An-Najah National University
Faculty of Engineering and Information Technology

جامعة النجاح الوطنية
كلية الهندسة و تكنولوجيا المعلومات

Graduation Project Report II

Structural Design of Residential Complex in Nablus City-Palestine

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DEDICATION

We humbly dedicate this scientific work to our beloved homeland, Palestine, a symbol of resilience and hope. In every page of this project, we hold the dream that freedom, peace and security will soon grace our land, and that a future filled with unity and prosperity awaits.

To our dearest parents, whose unwavering love, sacrifices, and endless support have been the pillars of our strength. This achievement would not have been possible without your guidance and belief in us. We are eternally grateful.

To our brothers, sisters, and friends, your motivation, tireless encouragement, and moral support have carried us through this journey. You have stood by us in both success and challenge, and for that, we extend our deepest thanks and appreciation. This success belongs to all of us.

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May Allah bless and reward all who contributed to the success of this project.

DISCLAIMER

This report was written by students at the Civil Engineering Department, Faculty of Engineering, An-Najah National University. It has not been altered or corrected, other than editorial corrections, as a result of assessment and it may contain language as well as content errors. The views expressed in it together with any outcomes and recommendations are solely those of the students. An-Najah National University accepts no responsibility or liability for the consequences of this report being used for a purpose other than the purpose for which it was commissioned.

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Abstract

This report presents the structural design of a six-story reinforced concrete residential building located in Nablus, Palestine, developed in two successive phases: a gravity-load-only design followed by a full seismic redesign. Although the study was not initiated as a comparison between gravity-only and seismic design methodologies, the dual-phase development offers an opportunity to reflect on key observations, particularly regarding the influence of early design decisions on later seismic performance.

The structure, with a footprint of approximately 30 × 20 meters and moderate spans not exceeding 6 meters, was modeled using ETABS. Gravity design considered typical floor loads of 4 kN/m² superimposed dead and 3 kN/m² live, with 12 kN/m² live loads at the roof and ground floors, in addition to lateral earth and water pressures reaching up to 10 kN/m² in the basement. The structural system comprised solid two-way slabs at all levels, reinforced concrete columns (ranging from 120×40 cm at lower stories to 30×30 cm at upper ones), and 20 cm thick perimeter boundary walls — all contributing to vertical load resistance and transferring loads to isolated and strip footings. Basement retaining walls, 30 cm thick, were designed to withstand lateral soil and water loads. All elements were designed according to ACI 318-19, using load combinations and parameters from ASCE 7-16.

In the second phase, a full seismic assessment and design were conducted using the same model, modified to incorporate seismic definitions and structural refinements based on engineering judgment. The lateral force-resisting system was classified as a bearing wall system consisting of special reinforced concrete shear walls. Seismic analysis was carried out using both equivalent static and response spectrum procedures. Checks for irregularities, story drift, and P-Delta effects (found negligible) were conducted, followed by seismic design of structural members in compliance with ACI 318-19 Chapter 18. The final design reflected the combined demands of gravity and seismic actions.

While not intended as a comparative study, the project highlighted how certain early gravity-based decisions — particularly the adoption of solid slabs and perimeter reinforced concrete walls — may later contribute positively to seismic performance. These boundary walls, though often overlooked in gravity-only designs, offered notable lateral resistance without interfering with spatial or architectural functionality. Such findings are especially relevant to low- to mid-rise residential buildings common in Palestinian urban settings.

PART I

GRAVITY LOAD DESIGN

CHAPTER 1: INTRODUCTION

1.1 General

This project is concerned with the analysis and design of a multi-storied concrete-framed building intended for residential accommodation for An-Najah National University students, situated near the new university campus. This reinforced concrete structure is designed to resist both gravity and seismic loads simultaneously. However, the project is divided into two parts: the first part, covered in Chapters 1, 2, and 3, focuses solely on gravity loads, while the second part, presented in Chapters 4, 5, and 6, builds upon the initial design to address seismic considerations, making any necessary modifications. This approach allows for a comparison between standard designs and seismic designs, highlighting the differences and adaptations required for each.

The chapter at hand introduces the general aspects of the project, providing a concise overview that includes the building's location, site elevation, and a description of its layout, along with the specific functions assigned to each floor.

The chapter then details the materials employed in both structural and non-structural elements, accompanied by their key characteristics relevant to the design, including mechanical properties, unit weights, and other essential parameters.

With regard to the design, the chapter specifies the fundamental information necessary for the design process. This includes the key design principles, the use of three-dimensional modeling and the selected software, in addition to modeling of supports, partitions, and tie beams. The discussion further covers the application of the design methods, each suited to its respective purpose. Additionally, relevant codes and standards for structural design are reviewed. The types of loads considered—along with their type and magnitude—are outlined, followed by an enumeration of the load combinations employed in the design process. Finally, the geotechnical investigation and its integration into the overall design approach are addressed to develop a design that aligns with the site's specific conditions.

1.2 Project description

The building is located near An-Najah University, specifically in the northern sector of the new university campus (32°13'45"N 35°13'35"E). **Figure 1.1** shows the location of the project. The structure consists of multiple levels, including a basement, a ground floor, five upper floors, a roof, and a top roof.

The design reflects careful consideration of both functionality and aesthetic appeal, aiming to offer a high-quality living environment that complements the surrounding campus infrastructure.

The total built-up area of the project encompasses 13,067.61 m² as shown in **Figure 1.1,** while the site area covers 2,142.66 m². This allocation ensures optimal use of the available land, allowing for a well-balanced design that meets both functional and spatial requirements.

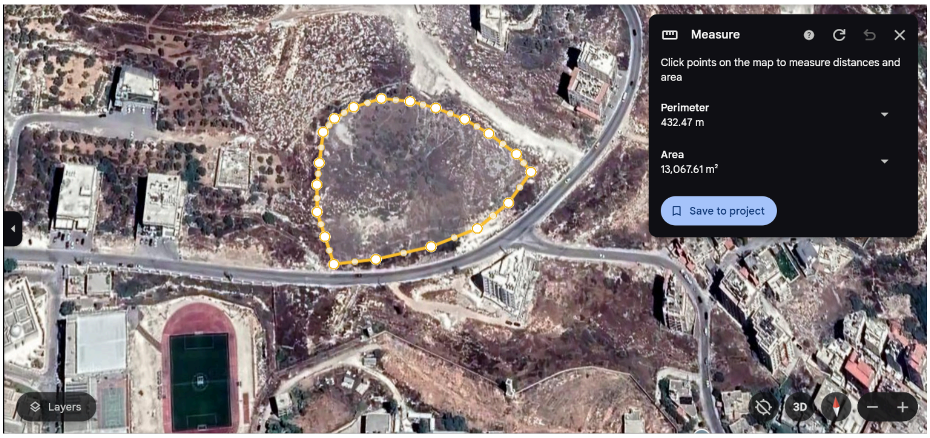


Figure 1.1: Site location and area

1.2.1 Project elevation

The elevation of the project site is **550m** above sea level as shown in **Figure 1.2** adapted from Google Earth. This elevation data provides a crucial understanding of the site's topography and will play a key role in the overall design and structural considerations of the building.

Figure 1.2: Site elevation

1.2.2 Floor details

Each floor features a unique description with varying areas, as detailed in **Table 1.1**. All floor areas are calculated approximately, the elevation of each floor is specified, along with a description of its intended use.

Table 1.1: Floor details

Floors	Area (m^2)	Level (m)	Description
Basement	123	-3.35	Fire fighting water tank, Domestic water tank, Pump room, Staircase-02, Lift 1, Lift 2.
Ground floor	463	0.6	Generator room, Disabled toilet, TEL. Room, CCTV Room, LV Room, Guard Room, Transformer RM, H.V Room, Staircase-01, Garbage Room, Entrance lobby, lift lobby, corridor, Staircase-02, pump room access, retail-01, retail-02, lift-01, lift-02, garbage room, Reception desk, FHR,W.M cabinet.
Typical floors **Floor (1 to 5)**	567	5 8.55 12.1 15.65 19.2	Lift-01, Lift-02, Staircase-01, Staircase-02, Corridor, Telephone room, Electric room, garbage room,Bedroom (#5), Master bedroom(#4), living room (#4), balcony (#4), kitchen (#4), bathroom (#12), FHR, store (#1).
Floors	**Area (m^2)**	**Level (m)**	**Description**
Roof	567	22.75	Lift-01, Lift-02, Staircase-01, Staircase-02, Tele room, Garbage room, GSM room, Chiller-01, Chiller-02, Ladder, Water tank, LPG tank, Electric room, MCC room, Chilled water pump room
Top roof	185	26.2	

1.2.3 Wall Specifications

The exterior walls of the building are composed of layers arranged from outside to inside as follows: 50 mm stone, 50 mm concrete, a steel mesh, 200 mm reinforced concrete, 30 mm foam, 100mm block wall, and 20mm plastering layer.This design enhances durability and structural integrity, ensuring durability and structural integrity around the structure. For the interior walls, 10 mm concrete blocks are used, with an additional 20 mm layer of plaster applied to each side, providing smooth finishes and improved insulation.

1.2.4 Main plans

The main plans of the project can be viewed in **Appendix A**. All plans and other drawings are included in appendices.

1.3 Analysis and design principles

The supports in the structure were considered as fixed supports to represent the foundations. External walls were included in the modeling as reinforced concrete bearing walls, while internal partitions were not modeled but their weight was incorporated into the calculation of superimposed dead loads. Additionally, tie beams between the foundations were adopted and designed using the ultimate design method. Computer applications (SAP 2000 & ETABS) were employed for structural modeling, where slabs and walls were represented as area elements and beams and columns as line elements.

1.4 Codes and standards

Codes give specific requirements for materials, structural analysis, reinforcement details, etc. These requirements work as a limitation for structural design to guarantee various aspects such as integrity of structure, avoiding sudden collapse, and other contexts. In general, many codes are used in design; however, the American Concrete Institute (ACI) code has long been a leader and widespread in many regions of the world, same was for Palestine. This code and other codes and standards illustrated below:

- **ACI 318-19, (American Concrete Institute)**.
- **ACI 350-20, (American Concrete Institute).**

- **ASCE 7-16, (American Society of Civil Engineers): Minimum Design Loads for Buildings and Other Structures.**
- **ASTM, (American Society for Testing and Materials)**.
- **Jordanian Code for Loads and Forces 2006.**

The analysis and design of reinforced concrete sections (structural elements) is based on **ACI 318-19 code**. **ASCE 7-16** is used for determination of loads such as live load, lateral load (soil, hydrostatic pressure, and seismic load later on); however, snow load, specifically, is based on the Jordanian Code rather than **ASCE 7-16** for more realistic condition and result as it is a local code. Load combinations are also connected with **ASCE 7-16**. For materials specifications, **ASTM** governs concrete materials and reinforcing steel characteristics in detail.

1.5 Materials

Materials of structural and non-structural elements included in this project are discussed below.

1.5.1 Materials of structural element

All structural members are made of reinforced concrete due to its ability to resist both tension and compression forces that would be applied simultaneously on a section. Reinforced concrete consists of:

- Reinforcing steel: grade-60 steel, ASTM A615 and ASTM A706. **Table 1.2** shows reinforcing steel properties.
- Concrete: normal weight concrete (cement in mixture: Type 1 portland cement). **Table 1.3** shows concrete properties.

Table 1.2: Reinforcing steel properties

Reinforcing steel bar	Value
Steel grade	Grade 60
Yield strength, f_y	420 MPa
Modulus of elasticity, E_s	200 Gpa
Ultimate strength, f_u	620 MPa

Table 1.3: Concrete properties

Concrete	Value
Unit weight , γ_c	25 KN/m^3
Poisson's ratio , ν	0.2
Compressive strength, f_c	28
Modulus of elasticity, E_c	$4700(\sqrt{f_c}) = 24870$ MPa
Modulus of rupture, f_r	$0.62(\sqrt{fc}) = 3.28$ MPa

1.5.2 Materials of non-structural elements

In any structure, some parts of it aren't structurally valuable but essential for many reasons related to architecture and utilization. These parts usually have an ignorable weight and create loads that should be taken into consideration for design. **Table 1.4** illustrates materials of non-structural elements with their purpose and expected loads.

Table 1.4: Materials of non-structural elements

Material	Description	Thickness / Notes	Unit Weight (KN/m^3)[1]
Tile in general	Durable material for floors and walls, moisture-resistant and versatile in design.	10 mm thick / Ceramic tiles	26
Fill under tiles	Crushed stone sand (fine aggregate)	120 mm thick	18
Cement mortar	Mixture of cement, sand, and water used to bind bricks, tiles, and blocks.	thickness: 20 mm under tiles and 10 mm between blocks	23
Granite tiles	Natural stone valued for its elegance and durability..	30 mm / used for stairs tiling works	26
Steel handrail	Handrail is erected along stair length for safety	900 mm height	0.36 KN/m
Masonry stone	Used in construction and landscaping, valued for their aesthetic appeal and versatility in enhancing design	50 mm/ Matabeh (composition: limestone, crystalline). covers the outer face of the exterior wall	26
Concrete blocks	A mix of Portland cement, aggregates, and water, used for interior partitions for sound insulation and for thermal insulation in exterior walls.	100 mm for interior walls	15
Plastering layer	Mix of water, sand, and cement or gypsum used to create a smooth finish on walls and ceilings	20 mm each side	23
Gypsum	Soft mineral used for plastering and drywall		13.7
Aluminum	Lightweight, corrosion-resistant metal for windows, doors, and cladding		27
Compacted fill	Compressed soil or aggregate providing a stable base for construction		18
Bitumen Rolls	Flexible sheets for waterproofing roofs and foundations.	4 mm	0.35
Concrete screed	Flat layer of concrete applied to floors, providing a smooth and level surface for flooring materials.	1% sloping	23
Spray foam	Used between the external wall and the internal concrete block wall beside in order to enhance thermal insulation.	30 mm	0.3

[1] The unit weights of nonstructural materials were adapted based on the minimum values specified in **ASCE 7-16, Table C3.1-1**, ensuring consistency with code-prescribed loading assumptions.

1.6 Loads

1.6.1 Gravity loads

- Dead load: self weight of structural elements.
- Superimposed dead load: (SD) KN/m^2, values of superimposed dead load are mentioned in **Table 1.5.**
- Live load: The weight of people, furniture and moving equipment, which can vary over time. Values of live load are mentioned in **Table 1.6.**
- Exterior wall load=8 kN/m^2.

Table 1.5: Superimposed Dead Load

Number of floor	Name	SD (kN/m^2)[2]
1	Basement	3.5
2	Ground floor	4
3	First Floor	3.5
4	Second Floor	3.5
5	Third Floor	3.5
6	Fourth Floor	3.5
7	Roof	3.5

[2] The superimposed dead load was calculated manually on a per-square-meter basis, using the unit weights listed in **Table 1.4**.

Table 1.6: Live Load

Building Type	**Load (kN/m²)[3]**
Ground Floor (retail commercial units)	5
Ground Floor (H.V, L.V, Generators)	12
Residential (Private rooms and their corridors)	2
Residential (public rooms and their corridors)	5
Stairs and Exit ways	5
Garages	3
Roof	5
Roof (side of water tanks and chillers)	12
Basement (contain pumps rooms and water tanks)	12

Live load values varied across the ground floor, with some areas having values of 5kN/m² and others at 12 kN/m². Both values were incorporated into the analysis and design, each applied according to its specific area. The roof was divided into two zones: one with a 12 kN/m² load for the chiller and water tank areas, and the other with a 5 kN/m² load for remaining sections. For residential floors, a uniform live load of 3 kN/m² was selected, as typical values ranged between 2 kN/m² and 5 kN/m².

1.6.2 Snow load

Snow loads were included in the calculations based on the equation shown in **Figure 1.3** from the **Jordanian Code for Loads and Forces (2006)**, which depends on the elevation above sea level.

- H= 550 m above mean sea level.
- (H-400)/320= (550-400)/320= 0.5 KN\m^2
- Snow load= **0.5 KN/m^2**

[3] The live load was determined based on the minimum values specified in **ASCE 7-16, Table 4.3-1**, ensuring alignment with code-prescribed occupancy-based loading requirements.

الجدول (٣ - ٥)

أحمـــال الثلـــوج

ارتفاع المنشأ عـن سطح البحر (h) (بالمتر)	حمل الثلـج (S_o) (كن/م٢)
250 > h	0
500 > h > 250	(h-250)/800
1500 > h > 500	(h-400)/320

Figure 1.3: Table 3-5, Jordanian Code for Loads and Forces (2006)

According to **ASCE 7-16**, Standards recommend a minimum value of snow load for low-slope roofs depending on risk categories which, in turn, depends on use of structure and occupancy.

Regarding the risk category, **Table 1.5-1** in **ASCE 7-16 standards**, Risk Category of Buildings and Other Structures for Flood, Wind, Tornado, Snow, Earthquake, and Ice Loads, risk category II is suitable for residential structures as shown in **Figure 1.4.** And referring to **Table 7.3-4 (from ASCE 7-16),** as shown in **Figure 1.5**.

Snow load will be considered as the minimum value **(from ASCE 7-16).**

Table 1.5-1. Risk Category of Buildings and Other Structures for Flood, Wind, Tornado, Snow, Earthquake, and Ice Loads.

Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent low risk to human life in the event of failure	I
All buildings and other structures except those listed in Risk Categories I, III, and IV	II
Buildings and other structures, the failure of which could pose a substantial risk to human life Buildings and other structures not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released*	III
Buildings and other structures designated as Essential Facilities Buildings and other structures, the failure of which could pose a substantial hazard to the community Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released* Buildings and other structures required to maintain the functionality of other Risk Category IV structures	IV

*Buildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower risk category if it can be demonstrated to the satisfaction of the Authority Having Jurisdiction by a hazard assessment as described in Section 1.5.3 that a release of the substances is commensurate with the risk associated with that risk category.

Figure 1.4: Risk Category of Buildings and Other Structures for Flood, Wind, Tornado, Snow, Earthquake, and Ice Loads (ASCE 7-16 Table 1.5-1)

Table 7.3-4. Minimum Snow Loads for Low-Slope Roofs.

Risk Category	$p_{m,max}$
I	25 lb/ft^2 (1.20 kN/m^2)
II	30 lb/ft^2 (1.44 kN/m^2)
III	35 lb/ft^2 (1.68 kN/m^2)
IV	40 lb/ft^2 (1.92 kN/m^2)

Figure 1.5: Minimum snow loads for Low-Slope roofs (ASCE 7-16 Table 7.3-4)

1.6.3 Lateral loads

1. Soil load:

   Lateral Load of soil = h * γ_{soil} * k.

2. water load:

   Lateral Load of water = h * γ_w.

   Where: h is the height (distance from water top surface to point of interest) (m). γ_{soil} and γ_w are the unit weights of soil and water respectively (*kN*/*m*³).

These loads take place on the retaining walls in the basement (soil pressure) and on the walls of the water tank (fluid pressure). The topic of seismic load and wind load will be studied and addressed in Graduation Project II.

1.7 Load combinations

As mentioned earlier, the Ultimate Design Method was considered for the design, referring to the following load combinations sourced from Section 2.3.1 of **ASCE 7-16**:

1.7.1 Symbols

1. D = Dead load
2. L = Live load
3. L_r = Roof live load
4. S = Snow load
5. R = Rain load

6. W = Wind load

1.7.2 Basic load combinations for strength design

The following load combinations are the basic combinations for strength design. They were sourced from section 2.3.1, Chapter 2, **ASCE 7-16**.

1. 1.4D
2. 1.2D + 1.6L
3. 1.2D + 1.6L + 0.5(L_r OR S OR R)
4. 1.2D + 1.6(L_r OR S OR R) + (L OR 0.5W)
5. 1.2D + 1.0W + L + 0.5(L_r OR S OR R)
6. 0.9D + 1.0W

1.7.3 Basic load combinations for allowable stress design

The following load combinations are the basic combinations for allowable stress design. They were sourced from section 2.4.1, Chapter 2, **ASCE 7-16**.

1. D.
2. D + L.
3. D + (L_r or 0.7 S or R).
4. D + 0.75L + 0.75 (L_r or 0.7 S or R).
5. D + 0.6(W or W_T).
6. D + 0.75L + 0.75 (0.6(W or W_T)) + 0.75L + 0.75 (L_r or 0.7 S or R).
7. 0.6D + 0.6(W or W_T).

1.7.4 Basic combinations with seismic load effects.

Seismic load combinations were sourced from the same standard, Section 2.3.6, and can be summarized as follows:

1. 1.2D+ E_v + E_h + L+ 0.2S
2. 0.9D − E_v + E_h

Where E_v and E_h are defined in Section 12.4.2 or 12.14.3.1 of the code:

$E_h = \rho Q_E$ (12.4-3)

$E_v = (0.2S_{DS})D$ (12.4-4a)

Where:

- Q_E = effects of horizontal seismic forces from V or F_p (where required by Section 12.5.3 or 12.5.4, such effects shall result from application of horizontal forces simultaneously in two directions at right angles to each other)
- ρ = redundancy factor, as defined in Section 12.3.4 of ASCE 7-16
- S_{DS} = design spectral response acceleration parameter at short periods obtained from Section 11.4.5 of ASCE 7-16
- D = effect of dead load

1.8 Geotechnical investigation

Geotechnical investigation is the major process for determining soil mechanical properties, through which a designer selects the most appropriate type of footings that withstand loads of the structure. No soil tests were made for the land of interest, that is, the land that the building is established on. However, credibility has been given to geotechnical data that pertains to a nearby land, 200 meters far away, **Figure 1.6** exhibits the exact location of the site with coordinates. Referring to Hijjawi Construction Labs, and after performing a borehole test for three soil samples, a geotechnical investigation report for this land was obtained. The main characteristic that is required in the design of the foundation phase is the soil allowable bearing capacity. The soil internal friction angle (ø) in lateral load calculation for retaining walls, in addition to unit weight. All of these characteristics are presented and discussed below. Other details are given in the attached file (geotechnical investigation report), including a description of test works, assumptions, and recommendations.

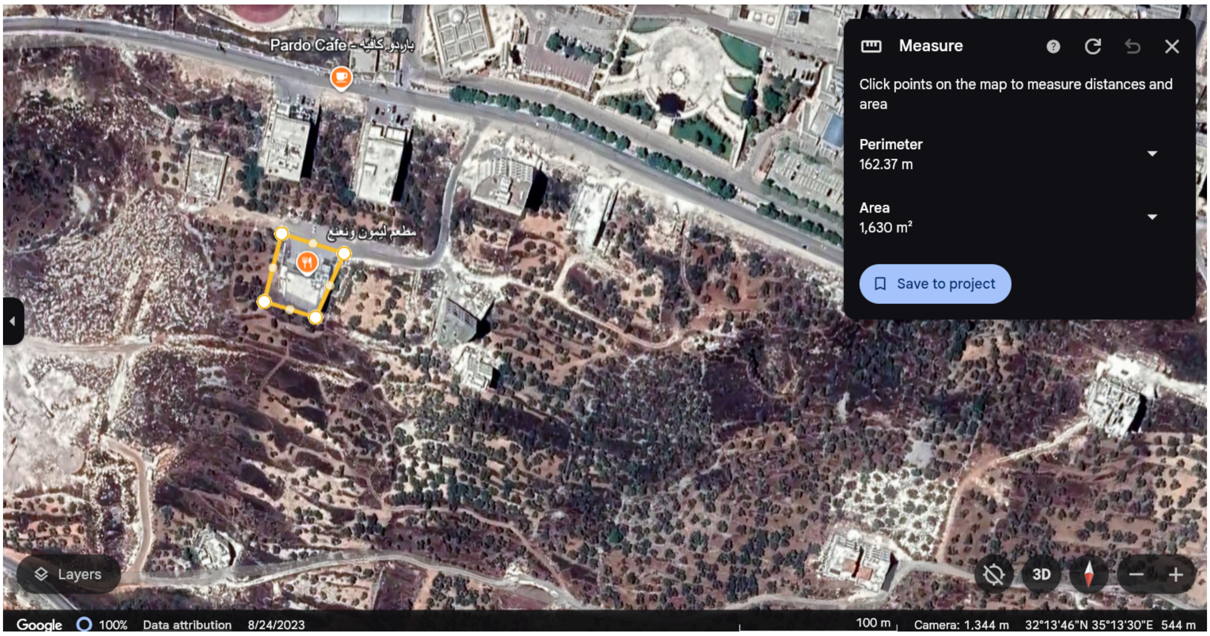


Figure 1.6: Nearby site location and coordinates

Calculations of the ultimate bearing capacity are based on Terzaghi equation:

$q_{ult} = C\,N_C + \gamma_o\,DN_q + 0.5\,\gamma_1\,B\,N\gamma$ (Terzaghi equation)

Results of this test are as follows:

- Soil internal friction angle(ø)=20∘. See **Figure 1.7**.
- Allowable bearing capacity=250 kN/m² (factor of safety=3). See **Figure 1.8**.
- Unit weight γ_S=18 kN/m^3. See **Figure 1.8**.

The coefficient of the soil lateral pressure (k) is crucial for soil pressure calculations in the retaining walls in the basement, it can be determined using the equation below, where the wall deflects away from soil and active behavior is considered.

$$K = \frac{1-\sin Ø}{1+\sin Ø}$$

For angle Ø equals 20°, **K=0.49**, where:

$$\frac{1-\sin 20}{1+\sin 20} = 0.49$$

K active= 0.49.

1-sin20= 0.66.

K static= 0.66.

Borehole No.	Depth (m)	Passing 200	PI	Cohesion (KN/m²)	Angle of internal friction ϕ (°)
1	0.0-7.0	28	3	11	22
	7.0-10.0	37	8		
	10.0-14.0	33	6		
2	0.0-6.5	30	4	11	21
	6.5-7.5	38	7		
	7.5-9.0	34	5		
3	0.0-6.0	32	4	12	20
	6.0-7.0	34	6		
	7.0-9.0	40	9		

Figure 1.7: Angle of internal friction for the three samples

BEARING CAPACITY OF SHALLOW FOUNDATIONS
Terzaghi Method

Date May 25, 2024

Identification

INPUT

Units of Measurement

	SI	SI or E

Foundation Information

Shape	SQ	SQ, CI, CO, or RE
B =	2	m
L =	2	m
D =	3	m

Soil Information

c =	12	kPa
phi =	20	deg
gamma =	18	kN/m^3
Dw = -		m

Factor of Safety

F =	3

Terzaghi Results

Bearing Capacity

q ult =	**741**	**kPa**
q a =	**247**	**kPa**

Allowable Column Load

P =	**705**	**kN**

Figure 1.8: Bearing capacity calculation

CHAPTER 2: PRELIMINARY DIMENSIONS

2.1 General

This chapter presents the preliminary dimensions of the project structural elements, including the slab, beams, columns, and walls. All dimensions were determined according to the specifications outlined in **ACI 318-19 (American Concrete Institute).** These preliminary dimensions must be verified and could be adjusted based on further checks in Chapter 3.

2.2 Lateral and gravity forces resisting systems

Dimensions and sections were determined assuming the structure is subjected only to gravity loads. Lateral load design will be addressed in Part II of the Project. The gravity load resisting system is composed of perimeter bearing walls, interior walls at staircases and elevators, and few interior columns.

2.3 Slabs structural systems

The structural slab for this project is a solid slab, which is commonly used in residential, commercial, and social buildings. This type of slab offers enhanced strength, greater durability, sound insulation, and improved fire resistance. Furthermore, the solid slab was chosen primarily for its ability to withstand seismic forces, making it an ideal choice for earthquake resistance. Solid slabs provide better performance in resisting seismic loads compared to other types of slabs. So it is better to be used in design.

The structural slab is a two-way solid slab, which ensures the structure's durability by effectively distributing loads to the supporting beams and columns. The slab's preliminary thickness is discussed in **Section 2.4.**

2.4 Preliminary slab thickness and loads

As mentioned in **Section 2.3**, the slab structural system is a two-way solid slab. The slab thickness is determined in **Figure 2.1**, based on the longest span in a typical floor plan, which measures **8.45** meters.

Table 8.3.1.1—Minimum thickness of nonprestressed two-way slabs without interior beams (mm)[1]

f_y, MPa[2]	Without drop panels[3]			With drop panels[3]		
	Exterior panels		Interior panels	Exterior panels		Interior panels
	Without edge beams	With edge beams[4]		Without edge beams	With edge beams[4]	
280	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$	$\ell_n/36$	$\ell_n/40$	$\ell_n/40$
420	$\ell_n/30$	$\ell_n/33$	$\ell_n/33$	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$
550	$\ell_n/27$	$\ell_n/30$	$\ell_n/30$	$\ell_n/30$	$\ell_n/33$	$\ell_n/33$

[1]ℓ_n is the clear span in the long direction, measured face-to-face of supports (mm).

[2]For f_y between the values given in the table, minimum thickness shall be calculated by linear interpolation.

[3]Drop panels as given in 8.2.4.

[4]Slabs with beams between columns along exterior edges. Exterior panels shall be considered to be without edge beams if α_f is less than 0.8.

Figure 2.1: Minimum thickness of two way slab[4]

Ln= 8.45m.

Preliminary thickness=8.45/33= 0.**25 m,** so the preliminary thickness is **250mm.**

2.5 Preliminary dimensions of beams: continuous beams and frames

Beams were placed along the exterior walls of the building, eliminating the need for interior beams. The longest span on a typical floor is shown in **Figure 2.2**, measuring 8.45 meters. The beam depth is determined based on **Figure 2.3**.

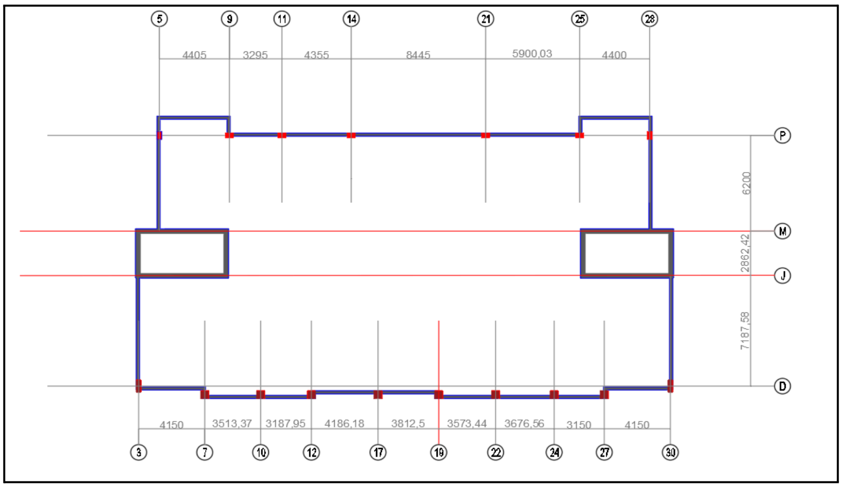


Figure 2.2: Typical floor plan

[4] Adapted from *ACI 318-19, Table 8.3.1.1 – Minimum Thickness of Nonprestressed Two-Way Slabs Without Interior Beams.*

Support condition	Minimum $h^{[1]}$
Simply supported	$\ell/16$
One end continuous	$\ell/18.5$
Both ends continuous	$\ell/21$
Cantilever	$\ell/8$

[1]Expressions applicable for normalweight concrete and f_y = 420 MPa. For other cases, minimum h shall be modified in accordance with 9.3.1.1.1 through 9.3.1.1.3, as appropriate.

Figure 2.3: Minimum depth of beam[5]

Based on the information in the **Figure 2.3**, minimum **depth h** is equal to **L/18.5.**

h (depth)= L/18.5.

h (depth)= 8.45/18.5= **0.456 m**. Take h (depth)= 0.5 m = **500 mm.**

Beam width **b= 300 mm.**

2.6 Preliminary dimensions of columns: tributary area computations

Exterior columns were used mainly for structural integrity, as beams with a 300 mm width are essential for adequate development length, whereas the 200 mm wall thickness alone could not support this requirement.Thus, these beams were supported by columns with dimensions of 300 mm x 500 mm for enhanced stability and integrity. Some of these columns (in the lower part of each floor) were 500 mm x 500 mm for architectural purposes in the ground floor, see **Figure 2.4**.

Interior columns were sized using the tributary area method to account for load distribution across each floor. Some interior columns were adjusted with a factor to increase their size, especially for columns subject to unbalanced moments due to irregular load distribution, particularly those on upper floors where axial load decreases, making moments more impactful. These columns were designed to reduce in size on higher floors to match the reduced load. **Table 2.1** illustrates the preliminary dimensions and tributary areas of the interior columns across floors, with more details available in the attached Excel sheet for this report. **Figure 2.4** shows gridlines and interior columns locations.

[5] Adapted from ACI 318-19, Table 9.3.1.1 – Minimum Depth of Nonprestressed Beams.

Table 2.1: Preliminary dimensions of the interior columns

Floors	Column	Suggested Length(m)	Suggested Width(m)	Tributary Area (m2)
Fifth Floor	G7	0.3	0.3	15.5
	N11	0.3	0.3	24
	G11	0.3	0.3	28
	N15	0.3	0.3	28
	N21	0.3	0.3	36
	G23	0.3	0.3	28.5
	G27	0.3	0.3	18
Fourth Third second Floors	G7	0.3	0.6	15.5
	N11	0.3	0.6	24
	G11	0.3	0.6	28
	N15	0.3	0.6	28
	N21	0.3	0.6	36
	G23	0.3	0.6	28.5
	G27	0.3	0.6	18
Ground and First Floors	G7	0.3	0.8	15.5
	N11	0.3	0.8	24
	G11	0.3	0.8	28
	N15	0.3	0.8	28
	N21	0.3	0.8	36
	G23	0.3	0.8	28.5
	G27	0.3	0.8	18
Basement Floor	N21	0.3	0.8	36
	G23	0.3	0.8	28.5
	G27	0.3	0.8	18

s.

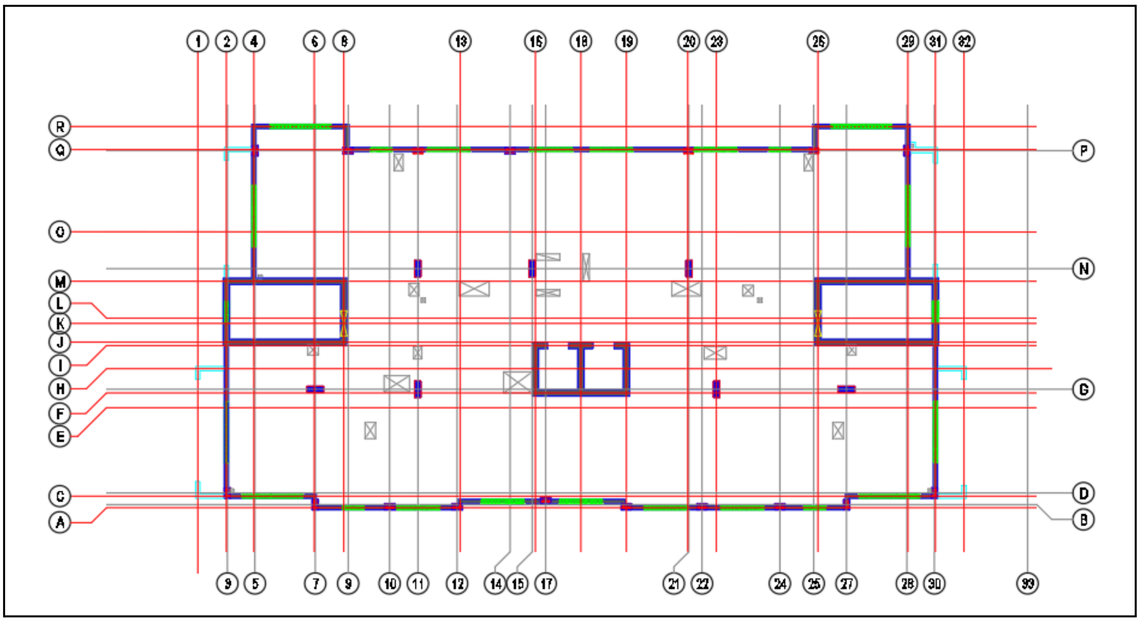


Figure 2.4: Gridlines and interior columns locations

2.7 Preliminary dimensions of walls

Walls in this project will function as bearing walls to handle structural loads. Bearing walls are used in the basement, staircase, and elevator areas. The wall thickness (h) will be determined based on **Figure 2.5.**

Table 11.3.1.1—Minimum wall thickness *h*

Wall type	Minimum thickness *h*		
Bearing[1]	Greater of:	100 mm	(a)
		1/25 the lesser of unsupported length and unsupported height	(b)
Nonbearing	Greater of:	100 mm	(c)
		1/30 the lesser of unsupported length and unsupported height	(d)
Exterior basement and foundation[1]	190 mm		(e)

[1]Only applies to walls designed in accordance with the simplified design method of 11.5.3.

Figure 2.5: Minimum thickness h and wall type.(ACI 318-19 Table 11.3.1.1)

According to **Figure 2.5**, the wall thickness h is determined as 1/25 of the lesser value between the unsupported length and unsupported height, so the exterior basement is measured to have a thickness of 190mm, (standardization of numbers). Therefore, a thickness of 300 mm for the interior walls of the elevator, stairwell, and walls of the basement. The perimeter wall of other floors is a bearing wall of 200 mm thickness.

CHAPTER 3: THREE-DIMENSIONAL ANALYSIS AND DESIGN

3.1 General

This chapter presents the analysis and design process conducted using ETABS software. It begins by outlining the modeling procedure, followed by verification of the preliminary member dimensions introduced in Chapter 2. Where necessary, adjustments were made based on analysis results. Key structural checks were then carried out, leading to the final design of all members, which are detailed in this chapter.

3.2 Structural modeling of the building

3.2.1 Units

The units used for designing the structure and interpreting the results from ETABS are illustrated in **Figure 3.1.** These units include meters (m) for length, Kilo Newtons (kN) for force, and degrees Celsius (°C) for temperature.

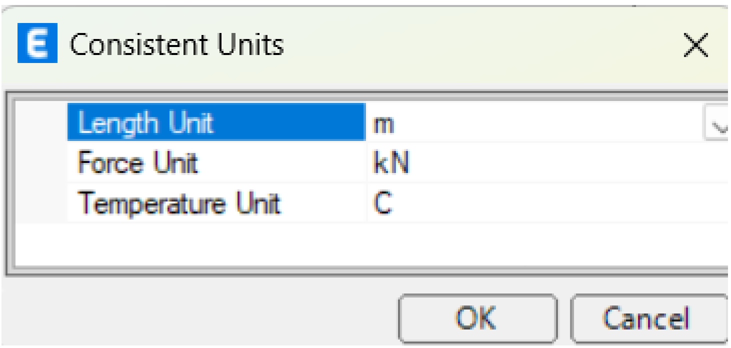


Figure 3.1: Consistent unit

3.2.2 Gridlines

The grid lines utilized in the ETABS model were determined based on column spacing, the interior walls of the elevator and stairwell, the basement layout, and the exterior walls. **Table 3.1** presents the distances between a grid line and another in both the X and Y directions. The elevations of each story were established from the floor finish level to the next floor finish level between stories. These elevations are detailed in **Table 3.2**, which illustrates the elevation of each level. The grid lines in X and Y axes are shown in **Figure 3.2**.

Table 3.1: Grid lines ordinates

Grid ID	X ordinate (m)	Grid ID	X ordinate (m)	Grid ID	Y ordinate (m)
A	0	O	23.8234	1	0
B	1.35	P	24.5	2	0.55
C	2.65	Q	27.5	3	4.75
D	5.5	R	29.3	4	5.45
E	6.899	S	30.65	5	6.6
F	9.063	T	33.55	6	7.7
G	10.4	U	34.85	7	7.862
H	12.401	V	36.2	8	8.75
I	14.755	W	39.205	9	9
J	15.95	-	-	10	10.75
K	16.4375	-	-	11	11.35
L	18.1	-	-	12	13.101
M	20.25	-	-	13	17
N	23.15	-	-	14	18.1

Table 3.2: Stories elevations

Story	Elevation (m)
Story9 (roof)	29.65
Story8 (5th)	26.2
Story7 (4th)	22.65
Story6 (3rd)	19.1
Story5 (2nd)	15.55
Story4 (1st)	12
Story3 (Ground Floor)	8.45
Story2 (Basement)	4.05
Story1	1
Base	0

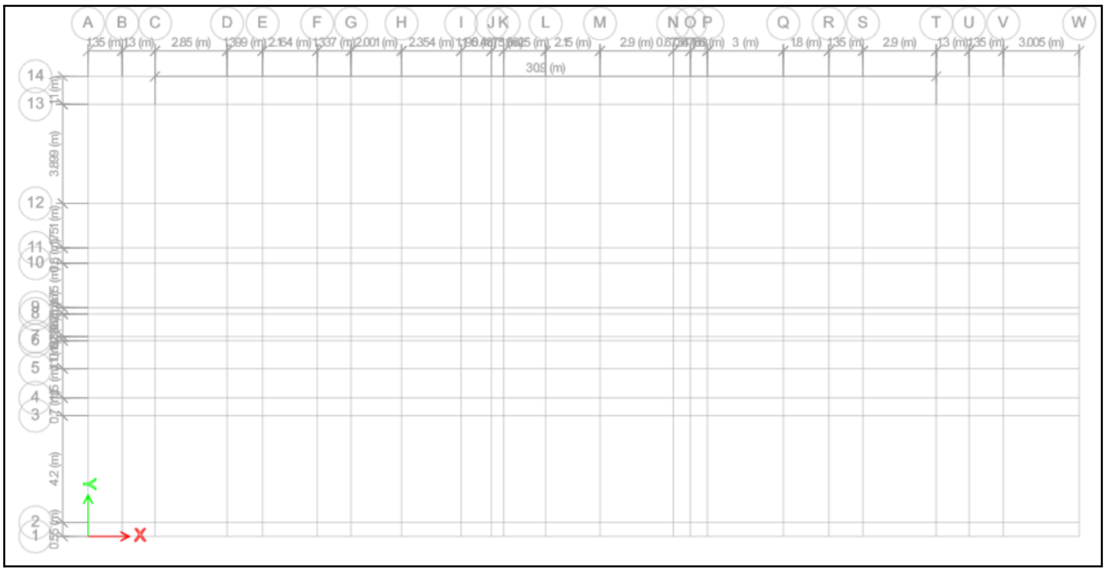


Figure 3.2: Grid lines in X and Y axes

3.2.3 Materials

The materials used in the design modeling are as following:

- Concrete material property is shown in **Figure 3.3**, and the compressive strength used is **28 MPa**.
- Rebar steel material property is shown in **Figure 3.4**, and the yielding strength of the tendon is **420 MPa** and the Ultimate strength is **620 MPa** with a **Grade 60.**

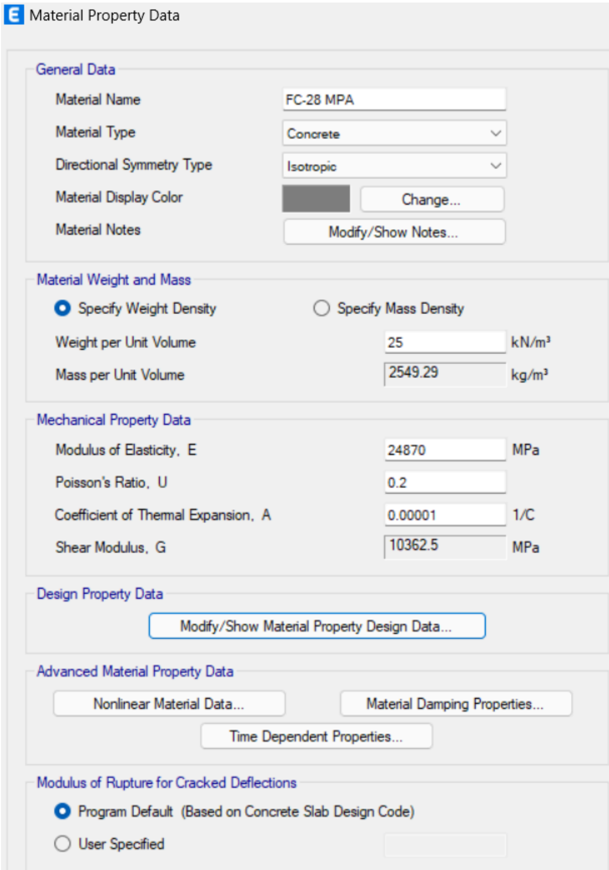


Figure 3.3: Concrete material property

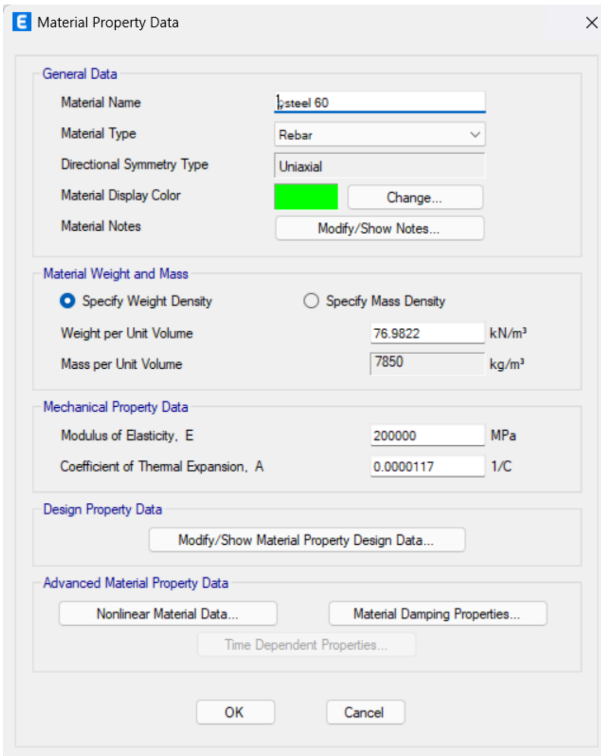


Figure 3.4: Rebar material property

3.2.4 Frame properties

The frame properties for beams and columns in the model include material specifications, section shapes, and dimensions. Beam properties are shown in **Figure 3.5**, while column dimensions and their respective frame properties are detailed in **Figure 3.6**. Columns oriented in opposite directions were defined with the same properties for consistency in the model.

Properties of slabs and walls were also defined in the model. Slab properties are shown in **Figure 3.7**. Wall properties were defined for two thicknesses: 200 mm, as shown in **Figure 3.8**. These definitions ensure accurate material behavior and load transfer in the structural analysis.

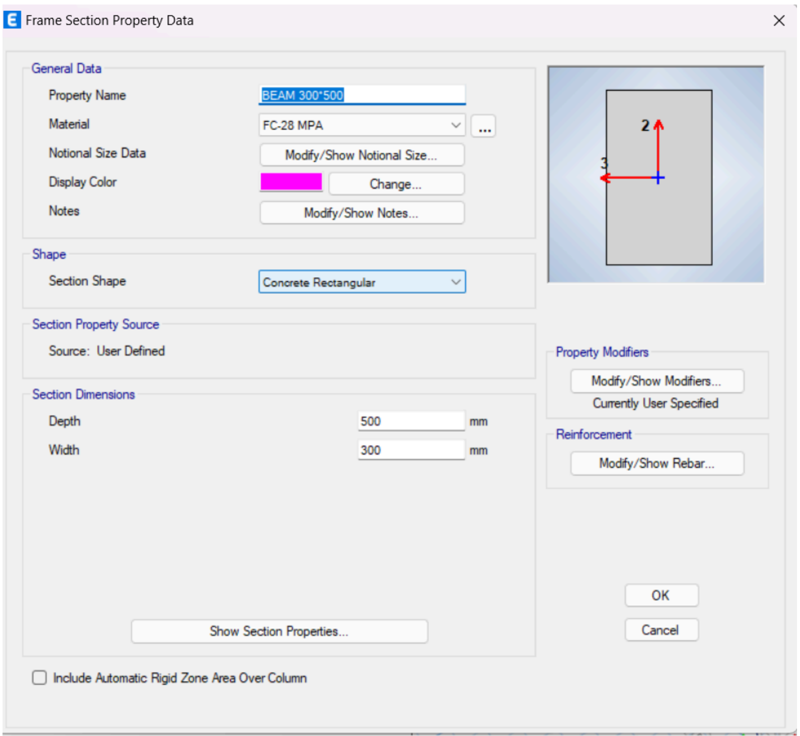


Figure 3.5: Beam frame section properties

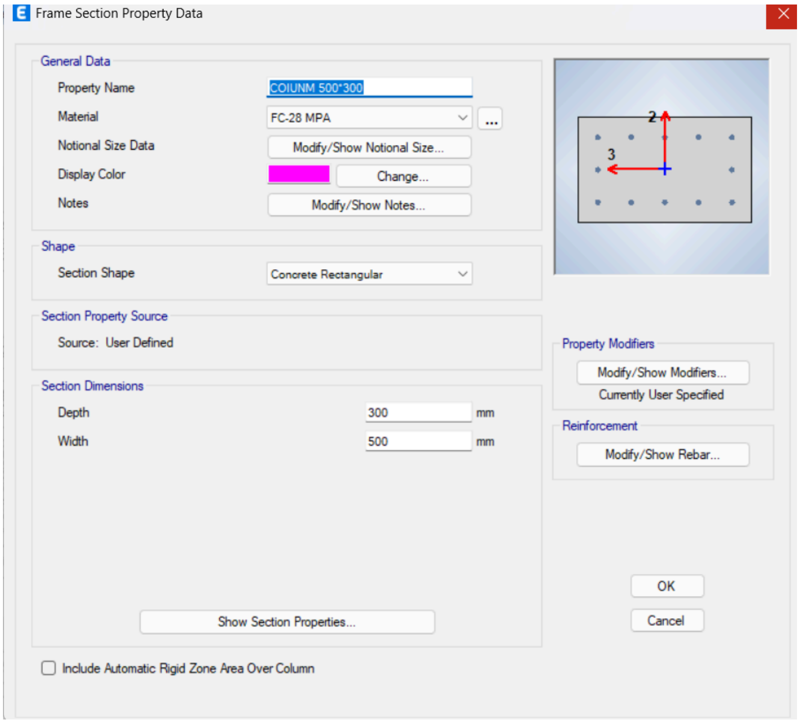


Figure 3.6: Column 300*500 frame section properties

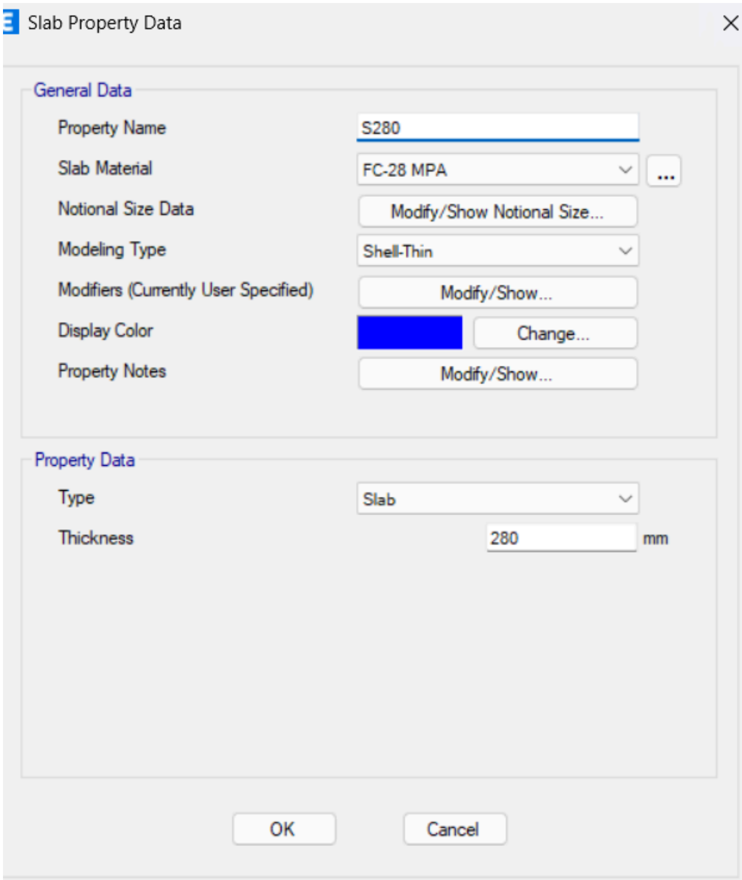


Figure 3.7: Slab section properties

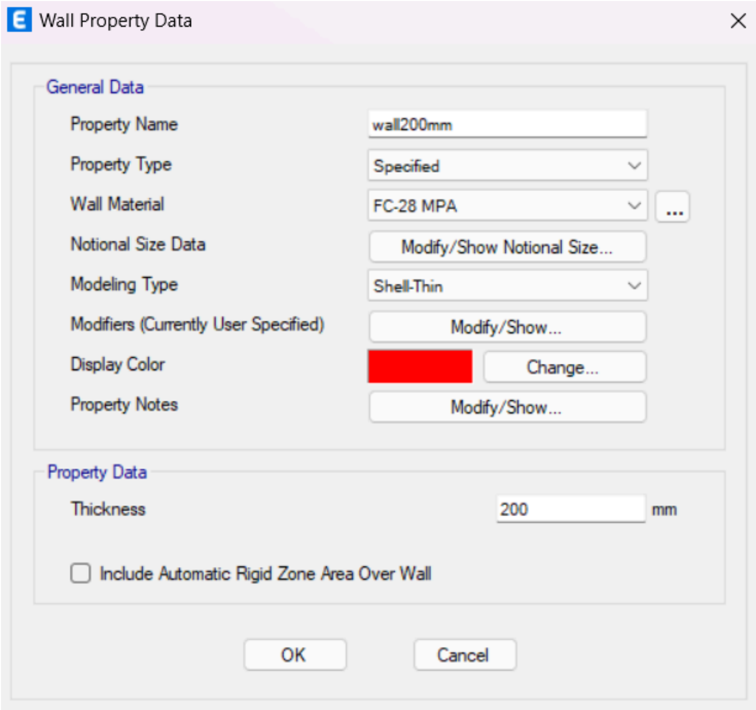


Figure 3.8: Wall section properties

3.2.5 Section modifiers

Section modifiers were applied in the ETABS model to adjust the stiffness properties of structural elements such as beams, columns, slabs, and walls. These modifiers account for material behavior under various loading conditions and ensure realistic simulation results by incorporating factors such as cracking, shrinkage, and creep. Applying section modifiers allows for more accurate analysis and design of the structural system.

Modifiers for each section were applied as follows:

- **Beams:** Modifier values are shown in **Figure 3.9**.
- **Columns:** Modifier values are shown in **Figure 3.10**.
- **Slabs:** Modifier factors are shown in **Figure 3.11**.
- **Walls:** Modifier details are shown in **Figure 3.12**.

These figures highlight the stiffness adjustment factors used in the ETABS model for each structural element.

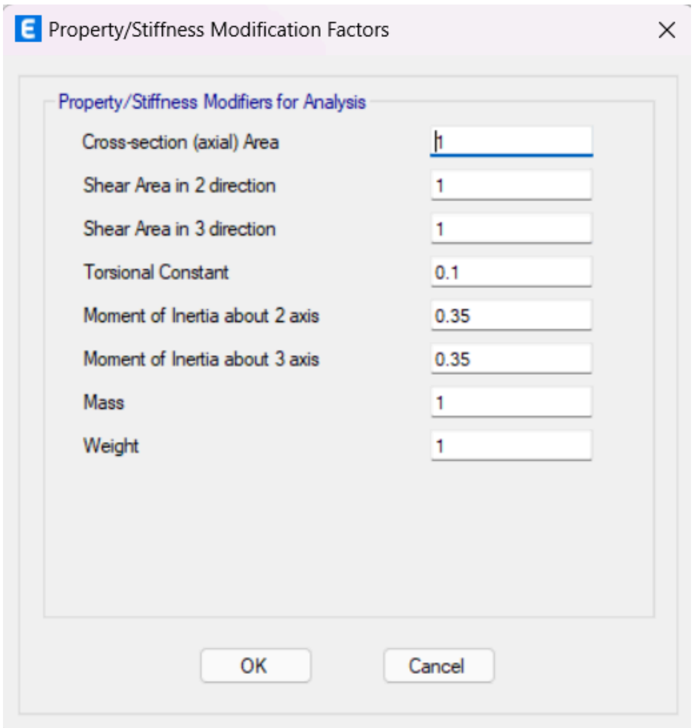


Figure 3.9: Beam section modifiers

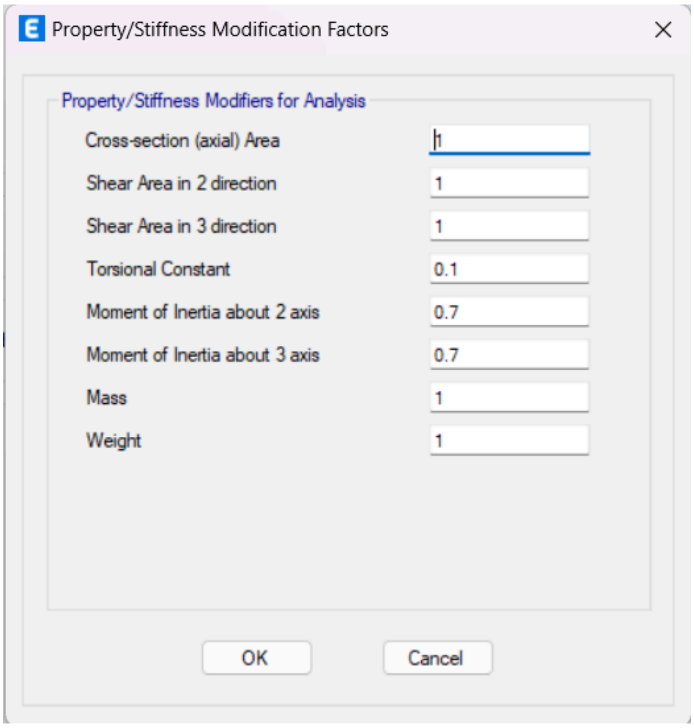


Figure 3.10: Column section modifiers

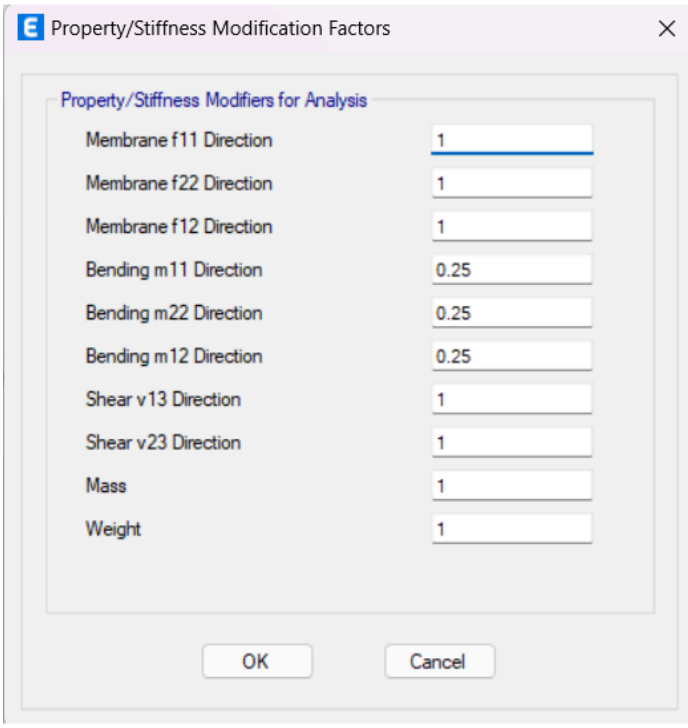


Figure 3.11: Slab section modifiers

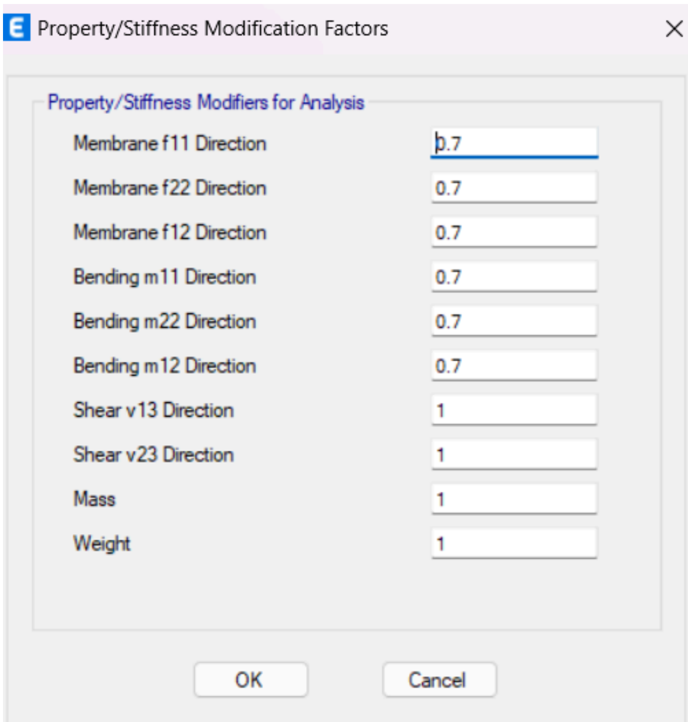


Figure 3.12: Wall section modifiers

3.2.6 Load patterns

The loads introduced into the ETABS model replicated the actual loads expected to act on the structure in real-life scenarios. These included gravity, live, superimposed, wind, snow, rain, and live roof, as defined in the project parameters. **Figure 3.13** presents the types of the loads as they were implemented in the modeling process.

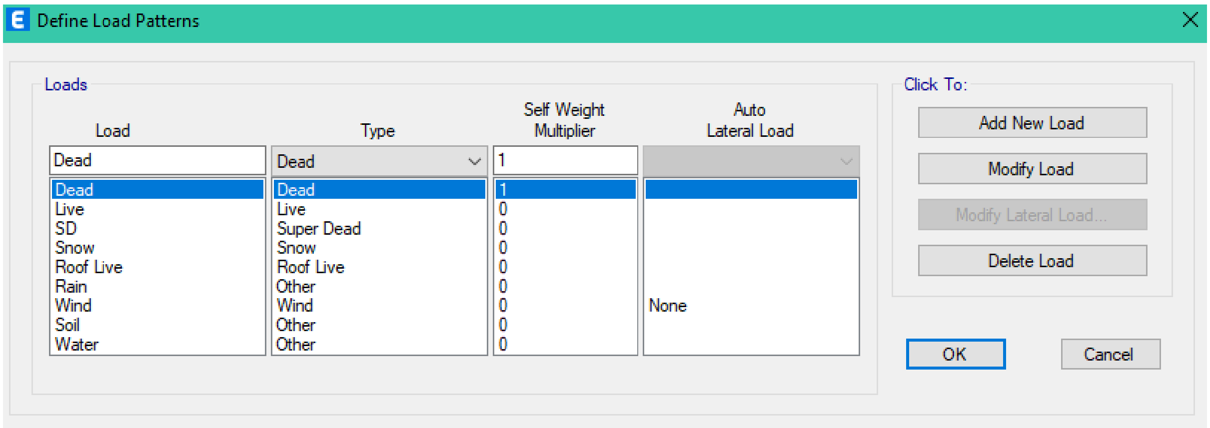


Figure 3.13: Load patterns

3.2.7 Load combinations

The load combination section introduces the various load combinations applied in the structural model. These combinations are crucial to ensure the structure's safety under normal and extreme conditions, reducing the risk of failure due to unexpected forces. By incorporating these combinations, the ETABS model checks the structural design against all possible scenarios to meet safety standards. **Figure 3.14** shows the load combinations, The load combinations were derived based on the guidelines in **ASCE 7-16**.

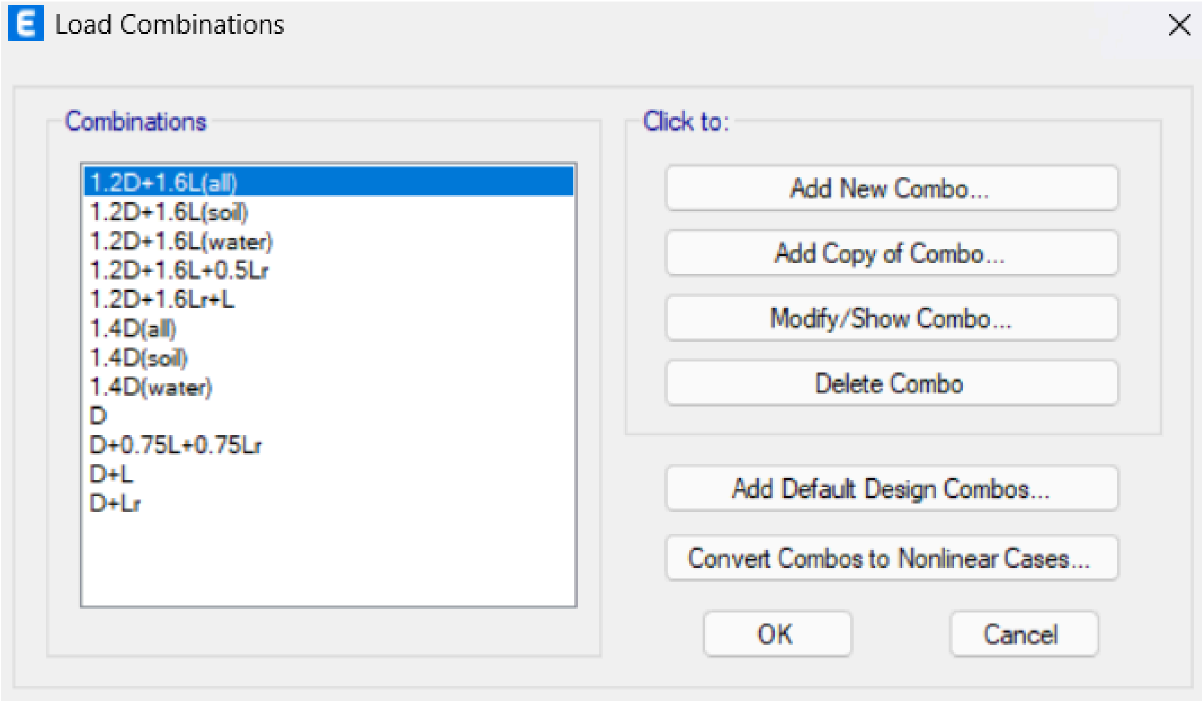


Figure 3.14: Load combinations

3.2.8 Loads assignments

The loads were assigned as outlined in **Tables 1.5** and **Table 1.6** presented in chapter 1. The superimposed dead load was applied to each slab on all floors, following the values specified in **Table 1.5**. The live load, however, varied across different panels and floors. As mentioned before, a uniform live load of 3 kN/m² was selected for residential floors for simplicity where typical values ranged between 2 kN/m² and 5 kN/m². Other live load values were applied following the values specified in **Table 1.6** that was presented earlier. Assigned load for various floors is shown in **Figures 3.15 to 3.18**.

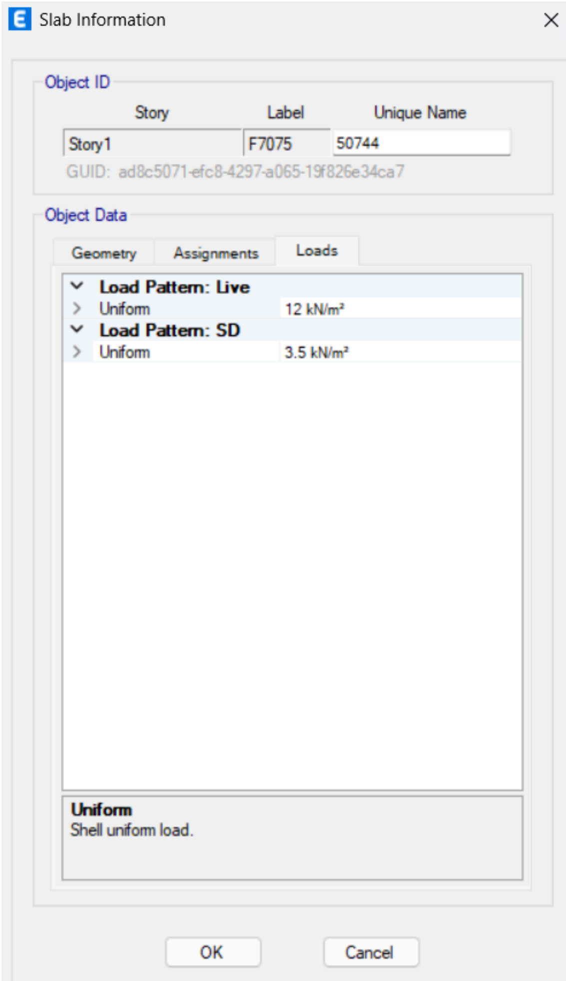


Figure 3.15: Assigned load at basement slab

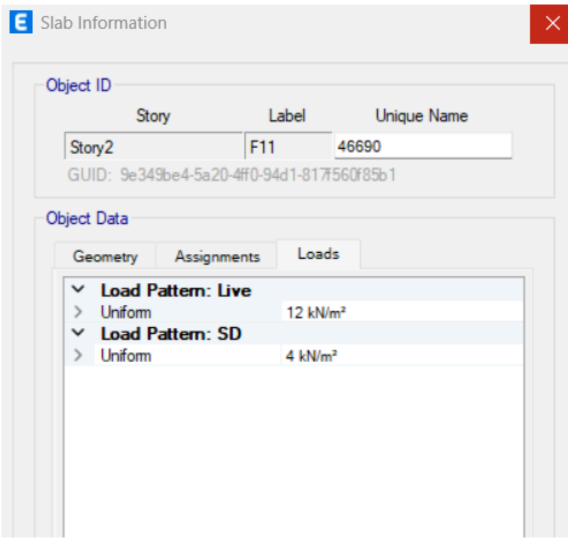


Figure 3.16: Assigned load at ground floor

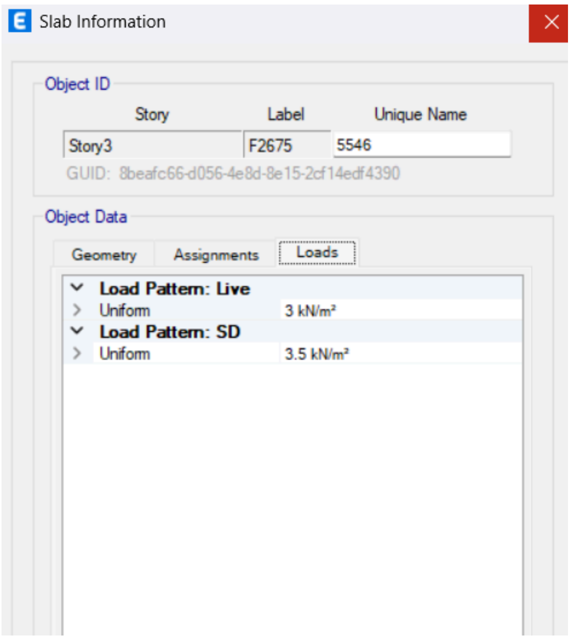


Figure 3.17: Assigned load at typical floors

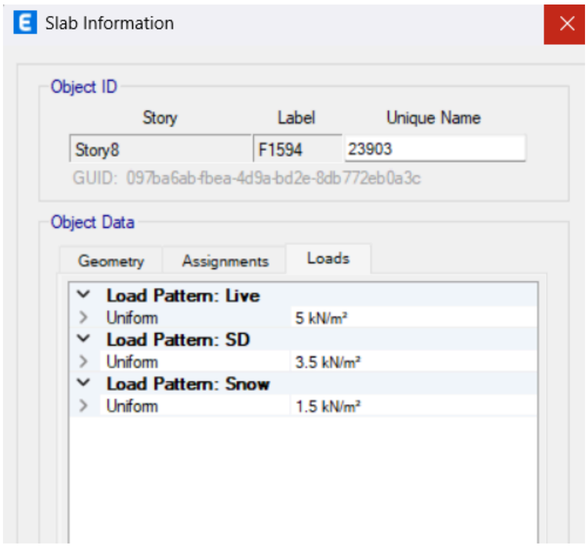


Figure 3.18: Assigned load at Roof floor

3.2.9 Springs

Springs were defined in ETABS to represent the soil's interaction with the foundation, ensuring the foundation's behavior closely mimics real-life conditions. **Figure 3.19** illustrates the spring definition. As shown, the allowable bearing capacity was set to 250 kN/m², corresponding to the soil's capacity.

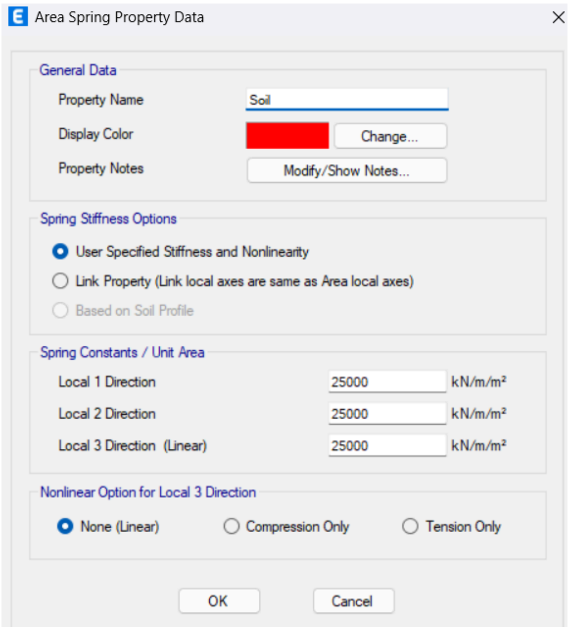


Figure 3.19: Area spring property data

3.2.10 Supports

Fixed supports were chosen to enhance the stability of the structure and to represent the foundation system accurately. These supports ensure the structure can resist rotations and displacements effectively under various loads, **Figure 3.20** shows joint assignment as fixed support. The fixed supports were used in the preliminary representation of the structural model before.

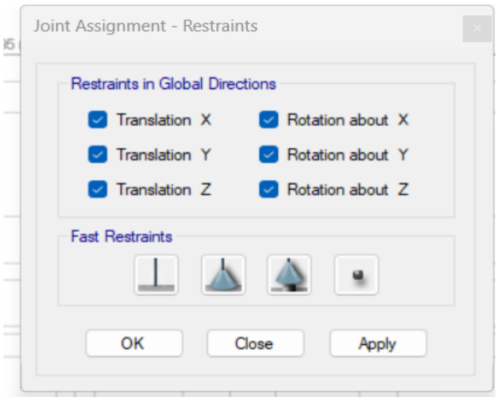


Figure 3.20: Assigned joint as fixed support

The fixed supports were used in the preliminary representation of the structural model before modeling the footing.

3.2.11 Codes

The primary code utilized for the structural modeling is **ACI 318-19**, which provides comprehensive guidelines for the design and construction of reinforced concrete structures. This standard ensures that all aspects of the structural system, including safety, durability, and performance under various loading conditions, comply with internationally recognized principles and practices, **Figure 3.21** shows codes used in modeling.

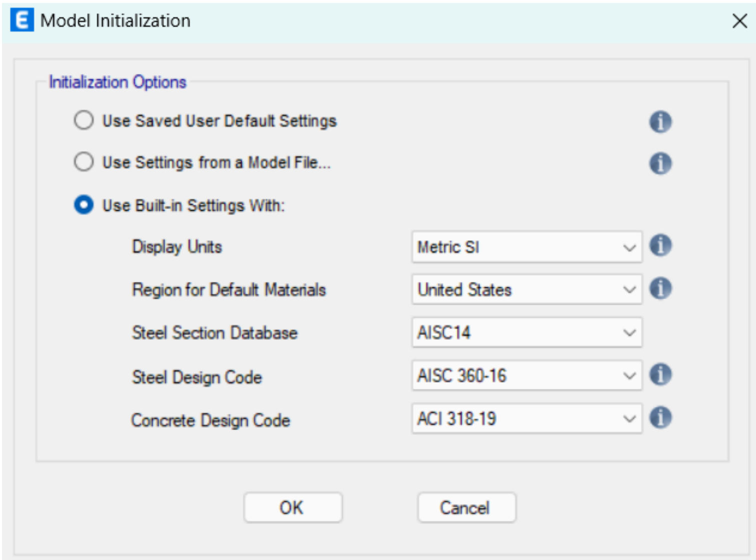


Figure 3.21: Selected codes for modeling

3.3 Evaluation of the preliminary design

3.3.1 Evaluation of the structural members

The model check indicated that some columns in both the ground floor and basement were overstressed, necessitating an adjustment in their dimensions. **Figures 3.22** and **3.23** illustrate the overstressed columns identified in the analysis. The slab thickness was adjusted to ensure that the punching shear force was adequately resisted on each story while optimizing the design for greater economic efficiency. The revised thickness values are detailed in **Table 3.3.** Meanwhile, the remaining parts of the building model showed satisfactory results within the allowable stress limits, large enough to withstand the applied loads and small enough to ensure an economical design. No modifications have been made to the dimensions of those parts. And finally, the modeled structural elements were enough for a good structural system; no additional elements were added.

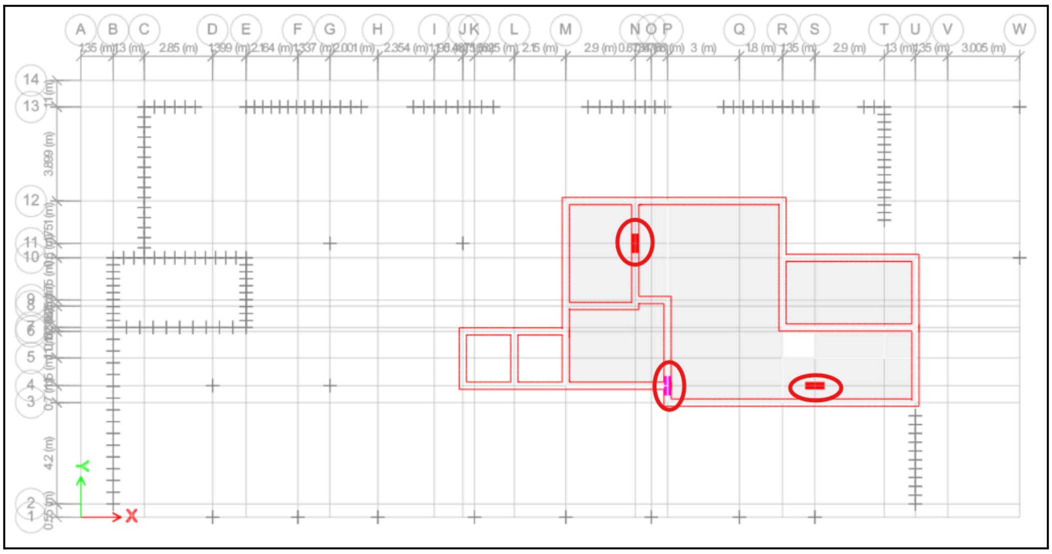


Figure 3.22: Basement over-stressed columns

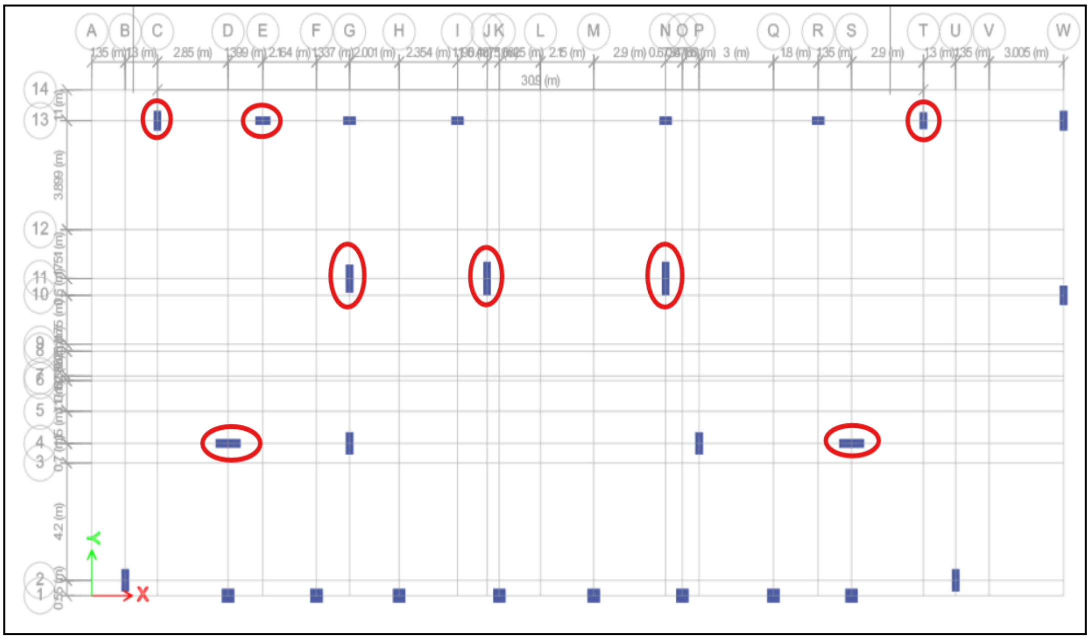


Figure 3.23: Ground Floor over stressed columns

3.3.2 Developed dimensions

Using the trial-and-error principle, the new dimensions for the over_stressed columns were determined to meet structural requirements. The updated dimensions are summarized in **Table 3.3**. This approach ensured that the columns could adequately resist the applied loads without exceeding allowable stress limits.

Table 3.3: Adjusted columns dimensions

Floor	Grid location	Previous dimension	Adjusted dimension (mm)
Basement	N11	800*300	1200*400
	P4	800*300	1000*300
	S4	800*300	1000*300
Ground floor	G11	800*300	1000*300
	J11	800*300	1200*300
	S4	800*300	1000*300
	N11	800*300	1200*300
	C13	300*500	300*700
	E13	300*500	300*600
	T13	300*500	300*600
	D4	800*300	1000*300

Table 3.4: Adjusted slab thickness

Floors	Previous thickness	Adjusted thickness
Ground, 1st, 2nd, 3rd, 4th, 5th	250 mm	220 mm
Roof	250 mm	280 mm

3.4 Verification of structural analysis

Software results strongly depend on modeling. Modeling mistakes lead to unrealistic results that do not simulate the real case. For this reason, checks are included in the report to ensure that the results are trustworthy. Compatibility check, gravity and soil loads, in addition to internal forces check are all discussed in this section.

3.4.1 Compatibility of the structure

The compatibility of the structure must be thoroughly checked to ensure its overall stability and to confirm the accuracy of the load calculations. This involves verifying that the loads determined through the ETABS model are consistent with the loads calculated manually, ensuring that both methods align. **Figure 3.24** shows that all parts work together and elements are compatible with each other.

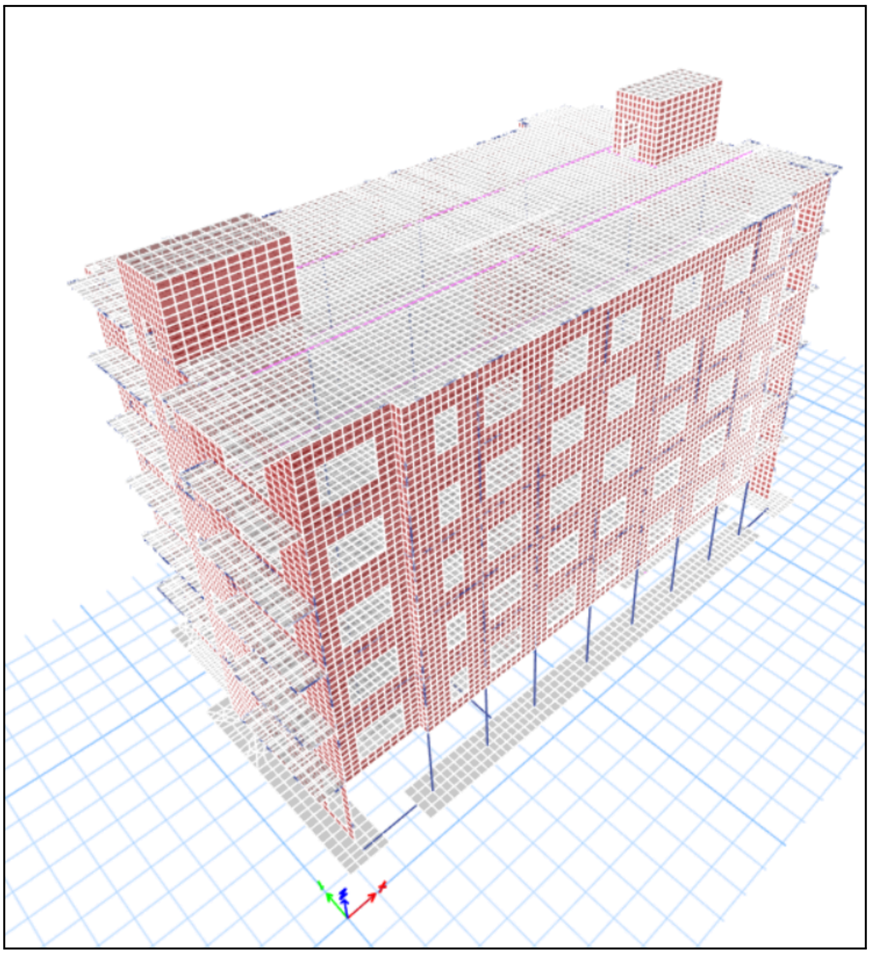
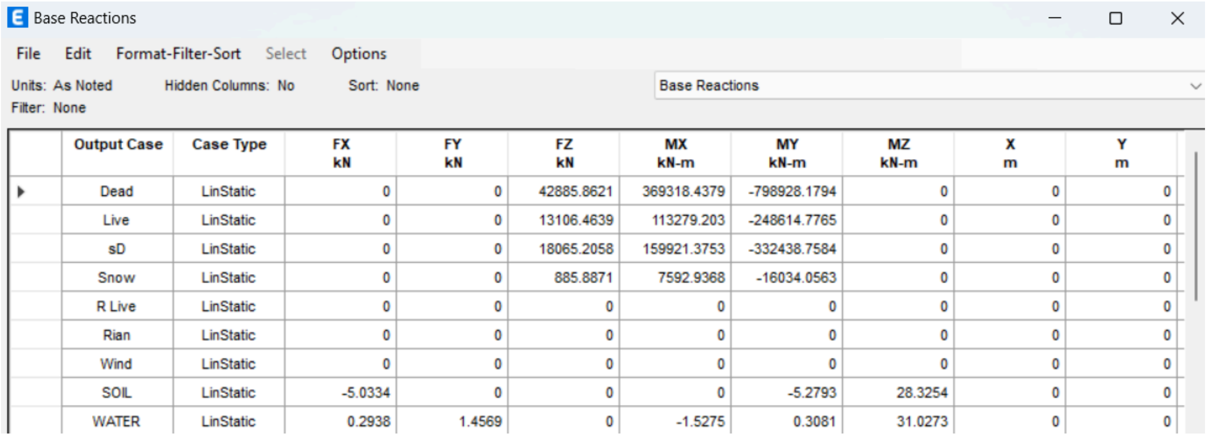


Figure 3.24: Compatibility checks

3.4.2 Gravity loads

Gravity loads are verified to ensure structural stability. Dead load, live load, superimposed dead load, and snow load are included in the analysis with a maximum allowable difference margin of 5% for each load. It is important to note that the gravity load check was applied without considering the footing. **Figure 3.25** shows the load values extracted from ETABS.


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
Dead	LinStatic	0	0	42885.8621	369318.4379	-798928.1794	0	0	0
Live	LinStatic	0	0	13106.4639	113279.203	-248614.7765	0	0	0
sD	LinStatic	0	0	18065.2058	159921.3753	-332438.7584	0	0	0
Snow	LinStatic	0	0	885.8871	7592.9368	-16034.0563	0	0	0
R Live	LinStatic	0	0	0	0	0	0	0	0
Rian	LinStatic	0	0	0	0	0	0	0	0
Wind	LinStatic	0	0	0	0	0	0	0	0
SOIL	LinStatic	-5.0334	0	0	0	-5.2793	28.3254	0	0
WATER	LinStatic	0.2938	1.4569	0	-1.5275	0.3081	31.0273	0	0

Figure 3.25: Gravity loads from ETABS

3.4.2.1 Check equilibrium for dead loads

A dead load equilibrium check was performed before modeling the foundations in ETABS. **Table 3.5** presents the calculated dead load values used for the equilibrium test.

Table 3.5: Calculated dead loads

Element	Dimensions	Dead load (kN)
Beams	300*500	2244.975
Slabs	thickness = 220 mm	21022.70153
Column 1	500*300	714.5625
Column 2	800*300	281.1
Column 3	300*300	55.9125
Column 4	300*600	335.475
Column 5	300*700	46.2
Column 6	300*1000	111.75
Column 7	300*1200	39.6
Column 8	1200*400	36.6
Column 9	500*500	1107.5
Wall 1	thickness = 200 mm	7439.130475
Wall 2	thickness = 300 mm	10754.15144
Total		**44189.65844**

As shown in **Table 3.5** The total calculated Dead Load value is **44189.65844** kN (hand calculated).

Dead Load from Etabs= 42885.86 kN.

Calculated Dead load= 44189.65 kN.

%difference = (Dead Load from Etabs- Calculate Dead load)/Calculate Dead load*100%.

%difference = (42885.86-44189.658)/44189.658*100% = **2.95% < 5% ✔**.

3.4.2.2 Check equilibrium for superimposed

Table 3.6 presents the calculated superimposed dead load values used for the equilibrium test.

Table 3.6: Calculated superimposed dead loads

Floor	Area(m^2)	SD(KN/m^2)	Wall 300(KN)	Wall 200(KN)	Beam(KN)	Slab(KN)	
Ground	105.27	4	97.5612	584.96196	62.04	421.08	
First	569.24535	3.5	80.640948	773.276	49.05	1992.358725	
Second	542.0266	3.5	80.640948	773.276	49.05	1897.0931	
Third	542.0266	3.5	80.640948	773.276	49.05	1897.0931	
Fourth	542.0266	3.5	80.640948	773.276	49.05	1897.0931	
Fifth	542.0266	3.5	80.640948	773.276	49.05	1897.0931	
Roof	558.5416	3.5	81.113	0	0	1954.8956	
Staircase	208.351872	3.5	0	0	0	729.231552	
Summation			581.87894	4451.34196	307.29	12685.93828	18026.45

As shown in **Table 3.6**, the total calculated Superimposed Dead Load value is 18026.45 kN.

