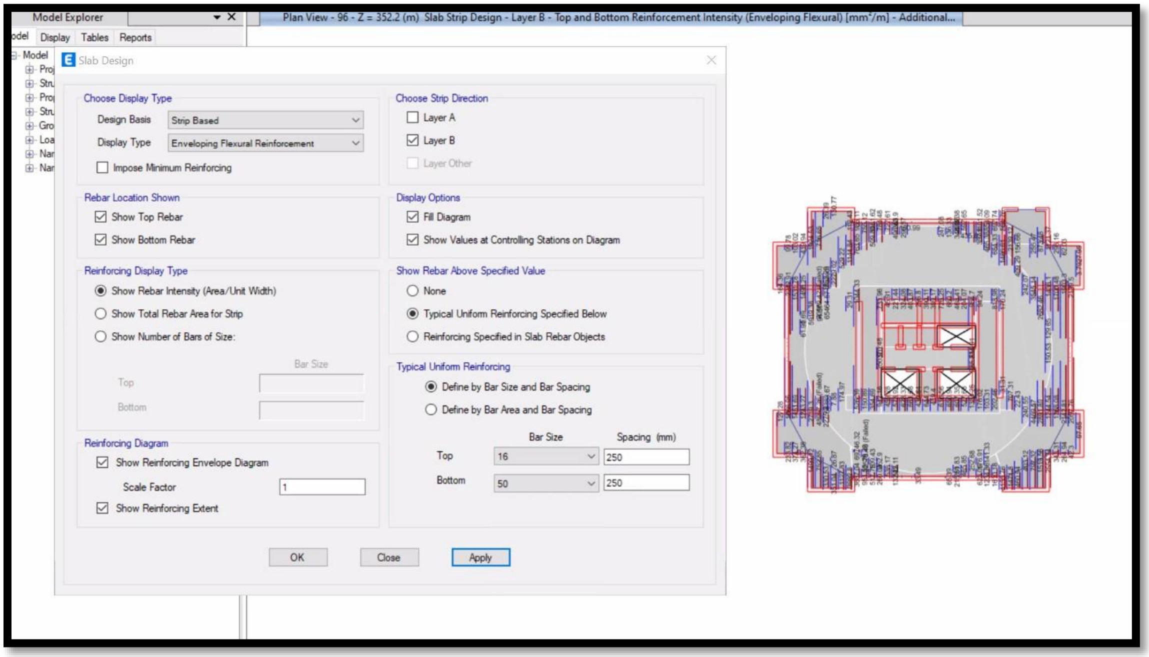


Figure 3.9.2.56 – Top Reinforcement in Y-Direction for Non-Drop Panels Regions in 96th Floor

Use Φ22/120 mm for bottom reinforcement for drop panels regions

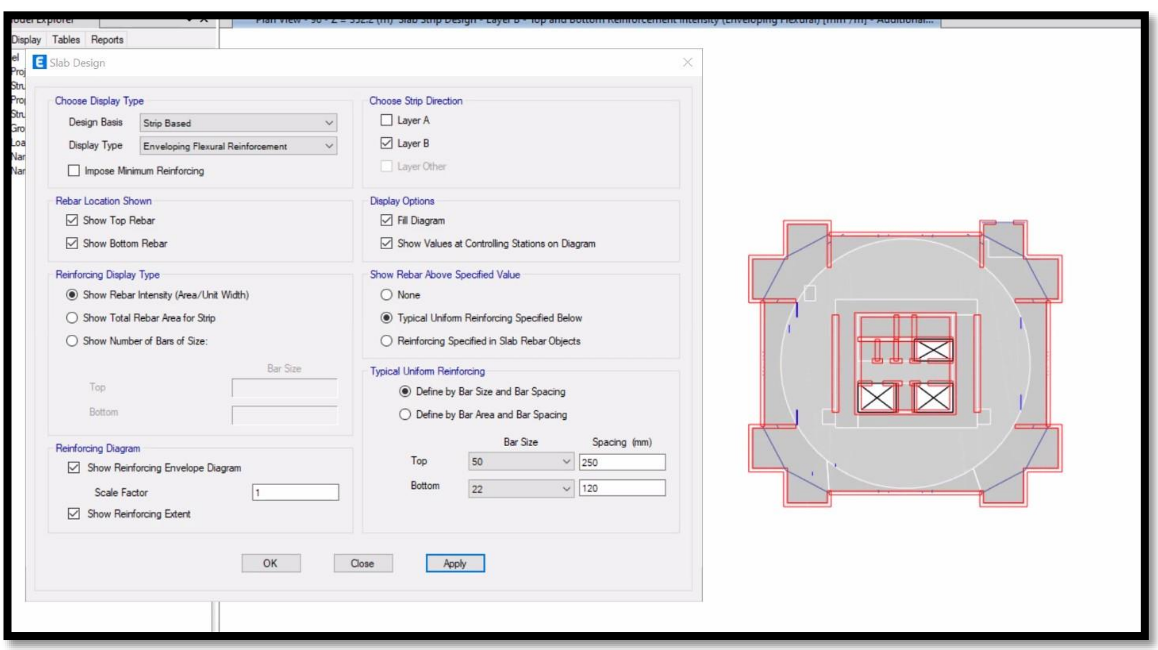


Figure 3.9.2.57 – Bottom Reinforcement in Y-Direction for Drop Panels Regions in 96th Floor

Use Φ25/120 mm for top reinforcement for drop panels regions

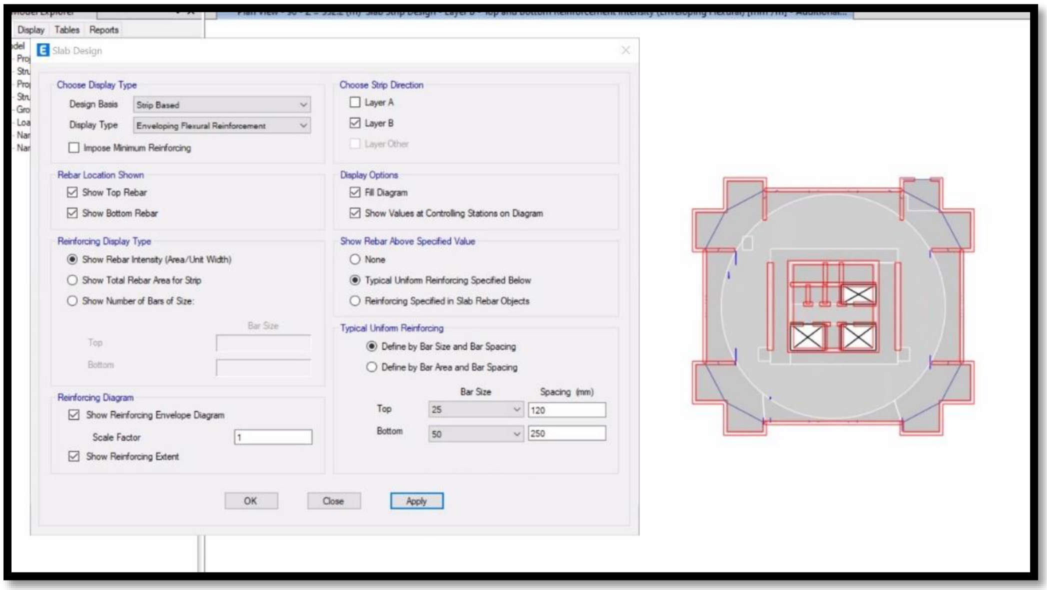


Figure 3.9.2.58 – Top Reinforcement in Y-Direction for Drop Panels Regions in 96th Floor

Shear Reinforcement

Shear Reinforcement is needed (stud rails)

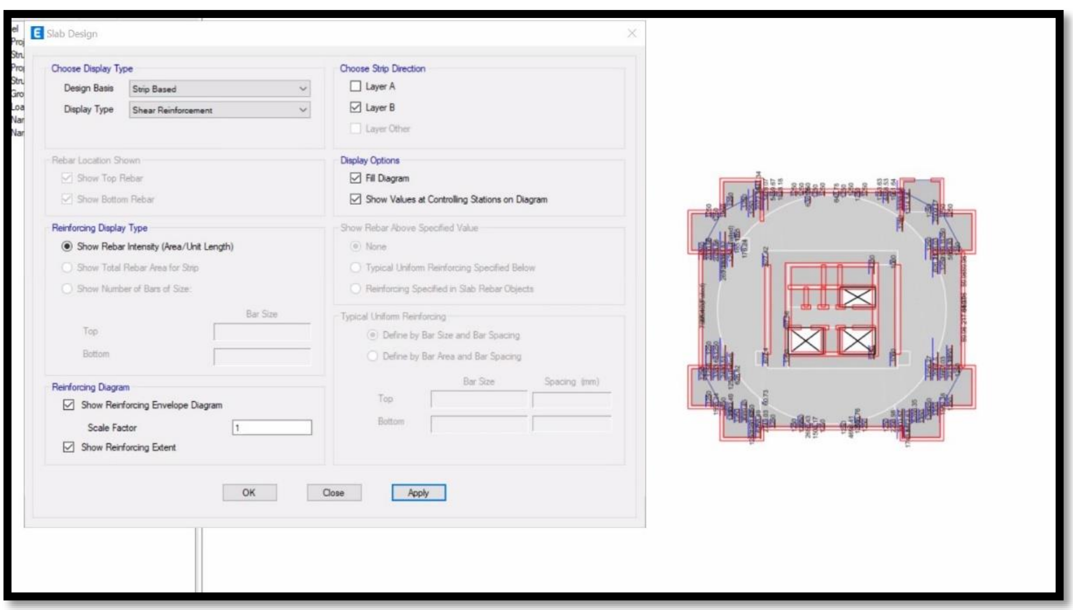


Figure 3.9.2.59 – Shear Reinforcement in Y-Direction for 96th Floor

Shear reinforcement needed for both directions:

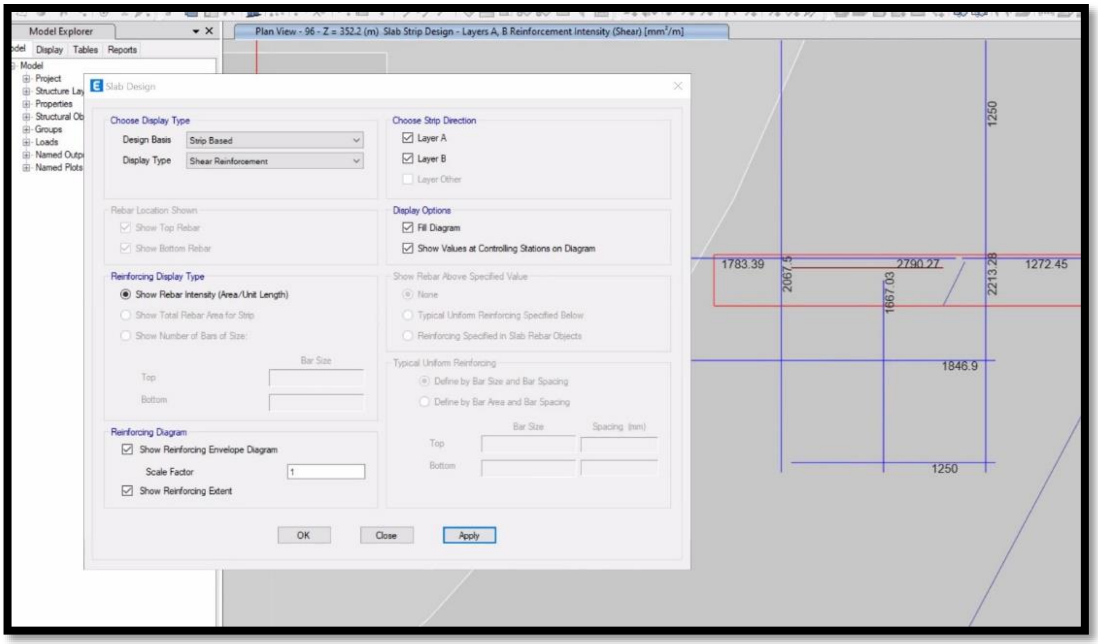


Figure 3.9.2.60 – Maximum Shear Reinforcement for Both Directions for 96th Floor

As noted from the figure above, the maximum shear reinforcement needed is 2791 mm^2/m

Take $A_v/s = 2.791\ mm^2/mm$

The studs' locations and spacings limits are shown in the table below

Table 8.7.7.1.2—Shear stud location and spacing limits

Direction of measurement	Description of measurement	Condition		Maximum distance or spacing, mm
Perpendicular to column face	Distance from column face to first peripheral line of shear studs	All		$d/2$
	Constant spacing between peripheral lines of shear studs	Nonprestressed slab with	$v_u \le 0.5\phi\sqrt{f_c'}$	$3d/4$
		Nonprestressed slab with	$v_u > 0.5\phi\sqrt{f_c'}$	$d/2$
		Prestressed slabs conforming to 22.6.5.4		$3d/4$
Parallel to column face	Spacing between adjacent shear studs on peripheral line nearest to column face	All		$2d$

Table 3.9.2.1 – Locations and Spacing Limits of Shear Studs as Per ACI 318 Code

Constant maximum spacing between peripheral lines of shear studs is d/2

$$\frac{d}{2} = \frac{1060}{2} = 530 \text{ mm}$$

Maximum spacing between adjacent shear studs on peripheral line nearest to column or wall face is 2d

2d = 2*1060 = 2120 mm

Maximum distance from the column or wall face to first peripheral line of shear studs is d/2

$$\frac{d}{2} = \frac{1060}{2} = 530 \text{ mm}$$

Use studs of diameter 30 mm at spacing of 250 mm and spacing between adjacent lines of 250 mm

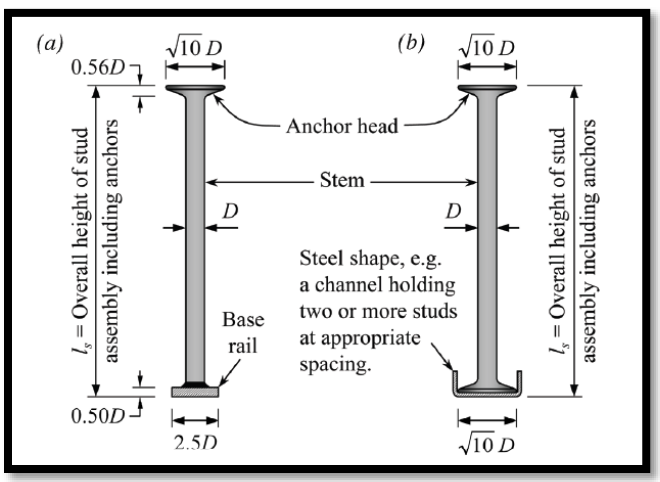


Figure 3.9.2.61 – Shear Stud Dimensions Recommendations

$$\text{Anchor head} = \sqrt{10}\text{D} = \sqrt{10} * 30 = 95 \text{ mm}$$

$$\text{Base rail} = 2.5\text{D} = 2.5 * 30 = 75 \text{ mm}$$

Thickness of anchor head = 0.56D = 0.56*30 = 16.8 mm take it as 20 mm

Thickness of base rail = 0.5D = 0.5*30 = 15 mm

$ls = h - C_t - C_b$

h: thickness of the slab

C_t: cover at top

C_b: cover at bottom

For studs at ring drop panel slab:

$l_s = 1100 - 40 - 40 = 1020$ mm

For studs at the floor slab:

$l_s = 550 - 40 - 40 = 470$ mm

7. Circular slab at 97th Floor

X-Direction

Use Φ18/120 mm for bottom reinforcement for all regions

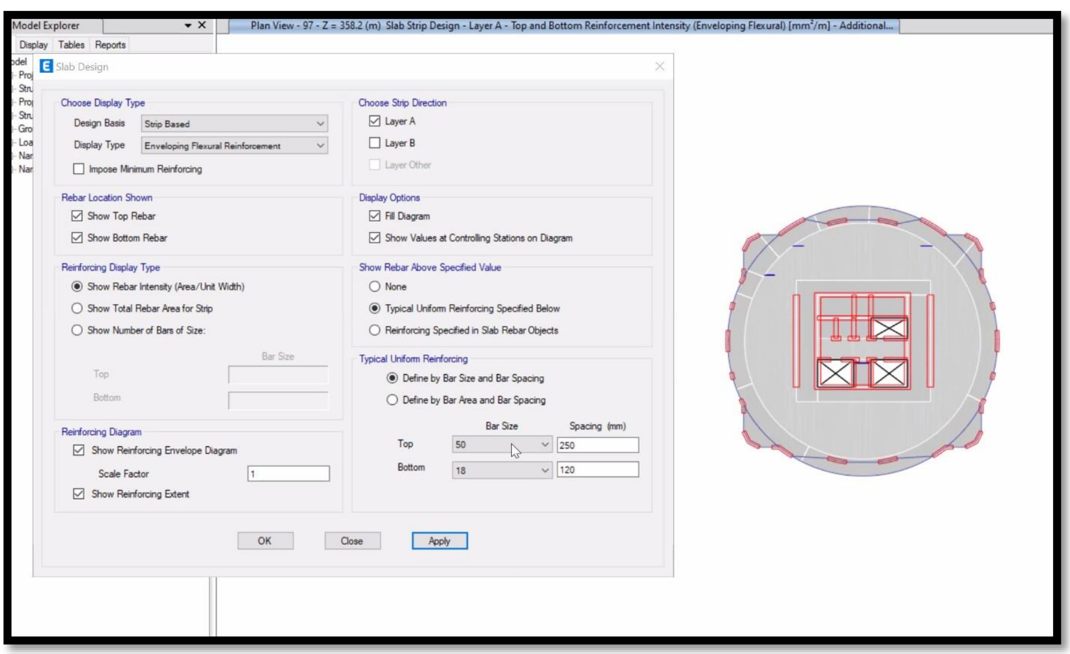


Figure 3.9.2.62 – Bottom Reinforcement in X-Direction for 97th Floor

Use Φ10/250 mm for top reinforcement for non-drop panel regions

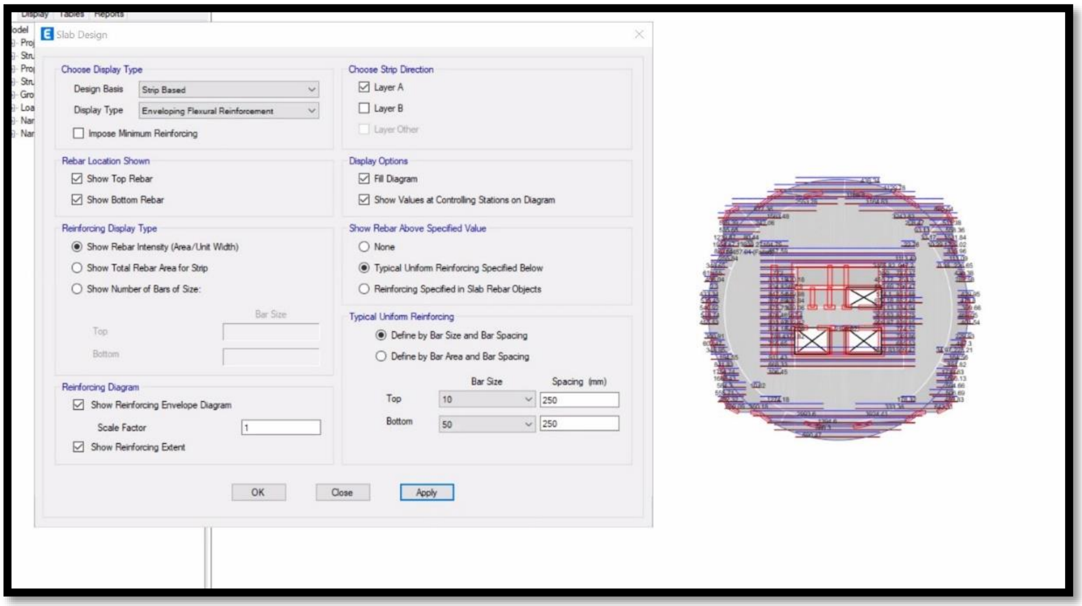


Figure 3.9.2.63 – Top Reinforcement in X-Direction for Non-Drop Panels Regions in 97th Floor

Use Φ20/150 mm for top reinforcement for drop panel regions

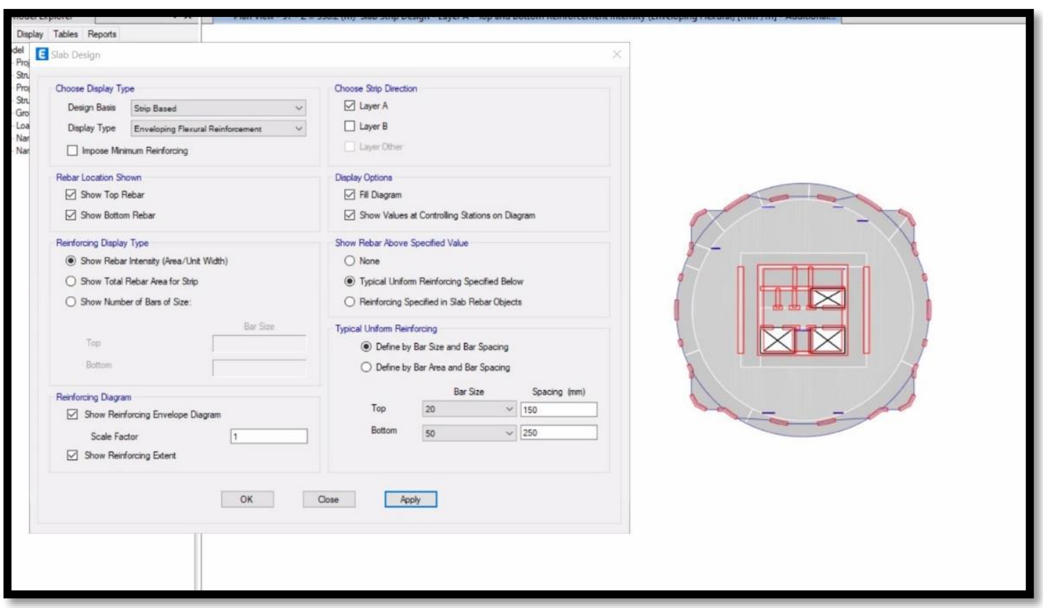


Figure 3.9.2.64 – Top Reinforcement in X-Direction for Drop Panels Regions in 97th Floor

Shear Reinforcement

Shear reinforcement is needed (stud rails)

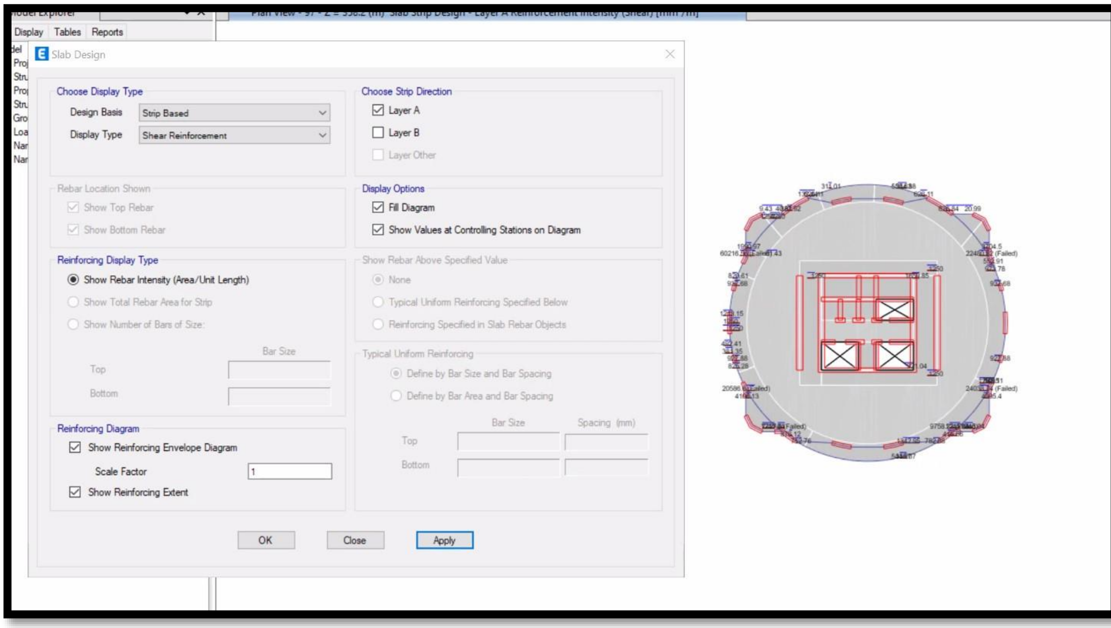


Figure 3.9.2.65 – Shear Reinforcement in X-Direction for 97th Floor

Y-Direction

Use Φ20/150 mm for bottom reinforcement for all regions

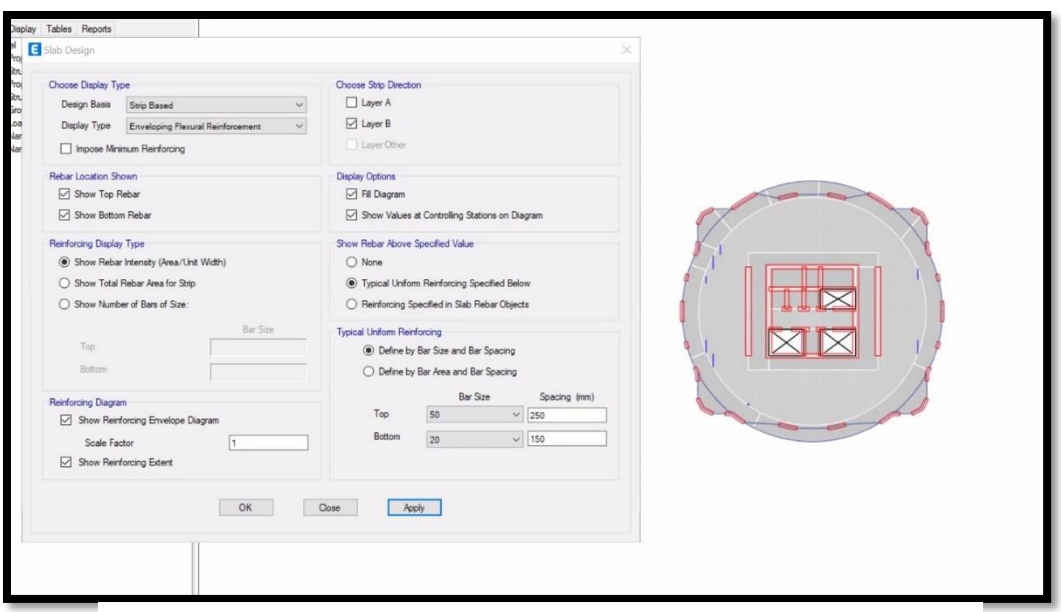


Figure 3.9.2.66 – Bottom Reinforcement in Y-Direction for 97th Floor

Use Φ12/250 mm for top reinforcement for Non-Drop panel regions

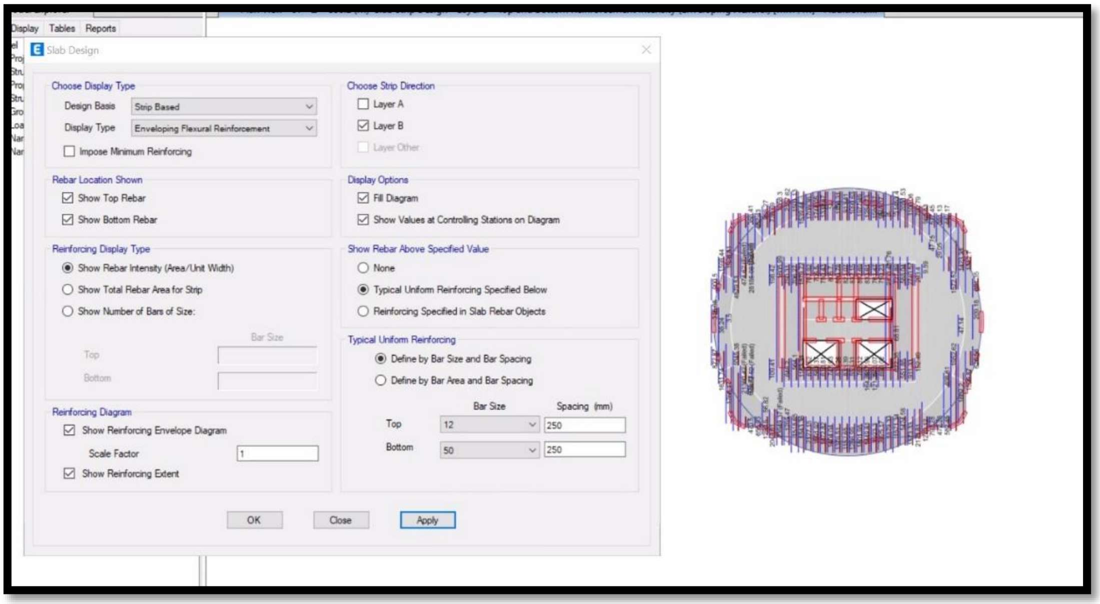


Figure 3.9.2.67 – Top Reinforcement in Y-Direction for Non-Drop Panel Regions in 97th Floor

Use Φ20/150 mm for top reinforcement for drop panel regions

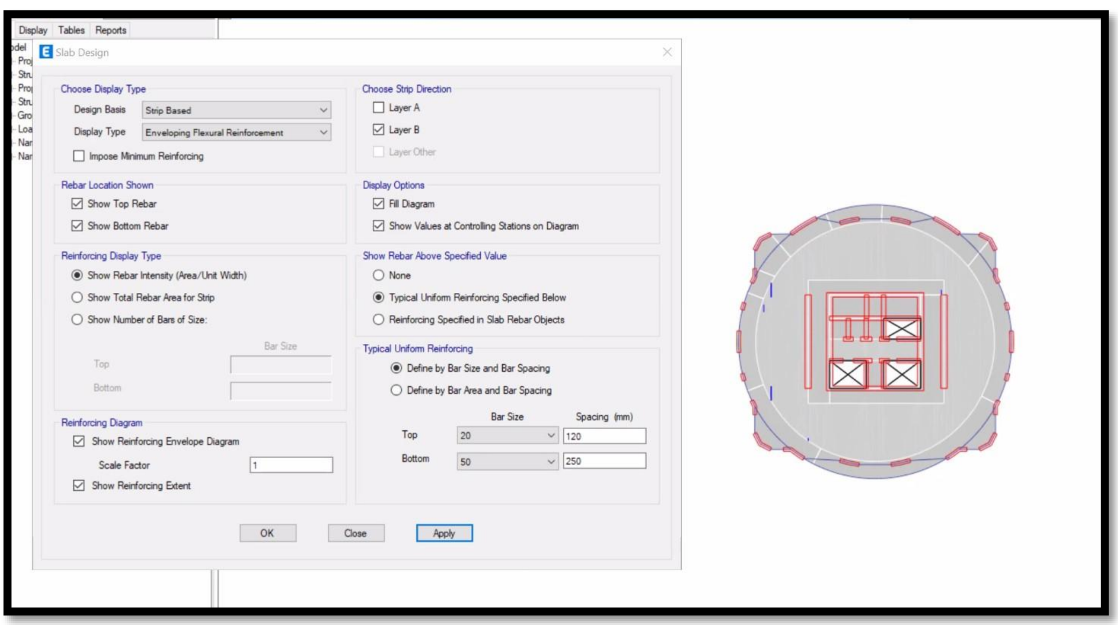


Figure 3.9.2.68 – Top Reinforcement in Y-Direction for Drop Panel Regions in 97th Floor

Shear Reinforcement

Shear reinforcement is needed (stud rails)

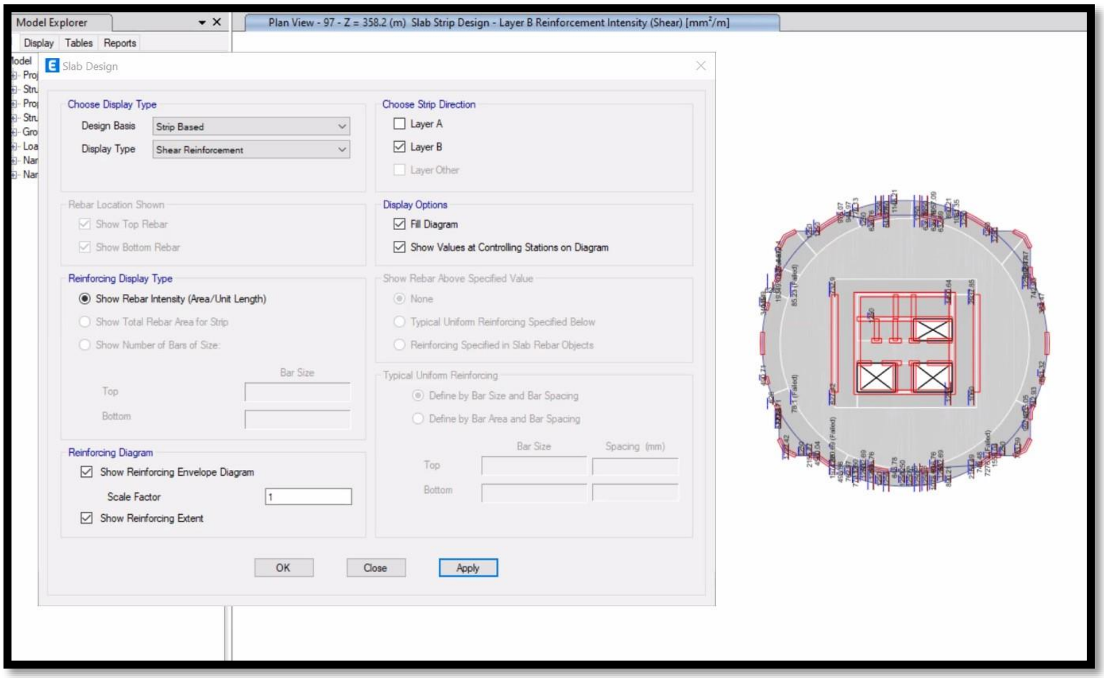


Figure 3.9.2.69 – Shear Reinforcement in Y-Direction for 97th Floor

Shear reinforcement for both directions

Shear reinforcement needed for both directions

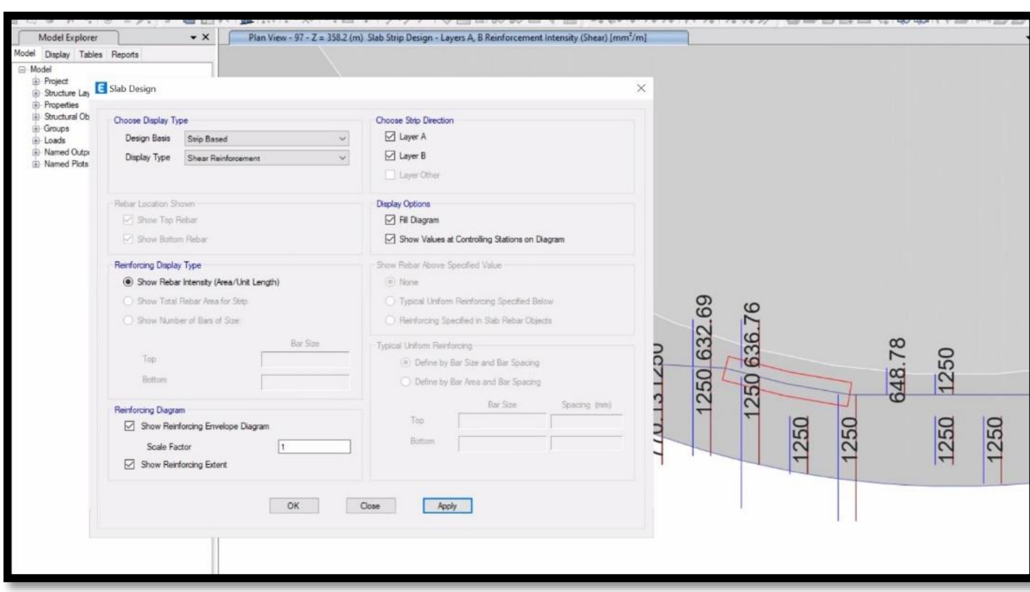


Figure 3.9.2.70 – Maximum Shear Reinforcement for 97th Floor

As noted from the figure above, the maximum shear reinforcement needed is 1250 mm^2/m