Superimposed Dead Load from Etabs=18065.02KN.

Calculated Superimposed Dead Load= 18026.45kN.

%difference = (S.D Load from Etabs- Calculate S.D Load)/Calculate S.D Load*100%.

%difference = (18065.02- 18026.45)/ 18026.45*100% = 0.2% < 5% ✔.

3.4.2.3 Check equilibrium for live loads

Table 3.7 presents the calculated live load values used for the equilibrium test.

Table 3.7: Calculated live loads

Floor	Area (m^2)	Live load (kN/m^2)	(kN)
Ground	105.27	12	1263.24
First	569.24535	3	1707.73605
Second	542.0266	3	1626.0798
Third	542.0266	3	1626.0798
Fourth	542.0266	3	1626.0798
Fifth	542.0266	3	1626.0798
Roof	558.5416	5	2792.708
Staircase	208.351872	5	1041.75936
Summation			**13309.76261**

As shown in **Table 3.7** the total calculated live load value is 13309.76261 kN.

Live Load from Etabs= 13106.5432 kN.

Calculated Live Load load= 13309.76261 kN.

%difference = (Live Load from Etabs- Calculate Live Load)/Calculate Live Load*100%.

%difference = (13106.5432- 13309.76261)/13309.7626*100% = **1.526% < 5% ✔**.

3.4.2.4 Check equilibrium for snow loads

Table 3.8 presents the calculated snow load values used for the equilibrium test.

Table 3.8: Calculated snow load

Floor	Area (m^2)	Snow load (kN/m^2)	Snow load (kN)
Roof	558.5416	1.5	837.8124
Top rood	32.053	1.5	48.0795
Total			885.8919

As shown in **Table 3.8** The total calculated snow load value is 885.8919 kN.

Snow load from Etabs= 885.887 kN.

Calculated snow load= 885.8919 kN.

%difference = (snow load from Etabs- Calculate snow load)/Calculate snow load*100%.

%difference = (885.887- 885.8919)/885.8919*100% = **0% < 5% ✔**.

3.4.3 Soil loads

Check equilibrium:

The force that was taken into consideration is shown in **Table 3.9**.

Table 3.9: Force taken into consideration

Loads	Dead load	Super dead load	Live load
Value(kN/m^2)	2.5	4	12

The coefficient of the soil lateral pressure (k)=0.66

Ɣs=18 kN/m^3

calculations :

q1= W*K= 18.5*0.66= 12.21kN/m^2

q2=Ɣs*h*k+(w*k)= 18*3.05*0.66+(12.21)= 48.44kN/m^2

final equation (q)=-11.88Z+36.55

3.4.4 Verification of internal forces

This section aims to verify whether the software analysis aligns with the expected behavior or if errors were made during modeling. Internal forces are manually calculated and compared with the software results.

The manual calculation methods used are approximate and intended to identify potential errors rather than provide precise verification. Due to the complexity of accurate analysis, software is required for detailed assessments, so minor differences are expected. Additionally, some methods, such as the Direct Design Method, have limitations. In this case, certain conditions, like the panel dimensions and span ratios in slabs, are not fully met, leading to an accepted difference of up to 30% in some areas after applying the method to the most representative part of the structure.

3.4.4.1 Beams

Bending moment and shear forces in beams are almost equal to zero. This expectation is true for those beams which are supported by bearing walls, and this is confirmed by software analysis. However, there are three beams in the ground floor that are supported by columns at their edges rather than a wall along their full length, thus, internal forces are expected to be higher in these beams and show necessity to be checked.

Bending moments and shear forces in beams are generally close to zero, which is true for beams supported by bearing walls, as confirmed by the software analysis. However, three beams on the ground floor are supported by columns at their edges, rather than being fully supported along their length by walls. As a result, higher internal forces are expected, and these beams require additional checks.

Bending moment check for column-supported beams:

The beams support a solid slab that is 220mm thick, resulting in a dead load of 5.5 kN/m². The slab is subjected to a live load of 3.0 kN/m² and a superimposed dead load of 3.5 kN/m². Based on these values, the ultimate load q_{ult}, in accordance with the ASCE 7-16 ultimate load combinations, is calculated as:

$$q_{ult} = 1.2D+1.6L$$

$$1.2 \times (5.5+3.5) + 1.6 \times (3) = 15.6 \text{ KN/m}^2$$

For this load combination, the bending moment M3-3 and shear 2−2 were obtained from ETABS, as shown in **Figures 3.26–3.29**. Focusing on the column-supported beam along grid line W (between grid lines 10 and 13), the beam is assumed to carry half the slab load in the X direction, following one-way slab behavior. By multiplying q_{ult} by half of the slab's span in the X direction, the equivalent distributed load w on the beam is calculated as:

$$w = (5.65/2) \times 15.6 = 44.07 \text{ KN/m}$$

In a single-span reinforced concrete beam with monolithic beam-column connections, partial fixity at the supports reduces the midspan positive moment and introduces negative moments near the supports, compared to a simply supported beam. A fully fixed beam experiences negative moments of $-wl^2/12$ and a reduced midspan moment of $+wl^2/24$, while the partially restrained beam behavior lies between these extremes, depending on the support stiffness and span. Shear force diagrams for such beams are similar to those of simply supported beams, but maximum shear is slightly reduced due to the moment resistance at the connections (Hibbeler, 2015, Chapter 6).

Fully fixed supports:

$$M_{support} = -wl^2/12 = 44.07 \times 5.55 \times 5.55 / 12 = 113.122 \text{ KN.m}$$

$$M_{midspan} = +wl^2/24 = 44.07 \times 5.55 \times 5.55 / 24 = 56.561 \text{ KN.m}$$

Simply supported:

$$M_{midspan} = +wl^2/8 = 44.07 \times 5.55 \times 5.55 / 8 = 169.683 \text{ KN.m}$$

$$V_{support} = wl/2 = (44.07 \times 5.55) / 2 = 122.294 \text{ KN}$$

ETABS results (partial fixity + two-way slab action):

- Midspan moment: $Mu \approx 58$ kN.m
- Support shear: $Vu \approx 78$ kN

These values lie between the fixed-end and simply-supported extremes and are considered acceptable. The differences arise from the actual rotational restraint at the beam-column connections and the one-way slab load transfer assumption.

In conclusion, **the internal forces of beams shown by software match the expected values**. **Figures 3.26** and **3.27** show the bending moment (M3-3) for the beams at the ground floor and typical floors, respectively, while **Figures 3.28 and 3.29** illustrate the corresponding shear force (Shear 2-2) for the same floors.

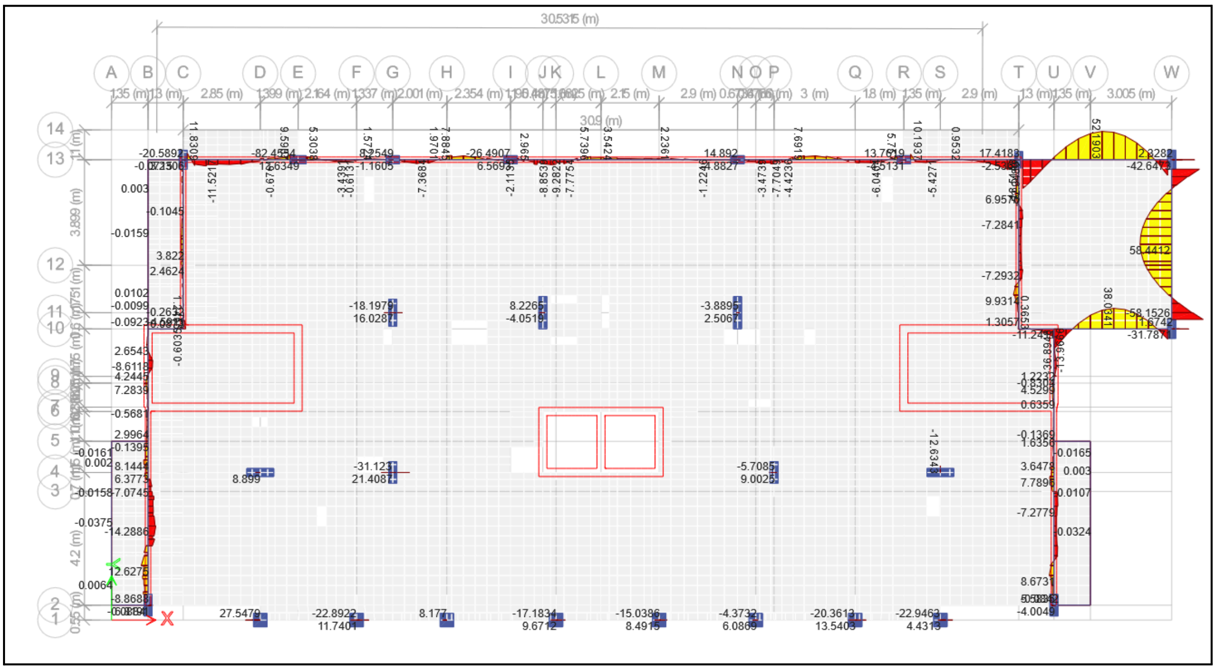


Figure 3.26: Bending moment M3-3 in beams at the ground floor

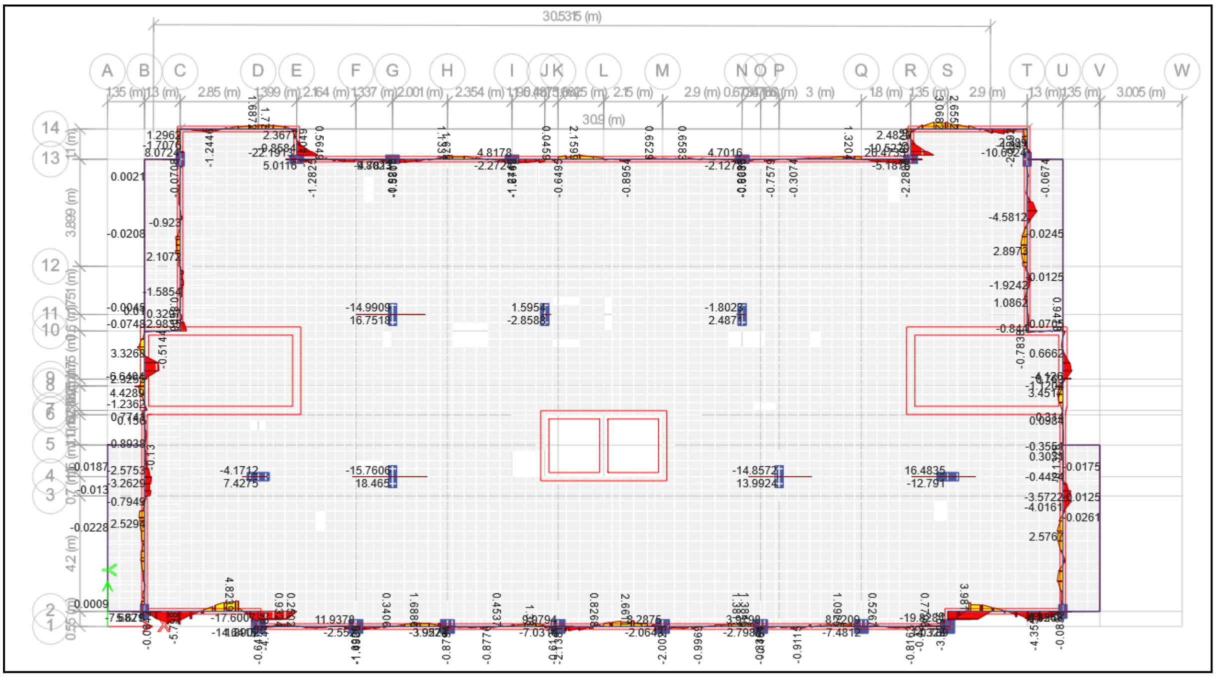


Figure 3.27: Bending moment M3-3 in beams at typical floors

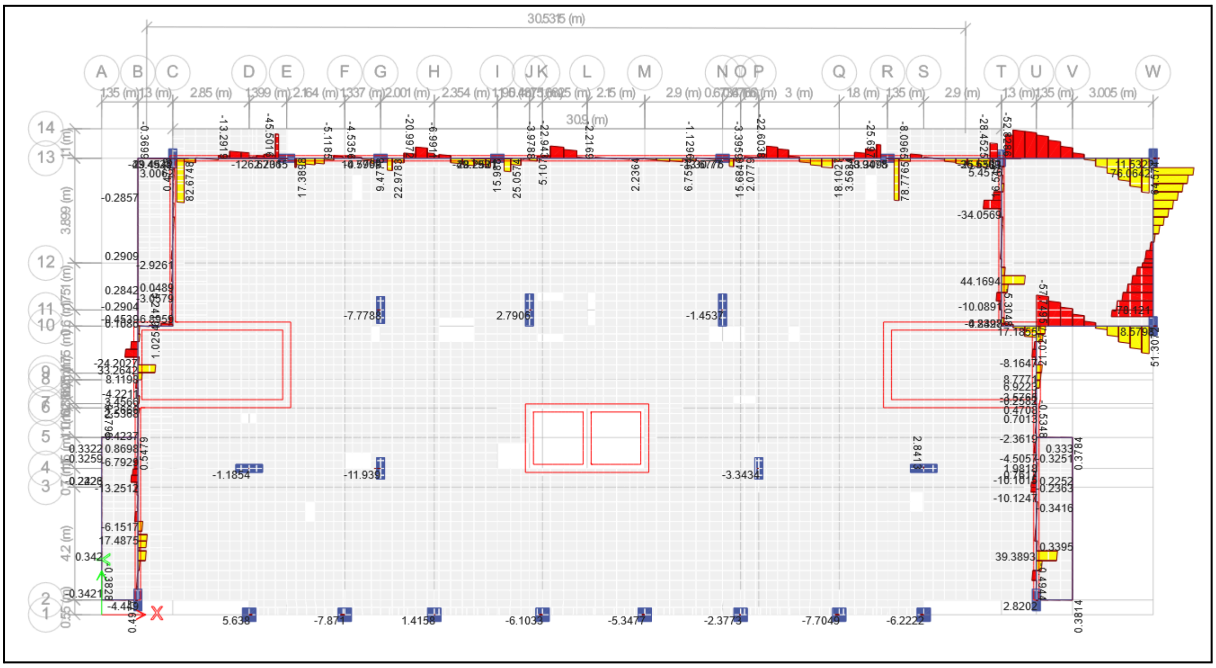


Figure 3.28 Shear 2-2 values in beams at the ground floor

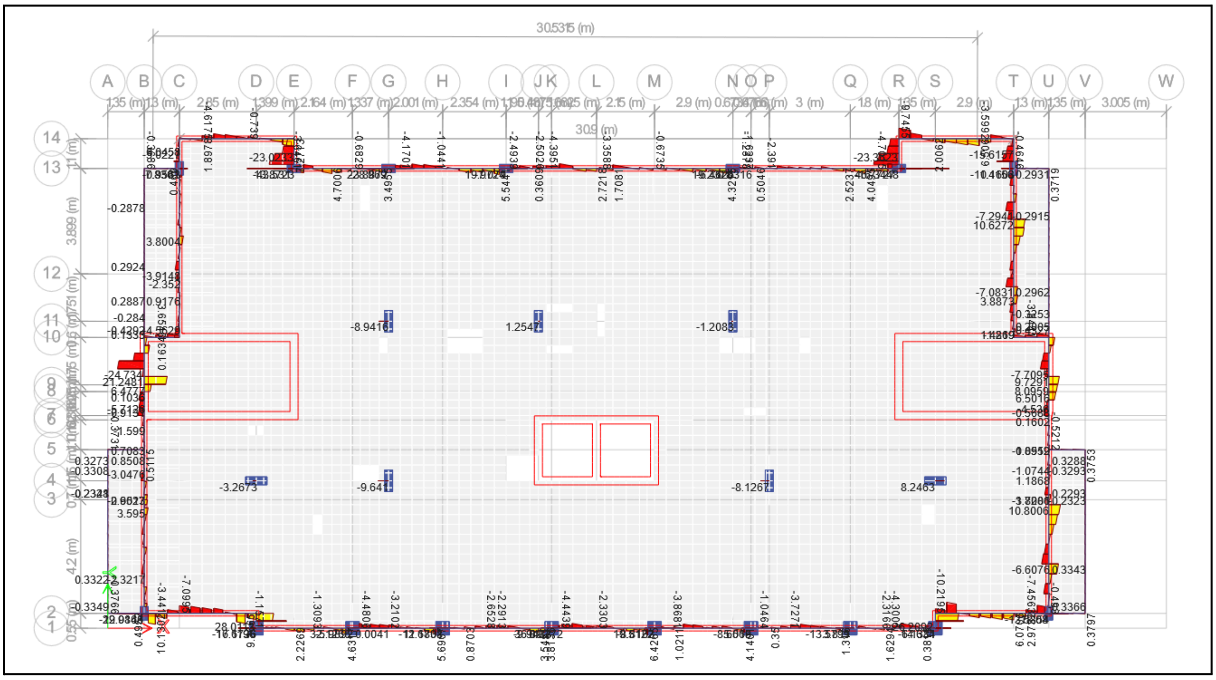


Figure 3.29: Shear 2-2 values in beams at typical floors

3.4.4.2 Columns

Axial loads of the internal columns can be calculated approximately using the **Tributary Area Method** where the axial load on a column equals the tributary area for the column (m^2) multiplied by ultimate load per square meter (kN/m^2). Hand calculations for axial loads of the internal columns in the second floor are illustrated in **Table 3.10** compared with the axial load results (from ETABS) for the same columns in the same floor.

External columns are expected to carry low axial load compared with internal columns. In this structure, external columns are located between bearing walls, this means an even lower axial load. **Figure 3.30** shows the axial loads of all columns in the second floor and the difference is noticeable and clear as expected.

Table 3.10: Axial loads in story5 (Second floor)

Column	Axial load kN(ETABS)	Axial load kN(calculated)	Difference %
D4	1100	1083.75	-1.499423299
G11	1659	1656.65	-0.141852534
G4	1656	1926.25	14.02985075
J11	1711	1926.25	11.17456197
N11	2148	2465.45	12.87594557
P4	1760	1959.95	10.20179086
S4	1097	1252.25	12.39768417

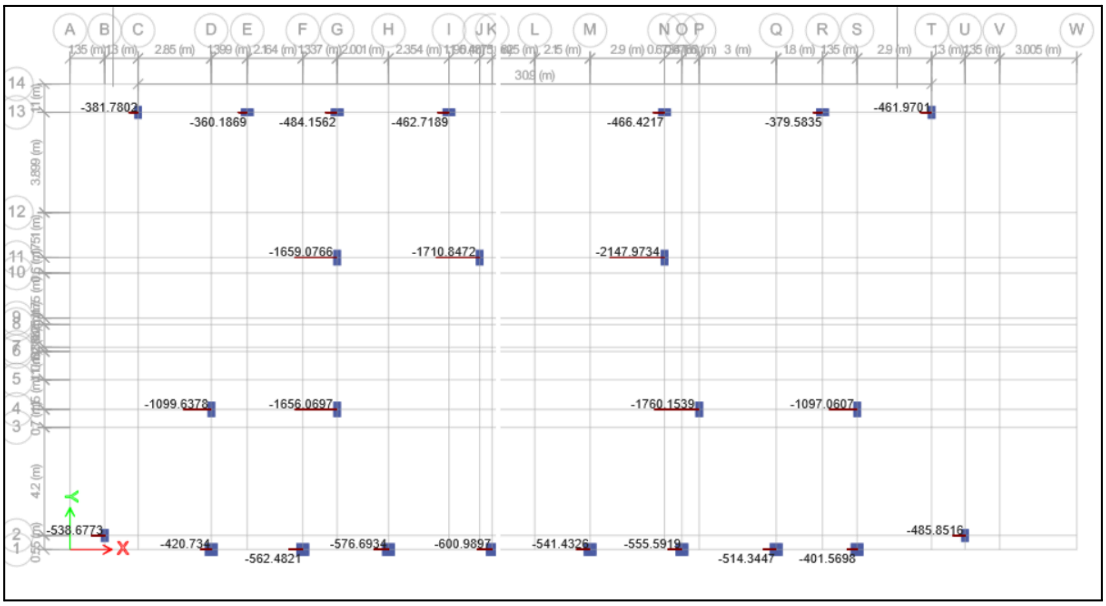


Figure 3.30: Axial load in story 5 from ETABS

3.5 Deflection computations

To perform the displacement check, the highest deflection value on each floor was identified, and the most critical panel was selected for detailed evaluation. **Figure 3.31** illustrates the panel exhibiting the maximum recorded deflection.

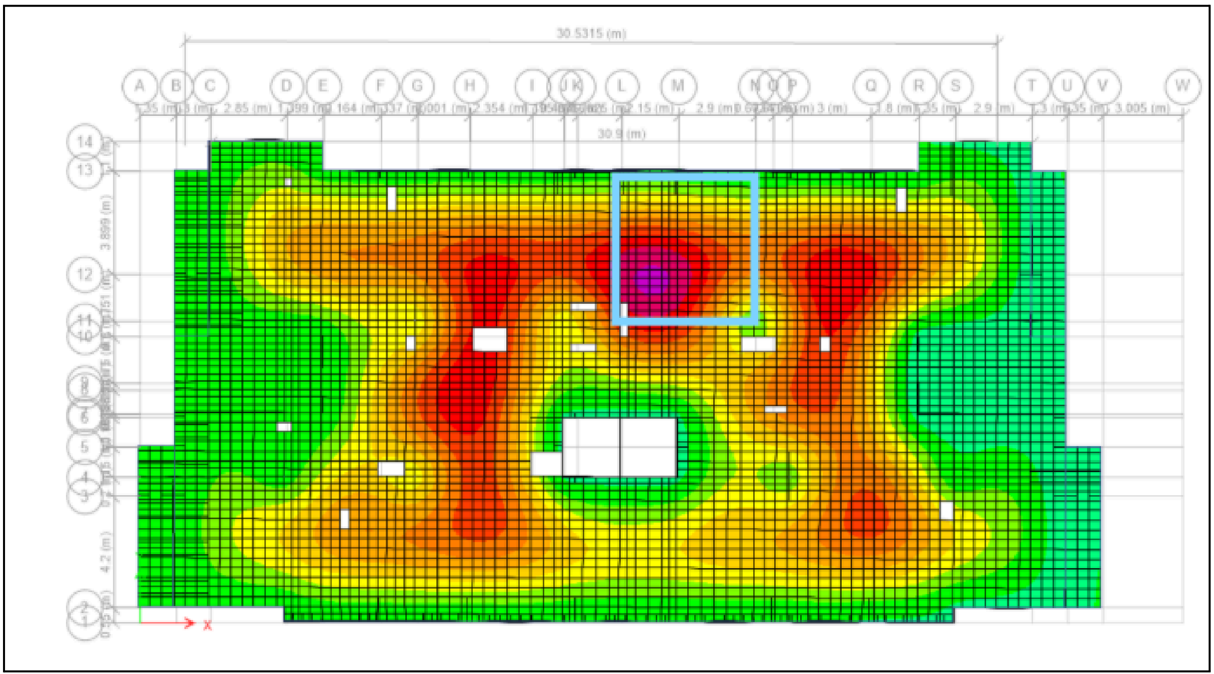


Figure 3.31: Panel with largest deflection

The deflection from point 1 to 9 is shown in **Table 3.11** below.

Note: that the points are arranged in a horizontal sequence

Table 3.11: Deflection from point 1 to 9

Point	1	2	3	4	5	6	7	8	9
Deflection (mm)	28.87	30.3	25.2	29.8	35.6	24.3	24.2	33.4	16.7

Table 3.12 shows the average deflection of the panel at level **z=22.65m**.

Table 3.12: Average deflection

Line	1,2,3	4,5,6	7,8,9	1,4,7	2,5,8	3,6,9
Average deflection(mm)	3.265	8.55	12.95	3.265	3.75	3.35
Allowable deflection(mm)	7712.5\240 =32.15	7712.5\240 =32.16	7712.5\240 =32.15	5650/240 =23.54	5650/240 =23.54	5650/240 =23.54
Acceptable or not	Acceptable	Acceptable	Acceptable	Acceptable	Acceptable	Acceptable

Calculation:

- Average deflection of line 1-2-3= 30.3 – (25.2+28.87)/2=3.265mm less than allowable 32.15mm (so ok).
- Average deflection of line 4-5-6= 35.6 – (24.3+29.8)/2 =8.55mm less than allowable 32.15mm (so ok).
- Average deflection of line 7-8-9= 33.4– (16.7+24.2)/2=12.952mm less than allowable 32.15mm (so ok).
- Average deflection of line 1-4-7= 29.8 – (24.24+28.87)/2=3.265mm less than allowable 23.54 mm (so ok).
- Average deflection of line 2-5-8= 35.6 – (30.3+33.4)/2=3.75 mm less than allowable 23.54 mm (so ok).
- Average deflection of line 3-6-9= 24.3 – (25.2+16.7)/2=3.35 mm less than allowable 23.45 mm (so ok).

The deflection is acceptable.

3.6 Verification of structural design

This section presents the comparison between manually calculated design requirements and the design output obtained from ETABS. A representative sample from each structural member type was selected for manual verification. This approach serves as a validation step to confirm the reliability of the software-generated results and to establish confidence in applying the remaining ETABS outputs during the full design phase. The comparison demonstrated consistency between manual checks and software results. Detailed comparisons for all member types are presented in the following subsections.

3.6.1 Beams

All beams were subjected to detailed structural verification under flexure, shear, and torsion to ensure compliance with the applicable design codes and to confirm the adequacy of the reinforcement provided. These checks were performed in accordance with the governing ACI 318-19 provisions and based on results obtained from ETABS analysis, supported by sample manual calculations where necessary. The following subsections summarize and validate the 12design performance of selected beams under each of these critical actions.

3.6.1.1 Flexural design

As previously noted, internal forces in wall-supported beams are negligible. Accordingly, the minimum flexural reinforcement ratio of 0.0033, as specified by ACI 318-19 Section 9.6.1.2, governs the design in such cases. This corresponds to a required steel area of:

Minimum area of steel = 0.0033 × 460 × 300 = 455 mm²,

This value aligns precisely with the ETABS output, which reported a required steel area of 455 mm², as shown in **Figure 3.32**.

For beams subjected to relatively higher internal forces, particularly those supported by columns, the required steel ratio was determined using Equation 3.12 from *Arman (2025b)*. For the critical beam located along the W-Grid Line — identified as the member with the highest design demands — the calculated steel ratio is:

$$\rho = \frac{0.85 f'_c}{f_y}\left(1 - \sqrt{1 - \frac{2.61 M_u}{b d^2 f'_c}}\right) \quad \text{(Arman, 2025b, Eq. 3.12)}$$

$$\rho = \frac{0.85 \times 28}{420}\left(1 - \sqrt{1 - \frac{2.61 \times 58E6}{28 \times 300 \times 460^2}}\right) = 0.0024$$

Where:

- f′c=28 MPa

- fy=420 MPa
- M_u = 58 kN.m (Shown in **Figure 3.33**)
- b= 300mm
- d= 460mm

The calculated required steel ratio for the critical beam was found to be 0.0024, which is less than the minimum ratio of 0.0033. Consequently, the minimum ratio of 0.0033 was adopted. This value was applied to all column-supported beams, as the selected member represents the most demanding case. **Figure 3.32** shows the corresponding steel area provided by ETABS based on this minimum ratio, confirming consistency with the manual calculations.

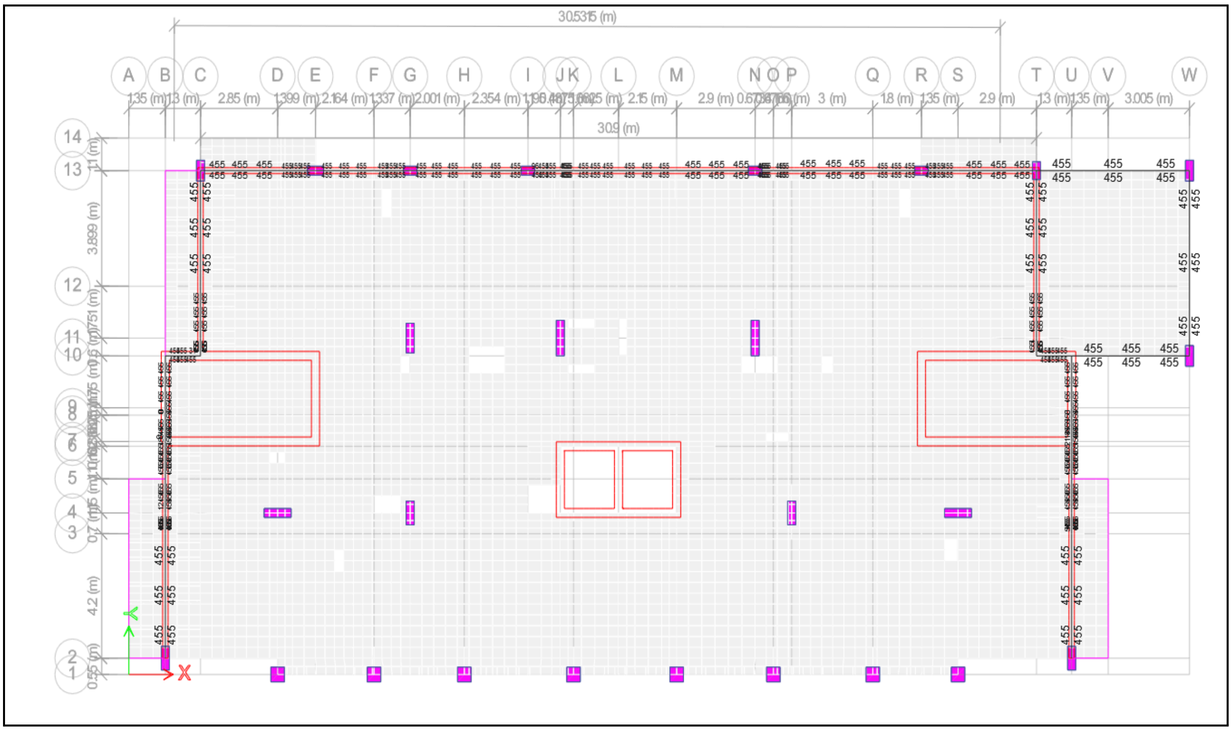


Figure 3.32: Flexural area of steel in beams

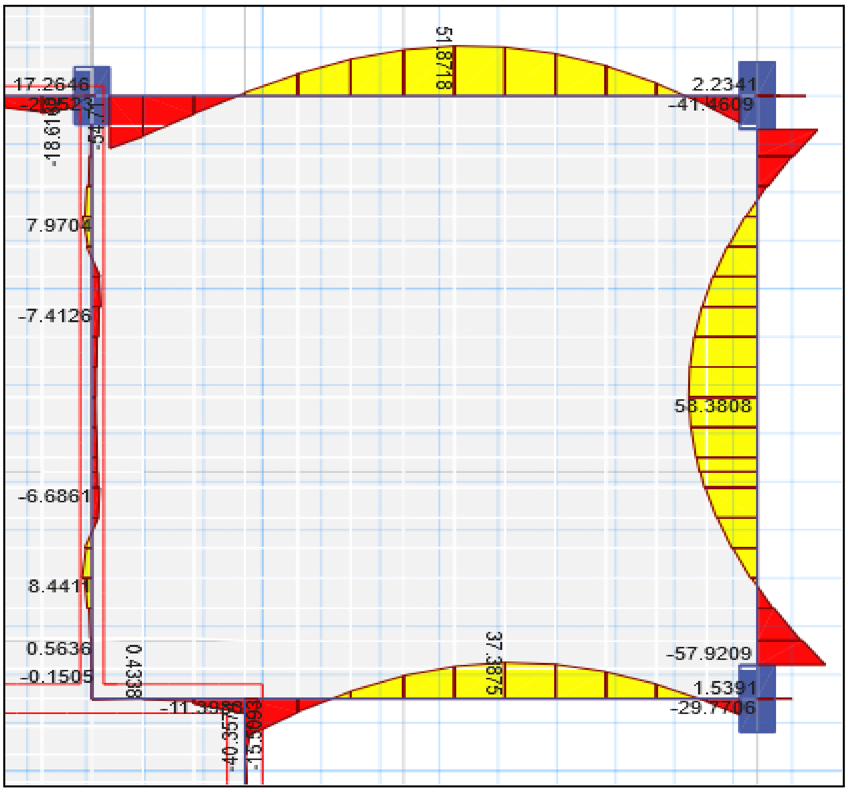


Figure 3.33: Maximum bending moment value in column-supported beams

3.6.1.2 Shear design

Beams, whether supported by structural walls or columns, exhibited relatively low shear values in the ETABS output. To verify shear reinforcement results, a manual check was performed on the beam with the highest reported shear demand. This beam was selected as the critical case for comparison. According to ETABS, the ultimate shear force in this beam was:

Vu = 78 kN (Shown in **Figure 3.34**)

To confirm this, the concrete shear capacity (ϕVc) was calculated manually and compared with the applied shear force. The calculation follows **ACI 318-19 Section 22.5.5.1**, which gives the expression for the nominal shear strength provided by concrete in non prestressed members without axial force:

$$V_c = \frac{1}{6}\lambda\sqrt{f_c'}b_w d \quad \text{(ACI 318-19, Section 22.5.5.1)}$$

Accordingly:

$$\phi V_c = \phi \cdot V_c = 0.75 \cdot \left(\frac{1}{6}\lambda\sqrt{f_c'}\, b_w d\right)$$

Assuming:

- $f'c=28$ MPa
- $b_w=300$ mm
- $d=460$ mm
- $\lambda=1.0$ (normal-weight concrete)

Then:

$$Vc = \frac{1}{6} \text{ x } 1 \text{ x } \sqrt{28} \text{ x } 300 \text{ x } 460 \approx 121.7\text{kN}$$

$$\text{Ø}Vc = 0.75 \text{ x } 121.7 = 91\text{kN}$$

Since:

$$\text{Ø}Vc = 91\text{kN} > Vu = 78\text{kN}$$

The manual check confirms the ETABS result that no shear reinforcement is required from a strength perspective. As shown in **Figure 3.35**, ETABS reports a required shear reinforcement area of 0 mm². While this aligns with the strength-based check, it does not account for minimum reinforcement provisions. According to ACI 318-19 Section 9.6.3.2, beams not forming part of slab or joist systems must still contain minimum transverse reinforcement, regardless of whether ϕVc exceeds Vu. This criterion is checked manually and enforced during the design process, even if not reflected in the software output.

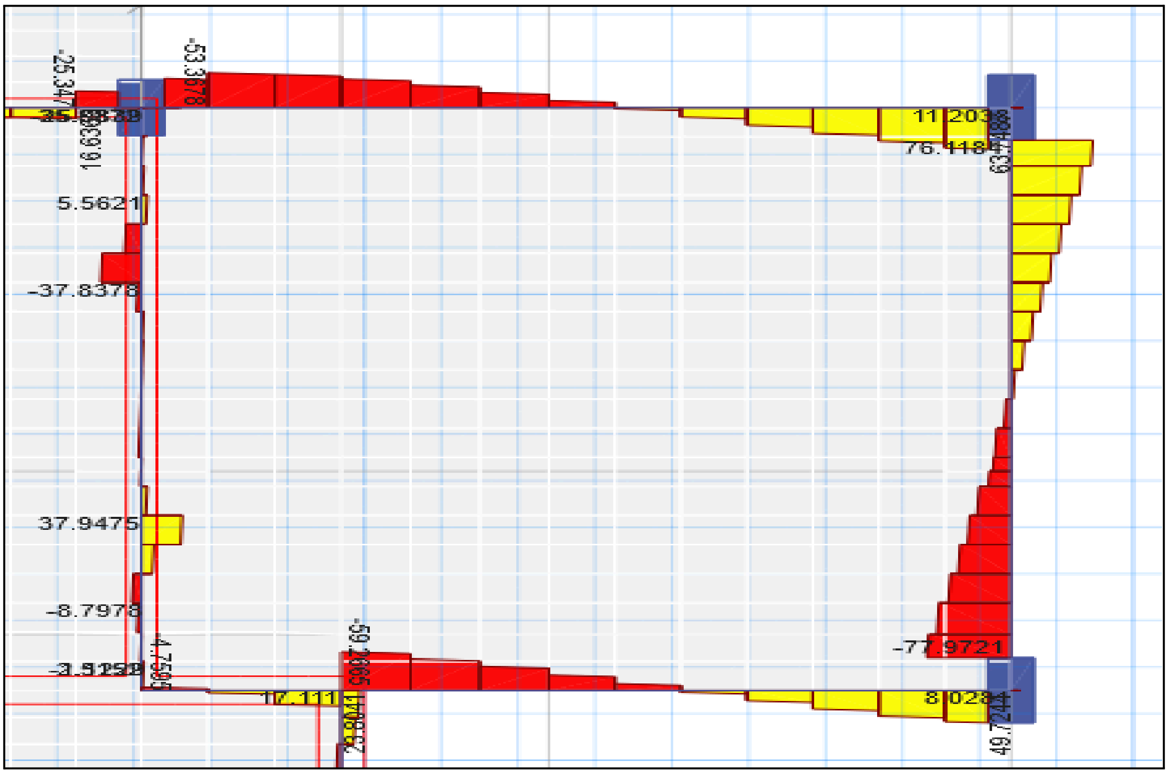


Figure 3.34: Shear forces in column-supported beams

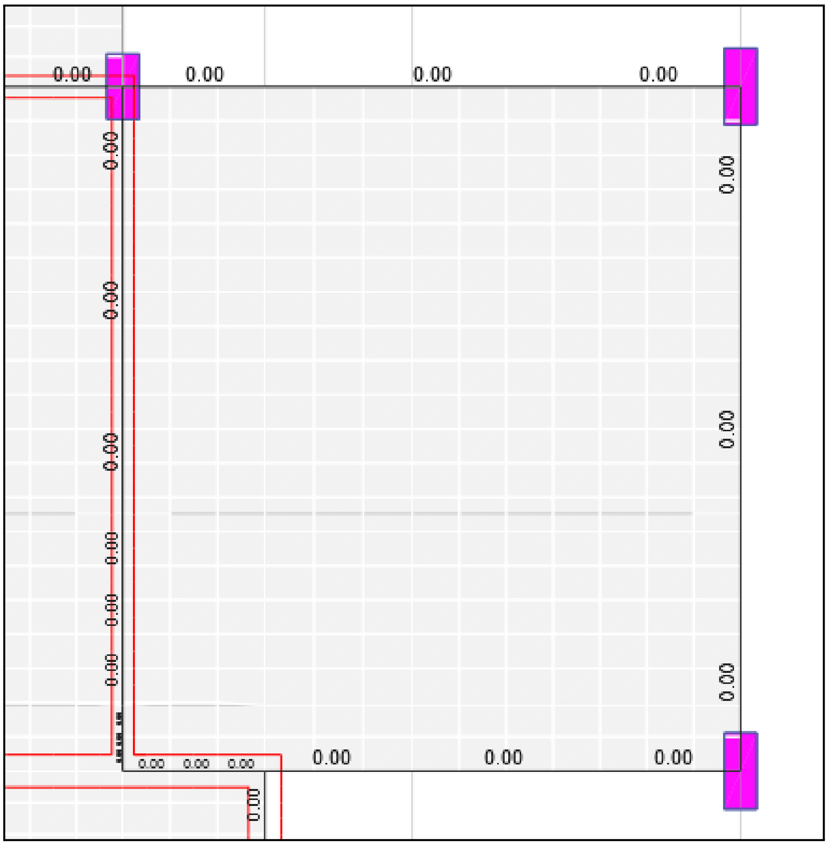


Figure 3.35: Shear reinforcement in critical beams supported by columns

3.6.1.3 Torsion design

Members subjected to a factored torsion Tu greater than the threshold torsion ϕT_{th} are considered as requiring torsional design. The value of Tu is obtained from the ETABS analysis, while ϕT_{th} is calculated according to provisions of ACI 318-19. **Figure 3.36** shows Table 22.7.4.1(a) from the code, which defines threshold torsion expressions for solid cross-sections and various member types.

For non-prestressed beams with a **solid cross section** (applicable in this case), the threshold torsion is calculated using Equation (a) in the table:

$$T_{th} = \frac{1}{12}\lambda\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right)$$

Substituting the values:

$$T_{th} = 0.083 \times 1 \times \sqrt{28} \times \left(\frac{(500 \times 300)^2}{2(500 + 300)}\right) = 6.176\,\text{kN}$$

Where:

- Width of section core, b_w = 300 mm.
- Height of the section core, h = 500 mm.
- $A_{cp} = b_w * h$.
- $P_{cp} = (2)(b_w + h)$.

Applying the strength reduction factor ϕ=0.75:

$$\phi T_{th} = 0.75 \times 6.176 = 4.632\,\text{kN}$$

Comparison with ETABS Output:

- **Beams supported by bearing walls**:

  Maximum torsion: Tu_1=3.7 kN (From **Figure 3.37**)

  Since $ØT_{th} > T_{u1}$, **torsional design is not required ✓**

- **Beams supported by columns**:

Maximum torsion: T_{u2}=12.67 kN.m (From **Figures 3.38** and **3.39**)

Since $ØT_{th} < T_{u2}$, **torsional design is required**

Table 22.7.4.1(a)—Threshold torsion for solid cross sections

Type of member	T_{th}	
Nonprestressed member	$0.083\,\lambda\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)$	(a)
Prestressed member	$0.083\,\lambda\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)\sqrt{1+\frac{f_{pc}}{0.33\lambda\sqrt{f_c'}}}$	(b)
Nonprestressed member subjected to axial force	$0.083\,\lambda\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)\sqrt{1+\frac{N_u}{0.33A_g\lambda\sqrt{f_c'}}}$	(c)

Figure 3.36 : Threshold torsion for solid cross sections

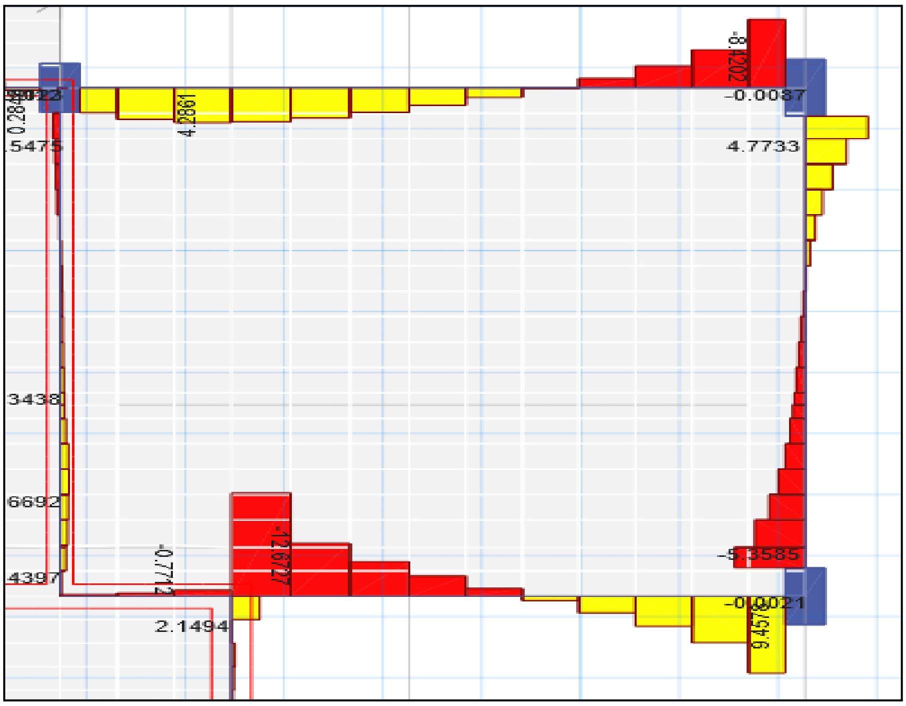


Figure 3.37 : Torsion values in column-supported beams

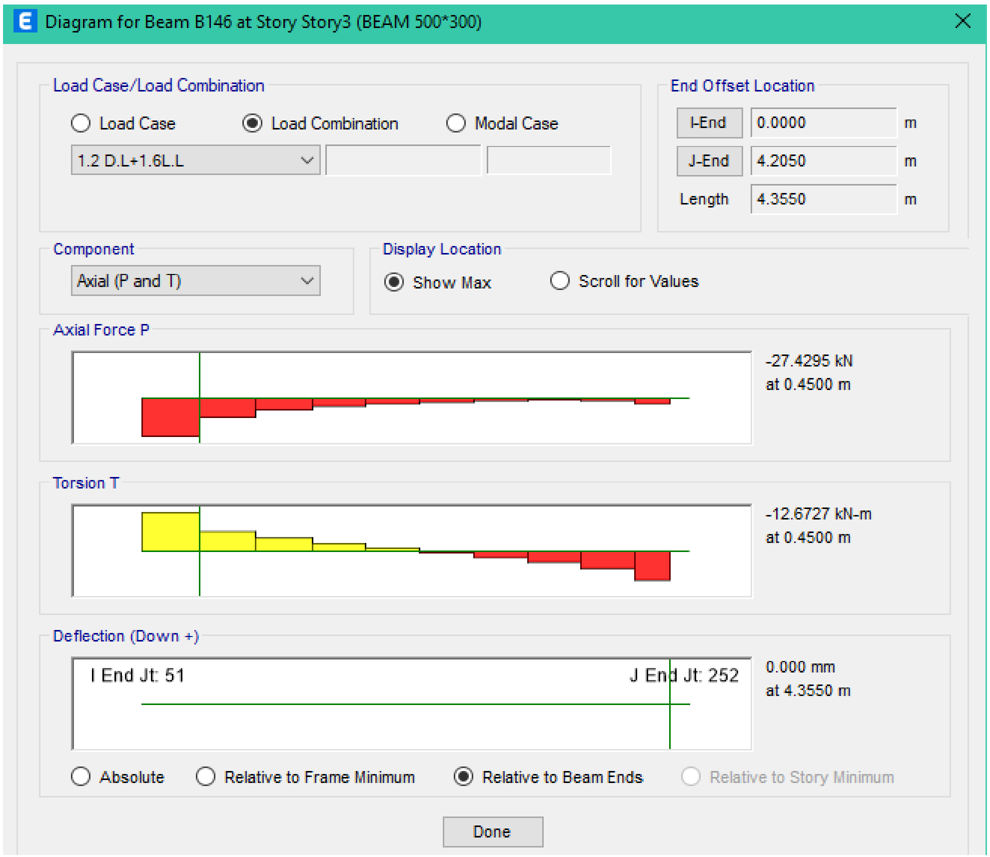


Figure 3.38 : Maximum torsion value for column-supported beams

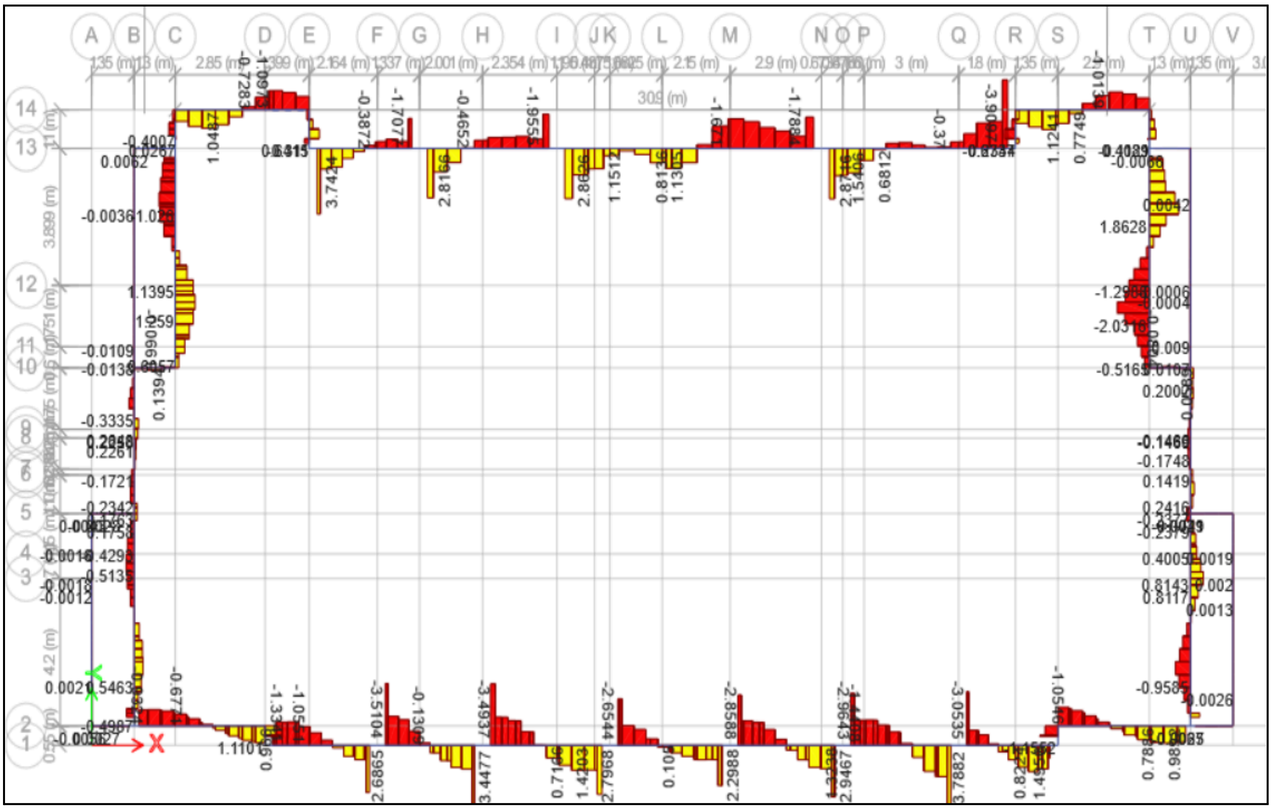


Figure 3.39: Torsion values in beams

In order to obtain meaningful results for comparison, it is essential to understand how **ETABS handles torsional design**. In the latest software versions (as used in this project), ETABS designs members for torsion using the larger of two values: the factored torsional demand Tu, and the factored cracking torsion ϕT_{cr}. This approach reflects code provisions that permit torsional reinforcement to be based on ϕT_{cr} in cases where the torsion is classified as **compatibility torsion**—that is, torsion not essential for equilibrium but induced by deformation compatibility. In such cases, although members may crack, **they remain structurally safe**, as failure is not expected to develop.

Figure 3.40 presents a screenshot from ETABS showing the torsional design of a sample beam, where the software selected ϕT_{cr} rather than Tu as the governing value. A sample calculation for the required torsional reinforcement in this beam is provided below, preceded by a check for adequacy of dimensions.

Torsion Force and Torsion Reinforcement for Torsion, T_u

T_u kN-m	$T_{u,Design}$ kN-m	ϕT_{th} kN-m	ϕT_{cr} kN-m	Rebar A_t /s mm²/m	Rebar A_l mm²
0.2863	19.9656	4.9914	19.9656	451.1	561

Figure 3.40: Torsion design value in a wall-supported beam along C grid-line

Adequacy of dimensions:

The following limiting expression, adapted from *Arman (2025b)*, is used to assess the adequacy of the existing dimensions:

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u P_h}{1.7A_{oh}^2}\right)^2} \le \phi\frac{5}{6}\sqrt{f_c'} \quad \text{(Arman, 2025b, Eq. 9.6)}$$

1.35 mpa < 3.31 mpa → adequate dimensions ✓

Where:

- V_u = 59.2 kN (maximum shear in this beam, value is shown in **Figure 3.34**).
- T_u = 12.67 kN.m.
- b_w = 300 mm.
- d = 440 mm.

- Width of section core, x_1 = 300-(40+ 12/2)(2) = 300-92 = 208mm.
- Height of the section core, y_1 = 500-92 = 408mm.
- $A_{oh} = x_1 * y_1$ = 208*408 = 84864 mm².
- $P_h = (2)(x1+ y_1)$ = (2)(208+408) = 1232mm.

Longitudinal reinforcement:

The design of torsional reinforcement follows the **thin-walled tube model** (also known as the **plastic space truss model**) and is based on the equations provided by the code. For the sample checked, the design torsion is T_{design}= 19.96 (see **Figure 3.40** presented earlier).

The required longitudinal reinforcement for torsion is calculated using the following equation, adapted from the same reference (Arman, 2025b):

$$A_l = \left(\frac{T_u/\phi}{2A_o f_y}\right) P_h = \left(\frac{A_t}{s}\right) P_h \left(\frac{f_{yt}}{f_y}\right) \quad \text{(Arman, 2025b, Eq. 9.11)}$$

Where:

- $T_n = T_u/\varnothing$ = 26.6 kN.m
- $A_o = 0.85*A_{oh}$ = 72134 mm²
- P_h = 1232mm

Thus, the calculated longitudinal torsional reinforcement is:

$$A_l\text{=541mm}^2$$

This closely matches the value reported by ETABS:

$A_{l,ETABS}$=561 mm² (as shown in **Figure 3.40**)

The percent difference between the manual calculation and the software result is:

$$\text{Percent difference}= \frac{561 - 541}{561} \text{ x } 100 \approx 3\ .57\%$$

This minor variation is considered acceptable and can be attributed to the method used, including potential simplifications in manual calculations versus the more refined procedures applied internally by the software.

Stirrups reinforcement:

Stirrups reinforcement for torsion can be calculated based on the following equation:

$$\frac{A_t}{s} = \frac{T_n}{2A_o f_{yt}} \quad \text{(Arman, 2025b, Eq. 9.8)}$$

Where:

$T_n = T_u/\varnothing = 26.6$ kN.m

$A_o = 0.85*A_{oh} = 72134$ mm²

The calculated distributed transverse torsional reinforcement is:

$A_t/S = 0.4389$ mm²/mm

$= 439$ mm²/m

This closely matches the value reported by ETABS:

$A_t/S = 451.1$ mm² (as shown in **Figure 3.40**)

The percent difference between the manual calculation and the software result is:

$$\text{Percent difference} = \frac{451 - 439}{451} \text{ x } 100 \approx 2.661\%$$

This minor discrepancy is considered acceptable and can be attributed to the specific assumptions or simplifications used in manual versus software calculations. Consequently, the torsional design results provided by ETABS are deemed accurate and may be reliably adopted for all similar beams throughout the structure.

3.6.2 Columns

Column N11 in the ground floor is subjected to 3200 kN ultimate axial load and 151 kN.m ultimate bending moment, this gives the following values:

$\frac{\Phi Pn}{bh} = \frac{3200*1000}{300*1200} = 8.888\ MPa$= 1.3 ksi.

$\frac{\Phi Mn}{bh^2} = \frac{151*1000*1000}{300*1200*1200}$=0.34 mpa = 0.049 ksi

And for 40mm clear cover, 10mm diameter of tie, and 10mm half diameter of longitudinal bar:

$\gamma = \frac{1200-120}{1200} = 0.9$

And for $f'c$= 28 MPa concrete and referring to the diagram in **Figure 3.45**, $\rho < 1\%$, thus, $\rho = 1\%$ is considered. **Figures 3.41** and **3.42** show ρ and area of steel results from ETABS, respectively. Area of steel = 0.01*1200*300 = 3600 mm^2 ✔.

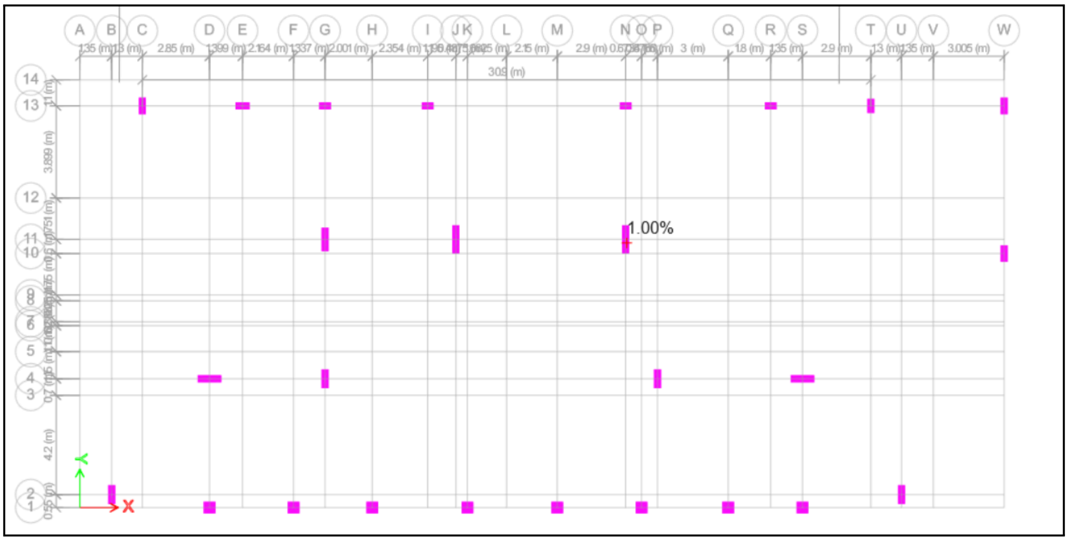


Figure 3.41: Total rebar percentage for column N11 ground floor

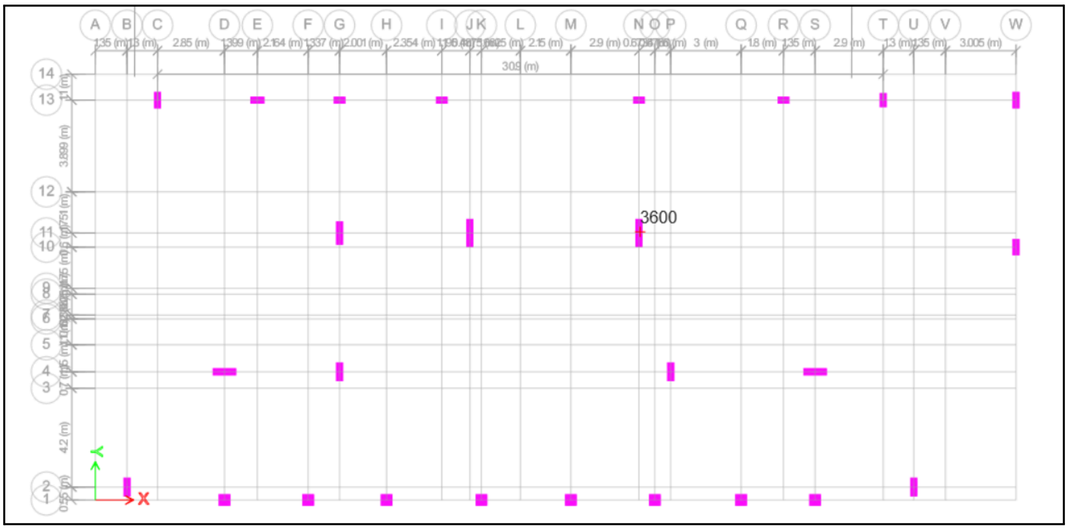


Figure 3.42: Total longitude bars (area) for column N11 ground floor

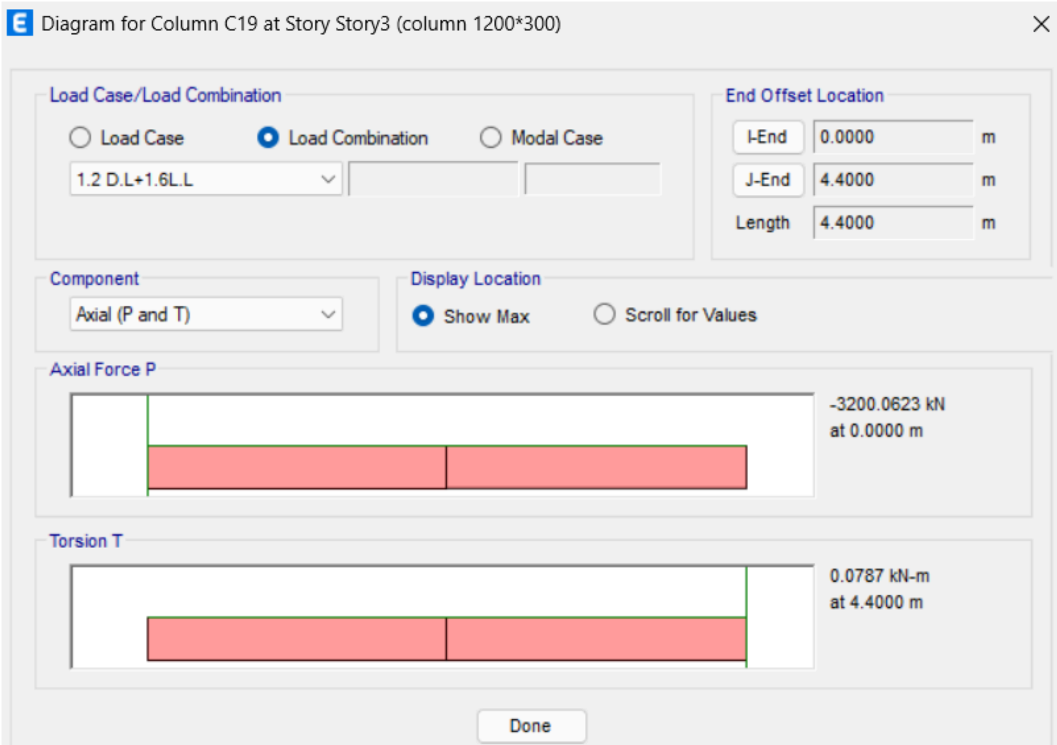


Figure 3.43: Axial load for column N11 ground floor

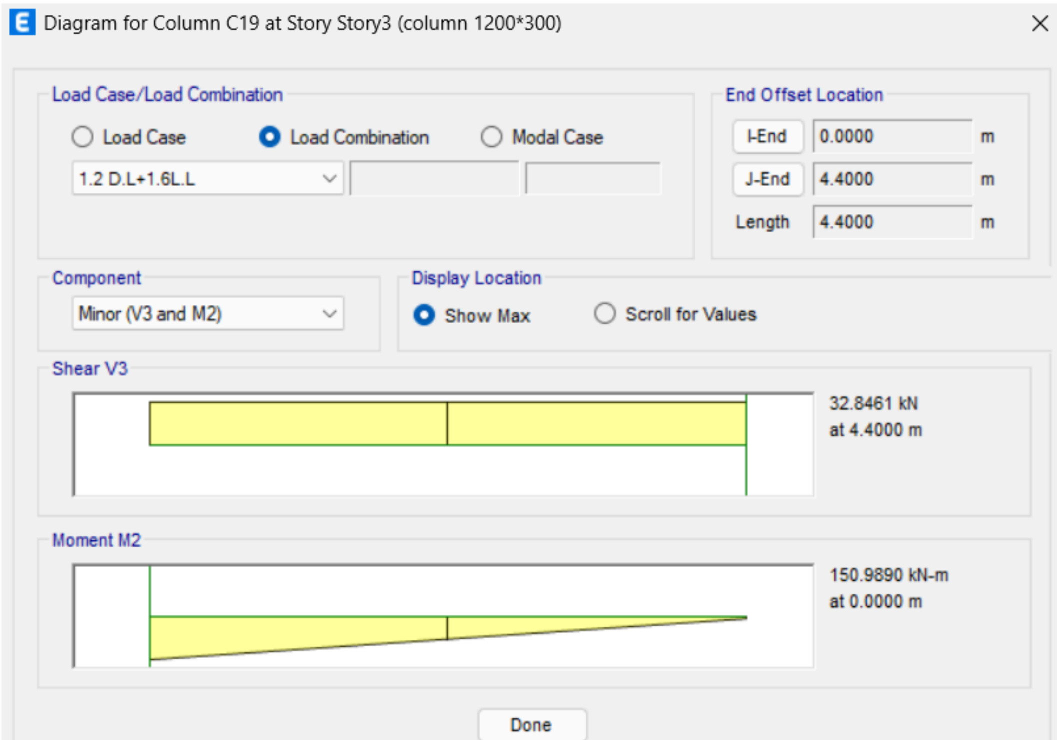


Figure 3.44: M22 for column N11 ground floor

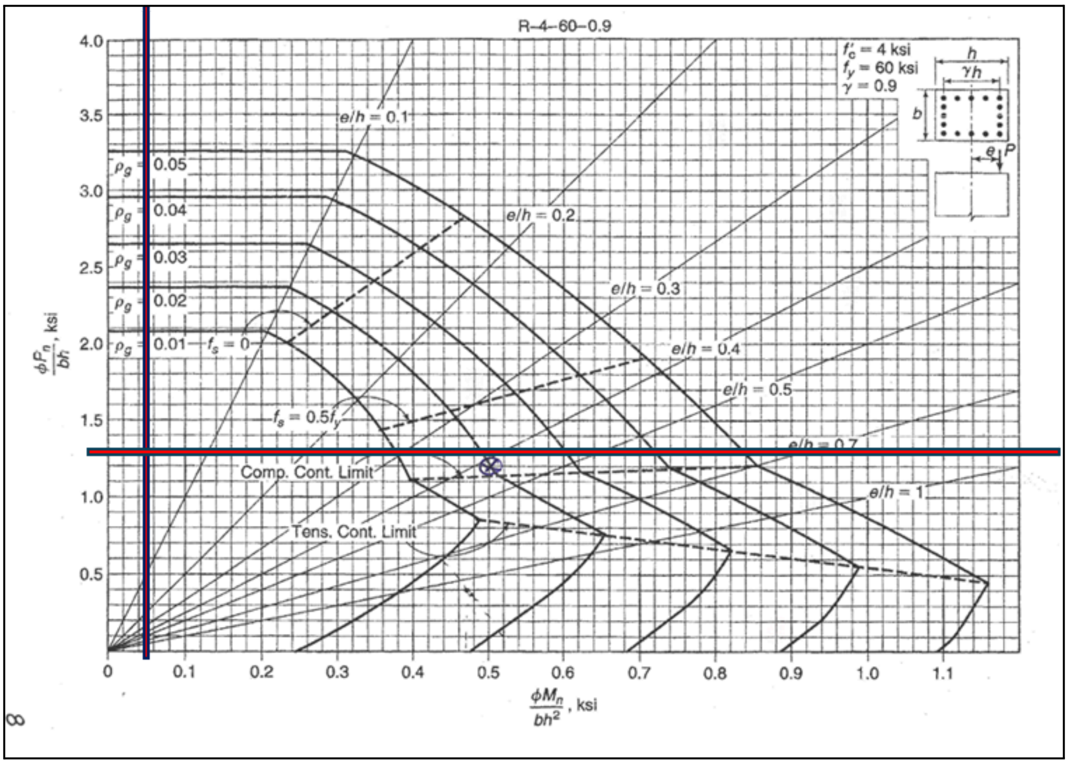


Figure 3.45: Moment-Axial force interaction diagram for γ= 0.9

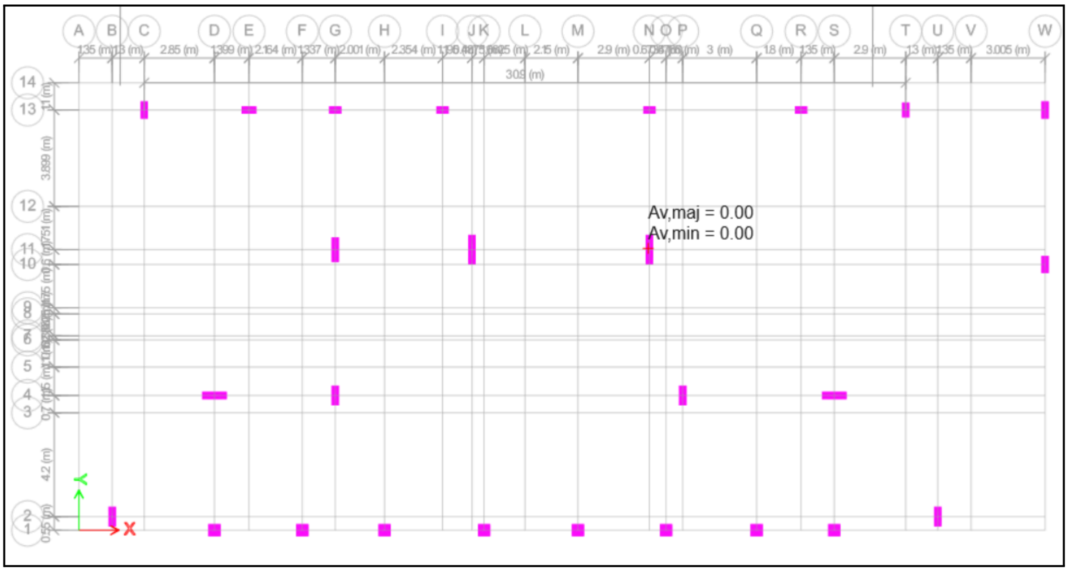


Figure 3.46: Shear reinforcement for column N11 ground floor

3.6.3 Footings

A flexural design check for a single footing and staircase footing is conducted below to ensure that the provided steel reinforcement is sufficient and meets the required safety and performance standards for the structural elements. **Figure 3.47** shows the software results for flexural reinforcement of footings.

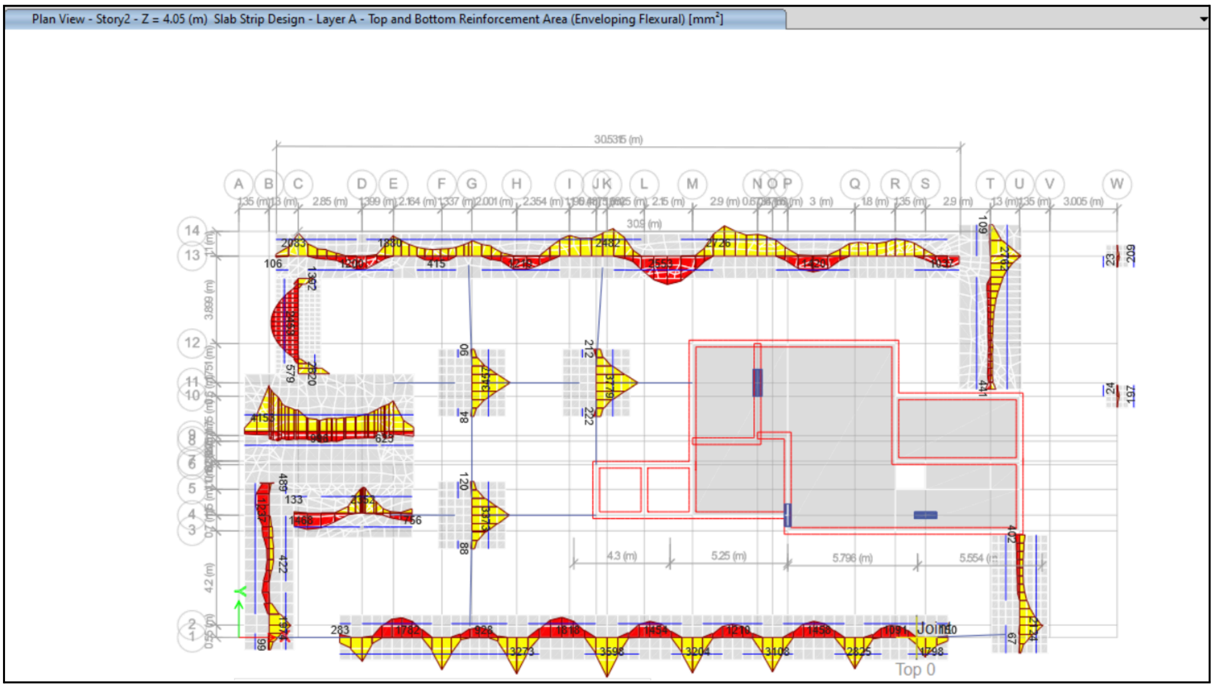


Figure 3.47: Flexure design from ETABS

Footing F1:

Mu = 320 KN.m

B = 3000 mm, h = 850 mm, d= 770 mm.

⇒ ρ = 0.0018 ⇒ As =0.0018*3000*770= 4158 mm^2

As min = 0.0018*b*h = 0.0018*3000*850 = 4590 mm^2 , use As = 4590 mm^2

As /m = 4590/330 = 1530 mm^2/m

%Difference = ((ETABS reading - Manual calculations)/Manual calculations)*100%

%Difference = (1530 - 1530)/1530 *100% = 0% < 25% acceptable.

- Wall footing:

-Upper wall.

Mu = 248 KN.m.

B = 1000 mm, d = 520 mm, h = 600mm.

⇒ ρ = 0.00247 ⇒ As = 0.00247*1000*520 = 1284..4 mm^2.

As min = 0.0018*b*h = 0.0018*1000*600=1080 mm^2 ⇒ use As =1284.4 mm^2.

%Difference = ((Etabs reading - Manual calculations)/Manual calculations)*100%.

%Difference = (1284.5-1284.4) / 1284.4*100% = 0% < 25% acceptable.

Staircase footing:

Mu = 240 KN.m.

B = 1000 mm, d = 620 mm, h = 700 mm.

⇒ ρ = 0.0018 ⇒ As = 0.0018*1000*620 = 1116 mm^2.

As min = 0.0018*b*h = 0.0018*1000*700=1260 mm^2 ⇒ use As =1260 mm^2.

%Difference = ((Etabs reading - Manual calculations)/Manual calculations)*100%.

%Difference = (1260-1260) / 1260*100% = 0% < 25% acceptable.

3.7 Design of slabs

3.7.1 Design slab for shear

Taking the slab thickness from ground floor to the fifth floor as 220mm and the total clear cover 40 mm, the slab design for shear (for a one meter strip) is as shown:

- Slab 220mm:

  h =220mm, d =180mm, b =1000mm, ρ= 0.002.

  ΦVc (without reinforcement) = $\Phi*\lambda*\lambda s*0.66*\rho_w^{1/3}*\sqrt{f`c}*b_w*d$ =

  $0.75*1*1*0.66*0.002^{1/3}*\sqrt{28}*1000*180/1000$= 59.4 kN

  λs = 1 ⇒ for d ≤ 250mm.

  V_u from ETABS =140kN/m.

  ∅Vc < Vu → Shear reinforcement is required.

  $\Phi V_s = \Phi V_u - \Phi V_c$ = 140-60 = 80 KN/m.

  Av/S = ΦVs/fy *d*Φ =80*1000/(420*0.75*180) =1.4mm^2/mm.

  ⇒ Smax = d/2 = 90mm.

  (Av/S)min = max of (0.062*(f′c)1/2 *bw/fy, 0.35*bw/fy) =0.834mm^2/mm.

  (Av/S)min < Av/S ⇒ it's ok.

  As=number of legs per meter *Ab

As=2*113=226mm^2

Use Φ12 stirrups ⇒ S =226/1.4=161mm >S max.

⇒ use S =75mm

The slab is reinforced for shear with 2 leg stirrup Φ12 /75mm

- Slab 280mm:

h =280mm   d =240mm   b =1000mm   assume ρ =0.002.

ΦV_c (without reinforcement) = $\Phi*\lambda*\lambda s*0.66*\rho_w^{1/3}*\sqrt{f`c}*b_w*d$ =

$0.75*1*1*0.66*0.002^{1/3}*\sqrt{28}*1000*240/1000=79.2$ kN

λs = 1 ⇒ for d ≤ 250mm

∅Vc < Vu → Shear reinforcement is required.

$\Phi V_s = \Phi V_u - \Phi V_c$ = 140-80 = 60 KN/m

Av/S = ΦVs/fy *d*Φ =60*1000/(420*0.75*240) =0.79mm^2/mm

⇒ Smax = d/2 = 120mm

(Av/S)min = max of (0.062*(f′c)1/2 *bw/fy, 0.35*bw/fy) =0.834mm^2/mm

(Av/S)min >Av/S ⇒ use (Av/S)min

Use Φ12stirrups ⇒ S =226/0.834=135mm > S max⇒ use S =75mm

The slab is reinforced for shear with 2 leg stirrup Φ12/75mm

Table 3.13: Value of Vu (Maximum shear)

Story	V13 (KN/m^2)	V23 (KN/m^2)	ΦVc (KN/m^2)
Ground	ok	ok	60
St3	140	135	60
St4	140	135	60
St5	140	135	60
St6	140	135	60
St7	140	135	60
St8	140	135	80
St9	ok	ok	60

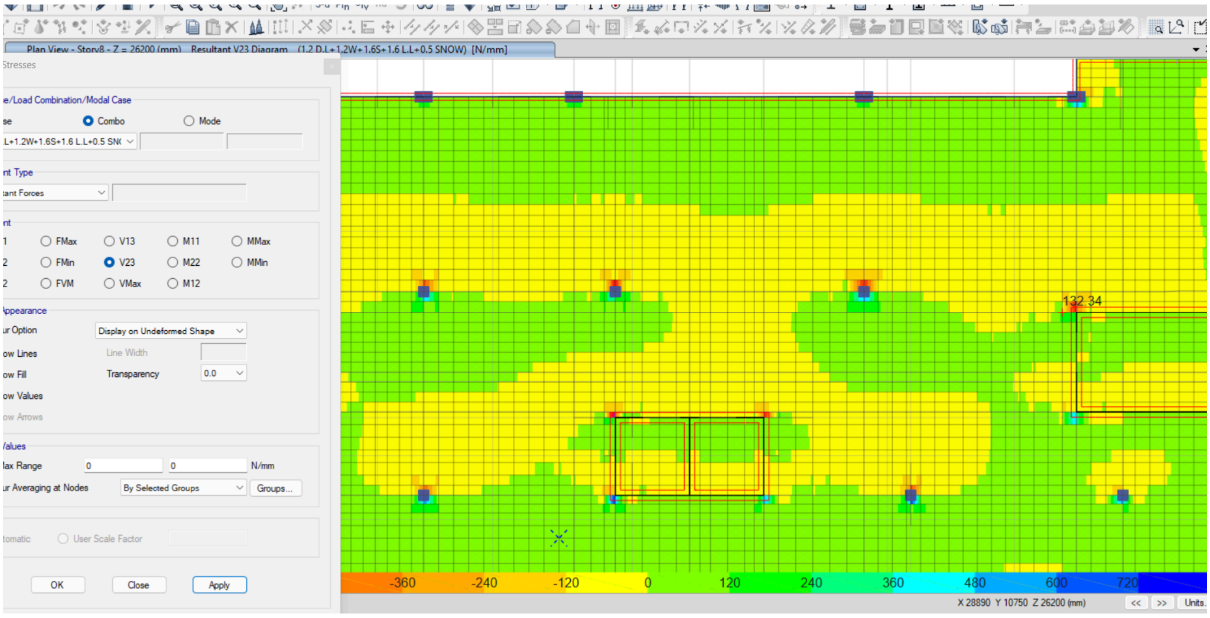


Figure 3.48: Max value V23 in story8

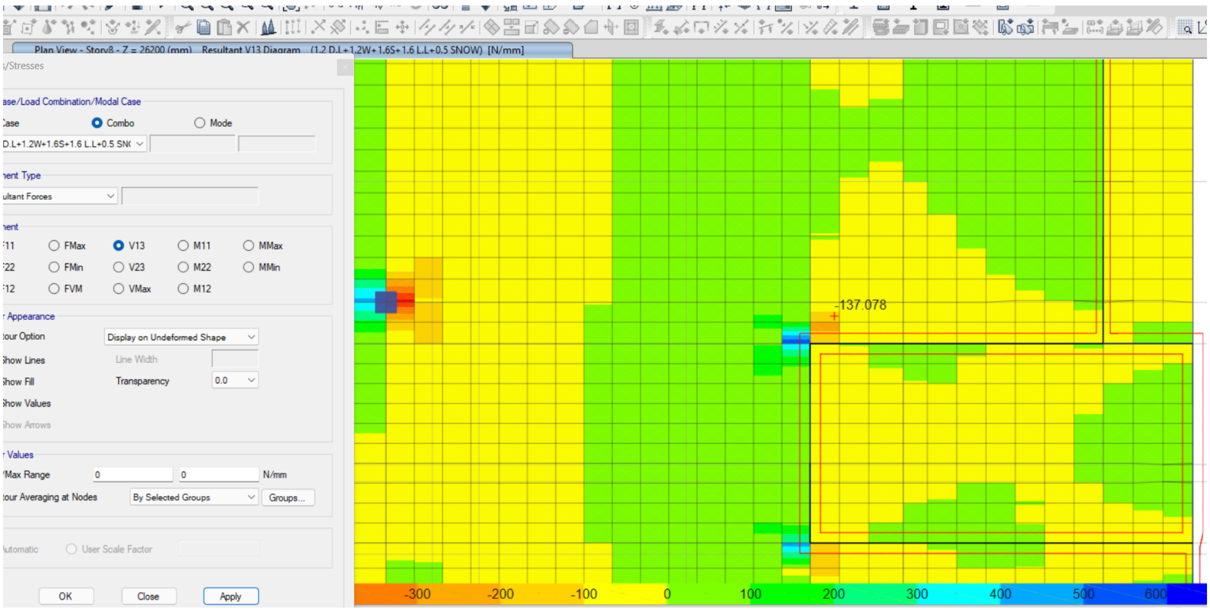


Figure 3.49: Max value V13 in story8

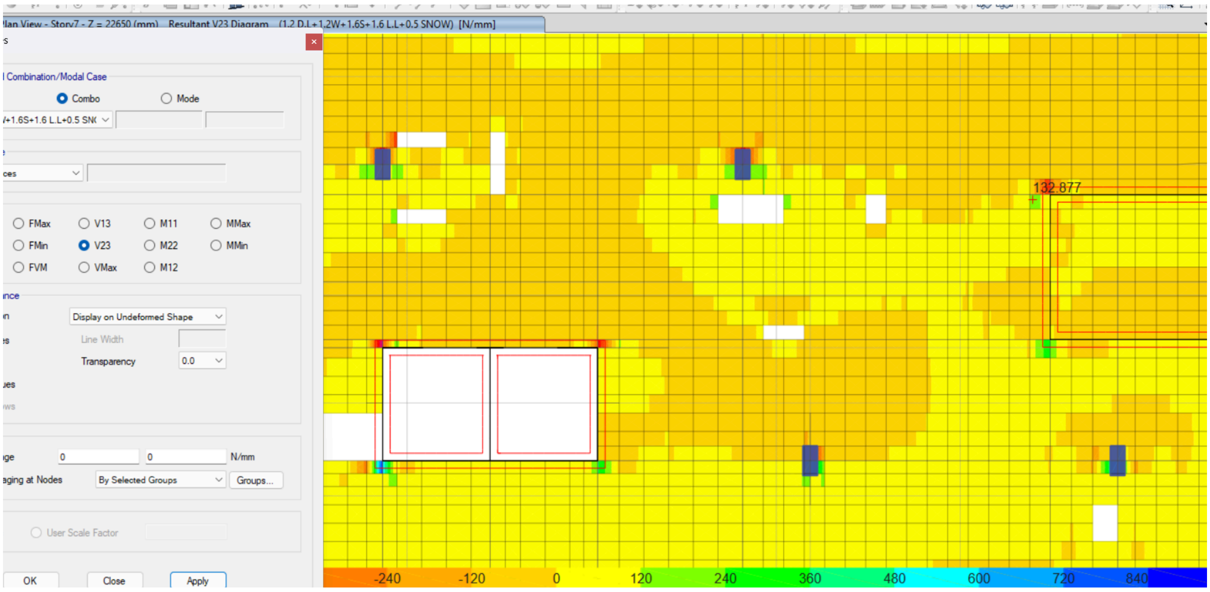


Figure 3.50: Max value V23 in other stories

Figure 3.51: Max value V13 in other stories

Shear reinforcing has been taken as shown in **Table 3.14,** the shear reinforcing area is shown in **Figure 3.52.**

Table 3.14: Shear reinforcement

Stirrups spacing (mm)	Bar diameter Φ
75	12
75	12

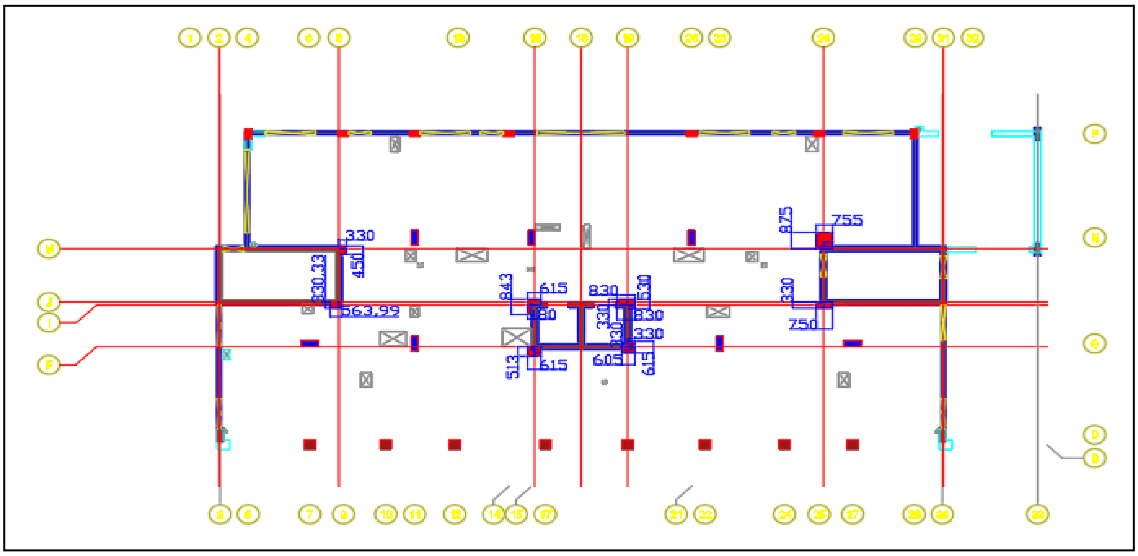


Figure 3.52: Shear reinforcement areas

3.7.2 Design slab for flexure

Table 3.15 presents the slab reinforcement details, including specifications for both the bottom and top rebar. This ensures that the slab is adequately reinforced to handle the applied loads and meet structural requirements. **Figures 3.53-3.56** shows that the slab reinforcement, both top and bottom, meets the minimum steel requirements.