Take $A_v/s = 1.25\ mm^2/mm$

Constant maximum spacing between peripheral lines of shear studs is d/2

$$\frac{d}{2} = \frac{560}{2} = 280 \text{ mm}$$

Maximum spacing between adjacent shear studs on peripheral line nearest to column or wall face is 2d

2d = 2*560 = 1120 mm

Maximum distance from the column or wall face to first peripheral line of shear studs is d/2

$$\frac{d}{2} = \frac{560}{2} = 280 \text{ mm}$$

Use Studs of diameter 20 mm at spacing of 250 mm and spacing between adjacent lines of 250 mm

Anchor head $= \sqrt{10}D = \sqrt{10} * 20 = 63.2$ mm take it as 65 mm

Base rail $= 2.5D = 2.5 * 20 = 50$ mm

Thickness of anchor head = 0.56D = 0.56*20 = 11.2 mm take it as 15 mm

Thickness of base rail = 0.5D = 0.5*20 = 10 mm

$ls = h - C_t - C_b$

h: thickness of the slab

C_t: cover at top

C_b: cover at bottom

For studs at ring drop panel slab:

$l_s = 600 - 40 - 40 = 520$ mm

For studs at the circular slab:

$l_s = 300 - 40 - 40 = 220$ mm

8. Circular slab at 98th Floor

X-Direction

Use Φ18/150 mm for bottom reinforcement for all regions

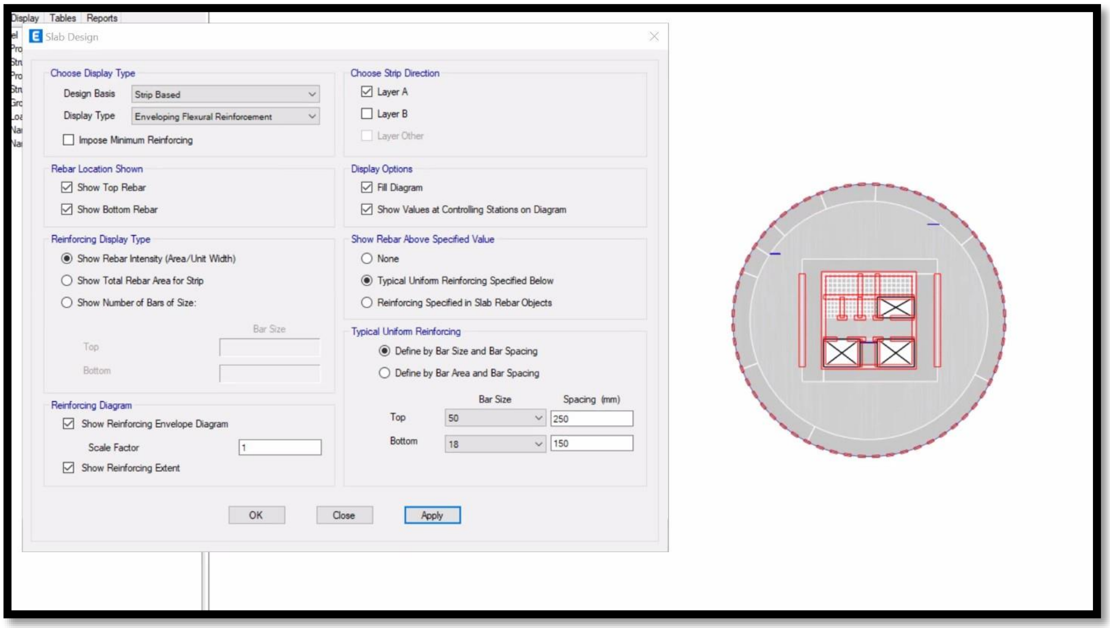


Figure 3.9.2.71 – Bottom Reinforcement in X-Direction for 98th Floor

Use Φ10/250 mm for top reinforcement for Non-Drop panel regions

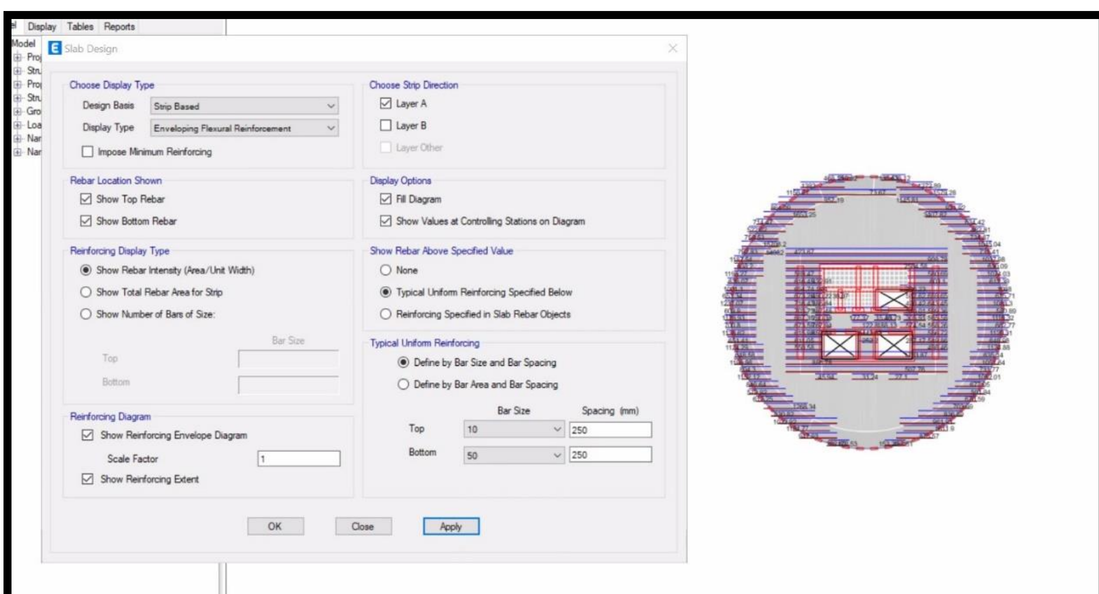


Figure 3.9.2.72 – Top Reinforcement in X-Direction for Non-Drop Panels Regions in 98th Floor

Use Φ18/120 mm for top reinforcement for drop panel regions

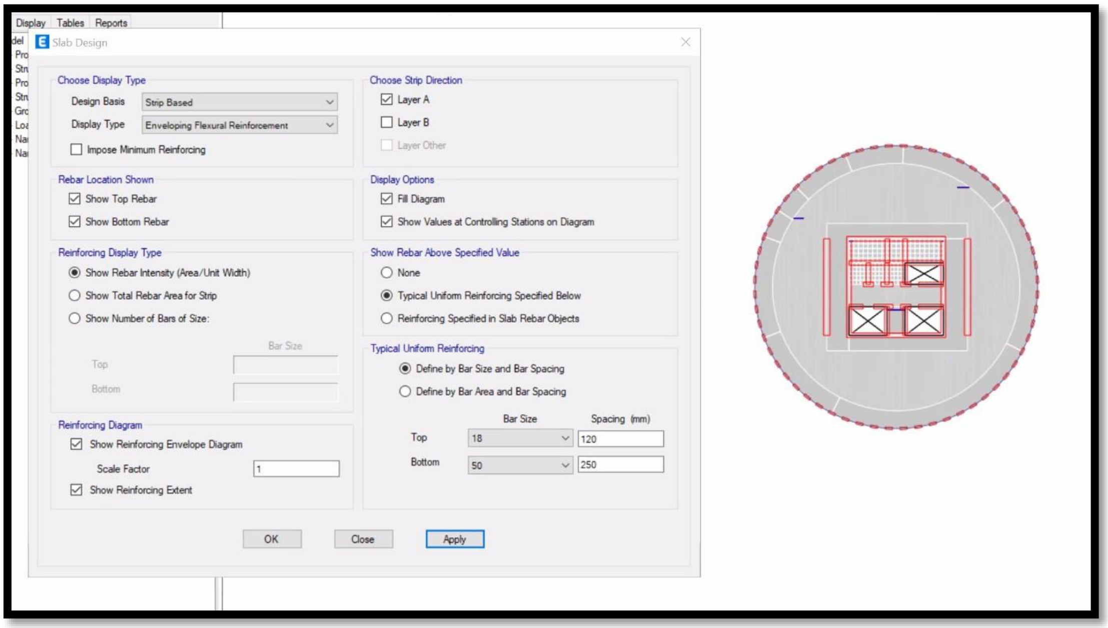


Figure 3.9.2.73 – Top Reinforcement in X-Direction for Drop Panels Regions in 98th Floor

Shear Reinforcement

No need for shear reinforcement

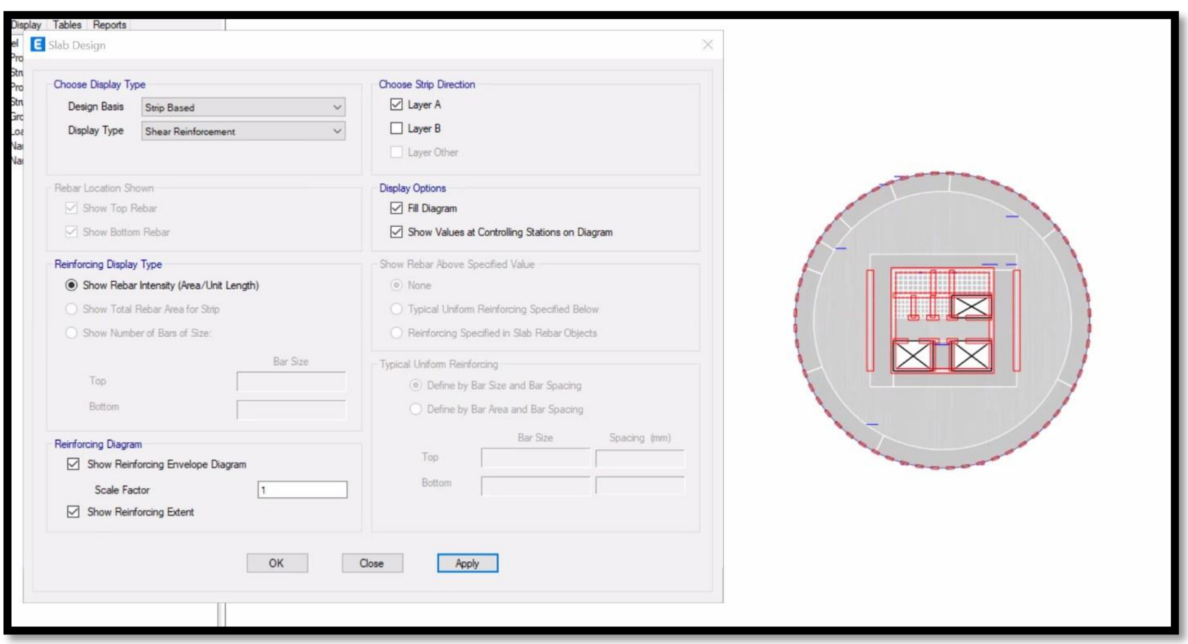


Figure 3.9.2.74 – Shear Reinforcement in X-Direction for 98th Floor

Y-Direction

Use Φ20/150 mm for bottom reinforcement for all regions

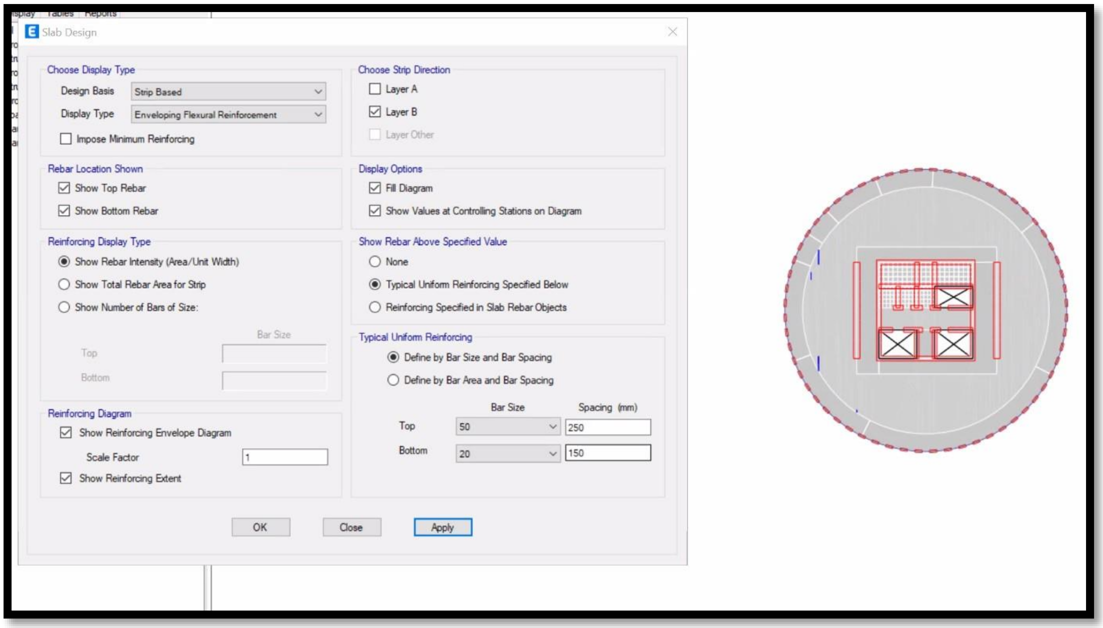


Figure 3.9.2.75 – Bottom Reinforcement in Y-Direction for 98th Floor

Use Φ12/250 mm for top reinforcement for Non-Drop panel regions

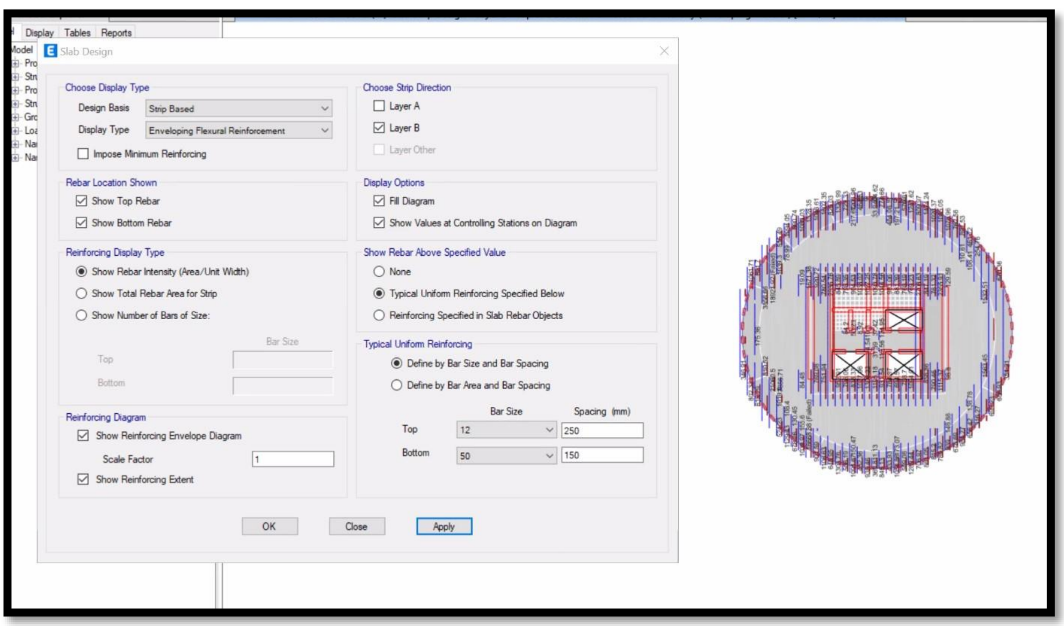


Figure 3.9.2.76 –Top Reinforcement in Y-Direction for Non-Drop Panels Regions in 98th Floor