Table 3.15: Slab reinforcement

Story	A_{st} min(mm^2/m)	Bottom	Top
Ground	396	Ø12 /200 mm	Ø14 /200 mm
St3	396	Ø12 /200 mm	Ø14 /200 mm
St4	396	Ø12 /200 mm	Ø14 /200 mm
St5	396	Ø12 /200 mm	Ø14 /200 mm
St6	396	Ø12 /200 mm	Ø14 /200 mm
St7	396	Ø12 /200 mm	Ø14 /200 mm
St8	504	Ø12 /200 mm	Ø14 /200 mm
St9	396	Ø12 /200 mm	Ø14 /200 mm

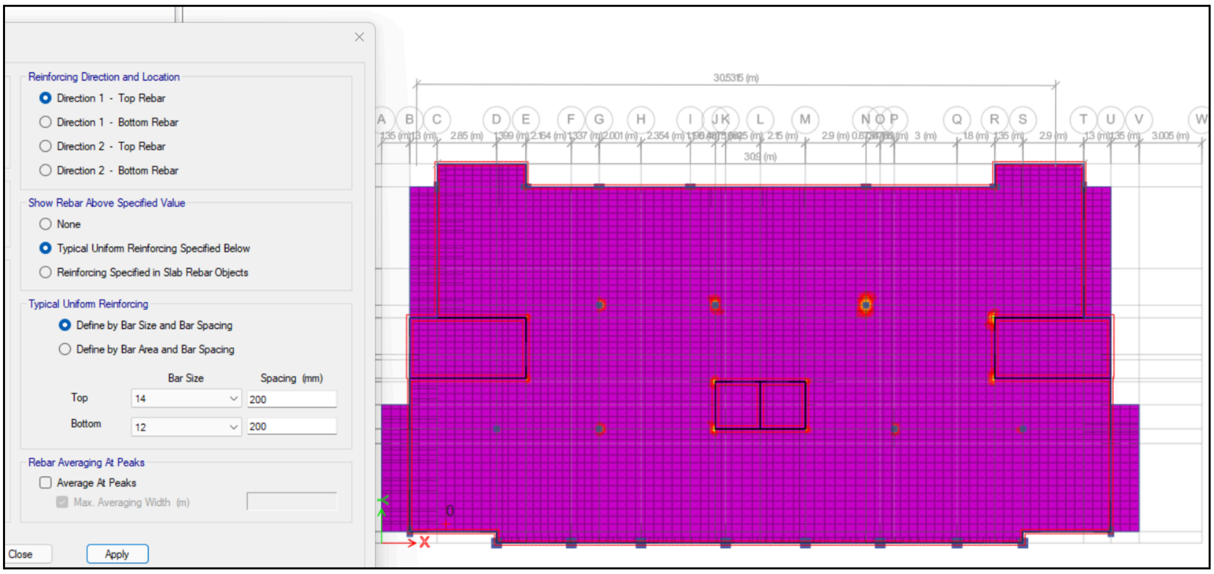


Figure 3.53: Direction 1 Top reinforcement

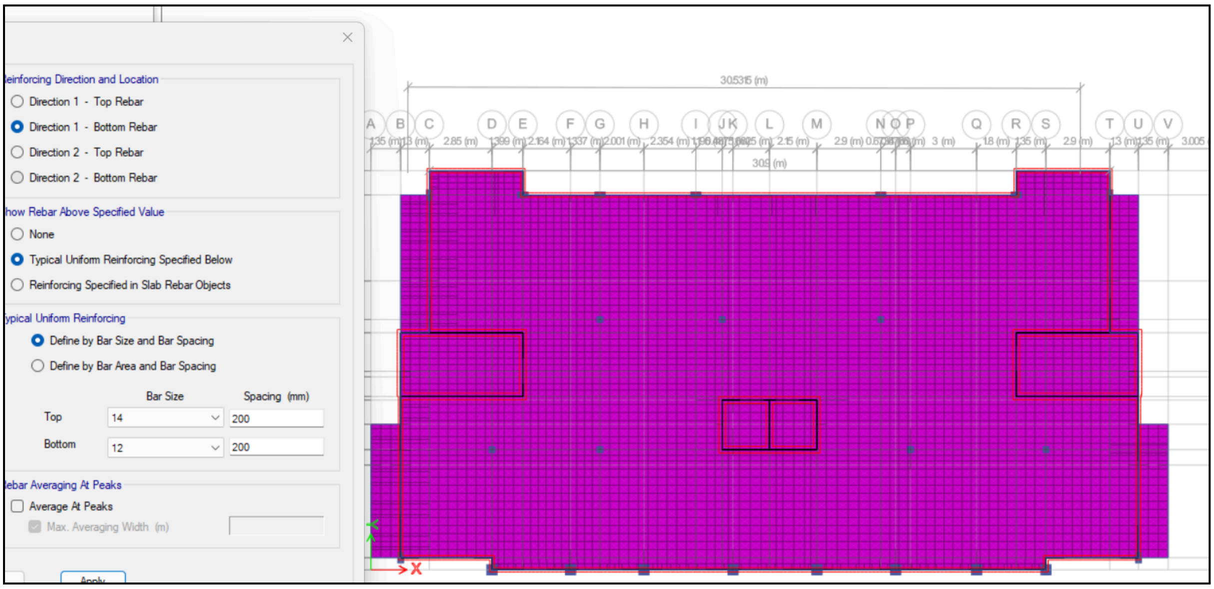


Figure 3.54: Direction 1 Bottom reinforcement

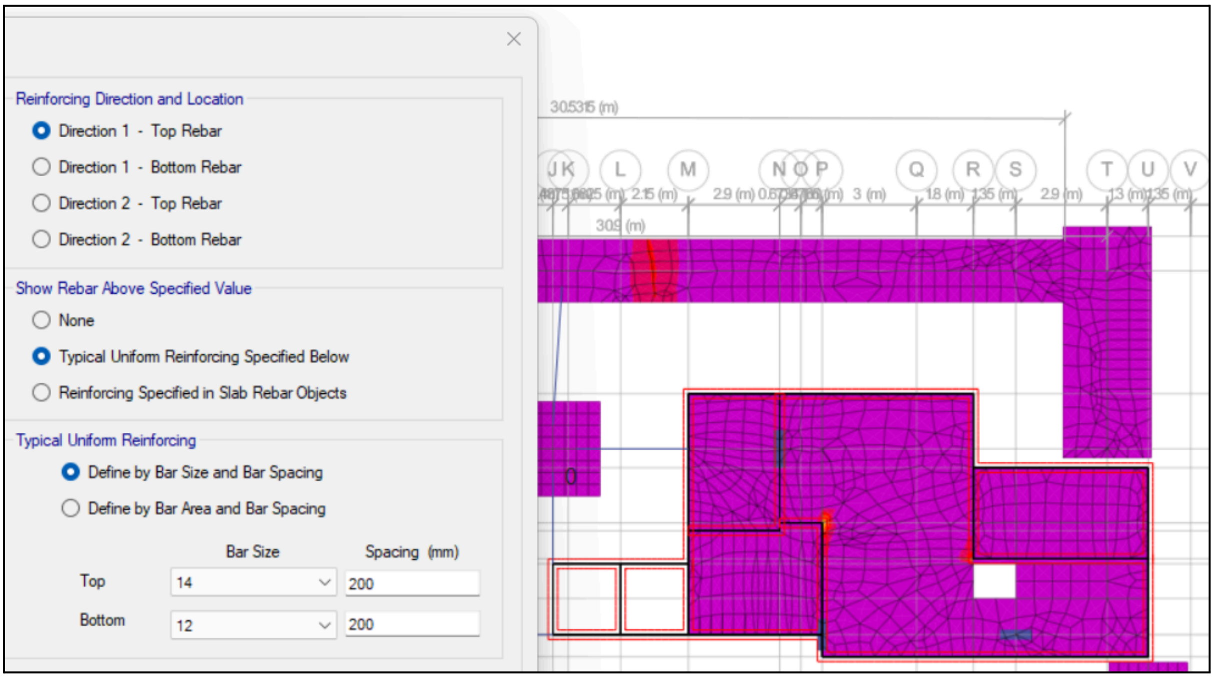


Figure 3.55: Direction 1 Top reinforcement

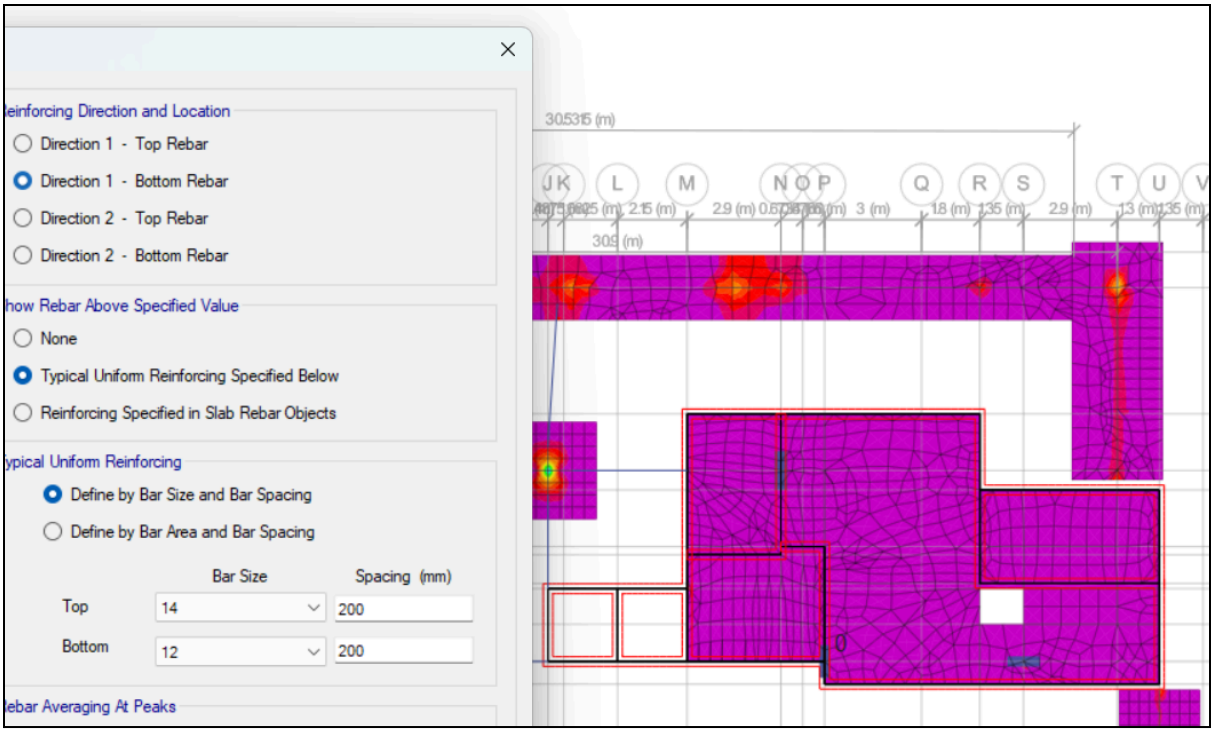


Figure 3.56: Direction 1 Bottom reinforcement

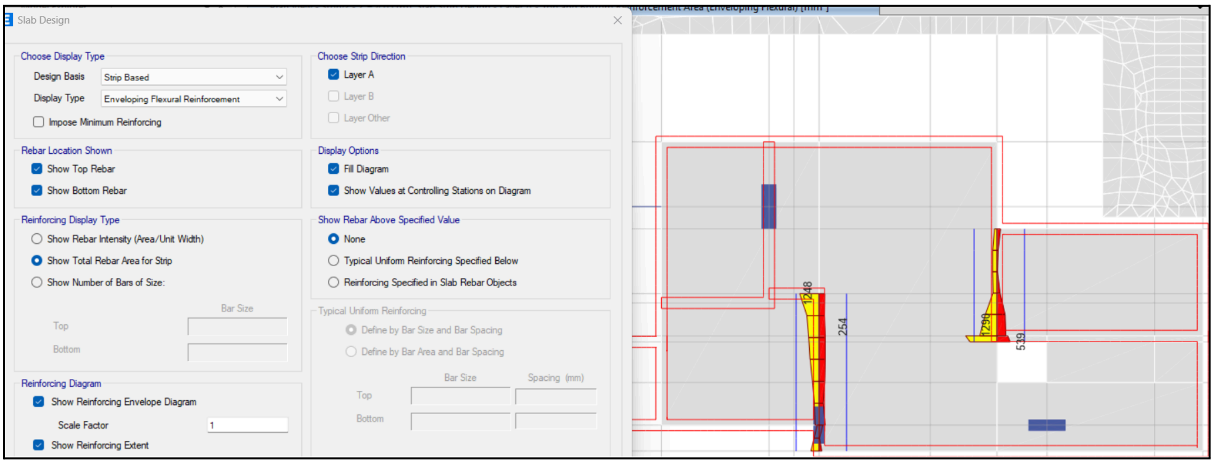


Figure 3.57: Extra moment reinforcement-1

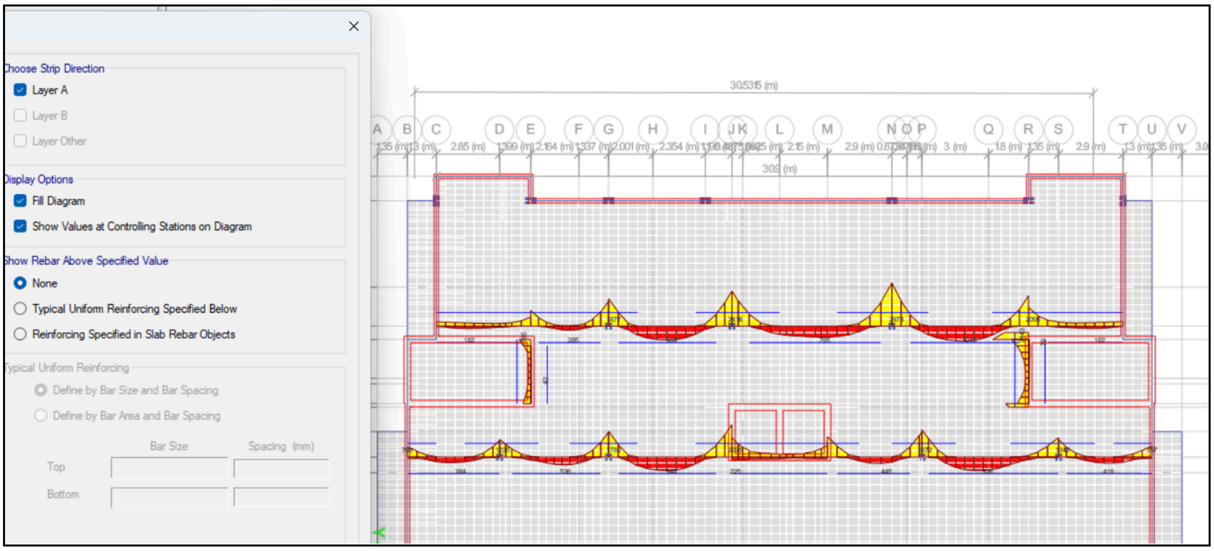


Figure 3.58: Extra moment reinforcement-2

Ast $_{Max}$ =3141 mm^2/2m =1570.5 mm^2/m

Bottom= Ø12 /200 mm

Top= Ø20/200 mm

3.7.3 Specifications for bars and stirrups distribution

1. According to ACI Code 20.6.1, for cast-in-place concrete, concrete protection at surfaces not exposed directly to the ground or weather should be not less than (3 / 4) in. (about 2 cm) for slabs. 1 in. to the center of the bar is ordinarily sufficient to give the required(3 / 4) in. cover.
2. ACI Code 25.2 specifies that the minimum clear distance between adjacent bars not be less than the nominal diameter of the bars, or 1 in.

3. In no case should the clear spacing of reinforcement be less than (4/3) of the maximum aggregate size.
4. Maximum spacing of legs of shear reinforcement along the length of the member and across the width of the member for non prestressed beam shall be:
   a. Required $V_s \leq 0.33\sqrt{f'c}\, b_w\, d$: The lesser of d or 600mm across the width and the lesser of d/2 and 600mm along the length.
   b. Required $Vs > 0.33\sqrt{f'c}\, b_w\, d$: The lesser of d/2 or 300mm across the width and the lesser of d/4 and 300mm along the length.

Slabs' reinforcement details and structural drawings are included in the **Appendix B**.

3.8 Design of beams

This section expands upon the individual beam design checks presented earlier by addressing the reinforcement requirements for all beams in the structure. It begins with the quantities obtained from software output, then examines how these are translated into practical bar layouts and spacing in accordance with code provisions. The discussion culminates in the finalized reinforcement design, ensuring both compliance and constructability.

A prior check of beam flexural design confirmed that all beams require no more than the minimum longitudinal reinforcement specified by code. In this section, that flexural reinforcement is combined with the longitudinal steel required to resist torsion to establish the final reinforcement demands needed to resist all internal actions.

Although the factored shear demand Vu was found to be less than the design shear strength ϕVc in the beams, minimum shear reinforcement was still required in accordance with ACI 318-19 Section 9.6.3. This requirement applies because the members do not qualify as slab-type sections and do not meet the dimensional or loading exceptions specified in Section 9.6.3.1. Consequently, closed stirrups were provided based on the minimum reinforcement provisions of the code. The resulting minimum shear reinforcement is then combined with the torsional reinforcement demands to determine the total required stirrups.

For simplification, the highest longitudinal and transverse torsional reinforcement demands among all beams were adopted as the design basis. These values were obtained from ETABS tables listing reinforcement quantities for all beams, which were sorted in ascending order to identify the lowest values and in descending order to determine the highest values. **Figure 3.59** illustrates these torsional reinforcement values, highlighting the beams with the lowest and highest longitudinal reinforcement demands, as well as those with the lowest and highest transverse reinforcement demands. The relatively small variation between beams confirms that adopting the highest reinforcement values for all beams is justified and efficient, as significant divergence would otherwise lead to uneconomical overdesign.

Maximum Longitudinal Reinforcement Demand for Torsion (Among All Beams):

Story	Label	DesignSect	TLngRebar mm²
Story3	B85	BEAM 500*300	596

Minimum Longitudinal Reinforcement Demand for Torsion (Among All Beams):

Story	Label	DesignSect	TLngRebar mm²
Story3	B86	BEAM 500*300	420

Maximum Transverse Reinforcement Demand for Torsion (Among All Beams):

Story	Label	DesignSect	TTrnRebar mm²/m
Story3	B85	BEAM 500*300	479.01

Minimum Transverse Reinforcement Demand for Torsion (Among All Beams):

Story	Label	DesignSect	TTrnRebar mm²/m
Story3	B86	BEAM 500*300	329.99

Figure 3.59: Maximum and minimum reinforcement demand for Torsion (Among All Beams)

3.8.1 Longitudinal reinforcement

Flexural reinforcement is placed as required at either the top or bottom face of the beam, depending on the moment demands. The torsional longitudinal reinforcement shown in **Figure 3.59** is distributed equally across the top, bottom, and side faces of the beam cross section, with overlapping areas combined with flexural steel where applicable.

Flexural Reinforcement:

- Steel ratio, $\rho_{min} = 0.0033$
- Required longitudinal steel area, $A_{s,min} = 455$ mm²
  (Uniform for all beams due to identical cross-section dimensions)

Torsional Longitudinal Reinforcement:

- Adopted value for all beams, $A_l = 596$ mm²
- Distributed equally in thirds across top face, bottom face, and middle region
- One-third of torsional steel ≈ 200 mm²

Combined longitudinal reinforcement:

- **Middle region (**torsion only**):**

  200 mm²

  → 2∅12

- **Tension face (**flexure + torsion**):**
  Flexural: **455 mm²**
  Torsional: **200 mm²**
  Total: 655 mm²

  → 3∅18

3.8.2 Transverse reinforcement

Minimum shear reinforcement ratio from code:

$$0.062\sqrt{f'_c}\frac{b_w}{f_{yt}} = 0.23$$

$$0.35\frac{b_w}{f_{yt}} = 0.25$$

Minimum shear reinforcement area:

$$\mathbf{(A_v /S)_{min} = 0.25\ mm^2/mm = 250\ mm^2/m}$$

Torsional transverse reinforcement:

A_t /S = 479 mm²/m (From **Figure 3.59**)

Combined transverse reinforcement (per *Arman, 2025b*):

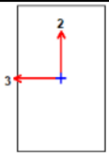
$$\frac{A_{v+t}}{s} = \frac{A_v}{s} + 2\frac{A_t}{s} \quad \text{for 2 legs closed stirrup} \quad (\text{Arman, 2025b, Eq. 9.9a})$$

250 + 2 x (479) = 1208 mm²/m

→ ∅12/90 mm

3.8.3 Samples for beams forces and design results from the software

Figure 3.60 shows a sample for design of a beam taken from ETABS.



Beam Element Details

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	LLRF	Type
Story5	B36	559	BEAM 500*300	1.2D+1.6L+0.5Lr	2340	3000	1	Sway Ordinary

Section Properties

b (mm)	h (mm)	b_f (mm)	d_s (mm)	d_{ct} (mm)	d_{cb} (mm)
300	500	300	0	60	60

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
24870	28	1	400	400

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)	0	-36.1513	0	42	0	42
Bottom (-2 Axis)	1.143	-36.1513	0	59	78	78

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
0.5298	86.9969	0	0	0

Figure 3.60: Data sheet for beam

3.8.4 Specifications for bars and stirrups distribution—Code provisions

3.8.4.1 Durability—Concrete cover

Table 20.5.1.3.1— in ACI 318-19 sets the specified concrete cover for cast-in-place non prestressed concrete members. For beams, a minimum cover of 40 mm is required, this limit was achieved in the early design phase for beams.

3.8.4.2 Bar Spacing

The code, in accordance with Section 25.2, specifies a minimum clear spacing between longitudinal bars along the beam length to ensure practical constructability and adequate concrete placement. According to the requirements, the clear spacing between two parallel adjacent bars in a layer must not be less than the largest of the following three values:

- 25 mm
- The diameter of the bar (d_b)

- (4/3) times the nominal maximum size of coarse aggregate (dagg) used in the concrete mix.

In all beam designs, the longitudinal bar spacing was carefully ensured to be sufficiently large and, in all cases, to exceed the minimum limits specified by the code.

3.8.4.3 Development Lengths and Splicing

Development lengths are governed by Section 25.4 of ACI 318-19, while splicing requirements fall under Section 25.5. The general arrangement of longitudinal reinforcement—including bar positioning, splicing locations, distribution, and extensions—is subject to detailing rules outlined in the code. These requirements are effectively summarized in schematic form rather than through lengthy textual descriptions, as done specifically in this report to enhance clarity.

Accordingly, the drawings shown in **Figures 61-Y** are adapted from an unpublished instructional document developed by M.Sc. Arman, which visually interprets the relevant ACI 318-19 provisions specifically for longitudinal reinforcement in beam detailing. These figures provide a practical and visual summary of code-based rules for longitudinal bar layout in beams, and they were used as a reference in the development of the structural drawings for this project. Where L_{dt} , L_{dh}, and L_s are the development length, hook length, and splicing length, respectively.

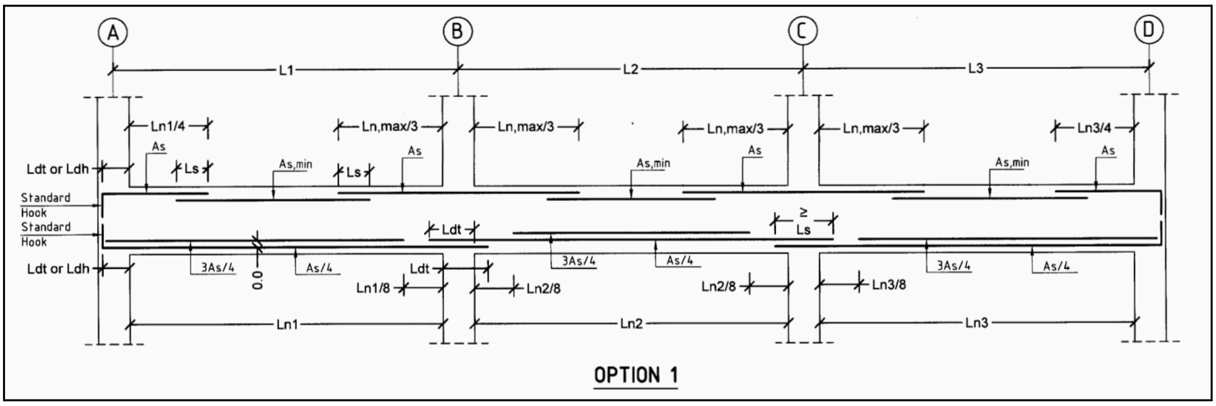


Figure 3.61: Schematic of ACI 318-19 beam detailing requirements, Option 1

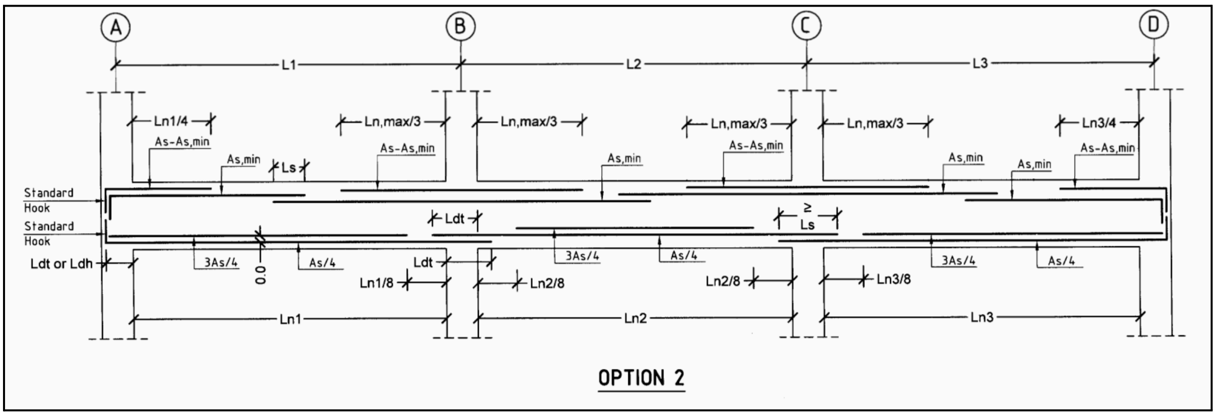


Figure 3.62: Schematic of ACI 318-19 beam detailing requirements, Option 2

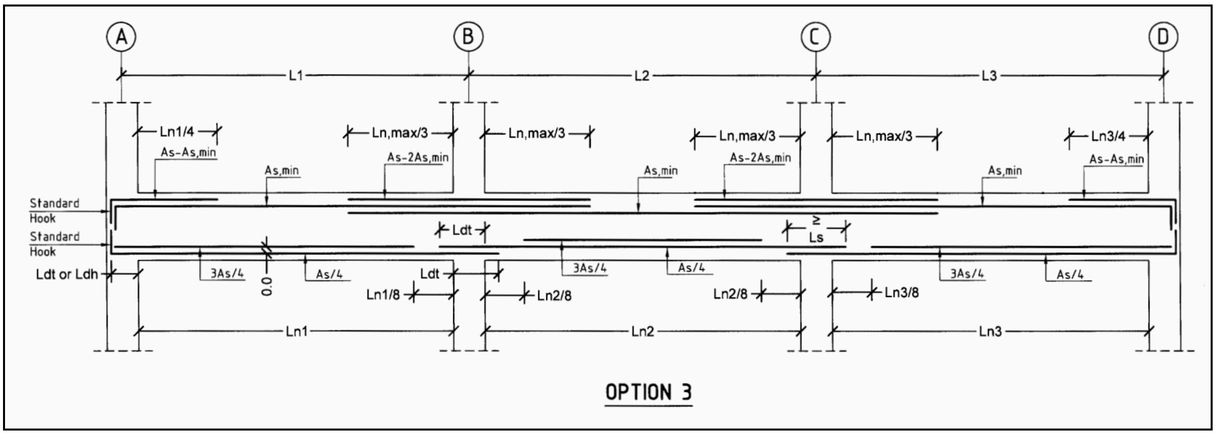


Figure 3.63: Schematic of ACI 318-19 beam detailing requirements, Option 3

3.9 Design of columns:

3.9.1 Design of all columns

Table 3.16 illustrates the column details across the project with the dimension, location, and reinforcing details.

Table 3.16: Column details

Columns	Column 1	Column 2	Column 3	Column 4	Column 5	Column 6	Column 7	Column 8	Column 9
Dimension	300*300	300*500	300*600	300*800	300*1000	500*500	300*1200	400*1200	300*700
Basement grid line	-	-	-		P4, S4			N11	
Ground floor grid line	-	G13, I13, N13, R13	E13, T13	B2, U2, G4, P4	G11, D4 , S4	D1, F1, H1, K1, M1, O1, Q1, S1	J11, N11		C13, W13, W10
Columns	**Column 1**	**Column 2**	**Column 3**	**Column 4**	**Column 5**	**Column 6**	**Column 7**	**Column 8**	**Column 9**
1st floor grid line	-	C13, E13, G13, I13, N13, R13, T13, B2, U2	-	G11, J11, N11, D4, G4, P4, S4		D1, F1, H1, K1, M1, O1, Q1, S1			
2nd floor grid line	-	C13, E13, G13, I13, N13, R13, T13, B2, U2	G11, J11, N11, D4, G4, P4, S4			D1, F1, H1, K1, M1, O1, Q1, S1			
3rd floor grid line	-	C13, E13, G13, I13, N13, R13, T13, B2, U2	G11, J11, N11, D4, G4, P4, S4			D1, F1, H1, K1, M1, O1, Q1, S1			
4th floor grid line	-	C13, E13, G13, I13, N13, R13, T13, B2, U2	G11, J11, N11, D4, G4, P4, S4			D1, F1, H1, K1, M1, O1, Q1, S1			
5th grid line	G11, J11, N11, D4, G4, P4, S4	C13, E13, G13, I13, N13, R13, T13, B2, U2	-			D1, F1, H1, K1, M1, O1, Q1, S1			

Continued in Next Page

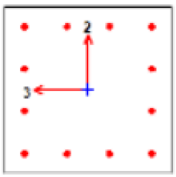
Area of steel %	1%	1%	1%	1%	1%	1%	1%	1%	1%
Area of steel	900	1500	1800	2400	3000	2500	3600	4800	2100
Longitudinal bars	8∅14	12∅14	12∅14	16∅14	18∅16	12∅18	20∅16	22∅18	16∅14
Stirrups	10∅200	10∅200	10∅200	10∅200	10∅250	10∅250	10∅250	10∅250	10∅200

3.9.2 Samples for columns forces and design results from the software

Figure 3.64 shows a sample for design of a column taken from ETABS.

ETABS Concrete Frame Design

ACI 318-19 Column Section Design (Summary)



Column Element Details

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	LLRF	Type
Story3	C36	13	colunm 500*500	1.2D+1.6L(all)	0	4400	0.573	Sway Ordinary

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
500	500	59	27.3

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
24870	28	1	420	420

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

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 % %
1398.2783	-6.6661	42.2839	42.2839	42.2839	2500	1

Axial Force and Biaxial Moment Factors

	C_m Factor Unitless	δ_{ns} Factor Unitless	δ_s Factor Unitless	K Factor Unitless	Length m	β_{dns} Unitless	EI_{eff} kN-m²	P_c kN
Major Bend(M3)	0.639795	1	1	1	4.4	0.80367	28726.1568	14644.4113
Minor Bend(M2)	0.483608	1	1	1	4.4	0.80367	28726.1568	14644.4113

Figure 3.64: Data sheet for column

3.9.3 Specifications for bars and ties distribution

Lateral reinforcement is required to prevent spalling of the concrete cover or local buckling of longitudinal bars. ACI code chapter 25.7.2 give rules and specifications of tie arrangement that might be summarized as:

1. The vertical spacing (center-to-center) of ties are the smaller of:

    - 48 d_s

- 16 d_b
- The least column dimension.

Where d_s is the diameter of the tie and d_b is the diameter of the longitudinal bars.

2. Rectilinear ties shall be arranged so that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 ° , and no bar shall be farther than 6 in. (150mm) clear on each side along the tie from a laterally supported bar.

Columns' reinforcement details and structural drawings are included in the **Appendix B**.

3.10 Design of walls

In this building structure, there are two types of walls with thicknesses of 200 mm and 300 mm, each assigned to a corresponding pier. Both wall types feature openings. The right, middle, and left walls—referred to as Wall 200 and Wall 300—are designated as piers P1 through P332, while the horizontal segments between the openings are assigned as spandrels S1 through S175.

The design of the shear walls in the building structure is based on the ETABS model, ensuring that the total base shear is fully resisted by these walls.

Define piers labels, the pier labels dialog box is displayed as shown in **Figure 3.65**.

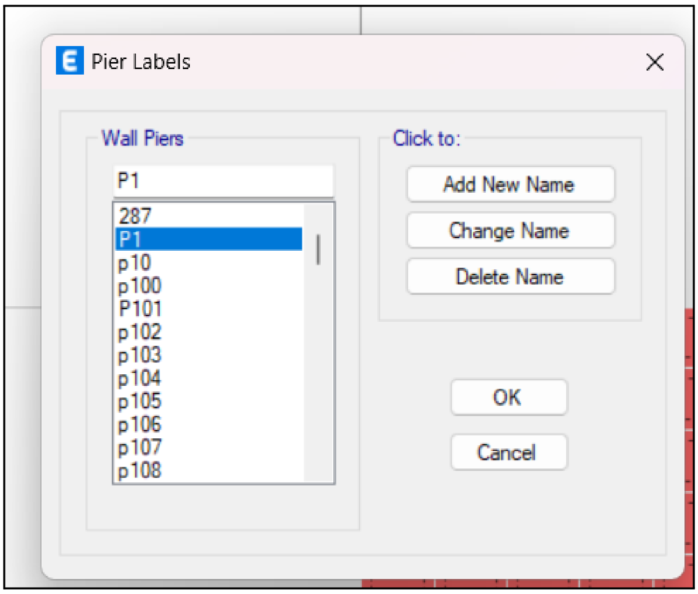


Figure 3.65: Pier labels dialog box in ETABS

The spandrel labels dialog box is displayed as shown in **Figure 3.66**.

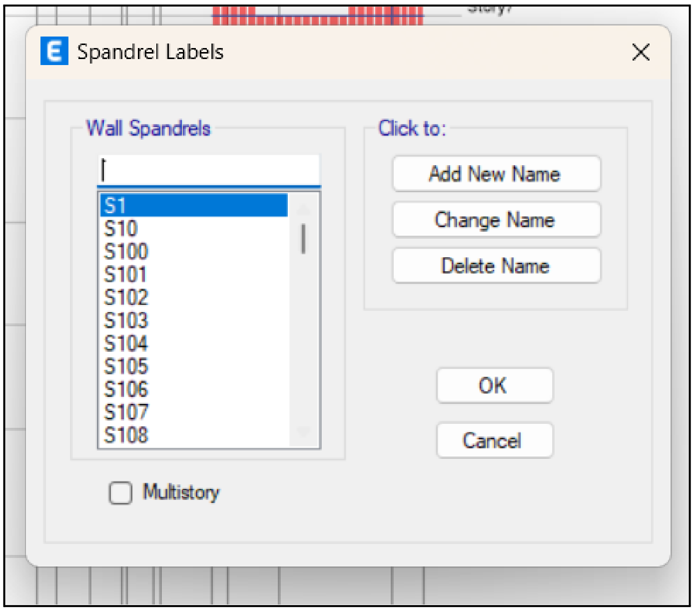


Figure 3.66: Spandrel labels dialog box in ETABS

The design steps are shown in **Figures 3.67-3.74**:

- Shear wall design preferences dialog box is displayed and edited as shown in **Figure 3.67**, design code ACI 318-19 ,wall ductility type is a special structural wall.

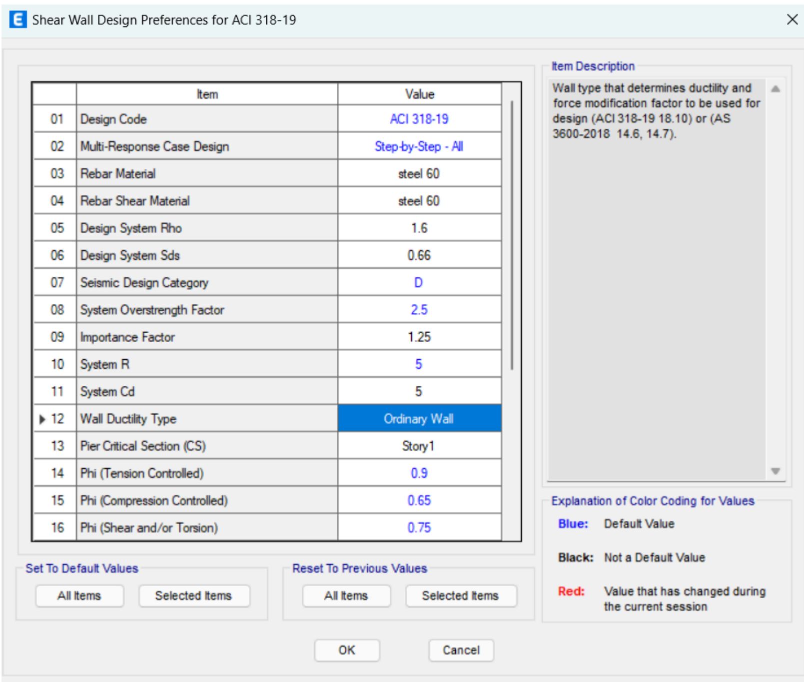


Figure 3.67: Shear wall design preferences dialog box in ETABS

- select a pier; design- shear wall designed -view/revise pier overwrites ,the wall pier design overwrites dialog box is displayed as shown in **Figure 3.68.**

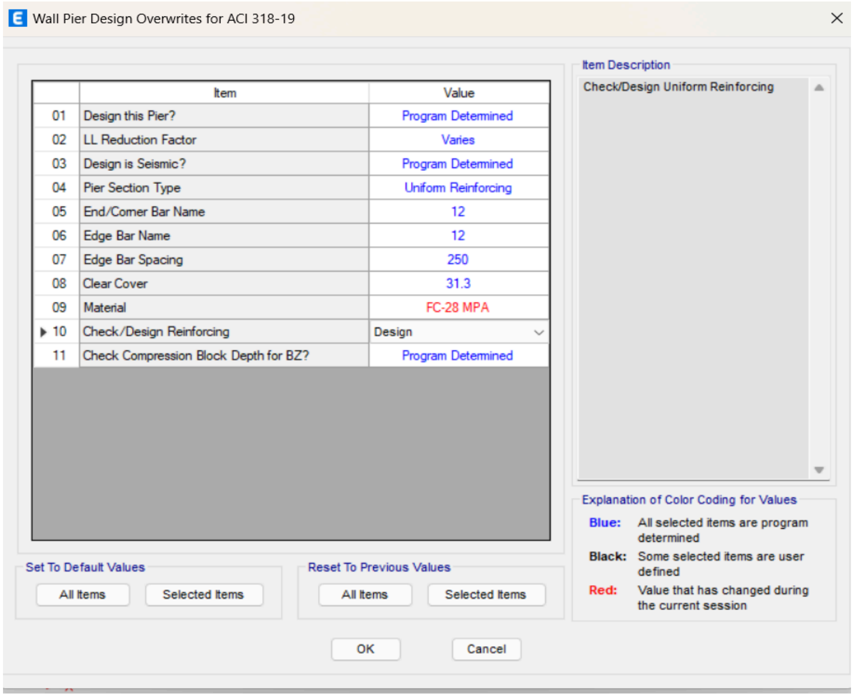


Figure 3.68: Wall pier design overwrites dialog box in ETABS

- Designing shear walls, select design combination, the design combination selection dialog box is displayed and edited as shown in **Figure 3.69**.

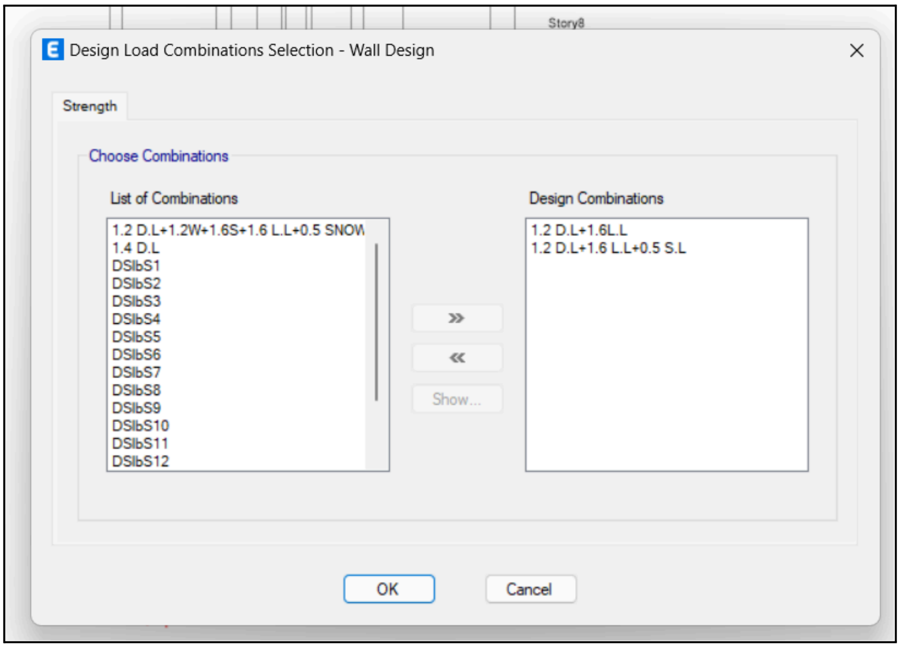


Figure 3.69: Design load combination -wall Design dialog box in ETABS

- Start design check, the program starts the design of the wall and the longitudinal reinforcements are displayed in a window .

- Design-shear wall design -display design info, the display shear wall design results dialog box is displayed as shown in **Figure 3.70**.

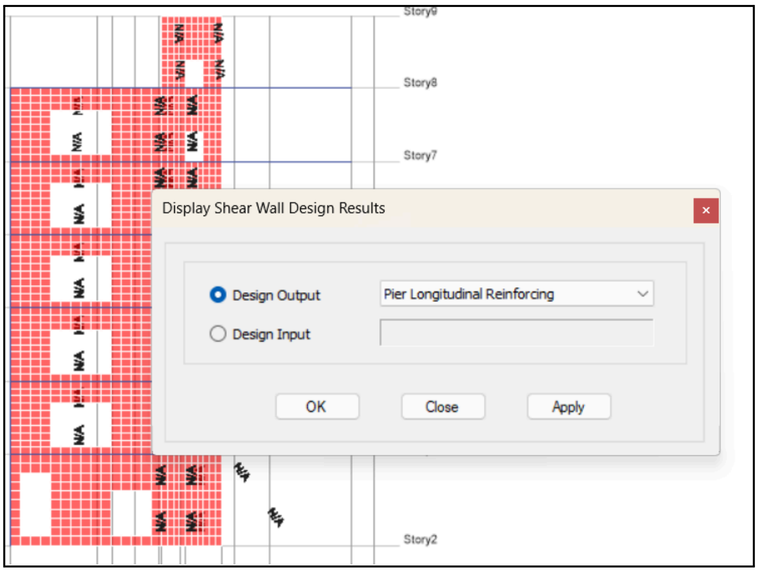


Figure 3.70: Display shear wall design results dialog box-longitudinal reinforcement in wall in ETABS

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
Story4	P2	1350	1524.9	1949.9	200	0.758

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
24870	28	1	420	420

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	1350	550	1350	2499.9	1949.9	200
Bottom	Leg 1	1350	550	1350	2499.9	1949.9	200

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	1044	0.0025	0.0022	SER	761.2216	4.5393	-44.2841	417430
Bottom	1044	0.0025	0.0022	SER	945.8409	1.3934	123.8061	417430

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	500	1.2 D.L+1.6 L.L+0.5 S.L	951.3213	-55.6555	93.0187	303.1184	610.2224
Bottom	Leg 1	500	1.4 D.L	1119.7492	154.3075	121.605	303.1184	610.2224

Figure 3.71: Data sheets for the vertical segments in the wall 200 in story 4 in ETABS

Flexural design for Pu, Mu2 and Mu3:

The design longitudinal reinforcement in the vertical direction is summarized in Table 3.17.

Table 3.17: Wall longitudinal reinforcement

Wall name	Wall thickness	Longitudinal reinforcement
W1, W2, W5, W6, W7, W8, W11, W21, W22, W23, W24, W25, W26, W27, W30, W31, W32, W33, W34, W35, W36, W37, W38, W39, W40, W41, W55, W56, W57, W58, W59, W60, W61, W62, W63, W64, W65, W66, W67, W68, W69	200 mm	Φ12/300
W3, W4, W9, W10, W12, W13, W14, W15, W16, W17, W18, W19, W20, W28, W29, W42, W43, W44, W45, W46, W47, W48, W49, W50, W51, W52, W53, W54	300 mm	Φ12/300

Same steps but for spandrels as shown in **Figure 3.72**:

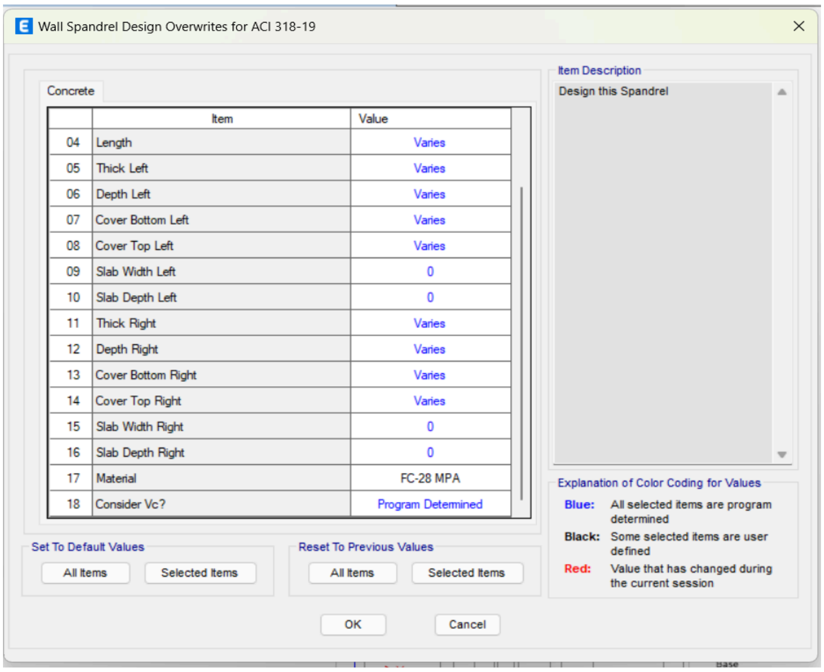


Figure 3.72: Wall pier design overwrites dialog box in ETABS

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
Story4	S26	6899	8439.6	1065	300	1

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
24870	28	1	420	420

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	0	0	SER	0
Right	0	0	SER	0

Spandrel Flexural Design—Bottom Reinforcement

Station Location	Reinf Area mm²	Reinf Percentage	Reinf Combo	Moment, M_u kN-m
Left	0	0	SER	0
Right	0	0	SER	0

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	750	750	1.2 D.L+1.6L.L	37.4203	194.3219	223.0411	417.363
Right	750	750	1.2 D.L+1.2W+1.6S+1.6 L.L+0.5 SNOW	35.8053	193.8851	223.0411	416.9262

Figure 3.73: Data sheets for the horizontal segments in the wall 300 in story 4 in ETABS

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
Story6	S11	1350	2499.9	1420	200	1

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
24870	28	1	420	420

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	33	0.01	1.4 D.L	0
Right	146	0.05	1.4 D.L	-104.4092

Spandrel Flexural Design—Bottom Reinforcement

Station Location	Reinf Area mm²	Reinf Percentage	Reinf Combo	Moment, M_u kN-m
Left	148	0.05	1.4 D.L	40.1164
Right	0	0	SER	0

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	500	500	1.2 D.L+1.6L.L	49.4663	167.1033	198.2588	365.3621
Right	500	500	1.2 D.L+1.2W+1.6S+1.6 L.L+0.5 SNOW	157.0295	176.5566	198.2588	374.8153

Figure 3.74: Data sheets for the horizontal segments in the wall 200 in story 4 in ETABS

The design of horizontal reinforcement is detailed in Table 3.18, which presents the horizontal reinforcement specifications for the walls.

Table 3.18: Wall horizontal reinforcement

reinforcement	Area of steel mm²/m	Walls
Φ10/300mm	500	200mm
Φ12/300mm	750	300mm

Walls reinforcement details and structural drawings are included in the **Appendix B**.

3.11 Design of footings

Design of footings is based on software analysis and design. A preliminary thickness and dimensions were determined by approximate and common methods then were checked by software. A check for stress under footings (bearing capacity check) was performed and it is discussed in this section too.

3.11.1 Ground floor foundations

3.11.1.1 Preliminary thickness

The footing dimensions were calculated as shown in **Table 3.19** and **Table 3.20**. **Table 3.20** provides the area of each footing, while **Table 3.19** details the thicknesses. The areas were determined using the equation $A_f = P_{service}/q_{allowable}$ (preliminary area that was modeled on ETABS) where $P_{service}$ represents the service load and $q_{allowable}$ is the allowable soil bearing pressure. The thickness of each footing was calculated based on the wide beam shear check to ensure adequate structural performance and safety. The assumed areas and thicknesses are verified by ETABS, that is, software results confirm adequacy of the assumed dimensions.

Figure 3.75 shows the numbering of footings.

Table 3.19: Thickness of footings

Footing	Thickness (mm)	d (mm)	Cover (mm)	Vu	qu	ΦVc	l n (m)	Steel ratio	Mu	Numbering
UPPER WALL	600	520.00	80.00	144.0	300.00	184.272	1	0.00247	248	Fwall 2
RIGHT WALL 1	700	620.00	80.00	175.5	225.00	197.545	1.4	0.0018	160	Fwall 4
RIGHT WALL 2	650	570.00	80.00	153.0	225.00	181.614	1.25	0.0018	160	Fwall 3
LEFT WALL 1	550	470.00	80.00	132.5	250.00	149.752	1	0.0018	80	Fwall 1
LEFT STAIRCASE	700	620.00	80.00	173.25	275.00	197.54	1.25	0.0018	240	Fwall 4
LEFT WALL 2	550	470.00	80.00	141.75	225.00	153.706	1.1	0.00194	160	Fwall 1

Footing	Thickness (mm)	d (mm)	Cover (mm)	Vu	qu	ΦVc	l n (m)	Steel ratio	Mu	Numbering
Down	550	470.00	80.00	159.0	300.00	176.479	1	0.00294	240	Fwall 1
F1	850	770.00	80.00	219.0	300.00	245.338	1.5	0.0018	320	F1
F2	850	770.00	80.00	219.0	300.00	245.338	1.5	0.0018	320	F1
F3	850	770.00	80.00	219.0	300.00	245.825	1.5	0.0018	320	F1

Table 3.20: Areas of footings

Type of footing in Ground floor	Label	F_z	Area	B	Dimensions
	unit	KN	m2	m	m
Single	G-11	1983.38	7.93	3.00	3*3
Single	J-11	2086.26	8.35	3.00	3*3
Single	D-4	1191.12	4.76	3.50	3.5*1.5
Single	G-4	2015.69	8.06	3.00	3*3

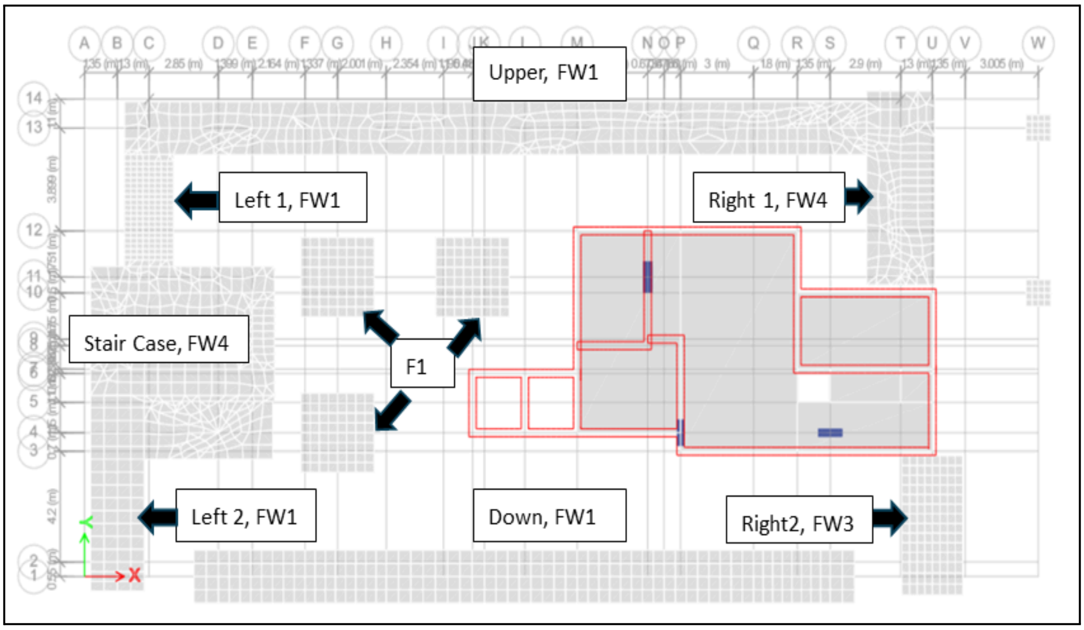


Figure 3.75: Footing numbering

3.11.1.2 Flexure design of footings

The area of steel for both bottom and top rebar was obtained from ETABS, as shown in **Figures 3.76** to **3.79**. These values are summarized in **Table 3.21**.

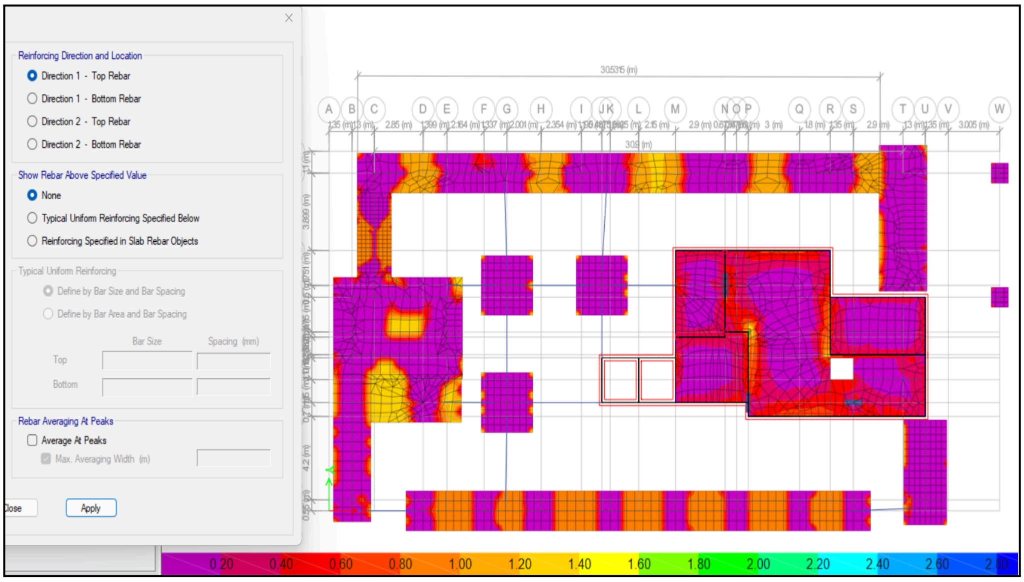


Figure 3.76: Top rebar reinforcing direction 1

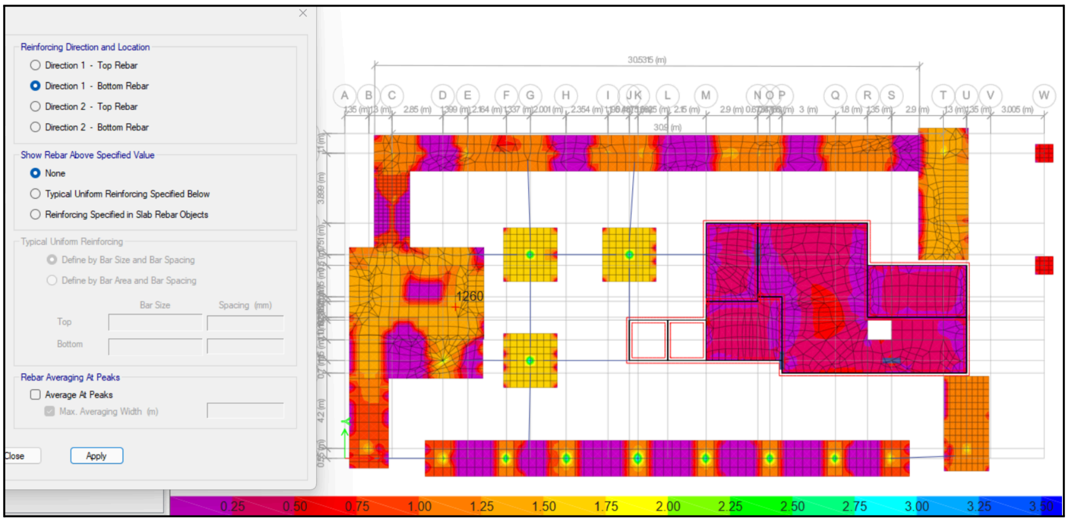


Figure 3.77: Bottom rebar reinforcing direction 1

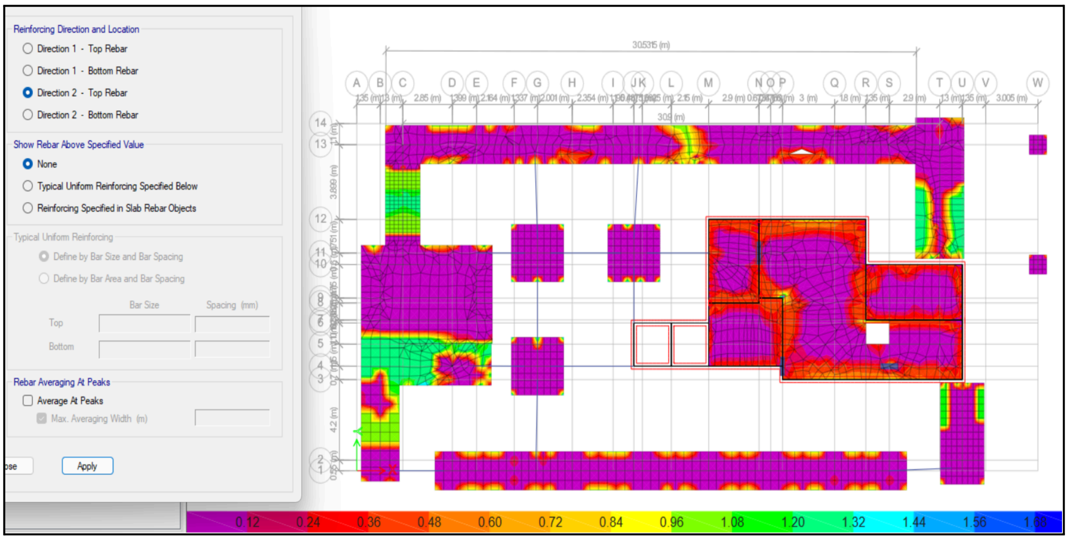


Figure 3.78: Top rebar reinforcing direction 2

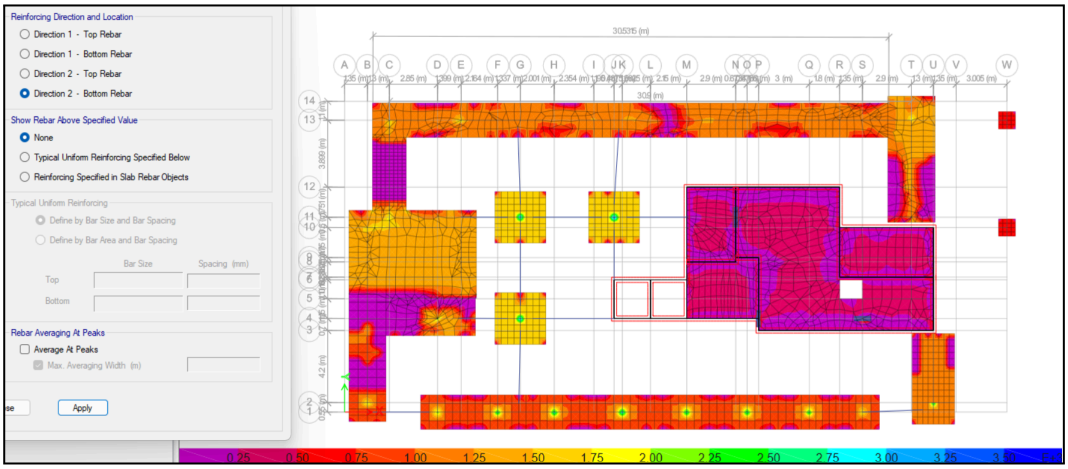


Figure 3.79: Bottom rebar reinforcing direction 2

Table 3.21: Footing reinforcement

Footing	Top	Bottom
UPPER WALL	Ø18 /200 mm	Ø20 /200 mm
RIGHT WALL 1	Ø18 /200 mm	Ø18 /200 mm
RIGHT WALL 2	Ø18 /200 mm	Ø18 /200 mm
LEFT WALL 1	Ø18 /200 mm	Ø20 /200 mm
LEFT STAIRCASE	Ø18 /200 mm	Ø18 /200 mm
LEFT WALL 2	Ø16 /200 mm	Ø16 /200 mm
down	Ø16 /200 mm	Ø20 /150 mm

Footing	Top	Bottom
F1	Ø20 /200 mm	Ø20 /200 mm
F2	Ø20 /200 mm	Ø20 /200 mm
F3	Ø20 /200 mm	Ø20 /200 mm
F4	Ø18 /200 mm	Ø18 /200 mm
F5	Ø16 /250 mm	Ø16 /250 mm
F6	Ø16 /250 mm	Ø16 /250 mm

Footings' reinforcement details and structural drawings are included in the **Appendix B**.

3.11.1.3 Punching check

A punching shear check must be performed to ensure the footing thickness is adequate to handle the column loads without exceeding the material's shear capacity. This ensures the foundation can safely transfer concentrated forces and prevents structural failure. **Figure 3.80** shows the punching operation for the ground floor foundations.

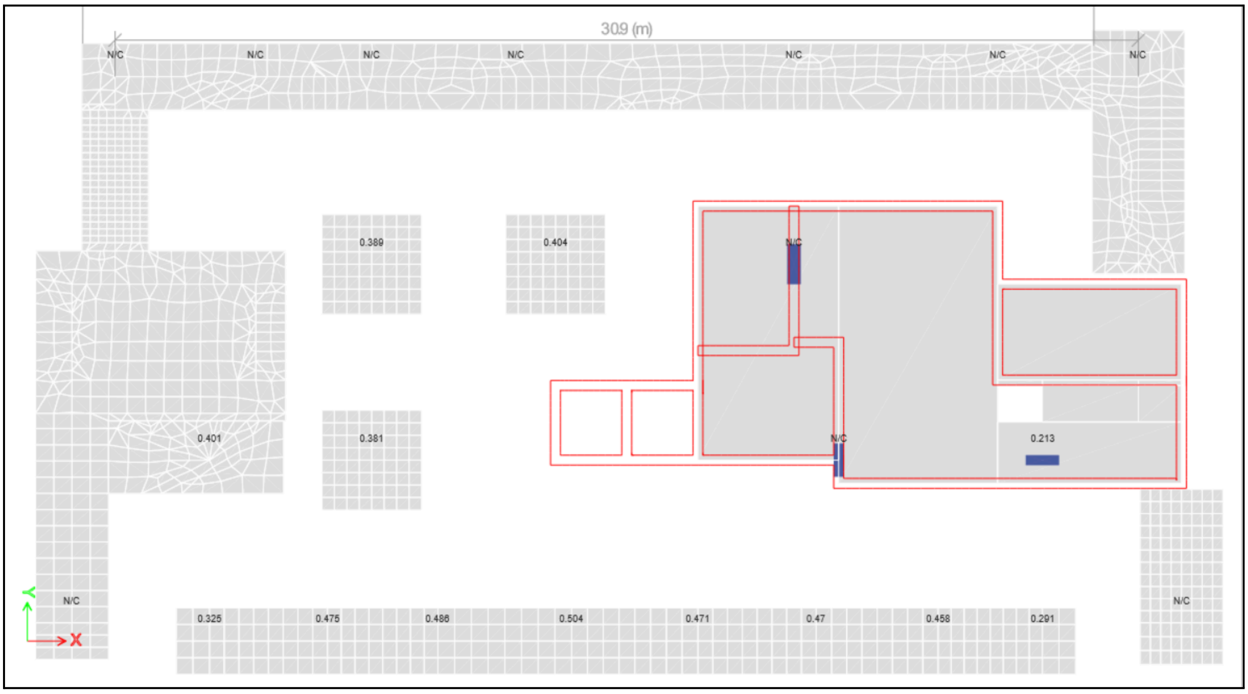


Figure 3.80: Punching operation from ETABS

As shown in the figure above, all punching shear values are less than one, indicating that the foundation is adequately designed to handle the applied loads.

3.11.1.4 Design summary

Table 3.22 presents the design of all footings including dimensions, thicknesses, and reinforcement.

Table 3.22: Flexure design for footings

Footing	Thickness (mm)	d(mm)	B(m)	Top Reinforcement (mm^2)	Bottom Reinforcement (mm^2)	Ast(min)(mm^2/m)	Top Ast(mm^2/m)	Bottom Ast(mm^2/m)
UPPER WALL	600	520	2	2553	2726	1080	1276.5	1363
RIGHT WALL 1	700	620	2.8	441	2762	1260	1260	1260
RIGHT WALL 2	650	570	2.5	402	2124	1170	1170	1170
LEFT 1 WALL	550	470	2	2458	2820	990	1229	1410
LEFT STAIRCASE	700	620	5.116	996	4153	1260	1260	1260
LEFT WALL 2	550	470	2.2	1237	1974	990	990	990
down	550	470	2	1818	3598	990	990	1799

3.11.2 Basement floor foundations (Mat)

Mat foundation system was selected for the basement foundation due to the limited basement area, making it an efficient and practical choice. **Figure 3.81** illustrates the mat foundation layout.

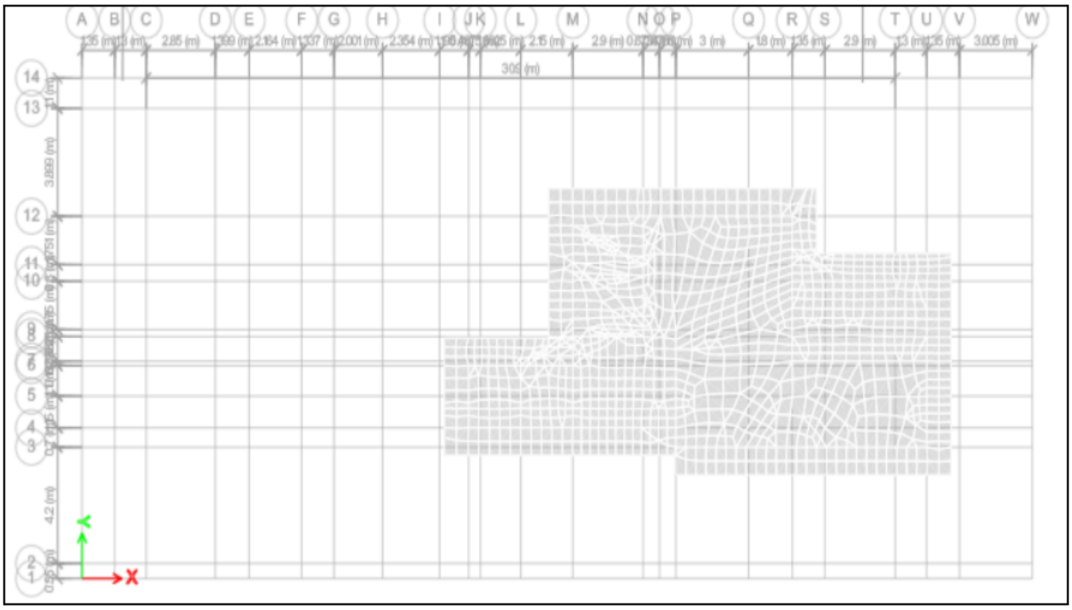


Figure 3.81: Mat foundation layout

3.11.2.1 Check the stress under the mat

The stress under the mat foundation had to be less than the allowable soil stress, which was 250 kN/m². The stress values were obtained from the ETABS, as shown in **Figure 3.82**.

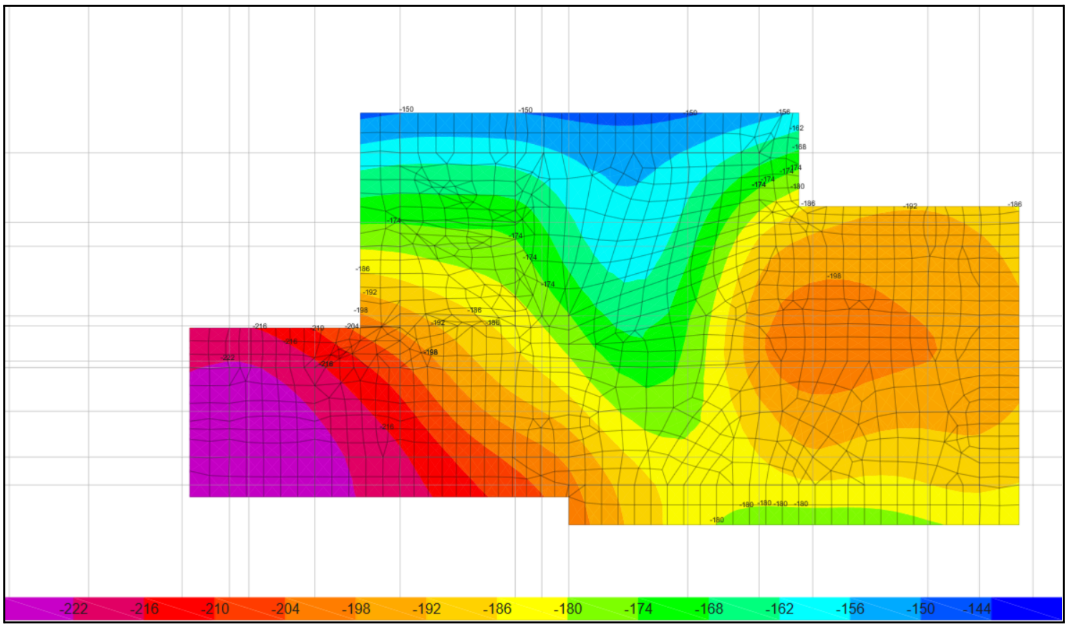


Figure 3.82: Stress under the mat foundation from ETABS

As shown in **Figure 3.82** above the stress under the mat foundation is smaller than the allowable bearing capacity of the soil which is 250 kN/m^2.

3.11.2.2 Check shear of the mat

Vu =**141**kN (from ETABS , **Figure 3.83** and **Figure 3.84**).

$\varnothing$Vc (no shear reinforcement) = $\varnothing \times 0.66 \times \lambda s \times \lambda \times \rho_w^{1/3} \times \sqrt{f`c} \times b_w \times d$

$= 0.75 \times 0.66 \times 0.72 \times 1 \times 0.002^{1/3} \times \sqrt{28} \times 720 \times 1000/1000 =$ **171** kN.

$\varnothing$Vc > Vu ✔ → no need for shear reinforcement.

Where: $\rho_w = (800/720) \text{ x } 0.0018=0.002$.

And $\lambda s = \sqrt{\dfrac{2}{1+0.004\, x\, 720}} = 0.72$

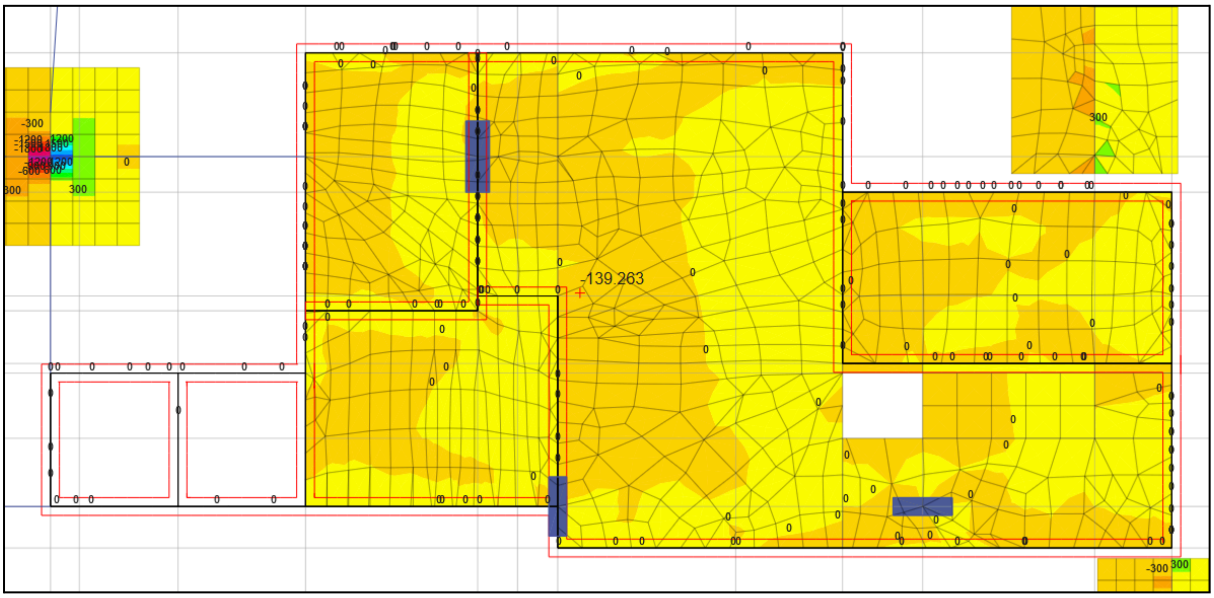


Figure 3.83: Shear values (V13) in the mat foundation (basement)

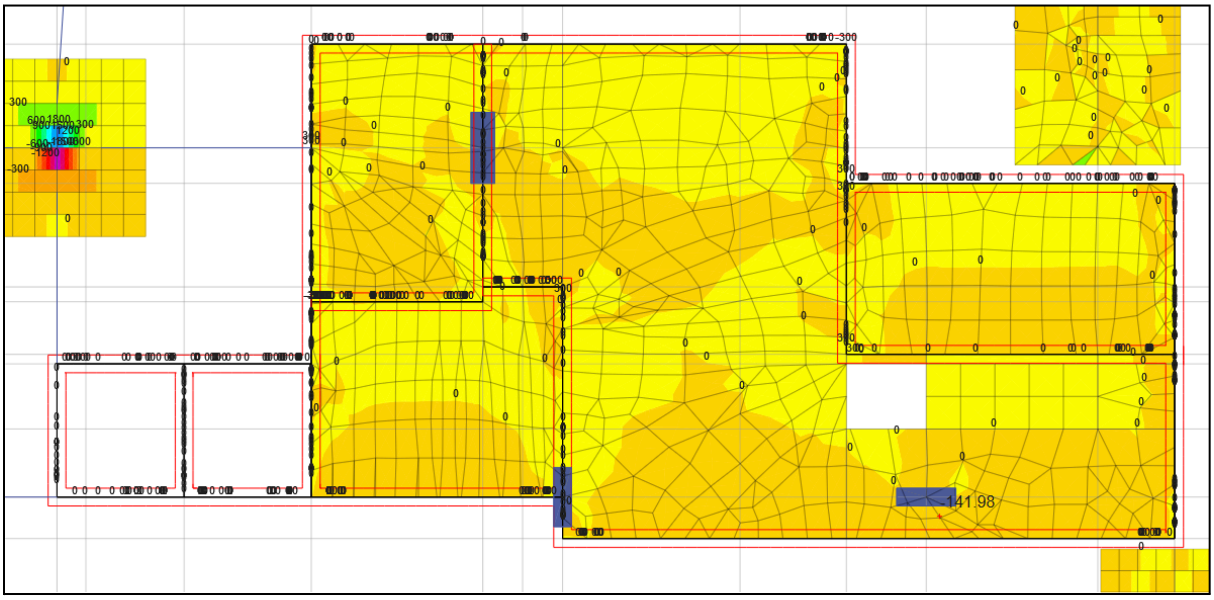


Figure 3.84: Shear values (V23) in the mat foundation (basement)

3.11.2.3 Flexural design of mat

Mat foundation reinforcement has been taken from ETABS directly as shown in **Figures 3.85 and 3.86**, the area of steel of both top and bottom has been taken as 1440 mm^2. Base of the water tank has been designed separately.

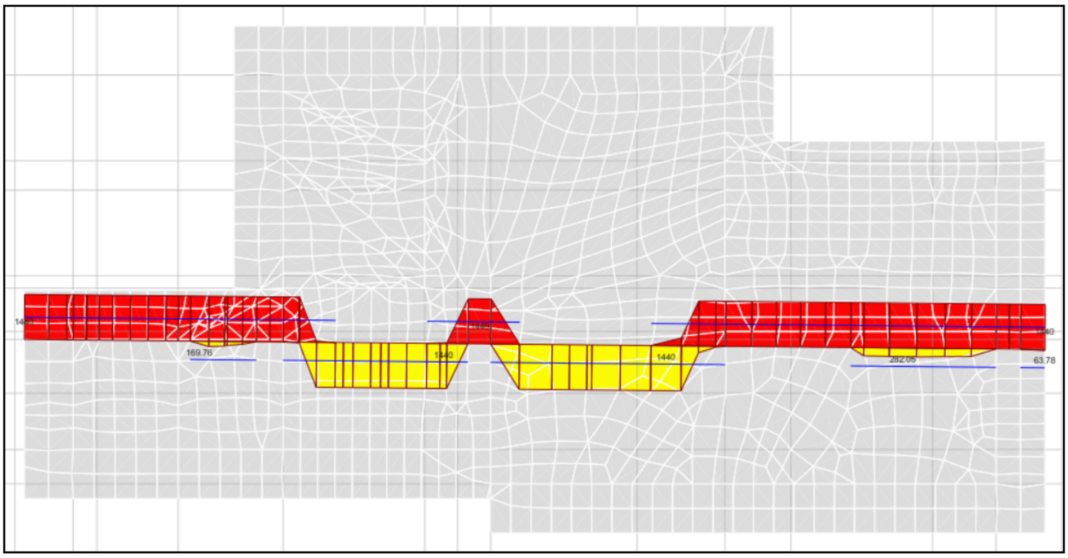


Figure 3.85: Flexural reinforcement of mat foundation in direction 1; X-direction

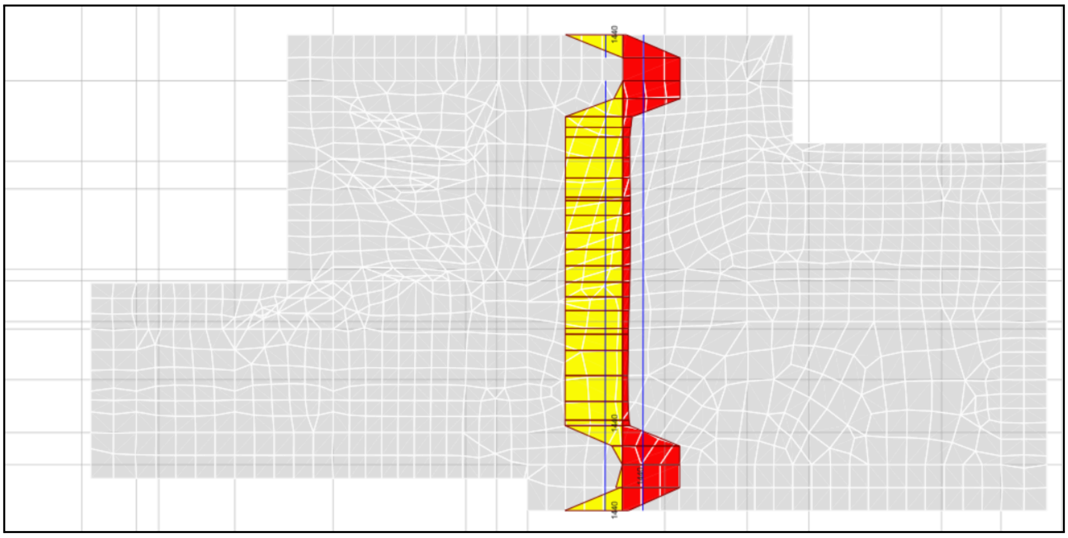


Figure 3.86: Flexural reinforcement of mat foundation in direction 2; Y-direction

As shown in **Figures 3.85** and **3.86** above the area of steel for both directions, top and bottom are 1440 mm^2, so the bars are distributed as **1Ø18/150mm**.

3.12 Design of tie beams

3.12.1 Dimensions and reinforcement

The following dimensions are used:

Width=300mm

Thickness=500mm

Cover=40mm

ETABS results for design of tie beams are illustrated in **Figure 3.87**. The area of steel given by software meets the minimum steel ratio (0.0033). This is the normal and usual case; results are reasonable. Minimum area of steel is given by:

Area of steel = 0.0033*460*300 = 455 mm^2

ETABS result = 455mm^2 (shown in **Figure 3.87**)

The same figure shows a value of 676 mm^2 at the top face of the middle region of a beam; therefore, bars details can be summed up as follows:

- 455 mm^2 → 3Ø14
- 676 mm^2 → 3Ø18

Tie beams details and structural drawings are all illustrated in **Appendix B**.

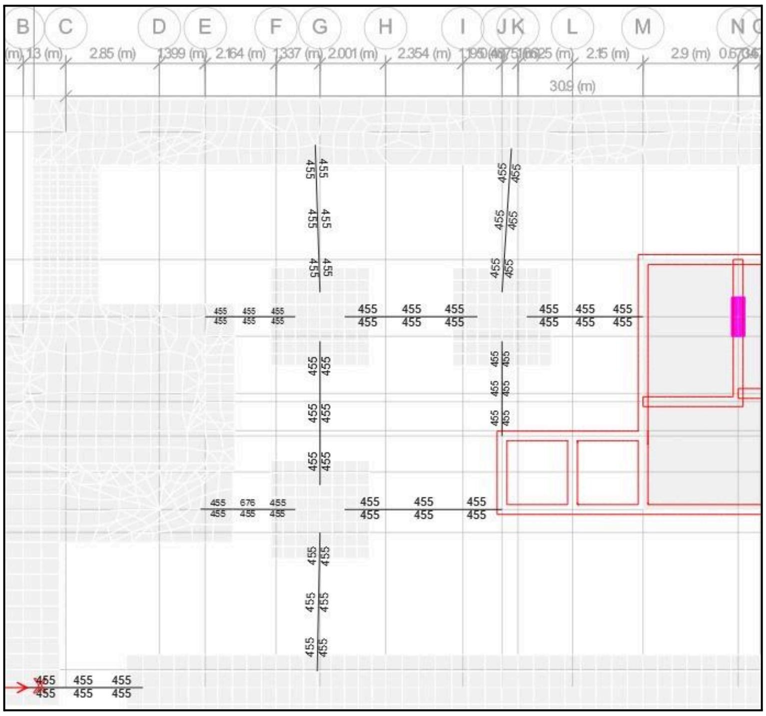


Figure 3.87: Design of tie beams from ETABS

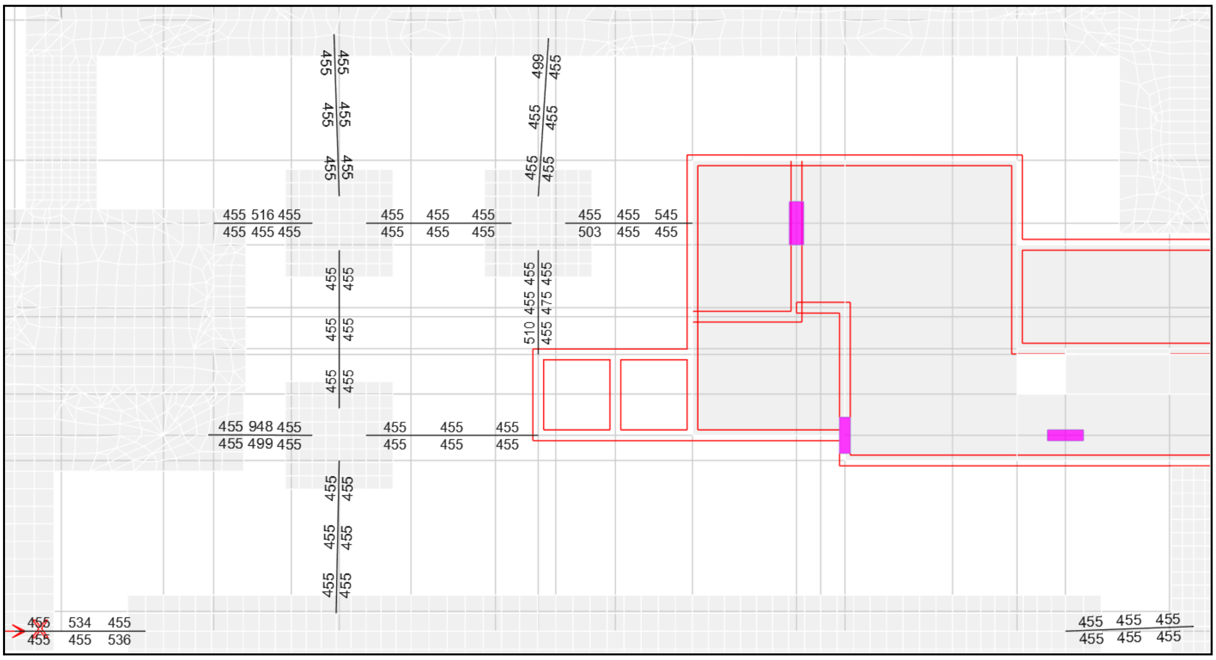


Figure 3.87a: Design of tie beams from ETABS (Total)

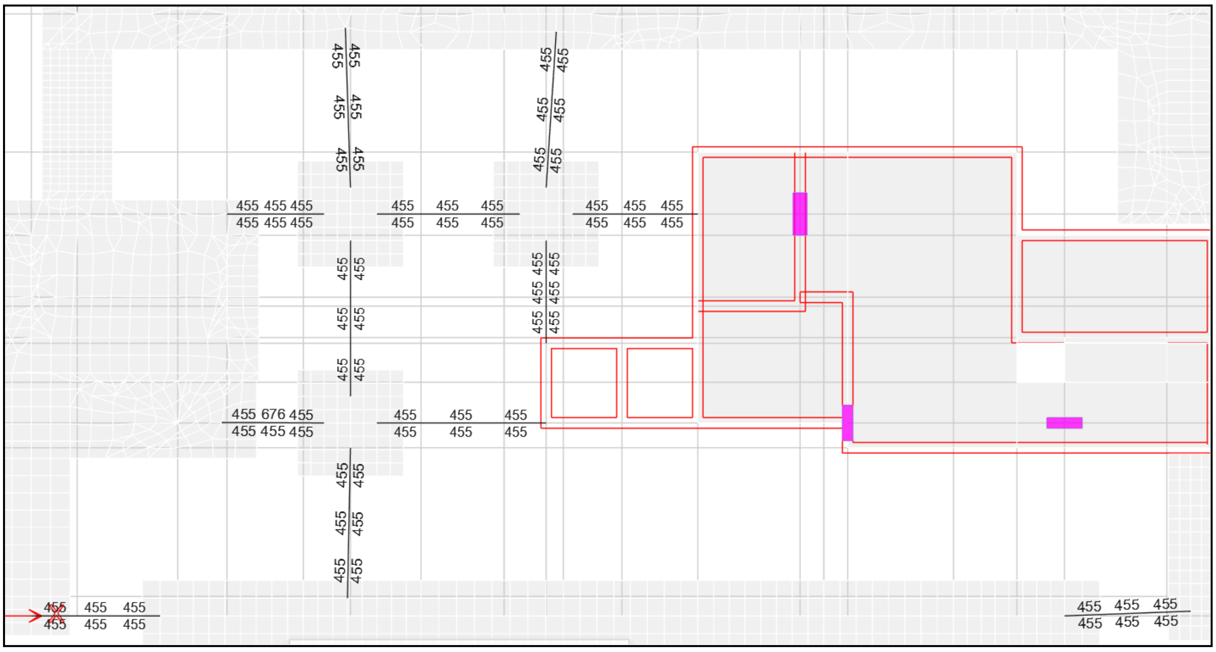


Figure 3.87b: Design of tie beams from ETABS (Flexural)

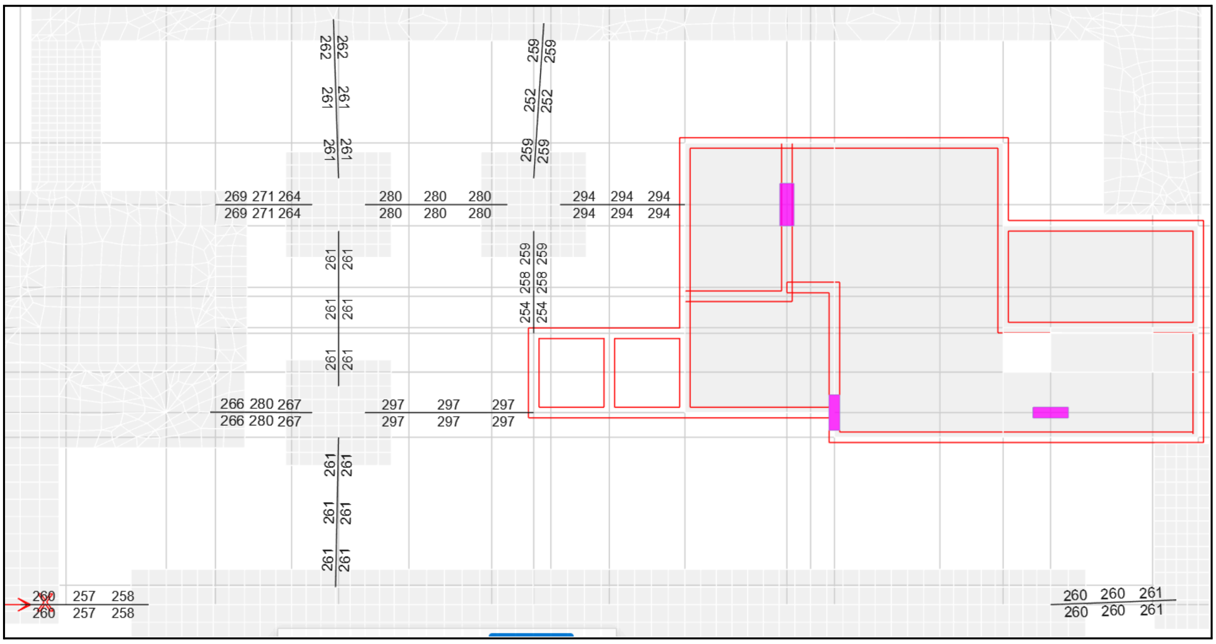


Figure 3.87c: Design of tie beams from ETABS (Torsional)

3.13 Design of stairs

3.13.1 Considerations and criteria

- This staircase is supported at edges of the long direction. Therefore, the structural model is a one-way solid slab, not a cantilever.
- In concrete structures, joints transfer internal forces with different ratios; supports are neither fixed nor pin. For this reason, and to be on the safe side, pin supports are assumed in calculations of flexural internal forces in the waist (flight). These forces are based on hand calculations rather than software analysis.
- The internal forces of landings are based on software analysis in order to take the effect of the surrounding structural elements into account.
- Live load and superimposed dead load are equal in landings and flights. **Table 3.23** shows live load and superimposed dead load in the staircase.
- Steps are treated as nonstructural elements and only the inclined slab is reinforced and resist the applied loads. Steps load is included in the superimposed dead load.
- Thickness of flight equals thickness of landings.
- Self weight (dead load) of the flight is calculated for the inclined length. And live load is calculated for the horizontal length.

- Dead load per horizontal meter length has a larger value in flights compared to landings for the same slab thickness. However, the larger value is considered in the design of flights to simplify analysis.
- Concrete cover = 40mm.

Table 3.23: Staircase loads

Objects	SD load (kN/m^2)	Live load (kN/m^2)
Landings	4	5
Flights	4	5

3.13.2 Calculation of dead load in stair case

Flight vertical length is the difference between any two successive elevations of those shown in **Figure 3.88 and Figure 3.89.**

span of flight=2.8+(2*0.25)=3.3m

Thickness of slab =3.3/20=170mm

Span of landing =2.6 (as shown in **Figure 3.91**)

Thickness of slab =2.6/20=130mm

Where the preliminary thickness of a one way solid slab = span length/20

flight dead load =0.17*25=4.25kN/m^2

dead load in landings=0.13*25=3.25kN/m^2

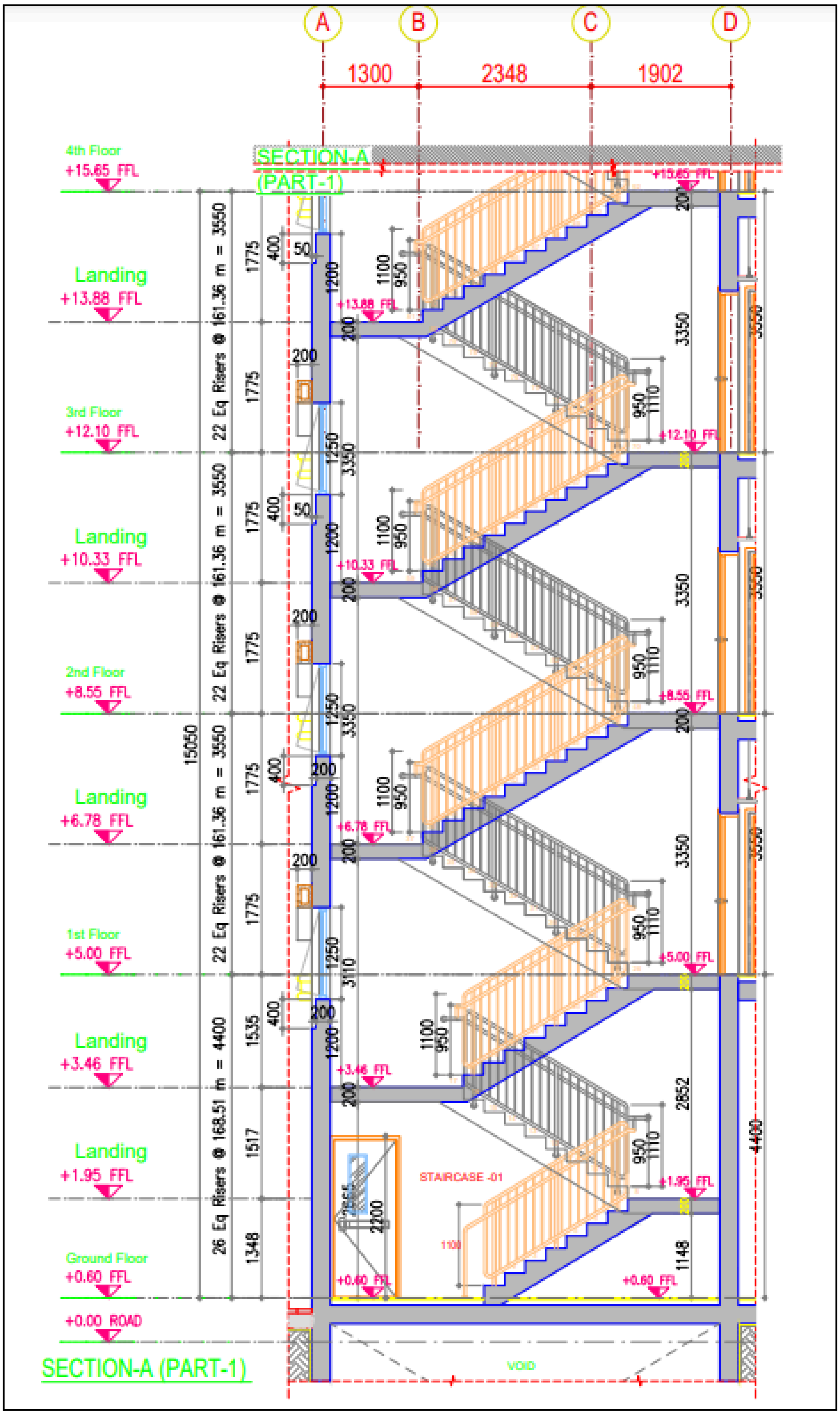


Figure 3.88: Detailed and elevation staircase_1

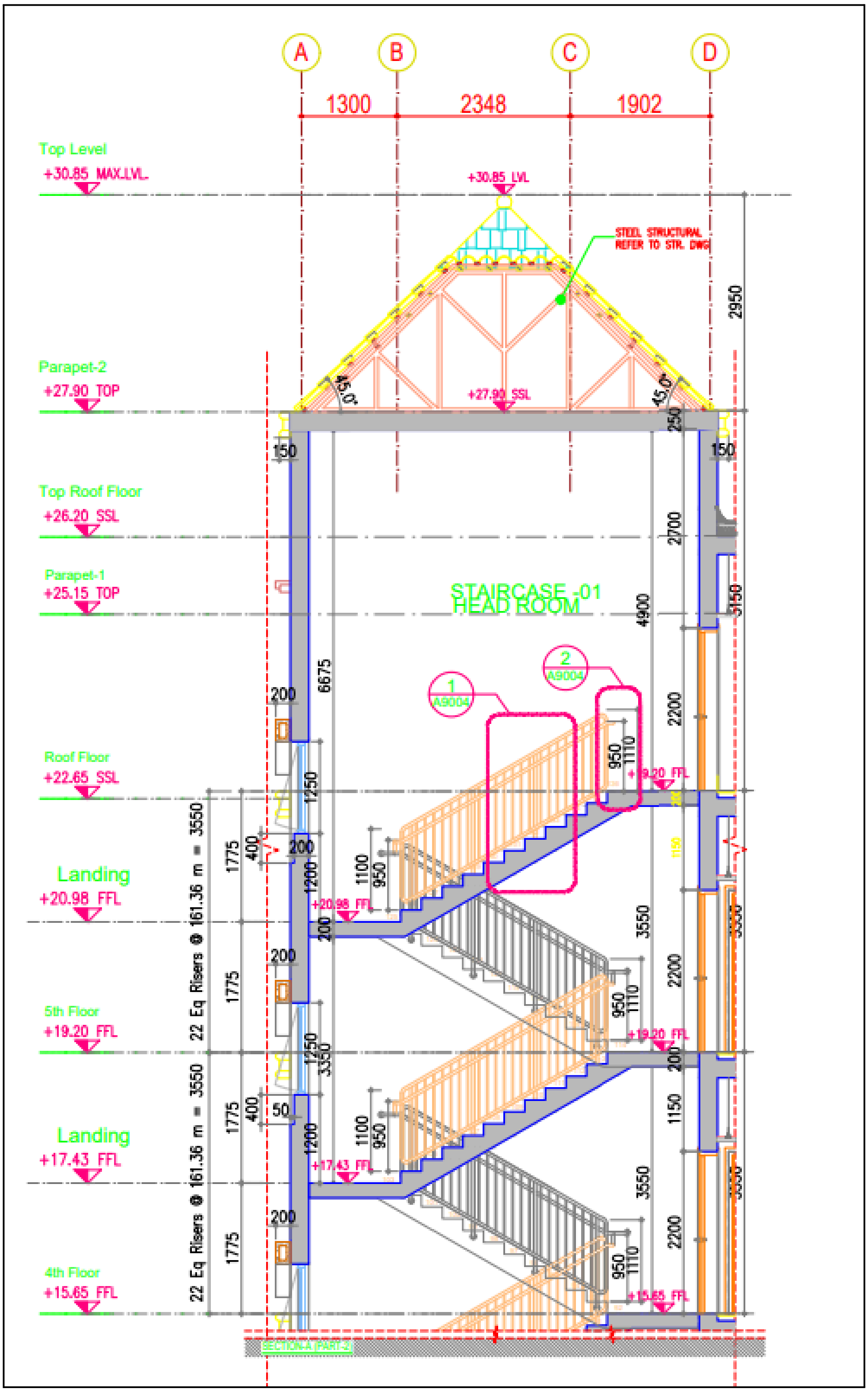


Figure 3.89: Detailed and elevation staircase_2

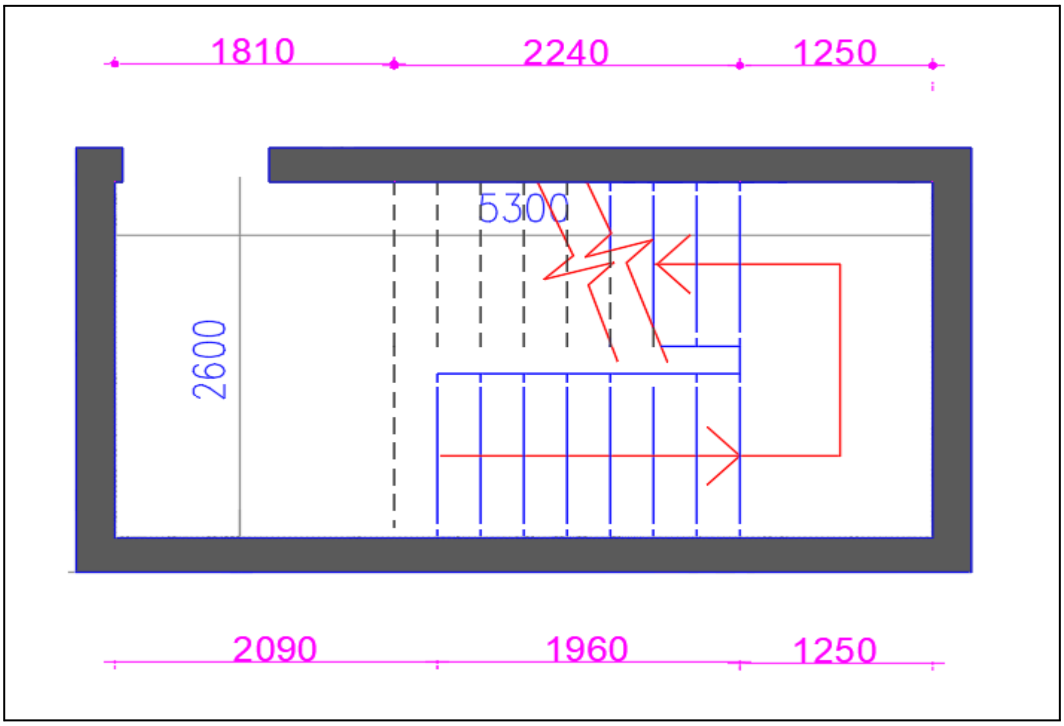


Figure 3.90: Dimensions of staircase in the ground floor

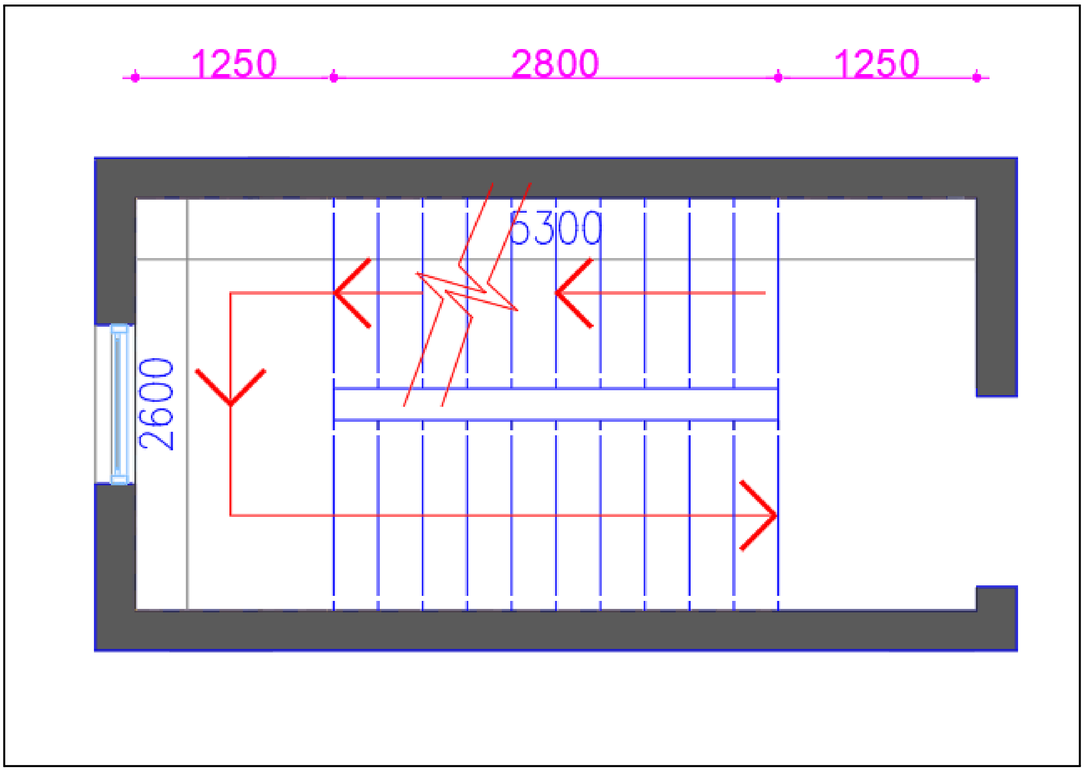


Figure 3.91: Dimensions of staircase in typical floors

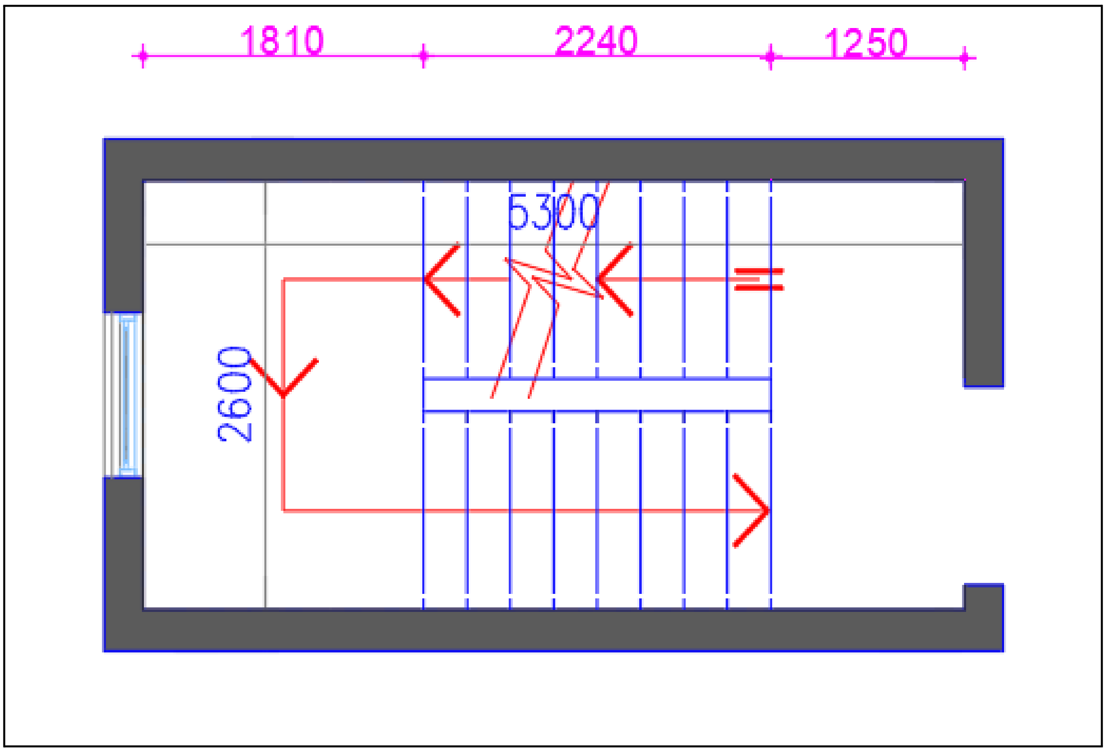


Figure 3.92: Dimensions of staircase in the first floor

3.13.3 Analysis and design calculations

Ultimate load= 1.2D+1.6L

Ultimate load= 1.2*(7+4)+1.6*(5)= 21.2 kN/m²

1.Shear check:

Vu = 21.2* (3.3/2)= 35kN

$\emptyset Vc = \emptyset \times 0.17 \times \lambda \times \sqrt{f`c} \times b_w \times d$

$= 0.75 \times 0.17 \times 1 \times \sqrt{28} \times 1000 \times 130/1000 = 88$ kN

$\emptyset Vc > Vu$ ✔ → No need for shear reinforcement. Concrete provides shear capacity.

2. Flexural design:

$M_u = w_u l^2/8 = 21.2*(3.3^2)/8$

$\mathbf{M_u = 29}$ kN.m

$$\rho = \frac{0.85 \times 28}{420} \left(1 - \sqrt{1 - \frac{2.61 \times 29E6}{28 \times 1000 \times 130^2}} \right) = 0.00473$$

0.00473 > 0.0018 → 0.00473 is used

Main flexural steel in flights → A_s =0.00473 * 1000 *130 =615mm²

Shrinkage steel = 0.0018 * 1000 * 170 = 306 mm²

Main flexural steel in landings → reinforcement → A_s = 615mm²

For landing:

Design the landing as a beam that carries its load plus reaction from the flight .

Ultimate load =21.2+13=34.2KN

1.Shear check:

Vu = 34.2* (2.6/2)= 44.5kN

$\varnothing Vc = \varnothing \times 0.17 \times \lambda \times \sqrt{f`c} \times b_w \times d$

$= 0.75 \times 0.17 \times 1 \times \sqrt{28} \times 1000 \times 90/1000 = 60.7$ kN

$\varnothing$Vc > Vu ✔ → No need for shear reinforcement. Concrete provides shear capacity.

2.Flexural design:

$M_u = w_u l^2/8 = 44.5*(2.6^2)/8$

$\mathbf{M_u = 37.6}$ kN.m

$$\rho = \frac{0.85 \times 28}{420} \left(1 - \sqrt{1 - \frac{2.61 \times 37.6E6}{28 \times 1000 \times 90^2}} \right) = 0.014$$

0.014 > 0.0018 → 0.014 is used

Main flexural steel in flights → A_s =0.014 * 1000 *90=1260mm²

Shrinkage steel = 0.0018 * 1000 * 130 = 234 mm²

Main flexural steel in landings → reinforcement → A_s = 1260mm²

3.13.4 Bars details

1.Flight:

- Bottom face and long direction of the flight → (1Ø12/200mm)
- Bottom face and short direction of the flight → (1Ø12/250mm)
- Top face and both direction of the flight → (1Ø12/250mm)

2.Landing:

- Bottom face and long direction of the landing→ (1Ø16/150mm)
- Bottom face and short direction of the landing→ (1Ø12/250mm)
- Top face and both direction of the landing→ (1Ø12/250mm)

Bars distribution, details, and structural drawings for stairs are all illustrated in **Appendix B**.

3.14 Design of water tank

3.14.1 Considerations and criteria

There are two tanks of interest in this design. The first is exposed to normal conditions. While the other has severe exposure. Despite that, and for simplicity, severe exposure is considered in both tanks. This consideration and other consideration for design are illustrated below:

- **Ultimate design method** with environmental durability factor S_d is used instead of **allowable stress method**.
- Water load used in design = (10 kN/m² * height(m)) for a strip of one meter length.
- Actual soil load = 0.66*18 * height = (11.88 kN/m² * height(m)).
- Water load acts on the whole surface of the wall from top to bottom. This guarantees a safe structure in case of floods.
- Some walls are loaded from both sides. This means that the net force acting on the walls is less than the water or soil loads because they are opposite in direction. However, the worst case is considered. In other words, these walls should withstand the applied load taking into account that case when the loads do not act simultaneously. This might occur in the construction phase and should not be ignored.
- For those walls in the previous point, and in order to simplify calculations, soil load is considered to be equal to water load, where water load is a little larger. Thus, flexural reinforcement does not vary in outer and inner faces of the walls.
- There are only two walls that are subjected to water load only. However, these walls are designed to withstand loads acting on both sides, same as the other walls that were mentioned in the two points above.

- Severe exposure is considered in both tanks.
- For cracks control, spacing between flexural bars does not exceed 150mm.
- Bar diameter is assumed to be equal to 32 mm in the calculations of f_s for flexure. This assumption leads to the maximum possible S_d factor. This permits changing diameter to any preferred value as extreme possible load is assumed (safe side).
- γ = 1.4 in S_d factor. Where the actual value equals 1.4 for load combination 1.4D and ranges from 1.2 to 1.6 for load combination 1.2D+1.6L (and closer to 1.2 as the major load is dead load).
- Minimum flexural steel ratio is 0.0033 rather than 0.0018 in the walls. However, base thickness is equal to basement thickness. Steel ratio is expected to be too small as the base is too thick; thus, 0.0018 is used and it matches ACI code specifications (0.0033 is too much larger than required; uneconomical design).
- Shrinkage steel ratio = 0.003 in both horizontal and vertical directions as specified by **ACI 350-20**. Where the maximum length in the horizontal direction is less than 9 m (30 ft). **Figure 3.93** shows the minimum shrinkage steel ratio, **Table 7.12.2.1** from the mentioned code.

TABLE 7.12.2.1—MINIMUM SHRINKAGE AND TEMPERATURE REINFORCEMENT

Length between movement joints, ft	Minimum shrinkage and temperature reinforcement ratio	
	Grade 40	Grade 60
Less than 20	0.0030	0.0030
20 to less than 30	0.0040	0.0030
30 to less than 40	0.0050	0.0040
40 and greater	0.0060*	0.0050*

*Maximum shrinkage and temperature reinforcement where movement joints are not provided.

Note: This table applies to spacing between expansion joints and full contraction joints. When used with partial contraction joints, the minimum reinforcement ratio shall be determined by multiplying the actual length between partial contraction joints by 1.5.

Figure 3.93: Minimum shrinkage and temperature reinforcement in environmental structures

3.14.2 S_d factor

$$S_d = \frac{\Phi * fy}{\gamma * fs}$$

f_s values for severe exposure:

- $f_s = \frac{45560}{\beta\sqrt{(s^2+4(50+\frac{d_b}{2})^2)}}$ MPa   For Flexure
- f_s = 117 MPa   For Tension
- f_s = 138 MPa   For Shear

Where:

- S: spacing between bars, mm.
- d_b: diameter of bar, mm.
- h: overall thickness of member, mm.
- d: effective depth of section, mm.
- c: distance from extreme compression fiber to neutral axis. It is calculated at service loads for cracked section, mm

Assumptions for calculations:

- s = 150 mm
- d_b =32 mm
- γ = 1.4

These assumptions give the following fs values:

- Flexure fs = 168.9 MPa (h<400mm, β=1.35)
- Flexure fs = 190 MPa (h>400mm, β=1.2)
- Tension fs = 117 MPa
- Shear fs = 138 MPa (Ignored; shear capacity is provided by concrete (wall thickness))

250 ≥ 190 , 168.9 > 117 ✔ (one way member, walls)

250 > 190 , 168.9 > 138 ✔ (two way member, base)

Thus; S_d equals:

- Flexure S_d = 1.59 ≈ **1.6** for walls (thickness<400mm , β=1.35).
- Flexure S_d = 1.42 for base (thickness>400mm , β=1.2).

- Tension $S_d = 2.3$.

3.14.3 Wall selection and design

Figure 3.94 illustrates the wall located at grid line P. Design calculations for this wall are discussed and explained in this section. The final design of all other walls is presented in **Table 3.24**.

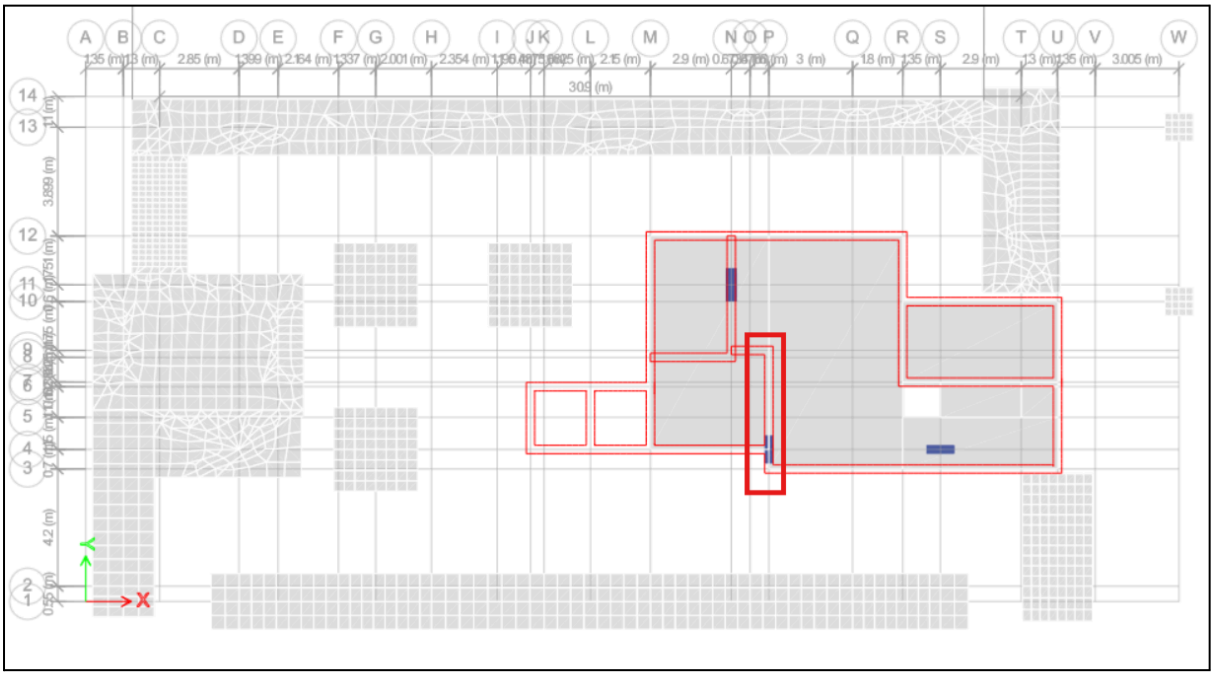


Figure 3.94: Water tank wall to be designed

1. **Flexural design:**

   $M_u = 26$ kN.m is the maximum moment in negative and positive regions.

   $M_u * S_d = 26 * 1.6 = 41.6$ kN.m

$$\rho = \frac{0.85 \times 28}{420} \left(1 - \sqrt{1 - \frac{2.61 \times 41.6E6}{28 \times 1000 \times 330^2}} \right) = 0.0022$$

0.0022 < 0.00333 → 0.0033 is used in this region

The moment values are less than 26 kN.m in the other regions; thus, a minimum flexural steel ratio is used in all regions for both Y and X directions.

Main flexural steel → As = 0.00333 * 1000 * 230 = 765.9 mm²

Shrinkage steel = (0.003/2) * 1000 * 300 = 450 mm²

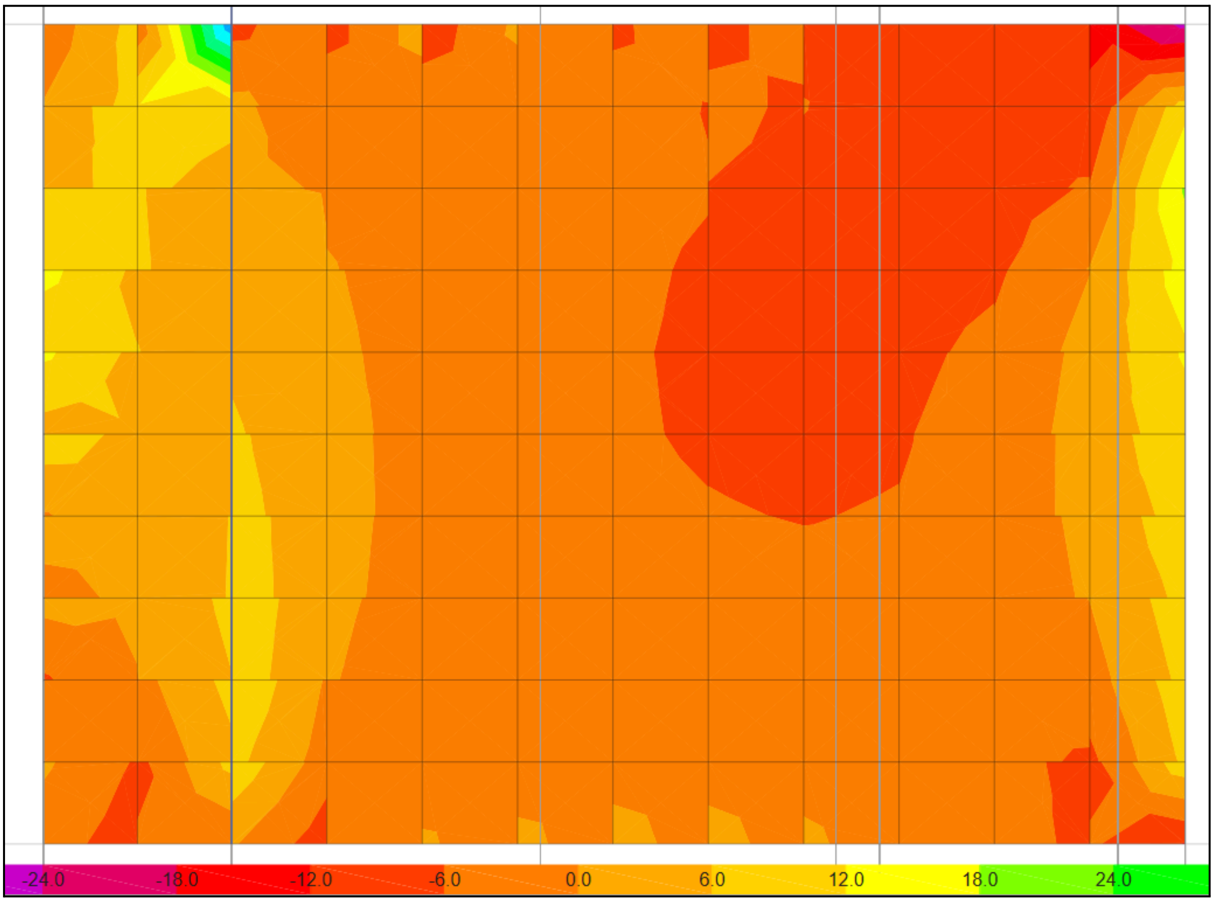


Figure 3.95: Moment contour map (m11) for the selected wall

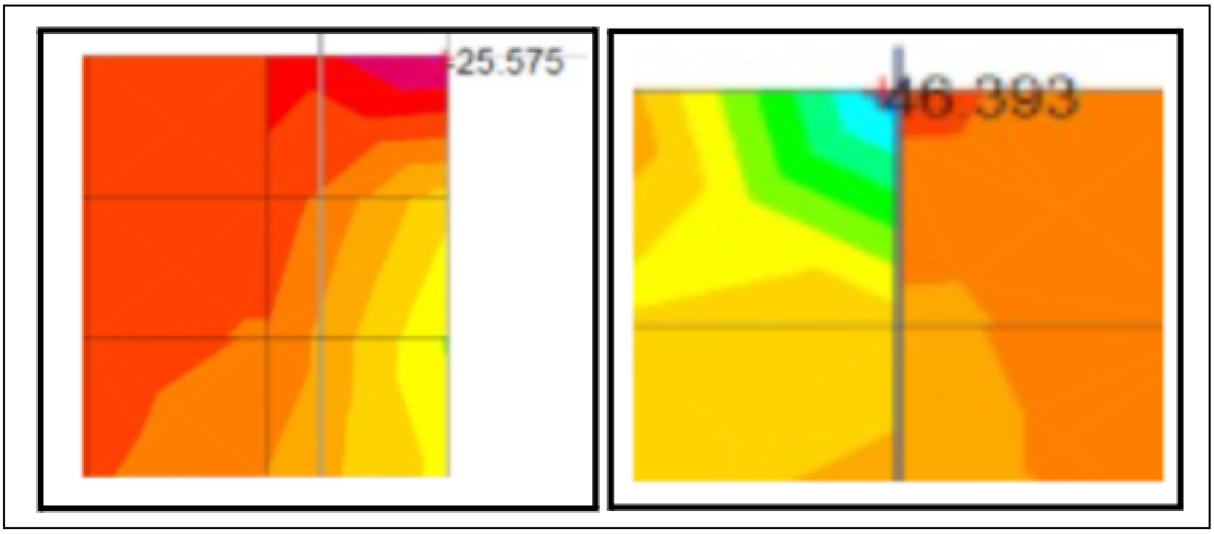


Figure 3.96: Moment values (m11) in edges (Corners and sides)

Note 1: m11=46.393 is at the column. Figure 3.97 shows the moment values at the wall (at the right and left sides of this column).

Note 2: m11=25.575 is at the extreme edge. Figure3.98 show the value of moment at distance 30 cm far from edge (at face of support)

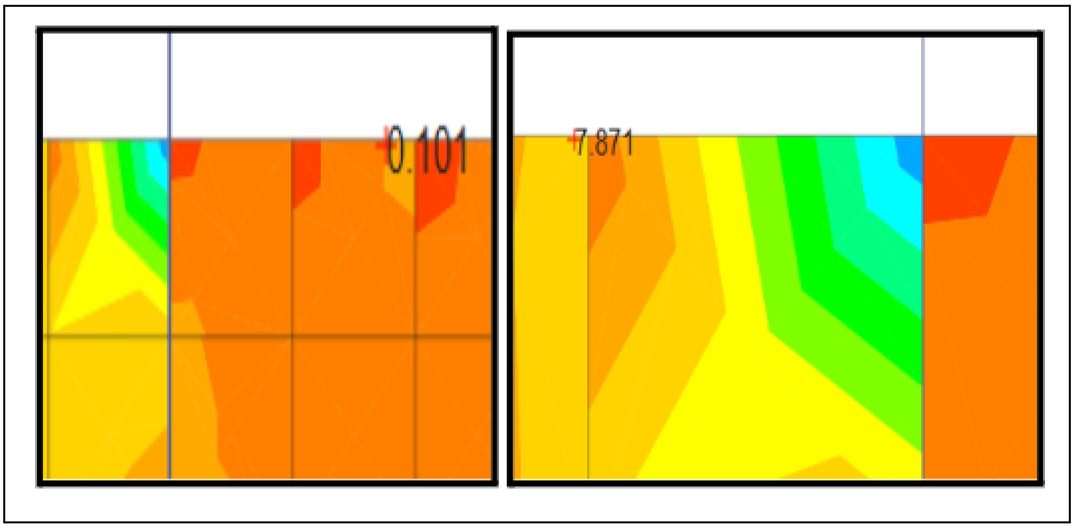


Figure 3.97: Moment values (m11) near the column

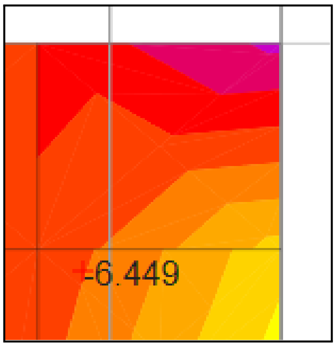


Figure 3.98: Moment values (m11) at distance 30 cm from edges (At face of support)

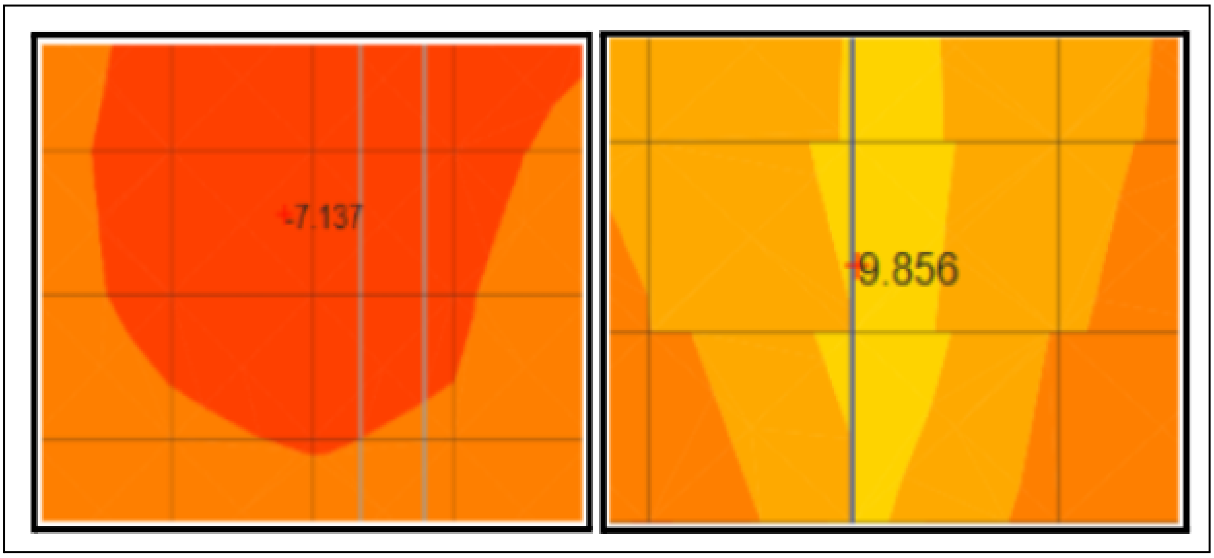


Figure 3.99: Moment values (m11) in central regions

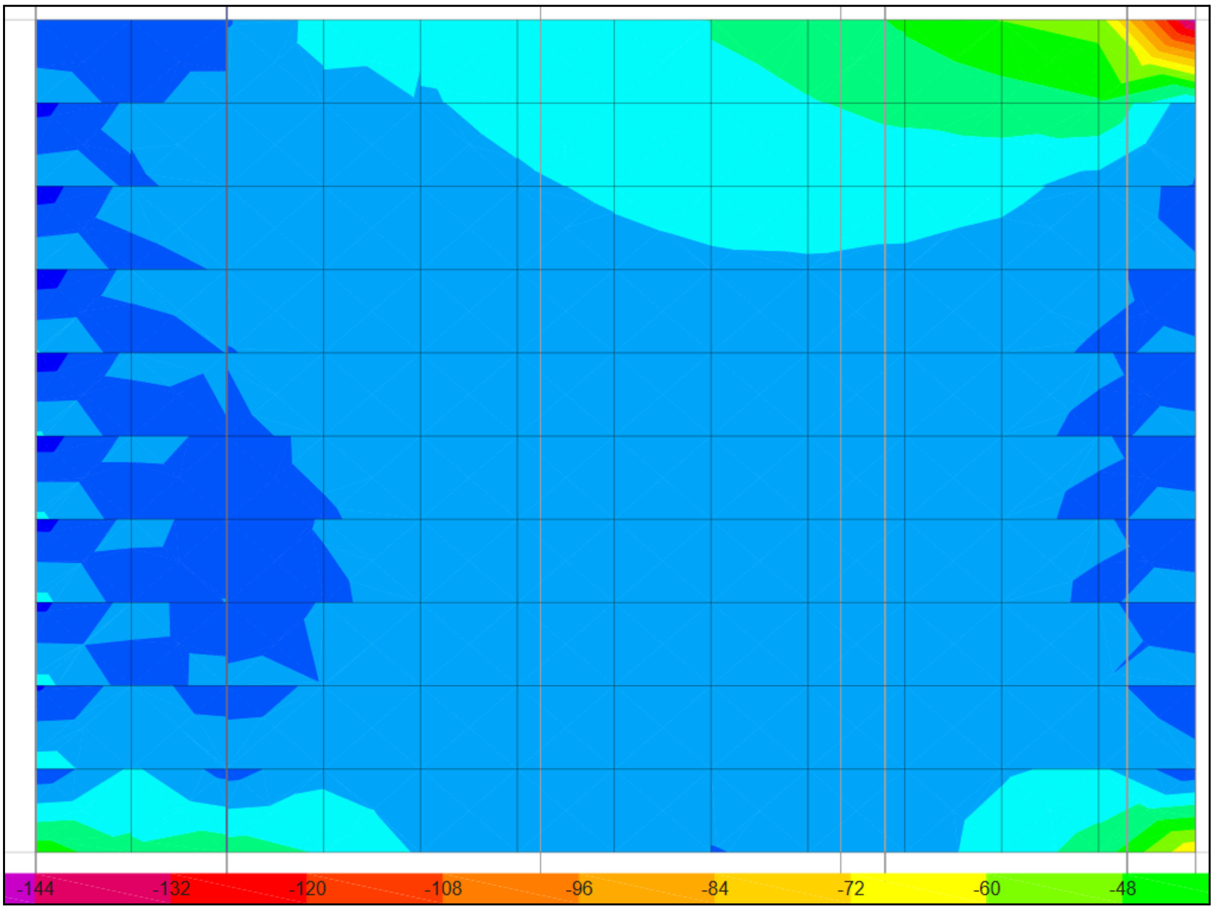


Figure 3.100: Moment contour map (m22) for the selected wall

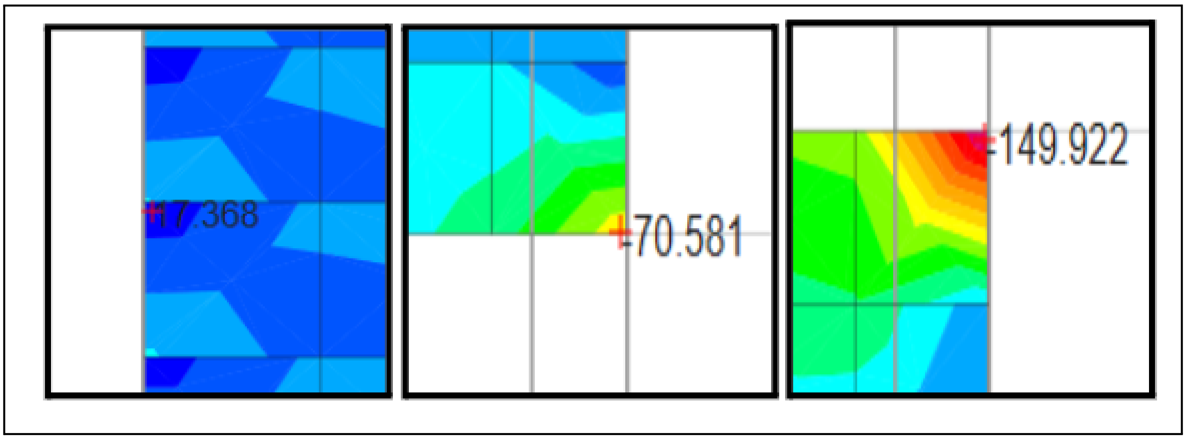


Figure 3.101: Moment values (m22) in edges (Corners and sides)

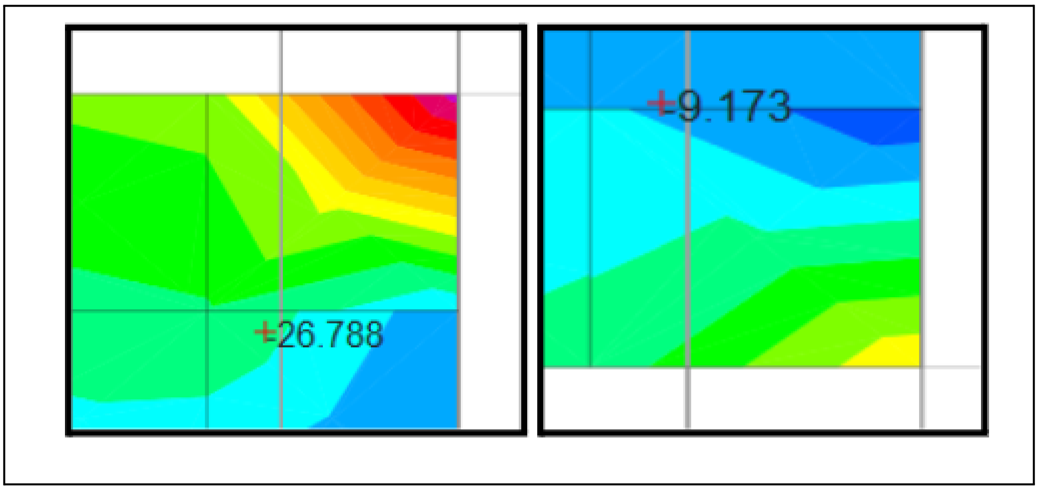


Figure 3.102: Moment values (m22) at distance 30 cm from edges (At face of support)

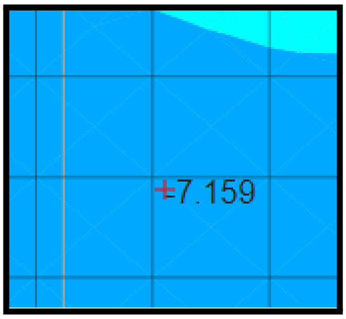


Figure 3.103: Moment values (m22) in central regions

2. **Shear Check and Design:**

Vu=45 kN (from ETABS)

(at distance d=230mm from face of support)

Figures 3.104-3.111 below which show shear values for both X and Y direction**.**

$\varnothing$Vc (no shear reinforcement) = $\varnothing \times (0.66 \times \lambda s \times \lambda \times \rho_w^{1/3} \times \sqrt{f`c} + \frac{Nu}{6\,Ag}) \times b_w \times d$

$= 0.75 \times (0.66 \times 1 \times 1 \times 0.0043^{1/3} \times \sqrt{28} - \frac{7.5}{6\times1000\times300}) \times 1000 \times 230/1000 = 98$ kN

Where: $\rho_w = (300/230) \times 0.0033 = 0.0043$

λ and $\lambda s = 1$

$\frac{Nu}{6\,Ag}$ is tension → negative sign. Nu equals(30*0.5*0.5)= 7.5KN (see **Figure 3.114**) in design for tension).

d= 300-70 = 230mm

∅Vc > Vu ✔ → no need for shear reinforcement

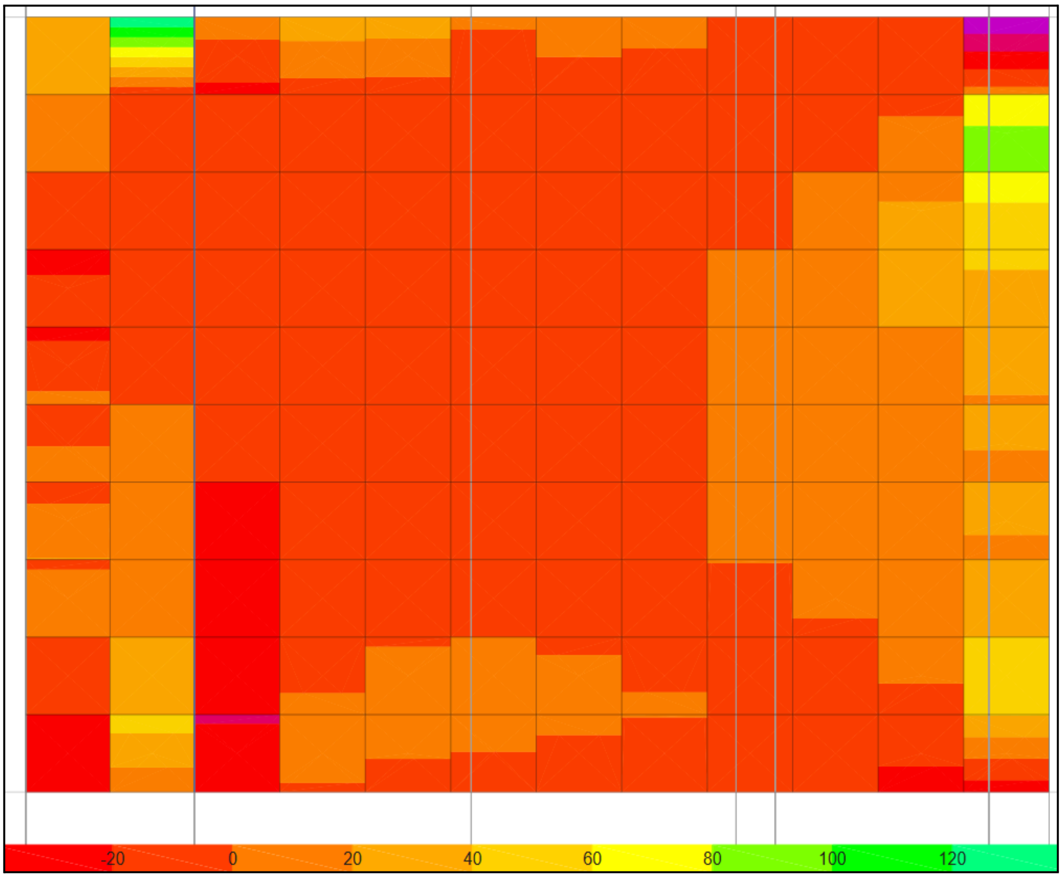


Figure 3.104: Shear contour map (V13) for the selected wall

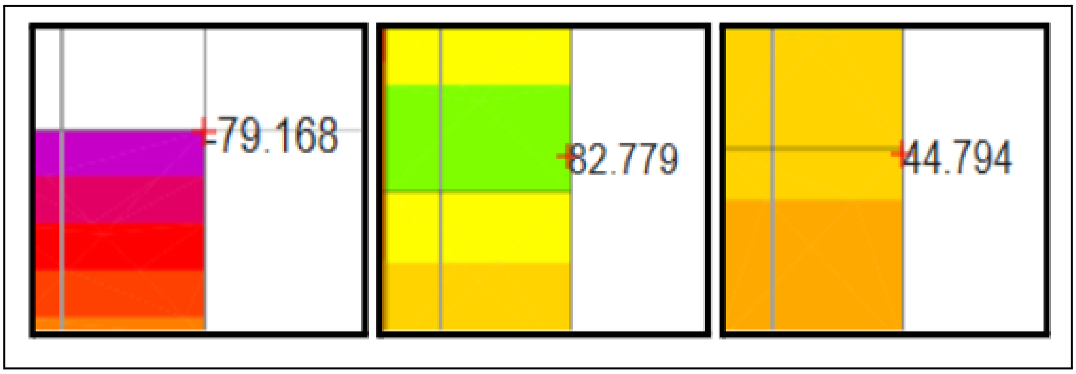


Figure 3.105: Shear values (V13) in edges (Corners and sides)

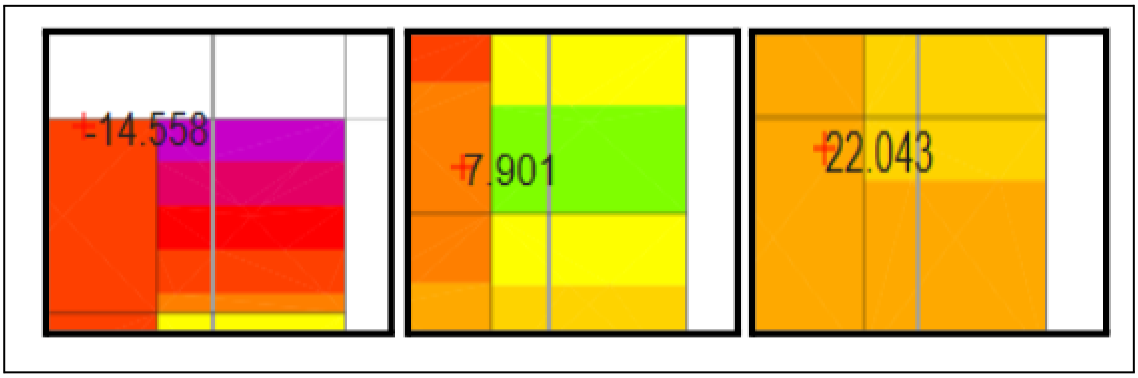


Figure 3.106: Shear values (V13) at distance d= 230mm from face of support

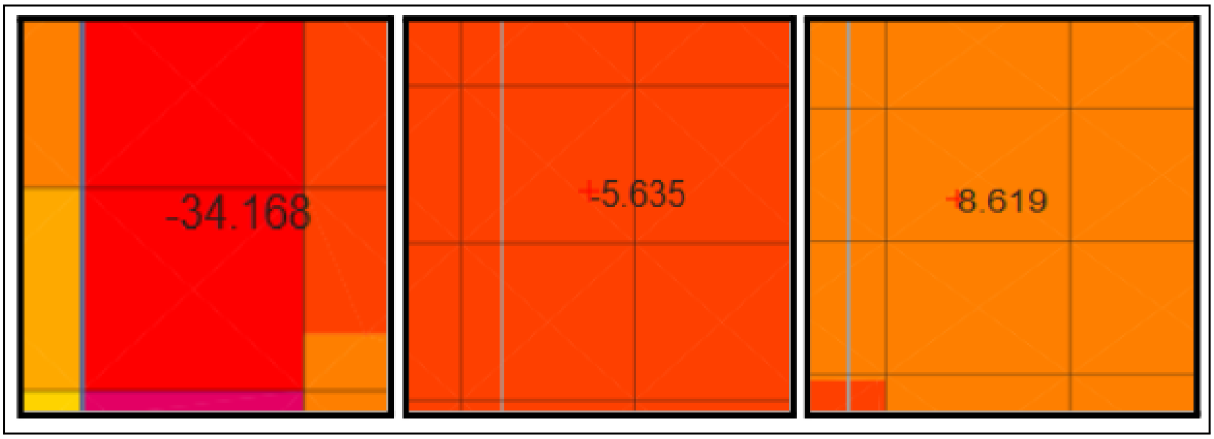


Figure 3.107: Shear values (V13) in central regions

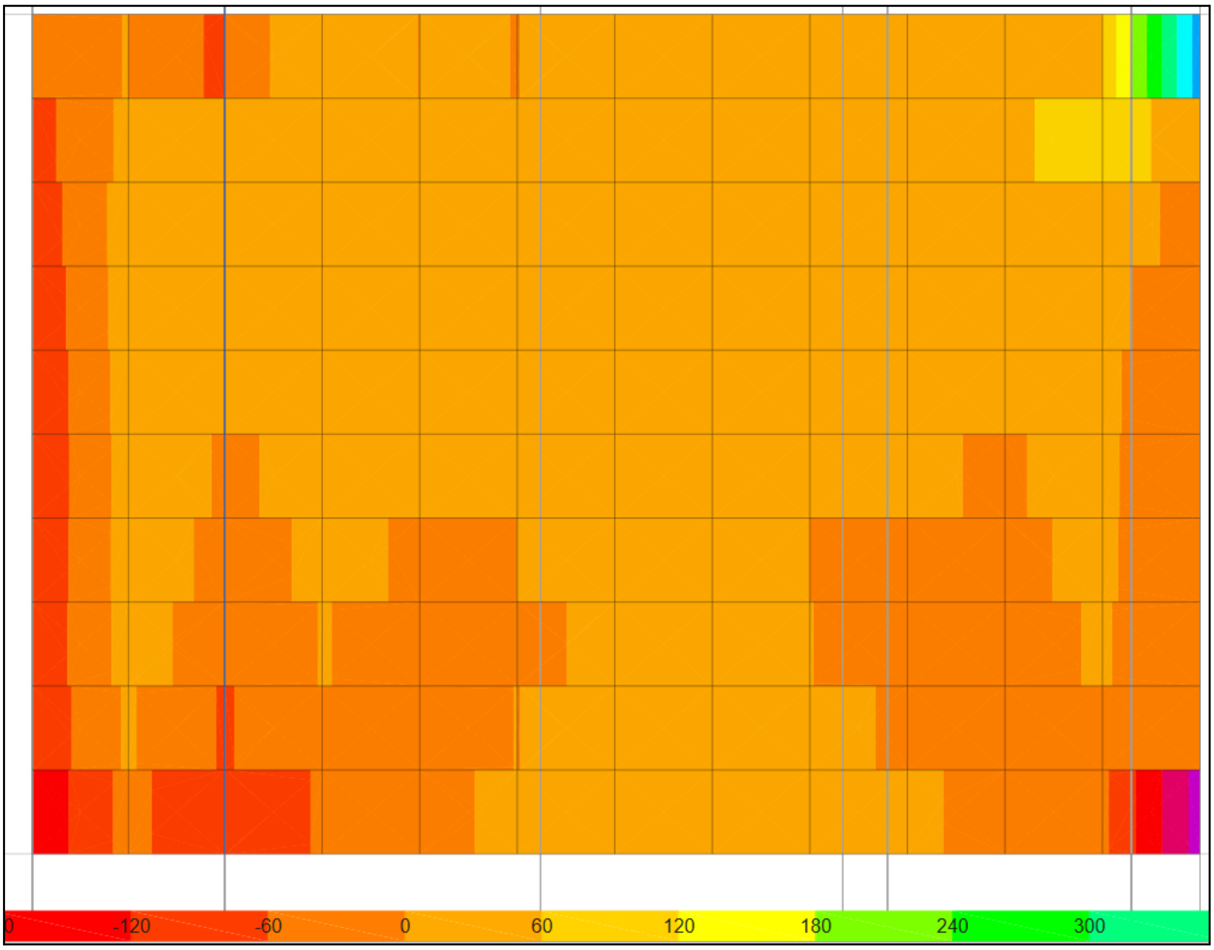


Figure 3.108: Shear contour map (V23) for the selected wall

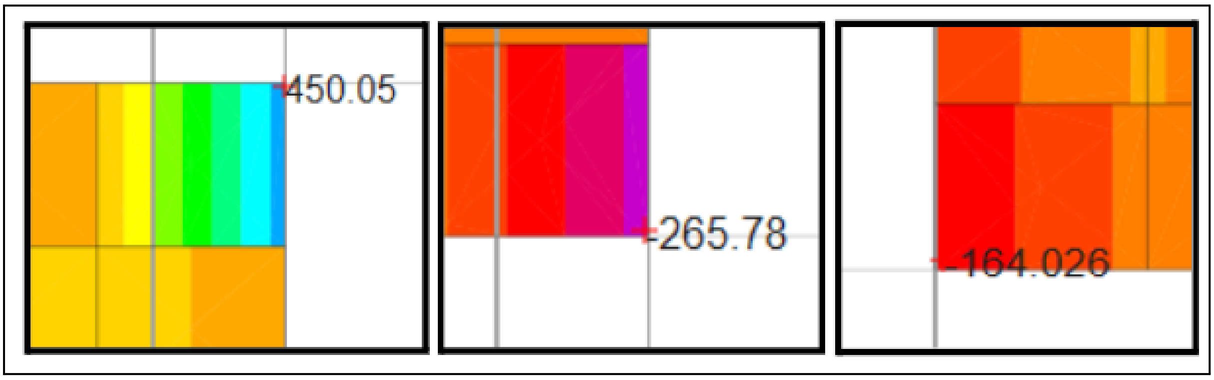


Figure 3.109: Shear values (V23) in edges (Corners and sides)

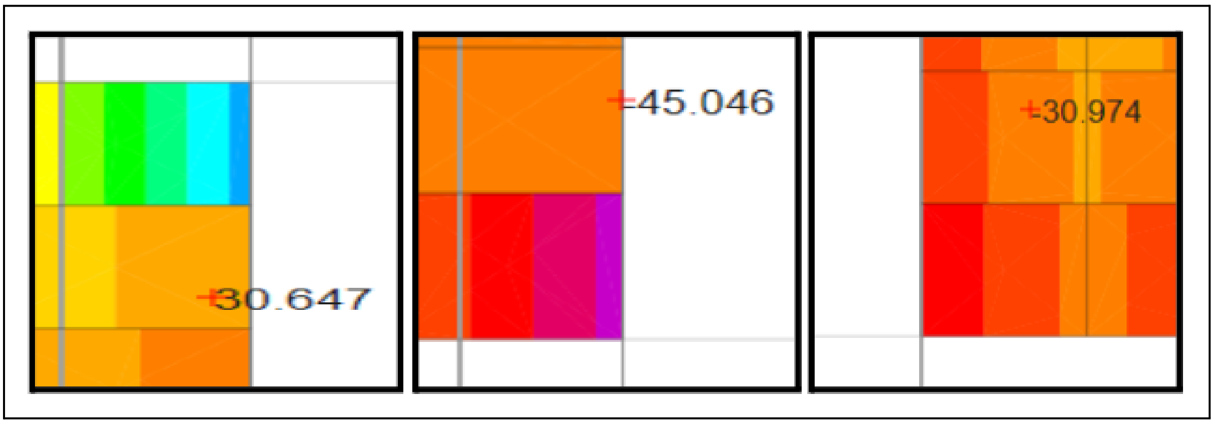


Figure 3.110: Shear values (V23) at distance d=230mm from face of support

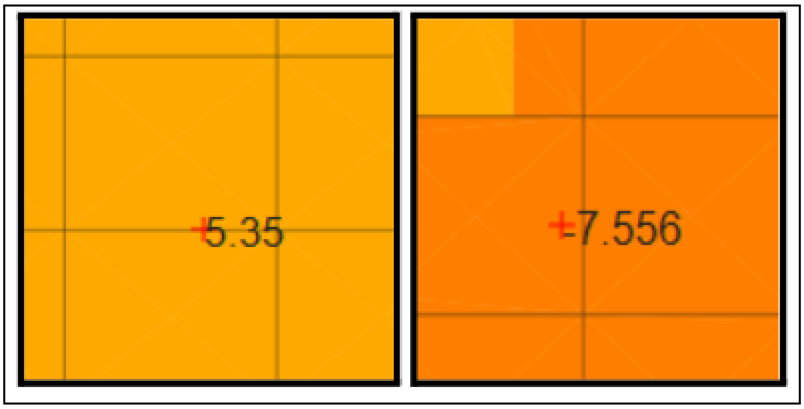


Figure 3.111: Shear values (V23) in central regions

3. **Tension Design:**

Figures 3.112 to **3.115** show the tensile forces acting on the wall in the horizontal direction. The wall is not too high to be divided into many parts for economical reasons, and the values of tension in the middle do not differ too much from the edge. The value shown is considered in the design along total height and width.

As $= \frac{29.6 \, x \, 2.3 \, x \, 1000}{0.9 \, x \, 420} = 180 \text{ mm}^2$.

Tension values in Y direction are presented to determine the critical value of tension. This value is required for calculating shear capacity (V_c) to confirm shear check.

Maximum f11= 29.6 kN/m^2. where f11=142 is located in the column.

Maximum f22= 25 kN.

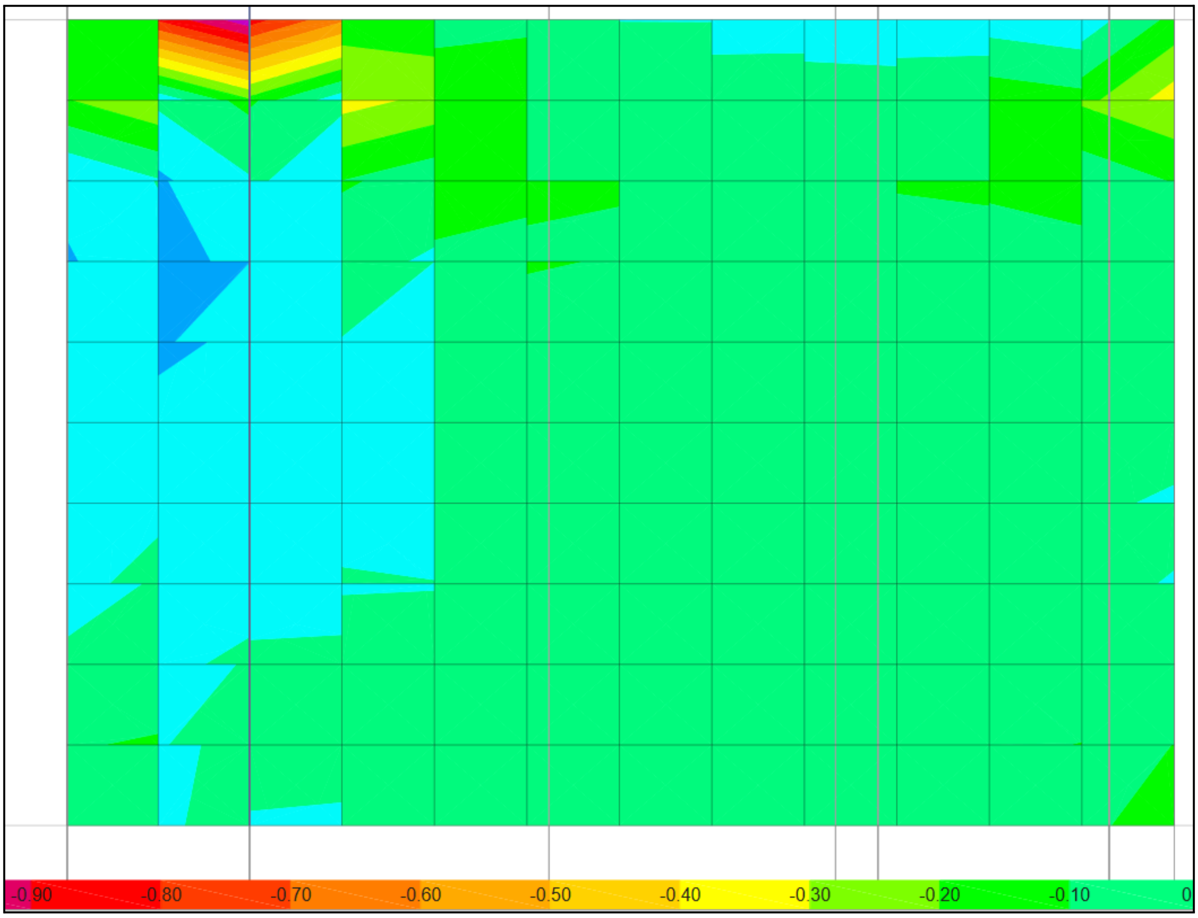


Figure 3.112: Tension contour map f11

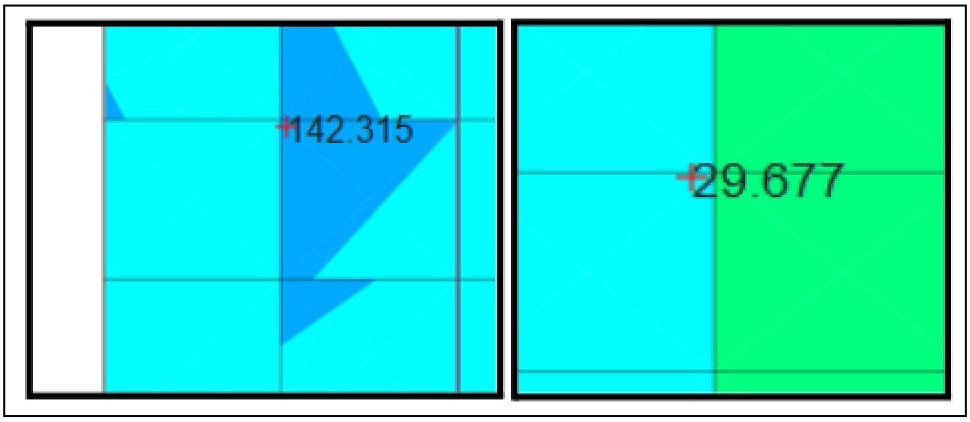


Figure 3.113: Maximum tension force f11

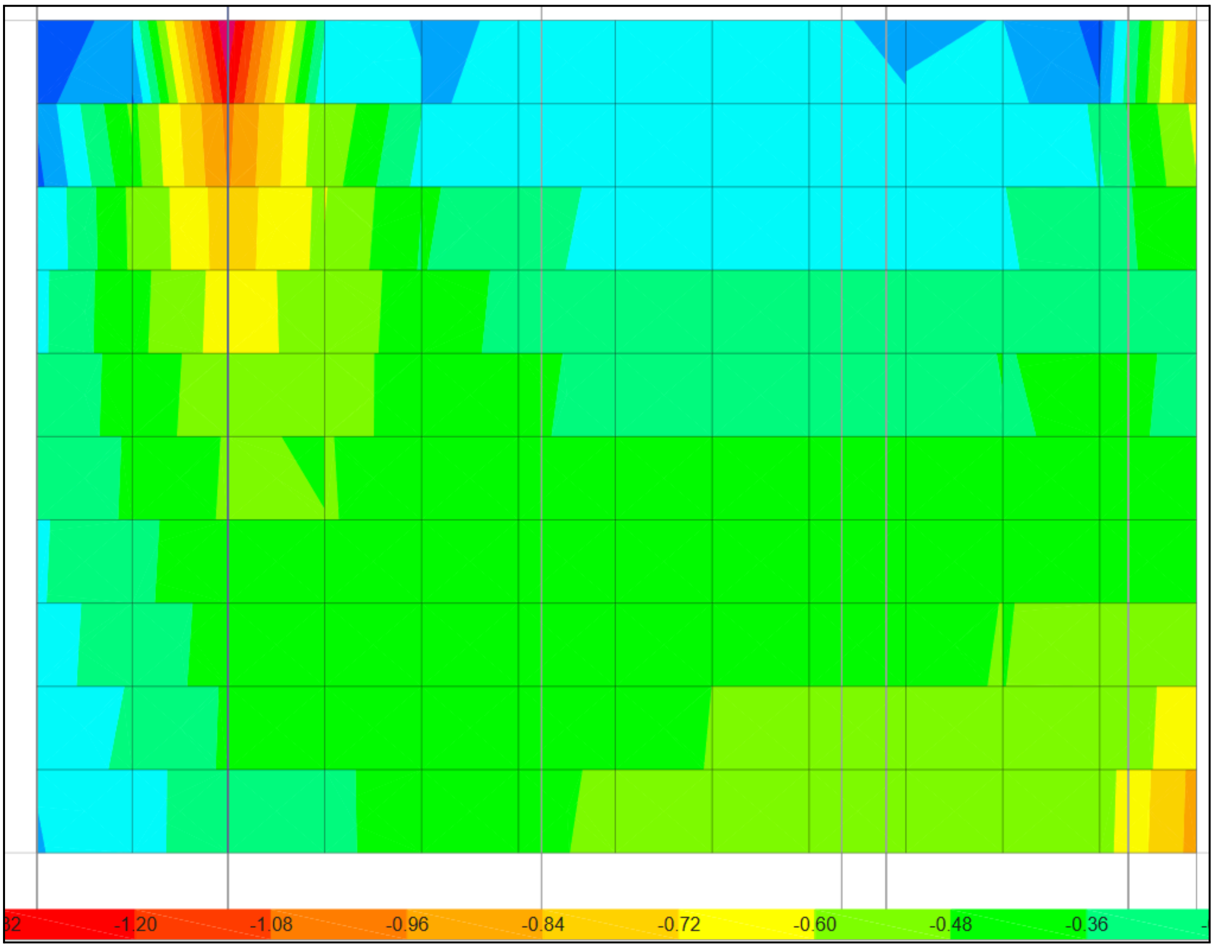


Figure 3.114: Tension contour map f22

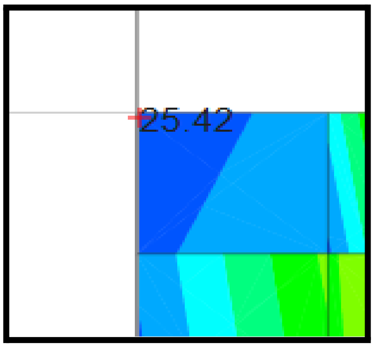


Figure 3.115: Maximum tension force f22

3.14.4 Design of water tank walls

Table 3.24 shows the total reinforcement required for all walls.

Table 3.24: Reinforcement for walls of the water tank

Direction		**X- Direction**		**Y- Direction**	
Face		**outer**	**inner**	**outer**	**inner**
W1	**A_{st}**	851.08	851.08	759.81	759.81
	Bars	1Ø14/150	1Ø14/150	1Ø14/150	1Ø14/150
W2	**A_{st}**	760.51	760.51	759.81	759.81
	Bars	1Ø14/150	1Ø14/150	1Ø14/150	1Ø14/150
W3	**A_{st}**	761.03	761.03	759.81	759.81
	Bars	1Ø14/150	1Ø14/150	1Ø14/150	1Ø14/150
W4	**A_{st}**	835.87	835.87	759.81	759.81
	Bars	1Ø14/150	1Ø14/150	1Ø14/150	1Ø14/150
W5	**A_{st}**	760.11	760.11	759.81	759.81
	Bars	1Ø14/150	1Ø14/150	1Ø14/150	1Ø14/150
W6	**A_{st}**	760.72	760.72	759.81	759.81
	Bars	1Ø14/150	1Ø14/150	1Ø14/150	1Ø14/150
W7	**A_{st}**	820.66	820.66	759.81	759.81
	Bars	1Ø14/150	1Ø14/150	1Ø14/150	1Ø14/150

3.14.5 Design of water tank base

A water tank base is within the mat foundation in the basement. **Figure 3.116** shows the two water tanks base layout.

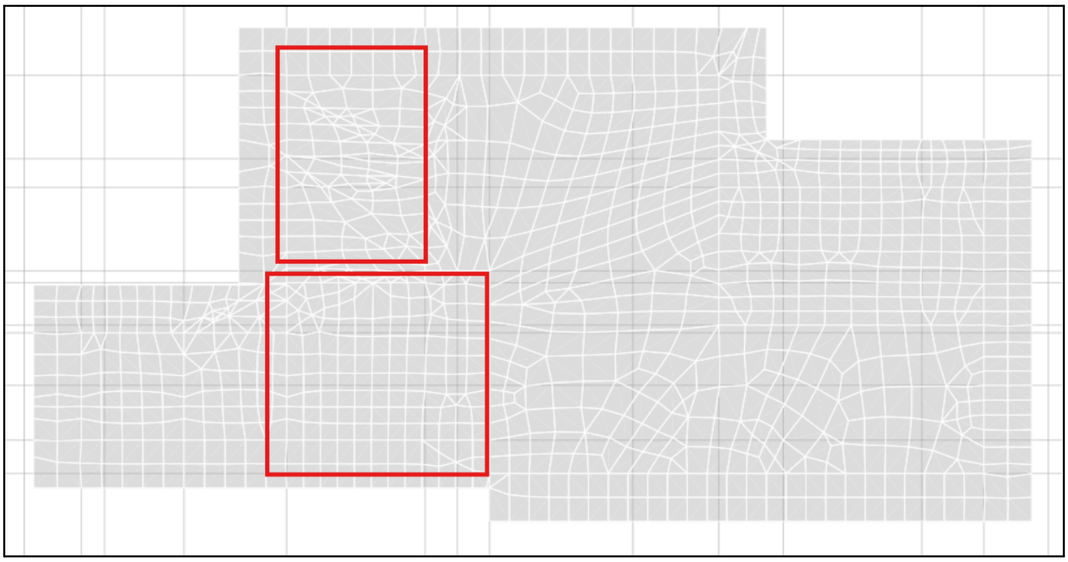


Figure 3.116: Water tank base

3.14.5.1 Flexure design

- For edge of the base:

As previously calculated, the value of S_d was determined to be 1.42, and the moment value at the edge of the water tank base was 90.7 kN·m. By multiplying this factor with the moment value, a new moment value of 128.8 kN·m was obtained. This new moment value was used to conduct the steel ratio calculation, as shown below.

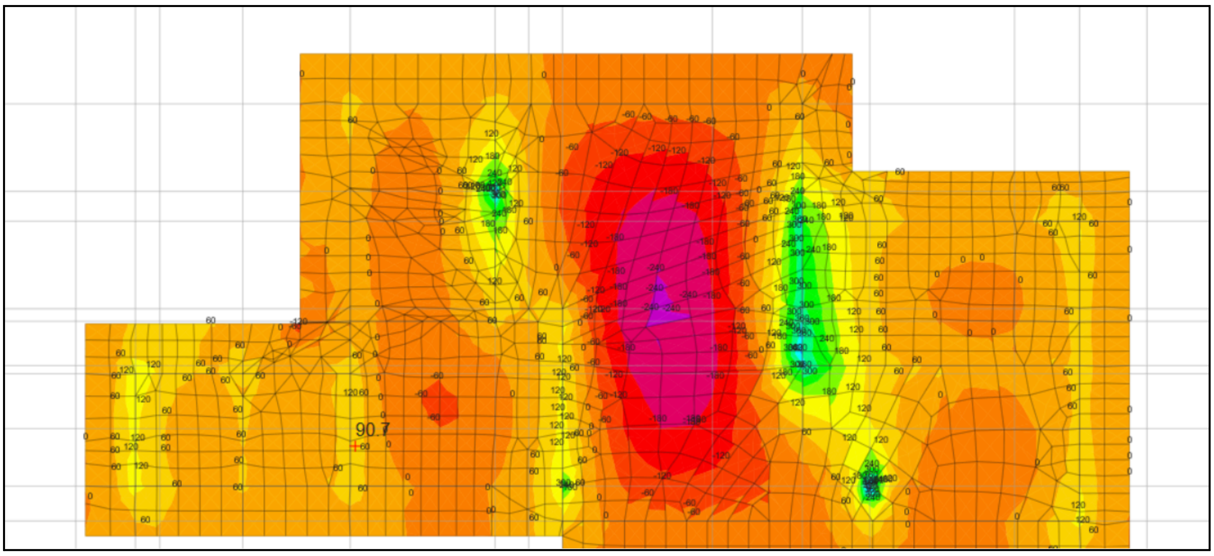


Figure 3.117: M11 value at the edge of the water tank

$$\rho = \frac{0.85 \times 28}{420}\left(1 - \sqrt{1 - \frac{2.61 \times 128.88E6}{28 \times 1000 \times 720^2}}\right) = 0.00066.$$

0.00066 < 0.0018 → 0.0018 is used in this region.

As= 0.0018*1000*800= 1440 mm^2/m.

Take 1ø18/150 mm as reinforcement for the edge of the water tank base.

- For center of the base:

As shown in **Figure 3.118,** the value of moment at center of the base, thickness of the center also is the same so the steel ratio of the center of the water tank is shown in the equation below, the factor S_d of the center is the same as the edge value which is 1.42.

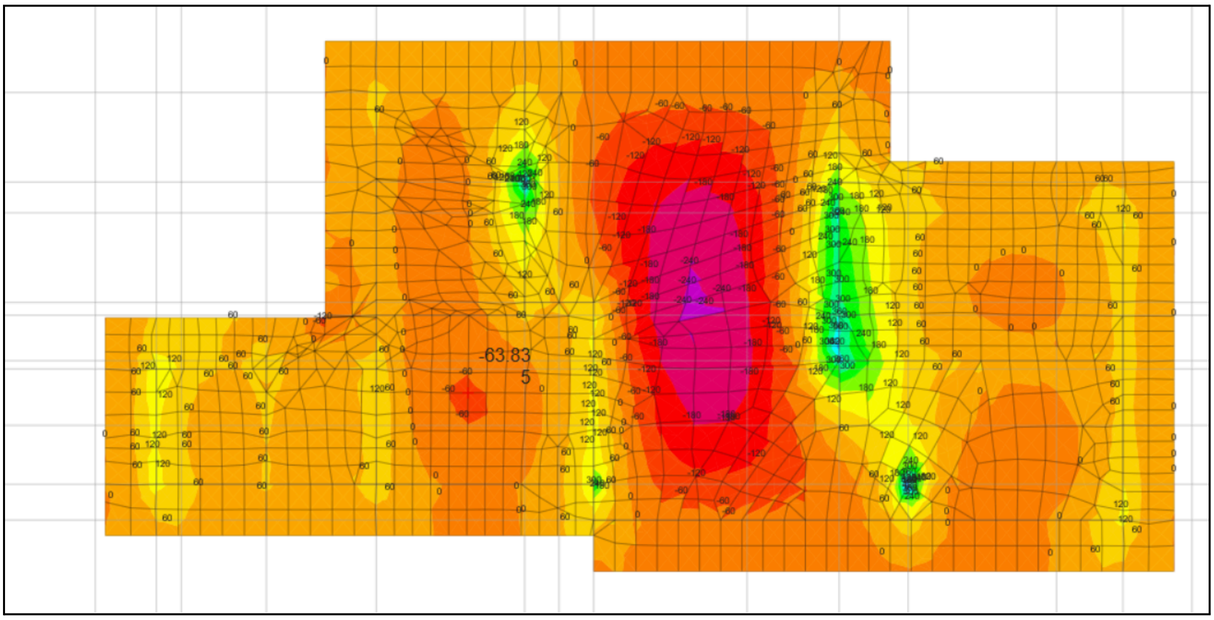


Figure 3.118: M11 values at the center of the water tank

$$\rho = \frac{0.85 \times 28}{420} \left(1 - \sqrt{1 - \frac{2.61 \times 90.56E6}{28 \times 1000 \times 720^2}} \right) = 0.000463.$$

0.000463 < 0.0018 → 0.0018 is used in this region.

As= 0.0018*1000*800= 1440 mm^2/m.

Center reinforcement is taken the same as the edge reinforcement which is ø18/150 mm.

3.14.5.2 Design for tension

Axial forces are compression or too small tension values in the X-direction. In contrast, tension forces in the Y-direction are high and can not be neglected. **Figure 3.119** and **Figure 3.120** show the maximum tension forces taken from ETABS followed by hand calculations for tension reinforcement and design.

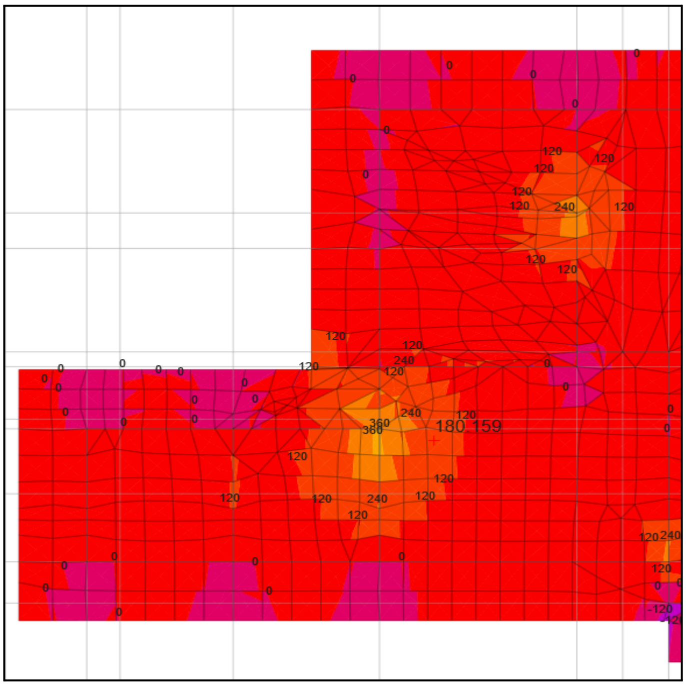


Figure 3.119: Maximum tension force f22 near the edge of the wall

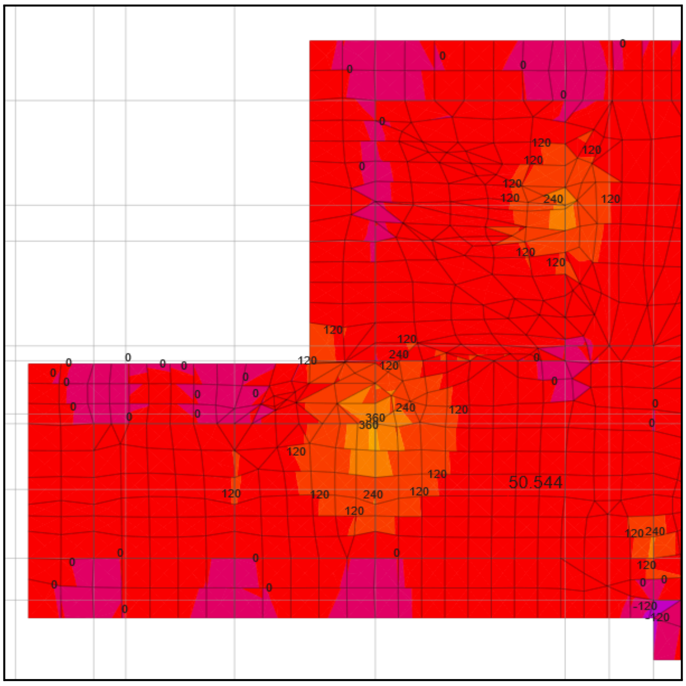


Figure 3.120: Maximum tension force f22 in the middle

Tension values in Y-direction are presented to determine the critical value of tension. This value is required for calculating shear capacity (V_c) to confirm shear check.

Maximum f11 is compression.

Maximum f22= 180 kN near edges (along the length of the wall).

Maximum f22= 50 kN in the middle region.

$A_s = \frac{180\ x\ 2.3\ x\ 1000}{0.9\ x\ 420}$ = 1095 mm^2. Near edges.

$A_s = \frac{50\ x\ 2.3\ x\ 1000}{0.9\ x\ 420}$ = 304 mm^2. Middle region.

3.14.5.3 Final reinforcement and bars details

X-direction:

1. **Edges:**

   A_s = 1269 mm² → 1ø18/150 mm **Top face.**

   Shrinkage steel = 1440/2 = 720 mm² → 1ø12/150 mm **Bottom face.**

2. **Middle:**

   A_s = 1269 mm² → 1ø18/150 mm **Bottom face.**

   Shrinkage steel = 1440/2 = 720 mm² → 1ø12/150 mm **Top face.**

Y-direction:

1. **Edges:**

   (left edge of the septic tank, three meters width)

   A_s =1440 + (1095/2) = 1987.5 mm² → 1ø20/150 mm **Top face.**

   Shrinkage steel = 1440/2 = 720 mm² → 1ø12/150 mm **Bottom face.**

   Note: (1095/2)= 547.5 mm² is the area required for tension forces on one face. This area is provided by shrinkage steel.

2. **Middle:**

   A_s = 1440 mm² → 1ø18/150 mm **Bottom face.**

   Shrinkage steel = 1440/2 = 720 mm² → 1ø12/150 mm **Top face.**

PART II

SEISMIC REDESIGN

CHAPTER 4: SEISMIC CONSIDERATIONS

4.1 General

This chapter marks the beginning of **Part II** of the project, which builds upon the foundations established in **Part I**. The first part focused on the **structural design of a residential complex in Nablus under gravity loads only**, including dead and live loads, in accordance with the relevant provisions of **ACI 318-19** (*Building Code Requirements for Structural Concrete*) and **ASCE 7-16** (*Minimum Design Loads for Buildings and Other Structures*). In contrast, **Part II** shifts the focus to the **seismic design of the same structure**, addressing the necessary modifications and considerations required to ensure resistance against seismic forces while continuing to follow the same design codes and standards to maintain consistency throughout the project.

While the reasoning for dividing the project into two parts was previously mentioned, it is worth restating here: the separation allows for a **clear and direct comparison** between the structural behavior and requirements under gravity-only design and those under combined gravity and seismic loadings. This comparative approach enhances understanding of the additional demands that seismic design imposes on structural systems.

The chapter at hand establishes the foundation for the project's seismic design. It opens with two general sections: one introduces the nature of earthquakes and their effects on reinforced concrete structures, and the other outlines the basic principles of seismic design. These sections are intended to support readers who may be less familiar with seismic concepts and to provide a smooth transition into the technical discussions. Experienced readers may choose to skim or skip them, as the core of the chapter lies in the subsequent sections, which present the specific seismic design criteria adopted for this project—including the selection of the seismic force-resisting system, key design parameters, and the coefficients that directly guide the redesign and structural modifications in the upcoming stages.

4.2 Understanding earthquake effects on structural performance

The Earth is composed of several distinct layers: the crust, mantle, and core, with the core often divided into outer and inner parts. The Earth’s crust, which forms the Earth’s outermost layer, is not a single solid piece but is made up of several large sections called tectonic plates, which are constantly in motion, albeit slowly. These plates move either toward each other (converging), away from each other (diverging), or slide past one another in regions known as fault zones. Although this movement is generally slow and small in scale, when it becomes strong and occurs suddenly, the energy stored in the plates is released and transmitted through the Earth in the form of seismic waves. These waves cause ground motion at distances far from the epicenter—the point on the Earth's surface directly above the earthquake's origin. This results in what is called an earthquake. Regions adjacent to active fault lines are most prone to experience earthquakes due to the heightened tectonic activity in these areas.

Earthquakes cause random horizontal and vertical movements of the Earth's surface, which impact structures. As the ground shakes, the structure experiences dynamic forces in multiple directions. Due to inertia, structures tend to remain in their original positions, resisting the motion. As the base of a structure moves with the ground, the superstructure lags behind due to inertia. This interaction induces vibrations or oscillations, leading to internal stresses (of main focus, **base shear**) and lateral displacements. These internal forces, if not well resisted by the structural elements, can cause severe structural damage leading to collapse and undesirable consequences.

Figure 4.1 illustrates this concept: as the ground moves laterally, the structure resists that movement, resulting in a distribution of inertial forces throughout the frame. The schematics in **Figures 4.2** and **4.3** further highlight how a structure responds to seismic excitation by illustrating the distribution of lateral shear forces and the resulting internal forces across the building's floors. These figures were adapted from *Design of Concrete Structures* (Darwin, Dolan, & Nilson, 2016).

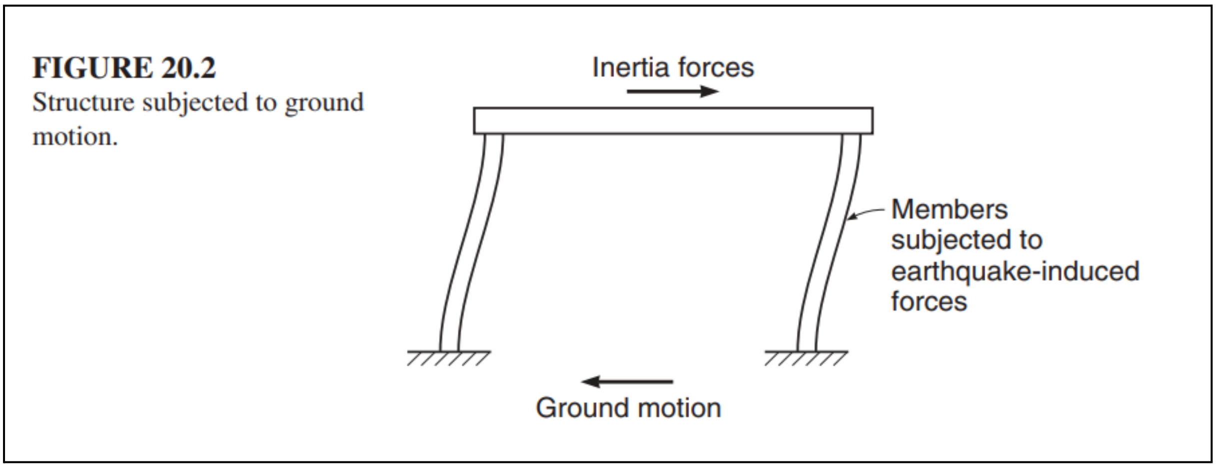


Figure 4.1: Ground motion-structural behaviour interaction

FIGURE 20.8
Frame subjected to lateral loading: (*a*) deflected shape; (*b*) moments acting on beam-column joint; (*c*) deflected shape and forces acting on a beam; and (*d*) forces acting on faces of a joint due to lateral load.