Use Φ20/120 mm for top reinforcement for drop panel regions

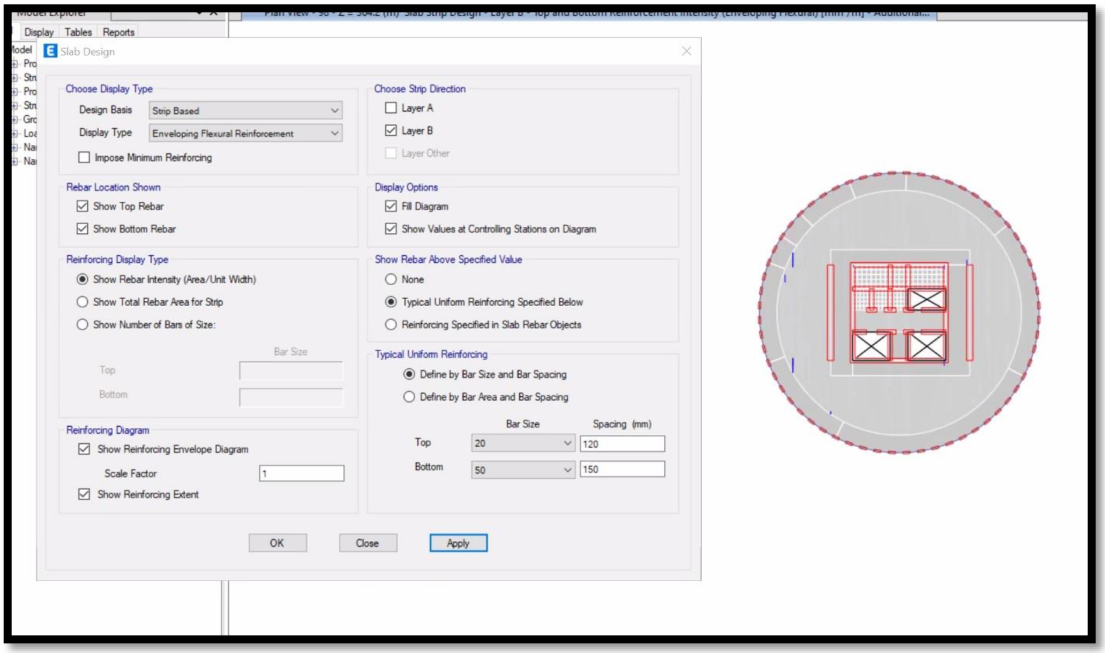


Figure 3.9.2.77 –Top Reinforcement in Y-Direction for Drop Panels Regions in 98th Floor