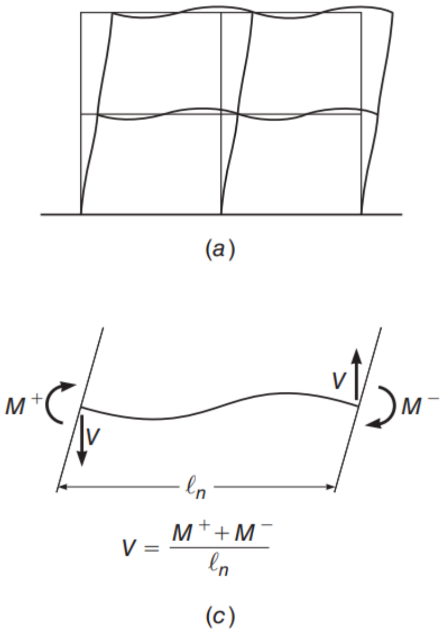
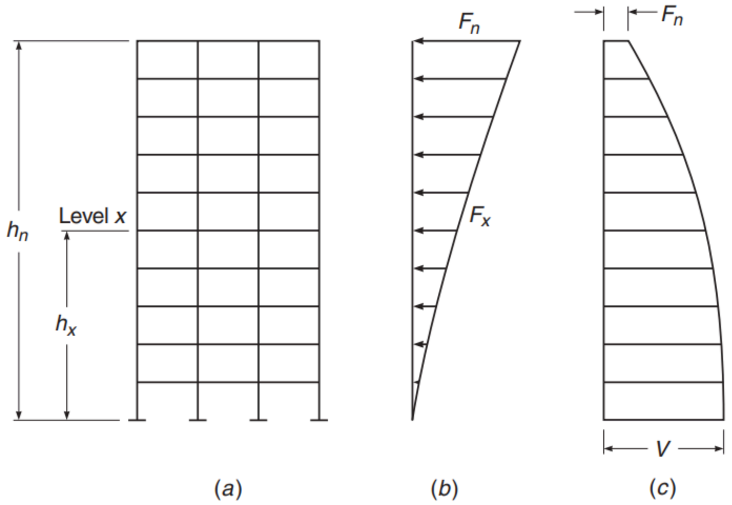


Figure 4.2: Induced internal stresses due to lateral loading

FIGURE 20.11
Forces based on ASCE/SEI 7 (Ref. 20.2) equivalent lateral force procedure: (*a*) structure; (*b*) distribution of lateral forces over height; and (*c*) story shears.



F_x = lateral seismic force induced at level x
w_x, w_i = portion of W at level x and level i, respectively
h_x, h_i = height to level x and level i, respectively
k = exponent related to structural period = 1 for $T \leq 0.5$ sec and = 2 for $T \geq 2.5$ sec. For $0.5 < T < 2.5$, k is determined by linear interpolation or set to a value of 2

Figure 4.3: Induced lateral shear forces in seismic events

As a rule, concrete structures respond to earthquakes based on how seismic energy interacts with their physical characteristics—primarily their mass and stiffness. These two factors define the **structure's natural period**, which represents how it vibrates when subjected to dynamic loading. Other structural characteristics, such as height, mass distribution, geometric irregularities, and damping capacity (the ability to dissipate energy), also influence seismic behavior, but the natural period remains one of the most critical parameters. Table 4.1 presents the factors affecting seismic forces, along with their explanations, in addition to other relevant factors.

Table 4.1: Factors affecting seismic forces

Factors Affecting Seismic Forces	
Factor	**Effect**
Structural System Type	Different systems (e.g., shear walls, moment-resisting frames) resist seismic forces in distinct ways.
Building Configuration (Structural Irregularities)	Irregular shapes or layouts can cause torsional effects, localized failures, and uneven force distribution.
Mass	More mass means larger inertial forces (Newton's law).
Stiffness	A stiffer structure resists more lateral displacement and experiences higher forces during seismic loading.
Ductility and Energy Dissipation	Affects how well the structure deforms without collapse and survives seismic events by dissipating energy.
Damping Ratio	Damping capacity, which varies depending on materials (e.g., concrete, steel), mitigates seismic response by dissipating energy.
Building Height	Taller buildings generally have longer natural periods, making them more affected by lower-frequency ground motions. In contrast to structures with short natural periods.
Soil Conditions	Soft soils amplify motion more than rock.
Proximity to Epicenter or Fault	Near-fault effects can be more intense.

The response of a structure is not determined by its properties alone, but also by the nature of the earthquake itself. Ground motion characteristics such as intensity, frequency content, acceleration, and duration play a major role—in addition to other factors related to proximity to the epicenter and soil conditions. Among these, the dominant periods of the ground motion—those frequencies at which the energy content is most significant—are especially important. When there is an alignment between the structure's natural period and the dominant periods of the earthquake ground motion, resonance can occur. This amplifies the structural response and may lead to catastrophic failure.

To better understand this, ASCE standard divides structures into two categories based on their natural period: short-period structures and long-period structures. Short-period structures—typically low-rise and stiff—respond more strongly to high-frequency components of ground motion, experiencing greater acceleration but relatively small displacements. In contrast, long-period structures—such as tall, flexible buildings—are more susceptible to low-frequency content, resulting in larger displacements but generally lower acceleration forces. This distinction underlines a fundamental aspect of seismic design: **different structures respond differently to the same earthquake**

To relate ground motion to structural behavior, seismic design standards introduce specific terms and definitions that capture both the characteristics of ground shaking and the dynamic properties of structures, particularly the natural period. These concepts are discussed in more detail in the subsequent sections. At this stage, it is important to recognize that a fundamental understanding of the interaction between ground motion and structural response lies at the core of all seismic analysis and design decisions.

4.3 Introduction to seismic design

4.3.1 Concepts and definitions

Seismic design is the discipline that addresses the challenge of ensuring that buildings can withstand the dynamic forces produced by earthquakes. These forces, which are lateral and time-dependent, arise from ground shaking and vary greatly in intensity and direction. Unlike gravity design, which deals primarily with predictable vertical loads, seismic design focuses on how structures respond to these unpredictable lateral forces, ensuring safety and preventing collapse.

At its core, seismic design aims to understand how seismic energy interacts with structures. Earthquake-induced ground motion generates inertial forces, leading to vibrations, internal stresses, and lateral displacements. The objective is to anticipate these effects and design systems that resist or dissipate these forces (ASCE 7-16).

The design philosophy—rooted in the concept of life safety—accepts that some damage may occur under strong shaking, but it ensures that the structure will not collapse, protecting its occupants (ACI 318-19, ASCE 7-16). In this context, seismic design is not meant to prevent all damage, but rather to safeguard human life and ensure essential structural integrity even under extreme events.

While seismic design aims to ensure life safety during severe earthquakes, it does not attempt to guard against the most extreme conceivable events. Instead, codes adopt a probabilistic approach to characterize earthquake hazard. A commonly used reference is the **Maximum Considered Earthquake (MCE)**, which corresponds to ground motion with a **2% probability of exceedance in 50 years**, as defined in ASCE 7-16. This means that there is a small chance the actual ground motion could exceed the design level within a typical building lifespan. Such a balance is intentional, as designing for even rarer events would often be impractical and economically unjustifiable.

4.3.2 Seismic design codes

To standardize seismic design practices, building codes like **ASCE 7-16** and **ACI 318-19** provide guidelines on performance objectives, design assumptions, and acceptable methods of analysis. These codes reflect decades of research, offering tools that engineers use to design structures capable of resisting seismic demands. The goal is to provide a consistent and reliable framework for anticipating earthquake effects and ensuring the building can resist lateral forces within acceptable safety margins (ASCE 7-16).

4.3.3 Performance objectives

While all seismic designs aim to prevent structural collapse under extreme shaking, they differ in how much damage is acceptable and what level of functionality is expected afterward. Depending on the building's importance and intended use, the design may aim to merely ensure life safety, or it may require the structure to remain operational with minimal repairs. These varying objectives influence the design approach and are guided by code-defined performance expectations.

4.3.4 Methodology of seismic design

Seismic design begins with analyzing how a structure responds to dynamic loading. Various analysis methods have been developed to predict the structural response to earthquake forces. While they differ in complexity and the level of accuracy they offer, all share a common objective: to inform the design of effective seismic force-resisting systems (ASCE 7-16, Chapters 12–16). A detailed overview of these methods, along with the specific method chosen for this project, is provided in Section 4.7.

Thereafter, the design process incorporates structural mechanisms, such as shear walls or moment-resisting frames, to provide the necessary strength, stiffness, and deformation capacity to withstand the seismic forces estimated from the analysis. These components not only contribute to the structural integrity but also enable energy absorption and dissipation during an earthquake (ASCE 7-16, Chapter 12).

4.3.5 Key seismic design parameters

Before seismic design can begin, the effects of earthquakes must be translated into a language that engineers can work with. This is the role of seismic design parameters. These values capture both the external characteristics of ground motion and the internal characteristics of the structure—such as its intended use or vulnerability—and express them in measurable terms.

Together, these parameters define the seismic environment in which the structure must perform. They serve not as a checklist of inputs, but as the foundation of the design approach, guiding the way seismic demands are understood and managed.

From this foundation, the structural system responsible for resisting earthquake forces, known as **Seismic Force-Resisting System**, is selected. This is a pivotal design decision: different systems offer different ways of responding to seismic events, and each comes with its own set of numerical factors. These factors—unique to the chosen system—become the tools by which seismic forces are introduced into the analysis and design process.

In this way, the design process flows from understanding the seismic context, to selecting a suitable structural strategy, to applying the resulting coefficients that drive technical decisions. The sections that follow reflect this progression.

4.4 Seismic design parameters: Concepts

The interaction between earthquakes and structures is shaped by a set of parameters that translate natural ground motion into usable design values. These parameters are not just numbers, they represent how the ground is expected to behave during a major earthquake, and how a particular structure is likely to respond. Establishing them correctly is essential to a safe and effective seismic design. In modern engineering practice, these parameters are defined according to well-established seismic design standards, with **ASCE 7-16** being one of the most commonly referenced examples. The conceptual foundations of these parameters are introduced in this section. For convenience, **Table 4.2** at the end of the section provides a summary of the key seismic design parameters covered, including both primary variables and supporting values.

4.4.1 Spectral accelerations

In **ASCE 7-16**, seismic demand is characterized by two spectral accelerations: the **short-period spectral acceleration (Ss)** and the **long-period spectral acceleration (S_1)** (ASCE, 2017). These accelerations are expressed as fractions of gravitational acceleration and help quantify the expected earthquake demand on structures with different natural periods.

The spectral accelerations **Ss** and **S_1** correspond to the peak spectral accelerations of **single-degree-of-freedom systems** with fundamental periods of **0.2 seconds** and **1.0 second**, respectively, each assuming **5% critical damping** (ASCE, 2017). These values are adjusted for local site conditions using historical earthquake records and geological assessments to provide site-specific seismic hazard parameters. A damping ratio of 5% is commonly used in seismic design of reinforced concrete structures, representing a practical balance between structural stiffness and energy dissipation, as supported by ASCE 7-16, Chapter 11 (ASCE, 2017).

National seismic hazard maps —typically developed by government agencies such as the United States Geological Survey (USGS)—represent expected ground accelerations for structures with varying vibration characteristics. These mapped values of **Ss** and **S_1**, as adopted from USGS hazard data and incorporated into ASCE 7-16, generally correspond to ground motion with a **2% probability of exceedance in 50 years**, corresponding to a **2500-year return period.** This probabilistic basis is widely adopted in seismic design codes and standards, including *ASCE 7-16*, to ensure structures are designed to withstand rare but potentially damaging earthquakes.

4.4.2 Long-period Transition Period

To further interpret the seismic environment, ASCE 7-16 introduces the **long-period transition period (TL)**, which serves as a threshold distinguishing short-period structures from long-period ones. This parameter shapes the form of the design response spectrum and helps determine which segment of the curve governs a structure's expected behavior, reinforcing the contextual role of Ss and S_1 in defining seismic demand.

4.4.3 Site classification

Ground motion varies significantly depending on local site conditions. The nature of the soil beneath a structure influences how seismic waves are transmitted and amplified. Recognizing this, **ASCE 7-16 introduces site classification**, which divides soils into six classes (A through F) based on parameters such as average shear wave velocity and soil profile depth as shown in **Figure 4.4** (ASCE, 2017, Chapter 20, Section 20.3). These site classes modify the mapped spectral accelerations **Ss** and $\mathbf{S_1}$ through amplification factors, reflecting the influence of local soil conditions on seismic demand (ASCE, 2017, Chapter 11, Tables 11.4-1 and 11.4-2).

Table 20.3-1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard rock	>5,000 ft/s	NA	NA
B. Rock	2,500 to 5,000 ft/s	NA	NA
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50 blows/ft	>2,000 lb/ft^2
D. Stiff soil	600 to 1,200 ft/s	15 to 50 blows/ft	1,000 to 2,000 lb/ft^2
E. Soft clay soil	<600 ft/s	<15 blows/ft	<1,000 lb/ft^2
	Any profile with more than 10 ft of soil that has the following characteristics: — Plasticity index $PI > 20$, — Moisture content $w \geq 40\%$, — Undrained shear strength $\bar{s}_u < 500$ lb/ft^2		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

Note: For SI: 1 ft = 0.3048 m; 1 ft/s = 0.3048 m/s; 1 lb/ft^2 = 0.0479 kN/m^2.

Figure 4.4: Site classification based on Table 20.3-1 in ASCE 7-16

Two **site coefficients**, $\mathbf{F_a}$ (for short-period motion) and $\mathbf{F_v}$ (for long-period motion), are used to adjust the spectral accelerations for site-specific conditions. These coefficients are selected from standard tables in the code, based on both the site class and the corresponding spectral accelerations, yielding the **maximum considered earthquake spectral accelerations** which reflect the most severe shaking a structure is expected to endure:

- **$S_{MS} = F_a \times Ss$**
- **$S_{M1} = F_v \times S_1$**

Since the design process does not rely on the full intensity of the maximum considered earthquake, a reduction is applied to account for controlled, inelastic behavior. The resulting values—**design spectral accelerations (S_{DS} and S_{D1})**—are what the structure is actually designed to resist. The design spectral accelerations are calculated by applying a reduction factor of 2/3:

- **$S_{DS} = (2/3) \times S_{MS}$**
- **$S_{D1} = (2/3) \times S_{M1}$**

4.4.4 Additional parameters and interdependence of seismic factors

An additional key consideration in seismic design is the **Occupancy Category** (also known as the **Risk Category**) of the building. This classification reflects the consequences of failure and the need to maintain post-earthquake functionality, especially for essential or high-occupancy structures. When combined with the mapped and adjusted spectral accelerations, this category informs the determination of the building's **Seismic Design Category (SDC)**. The SDC plays a pivotal role in shaping the overall seismic design strategy, influencing the selection of structural systems, analysis methods, and applicable detailing and code requirements.

All the parameters discussed thus far form the foundation for selecting an appropriate **seismic force-resisting system (SFRS)**—the backbone of the structural response to seismic loading. Rather than acting in isolation, these parameters operate as an interconnected network, guiding engineers toward systems that can effectively withstand anticipated seismic demands.

Figure 4.5 presents a dependency diagram developed by the authors to illustrate this conceptual network. The flowchart highlights the sequence of decisions and refinements: how initial ground motion values, such as mapped spectral accelerations, are adjusted for site effects and reduced using code-based factors, ultimately leading to the design spectral response and the selection of an appropriate lateral force-resisting system.

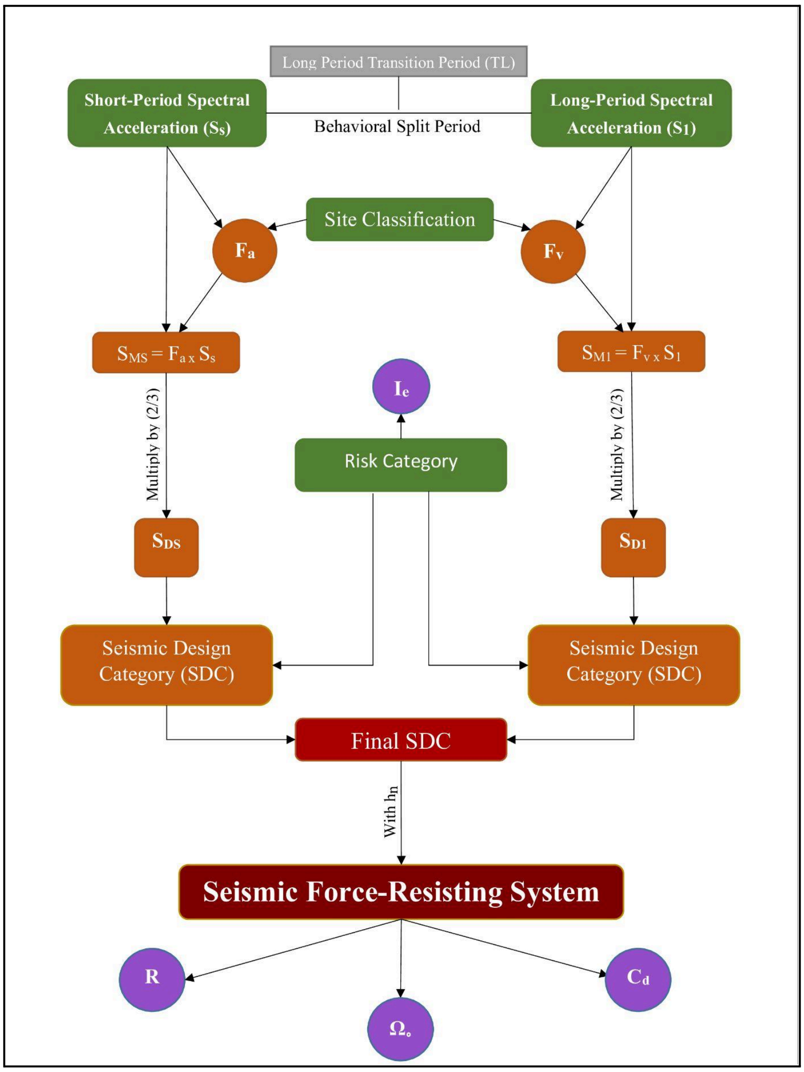


Figure 4.5: Dependency Diagram of Seismic Design Parameters

This flow clarifies the logic behind seismic parameter determination and supports a deeper understanding of how design choices are systematically shaped. It also provides a conceptual transition to the next section, where these parameters are applied to the actual site and structural characteristics of this project.

Additional factors such as the **importance factor (I_e)**, **response modification coefficient (R)**, **overstrength factor (Ω_0)**, **deflection amplification factor (C_d)**, and **period estimation coefficients (C_t and x)** play supporting roles at this stage, primarily influencing later phases of analysis and detailed design. While they are not central during the initial determination of seismic hazard and site effects, they become increasingly critical in shaping the final design forces, displacements, and detailing requirements. These parameters ultimately form the pillars of the structural response and will be discussed further once the seismic design approach and system selection are introduced.

Table 4.2: Summary of seismic design parameters

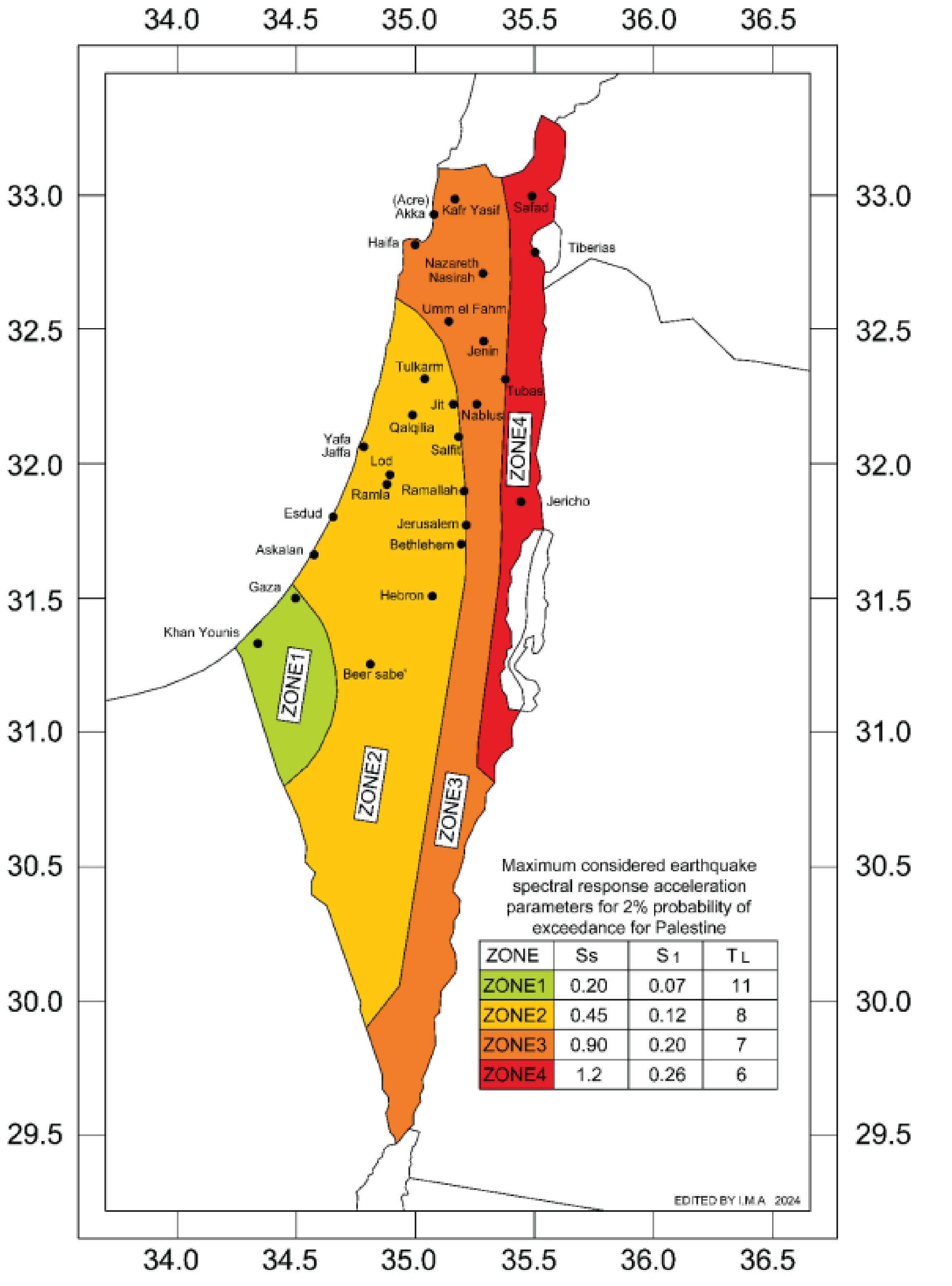
Parameter	Description	Notes
S_s	Mapped short-period spectral acceleration (0.2s).	Represents peak ground motion for short-period structures.
S_1	Mapped long-period spectral acceleration (1.0s).	Represents peak ground motion for long-period structures.
TL	Long-period transition period.	Separates short-period and long-period structure behavior in the design spectrum.
Site Classification	Describes local soil and rock conditions, affecting amplification of seismic waves.	It depends primarily on shear wave velocity, standard penetration resistance, and undrained shear strength
F_a	Site coefficient for short-period ground motion.	Depends on site class and S_s.
F_v	Site coefficient for long-period ground motion.	Depends on site class and S_1.
S_{MS}	Maximum considered earthquake spectral response acceleration at short periods.	Calculated as $F_a \times S_s$.
S_{M1}	Maximum considered earthquake spectral response acceleration at long periods.	Calculated as $F_v \times S_1$.
S_{DS}	Design spectral acceleration for short periods.	$S_{DS} = (2/3) \times S_{MS}$.
S_{D1}	Design spectral acceleration for long periods.	$S_{D1} = (2/3) \times S_{M1}$.
Risk Category	Classification of structure based on occupancy.	Used to determine Seismic Design Category and the Importance factor.
SDC	Seismic Design Category.	Determines seismic design requirements based on S_{DS}, S_{D1}, and Risk Category.
I_e	The Importance factor, reflects the importance of structure and controls the severity of earthquake damage.	Adjusts design forces based on building function.
R	Response modification coefficient.	Reduces elastic forces to account for inelastic behavior.
Ω_0	Overstrength factor.	Used in load combinations to account for extra strength beyond design level.
C_d	Deflection amplification factor.	Used to estimate expected displacements in seismic design.

4.5 Seismic design parameters: Determination and adaptation

This section presents the key seismic design parameters as adapted for this specific project, characterizing the expected earthquake demands at the site. These values were derived primarily from ASCE 7-16 and supplemented by additional references cited where relevant. Establishing these parameters is the first step in the seismic design process, as they define the conditions under which the structure must perform. Based on these values, the appropriate Seismic Force-Resisting System is selected in the following section. That system, in turn, provides the essential design coefficients that guide the structural analysis and detailing. In this way, the parameters discussed here initiate the design path followed throughout the rest of the seismic design.

4.5.1 S_s, S_1, and TL

Figure 4.6 presents the seismic hazard map for Palestine adapted from *Arman (2025a)*, which provides region-specific values for the mapped spectral accelerations and long-period transition periods. The derivation of these values follows the methodology described in detail by the original authors, and is based on probabilistic seismic hazard analysis calibrated for the region. For the purposes of this study, the focus is on the application of the provided values rather than their derivation.



ZONE	Ss	S1	TL
ZONE1	0.20	0.07	11
ZONE2	0.45	0.12	8
ZONE3	0.90	0.20	7
ZONE4	1.2	0.26	6

Figure 1.12: Palestine seismic map; seismic spectral acceleration parameters, S_s and S_1, for 2% probability of exceedance, modified.

Figure 4.6: Palestine seismic map

The values of Ss, S_1, and TL were determined through interpolation between the seismic data for Jett and Nablus, as the project site lies geographically between them. **Table 4.3** presents the resulting seismic parameters.

Table 4.3: Seismic factors

Factors	Value
S_s	0.6
S_1	0.14
T_L	7 Sec

4.5.2 Site classification and coefficients

4.5.2.1 Site classification

As shown previously in **Figure 4.4**, the site has been classified as **Site Class C**. The classification is supported by the following calculation, which utilizes **Terzaghi's classical bearing capacity equation** as presented in *Foundation Analysis and Design* by Bowles (1996):

$$q_u = N_C \text{ x } S_u$$

Given:

- q_u=741 kPa (as reported in the geotechnical study; see Section 1.8),
- N_c values vary depending on the footing type, as listed in Table 4-2 of the same reference (Bowles, 1996), and reproduced in **Appendix E**. Values are:
  - 5.14 for strip footings,
  - 5.7 for isolated square footings,
- A conservative intermediate value of **5.14** was adopted.

Solving for the undrained shear strength:

$$S_u = \frac{qu}{Nc} = \frac{741}{5.14}$$

Su = 0.144 MPa (or 144kPa) → **2715.05 Ib/ft^2**

Where:

q_u= ultimate bearing capacity.

N_c= Bearing capacity factor depended on shape of footing.

S_u= undrained shear strength.

This value of S_u, in combination with other geotechnical properties and site conditions, supports the assignment of Site Class C for seismic design purposes, in accordance with ASCE 7-16.

4.5.2.2 Site coefficients

Two site coefficients were adopted in accordance with *Tables 11.4-1* and *11.4-2* of *ASCE 7-16* (shown in **Figures 4.7** and **4.8**, respectively): the short-period site coefficient, $\mathbf{F_a}$, and the long-period site coefficient, $\mathbf{F_v}$. These coefficients are used to adjust the mapped spectral acceleration **Ss** and $\mathbf{S_1}$, respectively.

Table 11.4-1 Short-Period Site Coefficient, F_a

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at Short Period					
	$S_S \le 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S = 1.25$	$S_S \ge 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0
E	2.4	1.7	1.3	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use straight-line interpolation for intermediate values of S_s.

Figure 4.7: Short-period site coefficient (F_a) per ASCE 7-16

Table 11.4-2 Long-Period Site Coefficient, F_v

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at 1-s Period					
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \geq 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	2.2[a]	2.0[a]	1.9[a]	1.8[a]	1.7[a]
E	4.2	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use straight-line interpolation for intermediate values of S_1.
[a]Also, see requirements for site-specific ground motions in Section 11.4.8.

Figure 4.8: Long-period site coefficient (F_v) per ASCE 7-16

From **Figure 4.7** (Table 11.4-1 in ASCE 7-16), and for **Ss = 0.6** (i.e., 0.5 < Ss < 0.75):
→ $\mathbf{F_a}$ **= 1.26** (determined by linear interpolation between 1.3 and 1.2)

From **Figure 4.8** (Table 11.4-2 in ASCE 7-16), and for $\mathbf{S_1}$ **= 0.14** (i.e., 0.1 < S_1 < 0.2):
→ $\mathbf{F_v}$ **= 1.5**

4.5.3 Risk category and importance factor

4.5.3.1 Risk category

Risk Category is a classification system used to determine the level of importance of a building or structure based on its intended use and the consequences of its failure. This classification directly affects how conservative the seismic design should be.

In general, the selection of a facility's risk category is based on the engineer's evaluation of the building's function, its occupancy, and the level of performance required after an earthquake event. In ASCE 7-16, **Table 1.5-1** (shown in **Figure 4.9**) categorizes buildings into **Risk Categories I through IV**, ranging from lowest to highest importance, based on increasing hazard to human life and the building's role in supporting essential community or societal functions.

Table 1.5-1 Risk Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads

Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent low risk to human life in the event of failure	I
All buildings and other structures except those listed in Risk Categories I, III, and IV	II
Buildings and other structures, the failure of which could pose a substantial risk to human life	III
Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure	
Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released[a]	
Buildings and other structures designated as essential facilities	IV
Buildings and other structures, the failure of which could pose a substantial hazard to the community	
Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released[a]	
Buildings and other structures required to maintain the functionality of other Risk Category IV structures	

[a]Buildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the Authority Having Jurisdiction by a hazard assessment as described in Section 1.5.3 that a release of the substances is commensurate with the risk associated with that Risk Category.

Figure 4.9: Risk category of buildings. (ASCE 7-16, Table 1.5-1)

For this project, both **Risk Category II** and **Risk Category III** could initially be considered, based on the descriptions and criteria outlined in the referenced classification table. To resolve this ambiguity, Table 1604.5 of the *International Building Code* (IBC 2021) was consulted for clarification. While the complete table is included in **Appendix E**, only the relevant portion is cited here.

According to the IBC's description under **Risk Category III**, buildings are classified under this category if they:

> *"Represent a substantial hazard to human life in the event of failure, including but not limited to:*
>
> *• Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300."*[6]

Given that the facility consists of residential apartment units with a total occupancy of fewer than 300 residents, **Risk Category II** was ultimately selected as appropriate for the design.

Beyond its role in determining the Seismic Design Category—an essential step in selecting the appropriate Lateral Force Resisting System—the Risk Category also informs the selection of the seismic Importance Factor. Although the Importance Factor is not the primary focus of this discussion, it is directly derived from the Risk Category and is thus presented in the following subsection.

4.5.3.1 Importance Factor (I_e)

The seismic Importance Factor is a direct numerical expression of the previously determined Risk Category. It translates the qualitative assessment of a structure's importance into a quantitative factor used in seismic load calculations, ensuring that buildings with higher risk classifications are designed to withstand greater seismic demands.

In accordance with the Risk Category, *ASCE 7-16 Table 1.5-2* (shown in **Figure 4.10**) provides the corresponding Importance Factor. For the selected Risk Category II, the Importance Factor, I_e, is equal to **1.00**.

[6] This excerpt is one of several conditions listed under Risk Category III in **IBC Table 1604.5**. Only the relevant clause is included here, while the full list is available for reference in **Appendix G**.

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

Note: The component importance factor, I_p, applicable to earthquake loads, is not included in this table because it depends on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

Figure 4.10: Importance factor by risk category (ASCE 7-16, Table 1.5-2)

4.5.4 Seismic Design Category (SDC)

To determine the Seismic Design Category, a series of calculations must first be performed, beginning with the determination of **the design spectral response accelerations**, S_{DS} and S_{D1}.

S_{DS} and S_{D1}:

Using the site coefficients F_a and F_v previously determined, the mapped spectral response accelerations Ss and S_1 are adjusted to compute the Maximum Considered Earthquake (MCE) spectral accelerations:

- $S_{MS} = F_a \times Ss = 1.26 \times 0.6 = 0.756$

  → **$S_{MS} = 0.756$**

- $S_{M1} = F_v \times S_1 = 1.5 \times 0.14 = 0.21$

  → **$S_{M1} = 0.21$**

These values are then reduced by a factor of two-thirds (⅔) to obtain the **design spectral response accelerations**, which are used in structural design:

- $S_{DS} = (⅔) \times S_{MS} = (⅔) \times 0.756 = 0.504$

  → **$S_{DS} = 0.504$**

- $S_{D1} = (⅔) \times S_{M1} = (⅔) \times 0.21 = 0.14$

→ S_{D1} = **0.14**

The Seismic Design Category was then determined using two criteria from ASCE 7-16: Table 11.6-1 (**Figure 4.11**) for the short-period design spectral acceleration (SDS) and Table 11.6-2 (**Figure 4.12**) for the 1-second period spectral acceleration (SD1).

Based on the short-period acceleration SDS, the building falls under **Seismic Design Category D**. Based on the 1-second period acceleration SD1, it is classified as **Category C**. Following standard practice, the more critical classification governs; therefore, **Seismic Design Category D** was adopted.

TABLE 11.6-1 Seismic Design Category Based on Short-Period Response Acceleration Parameter

	Risk Category	
Value of S_{DS}	I or II or III	IV
$S_{DS} < 0.167$	A	A
$0.167 \le S_{DS} < 0.33$	B	C
$0.33 \le S_{DS} < 0.50$	C	D
$0.50 \le S_{DS}$	D	D

Figure 4.11: Seismic design category based on short period response acceleration parameter

TABLE 11.6-2 Seismic Design Category Based on 1-s Period Response Acceleration Parameter

	Risk Category	
Value of S_{D1}	I or II or III	IV
$S_{D1} < 0.067$	A	A
$0.067 \le S_{D1} < 0.133$	B	C
$0.133 \le S_{D1} < 0.20$	C	D
$0.20 \le S_{D1}$	D	D

Figure 4.12: Seismic design category based on 1-s period response acceleration parameter

4.6 Seismic-Force-Resisting System and design factors and coefficients

4.6.1 Seismic-Force-Resisting System

The Seismic Force Resisting System (SFRS) for this project was selected based on ASCE 7-16, Table 12.2-1, which outlines allowable systems by structural height (hn) and Seismic Design Category (SDC). With an hn of 29.65 meters and SDC D, the permitted systems were identified. While several options are allowed, the final choice also considered the existing structure to minimize modifications.

Special reinforced concrete shear walls were selected as the most suitable option, aligning with the original layout, which primarily includes load-bearing walls and lacks significant frame elements. Alternative systems like moment frames would require major changes, complicating the design. This selection supports a simpler, more compatible seismic upgrade. Figure 4.13 illustrates the chosen SFRS.

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_0^b	Deflection Amplification Factor, C_d^c	Structural System Limitations Including Structural Height, h_n (ft) Limits[d] — Seismic Design Category B	C	D^e	E^e	F^f
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls[g,h]	14.2	5	2½	5	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls[g]	14.2	4	2½	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls[g]	14.2	2	2½	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls[g]	14.2	1½	2½	1½	NL	NP	NP	NP	NP
5. Intermediate precast shear walls[g]	14.2	4	2½	4	NL	NL	40[i]	40[i]	40[i]
6. Ordinary precast shear walls[g]	14.2	3	2½	3	NL	NP	NP	NP	NP
7. Special reinforced masonry shear walls	14.4	5	2½	3½	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	14.4	3½	2½	2¼	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	14.4	2	2½	1¾	NL	160	NP	NP	NP
10. Detailed plain masonry shear walls	14.4	2	2½	1¾	NL	NP	NP	NP	NP
11. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
12. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
13. Ordinary reinforced AAC masonry shear walls	14.4	2	2½	2	NL	35	NP	NP	NP
14. Ordinary plain AAC masonry shear walls	14.4	1½	2½	1½	NL	NP	NP	NP	NP
15. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	6½	3	4	NL	NL	65	65	65
16. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	6½	3	4	NL	NL	65	65	65
17. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2	2½	2	NL	NL	35	NP	NP
18. Light-frame (cold-formed steel) wall systems using flat strap bracing	14.1	4	2	3½	NL	NL	65	65	65

Figure 4.13: The selected Seismic Force-Resisting System (ASCE 7-16, Table 12.2-1)

Therefore, the selected Seismic Force-Resisting System—**Special Reinforced Concrete Shear Walls** under the **Bearing Wall Systems** category—not only complies with seismic design requirements but also fits naturally within the original structural layout, minimizing the need for significant reconfiguration.

4.6.2 Seismic design factors and coefficients

With the system identified, the next step involves defining the key **seismic design parameters** associated with it. These include:

- **Response Modification Coefficient (R)**
- **Deflection Amplification Factor (C_d)**
- **Overstrength Factor (Ω_0)**

These coefficients are essential inputs in various stages of the seismic design process, such as the calculation of base shear, drift checks, and force amplification for detailing requirements.

Response Modification and Amplification Coefficients:

The values of these parameters are specified in **ASCE 7-16 Table 12.2-1**, corresponding to the selected system under the *Bearing Wall Systems* category. For **Special Reinforced Concrete Shear Walls**, the coefficients are:

- **Response Modification Coefficient (R) = 5.0**
- **Overstrength Factor (Ω_0) = 2.5**
- **Deflection Amplification Factor (C_d) = 5.0**

These values reflect the expected nonlinear behavior and energy dissipation capacity of the selected system. They will be applied in later design stages, ensuring compliance with the code's performance objectives related to strength, stiffness, and ductility.

Redundancy Factor (ρ)

The redundancy factor (ρ), defined in ASCE 7-16 Section 12.3.4, accounts for the structure’s ability to redistribute seismic forces through alternate load paths. A value of **1.3** was used in this project. This factor was assigned in ETABS and applied during the seismic design phase.

4.7 Seismic analysis methods: Comparison and selection

Seismic analysis methods reflect how a structure will respond under the influence of earthquake forces. In the context of seismic design, understanding these methods is essential to ensure that the structural response to seismic events is accurately captured and appropriately designed for.

There are various seismic analysis methods used in practice, ranging from simple to complex. The four most commonly used methods in seismic analysis are:

- Equivalent Static Force Procedure.
- Response Spectrum Analysis.
- Nonlinear Static (Pushover) Analysis.
- Time History Analysis.

Each of such provides a unique approach to modeling the dynamic response of buildings experiencing seismic effects, and each has its advantages and limitations. These methods vary in complexity, accuracy, and applicability depending on the building's size, design, and the expected seismic risk.

While these four are the most widely applied in engineering practice, there are other approaches available as well. Below is a detailed explanation of each method, followed by a comparison of their advantages and limitations. Finally, the most suitable method for this project is determined based on the project's specific requirements.

1. Equivalent Static Method

The Equivalent Static Method is one of the simplest and most widely used methods for seismic analysis. It assumes the seismic forces acting on the structure are equivalent to a static load, based on the building's mass, stiffness, and the seismic parameters defined for the site. This method is typically used for buildings that are relatively regular and of low to moderate height.

- **Advantages**:
  - Simple to apply.
  - Suitable for low-rise buildings and structures with minimal complexity.
- **Limitations**:
  - Does not account for dynamic effects such as frequency-dependent response.
  - Not suitable for highly irregular or tall structures.

2. Response Spectrum Method

The Response Spectrum Method accounts for the dynamic behavior of a structure by using a response spectrum, which represents the maximum peak response of a structure to seismic ground motion over a range of frequencies. This method considers the structure's natural frequency and mass distribution, allowing for a more accurate prediction of the structure's response to seismic forces.

- **Advantages**:
  - Accounts for dynamic effects and mode shapes.
  - Suitable for medium-rise buildings and buildings with higher complexity.
- **Limitations**:
  - Assumes linear elastic behavior.
  - Does not capture time-dependent effects such as duration of shaking.

3. Time History Analysis

The Time History Analysis is a dynamic analysis method that simulates the actual time-dependent behavior of the structure under a specific ground motion record. This method provides a detailed understanding of the structure's response, accounting for both the frequency content and the duration of the seismic event.

- **Advantages**:
  - Provides detailed and accurate predictions of structural behavior.
  - Accounts for both the magnitude and duration of seismic events.
- **Limitations**:
  - Requires significant computational resources.
  - Sensitive to the selection of the ground motion record.

4. Nonlinear Analysis

Nonlinear Analysis considers the inelastic behavior of the structure under seismic loading, taking into account factors such as material yielding, plastic deformations, and energy dissipation mechanisms. This method is particularly useful for evaluating the performance of structures in seismic zones with a higher risk of significant damage.

- **Advantages**:

○ Accurately captures the inelastic response of the structure.

○ Essential for highly irregular structures and for assessing post-yield behavior.

- **Limitations**:

  ○ More complex and computationally intensive.

  ○ Requires detailed material and structural modeling.

Each method offers specific advantages and limitations, reflecting varying levels of approximation and suitability for different project types. Some methods aim to capture peak structural response, while others provide a more detailed representation of time-dependent behavior during seismic events. **Table 4.4** provides a comparative summary of the four methods—highlighting their ease of application, strengths, drawbacks, and appropriate use cases—enabling informed selection based on the characteristics and requirements of the project.

Table 4.4: Comparison of seismic analysis methods

Method	Ease of Application	Advantages	Disadvantages	Suitability
Equivalent Static Method	Easy	Simple to apply, computationally efficient, suitable for small to medium buildings	Assumes linear behavior, not suitable for large/complex structures	Best for low seismic risk areas, simple buildings
Response Spectrum Analysis	Moderate	More accurate than static method, accounts for dynamic behavior	Requires more input date, more complex calculations	Suitable for medium to large buildings
Time History Analysis	Difficult	Most detailed, captures realistic seismic response, highly accurate	Computationally intensive, requires quality ground motion data	Used for high-risk structures, irregular geometries
Nonlinear Analysis	Difficult	Captures inelastic behavior, accurate for structures with significant deformation	Complex and time-consuming, requires detailed modeling	Suitable for structures expected to undergo significant inelastic behavior

Selection of Seismic Analysis Method for the Project:

In this project, the **Equivalent Equivalent Static Method** was selected as the primary approach for seismic analysis and structural checks. This choice reflects the project’s relative simplicity and the method’s ability to provide a reasonable balance between accuracy and computational efficiency for the given structural configuration and seismic hazard level. However, due to the presence of certain irregularities and the need to better capture the building's dynamic behavior, the potential necessity for a **Response Spectrum Analysis** remains. Therefore, both methods were defined in the ETABS model (see Section 5.3), and their application—along with an evaluation of their suitability for the project's specific requirements—is discussed in detail in Chapter 5.

CHAPTER 5: SEISMIC MODELING, ANALYSIS, AND CHECKS

5.1 General

This chapter presents the modeling strategy, analysis procedure, and the key checks performed to evaluate the structural behavior, compliance with code requirements, and suitability of the selected seismic analysis method. These steps are essential to ensure that the model accurately represents the real behavior of the structure under seismic demands, and to confirm that the adopted design approach remains both safe and code-compliant before moving to the final seismic design phase.

Minor preliminary modifications were made to the model prior to the checks in this chapter, based on engineering judgment and prior seismic design experience. These adjustments aimed to improve expected lateral performance. Details are provided in Section 6.4.

The chapter proceeds by presenting the base shear evaluation and scaling process, followed by a series of checks that address stability, irregularities, and story drifts. Each check is introduced with an explanation of its purpose and relevance, and results are interpreted in the context of seismic adequacy. Further refinement and design decisions—including structural modifications and reinforcement adjustments—are discussed in Chapter 6.

5.2 Modeling strategy and assumptions

The seismic modeling and analysis phase was executed using ETABS, extending from the gravity-load model developed in Part I. In this stage, additional considerations specific to seismic design were incorporated, including both loading definitions and lateral system configuration. ETABS provided a suitable platform to perform the required analyses, supporting both static and dynamic seismic procedures.

The scope of this section focuses solely on aspects related to seismic modeling. Elements from the gravity phase—such as cracked section modifiers, materials, and structural member dimensions—remained unchanged and are not revisited here. Instead, attention is directed to the adaptations and refinements applied specifically for seismic purposes, in line with ASCE 7-16 requirements.

5.2.1 Model framework and geometry

The starting point of the seismic modeling phase was the gravity-load model developed earlier. The same three-dimensional frame and shell-based structural representation was maintained. While the general layout, material assignments, and section properties were preserved, certain model-level settings and configurations were adjusted in the seismic stage.

These included updates to modeling preferences and element-specific overwrites. Such modifications were implemented in later stages of the design process to ensure compliance with seismic design requirements. Importantly, the lateral force-resisting system was refined through the introduction of a limited number of shear walls in targeted locations. These additions were made after preliminary evaluation indicated that the original configuration might exhibit torsional irregularities. Details on these adjustments are discussed further in Section 6.3.

5.2.2 Seismic parameters and load case definitions

The seismic parameters used in the model were determined based on ASCE 7-16 provisions and the site-specific hazard values discussed in Chapter 4. These included:

- Mapped spectral response accelerations at short periods (Ss) and at 1-second period (S_1)
- Site class and corresponding site coefficients
- Seismic Design Category (SDC)
- Long-period transition period (**TL**)
- Importance factor ($\mathbf{I_e}$), based on Risk Category II

These parameters were reflected in the definition of dynamic and static load cases and in the design spectrum function. Two seismic load cases were defined:

- **EX / EY**: Equivalent static in X and Y directions
- **EXD / EYD**: Response spectrum in X and Y directions

Modal analysis was conducted in both principal directions (**modal X** and **modal Y**), and a consistent mass source was defined incorporating 100% of the dead and superimposed dead load and 25% of the live load, as permitted by ASCE.

5.2.3 Load combinations and software automation

Load combinations involving seismic effects were applied according to ASCE 7-16. These included both strength and serviceability combinations accounting for seismic demands in various directions and scenarios. To streamline the modeling process and reduce human error, these combinations were not entered manually. Instead, ETABS's built-in combination generator was employed, enabling automatic creation of all code-compliant load combinations. However, additional combinations were defined manually for evaluating second-order effects (P-delta) and stress checks as they are not automatically defined by ETABS.

5.2.4 Code-Based modeling assumptions

In configuring the seismic model, various assumptions required by ASCE 7-16 were incorporated into ETABS. These assumptions reflect common modeling decisions that affect the distribution and interpretation of seismic forces, and they were resolved following code guidelines. These include:

- **Diaphragm constraint assignment**: rigid diaphragms were used for irregularity checks; semi-rigid diaphragms were applied in later design stages
- **Accidental torsion**: modeled using 5% accidental eccentricity
- **Mass source definition**: included 25% of live load in addition to full dead and superimposed dead load
- **Directional combination method**: 100% in one direction + 30% of the orthogonal
- **Modal combination method**: Complete Quadratic Combination (CQC)
- **Modal analysis type**: Ritz vectors
- **Number of modes**: 18
- **Directional combination method (in dynamic results)**: absolute value method
- **Damping ratio**: 5% in the response spectrum function
- **Response spectrum scale**: applied using the factor $\mathbf{g \cdot I_e / R}$, as prescribed by ASCE 7-16 Section 12.9.1.2

These modeling assumptions ensured that the behavior of the structure under seismic excitation was represented in a manner consistent with national standards and design expectations.

5.2.5 Secondary model for irregularity and drift checks

In addition to the primary seismic model, a secondary model was developed to isolate and evaluate potential irregularities and drift behavior more clearly. This model excluded the 30% orthogonal direction combination typically applied in seismic load cases. Its purpose was to assess story drift, plan irregularities, and determine the necessity of P-Delta analysis under pure directional loading conditions.

By distinguishing between the design model and the evaluation model, the structural assessment remained both accurate and code-compliant, facilitating targeted checks without compromising the integrity of the main analysis results.

5.3 Seismic load definitions

Seismic analysis and design requires extra software definitions. These definitions are related to identifying the seismic parameters (coefficients and factors) which are the basis that determine how the structure behaves due to assumed (conservative assumption and expected) of seismic loading. In addition to the definition of the analysis method the software uses in the analysis procedure.

Defined load patterns , seismic load in x and y direction in ETABS,as shown in **Figure 5.1**.

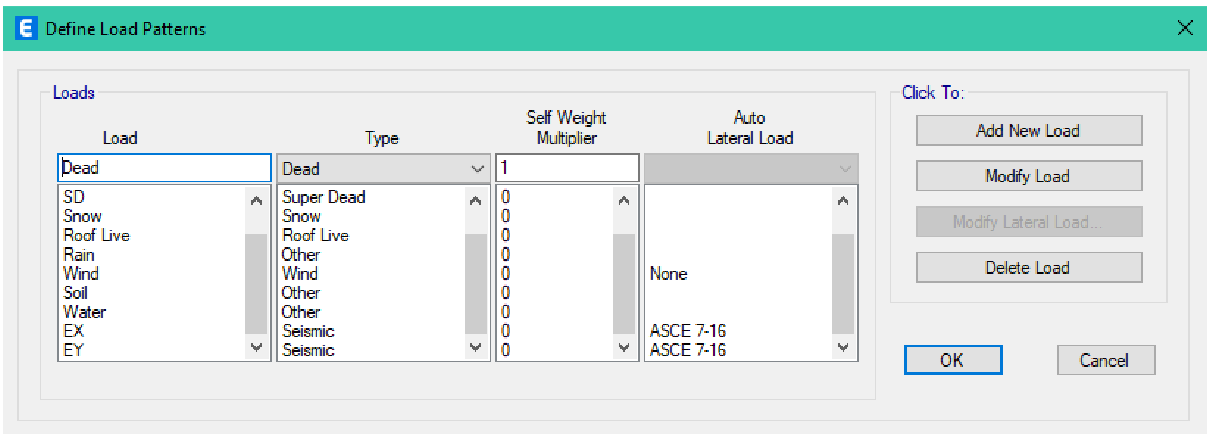


Figure 5.1: Defined load patterns for gravity and seismic in X and Y directions (ETABS)

Defined seismic load pattern in Y-direction and X-direction, as shown in **Figures 5.2** and **5.3**.

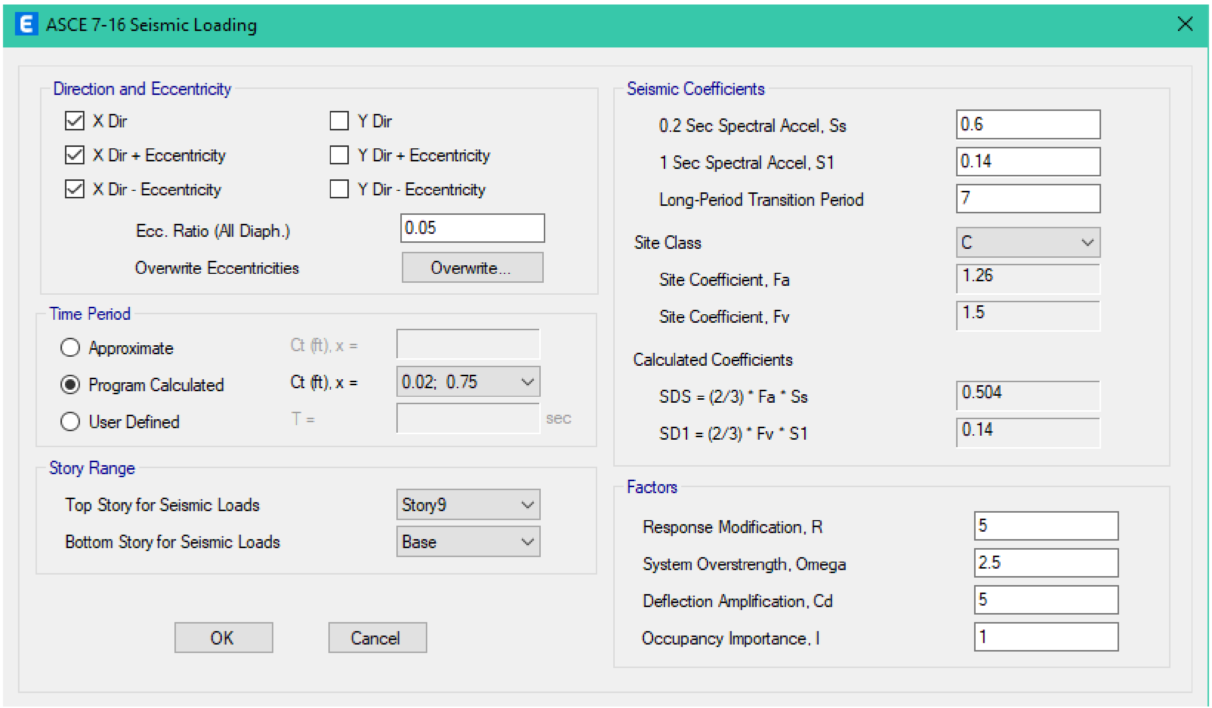


Figure 5.2: Defined load pattern, seismic load in X-direction in ETABS

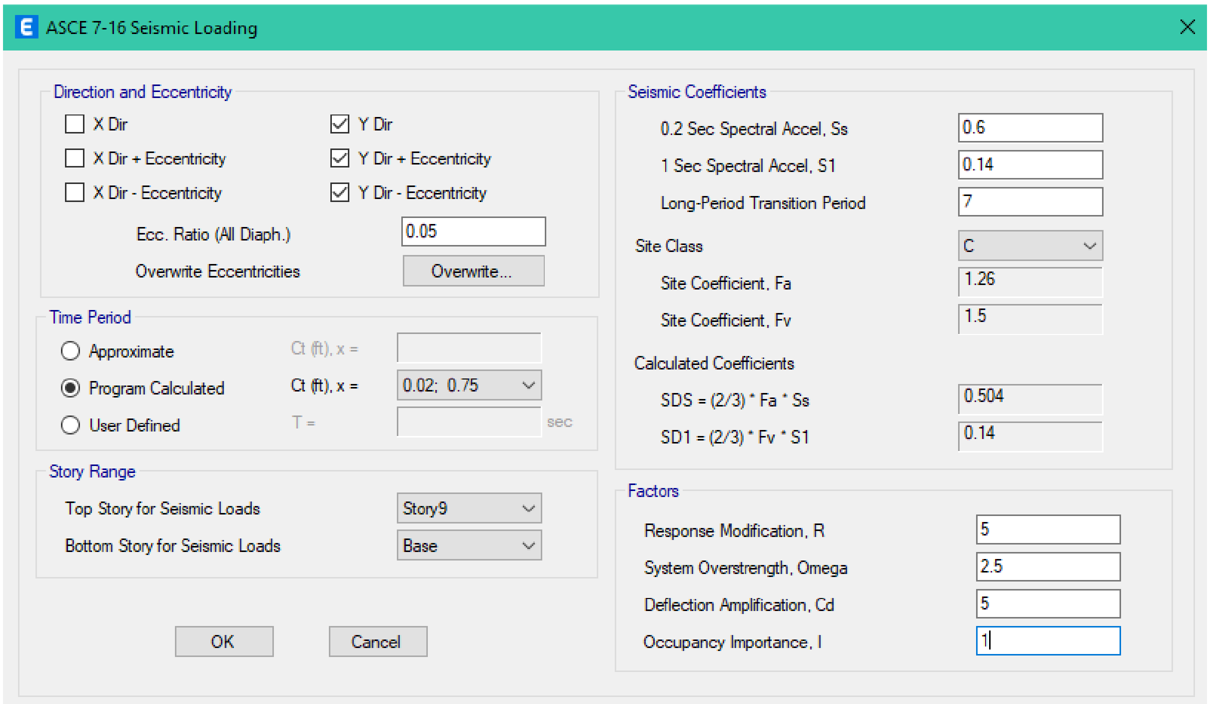


Figure 5.3: Defined load pattern, seismic load in Y-direction in ETABS

Definition of the mass source, as shown in **Figure 5.4**.

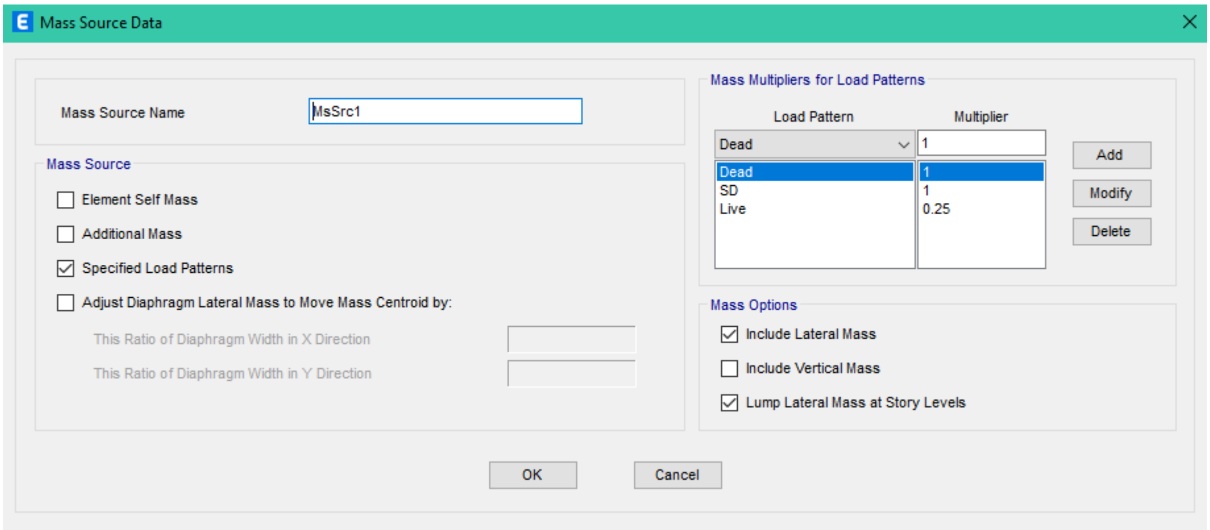


Figure 5.4: Mass source data in ETABS

Definition of the response spectrum function to be used in the response spectrum load case, as shown in **Figure 5.5**.

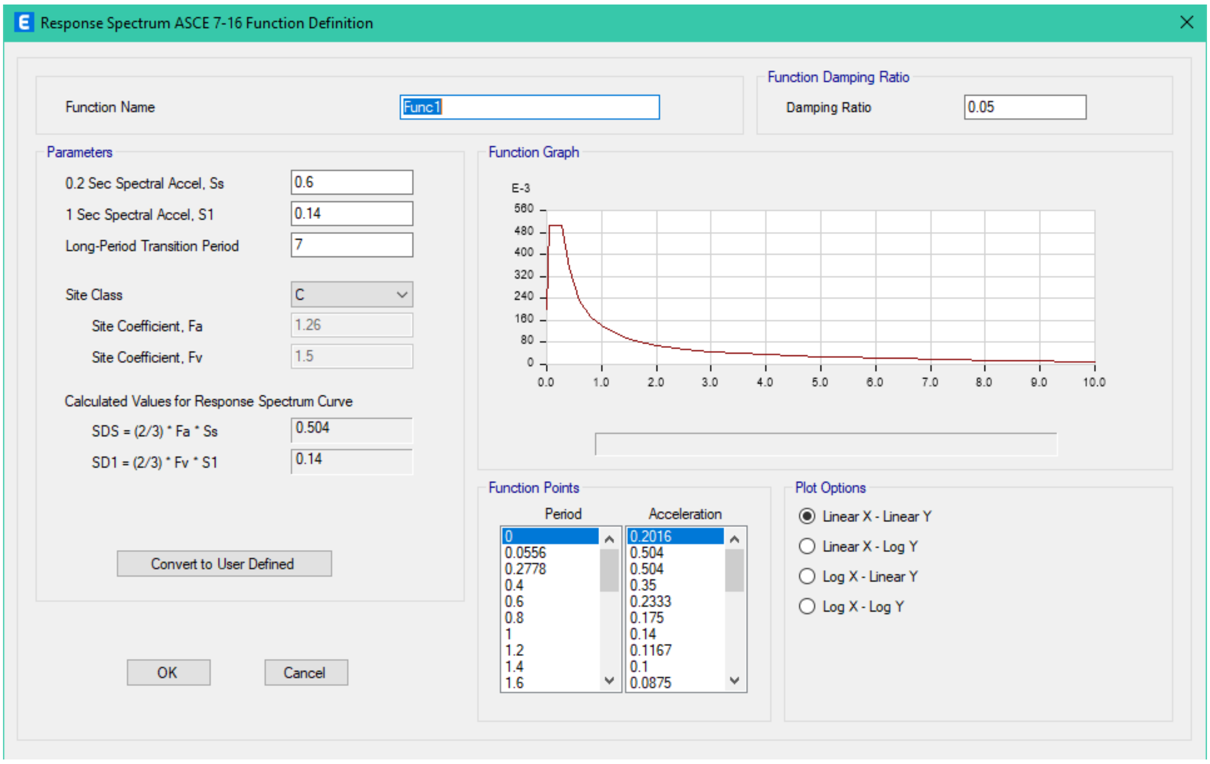


Figure 5.5: Response spectrum ASCE 7-16 function definition in ETABS

Both modal-X and modal-Y are defined in **Figure 5.6** and **5.7** as a ritz.

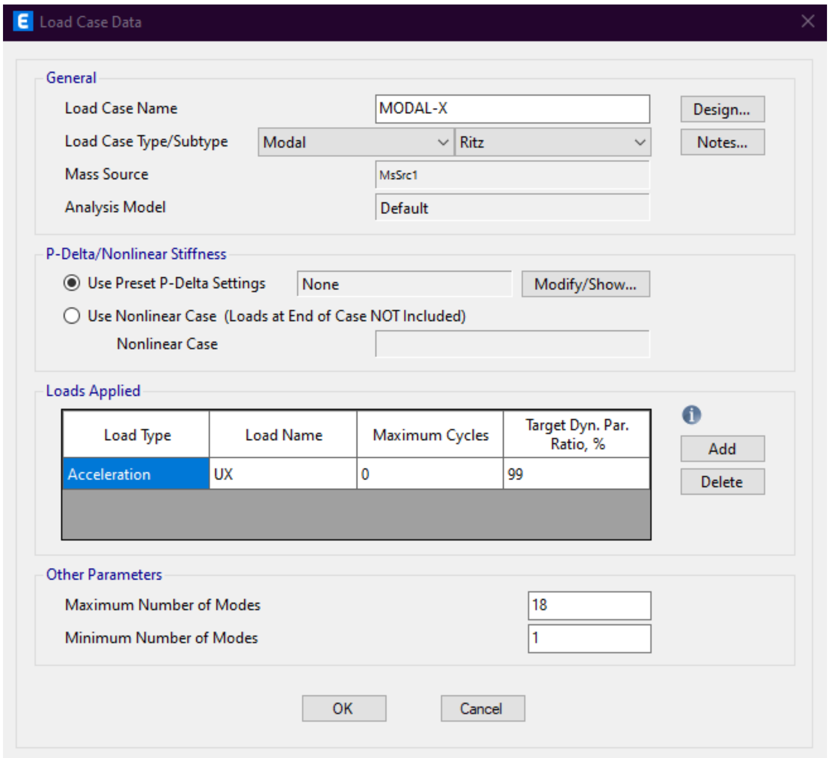


Figure 5.6: Definition of modal ritz in X-direction

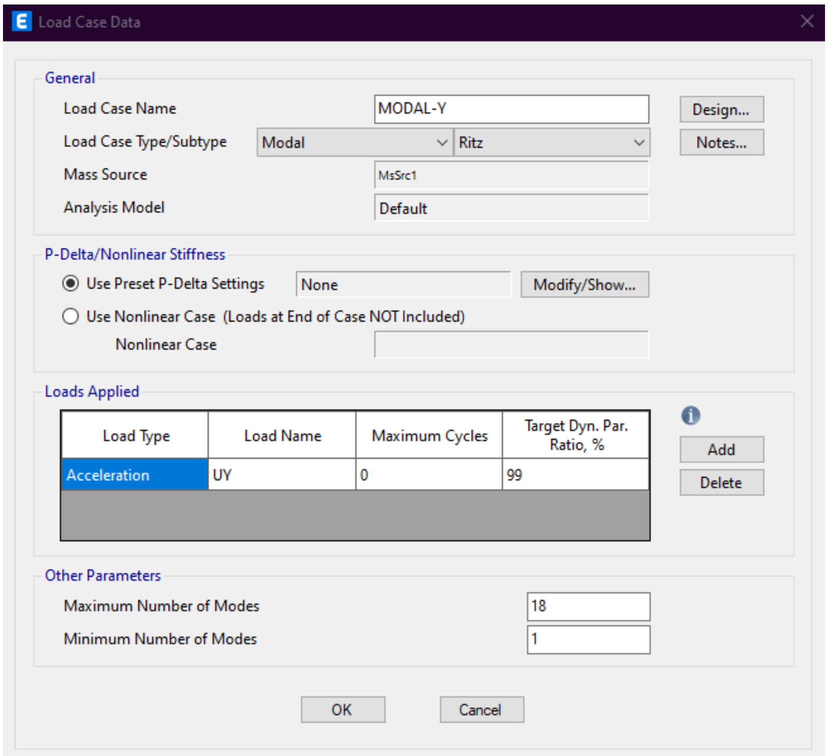


Figure 5.7: Definition of modal ritz in Y-direction

Definition of the response spectrum load cases in both x and y directions, is shown in **Figures 5.8** and **5.9**.

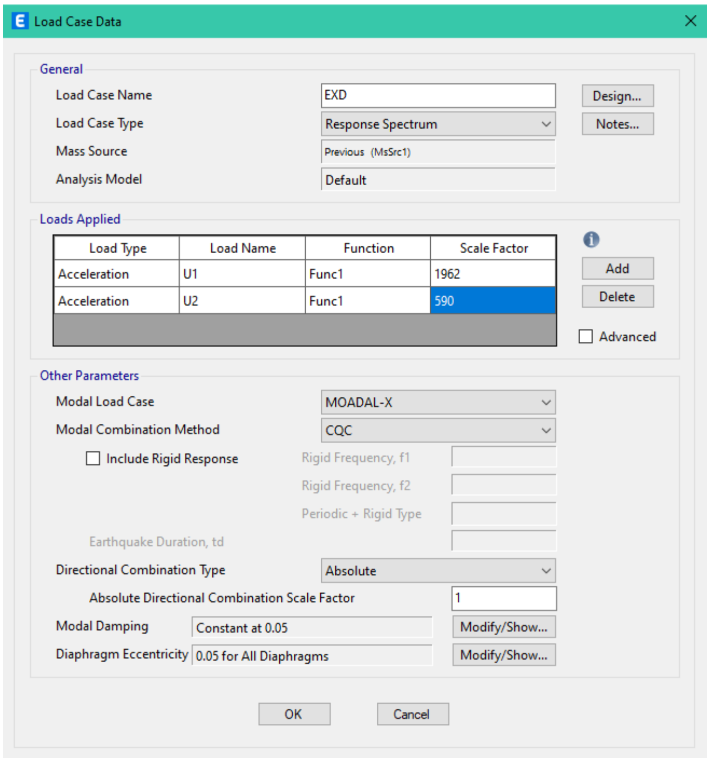


Figure 5.8: Load case data-definition of EDX load case in ETABS

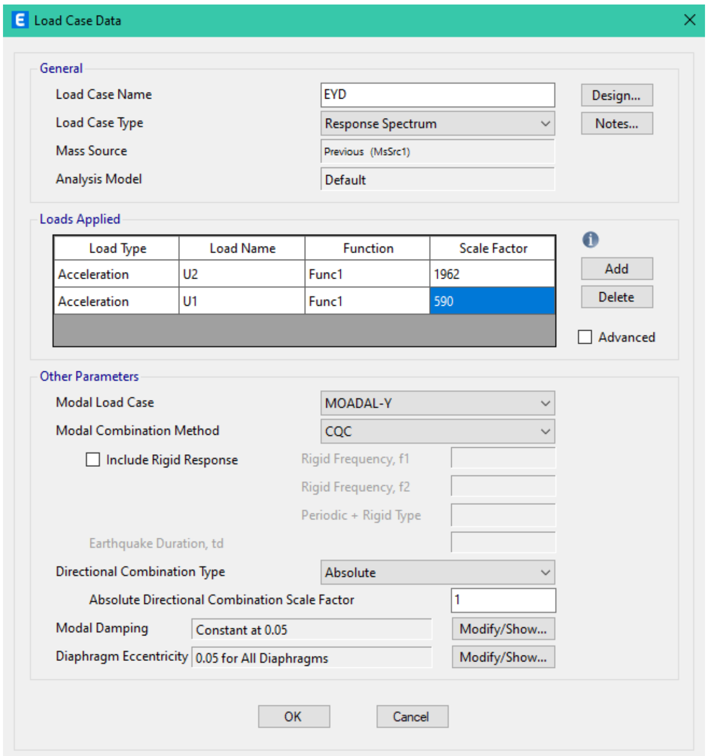


Figure 5.9: Load case data-definition of EDY load case in ETABS

30% of each seismic force EX and EY shall be added to the other direction as shown in both **Figures 5.10** and **5.11**.

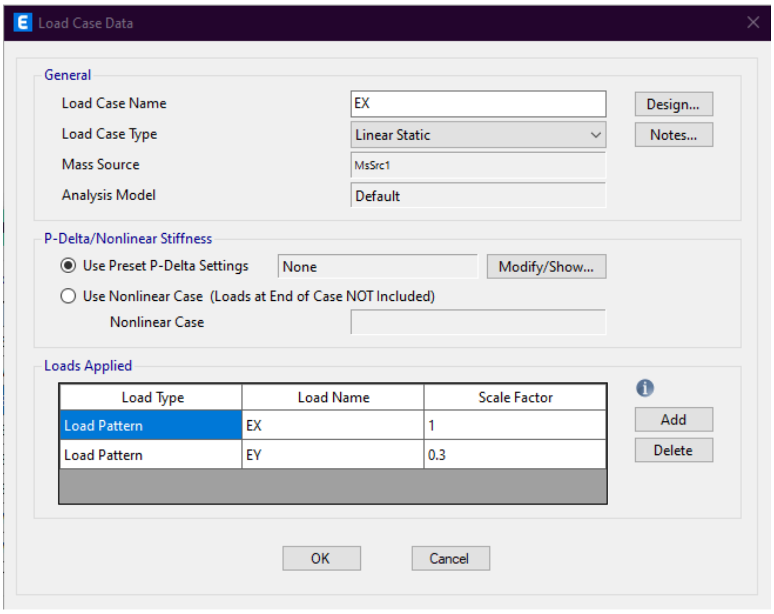


Figure 5.10: 30% of lateral load EY in EX

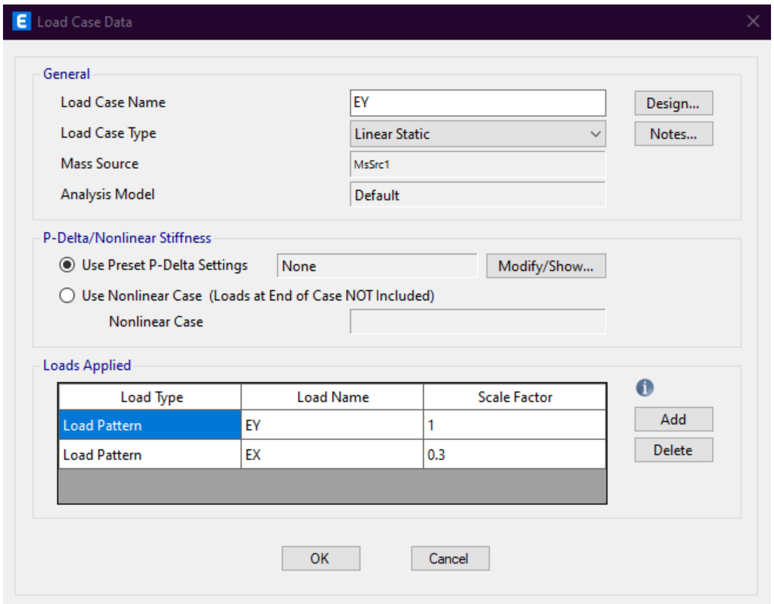


Figure 5.11: 30% of lateral load EX in EY

The concrete frame design preferences were adjusted to reflect the parameters relevant to our project. Figure 5.12 shows the updated settings used in the design.

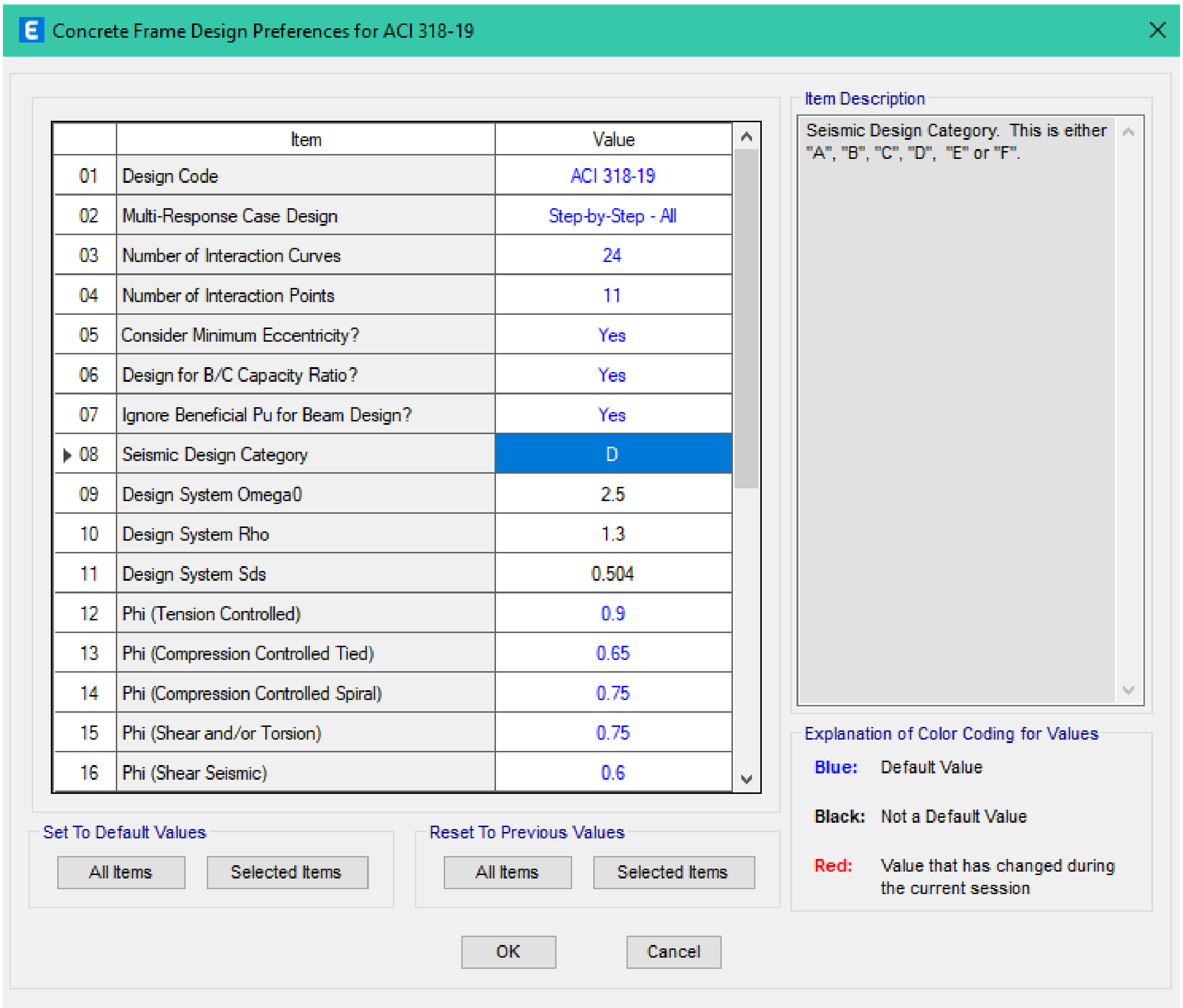


Figure 5.12: Adjusted concrete frame design preferences

Load combinations were added in ETABS to capture the most critical lateral load scenarios, ensuring a comprehensive design under seismic and wind actions. These combinations were created in accordance with ASCE 7-16, which outlines the proper procedures for combining various loads. **Figure 5.13** illustrates the specific load combinations used in the ETABS model.

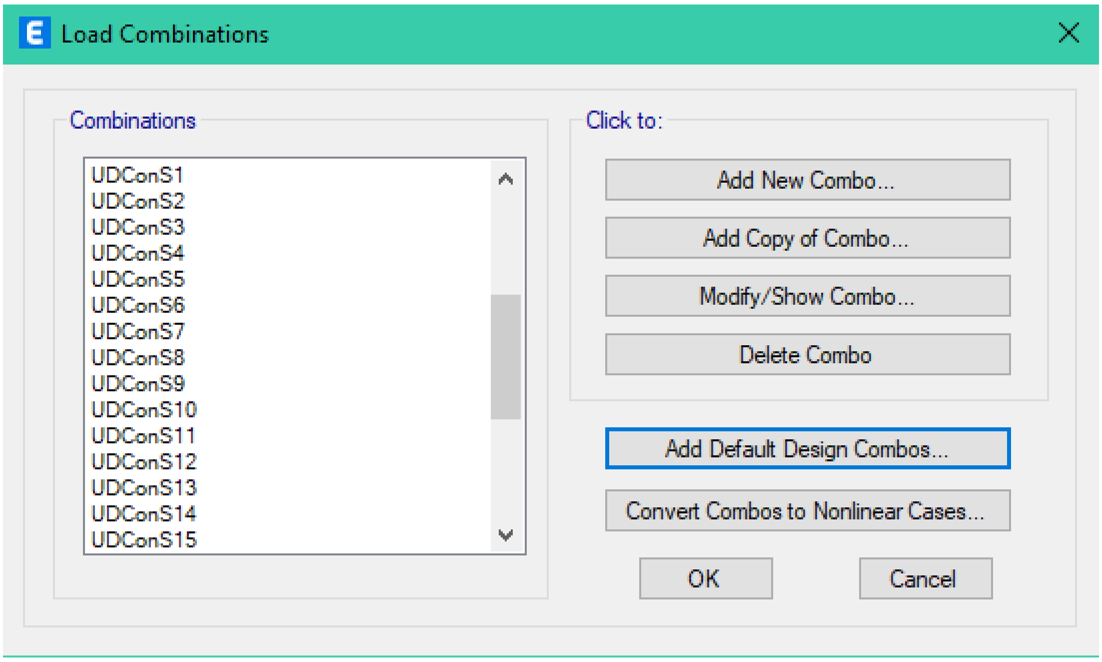


Figure 5.13: Lateral load combination

Diaphragms were added to define the center of mass and center of rigidity for each story throughout the building structure. This ensures accurate distribution of lateral loads and proper dynamic behavior under seismic forces. **Figures 5.14** and **5.15** show the definition and application of the diaphragms within the ETABS model.

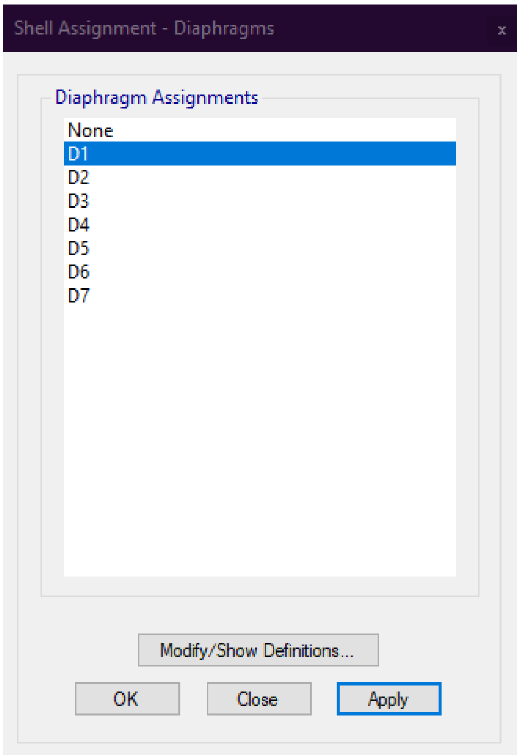


Figure 5.14: Diaphragms definitions

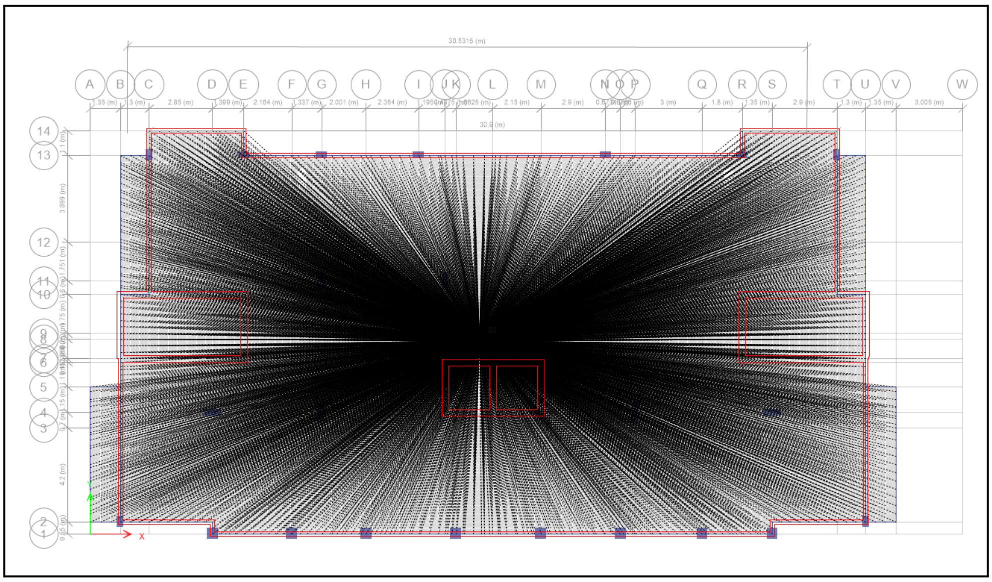


Figure 5.15: Diaphragms applied for each story

5.4 Base shear check and scaling

5.4.1 Modal mass participation check

To ensure that the modal response spectrum analysis adequately captures the dynamic behavior of the structure, a check was performed on the modal participating mass ratios. This process confirms that a sufficient number of vibration modes have been included to account for the majority of the structure's seismic response in both principal directions.

The **ASCE 7-16 Section 12.9.1.2 Exception** states the following:

> *"Alternatively, the analysis shall be permitted to include a minimum number of modes to obtain a combined modal mass participation of at least 90% of the actual mass in each orthogonal horizontal direction of response considered in the model."*

This provision establishes a minimum threshold for acceptable dynamic representation, allowing designers to limit the number of modes analyzed—**as long as 90% of the total mass is captured in each direction**. This simplifies the modeling effort while ensuring the primary modal contributions are included and the computed response reflects the actual dynamic characteristics of the structure. Failing to meet this threshold could result in underestimating lateral demands, potentially leading to an inaccurate or unconservative design, especially in flexible or irregular structures where higher modes may be significant.

The ETABS model was reviewed to verify compliance with this requirement. **Figures 5.16** and **5.17**, adapted from ETABS output tables, illustrate the modal participating mass ratios in the X and Y directions, respectively.

Case	Mode	Period sec	UX	UY	UZ	SumUX	SumUY
MODAL-X	1	0.686	0.0001	0.7103	0	0.0001	0.7103
MODAL-X	2	0.46	0.7596	0.0001	0	0.7596	0.7104
MODAL-X	3	0.328	0.0015	4.698E-05	0	0.7612	0.7104
MODAL-X	4	0.162	1.264E-05	0.1614	0	0.7612	0.8719
MODAL-X	5	0.115	0.1676	4.983E-06	0	0.9288	0.8719
MODAL-X	6	0.106	6.003E-07	0.0018	0	0.9288	0.8737
MODAL-X	7	0.088	1.316E-05	0.0573	0	0.9288	0.9309
MODAL-X	8	0.069	0.0023	0.0295	0	0.931	0.9604
MODAL-X	9	0.069	0.0565	0.0014	0	0.9876	0.9618
MODAL-X	10	0.065	0.0001	3.869E-05	0	0.9876	0.9619
MODAL-X	11	0.058	0.0006	0.0002	0	0.9882	0.9621
MODAL-X	12	0.053	0.0019	0	0	0.9901	0.9621
MODAL-X	13	0.044	0.0015	0.001	0	0.9917	0.9631
MODAL-X	14	0.041	0.0061	0.0003	0	0.9977	0.9634
MODAL-X	15	0.035	0.0015	0.0004	0	0.9992	0.9638
MODAL-X	16	0.031	0.0006	0.0002	0	0.9998	0.964
MODAL-X	17	0.022	0.0002	0.0001	0	1	0.9641
MODAL-X	18	0.016	1.883E-05	0.0001	0	1	0.9642

Figure 5.16: Modal participating mass ratios in the X-direction for the building structure

Case	Mode	Period sec	UX	UY	UZ	SumUX	SumUY
MODAL-Y	1	0.686	0.0001	0.7103	0	0.0001	0.7103
MODAL-Y	2	0.46	0.7596	0.0001	0	0.7596	0.7104
MODAL-Y	3	0.328	0.0015	4.698E-05	0	0.7612	0.7104
MODAL-Y	4	0.162	1.264E-05	0.1614	0	0.7612	0.8719
MODAL-Y	5	0.115	0.1676	4.983E-06	0	0.9288	0.8719
MODAL-Y	6	0.106	6.003E-07	0.0018	0	0.9288	0.8737
MODAL-Y	7	0.088	1.316E-05	0.0573	0	0.9288	0.9309
MODAL-Y	8	0.069	0.0021	0.0294	0	0.9309	0.9603
MODAL-Y	9	0.069	0.0567	0.0014	0	0.9876	0.9617
MODAL-Y	10	0.064	1.199E-05	0.0001	0	0.9876	0.9618
MODAL-Y	11	0.059	9.287E-06	0.0115	0	0.9876	0.9734
MODAL-Y	12	0.053	0	0.0169	0	0.9876	0.9903
MODAL-Y	13	0.048	3.135E-05	0.0055	0	0.9877	0.9958
MODAL-Y	14	0.044	6.213E-07	0.0027	0	0.9877	0.9985
MODAL-Y	15	0.038	2.594E-05	0.0007	0	0.9877	0.9993
MODAL-Y	16	0.031	2.351E-06	0.0006	0	0.9877	0.9999
MODAL-Y	17	0.023	5.169E-06	0.0001	0	0.9877	1
MODAL-Y	18	0.015	8.36E-06	1.198E-05	0	0.9877	1

Figure 5.17: Modal participating mass ratios in the Y-direction for the building structure

The cumulative modal mass participation ratios in **both the X and Y directions exceeded 90%**, confirming that the **number of modes considered in the analysis is adequate** in accordance with ASCE 7-16.

5.4.2 Base shear

The base shear resulting from the static seismic load case in the software analysis is compared with the manually calculated value to ensure that the ETABS result is trustworthy. This check is crucial, as base shear is the core of seismic design outputs. Manual calculations are based on **ASCE 7-16**. Section 12.8 provides the guidelines for calculating the seismic base shear V for the **equivalent static analysis method** (Equivalent Lateral Force (ELF) Procedure). The base shear V can be calculated using Equation 12.8-1, as follows:

$$V = C_s \cdot W \quad \text{(ASCE 7-16, Eq. 12.8-1)}$$

Where:

- C_s = the seismic response coefficient.
- W = the effective seismic weight (defined in mass-source in the ETABS model)

The effective seismic weight for the building includes the full dead load, full superimposed load, and a quarter of the live load, as defined in the mass-source section of the ETABS model. Therefore, it can be expressed as:

$$W = SD + D + 0.25\ L$$

Referring to **Figure 5.18** (ETABS output table), which shows the net load in the building for each load type, the effective seismic weight *W* can be calculated as:

$$18533+50829.4+(0.25x\ 14556.3)=73001.5\ KN$$

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN	FZ kN
Dead	LinStatic			9.746E-07	0	50829.3612
Live	LinStatic			0	0	14556.3182
sD	LinStatic			0	0	18532.9007

Figure 5.18: Total dead, superimposed, and live loads in the building

In order to calculate the seismic response coefficient C_s, a set of previously discussed parameters is required. A quick reference for these parameters is summarized in **Table 5.1**. Detailed calculations for each parameter can be found in their respective sections.

Table 5.1: Summary of parameters relevant to C_s calculations

Parameter	Value
R	5
I_e	1
TL	8
S_1	0.14
S_{D1}	0.14
S_{DS}	0.504
C_t[7]	0.0488
x	0.75
h_n	29.65

[7] The parameters C_t and x, used in the approximate period calculation, were not discussed in Chapter 4. These values are taken from ASCE 7-16 Table 12.8-2, which is provided in Appendix E.

The seismic response coefficient C_s is calculated using the following equation:

$$C_s = \frac{S_{DS}}{R/I_e} \quad \text{(ASCE 7-16, Eq. 12.8-2)}$$

$$= \frac{0.504}{5/1} = 0.1008$$

Where:

- S_{DS} = the design spectral response acceleration parameter in the short period range. Calculated previously in section 4.5.
- R = the response modification factor. Determined previously in section 4.6.
- I_e = the Importance Factor. Determined previously in section 4.5.

According to the requirements of the same section, the value of C_s computed shall not exceed a specific value depending on the relationship between the fundamental period of the structure T and the long-period transition period TL (i.e., $T \leq TL$ or $T > TL$).

For $T \leq TL$ (governing limit, which applies in the project case)::

$$C_s \leq \frac{S_{D1}}{T \cdot (R/I_e)} \quad \text{(ASCE 7-16, Eq. 12.8-3)}$$

In the **X-direction:**

$$C_{s,max} = \frac{0.14}{0.46(5/1)} = 0.061$$

In the **Y-direction:**

$$C_{s,max} = \frac{0.14}{0.686(5/1)} = 0.041 \text{ in } \textbf{Y direction}$$

The determination of period T used in the calculation, as per **Section 12.8.2**, must comply with the following limit:

> *"The fundamental period, T, shall not exceed the product of the coefficient for upper limit on calculated period (C_u) from Table 12.8-1 and the approximate fundamental period, T_a, determined in accordance with Section 12.8.2.1."*

This means the analysis-based period (e.g., from ETABS) can be used **only if**:

$$T_{ETABS} \leq C_u \cdot T_a$$

Approximate period from **ASCE 7-16, Equation 12.8-7**:

$$T_a = C_t \cdot h_n^x$$

$$T_a = C_t h_n^x \qquad \text{(ASCE 7-16 Eq. 12.8-7)}$$

$$= 0.0488 \text{ x } 29.65^{0.75} = 0.62 \text{ sec}$$

Upper limit on the period:

$$C_u . T_a = 1.62 \; . \; 0.62 = 1.004 \text{ sec}$$

Where:

- h_n: the structural height.
- C_t and x: approximate period parameters (See **Appendix D**)
- C_u : coefficient for upper limit on calculated period, based on ASCE 7-16 Table 12.8-1 provided in **Appendix D**.

ETABS analysis results (From Figure 5.17, corresponding to maximum UX and UY):

- X-direction: T=0.46 sec
- Y-direction: T=0.686 sec

Since both values are less than 1.004 sec, the ETABS values **are valid** and used.