Shear Reinforcement

No need for shear reinforcement

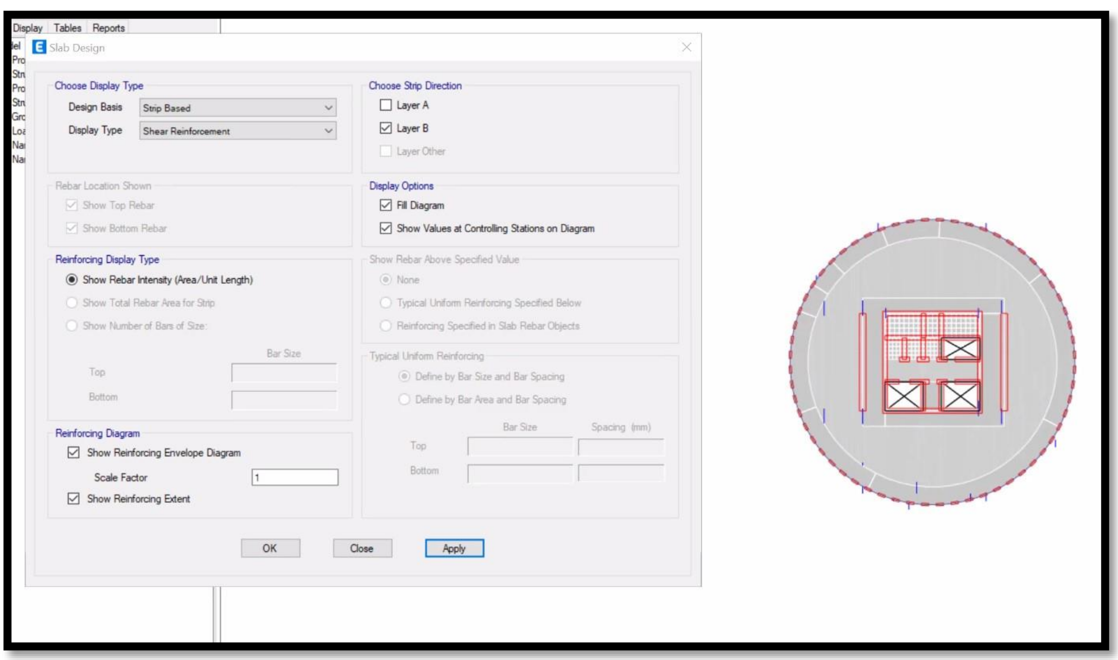


Figure 3.9.2.78 – Shear Reinforcement in Y-Direction for 98th Floor

Circular Slab at 99th Floor and 100th Floor

X-Direction

Use Φ16/120 mm for bottom reinforcement for all regions

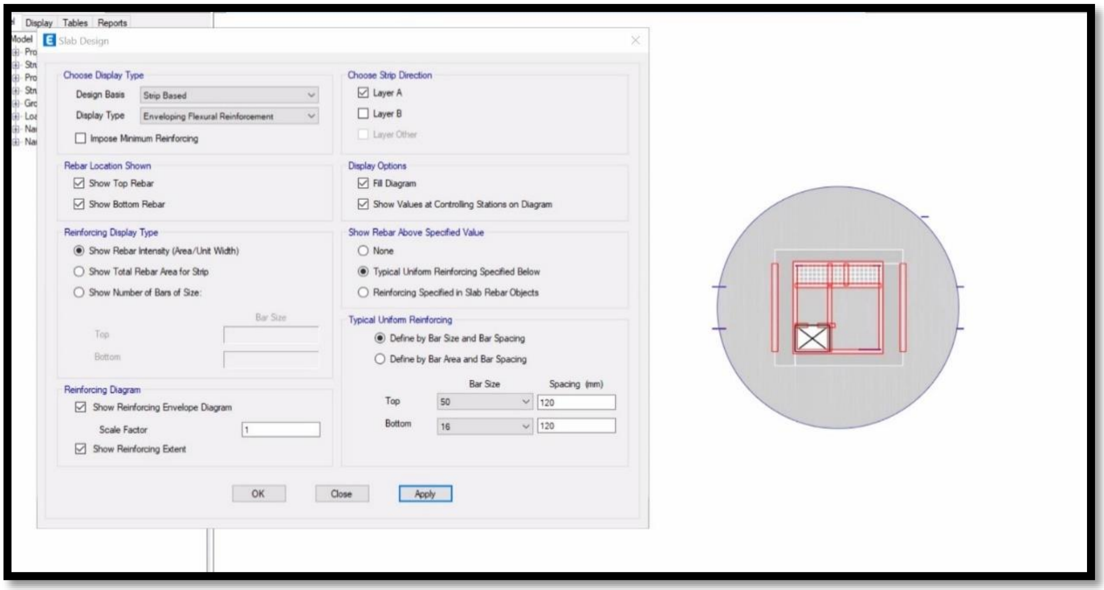


Figure 3.9.2.79 – Bottom Reinforcement in X-Direction for 99th Floor

Use Φ18/120 mm for top reinforcement for all regions

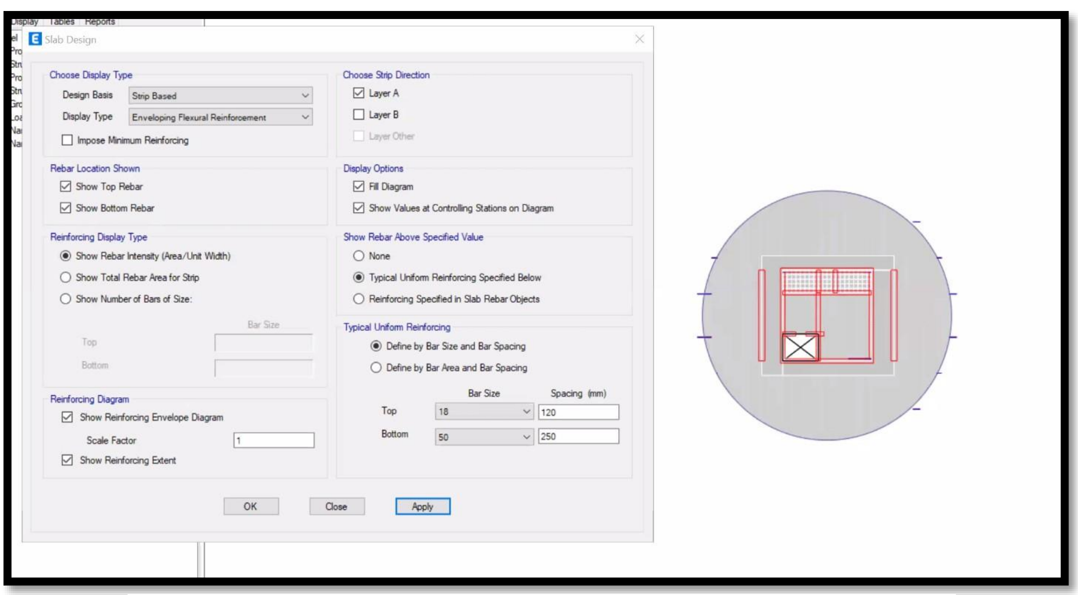


Figure 3.9.2.80 – Top Reinforcement in X-Direction for 99th Floor

Shear Reinforcement

No need for shear reinforcement

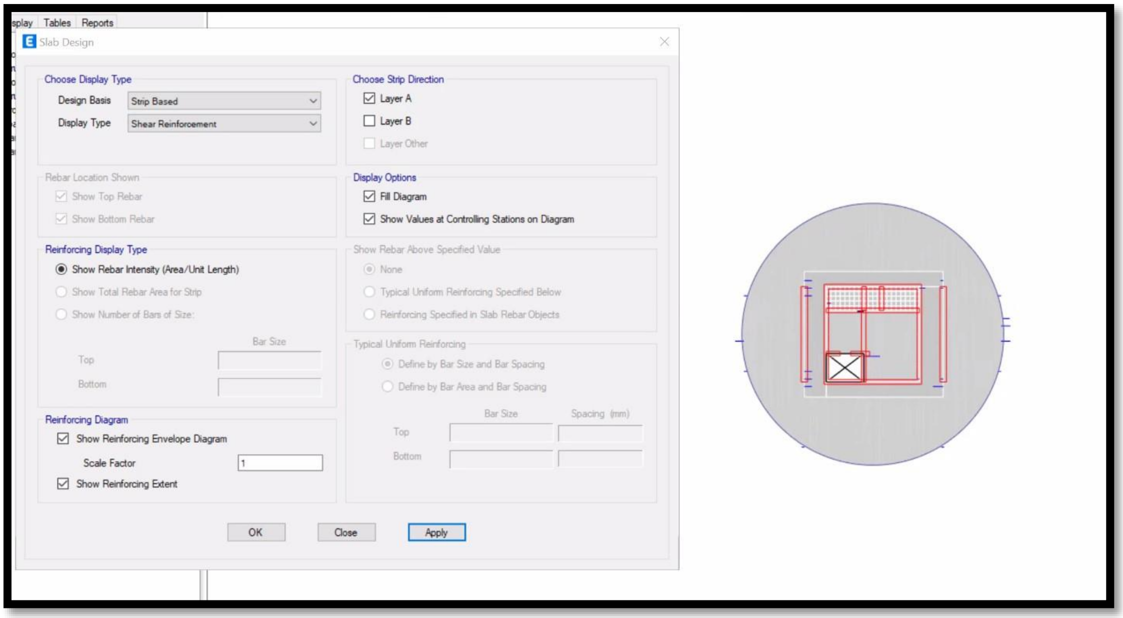


Figure 3.9.2.81 – Shear Reinforcement in X-Direction for 99th Floor

Y-Direction

Use Φ16/120 mm for bottom reinforcement for all regions

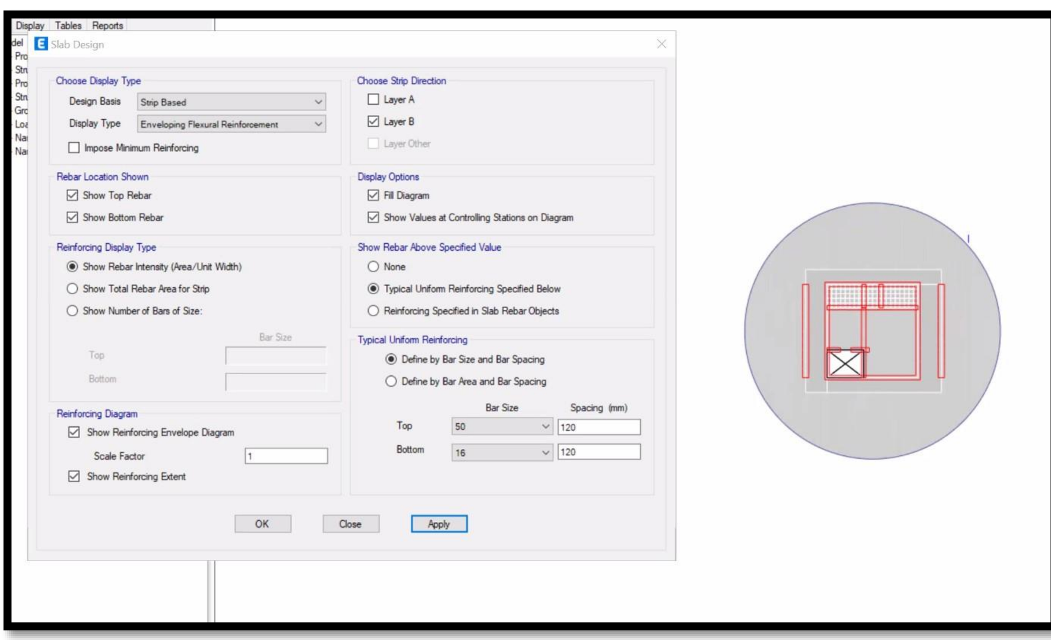


Figure 3.9.2.82 – Bottom Reinforcement in Y-Direction for 99th Floor

Use Φ18/120 mm for top reinforcement for all regions

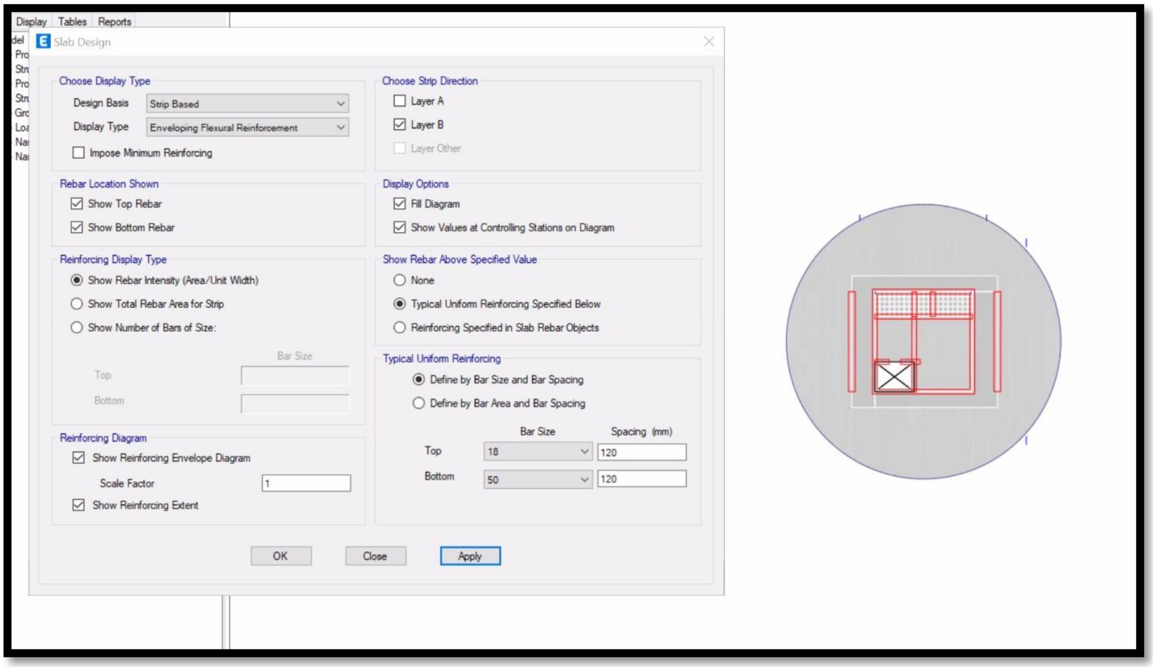


Figure 3.9.2.83 – Top Reinforcement in Y-Direction for 99th Floor

Shear Reinforcement

No need for shear reinforcement

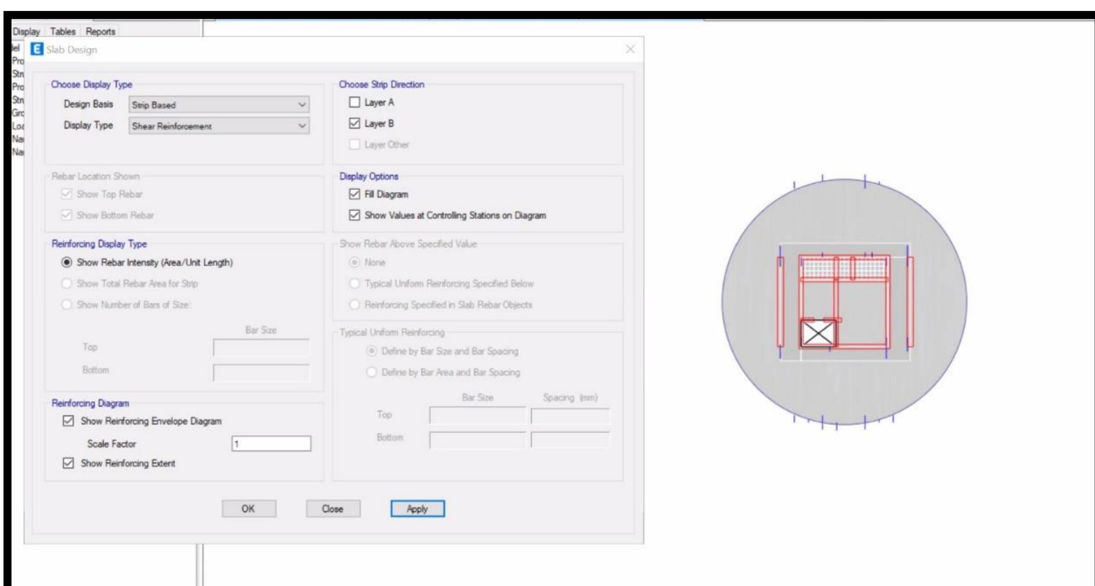


Figure 3.9.2.84 – Shear Reinforcement in Y-Direction for 99th Floor

3.10 Design of Foundations

3.10.1 General

Design of a safe and economical foundation system is an important task in tall building design. Deep foundations such as piled foundations are generally adopted to transfer heavy loads from superstructure to the bearing stratum. Providing adequate geotechnical capacity and limiting the deferential settlement are two important design considerations in the design of piled foundations. The foundation design becomes economical when both the criteria of bearing capacity and settlement are satisfied in an optimum way. A piled raft foundation is an advanced concept in which the total load coming from the superstructure is partly shared by the raft through bearing from soil and the remaining load is shared by piles through skin friction and end bearing. Consequently, piled raft system is generally adopted when pile foundations for tall buildings become uneconomical or unsatisfactory. Due to the three-dimensional nature of the load transfer, piled raft foundations are regarded as very complex systems involving many interaction factors such as pile-to-raft, raft-to-soil, and pile-to-soil.

As the inevitable results of rising population and growing land scarcity, high-rise buildings have become more predominant in many capitals in recent years. This phenomenon has forced the designers to design a suitable foundation system to satisfy the safety and economy for any kind of ground condition. The aspect of balancing the performance and cost of the foundation system is a challenge for the geotechnical and structural engineers. Also, geotechnical capacity and the settlement are the most key consideration during the designing of tall buildings. Due to the difficulty involved in the soil-structure interaction, proper foundation with systematic design guidelines, specified by both geotechnical and structural designers, is needed to predict the performance of the foundation system accurately. Raft, pile, and the piled raft foundations are the three major types of foundations commonly used in tall buildings. Generally, shallow foundations can be the most economical option for buildings loaded on subsoil with good load-bearing capacity. But the challenge in tall buildings is, due to heavy load from the super-structure, the thickness of the raft should be increased. It results in excessive settlement and expensive. Further, due to the relatively low thickness-to-width ratio, the raft foundations exhibit flexible characteristics to some extent even under stiff ground environments. Uneven loading and varying ground conditions may increase the flexible features and result the differential settlement. These effects made up the building foundation design a crucial aspect of the design process. Due to these limitations in the raft foundation systems, the idea of using piles as settlement-reducers started in the seventies.

The piled raft foundation is a combined system of piles and raft where both are partly shared loads of the superstructure. The Superstructure loads are shared between the piles using shaft friction and end bearing, while the raft supported on direct soil bearing. In this system the pile group typically carries about 80% of the total load directly into the deeper strata, and the raft carries the rest and transfers it to the beneath soil layer, but the soil bearing capacity is much smaller than the transferred pressure, so it is completely ignored in the foundation system, and the pile group is considered the only path for the loads to the ground, and the raft will play the role of confining the pile together and uniformly distribute the loads on them.

In this case of piled raft foundation system, three interactions are trendy namely,

1. Pile-Soil Interaction.
2. Pile-Pile interaction.
3. Pile-Raft Interaction.

In addition, the safety of the system depends on the combined system of raft, pile, and the soil instead of only in the pile group as in the pile foundation. Therefore, design and the analyses of the piled raft system seems complicated. Additionally, several issues are needed to be addressed when designing the piled raft foundation. Such as, ultimate load capacity for vertical, lateral loadings, maximum settlement, differential settlement, raft moments and shears for the structural design of the raft and pile loads and moments for the structural and geotechnical design of the piles.

3.10.2 Notes about Foundations Design

- The main factor that governed using this intensive number of piles is deflection, not the forces that are applied to the piles.
- According to Concrete Reinforcement Steel Institute (CRSI) Foundations Design Guide the embedment of piles reinforcement shall be a minimum of 4 inches, and a minimum cover of 3 inches for the mat foundation.
- There will be no punching shear check because punching will not occur due to not having any structural elements that have distance between them that is larger than the mat effective depth d which will result in merging the critical sections to form a large critical section that could resist the punching that is applying on the mat.

3.10.3 Deflection Check

The allowable maximum deflection according to the geotechnical investigation report is 200 mm

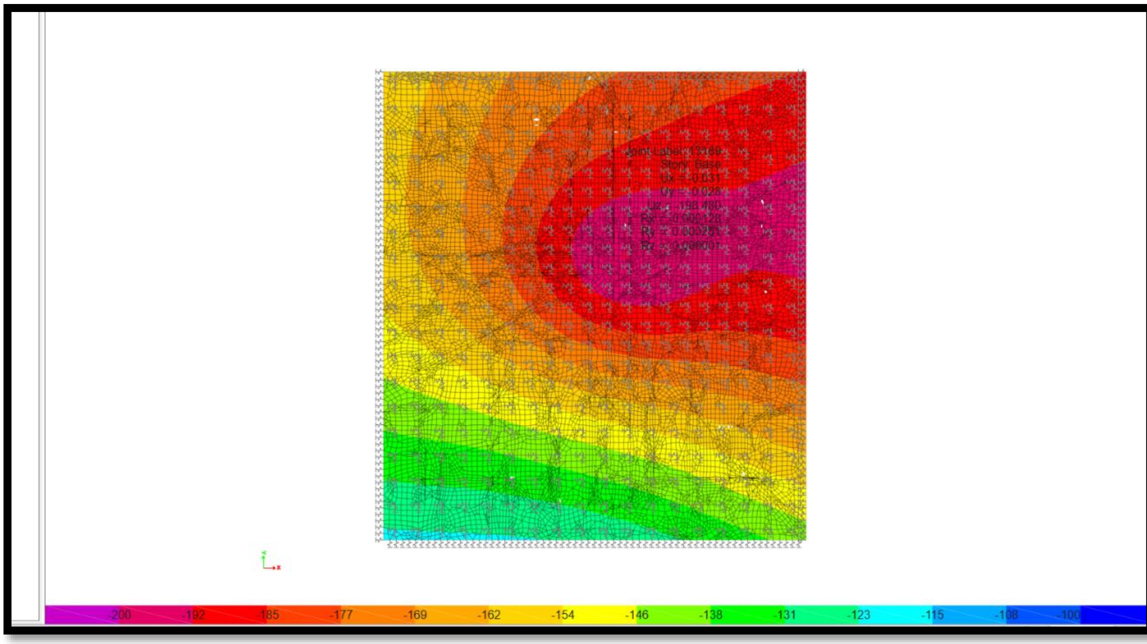


Figure 3.10.3.1 – Deflection in Mat Foundation

The maximum deflection is less than 200 mm

3.10.4 Mat Design

Flexural Design

Use Ø50/150 mm for the whole mat at bottom and top except for the inner core zone as shown

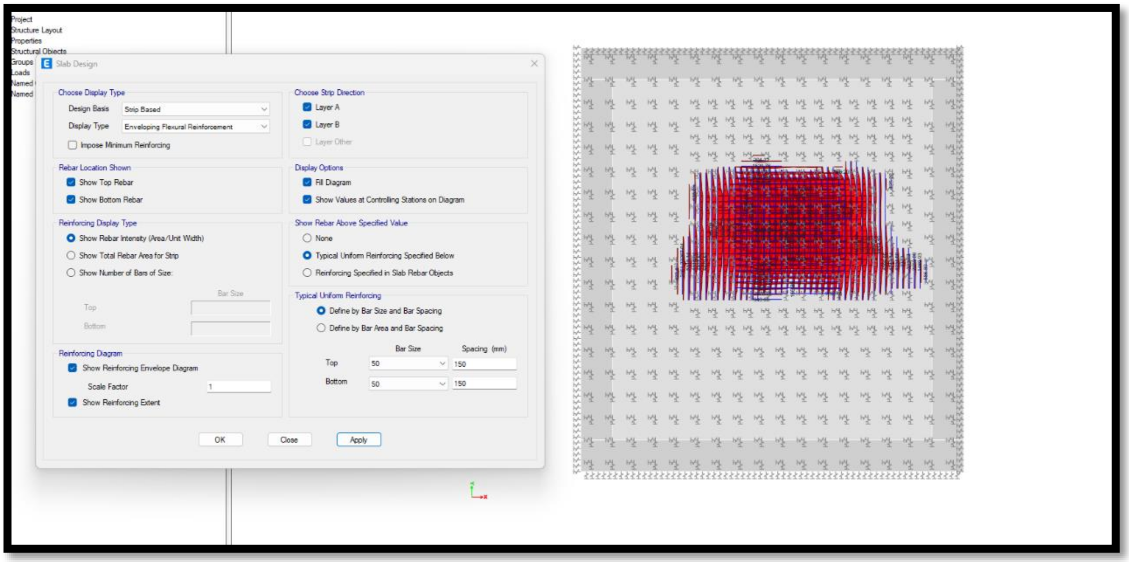


Figure 3.10.4.1 – Flexural Reinforcement for the Whole Footing

For the zone shown above

Use Ø50/200 mm for top reinforcement

Use double layers Ø50/120 mm for bottom reinforcement

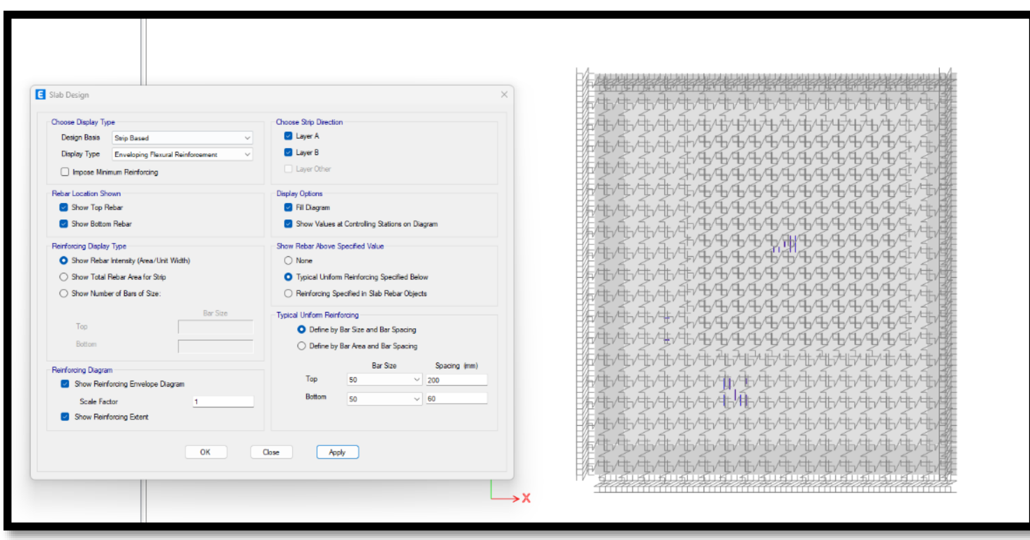


Figure 3.10.4.2 – Flexural Reinforcement for the Odd Zones in the Footing

Shear Design

Shear reinforcement is needed in the shown regions

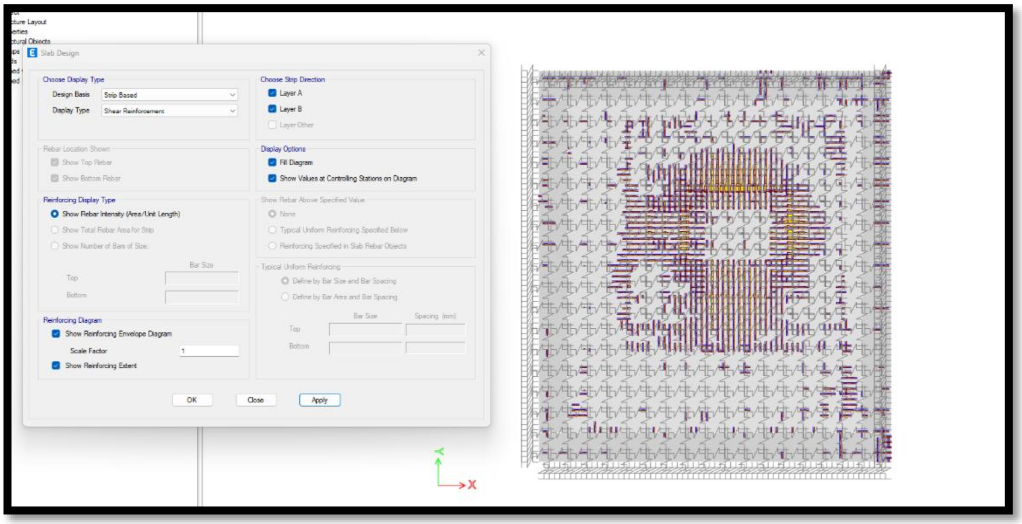


Figure 3.10.4.3 – Shear Reinforcement in the Mat Footing

The maximum shear reinforcement in both directions is 5655.56 mm^2/m

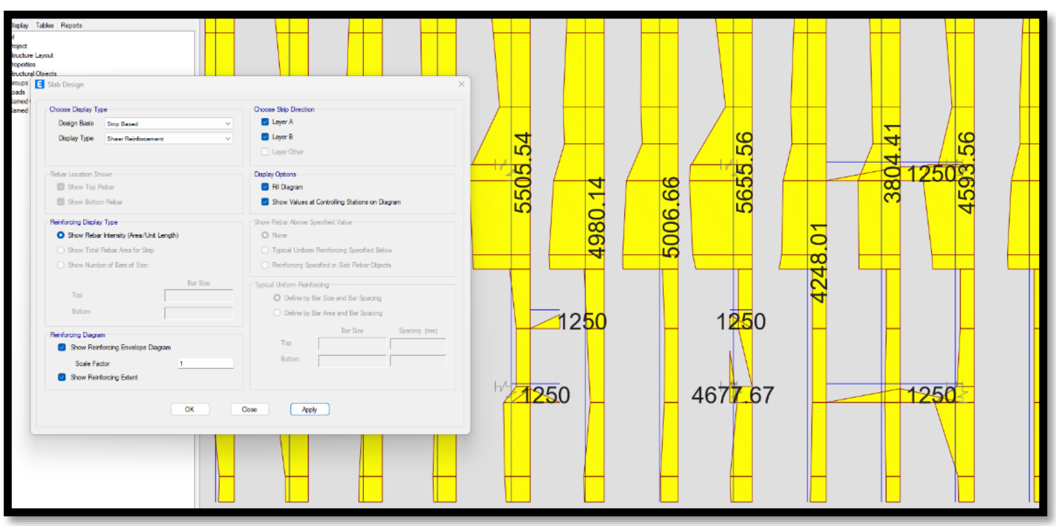


Figure 3.10.4.4 – Maximum Shear Reinforcement for Both Directions in the Mat

Use Ø40/200 mm vertical bars for both directions

3.10.5 Piles Design

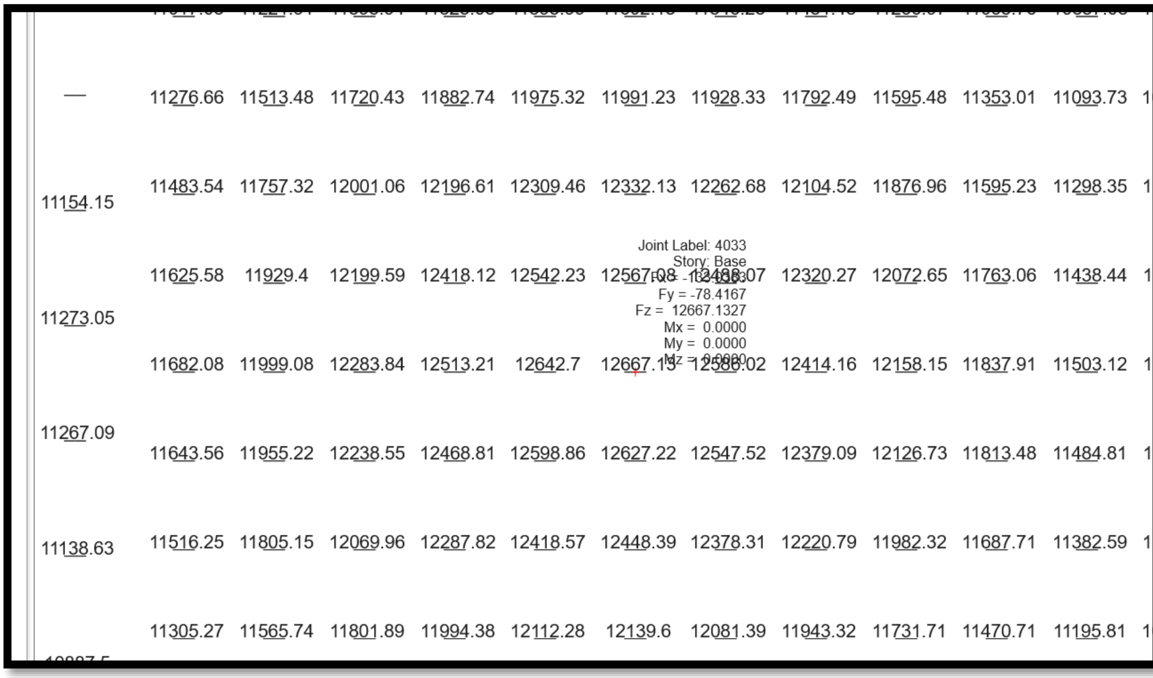


Figure 3.10.5.1 – Maximum Reaction at the Piles

The maximum axial force on the piles is 12667.1327 kN

Use minimum steel $\rho = 0.005$

$A_s = 0.005A_g$

$A_g = \pi*1200^2/4 = 1130973.36\ mm^2$

$A_s = 5655\ mm^2$

Use 20Ø20 at spacing of 165 mm

Minimum shear reinforcement = $0.062\sqrt{f'_c}\frac{D}{f_{yt}} = 0.062\sqrt{70}\frac{1200}{420} = 1.48\ mm^2/mm$

Use spirals of diameter 20 mm at spacing of 200 mm.

3.11 Performance Based Design

3.11.1 Introduction

In conventional elastic calculations practice, it has been created impression that the energy absorbing structures, the ability of ductile behavior in non-elastic stage, is able to withstand loads caused by earthquakes. But seismic events such as earthquakes in Northridge, California (1994), Kobe, Japan (1994), etc.., clarified the inefficiency of such existing methods. Thus, the engineering analysis of the consequences caused by earthquakes, structural damage control, respectively, initiated the idea of changing the existing design methodology. Such conclusions refer to Performance Based Design- BPD. To evaluate the performance of the structure, subjected to seismic action, the best way is non-linear time history analysis. This analysis can be accomplished and justified only in very special cases, and as such limits its application in daily practice or for solving normal engineering problems. Application of design principles need to be defined based on performance analysis procedures, bypassing the difficulties of calculating excess. Non-linear static procedures among others, are among the most rational, because they are simple to use and practical for solving of many engineering structures. Since the design is not focused on defining a single methodology of non-linear static procedures, the most suitable procedures that make use of analysis based on pushover, are summarized and clearly in many codes: FEMA 356 (ASCE, 2000), ATC-40 (ATC, 1996), FEMA 440 (ATC, 2005) and the EC 8 (ENV 1998-1, 1994).

3.11.2 Performance Based Seismic Engineering – PBSE

Design of building structures in seismic regions intended as a defense of their safe against possible earthquakes, seeking the correct engineering solution, but also economically acceptable. The experiences of many designs and constructions in seismic areas, but particularly the engineering analysis of the effects of real earthquakes remain major factors for the formulation, in principle, reasonable solutions, expressed in the basic requirements and the relevant anti-seismic design criteria. Any structural system is designed so that it has a capacity that exceeds seismic demand - previously known. In each concrete case, the ability of the designer lies in establishing the right relationship between reasonable seismic engineering requirements and structural capacity. But this should be a primary goal to determine the target performance of structure, choosing between options associated with a specific level of seismic hazard. The term concept of using "performance" relates to the provision of predetermined expected objectives of structural behavior during seismic operations data. Depending on the intensity of a significant earthquake, resulting seismic responses differentiated between themselves. Earthquake can be moderate, not strong, then the probability of major decline, but more important is the assessment of a possible strong earthquake or very strong, with low probability. In accordance with the intensity of earthquakes determined the so-called basic requirements and corresponding design criteria and the relevant boundary conditions.

Performance Goal has two essential parts: the damage state and a corresponding level of seismic hazard. So seismic performance described by reference to maximum allowed damages state (this is the level of performance), caused by a seismic risk level identified or expected ground motion during the earthquake. Among the most significant levels of structural performance are: immediate use of the building (Immediate Occupancy-IO); Safety of Life (Life Safety-LS) and collapse prevention (Collapse Prevention - CP).

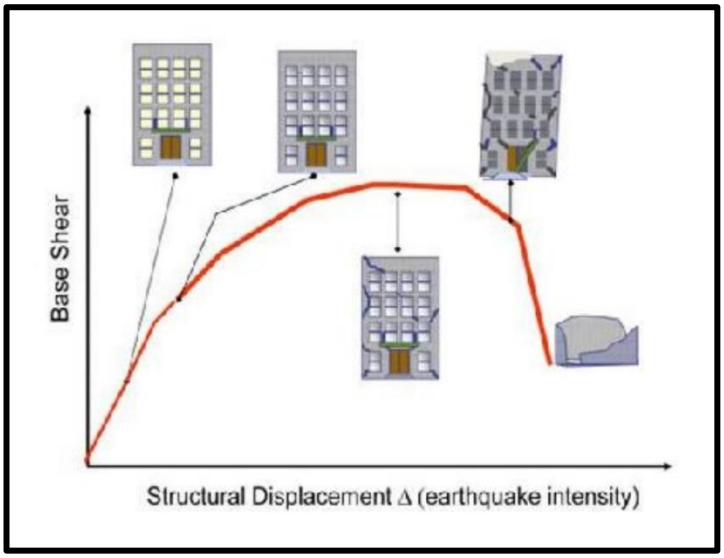


Figure 3.11.2.1 – Corresponding Levels of Damages

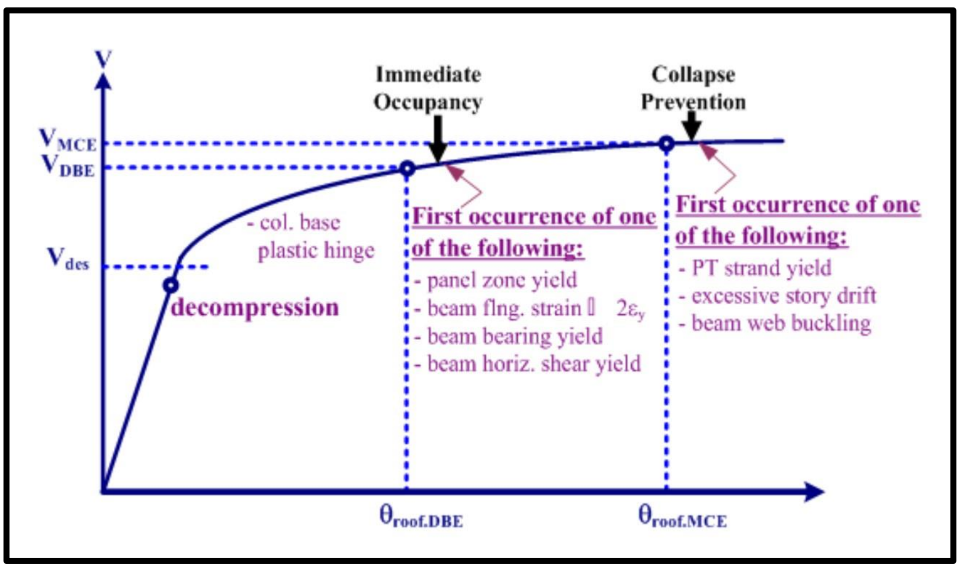


Figure 3.11.2.2 – Performance Levels and Corresponding Damage Levels

Expected performance can be assessed by comparing the seismic capacity requirements corresponding to the required level of performance. Thus, the global performance of the structure can be visualized graphically, comparing the capacity with the demand. On this basis, the analysis can be made of the expected damage to the structure. A detailed description of the damage done to the levels of performance, are given below:

Figure 3.11.2.3 – Performance Levels in Terms of Damages in Structural Elements

3.11.3 Nonlinear Static Analysis – Pushover Analysis

Referring to studies that have started application of Performance based Seismic Design, they have assessed, in the first place, the analysis of gradual loading–Pushover Analysis. Historically, this analysis is used in the 70's of last century. In the middle of 90s, the potential of pushover analysis is verified and found its way to the seismic requirements Guide SEAOC 1995, FEMA 273/274, 1997, 1997 ATC-40 Nowadays Pushover Analysis enjoys more popularity and is involved in almost modern design codes, such as FEMA 356/357 2000, ATC-55, 2005; FEMA-440 2006; EC-8 (ENV 1998 to 1.1994). Pushover analysis is in no way definitive answer to the problems of design - analysis, but is an important step forward, which gives importance to those elastic response characteristics that would distinguish between good performance and the poor, in the case of strong earthquakes.

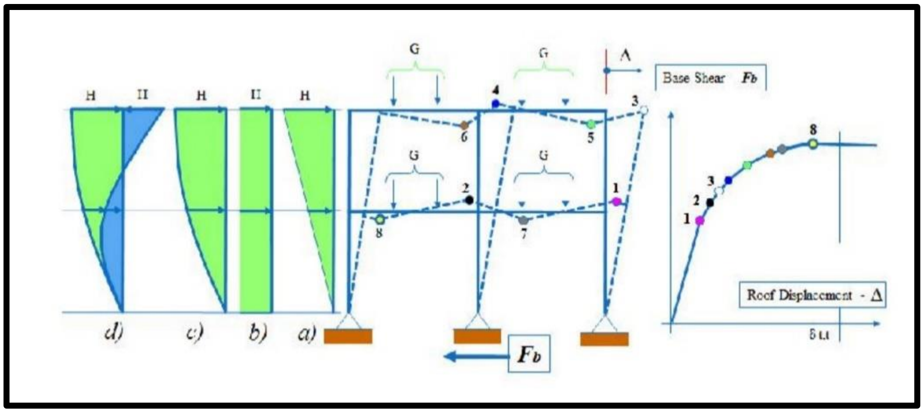


Figure 3.11.3.1 – Pushover Analysis Concept Horizontal loads and Plastic Mechanism

3.11.4 Implementation of Pushover Analysis on Princess Tower

Pushover analysis can be performed directly through the software that made structure modeling, ETABS in this scenario. However, the scheme of calculation and modeling assumptions accepted in non-elastic behavior of structural elements can affect the results of the analysis change with different software programs. Therefore, the basic principles of software programs used in pushover analysis should be well understood to interpret the results of this analysis.

Due to the long time taken by a nonlinear analysis when compared to a linear one, and due to the intensive data to be analyzed and computed in this project, pushover analysis will be performed in the X-direction only, and only for the basement floors shear walls that continue above to the typical floors. The following steps illustrate the implementation and definitions process of the pushover analysis.

Step I: Nonlinear Gravity and Pushover Load Cases Definition

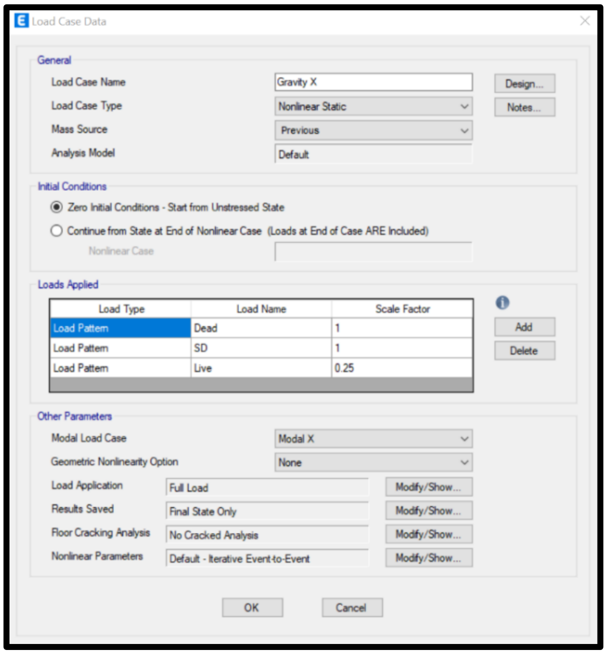


Figure 3.11.4.1 – Gravity X Load Case Definition Dialog Box in ETABS

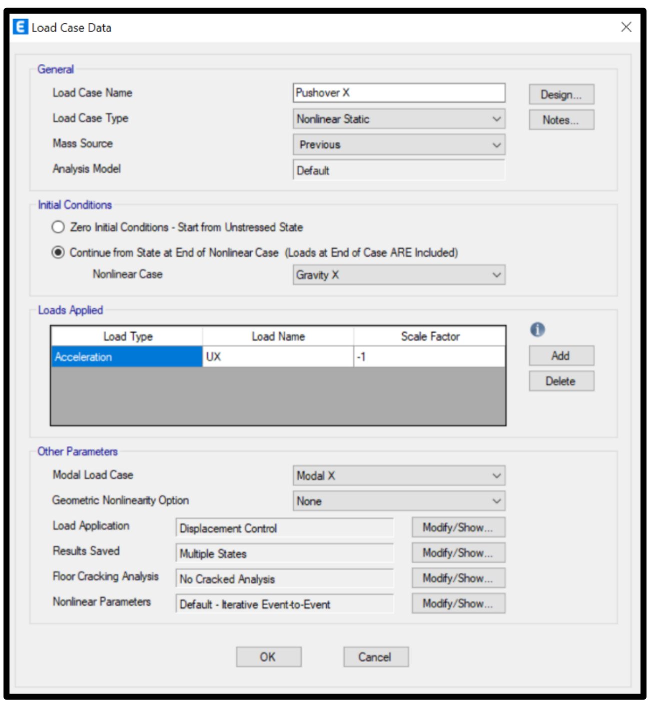


Figure 3.11.4.2 – Pushover X Load Case Definition Dialog Box in ETABS

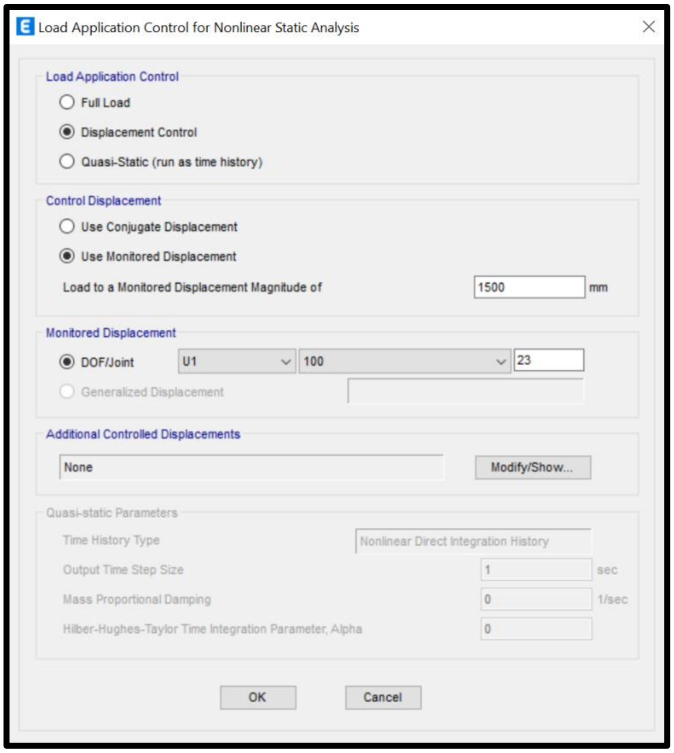


Figure 3.11.4.3 – Load Application Control for Pushover X Nonlinear Static Analysis Dialog Box in ETABS

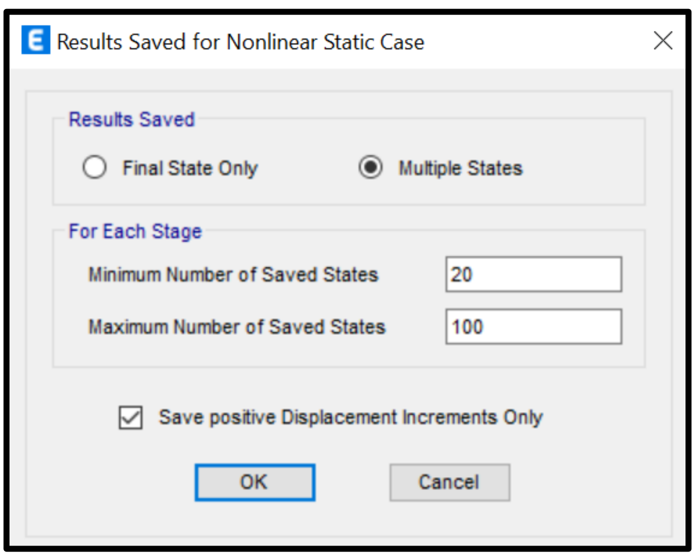


Figure 3.11.4.4 – Results Saved for Pushover X Nonlinear Static Analysis Dialog Box in ETABS