Governing C_s Value:

- In X-direction:

  Calculated C_s exceeds the upper limit:

  $C_s = 1.004 > C_{s,max} = 0.061 \rightarrow C_s = 0.061$ governs

- In Y-direction: 0.041 < 1.008 (**not satisfied**) → 0.041 governs

  Similarly:

  $C_s = 1.004 > C_{s,max} = 0.041 \rightarrow C_s = 0.041$ governs

Minimum C_s Check:

According to the lower bound requirement in **Section 12.8.1.1**, the seismic coefficient must not be less than:

$$C_s = 0.044 \cdot S_{DS} \cdot I_e \geq 0.01 \quad \text{(ASCE 7-16, Eq. 12.8-5)}$$

0.044 x 0.504 x 1 = 0.022 (>0.01)[8]

- **X-direction**: 0.061 > 0.022 → ✓ Satisfied
- **Y-direction**: 0.041 > 0.022 → ✓ Satisfied

Substituting W= 73001.5 KN, C_s= 0.061 (X-direction), and C_s= 0.041 (Y-direction) into the base shear equation **(ASCE 7-16, Eq. 12.8-1)**:

- X-direction:

  V= 0.061 x 73001.5 = 4453 KN

  Difference percentage = $(\frac{4453}{4448} - 1) * 100\% = 0.11\% < 2.0\%$ ✔

  (**Figure 5.19** presents the base reactions in both directions, as obtained from ETABS.)

- Y-direction:

  V= 0.041 x 73001.5 = 2933 KN

  Difference percentage=$(\frac{2993}{2978} - 1) * 100\% = 0.5\% < 2.0\%$ ✔

  (**Figure 5.19** presents the base reactions in both directions, as obtained from ETABS.)

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN
EX	LinStatic	Step By Step	1	-4447.8379	-893.3148
EX	LinStatic	Step By Step	2	-4447.8379	-893.3148
EX	LinStatic	Step By Step	3	-4447.8379	-893.3148
EY	LinStatic	Step By Step	1	-1334.3514	-2977.7161
EY	LinStatic	Step By Step	2	-1334.3514	-2977.7161
EY	LinStatic	Step By Step	3	-1334.3514	-2977.7161

Figure 5.19: Base reactions from static load cases EX and EY (Obtained from ETABS)

These results confirm that the difference between the manually calculated base shear and the ETABS output is within acceptable limits.

[8] The alternative minimum requirement (i.e., if $S_1 \geq 0.6g$) does **not** apply here, as it is not satisfied in this case.

5.4.3 Scaling

In ETABS dynamic analysis, if the calculated base shear is less than the static base shear obtained from the equivalent lateral force procedure, it is necessary to scale the dynamic base shear to the static value as mandated by ASCE 7-16 (Section 12.9.1.4).

Figure 5.20 indicates an exceedance of base shear in the static procedure compared to the dynamic response spectrum analysis, necessitating a correction by scaling. Scale factors were calculated and applied in both the EXD and EYD directions as follows:

In X-direction:

$$\frac{4448}{3759} = 1.183$$

In Y-direction :

$$\frac{2978}{2555} = 1.165$$

Adjusted dynamic accelerations:

- Acceleration U1= 1.183 x 1962 = 2322
- Acceleration U2= 1.165 x 1962 = 2287
- 30% of acceleration U1= 2322 x 0.3 = 697
- 30% of acceleration U2= 2287 x 0.3 = 686

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN
EX	LinStatic	Step By Step	1	-4447.8379	-893.3148
EX	LinStatic	Step By Step	2	-4447.8379	-893.3148
EX	LinStatic	Step By Step	3	-4447.8379	-893.3148
EY	LinStatic	Step By Step	1	-1334.3514	-2977.7161
EY	LinStatic	Step By Step	2	-1334.3514	-2977.7161
EY	LinStatic	Step By Step	3	-1334.3514	-2977.7161
EXD	LinRespSpec	Max		4448.6319	942.3521
EYD	LinRespSpec	Max		1388.326	2978.3866

Figure 5.20: Static vs. dynamic base shear by software analysis

Figures 5.21 and **5.22** provide the dynamic load case definition in ETABS after applying the required scaling adjustment.

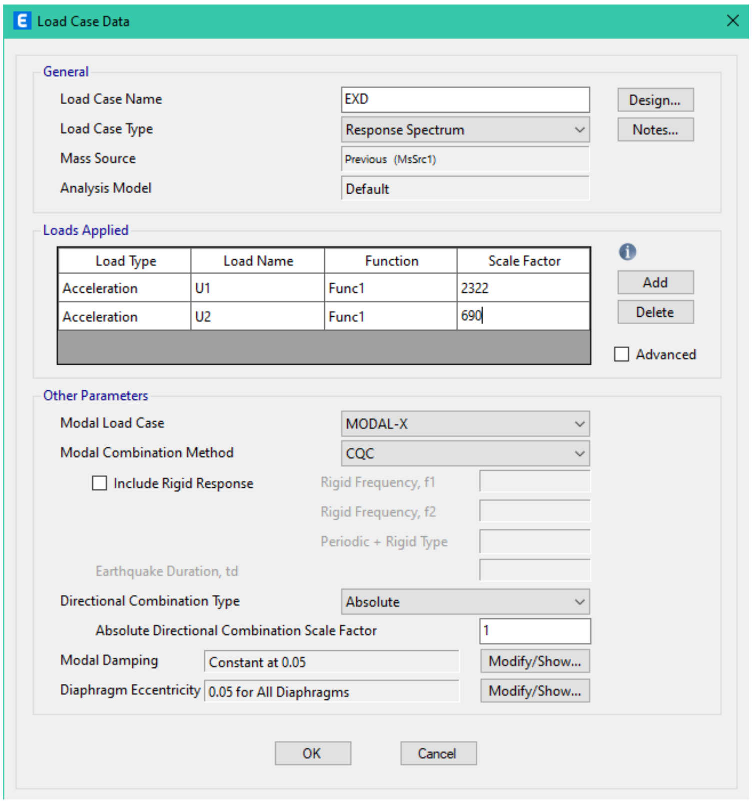


Figure 5.21: Scale factor adjustment in the dynamic load case in X

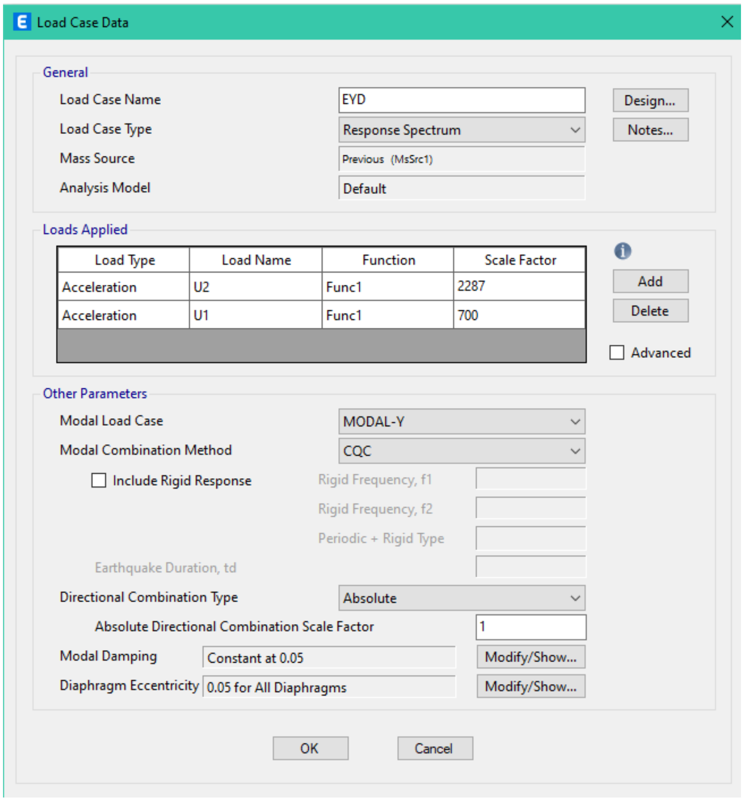


Figure 5.22: Scale factor adjustment in the dynamic load case in Y

5.5 Participation check

The participation of the members not supposed to be part of the seismic force resisting system should be minimized or diminished. The model is checked to estimate its convergence to the actual structural behaviour, applying any modifications needed.

5.5.1 Implementation and results

A section cut was placed at the base level joint to determine the lateral load contribution of the structural elements. As previously discussed, the structural system employed is a bearing wall system; thus, the shear walls are expected to resist the majority of seismic forces in both the X and Y directions.

Figures 5.23 and **5.24** present the total base shear resisted by all structural elements and the portion resisted by walls only under the **EY** seismic load case. Similarly, **Figures 5.25** and **5.26** illustrate the corresponding results for the **EX** seismic load case.

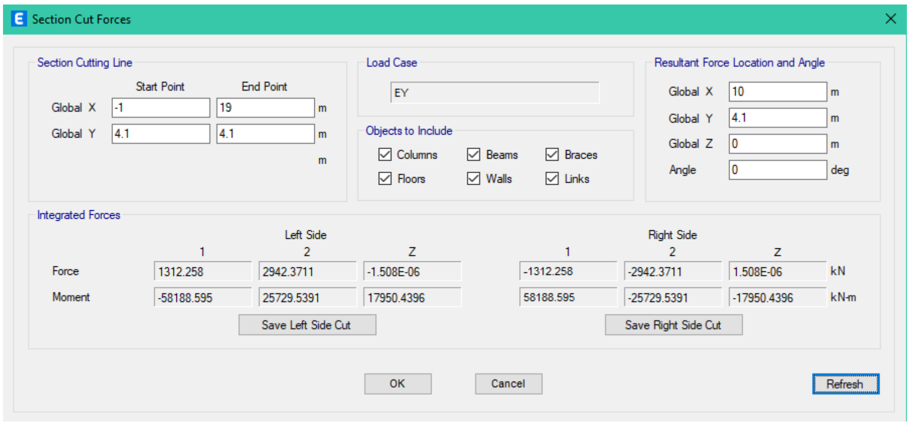


Figure 5.23: Base shear resisted by all structural elements under the EY seismic load case

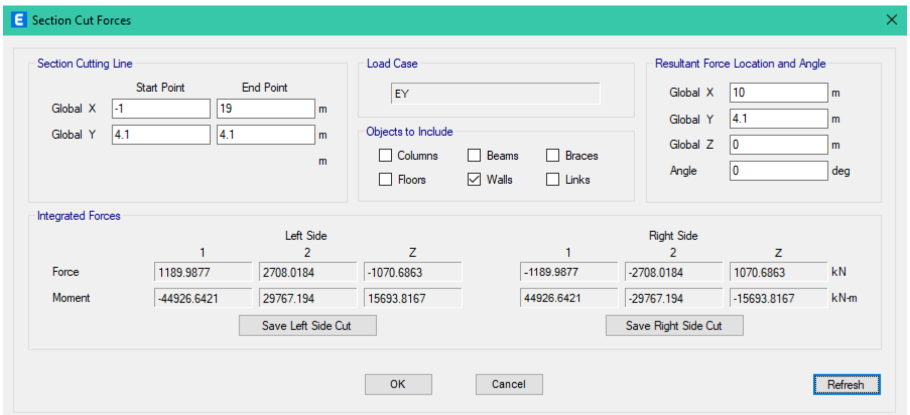


Figure 5.24: Base shear resisted by walls under the EY seismic load case

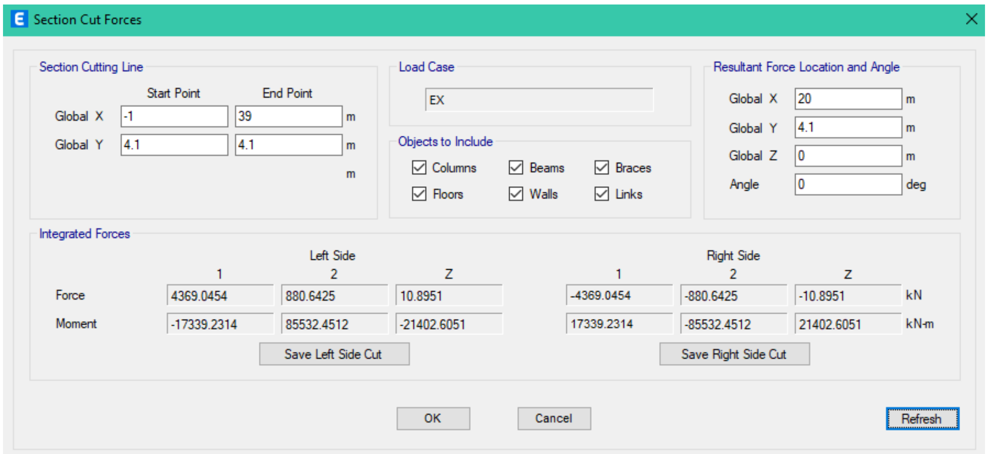


Figure 5.25: Base shear resisted by all structural elements under the EX seismic load case

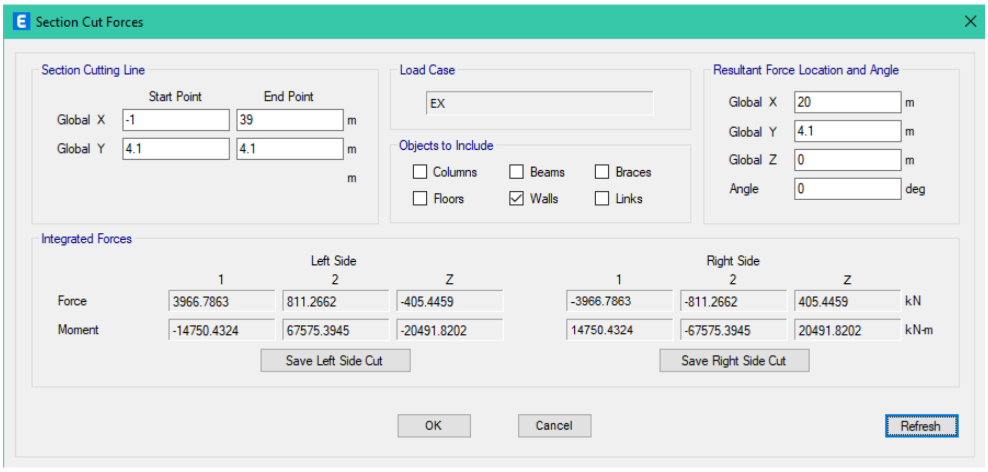


Figure 5.26: Base shear resisted by walls under the EX seismic load case

The obtained values confirm the predominance of shear walls in resisting lateral forces, where:

- Y-direction:

$$\frac{2708}{2942} \text{ x } 100 = \mathbf{92.046\ \%}$$

- X-direction:

$$\frac{3967}{4369} \text{ x } 100 = \mathbf{90.799\%}$$

5.5.2 Modifications

Although these percentages demonstrate that the majority of the lateral load is already carried by the walls, a further modification was introduced to isolate their contribution and obtain a more representative assessment. The ETABS model was adjusted to eliminate the lateral force-resisting role of the frames by assigning moment releases—specifically in Moment 22 and Moment 33—to all columns. This effectively prevents the columns from resisting lateral forces, thereby transferring the entire lateral demand to the shear walls.

Figures 5.27–5.28 illustrate the applied moment releases in the ETABS model.

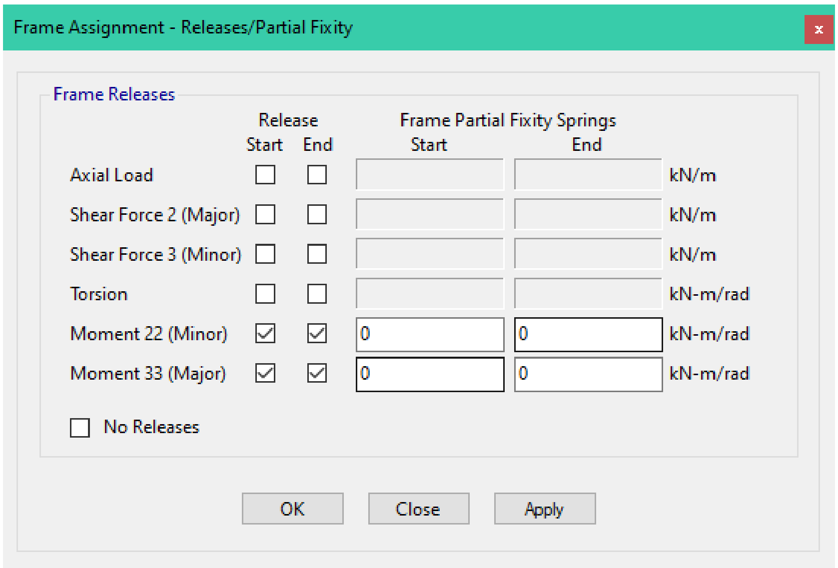


Figure 5.27: Assignment of column releases in ETABS

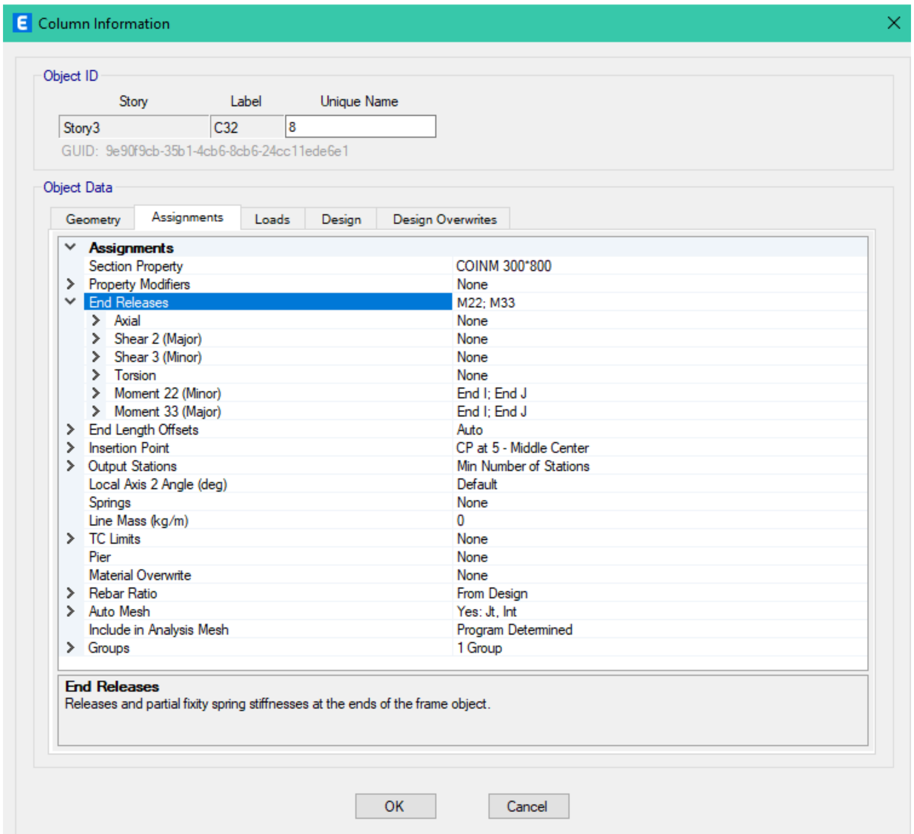


Figure 5.28: Assignments of a random column showing releases

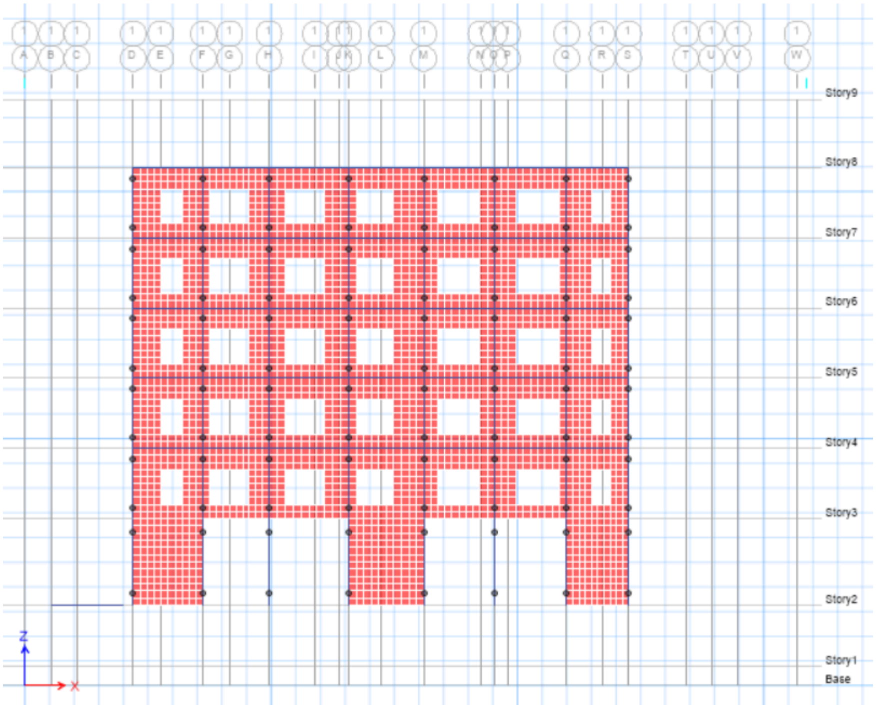


Figure 5.29: Screenshot showing column releases along a random gridline

5.5.3 Final result

The obtained results after applying the moment releases confirm a near-perfect convergence in total base shear between the “walls only” and “all structural members” cases—indicating that the entire lateral load is now effectively resisted by the walls. This outcome validates the intended modeling adjustment.

Figures 5.30-5.33 present the corresponding base shear values for both the EX and EY seismic load cases.

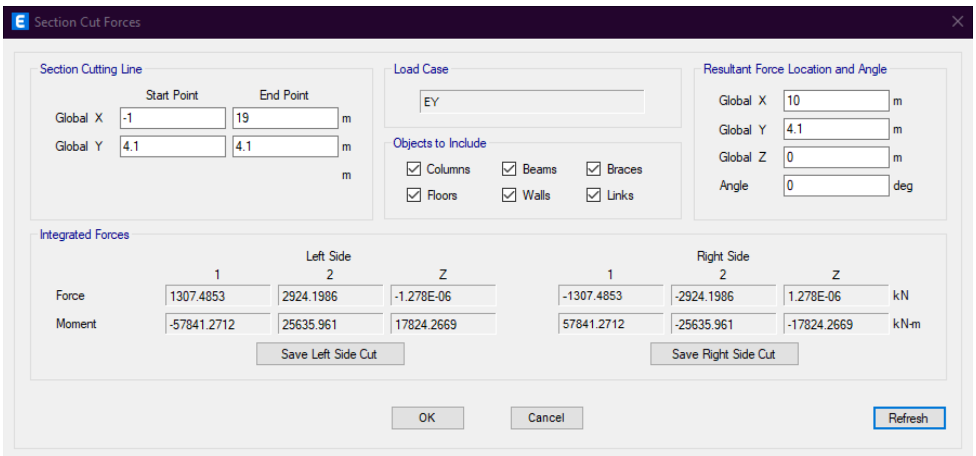


Figure 5.30: Base shear resisted by all structural elements under the EY seismic load case

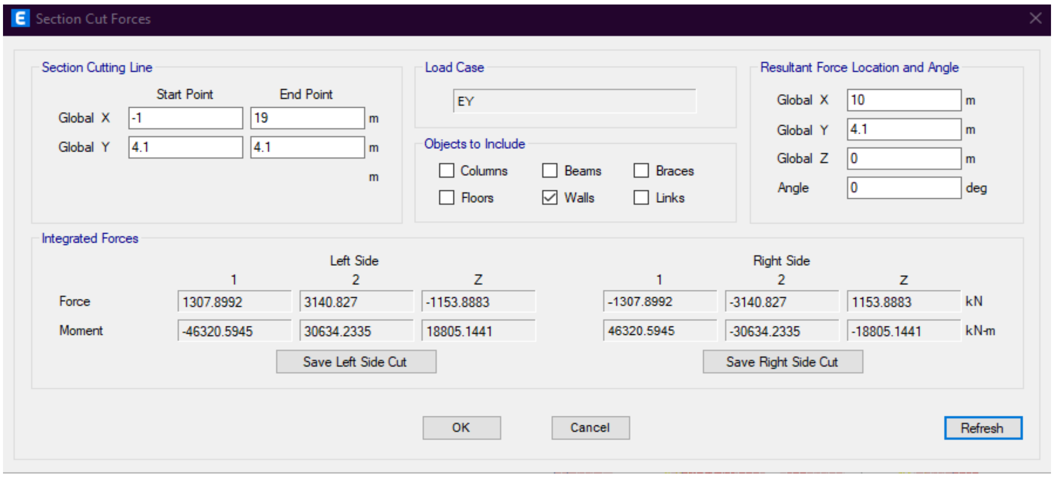


Figure 5.31: Base shear resisted by walls under the EY seismic load case

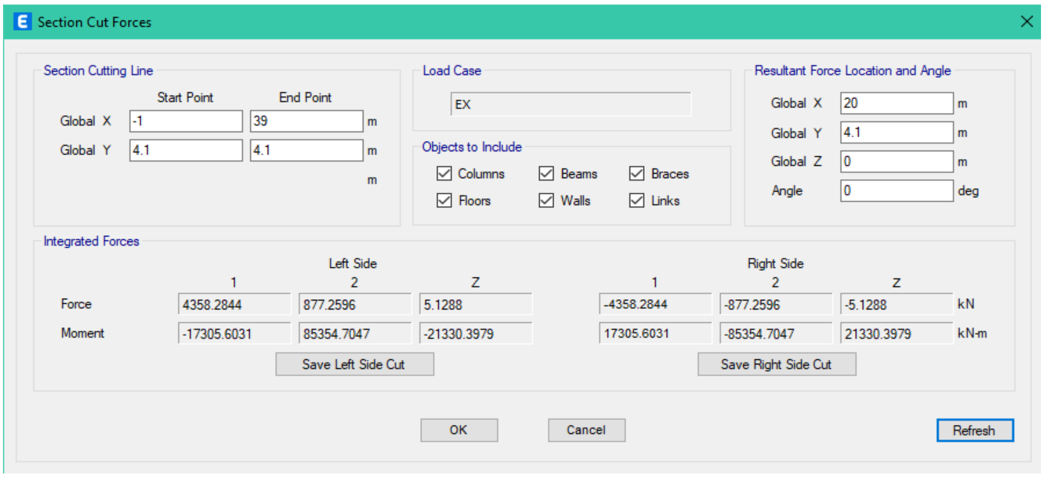


Figure 5.32: Base shear resisted by all structural elements under the EX seismic load case

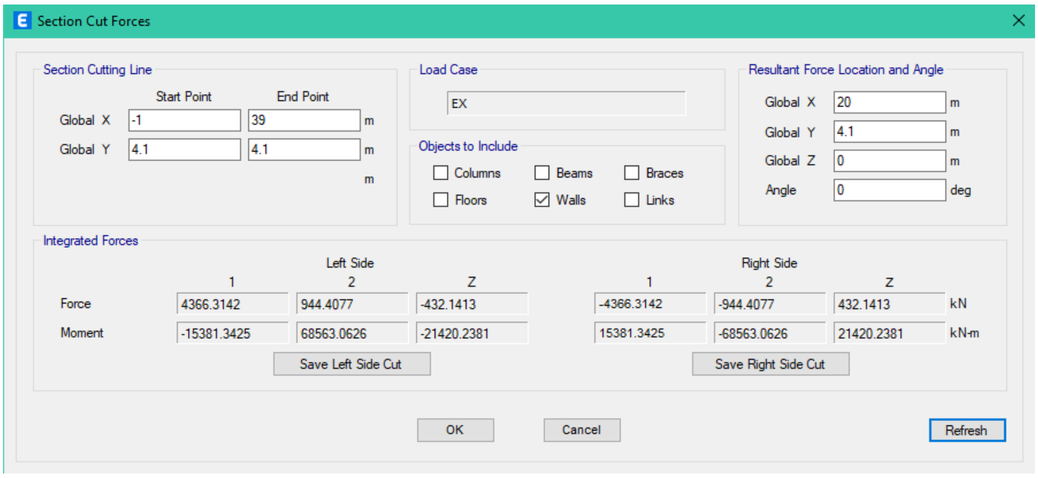


Figure 5.31: Base shear resisted by walls under the EX seismic load case

The values of base shears indicate that the structural walls are effectively resisting the majority of the seismic forces acting on the building. This confirms the efficiency of the bearing wall system in lateral load resistance, as intended in the structural design..

5.6 Check irregularities

According to seismic design provisions, when checking for structural irregularities, 30% of the seismic load in the opposite direction must be excluded. This ensures that the effects of accidental torsion and asymmetrical behavior are properly considered.

Figures 5.32 and **5.33** show the new EX and EY for the check.

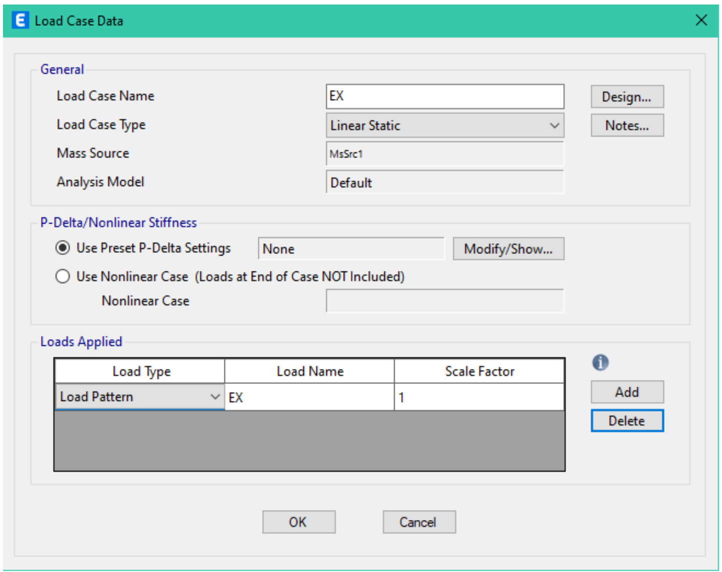


Figure 5.32: Adjusted EX for irregularity check

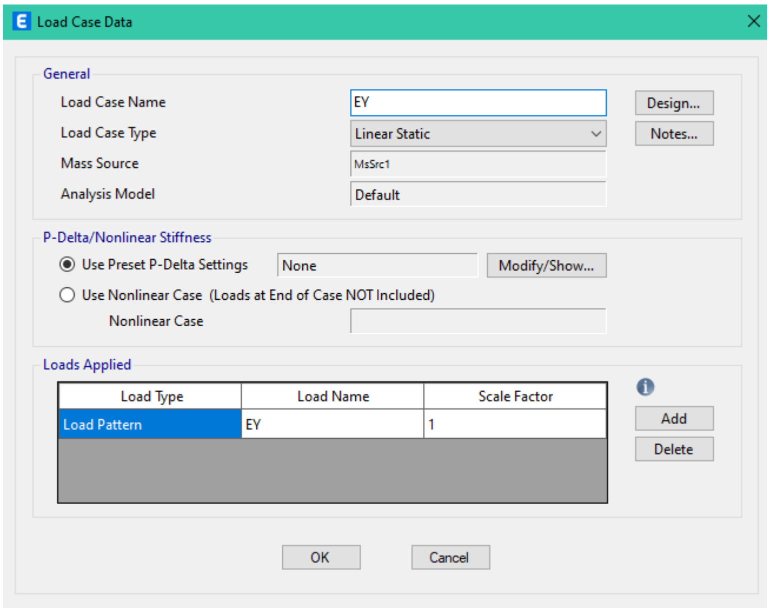


Figure 5.33: Adjusted EY for irregularity check

define combination , exclude 30% of other directions.

5.6.1 Horizontal structural irregularity

Horizontal Irregularity: Structures that have one or more of the irregularity types listed in Table 12.3-1 shall be designated as having a horizontal structural irregularity. Such structures assigned to the Seismic Design Categories listed in Table 12.3-1 shall comply with the requirements in the sections referenced in those tables, **Figure 5.34** shows the horizontal structural irregularity .

Table 12.3-1 Horizontal Structural Irregularities

Type	Description	Reference Section	Seismic Design Category Application
1a.	**Torsional Irregularity:** Torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_x = 1.0$, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 16.3.4	D, E, and F B, C, D, E, and F C, D, E, and F C, D, E, and F D, E, and F B, C, D, E, and F
1b.	**Extreme Torsional Irregularity:** Extreme torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_x = 1.0$, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.1 12.3.3.4 12.3.4.2 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 16.3.4	E and F D D B, C, and D C and D C and D D B, C, and D
2.	**Reentrant Corner Irregularity:** Reentrant corner irregularity is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
3.	**Diaphragm Discontinuity Irregularity:** Diaphragm discontinuity irregularity is defined to exist where there is a diaphragm with an abrupt discontinuity or variation in stiffness, including one that has a cutout or open area greater than 50% of the gross enclosed diaphragm area, or a change in effective diaphragm stiffness of more than 50% from one story to the next.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
4.	**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.	12.3.3.3 12.3.3.4 12.7.3 Table 12.6-1 16.3.4	B, C, D, E, and F D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F
5.	**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.	12.5.3 12.7.3 Table 12.6-1 16.3.4	C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F

Figure 5.34: Horizontal irregularities (ASCE 7-16, Table 12.3-1)

5.6.1.1 Type 1.a: Torsional irregularity

Check horizontal irregularities in the building structure (Torsional irregularity)

Torsional Irregularity: Torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with Ax = 1.0, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semi rigid.

In determining the torsional irregularities , the applied horizontal forces EX and EY shall consider the accidental eccentricity of ±0.05 and shall be aligned in the considered direction only without a ratio of the force in the perpendicular direction.

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2 \qquad (ASCE\ 7-16\ equation\ 12.8-14) \quad (5.1)$$

1.293 > 1.2 in X-Direction (Story3)

1.309 > 1.2 in Y-Direction (Story3)

$A_x = (\delta max/\delta avg)^2 = (1.073/1.2)^2 = 0.8$ in X-Direction

$A_x = (\delta max/\delta avg)^2 = (1.112/1.2)^2 = 0.86$ in Y-Direction

Figure 5.35 and **5.36** illustrate the Diaphragm Max. Over Avg Drift for both X and Y direction.

Story	Output Case	Case Type	Step Type	Step Number	Item	Max Drift	Avg Drift	Ratio	Label	Max Loc X m	Max Loc Y m	Max Loc Z m
Story8	EX	LinStatic	Step By Step	1	Diaph D6 X	0.00018	0.000177	1.016	3711	33.125	18.1	26.2
Story8	EX	LinStatic	Step By Step	2	Diaph D6 X	0.000183	0.000178	1.031	3711	33.125	18.1	26.2
Story8	EX	LinStatic	Step By Step	3	Diaph D6 X	0.000177	0.000177	1.002	3711	33.125	18.1	26.2
Story7	EX	LinStatic	Step By Step	1	Diaph D5 X	0.000197	0.000194	1.014	95	33.55	18.1	22.65
Story7	EX	LinStatic	Step By Step	2	Diaph D5 X	0.000201	0.000195	1.033	95	33.55	18.1	22.65
Story7	EX	LinStatic	Step By Step	3	Diaph D5 X	0.000195	0.000194	1.005	1192	30.335	0	22.65
Story6	EX	LinStatic	Step By Step	1	Diaph D4 X	0.000214	0.000212	1.01	95	33.55	18.1	19.1
Story6	EX	LinStatic	Step By Step	2	Diaph D4 X	0.000219	0.000212	1.032	95	33.55	18.1	19.1
Story6	EX	LinStatic	Step By Step	3	Diaph D4 X	0.000214	0.000212	1.012	1192	30.335	0	19.1
Story5	EX	LinStatic	Step By Step	1	Diaph D3 X	0.000228	0.000228	1.004	95	33.55	18.1	15.55
Story5	EX	LinStatic	Step By Step	2	Diaph D3 X	0.000234	0.000228	1.028	95	33.55	18.1	15.55
Story5	EX	LinStatic	Step By Step	3	Diaph D3 X	0.000232	0.000227	1.02	1192	30.335	0	15.55
Story4	EX	LinStatic	Step By Step	1	Diaph D2 X	0.000241	0.000239	1.011	1192	30.335	0	12
Story4	EX	LinStatic	Step By Step	2	Diaph D2 X	0.000242	0.000239	1.013	3711	33.125	18.1	12
Story4	EX	LinStatic	Step By Step	3	Diaph D2 X	0.000247	0.000239	1.035	1192	30.335	0	12
Story3	EX	LinStatic	Step By Step	1	Diaph D1 X	0.000391	0.000303	1.288	252	39.205	10.75	8.45
Story3	EX	LinStatic	Step By Step	2	Diaph D1 X	0.000392	0.000306	1.284	252	39.205	10.75	8.45
Story3	EX	LinStatic	Step By Step	3	Diaph D1 X	0.000389	0.000301	1.293	252	39.205	10.75	8.45
Story2	EX	LinStatic	Step By Step	1	Diaph D7 X	0.000138	0.000123	1.115	3652	34.85	7.4964	4.05
Story2	EX	LinStatic	Step By Step	2	Diaph D7 X	0.000137	0.000123	1.111	3652	34.85	7.4964	4.05
Story2	EX	LinStatic	Step By Step	3	Diaph D7 X	0.000138	0.000123	1.12	3652	34.85	7.4964	4.05

Figure 5.35: Diaphragm Max. Over Avg Drift X-direction

Story	Output Case	Case Type	Step Type	Step Number	Item	Max Drift	Avg Drift	Ratio	Label	Max Loc X m	Max Loc Y m	Max Loc Z m
Story8	EY	LinStatic	Step By Step	1	Diaph D6 Y	0.000325	0.000323	1.006	262	36.2	0.55	26.2
Story8	EY	LinStatic	Step By Step	2	Diaph D6 Y	0.000333	0.000324	1.028	262	36.2	0.55	26.2
Story8	EY	LinStatic	Step By Step	3	Diaph D6 Y	0.000328	0.000323	1.017	265	0	0.55	26.2
Story7	EY	LinStatic	Step By Step	1	Diaph D5 Y	0.000354	0.000352	1.007	262	36.2	0.55	22.65
Story7	EY	LinStatic	Step By Step	2	Diaph D5 Y	0.000365	0.000352	1.035	262	36.2	0.55	22.65
Story7	EY	LinStatic	Step By Step	3	Diaph D5 Y	0.000359	0.000351	1.022	265	0	0.55	22.65
Story6	EY	LinStatic	Step By Step	1	Diaph D4 Y	0.000378	0.000377	1.004	262	36.2	0.55	19.1
Story6	EY	LinStatic	Step By Step	2	Diaph D4 Y	0.000391	0.000377	1.038	262	36.2	0.55	19.1
Story6	EY	LinStatic	Step By Step	3	Diaph D4 Y	0.000387	0.000376	1.03	265	0	0.55	19.1
Story5	EY	LinStatic	Step By Step	1	Diaph D3 Y	0.000391	0.000391	1.002	265	0	0.55	15.55
Story5	EY	LinStatic	Step By Step	2	Diaph D3 Y	0.000405	0.000391	1.036	262	36.2	0.55	15.55
Story5	EY	LinStatic	Step By Step	3	Diaph D3 Y	0.000406	0.00039	1.039	265	0	0.55	15.55
Story4	EY	LinStatic	Step By Step	1	Diaph D2 Y	0.000388	0.000384	1.01	265	0	0.55	12
Story4	EY	LinStatic	Step By Step	2	Diaph D2 Y	0.000396	0.000384	1.03	262	36.2	0.55	12
Story4	EY	LinStatic	Step By Step	3	Diaph D2 Y	0.000403	0.000384	1.05	265	0	0.55	12
Story3	EY	LinStatic	Step By Step	1	Diaph D1 Y	0.000494	0.000389	1.268	78	16.4375	0	8.45
Story3	EY	LinStatic	Step By Step	2	Diaph D1 Y	0.000514	0.000405	1.269	251	39.205	17	8.45
Story3	EY	LinStatic	Step By Step	3	Diaph D1 Y	0.000513	0.000392	1.309	75	5.5	0	8.45
Story2	EY	LinStatic	Step By Step	1	Diaph D7 Y	0.000198	0.000187	1.059	3652	34.85	7.4964	4.05
Story2	EY	LinStatic	Step By Step	2	Diaph D7 Y	0.000205	0.000192	1.067	3652	34.85	7.4964	4.05
Story2	EY	LinStatic	Step By Step	3	Diaph D7 Y	0.00019	0.000181	1.051	3652	34.85	7.4964	4.05

Figure 5.36: Diaphragm Max. Over Avg Drift Y-direction

Both Figures 5.37 and 5.38 show the story max. over avg displacement X and Y -direction.

Story	Output Case	Case Type	Step Type	Step Number	Step Label	Direction	Maximum mm	Average mm	Ratio
Story8	EX	LinStatic	Step By Step	1		X	5.524	5.475	1.009
Story7	EX	LinStatic	Step By Step	1		X	4.905	4.846	1.012
Story6	EX	LinStatic	Step By Step	1		X	4.225	4.156	1.017
Story5	EX	LinStatic	Step By Step	1		X	3.48	3.403	1.023
Story4	EX	LinStatic	Step By Step	1		X	2.675	2.595	1.031
Story3	EX	LinStatic	Step By Step	1		X	1.818	1.748	1.04
Story2	EX	LinStatic	Step By Step	1		X	0.642	0.601	1.068
Story8	EX	LinStatic	Step By Step	2		X	5.563	5.476	1.016
Story7	EX	LinStatic	Step By Step	2		X	4.913	4.846	1.014
Story6	EX	LinStatic	Step By Step	2		X	4.199	4.155	1.011
Story5	EX	LinStatic	Step By Step	2		X	3.42	3.4	1.006
Story4	EX	LinStatic	Step By Step	2		X	2.594	2.592	1.001
Story3	EX	LinStatic	Step By Step	2		X	1.758	1.744	1.008
Story2	EX	LinStatic	Step By Step	2		X	0.627	0.6	1.046
Story8	EX	LinStatic	Step By Step	3		X	5.658	5.473	1.034
Story7	EX	LinStatic	Step By Step	3		X	5.031	4.846	1.038
Story6	EX	LinStatic	Step By Step	3		X	4.339	4.157	1.044
Story5	EX	LinStatic	Step By Step	3		X	3.579	3.405	1.051
Story4	EX	LinStatic	Step By Step	3		X	2.755	2.598	1.06
Story3	EX	LinStatic	Step By Step	3		X	1.879	1.751	1.073
Story2	EX	LinStatic	Step By Step	3		X	0.657	0.603	1.09

Figure 5.37: Story Max. Over Avg Displacement X-direction

Story	Output Case	Case Type	Step Type	Step Number	Step Label	Direction	Maximum mm	Average mm	Ratio
Story8	EY	LinStatic	Step By Step	1		Y	8.747	8.678	1.008
Story7	EY	LinStatic	Step By Step	1		Y	7.605	7.53	1.01
Story6	EY	LinStatic	Step By Step	1		Y	6.364	6.281	1.013
Story5	EY	LinStatic	Step By Step	1		Y	5.032	4.944	1.018
Story4	EY	LinStatic	Step By Step	1		Y	3.643	3.558	1.024
Story3	EY	LinStatic	Step By Step	1		Y	2.268	2.185	1.038
Story2	EY	LinStatic	Step By Step	1		Y	0.854	0.718	1.189
Story8	EY	LinStatic	Step By Step	2		Y	8.942	8.672	1.031
Story7	EY	LinStatic	Step By Step	2		Y	7.764	7.523	1.032
Story6	EY	LinStatic	Step By Step	2		Y	6.473	6.273	1.032
Story5	EY	LinStatic	Step By Step	2		Y	5.088	4.934	1.031
Story4	EY	LinStatic	Step By Step	2		Y	3.655	3.547	1.03
Story3	EY	LinStatic	Step By Step	2		Y	2.271	2.193	1.036
Story2	EY	LinStatic	Step By Step	2		Y	0.922	0.754	1.222
Story8	EY	LinStatic	Step By Step	3		Y	9.092	8.683	1.047
Story7	EY	LinStatic	Step By Step	3		Y	7.929	7.537	1.052
Story6	EY	LinStatic	Step By Step	3		Y	6.656	6.29	1.058
Story5	EY	LinStatic	Step By Step	3		Y	5.283	4.954	1.066
Story4	EY	LinStatic	Step By Step	3		Y	3.847	3.569	1.078
Story3	EY	LinStatic	Step By Step	3		Y	2.421	2.178	1.112
Story2	EY	LinStatic	Step By Step	3		Y	0.786	0.682	1.153

Figure 5.38: Story Max. Over Avg Displacement Y-direction

5.6.1.2 Type 1.b Extreme Torsional Irregularity

Extreme Torsional Irregularity: Extreme torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_1 = 1.0$, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semi-rigid.

5.6.1.3 Type 2: Reentrant Corner Irregularity

Reentrant corner irregularity is defined to exist when both plan projections of the structure extending beyond a reentrant corner exceed 15% of the plan dimension of the structure in that direction. **Figure 5.39** illustrates the building layout and relevant dimensions used to assess this condition.

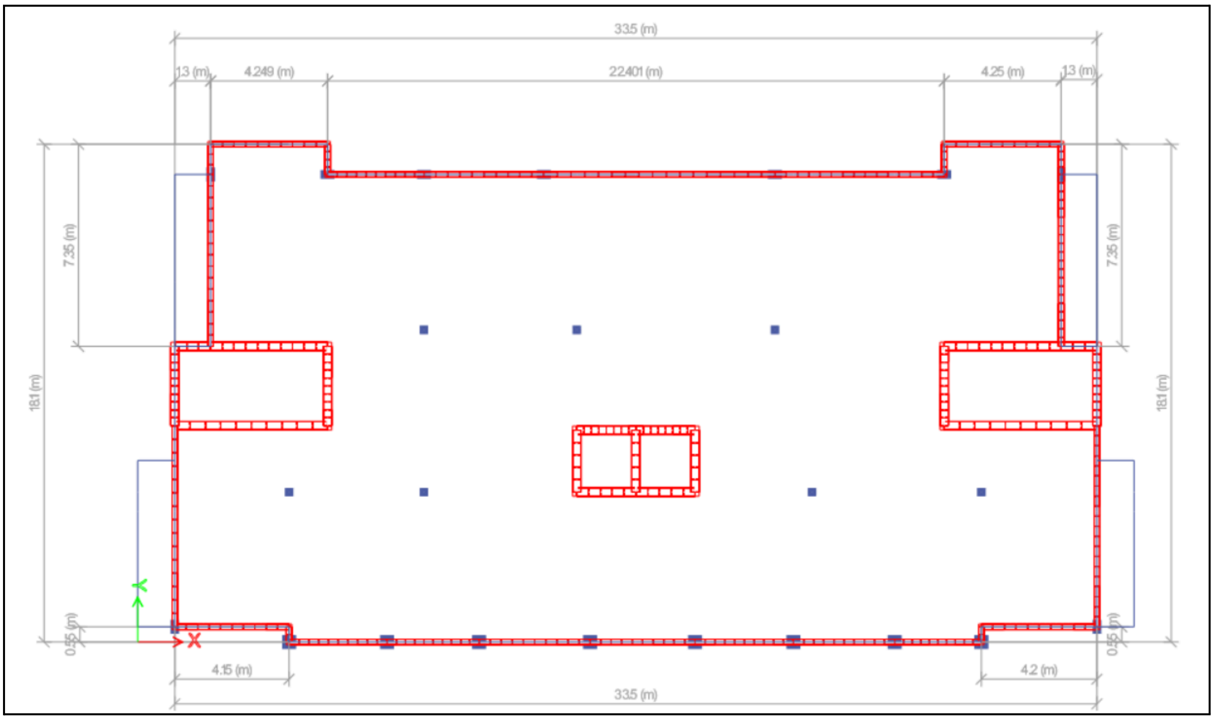


Figure 5.39: Building layout dimensions

In X direction - Ratio: 4.3/33.5= 0.128 < 0.15.

In Y direction - Ratio: 7.35/18.1= 0.41 > 0.15.

So, reentrant corner irregularity exists.

5.6.1.4 Type 3: Diaphragm Discontinuity Irregularity

Diaphragm discontinuity irregularity is defined to exist where there is a diaphragm with an abrupt discontinuity or variation in stiffness, including one that has a cutout or open area greater than 50% of the gross enclosed diaphragm area, or a change in effective diaphragm stiffness of more than 50% from one story to the next.

By inspection, this irregularity does not exist, here are openings. But less than 50%.

Figure 5.40 shows the building openings layout.

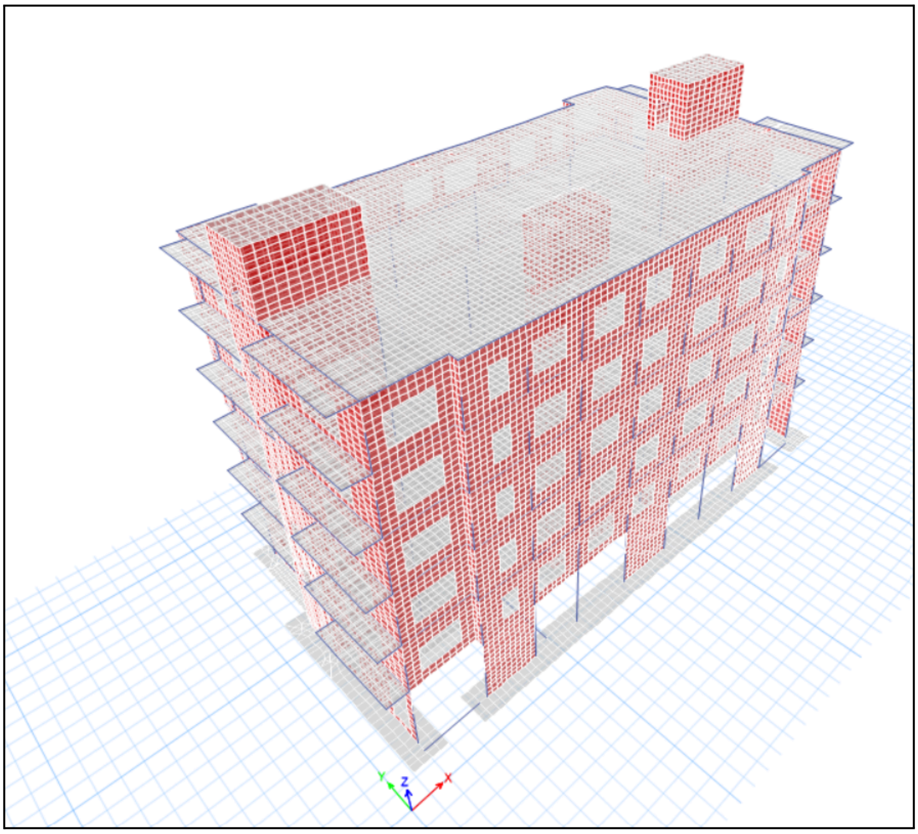


Figure 5.40: Building openings

5.6.1.5 Type 4: 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.

There is no discontinuity in the lateral forces system, all floors are symmetric in the direction of interest.

5.6.1.6 Type 5: Non Parallel 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.

Referring to the building layout, there is no inclined at the building, so no Non Parallel System Irregularity.

5.6.2 Forces and stiffness (Vertical irregularity)

ASCE 7-16 12.3.2.2 Vertical Irregularity: Structures that have one or more of the irregularity types listed in Table 12.3-2 shall be designated as having a vertical structural irregularity. Such structures assigned to the Seismic Design Categories listed in Table 12.3-2 shall comply with the requirements in the sections referenced in that table.

Figure 5.41 shows the vertical irregularities; ASCE 7-16 Table 12.3-2

Table 12.3-2 Vertical Structural Irregularities

Type	Description	Reference Section	Seismic Design Category Application
1a.	**Stiffness–Soft Story Irregularity:** Stiffness–soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.	Table 12.6-1	D, E, and F
1b.	**Stiffness–Extreme Soft Story Irregularity:** Stiffness–extreme soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.	12.3.3.1 Table 12.6-1	E and F D, E, and F
2.	**Weight (Mass) Irregularity:** Weight (mass) irregularity is defined to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	Table 12.6-1	D, E, and F
3.	**Vertical Geometric Irregularity:** Vertical geometric irregularity is defined to exist where the horizontal dimension of the seismic force-resisting system in any story is more than 130% of that in an adjacent story.	Table 12.6-1	D, E, and F
4.	**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.	12.3.3.3 12.3.3.4 Table 12.6-1	B, C, D, E, and F D, E, and F D, E, and F
5a.	**Discontinuity in Lateral Strength–Weak Story Irregularity:** Discontinuity in lateral strength–weak story irregularity is defined to exist where the story lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 Table 12.6-1	E and F D, E, and F
5b.	**Discontinuity in Lateral Strength–Extreme Weak Story Irregularity:** Discontinuity in lateral strength–extreme weak story irregularity is defined to exist where the story lateral strength is less than 65% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 12.3.3.2 Table 12.6-1	D, E, and F B and C D, E, and F

Figure 5.41: Vertical irregularities (ASCE 7-16, Table 12.3-2)

5.6.2.1 Type 1: Stiffness - Soft story

Stiffness–Soft Story Irregularity: "Stiffness–soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.

Figure 5.42 and 5.43 shows the Story stiffness in both X and Y directions.

Story	Output Case	Case Type	Step Type	Step Number	Shear X kN	Drift X mm	Stiff X kN/m	Shear Y kN	Drift Y mm	Stiff Y kN/m
Story9	EX	LinStatic	Step By Step	1	79.7754	0.641	124379.968	0	0.021	0
Story8	EX	LinStatic	Step By Step	1	1163.3164	0.66	1763885.662	0	0.031	0
Story7	EX	LinStatic	Step By Step	1	2090.1659	0.721	2899482.49	0	0.02	0
Story6	EX	LinStatic	Step By Step	1	2874.0711	0.779	3690686.683	0	0.019	0
Story5	EX	LinStatic	Step By Step	1	3512.2766	0.833	4214492.898	0	0.035	0
Story4	EX	LinStatic	Step By Step	1	4005.7274	0.847	4727083.28	0	0.013	0
Story3	EX	LinStatic	Step By Step	1	4366.4634	1.335	3271517.589	0	0.081	0
Story2	EX	LinStatic	Step By Step	1	911.4041	0.424	2147342.926	56.613	0.026	0
Story1	EX	LinStatic	Step By Step	1	0	0	0	0	0	0

Figure 5.42: Story stiffness X-direction

Story	Output Case	Case Type	Step Type	Step Number	Shear X kN	Drift X mm	Stiff X kN/m	Shear Y kN	Drift Y mm	Stiff Y kN/m
Story9	EY	LinStatic	Step By Step	1	0	0.006	0	55.6491	1.103	50456.759
Story8	EY	LinStatic	Step By Step	1	0	0.014	0	803.2764	1.19	675133.613
Story7	EY	LinStatic	Step By Step	1	0	0.007	0	1434.6108	1.285	1116297.663
Story6	EY	LinStatic	Step By Step	1	0	0.008	0	1960.5908	1.359	1443175.135
Story5	EY	LinStatic	Step By Step	1	0	0.014	0	2381.0967	1.357	1755213.02
Story4	EY	LinStatic	Step By Step	1	0	0.005	0	2698.8619	1.363	1980174.801
Story3	EY	LinStatic	Step By Step	1	0	0.059	0	2924.0716	1.713	1706744.165
Story2	EY	LinStatic	Step By Step	1	0.1446	0.027	0	504.0784	0.654	770347.171
Story1	EY	LinStatic	Step By Step	1	0	0	0	0	0	0

Figure 5.43: Story stiffness Y-direction

The stiffness irregularity is applied in both X and Y direction, the results and check are both shown in **Figures 5.44** and **5.45**.

Permitted analytical procedures are shown in **Figure 5.46**.

Stiffness Irregularity Check in X-Direction											
		Soft–Story Irregularity Check					Extreme Soft–Story Irregularity Check				
Story	Stiffness X–Direction	K_i / K_{i+1} 0.7	check	$K_{mi} = avg(K_{i-1,i-2,i-3})$	K_i / K_{mi} 0.8	check	K_i / K_{i+1} 0.6	check	$K_{mi} = avg(K_{i-1,i-2,i-3})$	K_i / K_{mi} 0.7	check
Story9	124379.97	-	-				-				
Story8	1763885.66	14.18	Regular				-				
Story7	2899482.49	1.64	Regular				-				
Story6	3690686.68	1.27	Regular	1595916.04	2.31	Regular	1.27	Regular	1595916.04	2.31	Regular
Story5	4214492.90	1.14	Regular	2784684.95	1.51	Regular	1.14	Regular	2784684.95	1.51	Regular
Story4	4727083.28	1.12	Regular	3601554.02	1.31	Regular	1.12	Regular	3601554.02	1.31	Regular
Story3	3271517.59	0.69	Soft Story	4210754.29	0.78	Soft Story	0.69	Regular	4210754.29	0.78	Regular
Story2	2147342.93	0.66	Soft Story	4071031.26	0.53	Soft Story	0.66	Regular	4071031.26	0.53	Ex-Soft Story

Figure 5.44: Stiffness irregularity check in X-direction

		Soft–Story Irregularity Check					Extreme Soft–Story Irregularity Check				
Story	**Stiffness Y–Direction**	$\frac{K_i}{K_{i+1}}$	**check**	$K_{ai} = avg(K_{i+1,i+2,i+3})$	$\frac{K_i}{K_{ai}}$	**check**	$\frac{K_i}{K_{i+1}}$	**check**	$K_{ai} = avg(K_{i+1,i+2,i+3})$	$\frac{K_i}{K_{ai}}$	**check**
		0.7			0.8		0.6			0.7	
Story9	50456.76	-	-				-				
Story8	675133.61	13.38	Regular				-				
Story7	1116297.66	1.65	Regular				-				
Story6	1443175.14	1.29	Regular	613962.68	2.35	Regular	1.29	Regular	613962.68	2.35	Regular
Story5	1755213.02	1.22	Regular	1078202.14	1.63	Regular	1.22	Regular	1078202.14	1.63	Regular
Story4	1980174.80	1.13	Regular	1438228.61	1.38	Regular	1.13	Regular	1438228.61	1.38	Regular
Story3	1706744.17	0.86	Regular	1726187.65	0.99	Regular	0.86	Regular	1726187.65	0.99	Regular
Story2	770347.17	0.45	Soft Story	1814044.00	0.42	Soft Story	0.45	Ex-Soft Story	1814044.00	0.42	Ex-Soft Story

Figure 5.45: Stiffness irregularity check in Y-direction

Table 12.6-1 Permitted Analytical Procedures

Seismic Design Category	Structural Characteristics	Equivalent Lateral Force Procedure, Section 12.8[a]	Modal Response Spectrum Analysis, Section 12.9.1, or Linear Response History Analysis, Section 12.9.2[a]	Nonlinear Response History Procedures, Chapter 16[a]
B, C	All structures	P	P	P
D, E, F	Risk Category I or II buildings not exceeding two stories above the base	P	P	P
	Structures of light-frame construction	P	P	P
	Structures with no structural irregularities and not exceeding 160 ft (48.8 m) in structural height	P	P	P
	Structures exceeding 160 ft (48.8 m) in structural height with no structural irregularities and with $T < 3.5T_s$	P	P	P
	Structures not exceeding 160 ft (48.8 m) in structural height and having only horizontal irregularities of Type 2, 3, 4, or 5 in Table 12.3-1 or vertical irregularities of Type 4, 5a, or 5b in Table 12.3-2	P	P	P
	All other structures	NP	P	P

[a]P: Permitted; NP: Not Permitted; $T_s = S_{D1}/S_{DS}$.

Figure 5.46: Permitted analytical procedures (ASCE 7-16, Table 12.6-1)

5.6.2.2 Type 2: Weight (Mass) Irregularity

Weight (mass) irregularity is defined to exist when the effective mass of any story exceeds 150% of the effective mass of an adjacent story. To evaluate this, a load combination of 1D + 1SD + 0.25L was used to represent the total structural weight (mass).

A roof that is lighter than the floor below need not be considered.

Tables 5.2 show the mass of the building:

Table 5.2: masses of each story

Story	Masses kg
Story8	1132183.739
Story7	990513.4736
Story6	990598.7919
Story5	990598.7919
Story4	994399.7236
Story3	1049118.478

Check story 3:

1049118.478 <1.5 *994399.7236

No weight (Mass) irregularity

Check story 4:

994399.7236 < 1.5*1049118.478

994399.7236 < 1.5*990598.7919

No weight (Mass) irregularity

Check story 5:

990598.7919 < 1.5*994399.7236

983280.22 < 1.5*990598.7919

No weight (Mass) irregularity

Check story 6:

990598.7919< 1.5*990598.7919

990598.7919 < 1.5*990513.4736

No weight (Mass) irregularity

Check story 7:

990513.4736< 1.5*990598.7919

990513.4736< 1.5*1132183.739

No weight (Mass) irregularity

Check story 8:

1132183.739 < 1.5*990513.4736

No weight (Mass) irregularity

5.6.2.3 Type 3: Vertical Geometric Irregularity

Vertical geometric irregularity is defined to exist where the horizontal dimension of the seismic force-resisting system in any story is more than 130% of that in an adjacent story.

From the layout of the building structure, there is no vertical geometric irregularity.

5.6.2.4 Type 4: 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.

From the layout of the building structure, there is no In-Plane Discontinuity.

5.6.2.5 Type 5: Discontinuity in Lateral Strength-Extreme Weak Story Irregularity

5.a Same as b but 80%

5.b Discontinuity in lateral strength-extreme weak story irregularity is defined to exist where the story lateral strength is less than 65% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.

If the checks show that there is no Type 5.a irregularity, thus, Type 5.b won’t exist.

5.7 P-delta

P-delta effects on the members internal forces and drifts can be neglected where the stability coefficient, θ, as calculated by ASCE 7-16 equation 12.8-16 is equal to or less than 0.1.

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d} \qquad ASCE\ 7-16\ equation\ 12.8-16 \quad (4.13)$$

ASCE 7-16 equation 12.8-16 (4.13)

Where:

P_x: total vertical design load at and above level x; where computing Px, no individual load factor need exceed 1.0.

Δ: design story drift as defined in ASCE 7-16 Section 12.8.6 occurring simultaneously with V_x.

I_e: importance factor determined in accordance with ASCE 7-16 Section 11.5.1.

V_x: seismic shear force acting between levels x and x - 1.

h_{sx}: story height below level x.

C_d: deflection amplification factor in ASCE 7-16 Table 12.2-1.

The deflection at level x, δx, used to compute the design story drift, 4, shall be determined in accordance with the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I_e} \qquad ASCE\ 7-16\ equation\ 12.8-15 \quad (4.10)$$

Where:

$$\theta = \frac{P_x \Delta_{xe}}{V_x h_{sx}}$$

Δxe: story drift from elastic analysis.

The stability coefficient, θ shall not exceed θ_{max}, determined as follows:

$$\theta_{max} = \frac{0.5}{\beta C_d} \le 0.25$$

Where ß is the ratio of shear demand to shear capacity for the story between levels x and x -

This ratio is permitted to be conservatively taken as 1.0, so:

θ_{max}= 0.5/(1*5)= 0.1 < 0.25 so take θ_{max}= 0.1.

Figures 5.47 and 5.48 illustrate the load combination definitions used for the **P-Delta check** in both the **X and Y directions**.

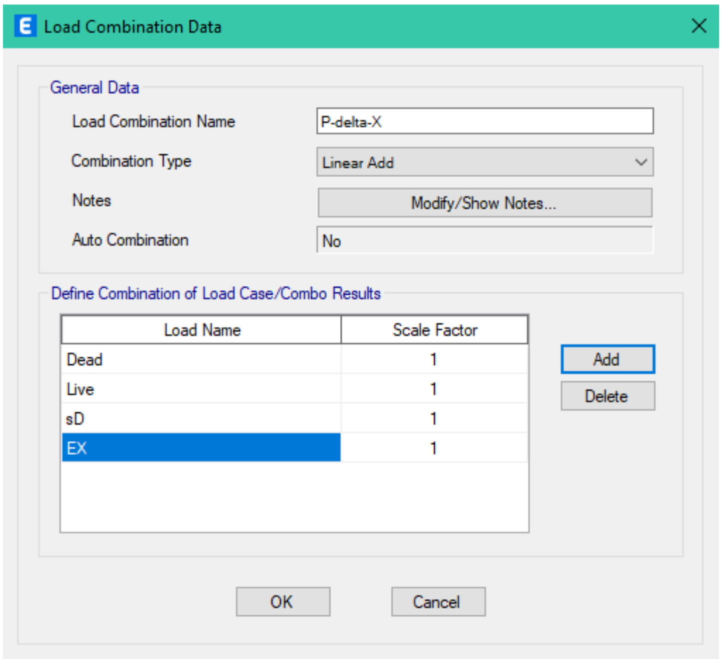


Figure 5.47: load combination for P-delta check in X direction

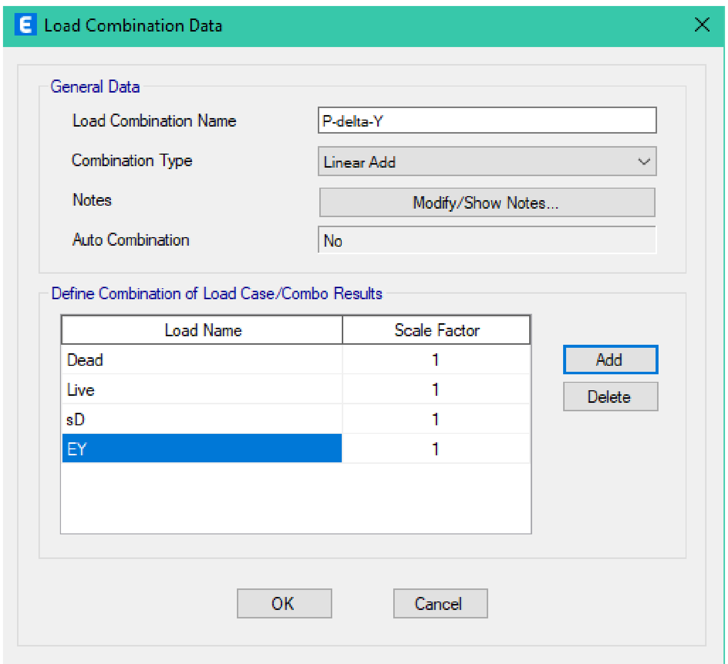


Figure 5.48: load combination for P-delta check in Y direction

define combination , exclude 30% of other directions.

Where the stability coefficient, θ is greater than 0.10 but less than or equal to max, the incremental factor related to P-delta effects on displacements and member forces shall be determined by rational analysis. Alternatively, it is permitted to multiply displacements and member forces by $1.0/(1 - \theta)$.

Where θ is greater than θmax, the structure is potentially unstable and shall be redesigned.

The used load combination in determining the need for P-delta effects in a direction is:

U = 1.0 DEAD + 1.0 SD + 1.0 LIVE + 1.0 E

This load combination is applied in both X and Y directions substituting EX and EY, respectively.

Figures 5.49 and 5.50 present the **story forces** in both the **X and Y directions**, while **Figures 5.51 and 5.52** show the **center of mass displacements of the diaphragms** across the building.

Story	Output Case	Case Type	Step Type	Step Number	Location	P kN	VX kN	VY kN	T kN-m	MX kN-m	MY kN-m
Story9	P-delta-X	Combination	Max		Bottom	1068.7855	-79.626	0	743.4582	9984.1299	-19567.5147
Story8	P-delta-X	Combination	Max		Bottom	14386.6094	-1161.1373	0	11026.5771	123857.675	-264106.6371
Story7	P-delta-X	Combination	Max		Bottom	25439.8917	-2086.2508	0	19766.3725	218434.2723	-471156.6609
Story6	P-delta-X	Combination	Max		Bottom	36494.2037	-2868.6876	0	27157.5323	313027.8927	-680990.1404
Story5	P-delta-X	Combination	Max		Bottom	47548.5156	-3505.6976	0	33174.9425	407621.5131	-893085.0057
Story4	P-delta-X	Combination	Max		Bottom	58640.1026	-3998.2241	0	37826.9812	502512.5347	-1107570
Story3	P-delta-X	Combination	Max		Bottom	70264.5845	-4358.2844	0	41395.4479	608459.3185	-1346096
Story2	P-delta-X	Combination	Max		Bottom	26368.0449	-954.8085	-583.0851	-6328.5907	217549.0164	-697137.6347

Figure 5.49: Screenshot of ETABS output table showing story forces in X-direction

Story	Output Case	Case Type	Step Type	Step Number	Location	P kN	VX kN	VY kN	T kN-m	MX kN-m	MY kN-m
Story9	P-delta-Y	Combination	Max		Bottom	1068.7855	0	-55.6511	-1005.5601	10176.1262	-19292.8051
Story8	P-delta-Y	Combination	Max		Bottom	14386.6094	1.435E-06	-803.3072	-13162.4275	126901.4117	-259709.89
Story7	P-delta-Y	Combination	Max		Bottom	25439.8917	2.56E-06	-1434.6672	-23428.6105	226571.0776	-459353.7236
Story6	P-delta-Y	Combination	Max		Bottom	36494.2037	3.483E-06	-1960.6699	-31979.7903	328125.0762	-659003.3623
Story5	P-delta-Y	Combination	Max		Bottom	47548.5156	4.366E-06	-2381.1953	-38816.2343	431171.94	-858653.0009
Story4	P-delta-Y	Combination	Max		Bottom	58640.1026	5.055E-06	-2698.9765	-43981.8597	535644.3281	-1058945
Story3	P-delta-Y	Combination	Max		Bottom	70264.5845	5.137E-06	-2924.1986	-47772.0374	654457.5855	-1278294
Story2	P-delta-Y	Combination	Max		Bottom	25687.8589	-28.9645	-1008.6539	-27160.3608	217822.3825	-665736.9901

Figure 5.50: Screenshot of ETABS output table showing story forces in Y-direction

Story	Diaphragm	Output Case	Case Type	Step Type	Step Number	UX mm	UY mm	RZ rad	Point	X m	Y m	Z m
Story8	D6	P-delta-X	Combination	Max		4.505	0.543	1E-06	53718	18.07	8.6031	26.2
Story7	D5	P-delta-X	Combination	Max		4.035	0.441	2E-06	53719	18.062	8.5649	22.65
Story6	D4	P-delta-X	Combination	Max		3.5	0.336	3E-06	53720	18.0585	8.5639	19.1
Story5	D3	P-delta-X	Combination	Max		2.9	0.226	4E-06	53721	18.0585	8.5639	15.55
Story4	D2	P-delta-X	Combination	Max		2.243	0.098	3E-06	53722	18.0569	8.5627	12
Story3	D1	P-delta-X	Combination	Max		1.548	-0.112	2E-06	53723	18.7844	9.0337	8.45
Story2	D7	P-delta-X	Combination	Max		0.557	-0.352	2.3E-05	53724	25.9944	8.4859	4.05

Figure 5.51: Screenshot of ETABS output table showing diaphragms center of mass displacements in X-direction

Story	Diaphragm	Output Case	Case Type	Step Type	Step Number	UX mm	UY mm	RZ rad	Point	X m	Y m	Z m
Story8	D6	P-delta-Y	Combination	Max		-1.012	9.304	-4E-06	53718	18.07	8.6031	26.2
Story7	D5	P-delta-Y	Combination	Max		-0.851	8.035	-4E-06	53719	18.062	8.5649	22.65
Story6	D4	P-delta-Y	Combination	Max		-0.696	6.662	-6E-06	53720	18.0585	8.5639	19.1
Story5	D3	P-delta-Y	Combination	Max		-0.543	5.197	-7E-06	53721	18.0585	8.5639	15.55
Story4	D2	P-delta-Y	Combination	Max		-0.391	3.668	-8E-06	53722	18.0569	8.5627	12
Story3	D1	P-delta-Y	Combination	Max		-0.229	2.076	-9E-06	53723	18.7844	9.0337	8.45
Story2	D7	P-delta-Y	Combination	Max		-0.072	0.328	3.3E-05	53724	25.9944	8.4859	4.05

Figure 5.52: Screenshot of ETABS output table showing diaphragms center of mass displacements in Y-direction

The **P-Delta output calculations** in both the **X and Y directions** are illustrated in **Figures 5.53 and 5.54**, respectively.

P-dealta , X-Direction							
Floor	**Axial Force, P(kN)**	**Shear Force, V(kN)**	**Displacement(mm)**	**Drift(mm)**	**Drift(m)**	**Story Height(m)**	**Stability Coeffecient, θ**
Story8	14387	1161	4.51	0.47	0.00047	3.55	**0.0016**
Story7	25440	2086	4.04	0.54	0.000535	3.55	**0.0018**
Story6	36494	2869	3.50	0.60	0.0006	3.55	**0.0022**
Story5	47549	3506	2.90	0.66	0.000657	3.55	**0.0025**
Story4	58640	3998	2.24	0.70	0.000695	3.55	**0.0029**
Story3	70265	4358	1.55	0.99	0.000991	4.4	**0.0036**
Story2	26368	955	0.56	0.56	0.000557	3.05	**0.0050**

Figure 5.53: Screenshot of EXCEL output table showing P-delta calculations in X-direction

P-dealta, Y-Direction							
Floor	**Axial Force, P(kN)**	**Shear Force, V(kN)**	**Displacement(mm)**	**Drift(mm)**	**Drift(m)**	**Story Height(m)**	**Stability Coeffecient, θ**
Story8	14387	803	9.30	1.27	0.001269	3.55	**0.0064**
Story7	25440	1435	8.04	1.37	0.001373	3.55	**0.0069**
Story6	36494	1961	6.66	1.47	0.001465	3.55	**0.0077**
Story5	47549	2381	5.20	1.53	0.001529	3.55	**0.0086**
Story4	58640	2699	3.67	1.59	0.001592	3.55	**0.0097**
Story3	70265	2924	2.08	1.75	0.001748	4.4	**0.0095**
Story2	25688	1009	0.33	0.33	0.000328	3.05	**0.0027**

Figure 5.54: Screenshot of EXCEL output table showing P-delta calculations in Y-direction

5.8 Check story drift

The values of displacements are obtained at the center of mass in the floors taking the load combination U= 1.0 DEAD + 1.0 SD + 1.0 LIVE + 1.0 E for drift computations.[9]

Drifts along all stories are shown in **Figures 5.55** and **5.56** for X and Y directions, respectively. The design story drift is the inelastic drift, so these elastic drifts are multiplied by the value C_d/I_e = 5.0/1.0= 5.0 in both directions. The final story drifts are shown in the same figures.

The maximum allowed story drift ratio is 0.020 (Risk category II, all other structures) according to ASCE 7-16 Table 12.12-1 (shown in **Figure 5.57**)and the maximum story drift

[9] If a torsional irregularity is found, the designer shall obtain the displacements and the drifts at an edge that gives the maximum values according to *Arman 2025a*. For this building structure, torsional irregularity can be neglected, so, the drifts were obtained at the center of mass.

ratio in Table 5.2 is 0.01405 which is less than the allowable limit, so, the story drifts are within the allowable limits.

Story Drift, X-Direction				
Floor	**Elastic Drift(m)**	**Inelastic Drift(m)**	**Story Height(m)**	**Inelastic Drift ratio**
Story8	0.00047	0.00235	3.55	**0.000662**
Story7	0.000535	0.002675	3.55	**0.000754**
Story6	0.0006	0.003000	3.55	**0.000845**
Story5	0.000657	0.003285	3.55	**0.000925**
Story4	0.000695	0.003475	3.55	**0.000979**
Story3	0.000991	0.004955	4.4	**0.001126**
Story2	0.000557	0.002785	3.05	**0.000913**

Figure 5.55: Screenshot of EXCEL output table showing story drift calculations in X-direction

Story Drift, Y-Direction				
Floor	**Elastic Drift(m)**	**Inelastic Drift(m)**	**Story Height(m)**	**Inelastic Drift ratio**
Story8	0.001269	0.006345	3.55	**0.001787**
Story7	0.001373	0.006865	3.55	**0.001934**
Story6	0.001465	0.007325	3.55	**0.002063**
Story5	0.001529	0.007645	3.55	**0.002154**
Story4	0.001592	0.007960	3.55	**0.002242**
Story3	0.001748	0.008740	4.4	**0.001986**
Story2	0.000328	0.001640	3.05	**0.000538**

Figure 5.56: Screenshot of EXCEL output table showing story drift calculations in Y-direction

Table 12.12-1 Allowable Story Drift, Δ_a [a,b]

	Risk Category		
Structure	**I or II**	**III**	**IV**
Structures, other than masonry shear wall structures, four stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	$0.025h_{sx}$ [c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures[d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below level x.
[b] For seismic force-resisting systems solely comprising moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.
[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support that are so constructed that moment transfer between shear walls (coupling) is negligible.

Figure 5.57: Allowable story drifts (ASCE 7-16, Table 12.12-1)

CHAPTER 6: STRUCTURAL DESIGN FOR SEISMIC LOADS

6.1 General

This chapter presents the redesign of the structural system to ensure reliable performance under seismic loading, building on the gravity-based design developed in Part I of the project. As seismic forces impose additional demands beyond those of gravity, several structural modifications were necessary to meet these requirements. These included both proactive adjustments—applied early based on prior experience—and reactive changes, introduced in response to the results of seismic analysis.

The redesign process considered the results of the seismic analysis model developed in ETABS, alongside code-based design. The structural system was evaluated in terms of strength, stiffness, and detailing requirements under seismic forces. The modifications aimed to retain as much of the original design intent as possible, while addressing deficiencies revealed through analysis and code checks.

This chapter also outlines the adopted seismic design methodology, including how structural elements were treated depending on their role in seismic resistance, the rationale behind specific changes, and how the final design complies with ACI seismic provisions. It also discusses the proactive and reactive design changes made to ensure the structure could safely resist both gravity and lateral loads.

The final seismic design is verified against the relevant seismic codes to ensure compliance. Detailed design checks are carried out for key structural components such as shear walls, columns, and foundations to guarantee the building's performance during an earthquake.

6.2 Seismic design approach

This section outlines the approach adopted for designing the structure under seismic loads. It begins with the conceptual framework that guides seismic design according to *ACI 318-19* and *ASCE 7-16*, emphasizing the performance-based philosophy that underpins modern code provisions. Building on this foundation, the section then moves to describe how these principles were interpreted and applied within the context of this project, including how design forces were evaluated, how structural responsibilities were distributed between walls and frames, and how code requirements were incorporated into the redesign process.

Together, these parts form a cohesive framework that connects code-driven objectives with project-specific decisions.

6.2.1 Seismic design philosophy (Conceptual Basis)

The seismic design philosophy, as previously introduced in Chapter 4, is grounded in the principle that structural design does not aim to eliminate all damage, but rather to protect human life and maintain core structural integrity during extreme seismic events. This fundamental concept is consistent across various building codes. In this section, the philosophy is examined from a more technical perspective, with emphasis on the specific interpretations and requirements of the adopted design code.

6.2.1.1 Code concepts and guidance

The redesign process under seismic loading follows the performance-based philosophy embedded in *ACI 318-19* and *ASCE 7-16*. These standards acknowledge that structural systems are not expected to remain fully elastic during strong ground shaking. Instead, the goal is to ensure that the structure can dissipate energy through controlled, ductile inelastic behavior—allowing damage to occur without leading to collapse.

ACI 318-19 defines the scope and intent of its seismic provisions in *Section 18.1.2*, stating:

> *"Structures designed according to the provisions of this chapter are intended to resist earthquake motions through ductile inelastic response of selected members."*

This philosophy assumes that structural members may yield under strong shaking, but they must do so in a safe, predictable manner. **Ductility** is therefore a fundamental requirement, especially in regions of high seismic risk. To this end, *ACI 318-19* includes special provisions to ensure that inelastic deformation occurs primarily in designated energy-dissipating zones—such as plastic hinges or confined boundary regions—while maintaining overall system integrity.

In addition to ductility, **interaction between axial and lateral demands** is a critical concern. A structural system must be capable of resisting the combined effects of gravity and earthquake-induced forces. **Stability** is another key consideration; members are required to remain stable under lateral displacements and to account for **second-order (P-Δ) effects**, especially in slender or flexible systems (ASCE/SEI 7-16, Section 12.8 and Commentary C12.8).

Design strength is still the initial measure used for member proportioning, but it is not the ultimate criterion. Instead, the code assumes that strength may be exceeded in localized areas, and that through appropriate detailing, the structure can continue to function without collapse. This places strong emphasis on reinforcement detailing—such as confinement, bar anchorage, and minimum steel ratios—and on ensuring that the system's geometry and member hierarchy support the intended inelastic mechanisms. The code also provides specifications for the relative strength of frame members—such as ensuring that columns are stronger than beams—to promote desirable failure modes and preserve overall system stability. These strength relationships support the core principles and objectives of seismic design.

6.2.1.2 Summary

Ultimately, the intent is to deliver a structural system that can sustain damage without collapse, while ensuring the safety and stability of the building during strong seismic events. To achieve this, **special attention must be given to the detailing of energy-dissipating regions, the relative strength and interaction of structural members, and the overall system stability under combined axial and lateral demands.** Other aspects of structural stability, including vertical and horizontal irregularities, story drift limits, and the need for P-Δ analysis, are important considerations. These aspects were addressed in **Chapter 5** through focused stability checks and are not discussed again in this chapter unless directly relevant to certain modifications. These principles serve as the foundation for the practical design approach adopted in this project, where code requirements are translated into engineering strategies and modeling decisions in the following section.

6.2.2 Application in this project (Seismic design approach in practice)

This section translates the seismic design philosophy outlined in Section 6.2.1 into a practical engineering approach tailored to the specific characteristics of the building. The focus is on how the three central design pillars—**strength**, **ductility**, and **stability**—were achieved through analysis, modeling decisions, and detailing strategies used to ensure seismic compliance. Each of these aspects was addressed through a combination of code-based requirements, engineering judgment, and interpretation of ETABS results.

Stability Considerations:

Stability was primarily addressed and verified in Chapter 5. It included checks for irregularities, story drift limits, and the need for second-order (P-Δ) effects, which was found to be negligible for this project. These assessments confirmed that the selected structural system maintains adequate stability under expected lateral displacements, and no further adjustments were required within this section unless needed for design modifications.

Ductility Considerations:

Ductility is a central objective in seismic design. As noted in the previous section, controlled inelastic behavior is essential for energy dissipation during major earthquakes. In this project, the provision of ductility is addressed in detail in **Section 6.6**, where reinforcement detailing and compliance with **ACI 318-19 Chapter 18** are examined.

Strength Considerations:

Strength, the focus of Section 6.4, is achieved by ensuring that member dimensions and reinforcement quantities are sufficient to resist the imposed seismic actions. Structural elements were evaluated and, where necessary, modified based on force demands obtained from the ETABS model developed for seismic considerations. Where results indicated deficiencies, cross-sectional capacities were increased accordingly.

While ETABS was used to perform linear static analysis and extract design forces, its primary role was to inform and verify decisions grounded in ACI 318-19. ETABS results were interpreted within the context of code provisions, not treated as standalone solutions.

Code Application:

Design decisions and detailing approaches were based on ACI 318-19, particularly Chapter 18 for seismic provisions. Since the lateral system was primarily composed of walls, relevant wall design criteria were applied. Frame members, though not designated as part of the LFRS, were still evaluated using code provisions to ensure adequate strength and confinement under accidental or secondary lateral demands. The requirements of Section 18.14 were referenced to address such cases.

Wall–Frame Interaction and Modeling Approach:

One of the key steps in implementing the seismic design was confirming the behavior of the designated lateral force-resisting system (LFRS). Based on initial design considerations, **reinforced concrete walls** were selected to act as the primary LFRS. To validate this assumption, lateral force participation was assessed using ETABS. As discussed in Chapter 5, the walls attracted approximately 90–95% of the total base shear, while the frames contributed only a small portion.

These findings supported the decision to treat the walls as the primary seismic-resisting elements and to model them accordingly. Within this framework, two modeling strategies were considered. **Model 1** assumes that **100% of the lateral load is resisted by shear walls**, as intended in the design (i.e., the frames are not part of the lateral force-resisting system). **Model 2**, however, reflects the **realistic structural behavior observed in the analysis**, where frame members resist a small portion of the seismic forces due to stiffness interaction, even though they are not intended to be part of the LFRS. Each for specific purpose mentioned below with their clarification:

Model 1 – Wall-Only Seismic Resistance:

In the first model, **moment releases (Moment 22 and Moment 33)** were assigned at both the **start and end of each column**, effectively preventing the frames from developing lateral stiffness. This ensured that nearly **100% of the lateral seismic forces were resisted by the structural walls**. The forces and moments from this model were used to design and detail wall elements in accordance with **ACI 318-19 Chapter 18**. This approach guaranteed that wall design reflected the full demand expected from seismic actions, with no unintended contribution from the frames.

Model 2 – Realistic Participation for Secondary Elements

The second model retained full continuity in the frame elements (i.e., no releases were applied), allowing them to contribute naturally to lateral resistance. This model represented a **more realistic interaction** between walls and frames, where the frames resisted 5–10% of the total base shear.

ACI 318-19 permits a simplified treatment for frame members not explicitly part of the LFRS. Sections **18.14.2.1** and **18.14.3.1** of ACI 318-19 states:

> *Section 18.14.2.1:*
>
> *"Members not designated as part of the seismic-force-resisting system shall be evaluated for gravity load combinations of 5.3 including the effect of vertical ground motion acting simultaneously with the design displacement δu."*
>
> *Section 18.14.3.1:*

"Cast-in-place beams, columns, and joints shall be detailed in accordance with 18.14.3.2 or 18.14.3.3 depending on the magnitude of moments and shears induced in those members when subjected to the design displacement δu. If effects of δu are not explicitly checked, the provisions of 18.14.3.3 shall be satisfied."

Although not explicitly stated in the code, two practical approaches can be inferred for dealing with the frame members in the case of this project:

1. The first approach involves directly considering the amplified displacement effects (δ_u).
2. The second allows these effects to be neglected provided certain relevant code requirements are satisfied.

While this provision allows for reduced requirements if the frame does not resist significant seismic actions, our design opted to check and detail all frame elements conservatively using results from the second model. ETABS outputs were used to size and reinforce beams and columns based on actual force demands, regardless of their primary role in resisting gravity only. This ensured that even minor participation was adequately addressed.

This second model also formed the basis for evaluating and modifying the **design of slabs, footings, and other members** that could be affected by seismic load paths. In cases where ETABS outputs indicated differences from gravity-only conditions, redesign was performed accordingly to ensure compliance with seismic design criteria.

6.3 Evaluation of gravity design response to seismic loading

This section presents a systematic evaluation of the original structural design, which was developed under gravity load assumptions, to determine its adequacy under seismic demands. The focus is on assessing the suitability of both the structural layout and the cross-sectional dimensions of members, as seismic effects may introduce additional force demands that could render the gravity-based design insufficient. Although reinforcement quantities and detailing requirements are also relevant, only changes in reinforcement ratios are addressed here; comprehensive reinforcement quantities and seismic detailing provisions are intentionally omitted and deferred to later sections of the report.

The evaluation is divided into two main parts. The first is a preliminary engineering assessment, relying on experience and conceptual seismic understanding to judge the general layout and configuration of the structure, particularly its ability to resist lateral loads through appropriate load paths. The second is a software-based assessment, which draws on ETABS seismic analysis results to evaluate the adequacy of individual member dimensions and reinforcement under lateral load effects.

By the end of this evaluation, a deliberate judgment is to be made as to whether the existing layout, member dimensions, and reinforcement can adequately resist seismic effects, or if structural modifications are required. It is important to emphasize that this section is limited to the evaluation process; any resulting structural modifications are presented in the subsequent main section.

6.3.1 Structural layout and resisting system

The structural layout—including the arrangement of vertical and horizontal elements as well as the distribution of non-structural mass—directly affects a building's response to seismic excitation. Particular attention was given to the placement and continuity of shear walls, which were intended to serve as the primary lateral force-resisting system. A preliminary assessment of the layout was conducted based on engineering judgment and conceptual understanding of seismic behavior, prior to running the seismic analysis. This early review aimed to identify potential sources of irregularity—both in plan and elevation—allowing proactive improvements to be made before the software modeling phase. This approach helped minimize the need for iterative corrections during the analysis stage.

The original ground floor layout exhibited asymmetry in the Y-direction due to irregular wall placement, in contrast to the basement and typical floors, which showed more symmetric wall distribution. This suggested a possible torsional irregularity, which was anticipated even before modeling. As a result, the initial configuration was flagged as requiring improvement in terms of symmetry and lateral stiffness distribution. Plan views of the basement, ground, and typical floors are shown in **Figures 6.1-6.3**.

In addition, an elevation view along grid-line 1 (**Figure 6.4**) suggested a potential vertical discontinuity in wall alignment at the ground level. Based on observation and prior experience, this type of discontinuity corresponds to a well-known form of vertical irregularity that may compromise seismic performance. While the exact impact would be clarified through detailed analysis, the condition was considered noteworthy during the early evaluation phase and was anticipated to require specific design attention in later stages.

Despite some layout concerns, the overall number and distribution of shear walls in the building were deemed sufficient in terms of strength and stiffness. The structure incorporates numerous vertical elements capable of resisting lateral forces. Based on engineering judgment, the building appeared strong enough to withstand seismic demands without the need for additional walls beyond those already included in the gravity design. Therefore, the primary concerns were related to layout irregularities, rather than a deficiency in lateral resistance.

It is important to note that the observations made during this preliminary evaluation—particularly the asymmetry identified in the ground floor wall layout—prompted early adjustments to the structural configuration prior to running the seismic model. Although these modifications are not discussed in detail here, they directly influenced the outcome of the seismic analysis. Accordingly, the ETABS model used for the seismic evaluation already incorporates the revised layout, and the results presented in **Section 6.3.2** are based on this updated configuration. The specific modifications and their justification are presented in **Section 6.4**.