Step II: Shear Walls Hinges Assignment

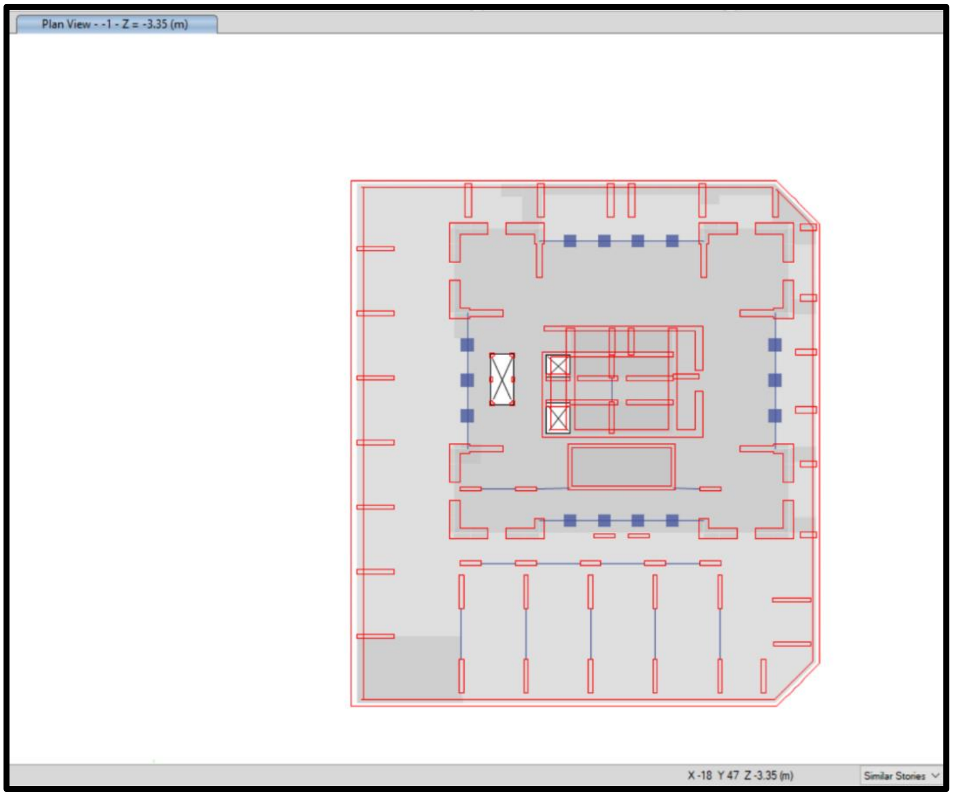


Figure 3.11.4.5 – 1st Basement Floor Plan with (Similar Stories) Option Activated

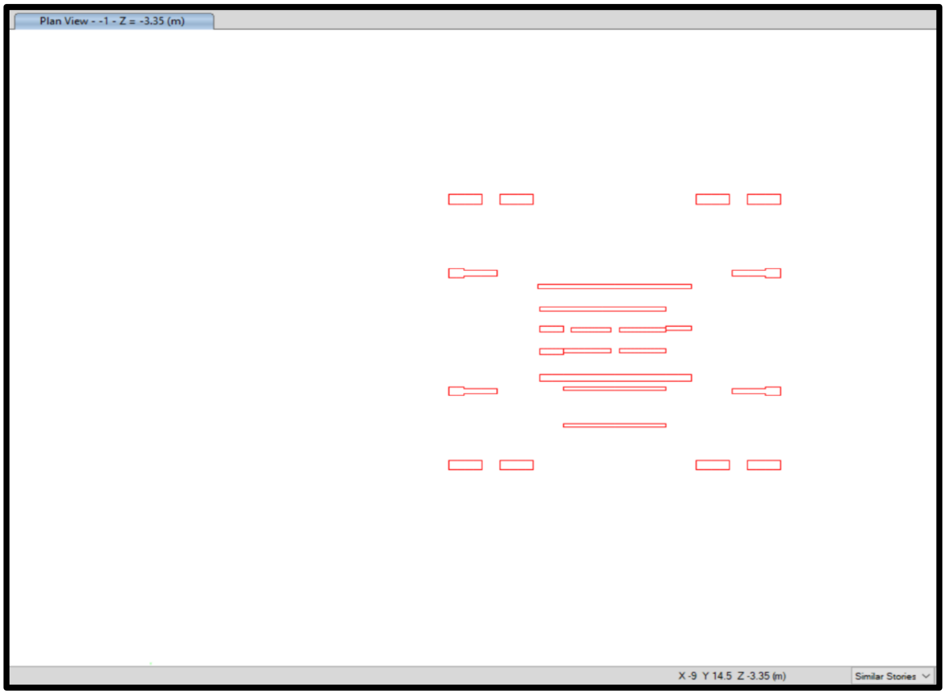


Figure 3.11.4.6 – 1st Basement Floor Shear Walls to be Assigned with (Similar Stories)

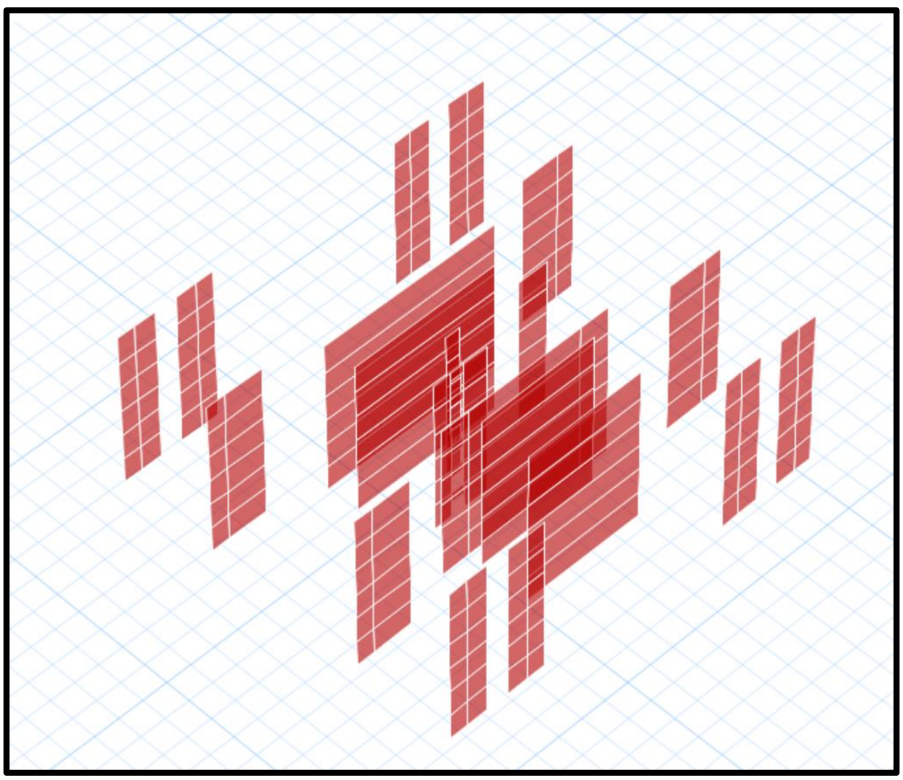


Figure 3.11.4.7 – 3D View of Basement Floors Shear Walls to be Assigned

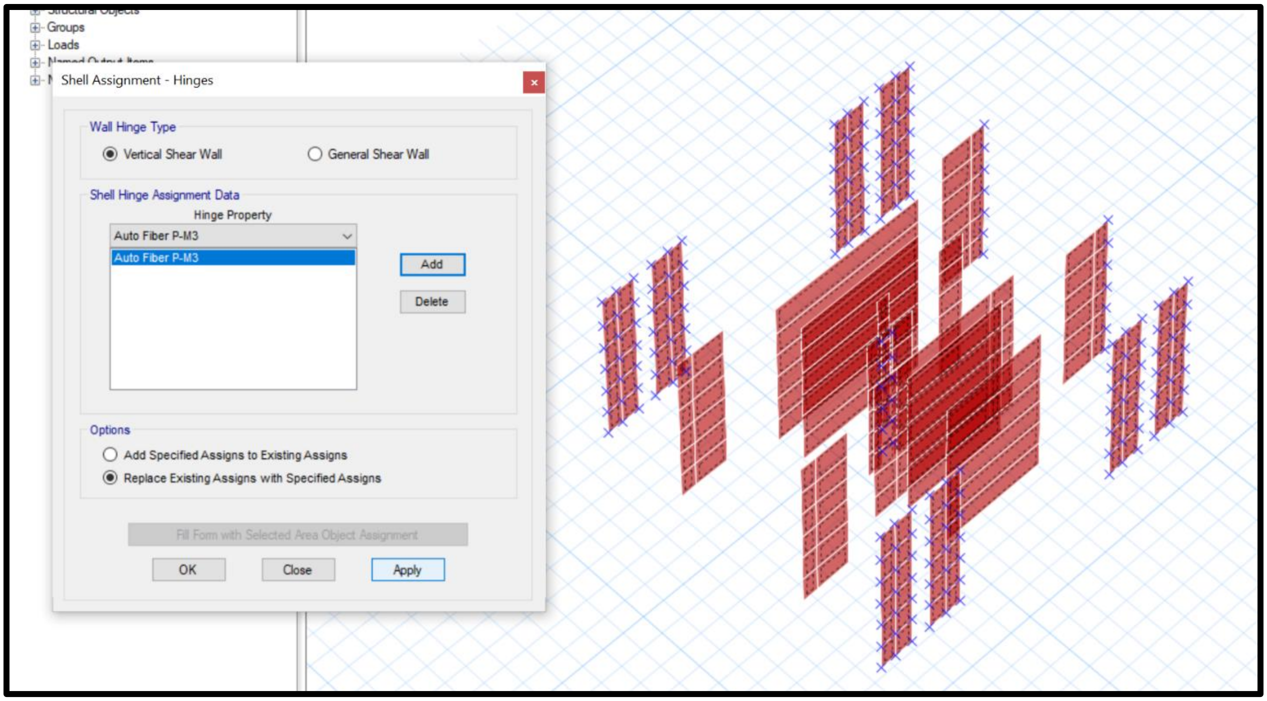


Figure 3.11.4.8 – Shell Hinges Assignment Dialog Box in ETABS

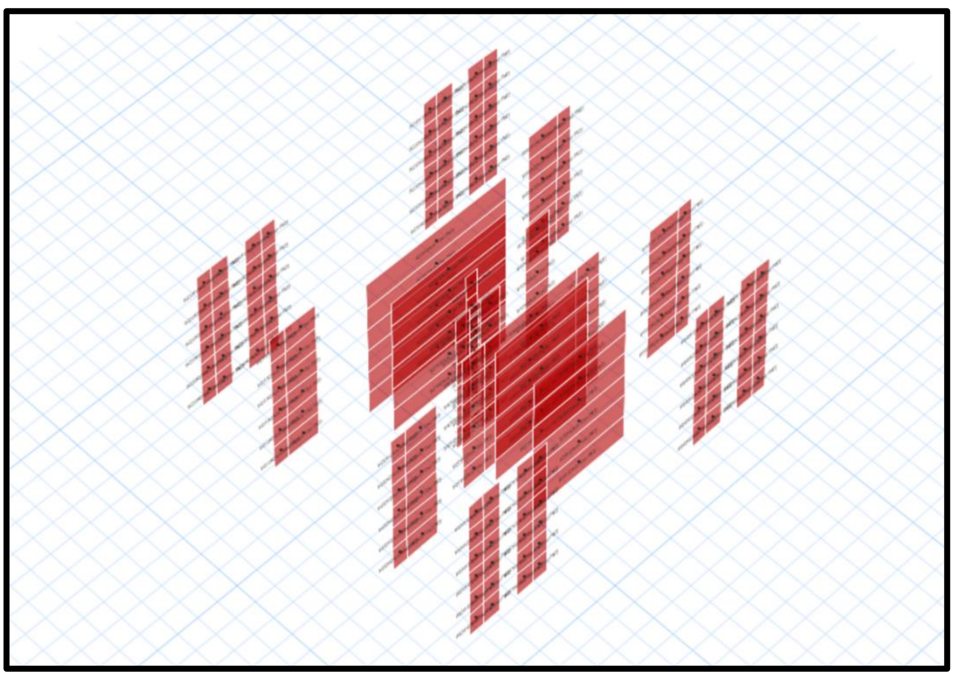


Figure 3.11.4.9 – Shear Walls Assigned with Fiber P-M3 Hinges

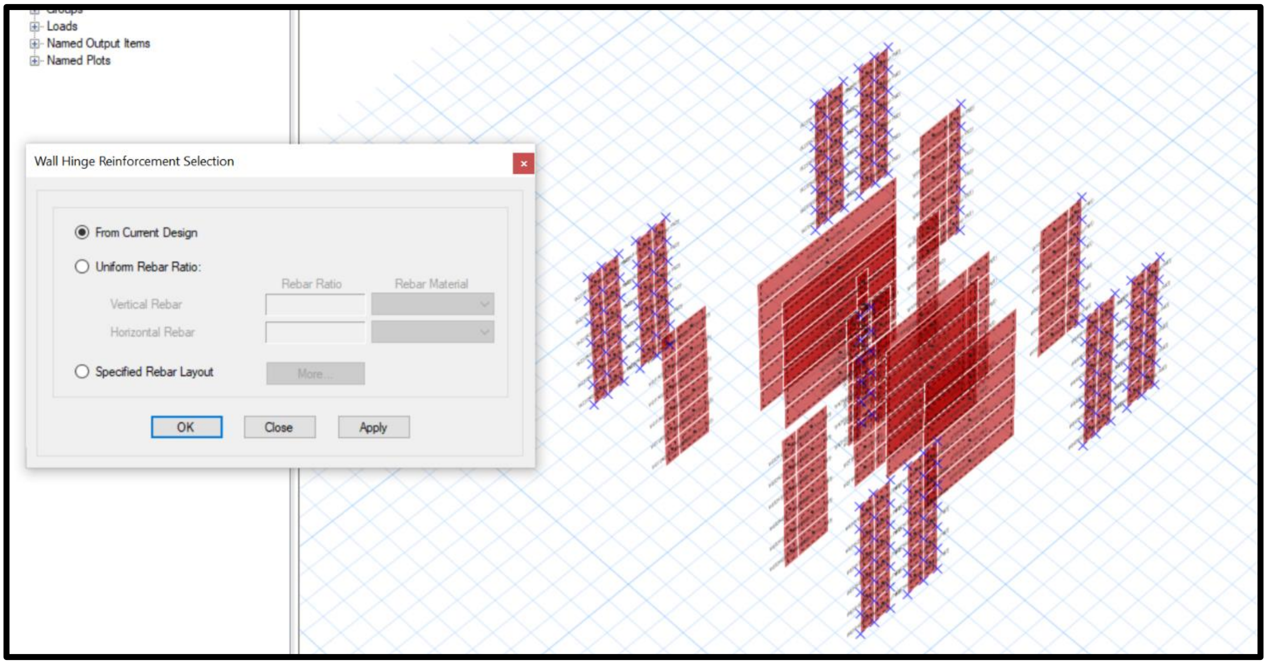


Figure 3.11.4.10 – Shear Walls Reinforcement Selection Dialog Box in ETABS

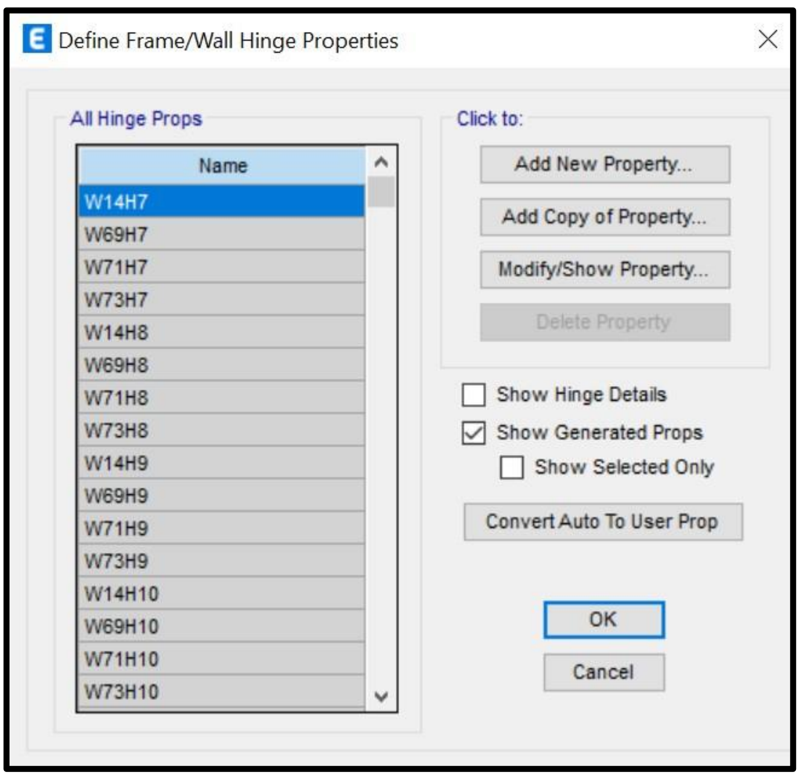


Figure 3.11.4.11 – Hinges Selection Dialog Box in ETABS

3.11.5 Results of Pushover Analysis

After carrying out the analysis process, the performance point should be determined and fiber plastic hinges should be checked to assure they are below the immediate occupancy performance level.