6.3.2 Software-Based evaluation of structural design

6.3.2.1 General

Following the preliminary layout-based assessment discussed earlier, a more rigorous evaluation of the structure was carried out using the updated ETABS model that incorporates both gravity and seismic loads. The primary aim of this stage was to determine whether the cross-sectional sizing of structural members—originally designed for gravity loads—remained adequate when subjected to code-defined earthquake effects. While the evaluation primarily focused on member sizing, reinforcement was only briefly addressed in terms of changes, with detailed quantities and provisions deferred to later sections.

Seismic loads are known to introduce new challenges not typically encountered in gravity-only design. These include significant increases in lateral forces, potential uplift or overturning in foundations, combined axial and flexural demands in columns, shear amplification in beams and walls, and overall changes in the distribution of internal forces. Anticipating such effects, it became important to assess whether the original member sizes could accommodate the added seismic demands or whether resizing would be required to maintain structural integrity and compliance with code provisions.

Overall, while many members retained acceptable performance under the revised loading, others—particularly walls—displayed increased force demands that signaled a need for resizing or further detailing. The remainder of this section presents a detailed evaluation of each element type based on these findings.

6.3.2.2 Evaluation strategy

To carry out this evaluation, the ETABS model enhanced by defining new lateral load cases—namely, EX and EY for static earthquake forces, and EDX and EDY to account for dynamic base shear components—was used as previously explained in Chapter 5[10]. The design modules within ETABS were then used to extract internal force demands across all relevant structural elements as follows:

- For beams and columns, the Concrete Frame Design module provided a direct assessment of force capacities and sizing adequacy.
- For shear walls, the Shear Wall Design module provided a full design, including boundary element checks, longitudinal and transverse reinforcement requirements, and detailing provisions in accordance with ACI 318-19. This process helped verify whether the existing wall dimensions could accommodate the required reinforcement and served to check the adequacy of wall thickness.
- For slabs and footings, internal force contours were reviewed under the updated seismic load combinations to capture any substantial shifts.

6.3.3.3 Frames (Beams and Columns)

The design output revealed increased demands in several columns. While some experienced minor overstressing, others—particularly those located on the upper floors—required modest increases in longitudinal reinforcement ratios. In contrast, the beams generally exhibited performance consistent with the gravity-only design. Both column-supported and

[10] The analysis results presented in this section are not solely a product of seismic loads being introduced. The preliminary modifications applied to the structural layout—most notably the redistribution of shear walls—also played a direct role in shaping the behavior of the system under seismic loading. These modifications were made before the software analysis, meaning the results here inherently reflect both the adjusted configuration and the expanded loading conditions.

wall-supported beams remained adequate in terms of cross-sectional dimensions and did not require reinforcement adjustments, as no significant increases in demand were observed.

Specifically, Columns E13 and R13 at the 5th floor (Story 8 in ETABS) showed increased reinforcement ratios, reaching up to approximately 1.38%. Minor overstressing was also observed in Columns D4, P4, G11, N11, and E13 at the ground floor (Story 3 in ETABS), primarily due to increased demands from seismic load combinations. These outcomes reflect localized effects rather than a systemic deficiency in the structural design.

Such results are considered reasonable, as the frame members (both columns and beams) are not part of the designated lateral force-resisting system. **Figures 6.1–6.4** illustrate the frame members that experienced overstressing or reinforcement changes, while the complete design results are provided in **Appendix B**.

It is also worth noting that several very short beams exhibited overstressing due to combined shear and torsion demands. However, this overstress was disregarded, as the affected beams are wall-supported and, consequently, are not expected to develop significant shear forces (design shear force Ve) during seismic events. Therefore, the software results in this case are not considered representative. The relevant ETABS output for these overstressed beams is also included in the appendix mentioned above.

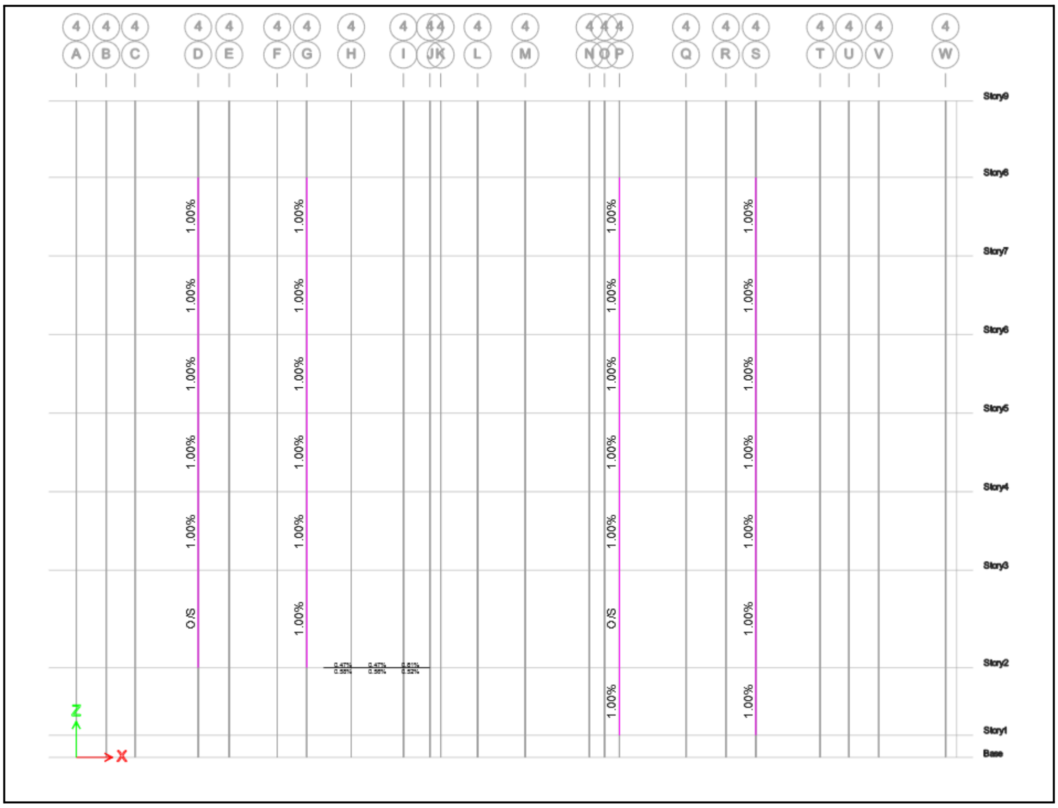


Figure 6.1: Design output of columns along grid-line 4 (Screenshot from ETABS)

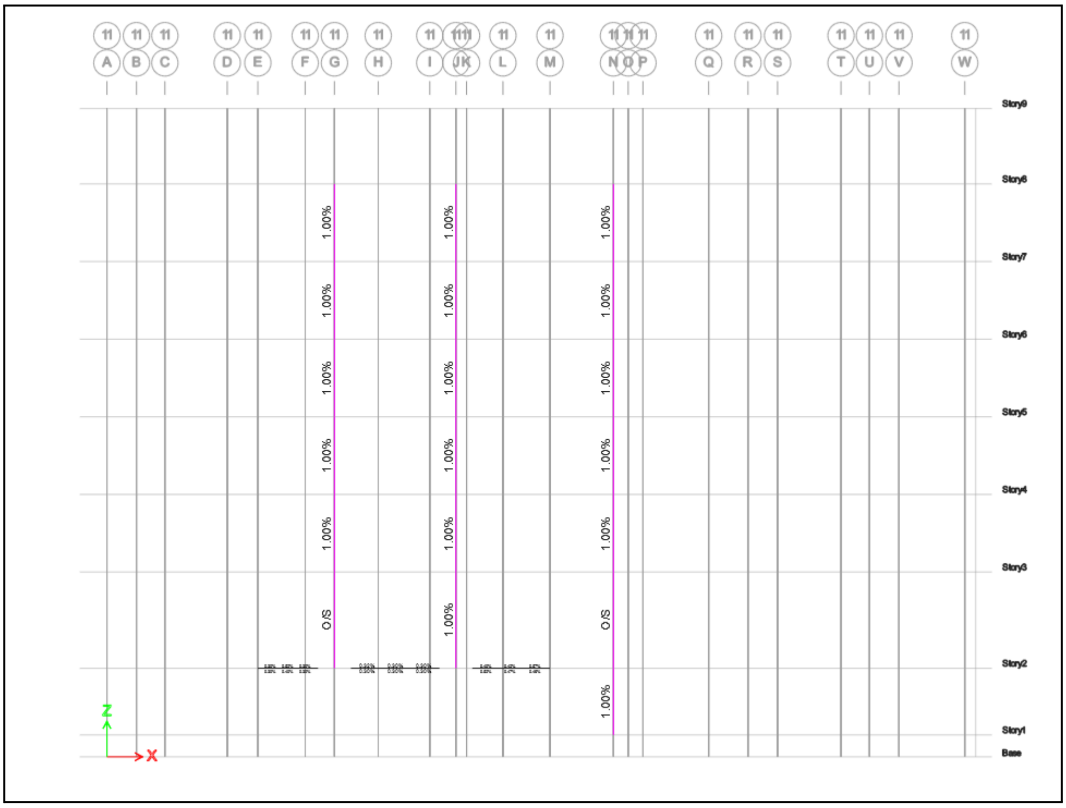


Figure 6.2: Design output of columns along grid-line 11 (Screenshot from ETABS)

Figure 6.3: Design output of columns along grid-line 13 (Screenshot from ETABS)

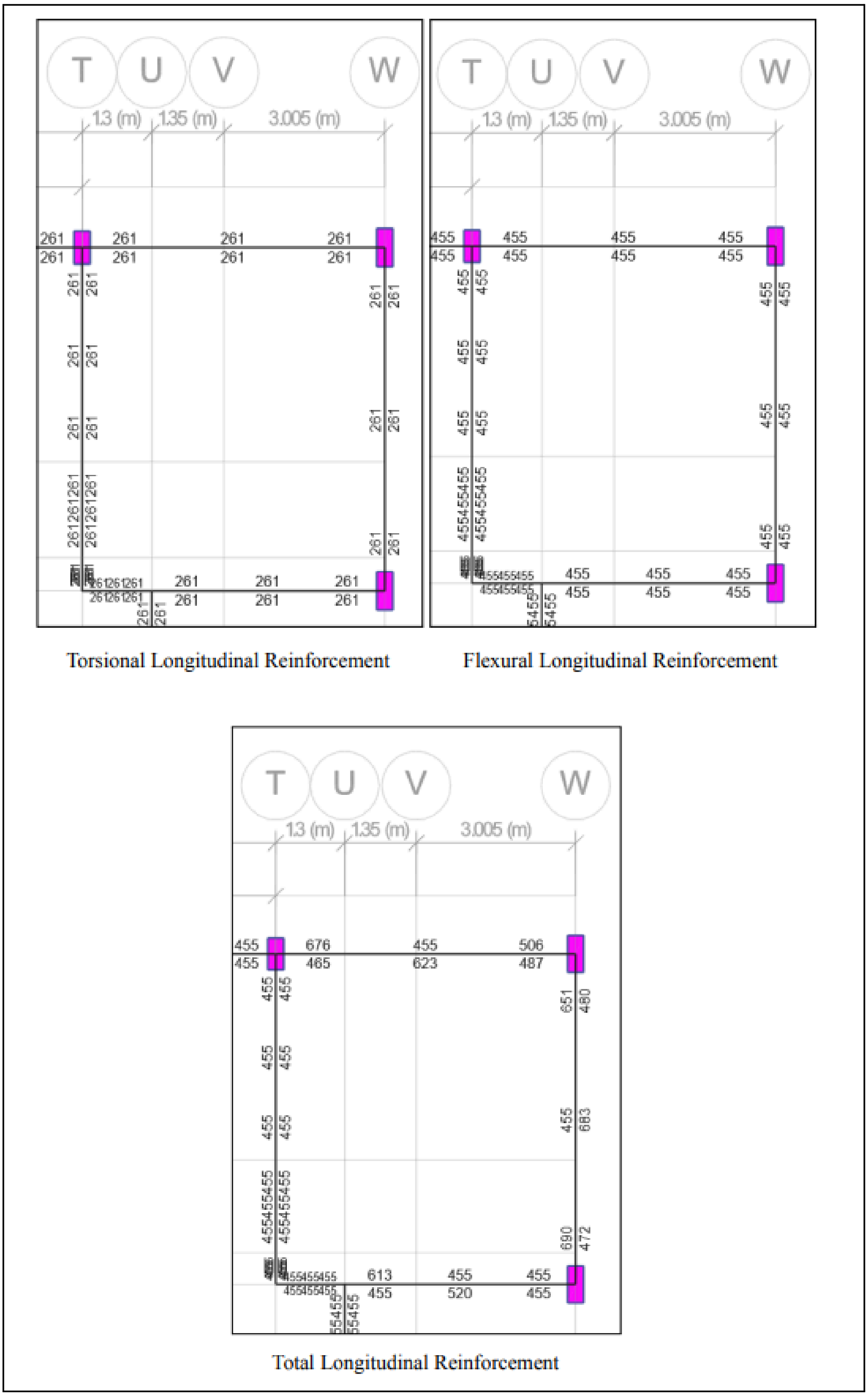


Figure 6.4: Longitudinal Reinforcement in column-supported beams (ETABS Output).

6.3.3.4 Shear walls

Wall thicknesses remain unchanged, with external walls having a thickness of 200 mm and internal walls 300 mm. However, the reinforcement layout was revised to meet ACI 318-19 requirements, ensuring adequate strength and ductility under seismic loading. These changes were made to enhance the wall's performance as a primary lateral force-resisting element in the structural system.

6.3.3.5 Slabs

While most slab zones remained within acceptable limits, internal force envelopes extracted under seismic load combinations revealed localized increases in bending moments, particularly near support lines and wall-slab connections. These stress concentrations may require additional top reinforcement to ensure adequate structural performance and crack control. **Figure 6.5** and **6.6** highlights the specific areas where slab reinforcement was enhanced accordingly.

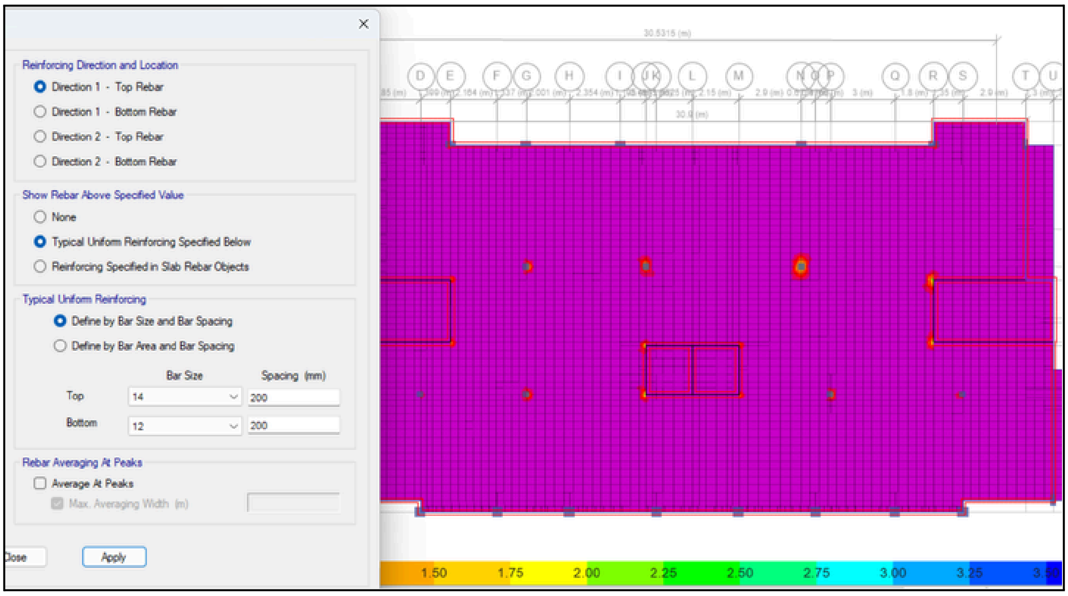


Figure 6.5: Enhanced reinforcement at story 8

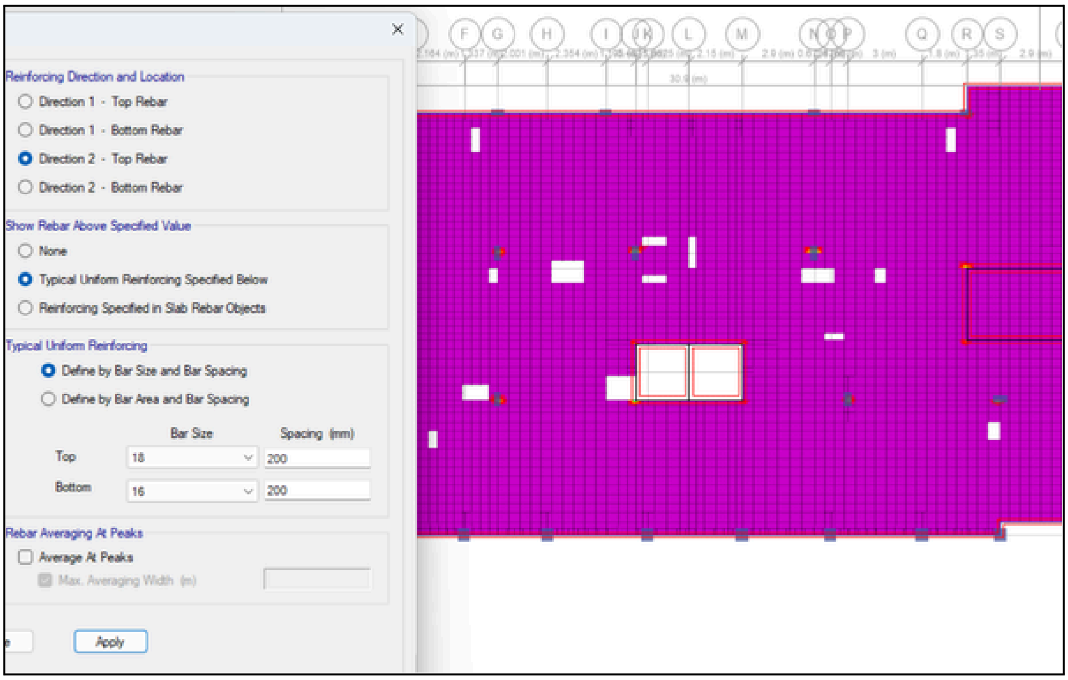


Figure 6.6: Enhanced reinforcement at story 7

6.3.3.6 Footings

In terms of sizing, footings are evaluated for the following:

- Soil bearing pressure: as the updated load combinations—namely, the seismic combinations—may lead to increased pressures that could exceed the allowable bearing capacity. This check is highly relevant to footings areas where an adjustment may be necessary.
- Overturning stability should also be checked to ensure that tension zones do not develop; otherwise, a different design approach may be required to address the resulting compression zones.
- Wide-beam shear and punching shear: a general verification of shear is conducted to confirm that the footing thicknesses are sufficient.

Although envelope results were initially reviewed to identify critical locations, final soil pressure values were extracted from individual governing load combinations to ensure realistic and consistent evaluation. The maximum values were found to occur under a specific combination which is [1.05 D+0.75 L+0.75 SD+0.68 EY] , which was used in the bearing capacity checks and footing design.

6.3.3.6.1 Assessment of soil pressure in footings

Although envelope results were initially reviewed to identify critical locations, final soil pressure values were extracted from individual governing load combinations to ensure realistic and consistent evaluation. The maximum values were found to occur under a specific combination which is [1.05 D+0.75 L+0.75 SD+0.68 EY] , which was used in the bearing capacity checks and footing design.

The soil pressure beneath the footings must be checked to verify that the provided footing areas are sufficient to safely transfer loads to the ground. The soil pressure values obtained from the structural analysis must not exceed the allowable bearing capacity, which is 250 kN/m² for this project.

As shown in **Figures 6.7** and **6.8**, the calculated soil pressures from ETABS remain below this limit, confirming that **the foundation dimensions are adequate with respect to soil pressure**.

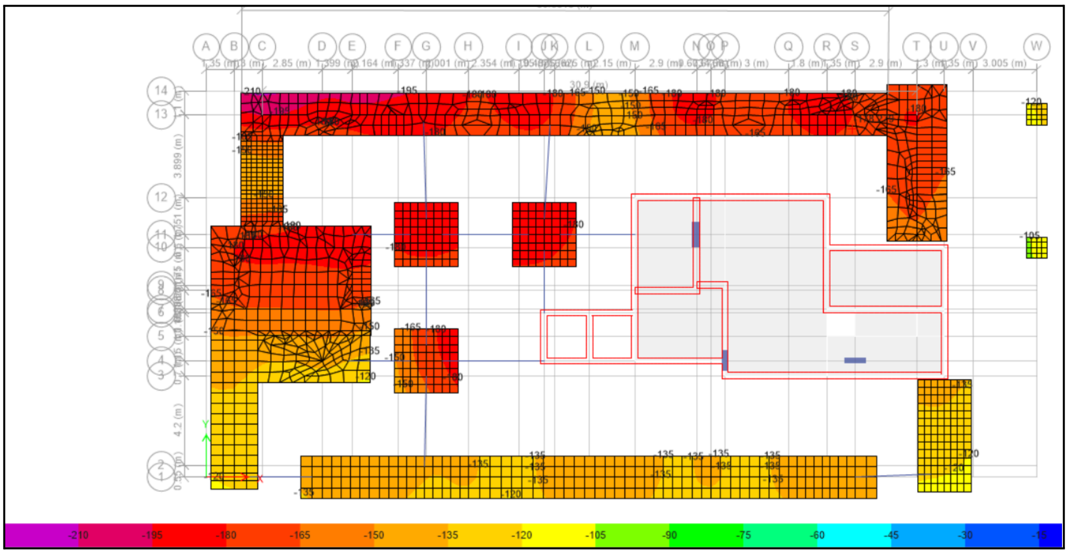


Figure 6.7: Contour map of soil pressure in footings for the determinant load combination (ETABS Output)

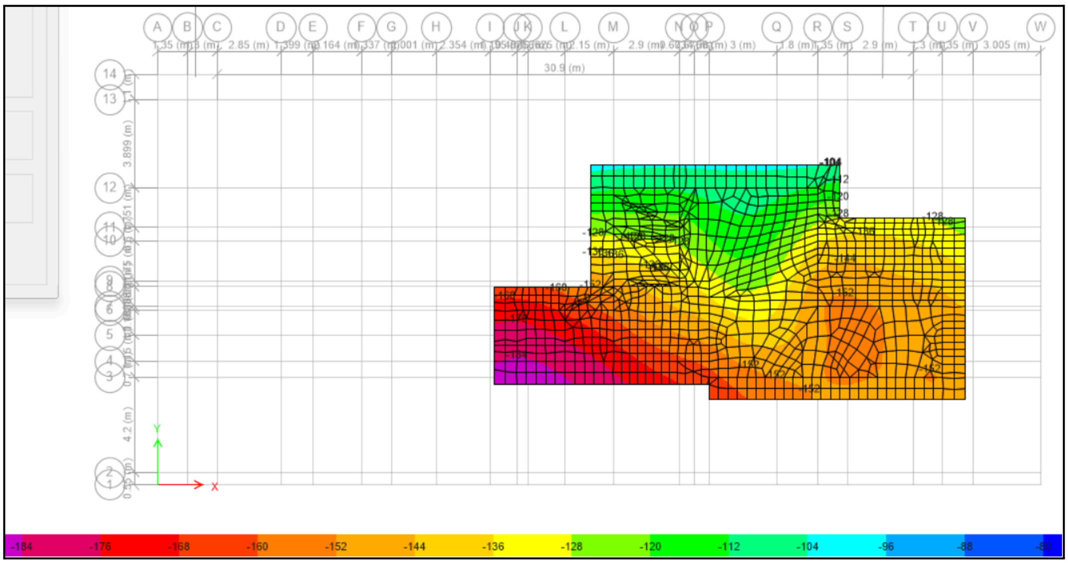


Figure 6.8: Contour map of soil pressure in basement (ETABS Output)

6.3.3.6.2 Shear check

Shear forces were evaluated using an envelope load combination defined in ETABS, which includes all ultimate seismic combinations as required by ASCE 7-16 for analysis and design. The resulting shear values were found to be satisfactory and, in all cases, did not exceed those obtained from gravity-only combinations. This indicates that **the existing footing thicknesses are adequate** and do not require modification. Contour maps of shear distributions in the X and Y directions are presented in **Figures 6.9** and **6.10**, respectively.

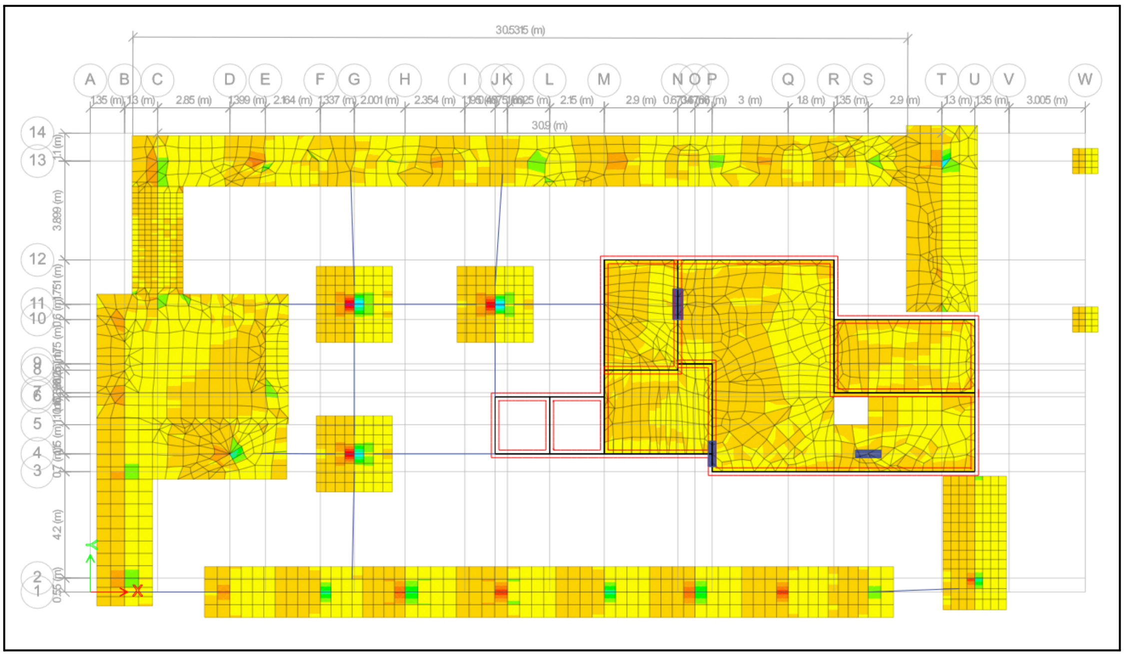


Figure 6.9: Contour map of shear force V13 in footings for the ENVELOPE load combination

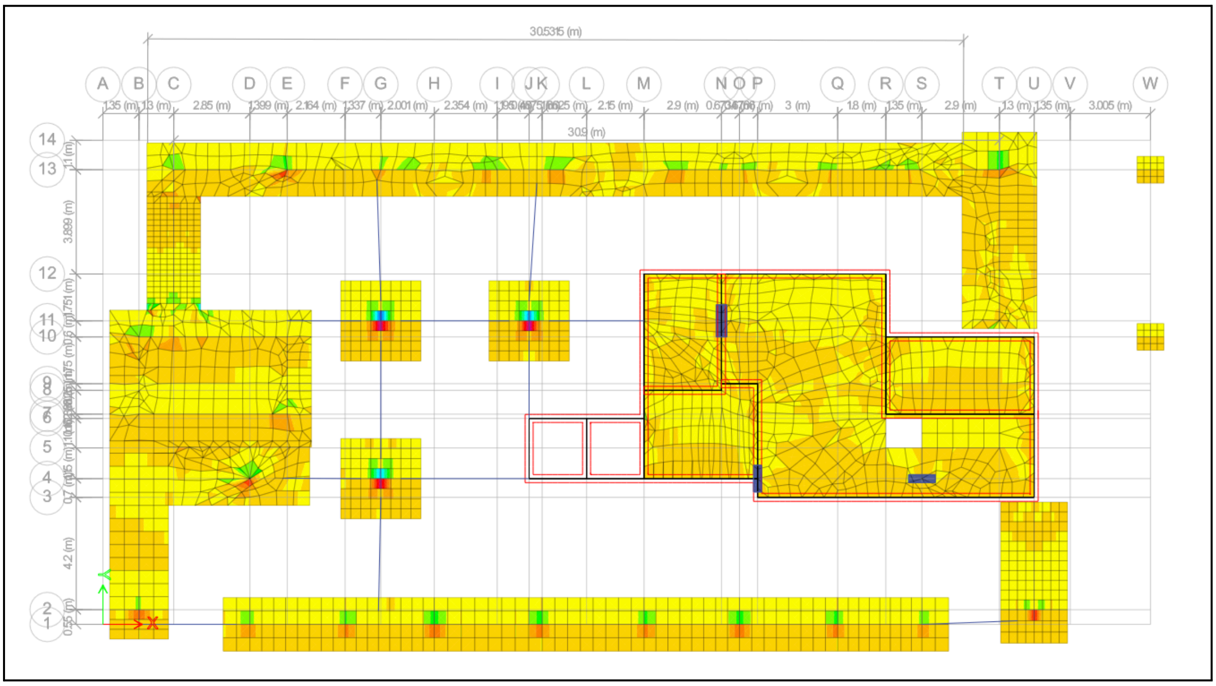


Figure 6.10: Contour map of shear force V23 in footings for the ENVELOPE load combination

6.3.3.6.3 Punching operation

Figure 6.11 shows the punching shear values, all of which are less than 1. This indicates that the provided footing thicknesses are adequate to resist punching shear.

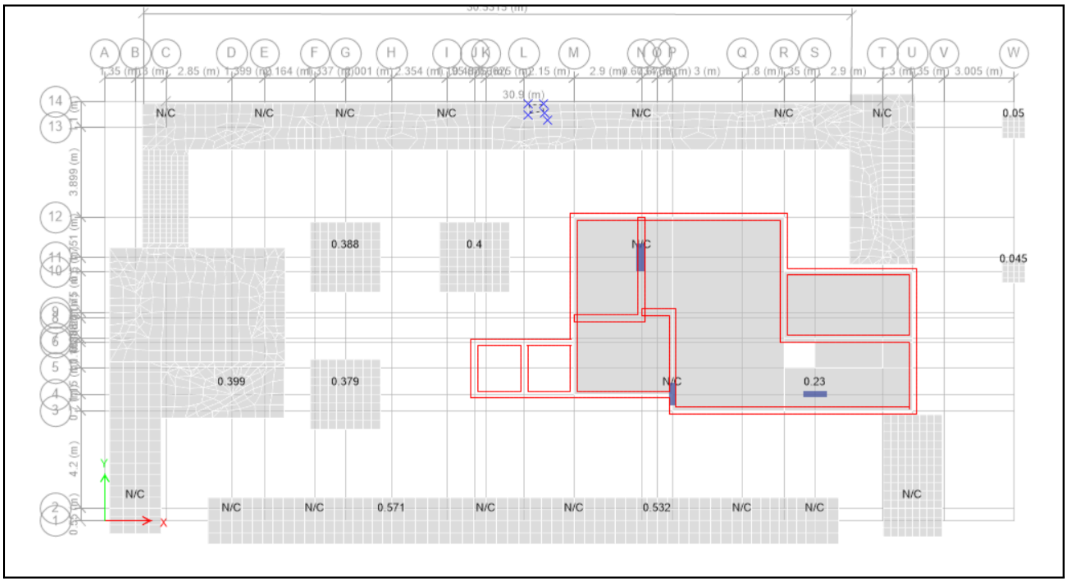


Figure 6.11: Punching shear values under seismic loading

6.3.3.6.4 Flexural reinforcement

Footing flexural reinforcement is shown later in section 6.7.4.

6.3.3.7 Tie beams

Tie beams at the foundation level are important structural elements that serve multiple purposes. They help connect individual footings together, improving the overall stability of the foundation system. **Figures 6.12**, **6.13** and **6.14** illustrate the total, flexural and torsional longitudinal reinforcement for tie beams

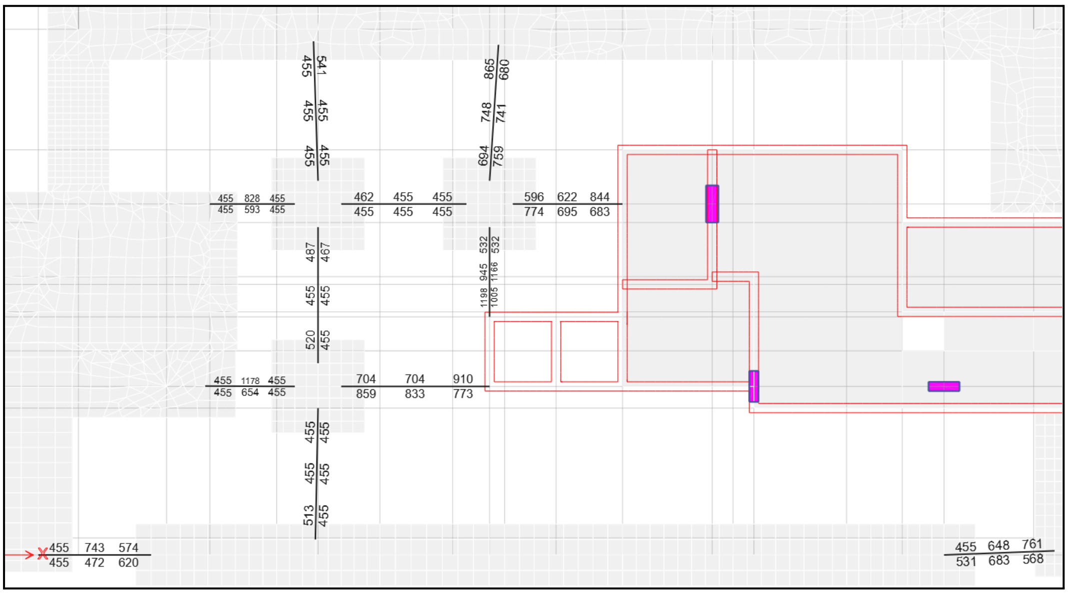


Figure 6.12: Total longitude reinforcement

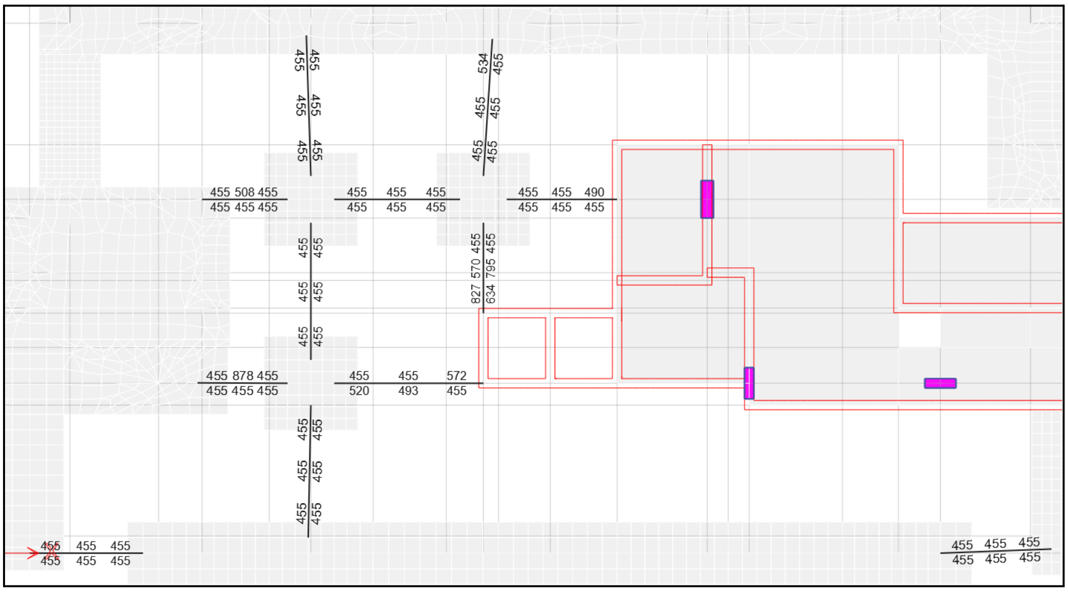


Figure 6.13: Total flexural longitude reinforcement

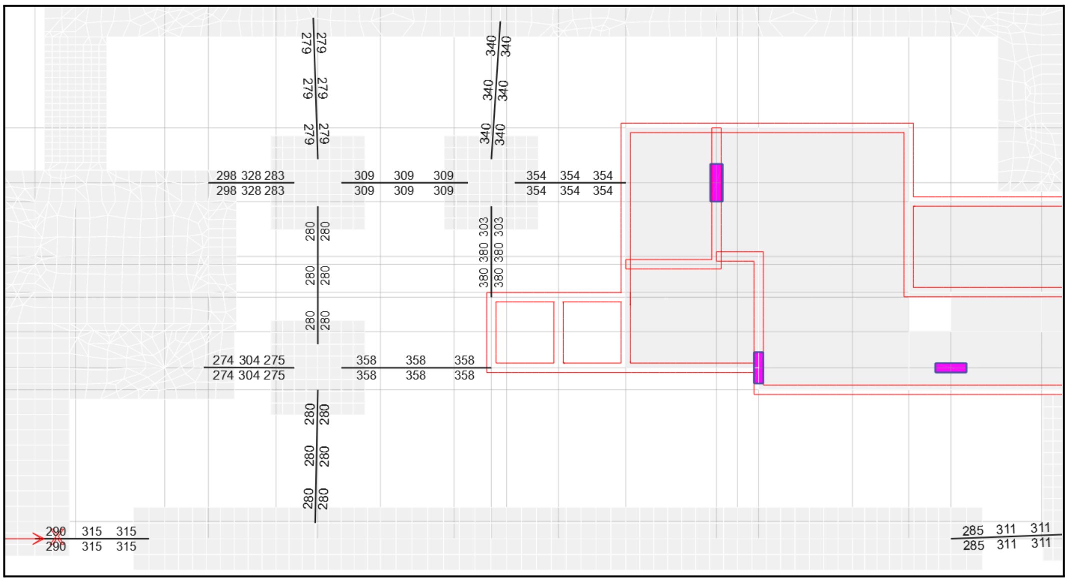


Figure 6.14: Total torsional longitude reinforcement

Comparing these values with those obtained in Chapter 3, it was found that the tie beams require adjustments. These modifications are necessary to ensure proper force transfer between footings and to improve overall foundation integrity, especially under seismic loading. The revised design will enhance the connectivity and stiffness of the foundation system, aligning it with ACI 318-19 requirements.

6.4 Structural modifications

This section focuses on the adjustments made to structural elements following the evaluation process. It outlines changes in dimensions, positioning, and the addition of structural components—such as walls, columns, or beams—introduced to accommodate seismic requirements. Where necessary, reinforcement modifications are also noted; however, a comprehensive discussion of reinforcement details is reserved for the final section presenting the completed design.

6.4.1 Frame members

The dimensions of beams remained unchanged, as they were found to be sufficient even under the additional seismic demands. In contrast, several overstressed columns required adjustments to ensure adequate load-carrying capacity. **Table 6.1** summarizes the dimensional changes made to these columns to address the increased demands, and **Table 6.2** shows the changes in steel ratio.

Table 6.1: Dimensional changes of columns

Floor	Grid location	Previous dimension	Adjusted dimension (mm)
Ground Floor	D4	300*1000	400*1000
	P4	300*800	400*1000
	N11	300*1200	400*1200
	G11	300*1000	400*1200
	E13	300*600	300*700

Table 6.2: Changes in steel ratio in columns

Column, Story	Previous steel ratio	Adjust steel ratio
E13, story 8	1%	1.4%
R13, story 8	1%	1.4%

As the gross cross-sectional area of certain columns increased—while maintaining the same steel ratio of 1%—the required area of steel also increased proportionally. This was addressed by using larger bar diameters. Additionally, some columns that were not overstressed still exhibited increased longitudinal reinforcement demands; in these cases as well, the bar diameters were adjusted accordingly. The final reinforcement layout, reflecting these diameter changes, is presented in Section 6.8 after the application of all relevant code detailing provisions.

6.4.2 Shear walls

Some modifications to the structural walls were implemented prior to conducting the software analysis and design, as previously discussed. Specifically, three new walls, each of 200 mm thickness, were added along grid line 1 at the ground floor. This modification was expected to play a crucial role in mitigating potential torsional irregularities and was therefore carried out in the early stages.. **Figure 6.15** presents a screenshot from ETABS illustrating the ground floor plan after this adjustment, highlighting the newly added walls.[11]

[11] The figure, originally intended to demonstrate wall preliminary modifications, was mistakenly captured post-analysis, and thus also shows column adjustments.

Wall thicknesses varied throughout the project, adapting to structural requirements. These changes are summarized in **Table 6.3**. **Figure 6.16-6.18** presents the corresponding walles naming for those presented in the table.

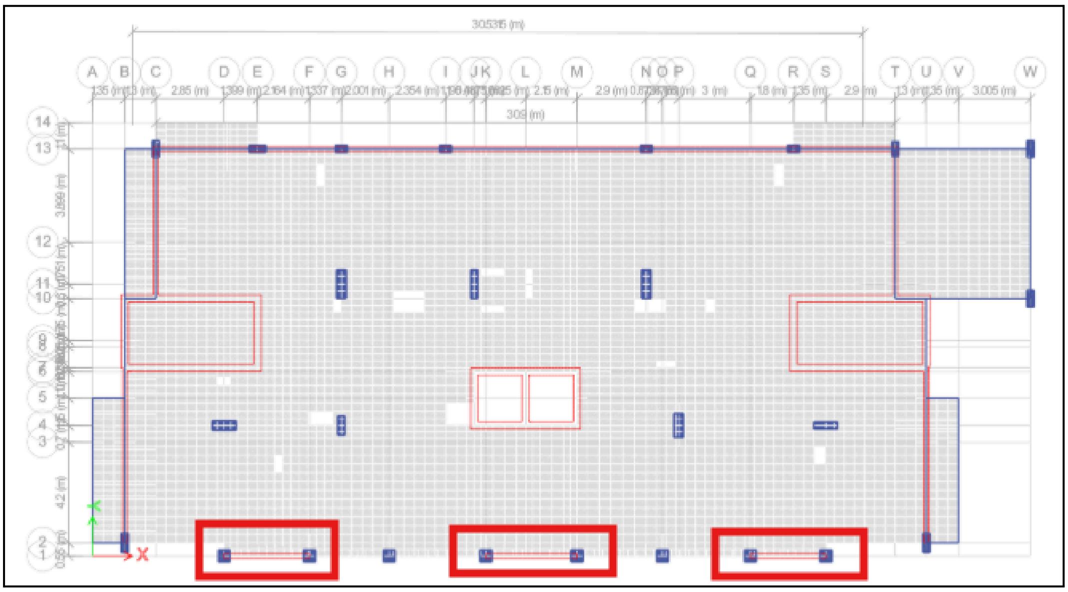


Figure 6.15: Plan view of ground floor, preliminary adjustment of walls distribution

Table 6.3: Wall modifications

Walls	Old thickness	Modified thickness
W8	300	450
W11	300	400
W14	300	400
W23	200	300
W24	200	300
W25	200	300
W26	200	300

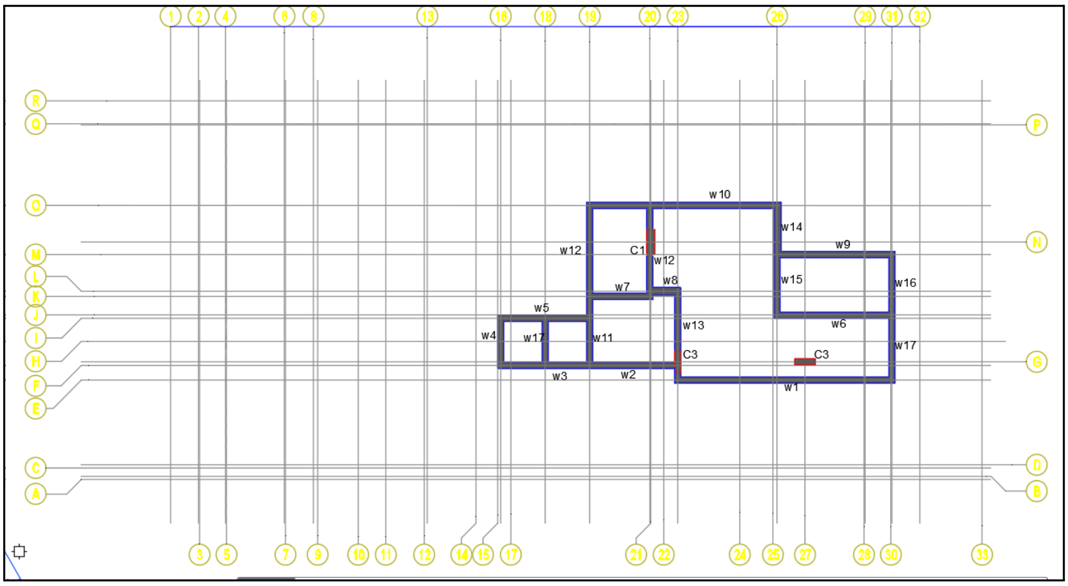


Figure 6.16: Wall naming layout at the basement level

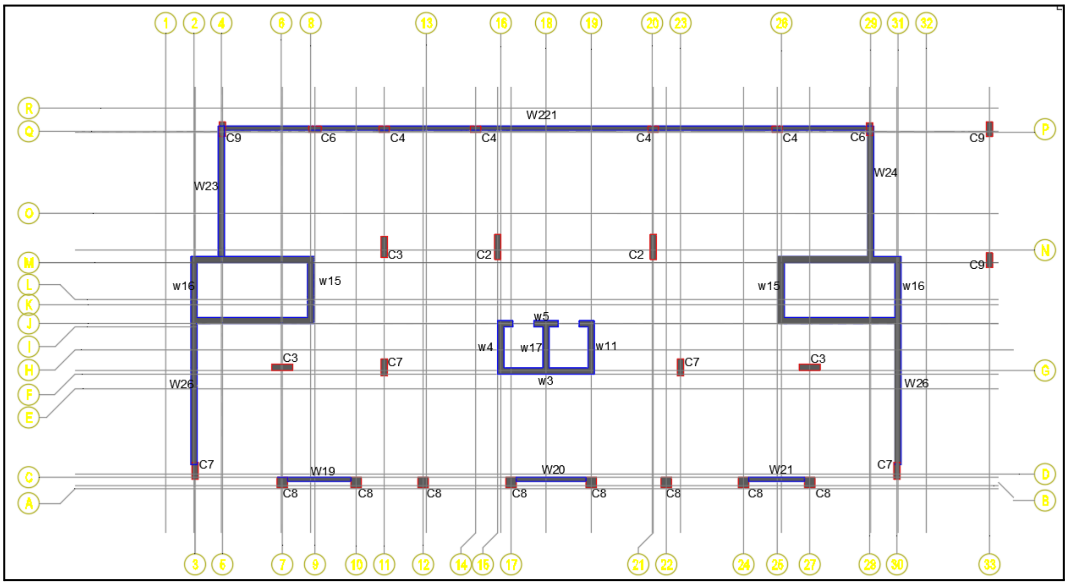


Figure 6.17: Wall naming layout at the ground floor level

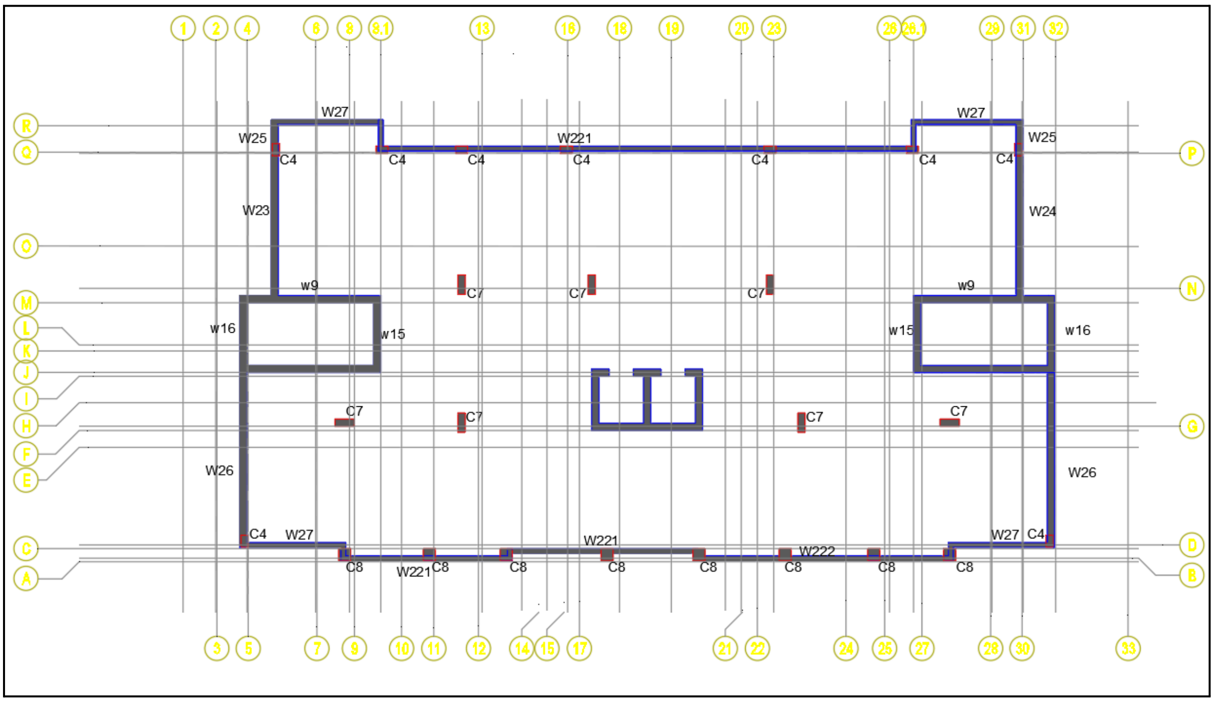


Figure 6.18: Wall naming layout at typical floors

6.4.3 Slabs

Slab thicknesses across the structure remained unchanged; however, in certain areas, the reinforcement was enhanced with additional top and bottom bars to meet higher demand requirements. **Figures 6.19** and **6.20** illustrate the regions where increased reinforcement was necessary to ensure adequate strength and serviceability under seismic loading conditions.

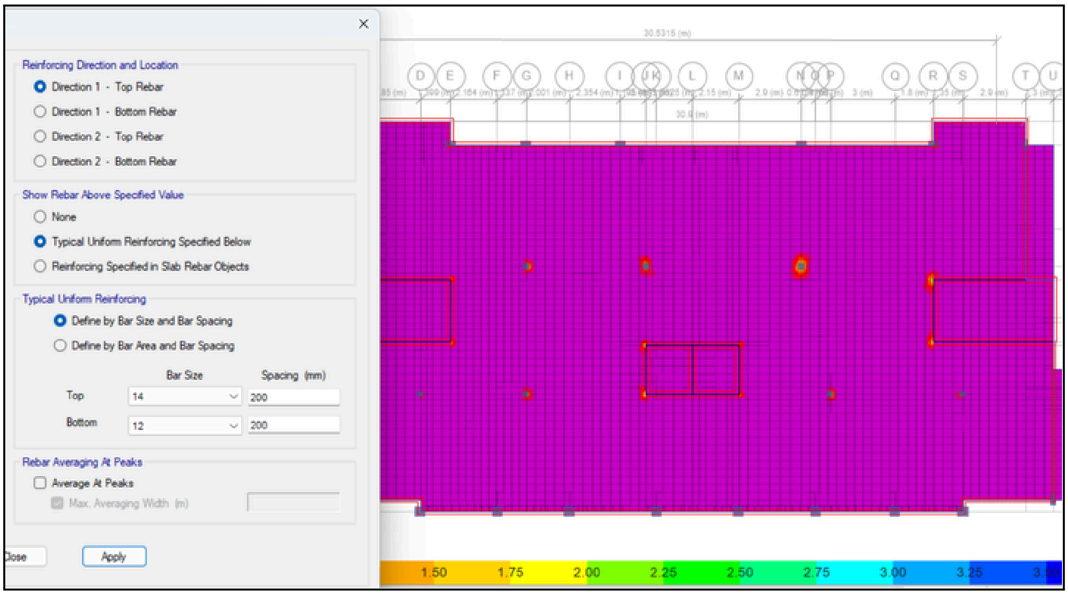


Figure 6.19: Enhanced reinforcement at story 8

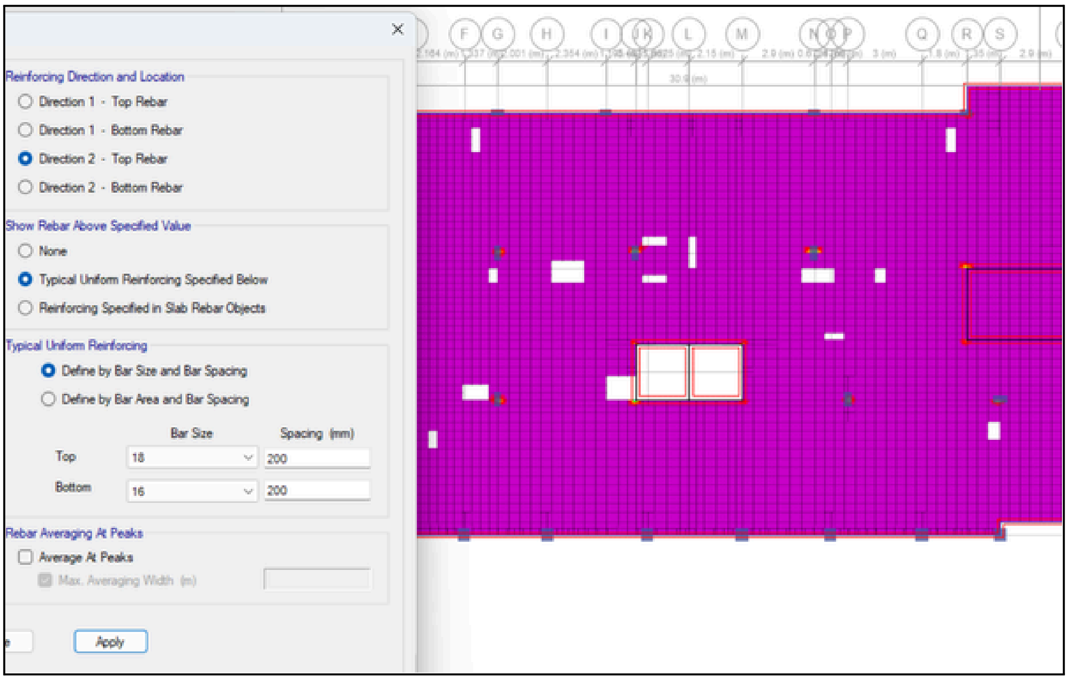


Figure 6.20: Enhanced reinforcement at story 7

6.4.4 Footings

Footing thicknesses and dimensions remained unchanged throughout the design process. The reinforcement layout also stayed consistent with the initial design, as structural analysis confirmed that the original configuration provided adequate strength and stability under the applied lateral loads.

6.4.5 Tie beams

The tie beam cross-section remains unchanged from the preliminary design; however, the reinforcement detailing has been modified. These changes were made based on updated analysis under seismic load combinations, which indicated increased demand in certain regions. The new reinforcement layout ensures adequate shear and flexural strength, meeting the requirements of ACI 318-19.

6.5 Software design vs. Code requirements

In this section, key design checks were performed to ensure that the seismic design results generated by ETABS are consistent with the provisions of ACI 318-19. The focus here is specifically on seismic-related checks, as gravity load design checks were already addressed in Part 1 of the project and are not repeated. The verification process concentrated on critical structural elements, including beams, columns, and structural walls. For each element type, representative members were selected and evaluated through code-based checks.

6.5.1 Beams

Beams were checked to ensure that the required reinforcement ratios correspond to singly reinforced sections, avoiding the need for doubly reinforced designs for practical execution. In addition, the beams were examined with respect to the design shear force (Ve).

6.5.1.1 Singly vs. Doubly reinforced sections

The maximum area of tensile reinforcement for singly reinforced sections is limited so that the net tensile strain in the extreme tension steel at nominal strength is at least 0.005, as stipulated by ACI 318-19. This strain threshold promotes ductile behavior and is a key criterion distinguishing singly from doubly reinforced sections. Based on strain compatibility and force equilibrium (assuming a balanced section where tensile strain just reaches 0.005), and for a rectangular cross-sectional beam, *Arman (2025b)* derives the following expression for the maximum reinforcement ratio in singly reinforced beams (Equation 3.26):

$$\rho_{\text{max,singly},\,\varepsilon_t=0.005} = 0.375\,\beta_1\,\frac{0.85 f'_c}{f_y} \qquad \textbf{(Arman, 2025b, Eq. 3.26)}$$

By applying Equation 3.26 with a concrete compressive strength **f'_c = 28 MPa** and a steel yield strength **f_y = 420 MPa**, the maximum reinforcement ratio for singly reinforced sections is calculated as **ρ_{max} = 0.0181**. Since ETABS expresses reinforcement ratios relative to the effective depth d, rather than the total section depth h, this theoretical value was adjusted by multiplying it with the ratio **d/h = 0.92**, resulting in a modified limit of approximately **1.66%**. This adjustment ensures consistency between the theoretical limit and the software output for comparison purposes.

Figures 6.21 show that the total rebar percentage in all beams ranges from 0.3% in the wall-supported beams, to 0.46% in the column-supported beams, well below the calculated limit of 1.66%. This confirms that all beams are singly reinforced and meet the criteria for ductile, tension-controlled behavior.

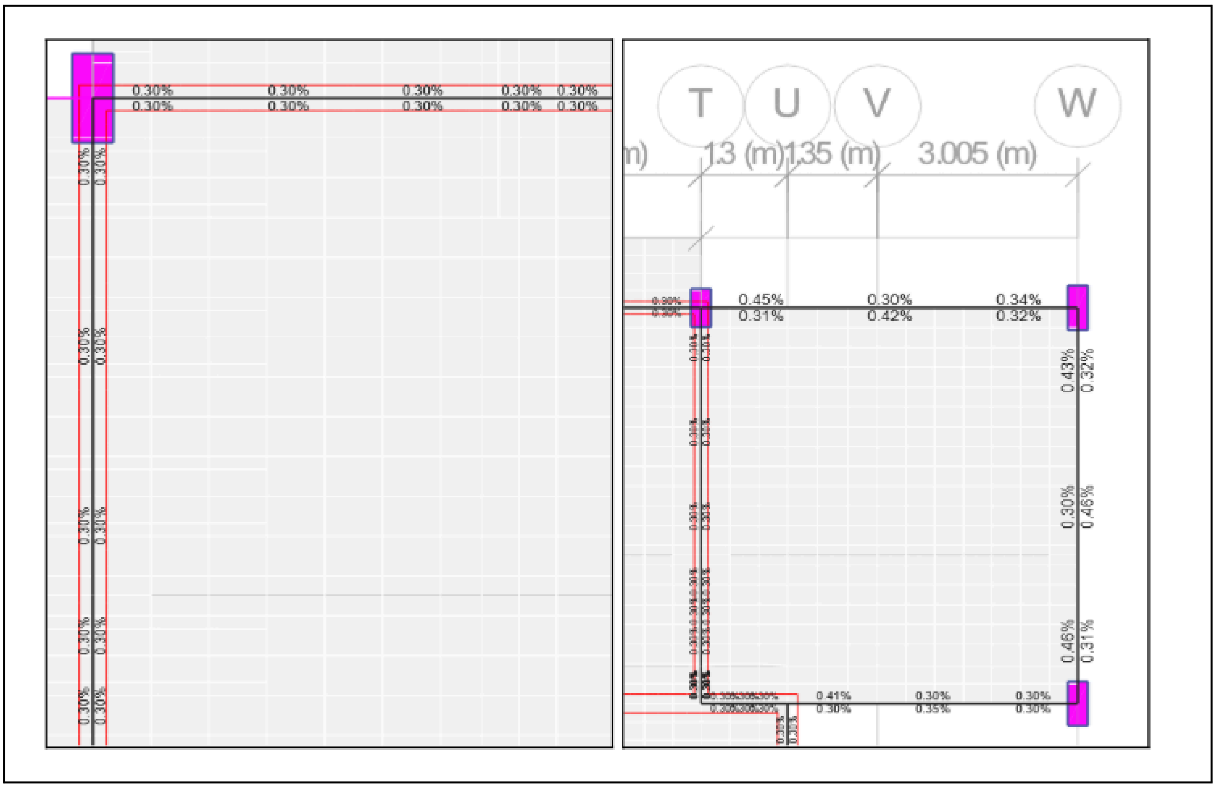


Figure 6.21: Rebar percentage in all beams

6.5.1.2 Design shear force check (V_e)

This check verifies the design shear force, **Ve**, in accordance with ACI 318-19 Section 18.6.5.1. The intent is to validate that the beam design accounts for critical shear demands arising from moment reversal due to seismic loading, in combination with factored gravity effects, thereby ensuring compliance with code-based strength assumptions.

Objective and methodology:

According to ACI 318-19 Section 18.6.5.1, the design shear force **Ve** must be evaluated for the segment of the beam between the faces of the joints, assuming moments of opposite sign at the supports equal to the *probable flexural strength* (**M_{pr}**) and considering the full factored gravity and vertical seismic loads. The equation applied is:

$$Ve = \frac{Mpr\ (left) + Mpr\ (right)}{L\ clear} + \text{gravity shear}$$

Where:

- M_{pr} = probable flexural strength at each end of the beam.

- L_{clear} = clear span between joint faces.

Unlike the conventional nominal moment $\varnothing M_n$, the **probable moment** M_{pr} excludes the strength reduction factor $\varnothing$ and includes a **multiplier of 1.25** to reflect strain-hardening and overstrength during seismic loading (per ACI commentary guidance):

$$M_{pr} = 1.25 \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$$

Where **1.25 fy**, representing the probable yield strength of transverse reinforcement, is commonly denoted as **fyt** in seismic design provisions.

Inputs and Assumptions:

- Beam section: 300 mm width × 500 mm total depth.
- Effective depth, d=460 mm.
- Flexural reinforcement area used for M_{pr} calculation: **455 mm²**, representing the minimum required area for flexure only, excluding any reinforcement added for torsion. This isolates the contribution relevant to flexural moment strength, which governs M_{pr}.
- Steel yield strength: f_y=420 MPa.
- As per ACI 318-19 Section 18.6.3, the probable yield strength is taken as:

  $$f_{yt}=1.25\times f_y=525 \text{ MPa}$$

- Concrete compressive strength: f′c=28 MPa.
- Clear span between joint faces: **5.55 m** (out of 6.25 m total length).

Calculation:

Step 1: Compute depth to the compression block (a) using the simplified equation derived from internal force equilibrium (tension = compression), as proposed by *Arman (2025b, Eq. 3.10)*:

$$a = \frac{A_s f_y}{0.85 f_c' b} \quad \text{(Arman, 2025b, Eq. 3.10)}$$

$$= \frac{455 \ x \ 525}{0.85 \ x \ 28 \ x \ 300} = 33.45 \text{ mm}$$

This calculated value uses the expected steel strength (f_{yt}) rather than the nominal yield strength (f_y) to maintain consistency with the moment strength calculation (M_{pr}), as the original equation is derived for standard conditions rather than for expected strength scenarios.

Step 2: Compute M_{pr}:

$$M_{pr} = A_s f_{yt} \left(d - \frac{a}{2}\right)$$

$$= 455 \text{ x } 525 \text{ x } (460 - \frac{33.45}{2}) = 105.88 \text{ KN.m}$$

Step 3: Compute Ve (due to seismic effects only):

$$\text{Ve} = \frac{2 \, x \, 105.88}{5.55} = 38.15 \text{ KN}$$

Comparison with ETABS Output:

- Seismic only:

  Manual Ve = 38.15 KN

  Software Ve = 34.77 KN (shown in **Figure 6.22)**

  % difference = (38.15 - 34.77) / 34.77 * 100 = 9.721 %

- Total (gravity + seismic):

  Manual V_{design} = 37.97 + 38.15 = 76.12 KN

  Software V_{design} = 37.97 + 34.77 = 72.74 KN (shown in **Figure 6.X)**

  % difference = (76.12 - 72.74) / 72.74 * 100 = 4.647 %

Design Forces

Factored V_{u2} kN	Factored M_{u3} kN-m	Design V_{u2} kN	Capacity V_p kN	Gravity V_g kN	Factored T_u kN-m	Design T_u kN-m
42.7345	-35.2825	72.7421	34.77	37.9721	1.6097	18.5363

Figure 6.22: Design shear force

Comment:

The manually calculated shear force exhibits a slight divergence from the ETABS design output. This discrepancy may be attributed to ETABS typically using the center-to-center distance between supports—in this case, the full span length of 6.25 m—which does not reflect the actual clear span used in the manual check. When this center-to-center length is used in the manual calculation instead, the resulting shear force Ve becomes 33.87 kN, leading to a percentage difference of only 2.56%, which is considered acceptable.

6.5.2 Columns

As required by ACI 318-19, all columns must undergo detailed checks to ensure they meet strength, ductility, and confinement requirements under both gravity and seismic loading conditions.

cheeks:

6.5.2.1 Axial force and biaxial moment design for P_u, M_{u2}, and M_{u3}

For column N11 at story 3:

Column element details are shown in **Figure 6.23**, where it is located at ground floor (story3).

Column Element Details

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	LLRF	Type
Story3	C19	626	column 1200*400	UDConS2	0	4400	0.421	Sway Special

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
1200	400	59	27.3

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
24870	28	1	420	420

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 % %
3192.1406	174.806	-86.9539	163.5653	86.9539	4800	1

Figure 6.23: Column N11 details

ACI commentary gives the following equation, originally presented byBresler, for calculating the capacity of a column under biaxial bending .

$$\frac{1}{\Phi pn} = \frac{1}{\Phi pnx} + \frac{1}{\Phi pny} - \frac{1}{\Phi pno}$$

ΦPno= reduced nominal axial nominal axial load capacity for ex and ey equal zero.

$\phi Pn=\phi*\lambda[0.85fc'(Ag-As)+fyAs]$

$\phi Pn=0.65*0.8*[0.85*28(480000-4800)+(420*4800]=6929.4KN$

Mu3=174.806 KN.m

Mu2= -86.95 KN.m

ϕPn=6929.4KN

ϕPnx=6929.4KN.m

ϕPny= 6929.4KN.m

Figure 6.24 shows the values of axial load, Mu2 and Mu3 along with their corresponding interaction diagrams, used to verify the column capacities under combined axial and bending moments.

M3(KN.m)	P(KN)
0	6929.395
208.734	6929.395
329.7806	6409.623
420.5055	5396.127
479.3275	4321.56
513.1731	3117.29
555.4237	2741.788
570.1162	2146.95
428.2448	912.8054
194.1004	-698.004
0	-1814.4

M2(KN.m)	P(KN)
0	6929.395
579.6141	6929.395
932.0331	6569.428
1195.321	5648.241
1376.074	4672.649
1484.892	3607.866
1681.01	3113.705
1785.44	2402.715
1413.853	1106.744
864.4184	-205.578
-1814.4	0

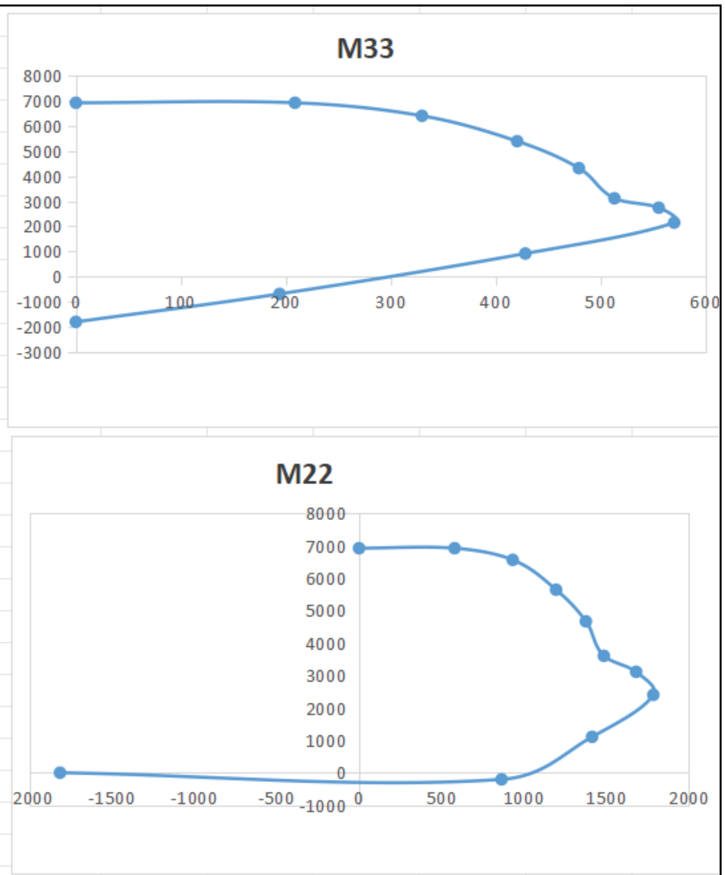


Figure 6.24: Values of axial load, Mu2 and Mu3 for column C3

$$\frac{1}{\Phi pn}=\frac{1}{6929.4}+\frac{1}{6929.4}-\frac{1}{6929.4}$$

Φpn=6929.4 KN

Pu=3192.14KN

Φpn > Pu → Satisfied

6.5.2.2 Check for V_p

Figure 6.25 illustrates the values of M_{pr} (up), M_{pr} (down), and V_P as obtained from ETABS.

for column W13

VP=Mpr(up) + Mpr(down)/L

L=4.4m

Mpr (up)=703.991KN.m

Mpr (down)=703.991 KN.m

VP=703.991+703.991/4.4=314 KN

VP from ETABS=0

314 <VP (ETABS) OK.

ETABS 22.4.0 License #*15RKRS4986Q44DA

Capacity Shear (Part 1 of 2)

	Shear V_p kN	Long.Rebar $A_{s(Bot)}$ %	Long.Rebar $A_{s(Top)}$ %	Cap.Moment M_{posBot} kN-m
Major Shear(V2)	0	1	1	703.991
Minor Shear(V3)	0	1	1	703.991

Capacity Shear (Part 2 of 2)

Cap.Moment M_{negTop} kN-m	Cap.Moment M_{negBot} kN-m	Cap.Moment M_{posTop} kN-m
703.991	703.991	703.991
703.991	703.991	703.991

Design Basis

Shr Reduc Factor Unitless	Strength f_{ys} MPa	Strength f_{cs} MPa	Area A_g cm²
1	420	28	3000

Concrete Shear Capacity

	Design V_u kN	Conc.Area A_{cv} cm²	Tensn.Rein A_{st} mm²
Major Shear(V2)	17.881	2826	1500
Minor Shear(V3)	25.103	2826	1500

Shear Rebar Design

	Stress v MPa	Conc.Cpcty v_c MPa	Uppr.Limit v_{max} MPa	ϕv_c MPa	ϕv_{max} MPa	RebarArea A_v /s mm²/m
Major Shear(V2)	0.06	0.96	4.48	0.58	2.69	820.8
Minor Shear(V3)	0.09	0.96	4.48	0.58	2.69	246.24

Figure 6.25: Column details of M_{pr} and V_p

6.5.3 Walls

Walls must be checked in accordance with ACI 318-19. The shear check was performed based on the provisions of Section 18.10.4 of ACI 318-19. This check was specifically applied to Wall W1, located in the basement, as illustrated in **Figure 6.26**. The corresponding shear design results obtained from ETABS are shown in **Figure 6.27**.

In addition, walls must also be checked for boundary elements in accordance with Section 18.10.6 of ACI 318-19. This check was also conducted for Wall W1 to ensure it meets the required detailing and confinement criteria. The axial load and moment used for the boundary check are shown in **Figure 6.28**.

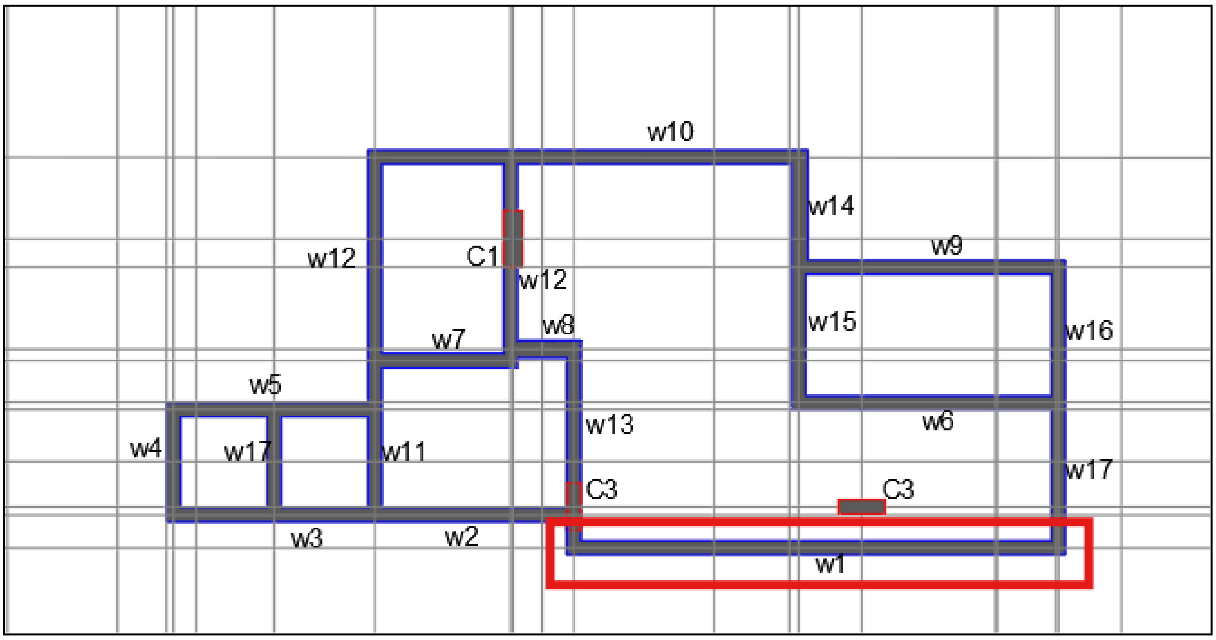


Figure 6.26: Wall W1 layout

6.5.3.1 Checks for walls

6.5.3.1.1 shear check

Shear Design (Part 2 of 2)

ϕV_c kN	ϕV_n kN
2455.6849	4411.8349
2455.6849	4411.8349

Boundary Element Check (ACI 18.10.6.3, 18.10.6.4)

Station Location	ID	Edge Length (mm)	Governing Combo	P_u kN	M_u kN-m	Stress Comp MPa	Stress Limit MPa	C Depth mm	C Limit mm
Top–Left	Leg 1	Not Required	UDConS25	803.1417	877.8558	0.09	5.6		
Top–Right	Leg 1	Not Required	UDConS25	803.1417	877.8558	0.42	5.6		
Bottom–Left	Leg 1	Not Required	UDConS24	3290.6593	641.8191	0.94	5.6		
Botttom–Right	Leg 1	Not Required	UDConS24	3290.6593	641.8191	1.18	5.6		

Figure 6.27: Shear design from ETABS

for W1

L=10.35m

thickness=0.3m

hw=3.05m

$$V_n = \left(\alpha_c \lambda \sqrt{f'_c} + \rho_t f_{yt}\right) A_{cv}$$

hw/Lw=3.05/10.35=0.29<1.5

αc=0.25

Vn=(0.25*1*√28+0.0025*420)*300*10350/1000=7367.78KN

ΦVn=0.6*7367.78=4420 KN

ΦVn(ETABS)=4411.84 KN

%different percentage=((4420/4411.84)-1)=0.2%

$$\emptyset V_c = \emptyset \alpha_c \lambda \sqrt{f'_c} A_{cv}$$

ΦVc=0.6*0.25*1*√28*10350*300/1000=2464.52KN.

ΦVc(ETABS)=2455.69KN

%different percentage=((2464.52/2455.69)-1)=0.4%

the maximum shear strength for the wall:

$$\emptyset V_n = \emptyset 0.66 \sqrt{f'_c} A_{cv}$$

=0.6*0.66*√28*10350*300/1000=6506.32 KN>4411.84 OK

6.5.3.1.2 Check for boundary elements

Flexural Design for P_u, M_{u2} and M_{u3}

Station Location	Required Rebar Area (mm^2)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P_u kN	Minimum M_2 kN-m	M_{u2} kN-m	M_{u3} kN-m
Top	7762	0.0025	0.0022	ENVELOPE-SER	442.2224	10.7195	33.0984	-109.3994
Bottom	7762	0.0025	0.0022	ENVELOPE-SER	1927.9682	46.7339	-187.0661	-920.542

Axial Force and Minor Moment Factors

Station Location	C_m Factor Unitless	δ_{ns} Factor Unitless	β_{dns} Unitless	EI_{eff} kN-m^2	P_c kN
Top Minor Bend(M2)	1	1.003853	0.6	144790.0313	153616.8051
Bottom Minor Bend(M2)	1	1.017019	0.6	144790.0313	153616.8051

Shear Design Capacity Parameters

Station Location	ID	Shear Combo	$M_{pr,major}$ kN-m	Ω_v Unitless	ω_v Unitless	$\Omega_v * \omega_v$ Unitless
Top	Leg 1	ENVELOPE	27840.4677	30.243561	1	1
Bottom	Leg 1	ENVELOPE	27840.4677	30.243561	1	1

Shear Design (Part 1 of 2)

Station Location	ID	Rebar mm^2/m	Shear Combo	P_u kN	M_u kN-m	V_u kN	$V_{u,design}$ kN	h_w / l_w Unitless	α_c Unitless
Top	Leg 1	750	ENVELOPE	192.6186	877.8558	985.7966	985.7966	0.294686	3
Bottom	Leg 1	750	ENVELOPE	1031.7894	858.8949	957.9774	957.9774	0.294686	3

Figure 6.28: Axial load and moment used for the boundary

For W1

Pu=Nu=1031.789 KN

Mu=920.542 KN.m

$$\sigma = \frac{N_u}{A} \mp \frac{M_u y}{I}$$

=((1031.789*1000)/(300*1030))+((920.542*10^6*5175)/(½)(300)(10350)^3

=0.3323+0.172=0.5043 < 5.6 OK.

no boundary needed.

6.6 ACI provisions for detailing considering seismic effects

This section delves into the **detailing provisions** outlined by ACI 318-19, specifically for the elements of the structure under seismic loads. It discusses how the structural components (such as walls, beams, and columns) are designed in compliance with ACI's requirements for seismic detailing. The section addresses things like reinforcement detailing, minimum reinforcement ratios, and any other relevant code specifications to ensure seismic resilience.

Frame members include the beams and columns. Frame members that are designated as part of the seismic-force-resisting system are treated differently from those dedicated to gravity-only resistance. ACI 318-19, Section 18.14 specifies the seismic design and detailing requirements of members not designated as part of the seismic-force-resisting system. Section 18.14.3 is concerned with cast-in-place beams, columns, and joints specifications. Slab-column connections are also of concern, detailed in Section 18.14.5 of the same code. Wall piers – Section 18.14.6. These requirements ensure wall piers have adequate strength and ductility to resist seismic forces, with proper reinforcement and detailing to prevent failure under lateral loads.

6.6.1 Beams

During this process, several conditional code provisions were reviewed. However, these provisions are triggered only under specific conditions that did not occur in this project.

As such, they were not applicable and are not discussed further. The requirements presented here are those directly applicable to the beam design without additional conditions, and thus represent the relevant limits that needed to be satisfied.

During this process, several code provisions were reviewed. However, many of these provisions are conditional in nature, that is, they apply only when specific criteria or circumstances are met. Since such conditions did not arise in this project, those provisions were not applicable. Therefore, they are not included or discussed in this report. The requirements presented here are limited to those that were directly applicable to the beam design, without reliance on additional or untriggered conditions. This approach ensures the report remains focused on the relevant and necessary design considerations.

The seismic evaluation confirmed that all relevant reinforcement requirements for beam members, as specified in *ACI 318-19*, were already satisfied in the original gravity design. In particular:

- **Section 18.14.3.2(a)** limits the maximum spacing of transverse reinforcement to *d/2*. With an effective depth *d* of 460 mm, the allowable spacing was 230 mm. The provided stirrup spacing was 90 mm for all beams, which satisfies this requirement.

- **Section 9.6.1.2** specifies a minimum required flexural steel area, $A_{s,min}$. This area corresponds to a reinforcement ratio of approximately $\rho = 0.0033$. The steel ratio provided matches this minimum, confirming compliance.
- **Section 18.6.3.1** requires a minimum of two longitudinal bars at both the top and bottom faces of the beam and limits the reinforcement ratio, ρ, to a maximum of 0.025. All beam sections were detailed with 3Ø18 bars at both the top and bottom faces, thereby satisfying the minimum bar count requirement. However, since a portion of this reinforcement is designated for torsional resistance, only the remaining portion—intended for flexural action—was considered in the steel ratio calculation, which resulted in $\rho = 0.0033$. This value is well below the maximum limit specified by the code. ETABS results for steel ratios, discussed in a previous section, align with this detailing approach too.

All of these conditions were already met in the original gravity design, and no revisions were necessary during the seismic detailing phase.

Note: It is not immediately obvious why provisions from sections typically not associated with LFRS members are applied here. However, Section 18.14 explicitly refers to earlier parts of the code — including Sections 18.6.3.1 and 9.6.1.2 — to define limits and detailing rules. This reference chain explains the necessity of considering requirements from other parts of the code.

6.6.2 Columns

According to Section 18.14.3.3(c) of the code, columns must comply with Sections 18.7.4, 18.7.5, and 18.7.6. Sections 18.7.4 and 18.7.5 address detailing provisions for longitudinal and transverse reinforcement, respectively, while Section 18.7.6 ensures that design provisions are sufficient to maintain safe behavior during seismic events. The latter has been handled by the software design and verified in Section 6.5.2.

6.6.2.1 Longitudinal reinforcement requirements

- **Steel Ratio:**
  The total area of longitudinal reinforcement must be at least 0.01Ag and not more than 0.06Ag.

- **Bar Development Length:**

  The selection of bar diameters must ensure that $1.25\ \ell d \le \ell u/2$, where ℓd is the development length and ℓu is the clear column height.

- **Lap Splices:**

  Lap splicing is allowed only within the central half of the column clear height and must be designed as tension lap splices.

  These splices must also be enclosed by transverse reinforcement in accordance with Sections 18.7.5.2 and 18.7.5.3, to be examined after the current detailing requirements.

Application to the Project:

- Regarding the **steel ratio**, although the upper limit in the code is 0.06Ag, a more conservative limit of 0.03Ag is typically adopted near joints due to localized increases in reinforcement. All columns in the project satisfy this, with steel ratios ranging between 1% and 3%. The detailed output from the software is included in **Appendix C**.

- The **development length condition** was satisfied by selecting bar diameters that meet the specified inequality. A practical check was performed using the worst-case combination of maximum bar diameter and minimum clear height. The detailed methodology and calculations are presented following the third point, confirming that the condition is satisfied.

- For **lap splicing**, the gravity design did not restrict splice placement to the central region. This issue was addressed in the seismic design phase, and the updated lap splice locations now comply with the code requirement. These are shown in the structural drawings provided in **Appendix B**.

  Furthermore, the required transverse reinforcement around lap splices is considered in the following discussion of Section 18.7.5.

Development Length Check for Bar Diameters:

To verify compliance with development length requirements, the check focused on the most critical group of columns: those along grid line 1 in the upper floors. These columns have the following specifications:

- Column dimensions: 500 mm × 500 mm

- Story height: 3.55 m (shortest in the project)
- Longitudinal bar diameter: 18 mm (largest in the project)

If this most critical case satisfies the condition, then all other columns—having either smaller bar diameters or greater clear heights—will inherently comply. This approach was chosen to streamline and expedite the verification process, as satisfying this condition for the most critical case ensures compliance for all other columns, removing the need for individual checks.

The required development length was calculated using the simplified expression proposed by *Arman (2025b, Eq. 5.1)*, which is based on Table 25.4.2.2 of the code:

$$l_{dt} = \frac{0.48 f_y}{\sqrt{f'_c}} d_b \quad \text{for bars less than 20 mm} \quad \text{(Arman, 2025b, Eq. 5.1)}$$

Where:

- $f_y = 420$ MPa
- $f'c = 28$ MPa
- $d_b = 18$ mm

Substituting values:

$$\frac{0.48 \text{ x } 420}{\sqrt{(28)}} \text{ x } 18 = \frac{201.6}{5.29} \text{ x } 18 \approx 686 \text{ mm}$$

The check condition is:

$$\mathbf{1.25}\boldsymbol{\ell_{dt}} \leq \frac{\boldsymbol{\ell_u}}{\mathbf{2}}$$

$$1.25 \times 686 = 857.5\text{mm} \leq \frac{3550}{2} = 1775\text{mm}$$

The condition is satisfied, confirming that development length requirements are met in the worst-case columns. As this scenario involves the shortest columns and largest bar diameter used, all other columns in the structure can be considered compliant as well.

6.6.2.2 Transverse reinforcement requirements

This section outlines the required configurations for transverse reinforcement of columns in special frames. Transverse reinforcement requirements include proper anchorage and engagement with longitudinal bars, acceptable crosstie arrangements, maximum allowable spacing, and additional provisions relevant to minimum transverse reinforcement area required.

Firstly, Section 18.7.5.1 mandates that closely spaced transverse reinforcement be provided over a defined length ℓ_o at both the top and bottom ends of each column to enhance ductility and confinement in critical regions. Then Section 18.7.5.3 sets maximum spacing limits for transverse reinforcement to ensure adequate shear strength in seismic loading. Section 18.7.5.4 then establishes the minimum required area of transverse reinforcement.

6.6.2.2.1 Zones of transverse reinforcement densification (ℓ_o)

The length ℓ_o is taken as the greatest of the following three values:

(a) The depth of the column at the joint face or at the section where flexural yielding is expected.

(b) One-sixth of the clear span of the column.

(c) 450 mm.

For criterion (a), the maximum **column depth** at locations where flexural yielding is expected is summarized in **Table 6.3**, which presents the dimensions of each column across different stories of the structure. This table also includes the final ℓ_o values, determined by comparing all three criteria.

For criterion (b), one-sixth of the clear span of the column is calculated as follows:

- **Ground floor:** (1/6) x 4400 mm = **733 mm**, rounded to **750 mm**

- **Typical floors:** (1/6) x 3550 mm = **591.6 mm**, rounded to **600 mm**

Since this criterion governs for certain columns (i.e., provides the maximum among the three cases), the required **transverse reinforcement length** is taken as:

- **ℓ_o = 750 mm** for columns at the ground floor **(story3)**
- **ℓ_o = 600 mm** for columns at typical floors (**stories 4 to 8)**

These values are used in cases where **n**either the column depth (criterion a) nor the minimum limit of 450 mm (criterion c) controls, and **one-sixth of the span (criterion b)** governs.

Table 6.4: Column expected depth to undergo flexural yielding

Column	**Story**	**Dimension**	**Flexural yielding dimension**	**ℓo**
G11	3	1200*400	1200	**1200**
J11	3	1200*300	1200	**1200**
D4	3	1000*400	1000	**1000**
G4	3	800*300	800	**800**
S4	3	1000*300	1000	**1000**
D1	3	500*500	500	**750**
E13	3	700*300	700	**750**
G13	3	500*300	500	**750**
G11	5	600*300	600	**600**
G11	7	300*300	300	**600**

6.6.2.2.2 Spacings of transverse reinforcement

Spacing is applied according to section 18.7.5.3, spacing shall not exceeds the lesser of four values:

(a) One-fourth of the minimum column dimension

(b) For Grade 420, 6db of the smallest longitudinal bar

(c) For Grade 550, 5db of the smallest longitudinal bar

(d) s_o, as calculated by:

Case (a) the minimum dimension of columns are 400mm, 500mm and 300mm, so the spacing of transverse reinforcement for those columns are, ¼*400= 100 mm, ¼*500= 125 mm and ¼*300= 75 mm.

Between both cases (b) and (c), case (b) is selected due to steel grade 420, the spacing between transverse reinforcement is different from each column, it relies on the bar diameter. Table 6.4 shows the 6db of the smallest longitudinal bar:

Table 6.5: $6d_b$ of the smallest longitudinal bar for each column

Column dimension (mm)	Bar Diameter d_b (mm)	$6 * d_b$
300*800	14	84
300*600	14	84
500*500	18	108
300*700	14	84
1200*400	18	108
1200*300	16	96
1000*300	16	96
1000*400	16	96
300*300	14	84
500*300	14	84

Lastly case (d), a value of S_o shall be calculated for each column, **Table 6.5** shows S_o value for each column.

Table 6.6: Value of S_o for columns

column dimension (mm)	S_o (mm)
300*800	130
300*600	130
500*500	150
300*700	150
1200*400	120
1200*300	120
1000*300	130
1000*400	130
300*300	150
500*300	150

Table 6.6 shows the value transverse reinforcement spacing within ℓ_o for each columns section across the building.

Table 6.7: transverse reinforcement spacing and length

Column dimension (mm)	ℓ_o	Transverse reinforcement spacing within ℓ_o (mm)
300*800	800	75
300*600	600	75
500*500 (Ground floor)	750	100
500*500 (typical floors)	600	100
300*700	750	75
1200*400	1200	100
1200*300	1200	75
1000*300	1000	75
1000*400	1000	100
300*300	750	75
500*300	600	75

6.6.2.2.3 Minimum transverse reinforcement ratio evaluation

Table 18.7.5.4 in ACI 318-19 specifies transverse reinforcement for columns of special frames. In this table, the transverse reinforcement ratios are calculated for the condition that complies with the case, taking the greatest value from them, **Figure 6.29** illustrates transverse moment reinforcement for columns.

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)

Figure 6.29: Transverse moment reinforcement for columns

In the case of this project, **Equations (a)**, **(b)** and **(c)** are those of interest, (a) is calculated and presented in **Table 6.8**, while (b) and (c) are as follows:

(b) 0.09 * (28/420) = 0.006

As for **Equation (c)**, it was evaluated under an *imaginary extreme case* that does not actually occur in any column, but rather assumes the worst combination of parameters across all columns. This approach **eliminates the need to repeat the complex and time-consuming calculations of Equation (c) for every individual column**, as even the most extreme theoretical case does not exceed the minimum value dictated by Equation (a). :

- Maximum k_n=1.3 (found in 300×300, 300×500, and 300×600 mm columns)
- Minimum transverse confined core area: A_{ch}=48400 mm^2 (found in columns 300×300)
- Maximum axial load: Pu=3231032 N (found in column 1200×400 mm at the ground floor — see **Figure 6.30**)
- k_f: $\frac{28}{175} + 0.6 = 0.76 < 1 \rightarrow 1$ is considered

- Yield strength: fy=420 MPa

Substituting into Equation (c):

$$0.2 * 1.33 * 1 * 3231032 / (420 * 358400) = 0.0057$$

Thus, even under an artificially constructed worst-case scenario that combines the most unfavorable values of all parameters in Equation (c), the resulting ratio is approximately **0.0057**. This value remains **lower than the corresponding results from both Equation (a) and Equation (b)**. Since Equation (a) provides the highest required reinforcement ratio, it is clearly the **governing criterion** in all cases. Accordingly, Equation (a) is adopted as the **controlling minimum transverse reinforcement requirement** across all columns in the project.

Figure 6.30 shows the axial force Pu in column 1200*400 at the ground floor.

Table 6.8 presents the calculated values of equation (a) for all column sections throughout the structure.

Story	Label	UniqueName	Section	Location	P kN	M Major kN-m	M Minor kN-m	PMM Combo
Story3	C19	626	column 1200*400	Bottom	3231.0318	-88.0133	150.8212	UDConS2

Figure 6.30 : Axial force Pu in column 1200*400 at the ground floor

Table 6.8: Values of equation (a) for all column sections

column dimension (mm)	A_g (mm2)	A_{ch} (mm2)	n	k_n	Equation (a)
300*800	240000	158400	1.25	1.25	0.01030303
300*600	180000	114400	1.333333333	1.333333333	0.011468531
500*500	250000	176400	1.2	1.2	0.008344671
300*700	210000	136400	1.25	1.25	0.010791789
1200*400	480000	358400	1.166666667	1.166666667	0.006785714
1200*300	360000	246400	1.2	1.2	0.009220779
1000*300	300000	202400	1.2	1.2	0.009644269
1000*400	400000	294400	1.166666667	1.166666667	0.007173913
300*300	90000	48400	1.333333333	1.333333333	0.017190083
500*300	150000	92400	1.333333333	1.333333333	0.012467532

According to Table 18.7.5.4 from ACI 318-19 the largest value from the three equations (a), (b), and (c), must be compared with Ash/sbc value, which is shown in **Table 6.8**.

Table 6.9: Value of Ash/sbc for columns across the structure

Column	Bars area x	bc x	Bars area y	bc y	S	Ash/sbc (x)	Ash/sbc (y)	Ash/sbc
300*800	235.5	220	314	720	75	0.014272727	0.005814815	**0.0142**
300*600	235.5	220	235.5	520	75	0.014272727	0.006038462	**0.0142**
500*500	314	420	314	420	100	0.00747619	0.00747619	**0.0074**
300*700	235.5	220	314	620	75	0.014272727	0.006752688	**0.0142**
1200*400	314	320	392.5	1120	100	0.0098125	0.003504464	**0.0098**
1200*300	235.5	220	392.5	1120	75	0.014272727	0.004672619	**0.0142**
1000*300	235.5	220	392.5	920	75	0.014272727	0.005688406	**0.0142**
1000*400	314	320	392.5	920	100	0.0098125	0.004266304	**0.0098**
300*300	314	220	314	220	75	0.019030303	0.019030303	**0.0190**
300*500	235.5	220	235.5	420	75	0.014272727	0.00747619	**0.0142**

The spacing between transverse reinforcement bars in the 500×500 mm column was selected as 75 mm, which is greater than the required minimum value of *(a)*.
Ash/sbc= 314/75*420= 0.00996> 0.0083

After comparing the values in **Table 6.7** and **Table 6.8**, **Table 6.9** summarizes the final transverse reinforcement requirements for all columns across the structure.

Table 6.10: value of minimum Transverse reinforcement across the section

Column	Transverse reinforcement
300*800	0.0142
300*600	0.0142
500*500	0.00996
300*700	0.0142
1200*400	0.0098
1200*300	0.0142
1000*300	0.0142
1000*400	0.0098
300*300	0.019
300*500	0.0142

6.6.3 Walls

According to ACI 318-19, wall transverse reinforcement vertical spacing shall satisfy the provisions of Section 18.10.6.5(b). The detailing and placement of this reinforcement, ensuring adequate resistance to out-of-plane moments, is illustrated in **Figure 6.31**.

Table 18.10.6.5(b)—Maximum vertical spacing of transverse reinforcement at wall boundary

Grade of primary flexural reinforcing bar	Transverse reinforcement required	Maximum vertical spacing of transverse reinforcement[1]	
420	Within the greater of ℓ_w and $M_u/4V_u$ above and below critical sections[2]	Lesser of:	$6d_b$
			150 mm
	Other locations	Lesser of:	$8d_b$
			200 mm
550	Within the greater of ℓ_w and $M_u/4V_u$ above and below critical sections[2]	Lesser of:	$5d_b$
			150 mm
	Other locations	Lesser of:	$6d_b$
			150 mm
690	Within the greater of ℓ_w and $M_u/4V_u$ above and below critical sections[2]	Lesser of:	$4d_b$
			150 mm
	Other locations	Lesser of:	$6d_b$
			150 mm

[1]In this table, d_b is the diameter of the smallest primary flexural reinforcing bar.

[2]Critical sections are defined as locations where yielding of longitudinal reinforcement is likely to occur as a result of lateral displacements.

Figure 6.31: Transverse reinforcement spacing for boundaries (ACI 318-19, Table 18.10.6.5(b))

The vertical spacing for transverse reinforcement in all boundary elements has been uniformly taken as 150 mm, ensuring compliance with ACI 318-19 requirements for confinement and structural integrity.

6.7 Final design with specified detailing requirements

This section highlights the finalized design under combined gravity and seismic loading, emphasizing the resulting cross-sectional dimensions, reinforcement quantities, and detailing of all structural elements. It provides a clear overview of the structural outcomes developed throughout this chapter, serving as a concise reference to the adopted design solutions.

6.7.1 Frame members design

Frame members generally retained their original gravity-only design requirements, with only minor increases in demand observed in a limited number of elements. Although these increases were modest—given that the frames do not serve as the primary lateral resisting system—they were addressed with full consideration. The final design presented here reflects these adjustments.

6.7.1.1 beams design

The beam design remained unchanged in terms of longitudinal reinforcement, transverse reinforcement, and cross-sectional dimensions. The original gravity design was found to be satisfactory in resisting the relatively minor seismic effects. The complete design is presented in Chapter 3 (Part One) and can be referred to for full design details.

6.7.1.2 columns design

Column flexural and transverse final design will be discussed in this section. The design includes the final selection of longitudinal reinforcement to resist bending moments, as well as transverse reinforcement to ensure adequate confinement and shear capacity. Design checks were performed according to ACI 318M-19 provisions to ensure the columns meet strength and ductility requirements under seismic loading conditions.

6.7.1.2.1 flexural design

Table 6.10 illustrates the flexure design for all columns in each floor.

Table 6.11: Column flexural design

Columns	Column 1	Column 2	Column 3	Column 4	Column 5	Column 6	Column 7	Column 8	Column 9	Column 10	Column 11
Dimension	300*300	300*500	300*600	300*800	300*1000	500*500	300*1200	400*1200	300*700	1000*400	300*500
Basement grid line	-	-	-		P4, S4			N11			
Ground floor grid line	-	G13, I13, N13, R13	T13	B2, U2, G4	S4	D1, F1, H1, K1, M1, O1, Q1, S1	J11	G11, N11	C13, W13, W10, E13	D4, P4	
1st floor grid line	-	C13, E13, G13, I13, N13, R13, T13, B2, U2	-	G11, J11, N11, D4, G4, P4, S4		D1, F1, H1, K1, M1, O1, Q1, S1					
2nd floor grid line	-	C13, E13, G13, I13, N13, R13, T13, B2, U2	G11, J11, N11, D4, G4, P4, S4			D1, F1, H1, K1, M1, O1, Q1, S1					
3rd floor grid line	-	C13, E13, G13, I13, N13, R13, T13, B2, U2	G11, J11, N11, D4, G4, P4, S4			D1, F1, H1, K1, M1, O1, Q1, S1					
4th floor grid line	-	C13, E13, G13, I13, N13, R13, T13, B2, U2	G11, J11, N11, D4, G4, P4, S4			D1, F1, H1, K1, M1, O1, Q1, S1					
5th grid line	G11, J11, N11, D4, G4, P4, S4	C13, G13, I13, N13, T13, B2, U2	-			D1, F1, H1, K1, M1, O1, Q1, S1					E13, R13
Area of steel %	1%	1%	1%	1%	1%	1%	1%	1%	1%	1%	1.40%
Area of steel	900	1500	1800	2400	3000	2500	3600	4800	2100	4000	2100
Longitudinal bars	8ø14	12ø14	12ø14	16ø14	18ø16	12ø18	20ø16	22ø18	16ø14	20ø16	12ø16

Table 6.10 illustrates the area of steel for each column along with the bar distribution. The development length for each section will be indicated in the column drawings.

6.7.1.2.2 Transverse reinforcement

Transverse reinforcement will be designed in accordance with ACI 318M-19. The transverse reinforcement length ℓ_o for each column in Story 3 is shown in **Table 6.11**, while the ℓ_o values for the remaining stories are provided in **Table 6.12**.

Table 6.12: Transverse length ℓ_o of each column in story 3

Column Dimension	300*700	300*500	300*600	1200*400	1200*300	1000*400	1000*300	800*300	500*500
ℓo (mm)	750	750	750	1200	1200	1000	1000	800	750

Table 6.13: Transverse length ℓ_o of each column in all stories expect story 3

Column Dimension	300*700	300*500	300*600	1200*400	1200*300	1000*400	1000*300	800*300	500*500	300*300
ℓo (mm)	600	600	600	1200	1200	1000	1000	800	600	600

Spacing between each bar of transverse reinforcement is shown in **Table 6.13**.

Table 6.14: Spacing between each bar of transverse reinforcement

Column dimension (mm)	Transverse reinforcement spacing within ℓ_o (mm)
300*800	75
300*600	75
500*500	75
300*700	75
1200*400	100
1200*300	75
1000*300	75
1000*400	100
300*300	75
500*300	75

Minimum Transverse Reinforcement for each column

Table 6.14 shows the amount of transverse reinforcement provided in each column throughout the structure.

Table 6.15: Minimum Transverse Reinforcement for each column

Column	Transverse reinforcement
300*800	0.0142
300*600	0.0142
500*500	0.0093
300*700	0.0142
1200*400	0.0098
1200*300	0.0142
1000*300	0.0142
1000*400	0.0098
300*300	0.019
300*500	0.0142

summary

The column transverse reinforcement design will be taken as shown in **Table 6.15**, bars distribution within ℓ_o is shown in **Table 6.16**, and out side ℓ_o is shown in **Table 6.17** .

Table 6.16: Transverse reinforcement design

Column	Transverse reinforcement	Transverse reinforcement spacing within ℓ_o (mm)	Transverse length ℓ_o
300*800	0.0142	75	800
300*600	0.0142	75	750 (story 3) 600 (other stories)
500*500	0.0083	100	750 (story 3) 600 (other stories)
300*700	0.0142	75	750 (story 3) 600 (other stories)
1200*400	0.0098	100	1200
1200*300	0.0142	75	1200
1000*300	0.0142	75	1000
1000*400	0.0098	100	1000
300*300	0.019	75	600
300*500	0.0142	75	750 (story 3) 600 (other stories)

Table 6.17: Distribution of transverse bars within ℓ_o.

Column	Transverse reinforcement	Reinforcement within ℓ_o
300*800	0.0142	ø10/75 mm
300*600	0.0142	ø10/75 mm
500*500	0.0083	ø10/75 mm
300*700	0.0142	ø10/75 mm
1200*400	0.0098	ø10/100 mm
1200*300	0.0142	ø10/75 mm
1000*300	0.0142	ø10/75 mm
1000*400	0.0098	ø10/100 mm
300*300	0.019	ø10/75 mm
300*500	0.0142	ø10/75 mm

Table 6.18: Distribution of transverse bars outside ℓ_o

Column	Reinforcement outside ℓ_o
300*800	ø10/200 mm
300*600	ø10/200 mm
500*500	ø10/250 mm
300*700	ø10/200 mm
1200*400	ø10/250 mm
1200*300	ø10/250 mm
1000*300	ø10/250 mm
1000*400	ø10/250 mm
300*300	ø10/200 mm
300*500	ø10/200 mm

6.7.2 Structural walls design

Structural wall design was carried out in accordance with the provisions of ACI 318M-19. The naming and layout of the structural walls are illustrated in **Figures 6.32-6.34**. **Tables 6.18-6.20** present the reinforcement details of the structural walls across the various elevations of the building, including both vertical and horizontal reinforcement. For walls requiring boundary elements, the corresponding reinforcement details are provided in **Table 6.21**. The reinforcement of spandrels, both vertically and horizontally, is shown in **Table 6.22**.

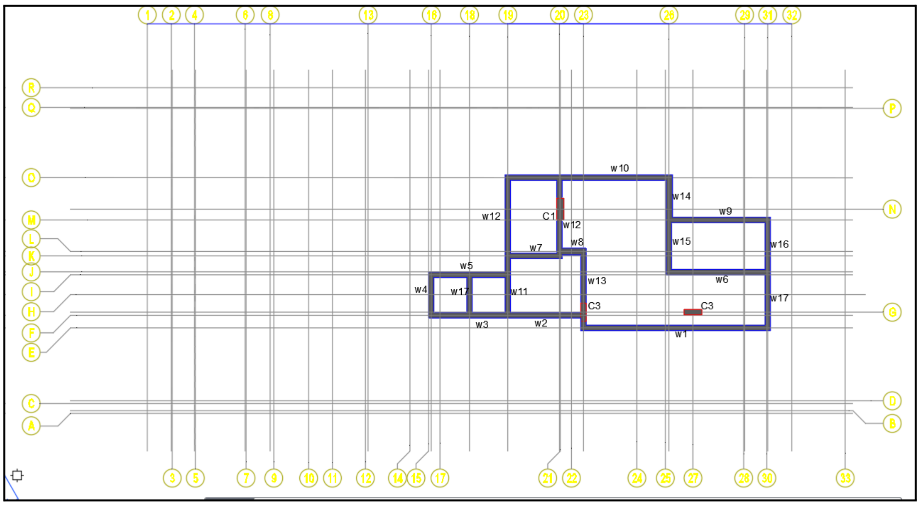


Figure 6.32: Wall naming layout at the basement level

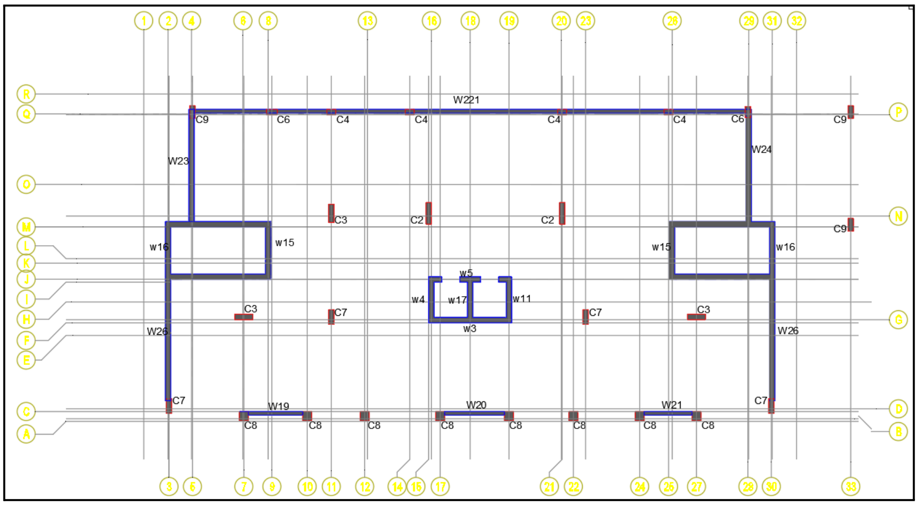


Figure 6.33: Wall naming layout at the ground floor level

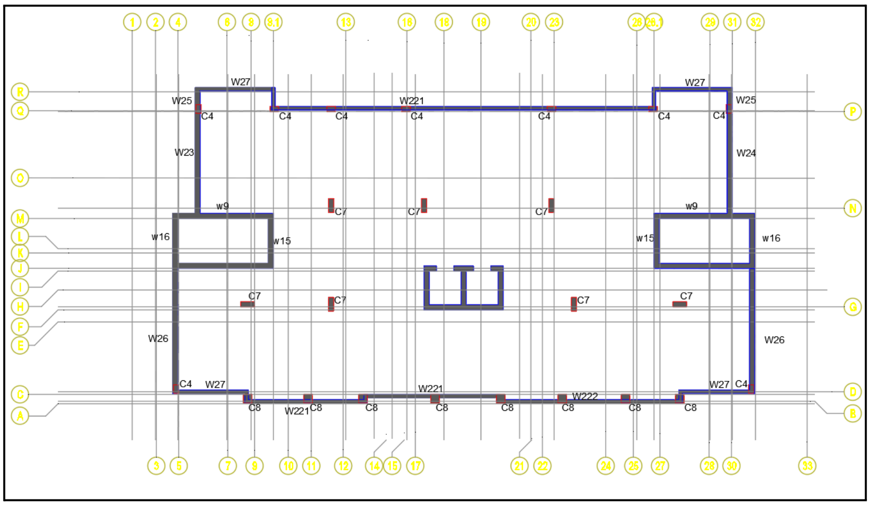


Figure 6.34: Wall naming layout at typical floors

Table 6.19: Walls piers reinforcement across structure

Grid	Walls	Thicknesses (mm)	Boundary	Vertical reinforcement	Horizontal reinforcement
3	W1	300	no need	Φ12/200mm	Φ14/300mm
4	W2	300	no need	Φ12/200mm	Φ14/300mm
	W3	300	no need	Φ12/200mm	Φ14/300mm
6	W5	300	need one side	Φ12/200mm	Φ14/300mm
7	W6	300	no need	Φ12/200mm	Φ14/300mm
8	W7	300	no need	Φ12/200mm	Φ14/300mm
9	W8	450	no need	Φ12/200mm	Φ20/250mm
10	W9	300	no need	Φ12/200mm	Φ14/300mm
12	W10	300	no need	Φ12/200mm	Φ14/300mm
J	W4	300	no need	Φ12/200mm	Φ14/300mm
L	W17	300	no need	Φ12/200mm	Φ14/300mm
M	W11	300	no need	Φ12/200mm	Φ14/300mm
	W12	300	no need	Φ12/200mm	Φ14/300mm
N	W12	300	no need	Φ12/200mm	Φ14/300mm
P	W13	300	no need	Φ12/200mm	Φ14/300mm
R	W14	400	no need	Φ12/200mm	Φ16/300mm
	W15	300	need	Φ12/200mm	Φ14/300mm
U	W16	300	no need	Φ12/200mm	Φ14/300mm
	W17	300	no need	Φ12/200mm	Φ14/300mm

Table 6.20: Walls piers reinforcement across structure

Grid	**Walls**	**Thicknesses (mm)**	**Boundary**	**Vertical reinforcement**	**Horizontal reinforcement**
	W19	200	no need	Φ12/300mm	Φ12/300mm
	W20	200	no need	Φ12/300mm	Φ12/300mm
1	W21	200	no need	Φ12/300mm	Φ12/300mm
4	W3	300	no need	Φ14/300mm	Φ14/300mm
6	W5	300	need one side	Φ12/300mm	Φ14/300mm
	W22.1L	200	no need	Φ16/300mm	Φ12/300mm
	W22.1M	200	no need	Φ20/300mm	Φ12/300mm
13	W22.1R	200	no need	Φ20/300mm	Φ12/300mm
	W16	300	no need	Φ12/200mm	Φ14/300mm
B	W26	300	need one side	Φ14/300mm	Φ20/300mm
C	W23	300	need	Φ14/300mm	Φ16/300mm
J	W4	300	no need	Φ12/300mm	Φ14/300mm
L	W17	300	no need	Φ12/300mm	Φ14/300mm
M	W11	300	no need	Φ12/300mm	Φ14/300mm
T	W23	300	need	Φ14/300mm	Φ16/300mm
	W16	300	no need	Φ12/200mm	Φ14/300mm
U	W26	300	need one side	Φ14/300mm	Φ20/300mm

Table 6.21: Walls piers reinforcement across structure

Gride	Walls	Thicknesses (mm)	Boundary	Vertical reinforcement	Horizontal reinforcement
1	W22.1L	200	no need	Φ16/300mm	Φ12/300mm
	W22.1M	200	no need	Φ20/300mm	Φ12/300mm
	W22.1R	200	no need	Φ20/300mm	Φ12/300mm
	W22.2L	200	no need	Φ16/300mm	Φ12/300mm
	W22.2R	200	no need	Φ20/300mm	Φ12/300mm
2	W27	200	no need	Φ12/300mm	Φ12/300mm
10	W9	300	no need	Φ12/300mm	Φ14/300mm
13	W22.1L	200	no need	Φ14/300mm	Φ12/300mm
	W22.1R	200	no need	Φ12/300mm	Φ12/300mm
14	W27L	200	no need	Φ12/300mm	Φ12/300mm
	W27R	200	no need	Φ12/300mm	Φ12/300mm
B	W16L	300	no need	Φ12/300mm	Φ14/300mm
	W16R	300	no need	Φ12/300mm	Φ14/300mm
	W26L	300	need	Φ14/300mm	Φ14/300mm
	W26R	300	no need	Φ14/300mm	Φ14/300mm
C	W25	300	no need	Φ14/300mm	Φ14/300mm
	W23.L	300	no need	Φ14/300mm	Φ14/300mm
	W23.R	300	no need	Φ14/300mm	Φ14/300mm
E	W15	300	need	Φ12/300mm	Φ14/300mm

Table 6.22: Boundary walls reinforcement

Walls	length walls(mm)	long .ren	boundary .length	b.boundary	#long bar	PU(kn)	cheak(equ)	spacing between hoops legs
w5L	805	708	300	300	Φ12/200mm	858.4	2028.6	300
w11	2250	1688	300	300	Φ12/200mm	2509.59	5670	350mm,300
w15L	1152	886	200	300	Φ12/200mm	1374.23	1935.36	350mm,300
w15R	577.4	403	100	300	Φ12/200mm	510.3	485.016	200
w23L	833	625	150	300	Φ14/300mm	941.6	1049.58	350mm,300
w23R	833	1757.87	100	300	Φ14/300mm	848.88	699.72	350mm,300
w24B	6250	4688	250	300	Φ14/300mm	1094	13125	200mm
w24L	1562	1238	250	300	Φ14/300mm	1788.45	3280.2	350mm,300
w24R	1562	1172	250	300	Φ14/300mm	1879.66	3280.2	350mm,300
w26M	2925	2194	350	300	Φ14/300mm	2849.68	8599.5	350mm,350
w26R	487	366	100	300	Φ14/300mm	521.29	409.08	200mm
w26L	1850	1462	200	300	Φ14/300mm	1757.87	3108	350mm,200

The spandrel reinforcement for all structural walls across the building is detailed in **Table 6.22.**

Table 6.23: Spandrel reinforcement

Wall	Stories	Horizontal bars	Vertical bars
W5	1-8	Φ14/300mm	Φ14/300mm
W15	1-9	Φ14/300mm	Φ14/300mm
W19.200	2-8	Φ12/300mm	Φ12/300mm
W20.200	2-8	Φ12/300mm	Φ12/300mm
W21.200	2-8	Φ12/300mm	Φ12/300mm
W221,200	2-8	Φ12/300mm	Φ12/300mm
W222.200	2-8	Φ12/300mm	Φ12/300mm
Wall	**Stories**	**Horizontal bars**	**Vertical bars**
W23	3-8	Φ14/300mm	Φ14/300mm
	8 Top	Φ14/300mm	Φ14/300mm
W24	2-7	Φ14/300mm	Φ14/300mm
W26	1-7	Φ14/300mm	Φ14/300mm
W27	3-8	Φ12/300mm	Φ12/300mm

6.7.3 Slabs design

The flexural reinforcement design of the slabs remained consistent with the initial design across most of the structure. However, in stories 7 and 8, specific areas exhibited increased internal forces requiring additional longitudinal reinforcement. These adjustments were made to enhance performance under seismic and gravity loading.

Figure 6.5 and **6.6** presented earlier illustrate the revised reinforcement zones in the affected slab regions.

6.7.4 Footings design

The reinforcement design for both the ground floor isolated footings and the basement mat foundation remains unchanged from the initial design presented in Chapter 3. **Figures 6.35-6.38** illustrate the reinforcement layout for the ground floor foundations. **Table 6.23** summarizes the required steel areas across all footing types, confirming that the reinforcement quantities are consistent with those previously calculated.

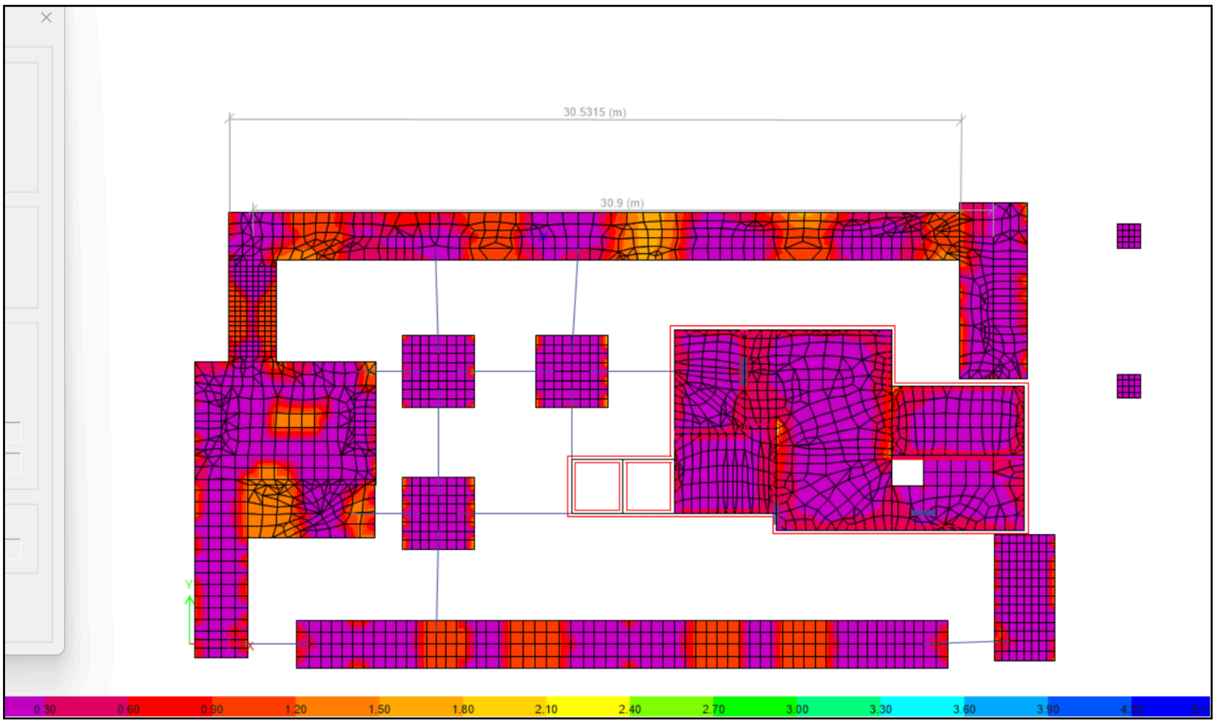


Figure 6.35: Top rebar of footings direction 1

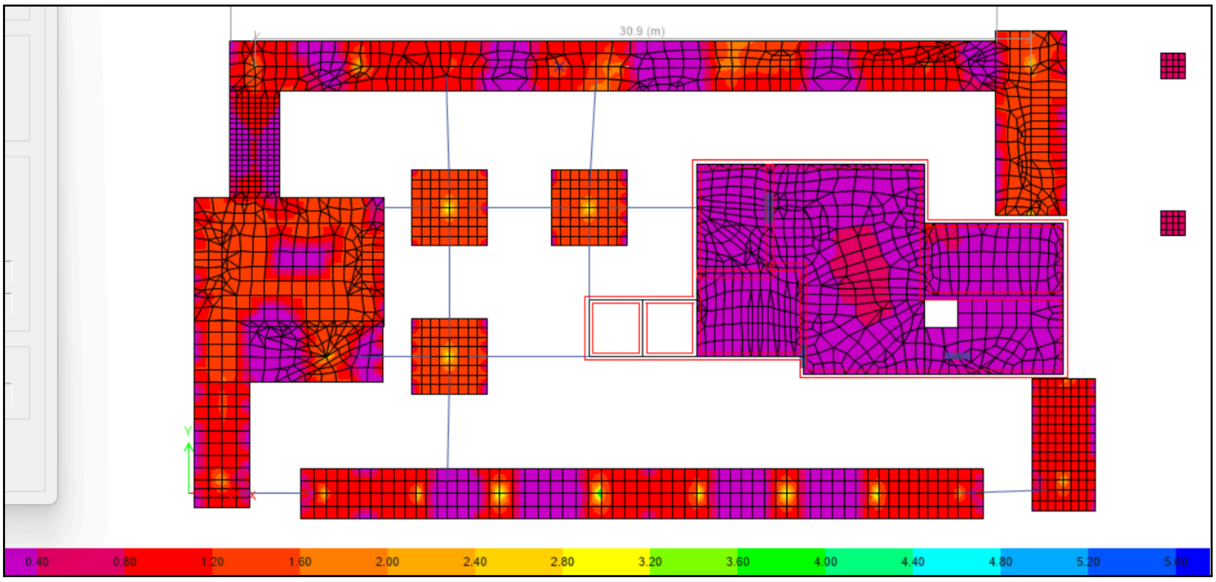


Figure 6.36: Bottom rebar of footings direction 1

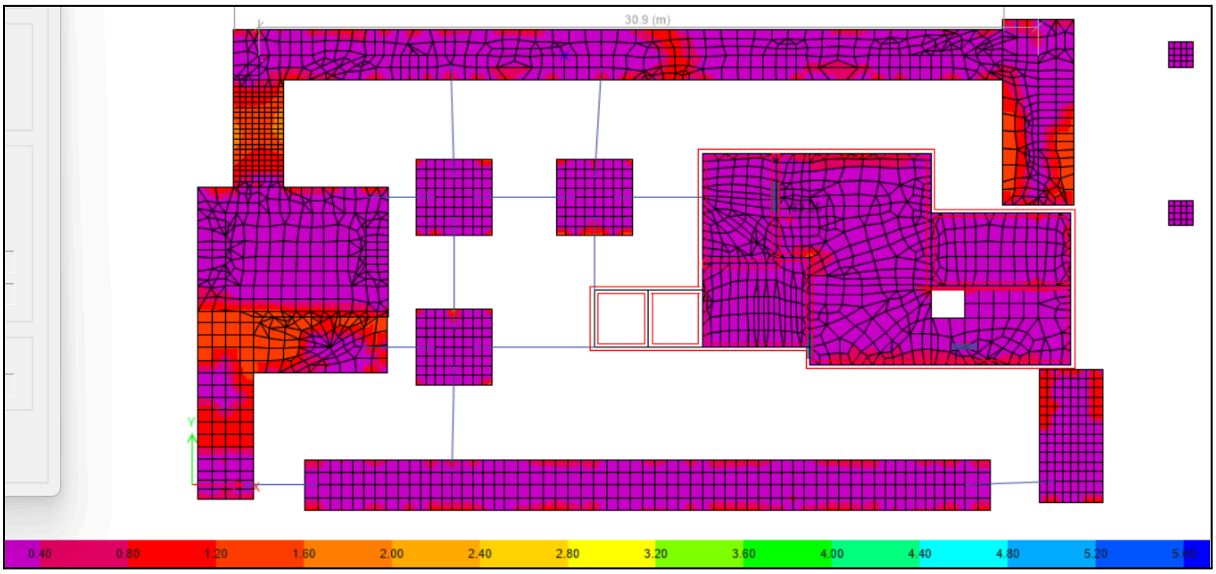


Figure 6.37: Top rebar of footings direction 2

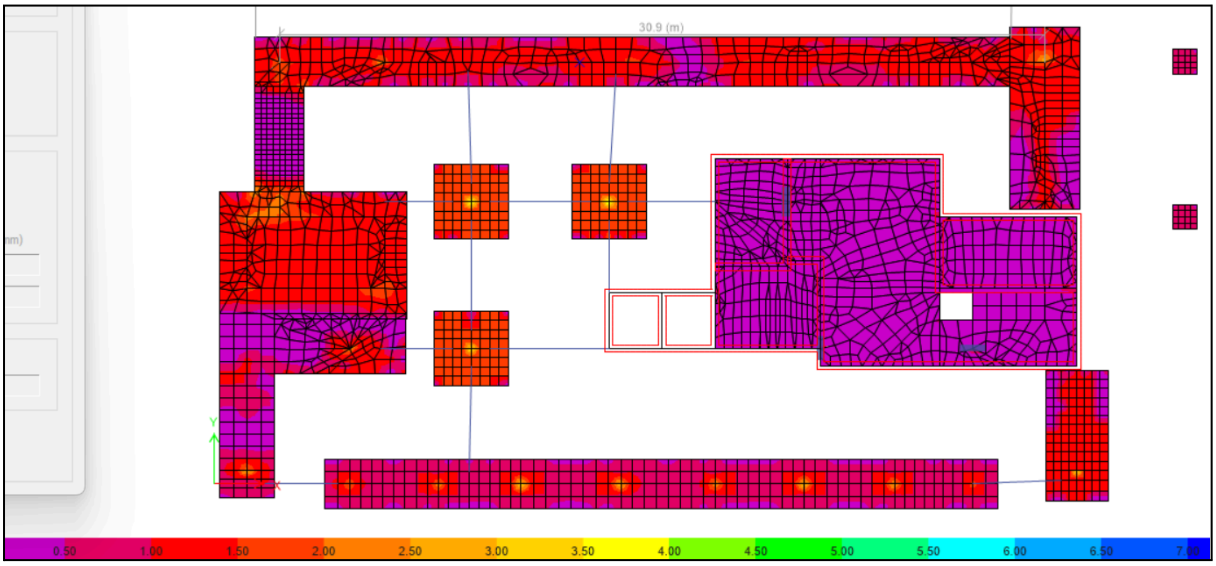


Figure 6.38: Bottom rebar of footings direction 2

Table 6.24: Footings area of steel in each direction

Footings	F1	Upper	Left 1	Stair case	Left 2	Down	Right 1	Right 2
Top rebar 1	1530	1600	990	1260	990	1200	1260	1260
Bottom rebar 1	1530	1080-1600	990	1260	1260	990	1260	1170
Top rebar 2	1530	1080	1550	1260	1260	990	1260	1170
Bottom rebar 2	1530	1080	990	1260	990	990	1260	1170

6.7.5 Tie beams design

The tie beam naming layout is illustrated in **Figure 6.39**, and the corresponding reinforcement details are provided in **Table 6.24**. The largest reinforcement values were adopted to ensure enhanced stability and strength for the ground beams.

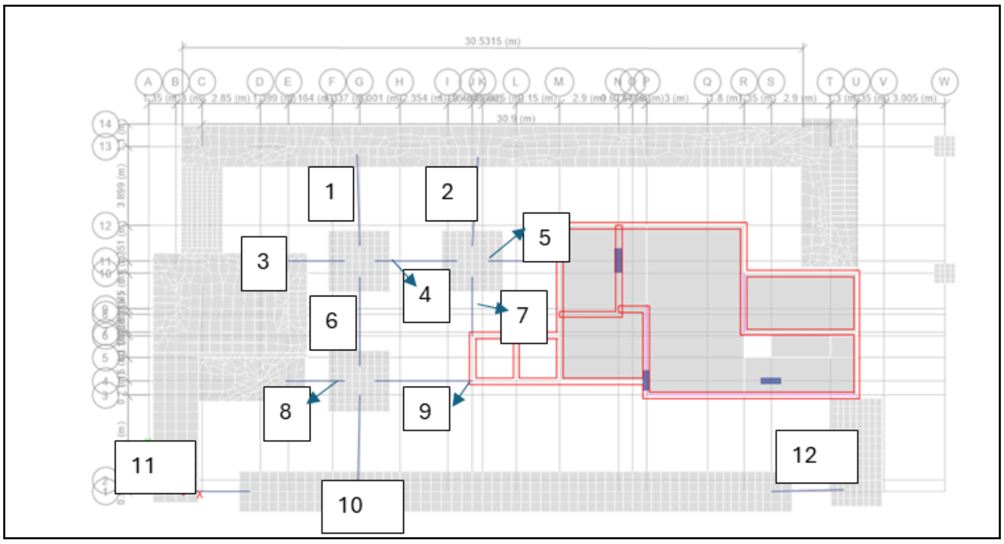


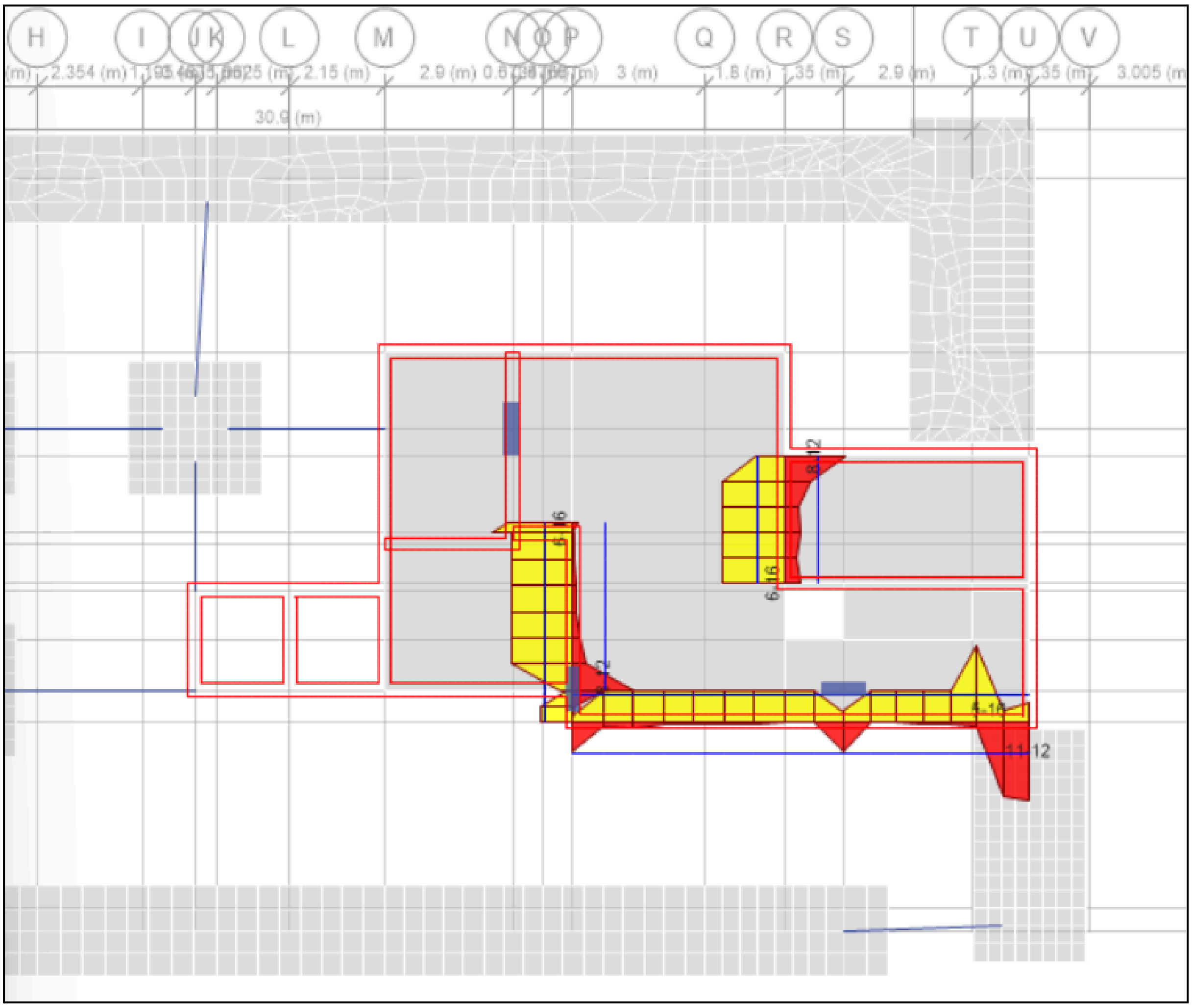
Figure 6.39: Ground beams naming layout

Table 6.25: Ground beams reinforcement

Tie Beam	Flexural Top	Flexural Bottom	Torsional	Stirrups	Spacing Flexural (mm)
1	6Φ14	6Φ14	2Φ16	2Φ10/(20)cm	40
2	6Φ14	6Φ14	2Φ16	2Φ10/(20)cm	40
3	6Φ14	6Φ14	2Φ16	2Φ10/(20)cm	40
4	6Φ14	6Φ14	2Φ16	2Φ10/(20)cm	40
5	6Φ14	6Φ14	2Φ16	2Φ10/(20)cm	40
6	6Φ14	6Φ14	2Φ16	2Φ10/(20)cm	40
7	6Φ14	6Φ14	2Φ16	2Φ10/(20)cm	40
8	6Φ14	6Φ14	2Φ16	2Φ10/(20)cm	40
9	6Φ14	6Φ14	2Φ16	2Φ10/(20)cm	40
10	6Φ14	6Φ14	2Φ16	2Φ10/(20)cm	40
11	6Φ14	6Φ14	2Φ16	2Φ10/(20)cm	40
12	6Φ14	6Φ14	2Φ16	2Φ10/(20)cm	40

Design of slabs:

reinforce the slabs in accordance with Chapter 3 **Table 3.15** except in specific areas that require additional reinforcement**.**



top 4Φ16

bottom2Φ12

zones around column :

top 1 and 2 direction , use top Φ18/200mm ,bottom Φ16/200mm

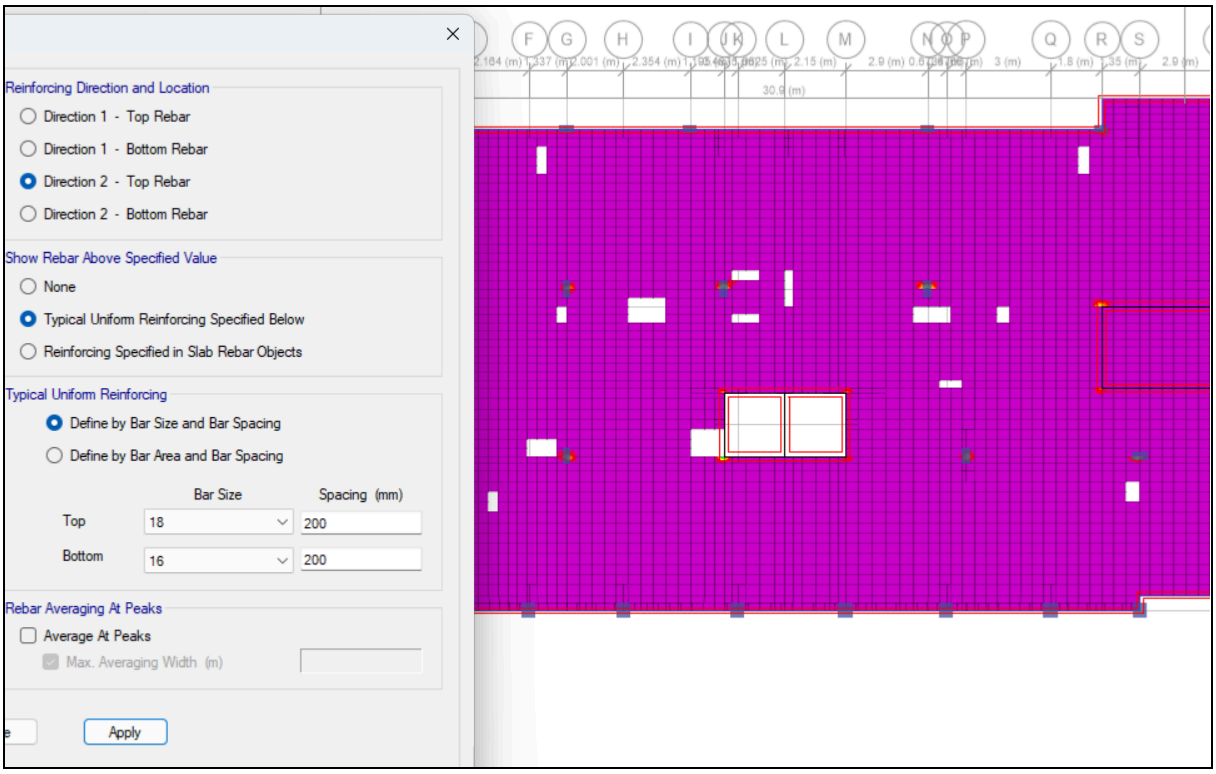

Reinforcing Direction and Location
Direction 1 - Top Rebar
Direction 1 - Bottom Rebar
Direction 2 - Top Rebar
Direction 2 - Bottom Rebar
Show Rebar Above Specified Value
None
Typical Uniform Reinforcing Specified Below
Reinforcing Specified in Slab Rebar Objects
Typical Uniform Reinforcing
Define by Bar Size and Bar Spacing
Define by Bar Area and Bar Spacing
Bar Size
Spacing (mm)
Top
18
200
Bottom
16
200
Rebar Averaging At Peaks
Average At Peaks
Max. Averaging Width (m)
Apply
F G H I J K L M N O P Q R S
30.9 (m)

Table 6.26:column design

Floors	Column	New dimensions	ρ	Area of steel (mm)^2
8	C13	500*300	1%	1500
	E13	300*500	1.4%	1500
	G13	300*500	1%	1500
	I13	300*500	1%	1500
	N13	300*500	1%	1500
	R13	300*500	1.4%	1500
	T13	500*300	1%	1500
	G11	300*300	1%	900
	J11	300*300	1%	900
	N11	300*300	1%	900
	D4	300*300	1%	900
	G4	300*300	1%	900
	P4	300*300	1%	900
	S4	300*300	1%	900
	B2	500*300	1%	1500
	V2	500*300	1%	1500
	D1	500*500	1%	2500
	F1	500*500	1%	2500
	H1	500*500	1%	2500
	K1	500*500	1%	2500
	M1	500*500	1%	2500

	O1	500*500	1%	2500
	Q1	500*500	1%	2500
	S1	500*500	1%	2500
	C13	500*300	1%	1500
	E13	300*500	1%	1800
	G13	300*500	1%	1800
	I13	300*500	1%	1800
	N13	300*500	1%	1800
	R13	300*500	1%	1800
	T13	500*300	1%	1800
	G11	300*600	1%	1800
	J11	300*600	1%	1800
	N11	300*600	1%	1800
7	D4	300*600	1%	1800
	G4	300*600	1%	1800
	P4	300*600	1%	1800
	S4	300*600	1%	1800
	B2	500*300	1%	1500
	V2	500*300	1%	1500
	D1	500*500	1%	2500
	F1	500*500	1%	2500
	H1	500*500	1%	2500
	K1	500*500	1%	2500
	M1	500*500	1%	2500

	O1	500*500	1%	2500
	Q1	500*500	1%	2500
	S1	500*500	1%	2500
6	C13	500*300	1%	1500
	E13	300*500	1%	1800
	G13	300*500	1%	1800
	I13	300*500	1%	1800
	N13	300*500	1%	1800
	R13	300*500	1%	1800
	T13	500*300	1%	1800
	G11	300*600	1%	1800
	J11	300*600	1%	1800
	N11	300*600	1%	1800
	D4	300*600	1%	1800
	G4	300*600	1%	1800
	P4	300*600	1%	1800
	S4	300*600	1%	1800
	B2	500*300	1%	1500
	V2	500*300	1%	1500
	D1	500*500	1%	2500
	F1	500*500	1%	2500
	H1	500*500	1%	2500
	K1	500*500	1%	2500
	M1	500*500	1%	2500

	O1	500*500	1%	2500
	Q1	500*500	1%	2500
	S1	500*500	1%	2500
5	C13	500*300	1%	1500
	E13	300*500	1%	1800
	G13	300*500	1%	1800
	I13	300*500	1%	1800
	N13	300*500	1%	1800
	R13	300*500	1%	1800
	T13	500*300	1%	1800
	G11	300*600	1%	1800
	J11	300*600	1%	1800
	N11	300*600	1%	1800
	D4	300*600	1%	1800
	G4	300*600	1%	1800
	P4	300*600	1%	1800
	S4	300*600	1%	1800
	B2	500*300	1%	1500
	V2	500*300	1%	1500
	D1	500*500	1%	2500
	F1	500*500	1%	2500
	H1	500*500	1%	2500
	K1	500*500	1%	2500
	M1	500*500	1%	2500

	O1	500*500	1%	2500
	Q1	500*500	1%	2500
	S1	500*500	1%	2500
4	C13	500*300	1%	1500
	E13	300*500	1%	1500
	G13	300*500	1%	1500
	I13	300*500	1%	1500
	N13	300*500	1%	1500
	R13	300*500	1%	1500
	T13	500*300	1%	1500
	G11	300*800	1%	2400
	J11	300*800	1%	2400
	N11	300*800	1%	2400
	D4	800*300	1%	2400
	G4	300*800	1%	2400
	P4	300*800	1%	2400
	S4	800*300	1%	2400
	B2	500*300	1%	1500
	V2	500*300	1%	1500
	D1	500*500	1%	2500
	F1	500*500	1%	2500
	H1	500*500	1%	2500
	K1	500*500	1%	2500
	M1	500*500	1%	2500

	O1	500*500	1%	2500
	Q1	500*500	1%	2500
	S1	500*500	1%	2500
3	C13	300*700	1%	2100
	E13	700*300	1%	2100
	G13	300*500	1%	1500
	I13	300*500	1%	1500
	N13	300*500	1%	1500
	R13	300*500	1%	1500
	T13	300*600	1%	1800
	W13	300*1000	1%	3000
	G11	1200*400	1%	4800
	J11	1200*300	1%	3600
	N11	1200*400	1%	4800
	W10	300*1000	1%	3000
	D4	1200*400	1%	4000
	G4	300*800	1%	2400
	P4	1000*400	1%	4000
	S4	300*1000	1%	3000
	B2	300*800	1%	2400
	V2	300*800	1%	2400
	D1	500*500	1%	2500
	F1	500*500	1%	2500
	H1	500*500	1%	2500

	K1	500*500	1%	2500
	M1	500*500	1%	2500
	O1	500*500	1%	2500
	Q1	500*500	1%	2500
	S1	500*500	1%	2500
BASEMENT	N11	1200*400	1%	4800
	P4	300*1000	1%	3000
	S4	300*1000	1%	3000

References

- *American Concrete Institute*, *"Building code requirements for structural concrete (ACI 318-19) and commentary"*, American Concrete Institute, Farmington Hills, MI, 2019.
- *American Concrete Institute*, *"Code requirements for environmental engineering concrete structures (ACI 350-20) and commentary"*, American Concrete Institute, Farmington Hills, MI, 2020.
- *American Society of Civil Engineers*, *"Minimum design loads for buildings and other structures (ASCE/SEI 7-16)"*, ASCE, Reston, VA, 2017.
- *Arman, I.*, *"Design of reinforced concrete building structures with computer applications (3rd ed.)"*, 2025a.
- *Arman, I.*, *"Design of reinforced concrete structures (6th ed.)"*, 2025b.
- *ASTM International*, *"Standards for materials and testing"*, ASTM, West Conshohocken, PA, n.d.
- *Bowles, J. E.*, *"Foundation analysis and design (5th ed.)"*, McGraw-Hill, 1996.
- *Darwin, D., Dolan, C. W., Nilson, A. H.*, *"Design of concrete structures (15th ed.)"*, McGraw-Hill Education, 2016.
- *Hibbeler, R. C.*, *"Structural Analysis (10th ed.)"*, Pearson Education, 2015.
- *International Code Council*, *"International Building Code (IBC)"*, International Code Council, Country Club Hills, IL, 2021.
- *Jordanian Standards and Metrology Organization*, *"Jordanian code for loads and forces"*, JSMO, Amman, Jordan, 2006.

APPENDIX

A Architectural Drawings

The architectural drawings for the project are provided in the accompanying AutoCAD file. The following is a summary of the architectural drawings:

B Structural Drawings

The structural drawings for the project are provided in the accompanying AutoCAD file. The following is a summary of the structural drawings:

C ETABS Design Results for Combined Gravity and Seismic Forces

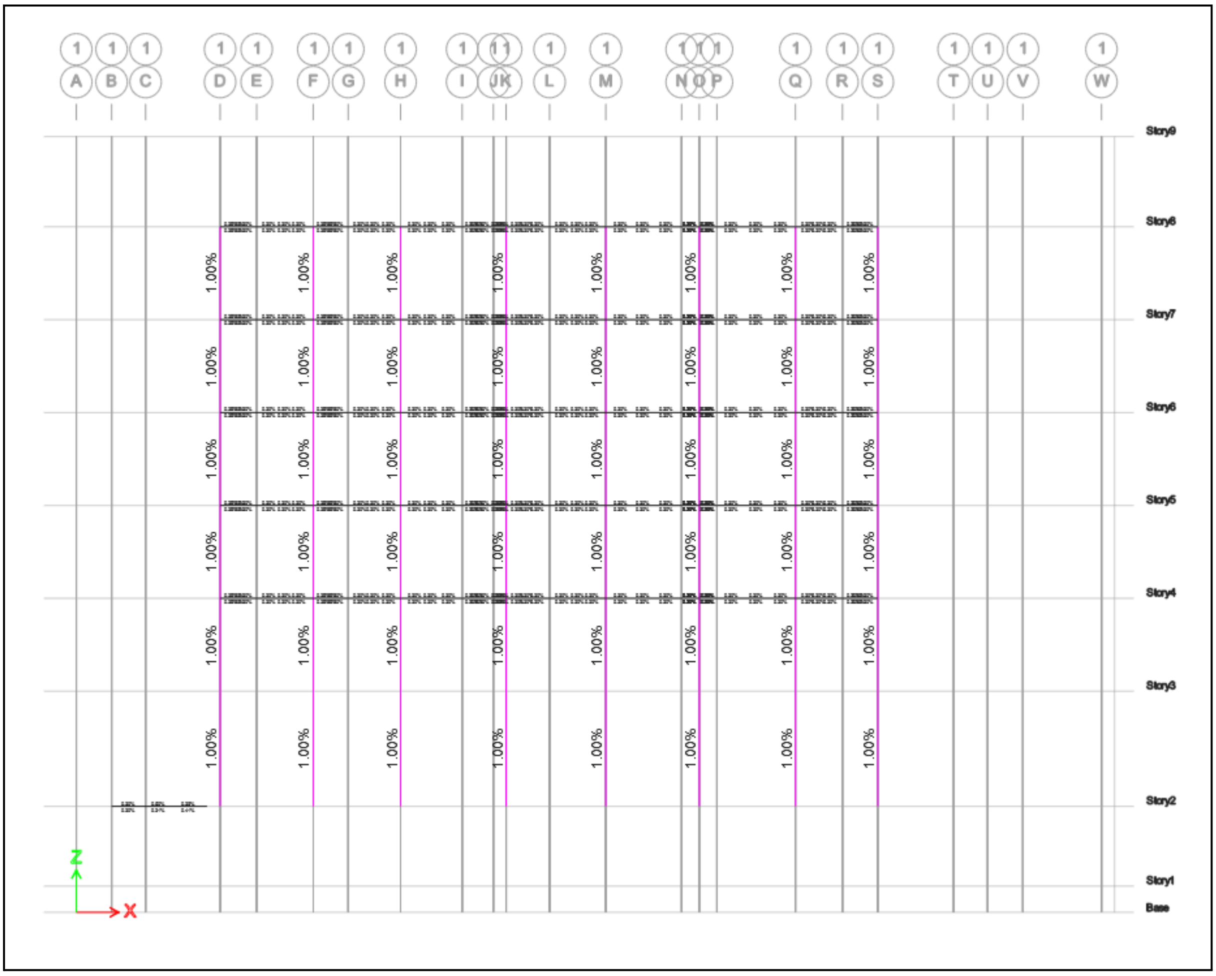


Figure D.1: Design output of columns along grid-line 1

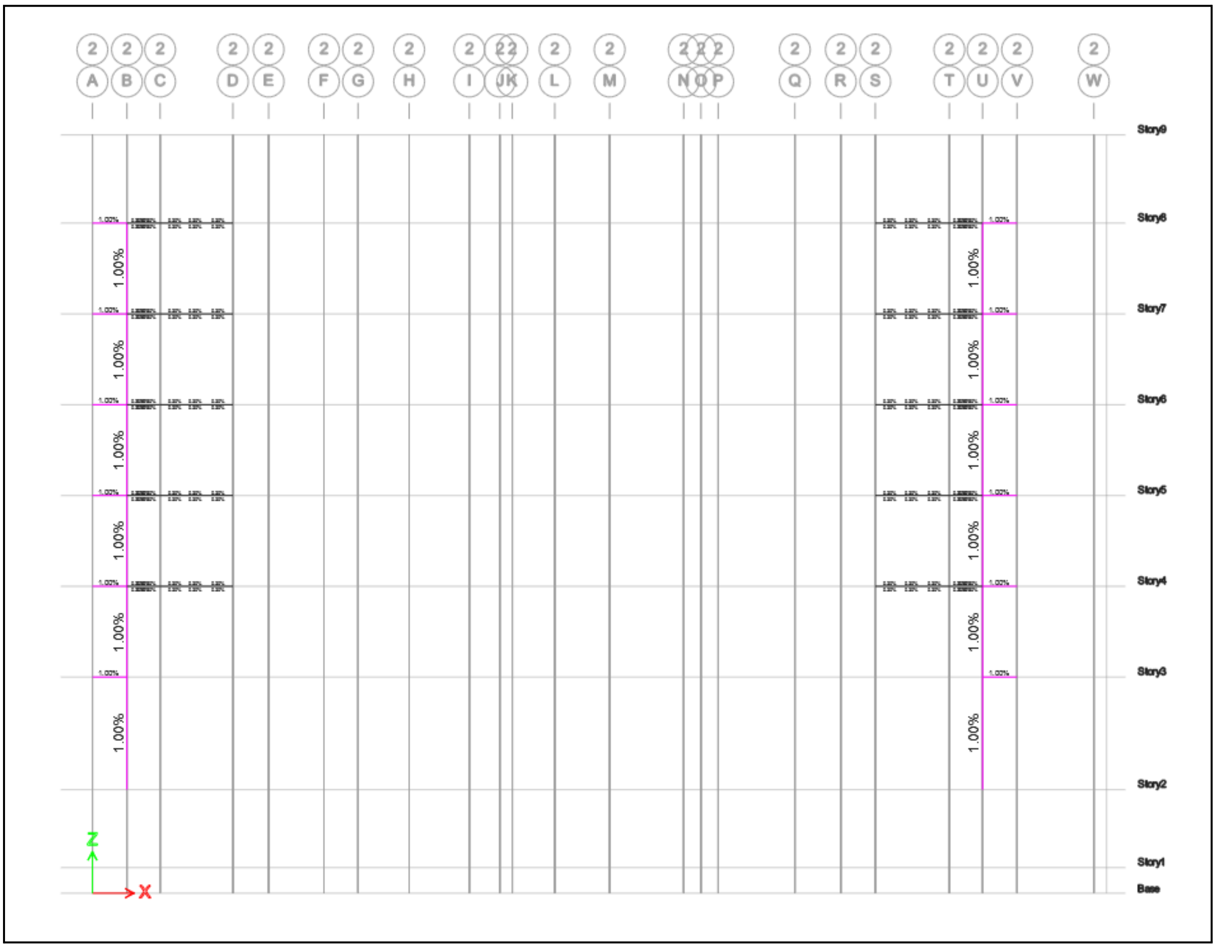


Figure D.2: Design output of columns along grid-line 2

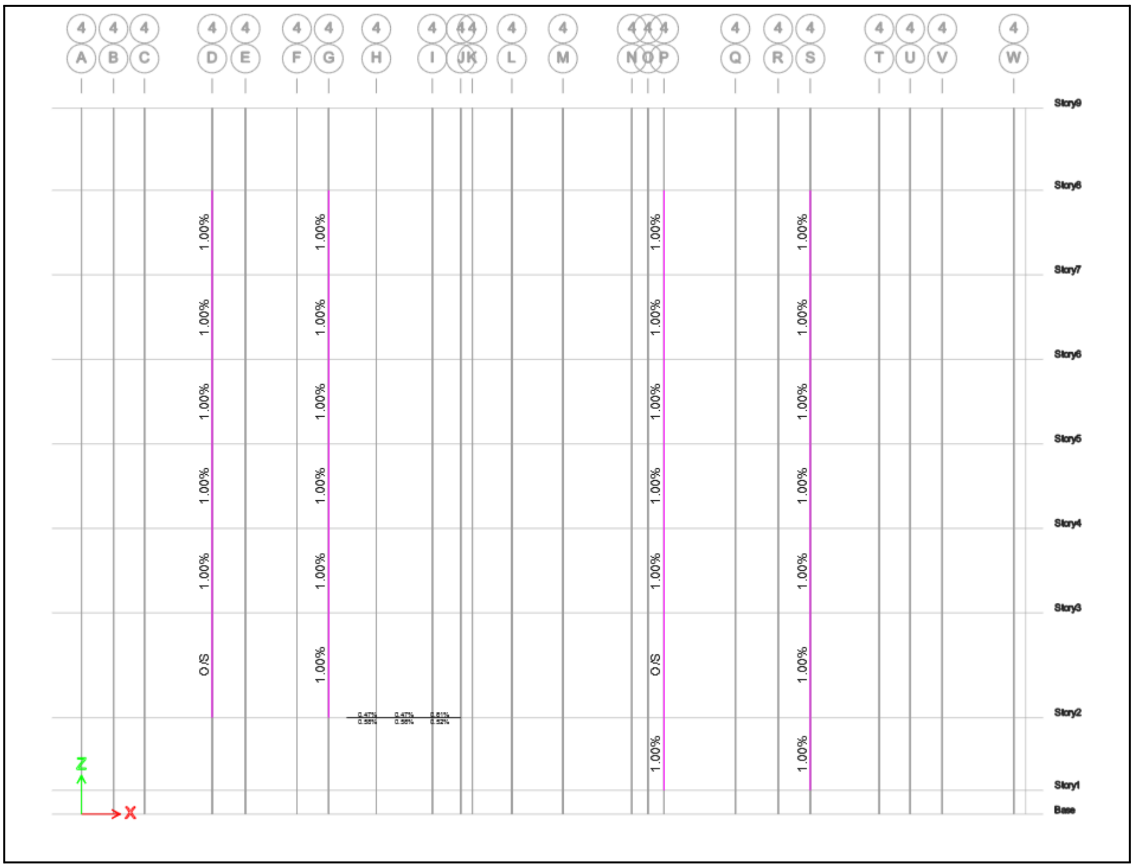


Figure D.3: Design output of columns along grid-line 4

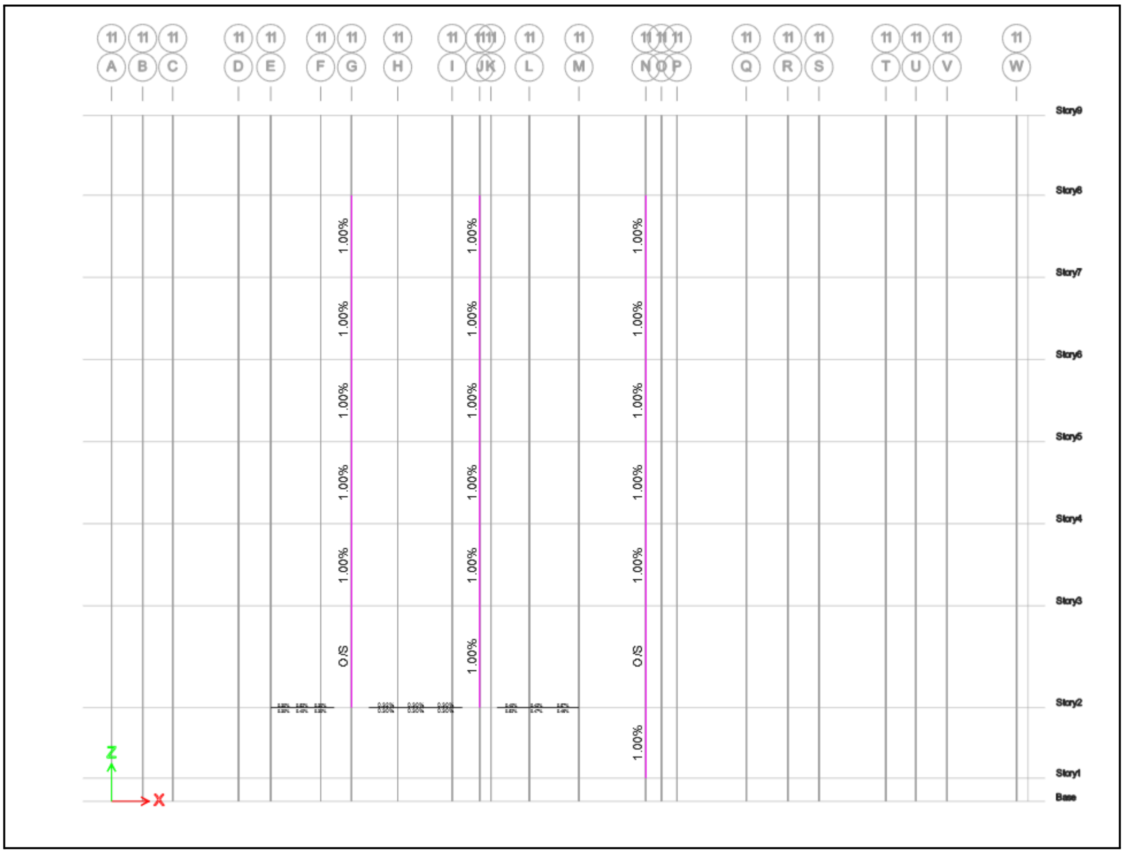


Figure D.4: Design output of columns along grid-line 11

Figure D.5: Design output of columns along grid-line 13

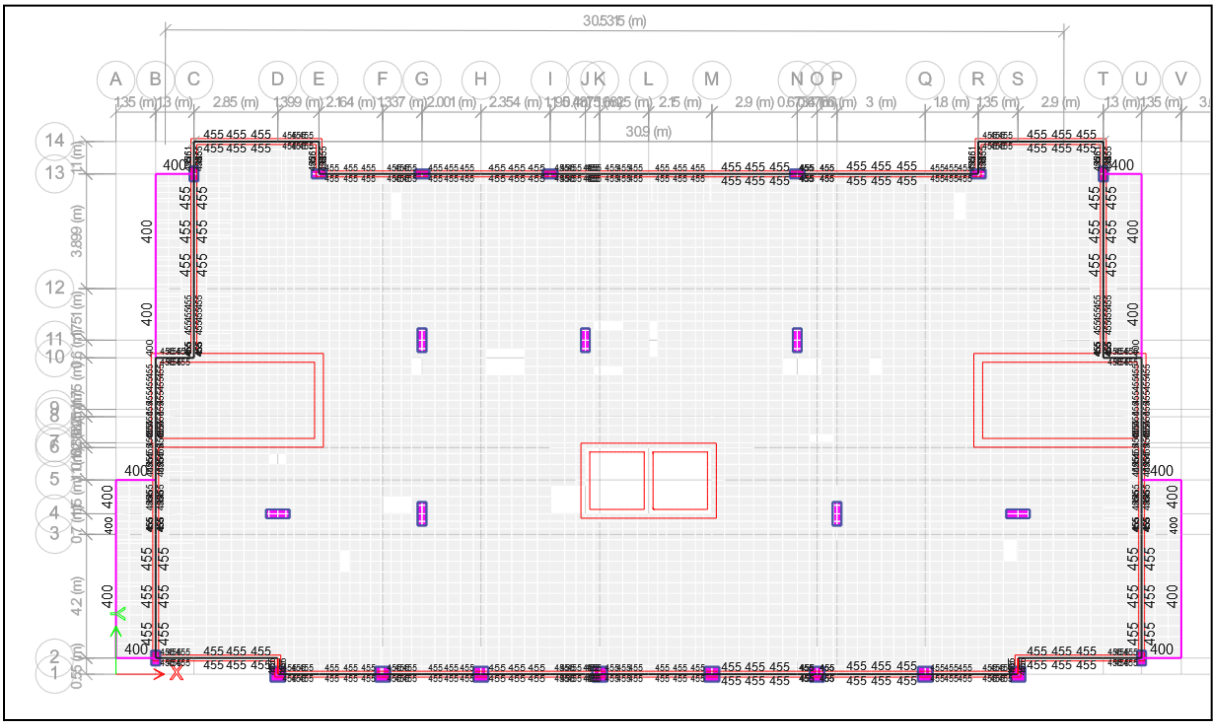
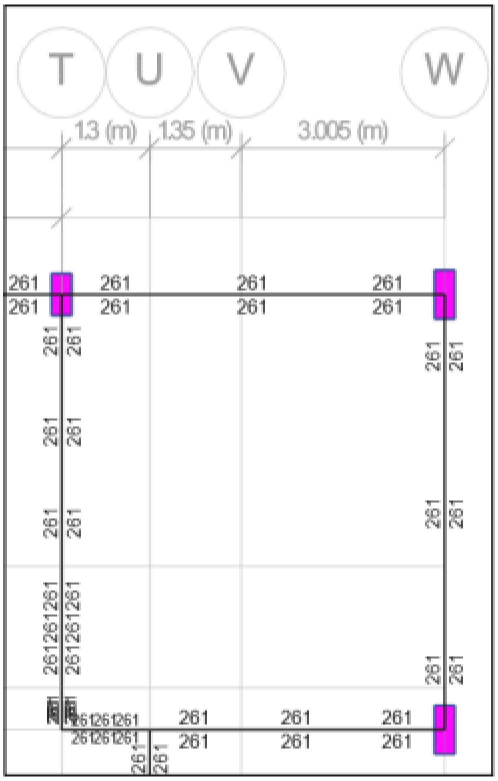
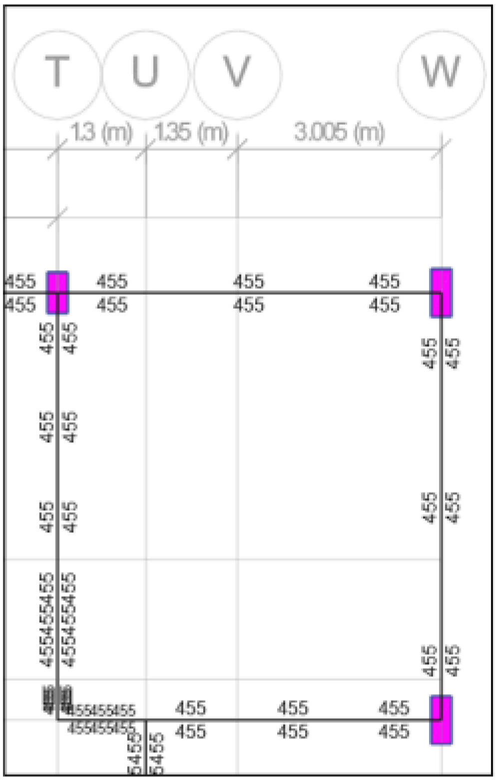


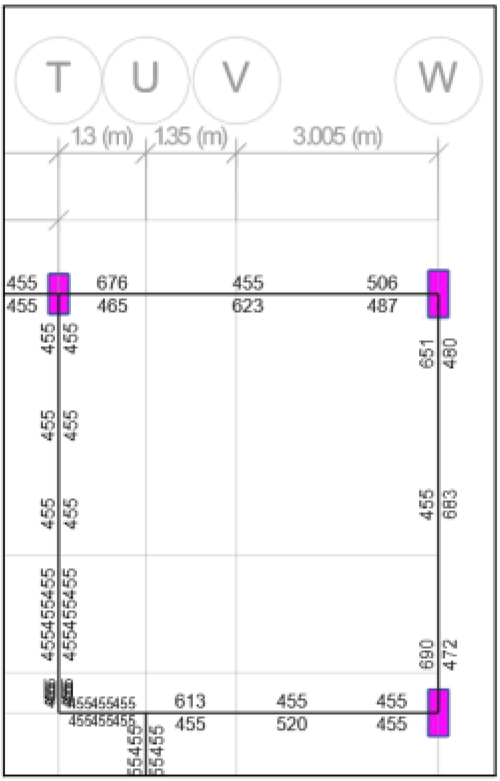
Figure D.6: Longitudinal Reinforcement in wall-supported beams



Torsional Longitudinal Reinforcement



Flexural Longitudinal Reinforcement



Total Longitudinal Reinforcement

Figure D.7: Longitudinal Reinforcement in column-supported beams

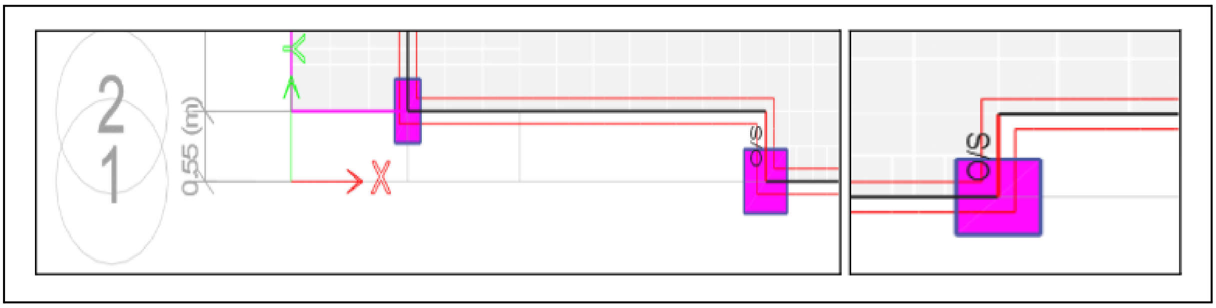


Figure D.8: Overstressing in very short wall-supported beams

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, Ω_0[b]	Deflection Amplification Factor, C_d[c]	Structural System Limitations Including Structural Height, h_n (ft) Limits[d] — Seismic Design Category: B	C	D[e]	E[e]	F[f]
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls[g,h]	14.2	5	2½	5	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls[g]	14.2	4	2½	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls[g]	14.2	2	2½	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls[g]	14.2	1½	2½	1½	NL	NP	NP	NP	NP
5. Intermediate precast shear walls[g]	14.2	4	2½	4	NL	NL	40[i]	40[i]	40[i]
6. Ordinary precast shear walls[g]	14.2	3	2½	3	NL	NP	NP	NP	NP
7. Special reinforced masonry shear walls	14.4	5	2½	3½	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	14.4	3½	2½	2¼	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	14.4	2	2½	1¾	NL	160	NP	NP	NP
10. Detailed plain masonry shear walls	14.4	2	2½	1¾	NL	NP	NP	NP	NP
11. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
12. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
13. Ordinary reinforced AAC masonry shear walls	14.4	2	2½	2	NL	35	NP	NP	NP
14. Ordinary plain AAC masonry shear walls	14.4	1½	2½	1½	NL	NP	NP	NP	NP
15. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	6½	3	4	NL	NL	65	65	65
16. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	6½	3	4	NL	NL	65	65	65
17. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2	2½	2	NL	NL	35	NP	NP
18. Light-frame (cold-formed steel) wall systems using flat strap bracing	14.1	4	2	3½	NL	NL	65	65	65
B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames	14.1	8	2	4	NL	NL	160	160	100
2. Steel special concentrically braced frames	14.1	6	2	5	NL	NL	160	160	100
3. Steel ordinary concentrically braced frames	14.1	3¼	2	3¼	NL	NL	35[j]	35[j]	NP[j]
4. Special reinforced concrete shear walls[g,h]	14.2	6	2½	5	NL	NL	160	160	100
5. Ordinary reinforced concrete shear walls[g]	14.2	5	2½	4½	NL	NL	NP	NP	NP
6. Detailed plain concrete shear walls[g]	14.2 and 14.2.2.7	2	2½	2	NL	NP	NP	NP	NP
7. Ordinary plain concrete shear walls[g]	14.2	1½	2½	1½	NL	NP	NP	NP	NP
8. Intermediate precast shear walls[g]	14.2	5	2½	4½	NL	NL	40[i]	40[i]	40[i]
9. Ordinary precast shear walls[g]	14.2	4	2½	4	NL	NP	NP	NP	NP
10. Steel and concrete composite eccentrically braced frames	14.3	8	2½	4	NL	NL	160	160	100
11. Steel and concrete composite special concentrically braced frames	14.3	5	2	4½	NL	NL	160	160	100
12. Steel and concrete composite ordinary braced frames	14.3	3	2	3	NL	NL	NP	NP	NP
13. Steel and concrete composite plate shear walls	14.3	6½	2½	5½	NL	NL	160	160	100
14. Steel and concrete composite special shear walls	14.3	6	2½	5	NL	NL	160	160	100
15. Steel and concrete composite ordinary shear walls	14.3	5	2½	4½	NL	NL	NP	NP	NP
16. Special reinforced masonry shear walls	14.4	5½	2½	4	NL	NL	160	160	100
17. Intermediate reinforced masonry shear walls	14.4	4	2½	4	NL	NL	NP	NP	NP

Figure D.1: ASCE 7-16 Table 12.2-1 (Part 1 of 3) – Permitted Seismic Force-Resisting Systems

18. Ordinary reinforced masonry shear walls	14.4	2	2½	2	NL	160	NP	NP	NP
19. Detailed plain masonry shear walls	14.4	2	2½	2	NL	NP	NP	NP	NP
20. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
21. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
22. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	7	2½	4½	NL	NL	65	65	65
23. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	7	2½	4½	NL	NL	65	65	65
24. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2½	2½	2½	NL	NL	35	NP	NP
25. Steel buckling-restrained braced frames	14.1	8	2½	5	NL	NL	160	160	100
26. Steel special plate shear walls	14.1	7	2	6	NL	NL	160	160	100
C. MOMENT-RESISTING FRAME SYSTEMS									
1. Steel special moment frames	14.1 and 12.2.5.5	8	3	5½	NL	NL	NL	NL	NL
2. Steel special truss moment frames	14.1	7	3	5½	NL	NL	160	100	NP
3. Steel intermediate moment frames	12.2.5.7 and 14.1	4½	3	4	NL	NL	35[k]	NP[k]	NP[k]
4. Steel ordinary moment frames	12.2.5.6 and 14.1	3½	3	3	NL	NL	NP[l]	NP[l]	NP[l]
5. Special reinforced concrete moment frames[m]	12.2.5.5 and 14.2	8	3	5½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2½	NL	NP	NP	NP	NP
8. Steel and concrete composite special moment frames	12.2.5.5 and 14.3	8	3	5½	NL	NL	NL	NL	NL
9. Steel and concrete composite intermediate moment frames	14.3	5	3	4½	NL	NL	NP	NP	NP
10. Steel and concrete composite partially restrained moment frames	14.3	6	3	5½	160	160	100	NP	NP
11. Steel and concrete composite ordinary moment frames	14.3	3	3	2½	NL	NP	NP	NP	NP
12. Cold-formed steel—special bolted moment frame[n]	14.1	3½	3[o]	3½	35	35	35	35	35
D. DUAL SYSTEMS WITH SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES	12.2.5.1								
1. Steel eccentrically braced frames	14.1	8	2½	4	NL	NL	NL	NL	NL
2. Steel special concentrically braced frames	14.1	7	2½	5½	NL	NL	NL	NL	NL
3. Special reinforced concrete shear walls[g,h]	14.2	7	2½	5½	NL	NL	NL	NL	NL
4. Ordinary reinforced concrete shear walls[g]	14.2	6	2½	5	NL	NL	NP	NP	NP
5. Steel and concrete composite eccentrically braced frames	14.3	8	2½	4	NL	NL	NL	NL	NL
6. Steel and concrete composite special concentrically braced frames	14.3	6	2½	5	NL	NL	NL	NL	NL
7. Steel and concrete composite plate shear walls	14.3	7½	2½	6	NL	NL	NL	NL	NL
8. Steel and concrete composite special shear walls	14.3	7	2½	6	NL	NL	NL	NL	NL
9. Steel and concrete composite ordinary shear walls	14.3	6	2½	5	NL	NL	NP	NP	NP
10. Special reinforced masonry shear walls	14.4	5½	3	5	NL	NL	NL	NL	NL
11. Intermediate reinforced masonry shear walls	14.4	4	3	3½	NL	NL	NP	NP	NP
12. Steel buckling-restrained braced frames	14.1	8	2½	5	NL	NL	NL	NL	NL
13. Steel special plate shear walls	14.1	8	2½	6½	NL	NL	NL	NL	NL
E. DUAL SYSTEMS WITH INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES	12.2.5.1								
1. Steel special concentrically braced frames[p]	14.1	6	2½	5	NL	NL	35	NP	NP
2. Special reinforced concrete shear walls[g,h]	14.2	6½	2½	5	NL	NL	160	100	100
3. Ordinary reinforced masonry shear walls	14.4	3	3	2½	NL	160	NP	NP	NP
4. Intermediate reinforced masonry shear walls	14.4	3½	3	3	NL	NL	NP	NP	NP

continues

Figure D.2: ASCE 7-16 Table 12.2-1 (Part 2 of 3) – Permitted Seismic Force-Resisting Systems

Table 12.2-1 *(Continued)* 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, Ω_0[b]	Deflection Amplification Factor, C_d[c]	Structural System Limitations Including Structural Height, h_n (ft) Limits[d] — Seismic Design Category: B	C	D[e]	E[e]	F[f]
5. Steel and concrete composite special concentrically braced frames	14.3	5½	2½	4½	NL	NL	160	100	NP
6. Steel and concrete composite ordinary braced frames	14.3	3½	2½	3	NL	NL	NP	NP	NP
7. Steel and concrete composite ordinary shear walls	14.3	5	3	4½	NL	NL	NP	NP	NP
8. Ordinary reinforced concrete shear walls[g]	14.2	5½	2½	4½	NL	NL	NP	NP	NP
F. SHEAR WALL-FRAME INTERACTIVE SYSTEM WITH ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS[g]	12.2.5.8 and 14.2	4½	2½	4	NL	NP	NP	NP	NP
G. CANTILEVERED COLUMN SYSTEMS DETAILED TO CONFORM TO THE REQUIREMENTS FOR:	12.2.5.2								
1. Steel special cantilever column systems	14.1	2½	1¼	2½	35	35	35	35	35
2. Steel ordinary cantilever column systems	14.1	1¼	1¼	1¼	35	35	NP[l]	NP[l]	NP[l]
3. Special reinforced concrete moment frames[m]	12.2.5.5 and 14.2	2½	1¼	2½	35	35	35	35	35
4. Intermediate reinforced concrete moment frames	14.2	1½	1¼	1½	35	35	NP	NP	NP
5. Ordinary reinforced concrete moment frames	14.2	1	1¼	1	35	NP	NP	NP	NP
6. Timber frames	14.5	1½	1½	1½	35	35	35	NP	NP
H. STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE, EXCLUDING CANTILEVER COLUMN SYSTEMS	14.1	3	3	3	NL	NL	NP	NP	NP

[a]Response modification coefficient, R, for use throughout the standard. Note that R reduces forces to a strength level, not an allowable stress level.
[b]Where the tabulated value of the overstrength factor, Ω_0, is greater than or equal to 2½, Ω_0 is permitted to be reduced by subtracting the value of 1/2 for structures with flexible diaphragms.
[c]Deflection amplification factor, C_d, for use in Sections 12.8.6, 12.8.7, and 12.9.1.2.
[d]NL = Not Limited, and NP = Not Permitted. For metric units, use 30.5 m for 100 ft and use 48.8 m for 160 ft.
[e]See Section 12.2.5.4 for a description of seismic force-resisting systems limited to buildings with a structural height, h_n, of 240 ft (73.2 m) or less.
[f]See Section 12.2.5.4 for seismic force-resisting systems limited to buildings with a structural height, h_n, of 160 ft (48.8 m) or less.
[g]In Section 2.3 of ACI 318. A shear wall is defined as a structural wall.
[h]In Section 2.3 of ACI 318. The definition of "special structural wall" includes precast and cast-in-place construction.
[i]An increase in structural height, h_n, to 45 ft (13.7 m) is permitted for single-story storage warehouse facilities.
[j]Steel ordinary concentrically braced frames are permitted in single-story buildings up to a structural height, h_n, of 60 ft (18.3 m) where the dead load of the roof does not exceed 20 lb/ft^2 (0.96 kN/m^2) and in penthouse structures.
[k]See Section 12.2.5.7 for limitations in structures assigned to Seismic Design Categories D, E, or F.
[l]See Section 12.2.5.6 for limitations in structures assigned to Seismic Design Categories D, E, or F.
[m]In Section 2.3 of ACI 318. The definition of "special moment frame" includes precast and cast-in-place construction.
[n]Cold-formed steel—special bolted moment frames shall be limited to one story in height in accordance with ANSI/AISI S400.
[o]Alternately, the seismic load effect including overstrength, E_{mh}, is permitted to be based on the expected strength determined in accordance with ANSI/AISI S400.
[p]Ordinary moment frame is permitted to be used in lieu of intermediate moment frame for Seismic Design Categories B or C.

Figure D.3: ASCE 7-16 Table 12.2-1 (Part 3 of 3) – Permitted Seismic Force-Resisting Systems

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

Figure D.4: ASCE 7-16 Table 12.8-1 – Coefficient for Upper Limit on Calculated Period

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.

Figure D.5: ASCE 7-16 Table 12.8-2 – Values of Approximate Period Parameters C_t and x

TABLE 4-2

Bearing-capacity factors for the Terzaghi equations

Values of N_γ for ϕ of 0, 34, and **48**° are original Terzaghi values and used to back-compute $K_{p\gamma}$

ϕ, deg	N_c	N_q	N_γ	$K_{p\gamma}$
0	5.7*	1.0	0.0	10.8
5	7.3	1.6	0.5	12.2
10	9.6	2.7	1.2	14.7
15	12.9	4.4	2.5	18.6
20	17.7	7.4	5.0	25.0
25	25.1	12.7	9.7	35.0
30	37.2	22.5	19.7	52.0
34	52.6	36.5	36.0	
35	57.8	41.4	42.4	82.0
40	95.7	81.3	100.4	141.0
45	172.3	173.3	297.5	298.0
48	258.3	287.9	780.1	
50	347.5	415.1	1153.2	800.0

*$N_c = 1.5\pi + 1$. [See Terzaghi (1943), p. 127.]

Figure E.1: Bearing-capacity factors for the Terzaghi equations (Table 4-2, Bowles 1996)

TABLE 1604.5
RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES

RISK CATEGORY	NATURE OF OCCUPANCY
I	Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to: • Agricultural facilities. • Certain temporary facilities. • Minor storage facilities.
II	Buildings and other structures except those listed in Risk Categories I, III and IV.

III	Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to:
	• Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300.
	• Buildings and other structures containing one or more public assembly spaces, each having an occupant load greater than 300 and a cumulative occupant load of the public assembly spaces of greater than 2,500.
	• Buildings and other structures containing Group E or Group I-4 occupancies or combination therof, with an occupant load greater than 250.
	• Buildings and other structures containing educational occupancies for students above the 12th grade with an occupant load greater than 500.
	• Group I-2, Condition 1 occupancies with 50 or more care recipients.
	• Group I-2, Condition 2 occupancies not having emergency surgery or emergency treatment facilities.
	• Group I-3 occupancies.
	• Any other occupancy with an occupant load greater than 5,000.[a]
	• Power-generating stations, water treatment facilities for potable water, wastewater treatment facilities and other public utility facilities not included in Risk Category IV.
	• Buildings and other structures not included in Risk Category IV containing quantities of toxic or explosive materials that:
	  • Exceed maximum allowable quantities per control area as given in Table 307.1(1) or 307.1(2) or per outdoor control area in accordance with the *International Fire Code*; and
	  • Are sufficient to pose a threat to the public if released.[b]

IV	Buildings and other structures designated as essential facilities, including but not limited to:
	• Group I-2, Condition 2 occupancies having emergency surgery or emergency treatment facilities.
	• Ambulatory care facilities having emergency surgery or emergency treatment facilities.
	• Fire, rescue, ambulance and police stations and emergency vehicle garages
	• Designated earthquake, hurricane or other emergency shelters.
	• Designated emergency preparedness, communications and operations centers and other facilities required for emergency response.
	• Power-generating stations and other public utility facilities required as emergency backup facilities for Risk Category IV structures.
	• Buildings and other structures containing quantities of highly toxic materials that:
	  • Exceed maximum allowable quantities per control area as given in Table 307.1(2) or per outdoor control area in accordance with the *International Fire Code*; and
	  • Are sufficient to pose a threat to the public if released.[b]
	• Aviation control towers, air traffic control centers and emergency aircraft hangars.
	• Buildings and other structures having critical national defense functions.
	• Water storage facilities and pump structures required to maintain water pressure for fire suppression.

a. For purposes of occupant load calculation, occupancies required by Table 1004.5 to use gross floor area calculations shall be permitted to use net floor areas to determine the total occupant load.

b. Where approved by the building official, the classification of buildings and other structures as Risk Category III or IV based on their quantities of toxic, highly toxic or explosive materials is permitted to be reduced to Risk Category II, provided that it can be demonstrated by a hazard assessment in accordance with Section 1.5.3 of ASCE 7 that a release of the toxic, highly toxic or explosive materials is not sufficient to pose a threat to the public.

Figure E.2: Risk Category Classification Table (IBC 2021 Table 1604.5)