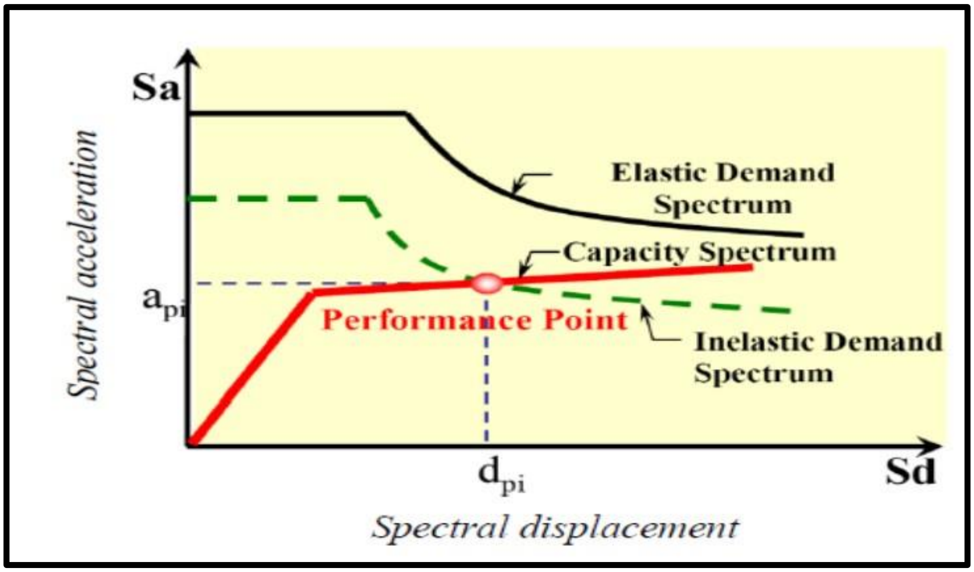


Figure 3.11.5.1 – Pushover Curve as per ATC-40

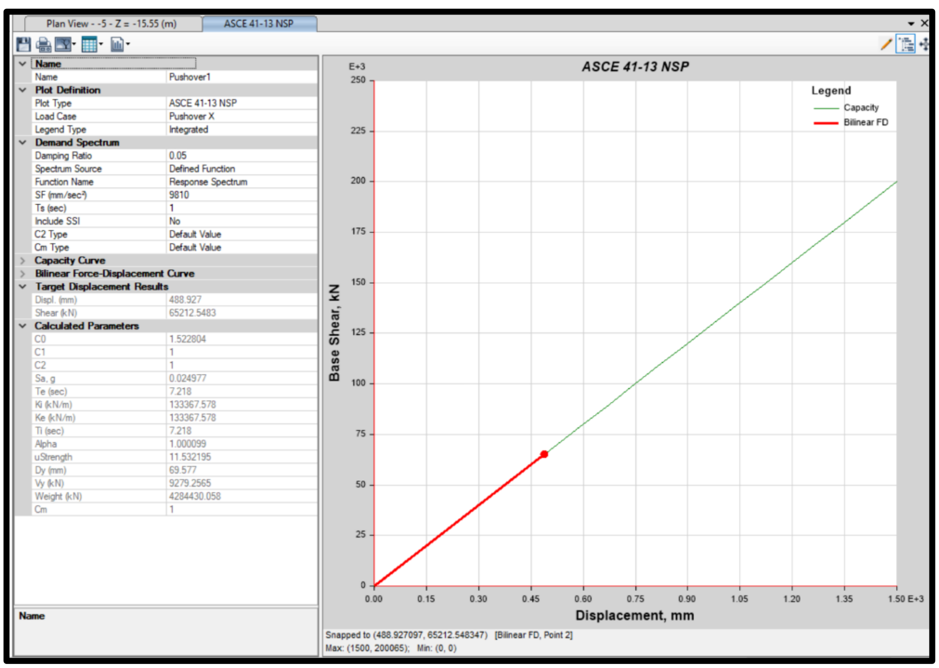
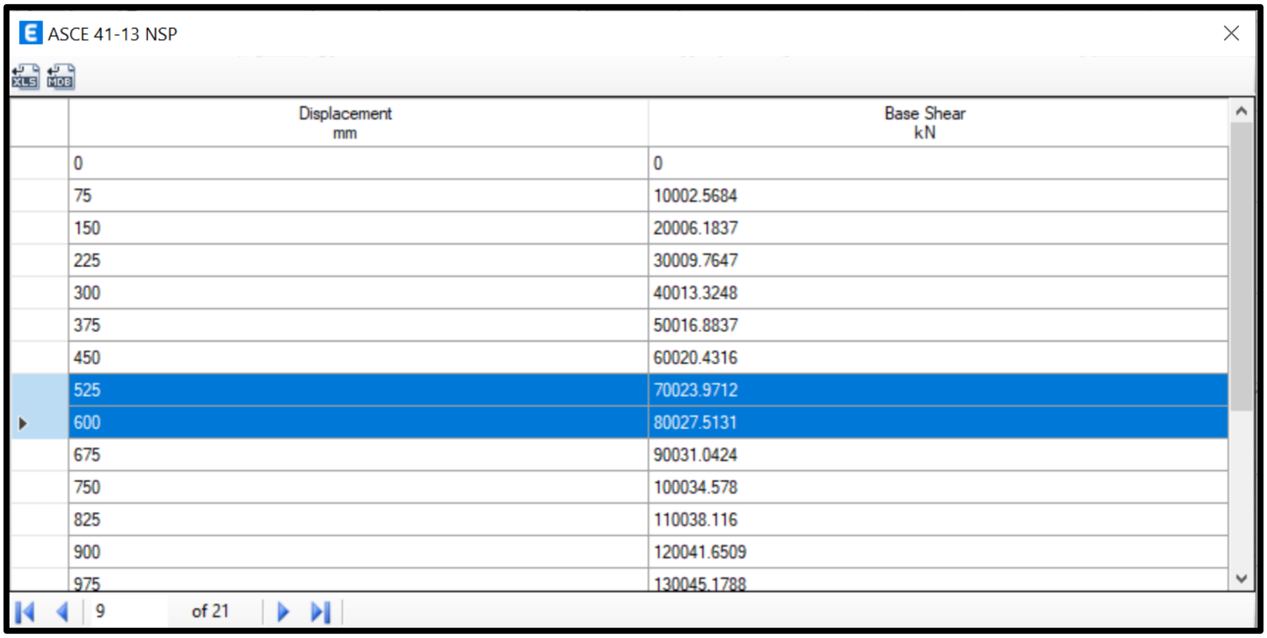


Figure 3.11.5.2 – Pushover Curve for Pushover X Load Case, ASCE 41-13 NSP, in ETABS

As shown in the figure above, the performance point displacement value is equal to 488.93 mm, which is located between step 6 and step 7 as shown in **Figure 3.11.5.3**, and the deformed shapes for steps 7 and 8 are shown in **Figure 3.11.5.4** and **Figure 3.11.5.5** respectively.


ASCE 41-13 NSP

Displacement mm	Base Shear kN
0	0
75	10002.5684
150	20006.1837
225	30009.7647
300	40013.3248
375	50016.8837
450	60020.4316
525	70023.9712
600	80027.5131
675	90031.0424
750	100034.578
825	110038.116
900	120041.6509
975	130045.1788

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Figure 3.11.5.3 – Pushover X Load Case Table in ETABS

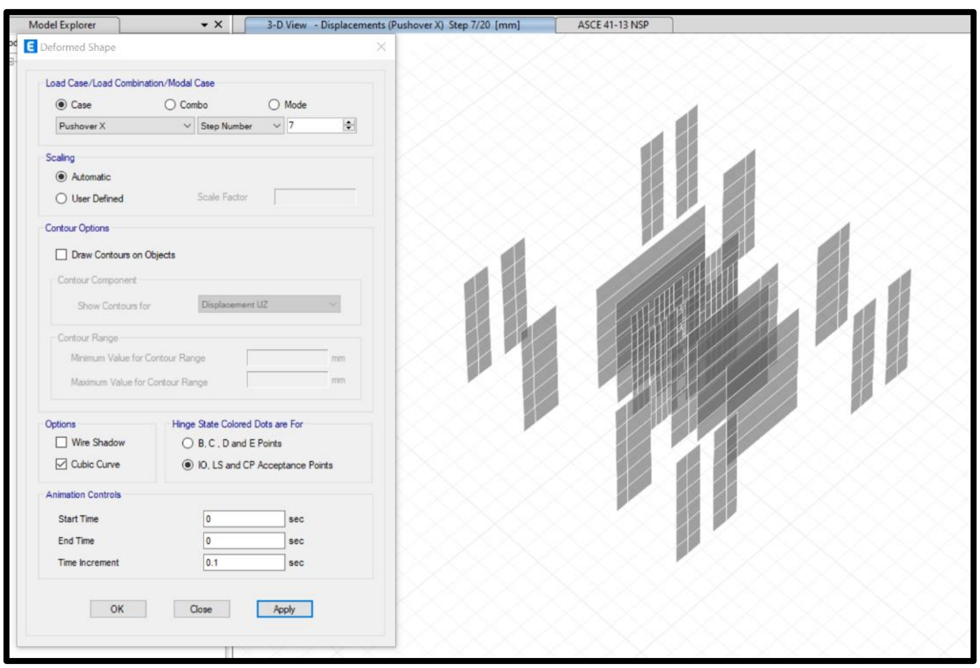

Figure 3.11.5.4 – Deformed Shape for Pushover X Load Case at Step 7 in ETABS

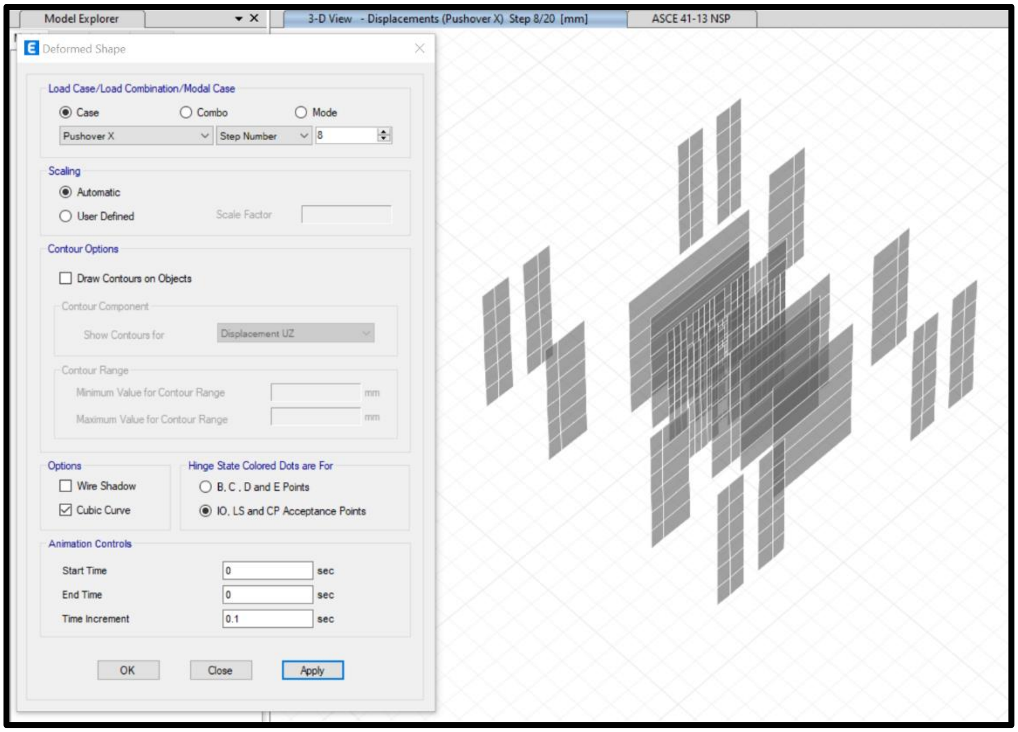

Figure 3.11.5.5 – Deformed Shape for Pushover X Load Case at Step 8 in ETABS

The maximum elastic displacement in X direction due to EDX load case was 410.638 mm, so the inelastic displacement would be (410.638 * 5 / 1.25) or 1642.55 mm, which is greater than the performance point displacement.

For the base shear, the performance pint base shear, by interpolation, would be about 65000 KN, which is 62.2% of the base shear resulted from the EDX load which is equal to 104480 KN.

As a result of these pushover analysis data, the assigned structural shear walls were designed to withstand a much greater base shear than the performance point one, and this might be due to the previously increased lateral stiffnesses of the floors to avoid performing a p-delta analysis, which takes a long time due to the intensive data being computed. In addition, and as shown in **Figure 3.11.5.6**, the performance check passed for the selected structural shear walls in the X direction with very small D/C values, which can indicate that the design of the whole structure is adequate, because the response of the stricture in both directions, X and Y, is almost the same, and the behaviors do not differ much.

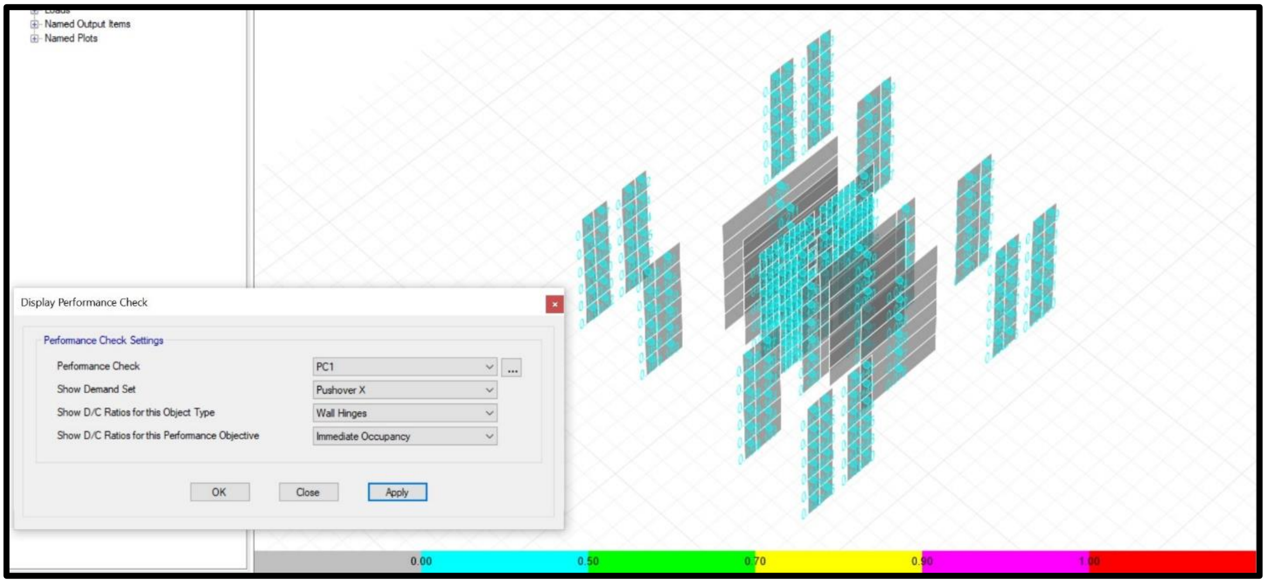

Figure 3.11.5.6 – Performance Check for Pushover X Load Case Demand Set in ETABS

REFERENCES

- ACI Committee 318, “Building Code Requirements for Structural Concrete (ACI 318M-19)”, American Concrete Institute, Michigan, 2019.
- ASCE Committee, “American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures” Edition 7-10.
- ASCE Committee, “American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures” Edition 7-22.
- ASCE Committee, “American Society of Civil Engineers, Seismic Evaluation and Retrofit of Existing Buildings” Edition 41-13.
- Tall Buildings Initiatives (TBI), May 2017. “Guidelines for Performance-Based Seismic Design of Tall Buildings.” Pacific Earthquake Engineering Research Center (PEER).
- Arman, Ibrahim “Design of Reinforced Concrete Building Structures with Computer Applications,” First Edition, Palestine, 2022.
- Arman, Ibrahim “Design of Reinforced Concrete Structures,” Fourth Edition, Palestine, 2022.
- Government of Dubai. “Dubai Building Code,” 2021 Edition.
- AASHTO LRFD Bridge Design Specifications 8th Edition. 2017.
- Arthur H. Nilson, David Darwin and Charles W. Dolan, Design of Concrete Structures, fourteenth edition in SI units, McGraw Hill, 2010.
- Jack C. McCormac and Russell H. Brown, Design of Reinforced Concrete, 9th Edition 2014.
- James K. Wight, University of Michigan, Reinforced Concrete: Mechanics and Design, 8th Edition, Pearson, 2022.
- Hi Sun Choi, Goman Hu, Leonard Joseph, Neville Mathias. 2014. “Outrigger Design for High-Rise Buildings”. Council on Tall Buildings and Urban Habitat.
- Joseph E. Bowles, 1997. “Foundation Analysis and Design.” Fifth Edition. The McGraw-Hill Companies.
- CRSI Committee, “Concrete Reinforcing Steel Institute, Design Guide for Pile Caps” First Edition, 2015.