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Faculty of Engineering and Information Technology

جامعة النجاح الوطنية

كلية الهندسة و تكنولوجيا المعلومات

Graduation Project Report II

Structural Analysis and Design of Commercial Complex in Haifa City-Palestine

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DEDICATION (الإهداء)

الحمد لله سبحانه واسع الفضل كريم العطايا، الحمدلله الذي لا يشكر ولا يحمد على فضل سواه، الحمدلله الذي أنعم علينا وما زال فله الحمد من قبل ومن بعد حمداً كثيراً طيباً مباركاً فيه.

إلى من ربياني صغيرا...

إلى من أضائوا لي منارة العلم، إلى من بذل الغالي و النفيس من أجلي ،،، أبي

إلى من سهرت الليالي من أجل راحتي، إلى القلب الدافئ الحنون ،،، أمي

إلى كل من علمني حرفاً و كان له فضل إتمام مسيرتي إلى النجاح

إلى كل من منحني من ينابيع العلم و المعرفة ،،، أساتذتي الكرام

إلى من تعب و بذل قصارى جهده وكان بمثابة الاخ و المعلم و العين الساهرة

إلى من كان لنا نموذجاً يحتذى به (المهندس إبراهيم عرمان)

إلى كل من ساندني وكان بمثابة الاخ و السند

إلى كل من منحني الطاقة ووقف بجانبي دوماً ،،، أصدقائي و اخوتي

إلى من منحوني الثقة، وكان لهم الأثر الطيب ،،،، عائلتي الكريمة

ACKNOWLEDGEMENT (شكر وعرفان)

بسم الله الرحمن الرحمن الرحيم

الحمد لله رب العالمين الذي وفقنا وأعاننا على إنهاء مشروعنا والخروج به بهذه الصورة المميزة، فبالأمس القريب بدأنا مسيرتنا التعليمية ونحن نتحسس الطريق برهبة وارتباك، فرأينا أن الهندسة المدنية هدفًا ساميًا وحبًا وغاية تستحق السير لأجلها، وإن مشروعنا يحمل في طياته طموح شباب يحلمون أن تكون أمتهم العربية كالشامة بين الأمم.

المعلمين والدكاترة المشرفين على المشروع عادة ما يكون لهم فضل كبير علينا في إنجاز مشروع التخرج من خلال توجيهاتهم لنا وتقديم الدعم الكبير، ويتوجب علينا أن نتوجه لهم بالشكر والثناء وإن كان لا يفيهم حقهم إلا أنها طريقة جيدة لحفظ الجميل، ونسرد لكم فيما يأتي عبارات شكر وتقدير للمهندس الاستاذ إبراهيم عرمان.

نود أن نعرب عن تقديرنا لخدمتك الرائعة لنا وللمساهمات التي قدمتها لنا في الفترة الماضية، فقد كانت لا تقدر بثمن بالنسبة لنا، نتمنى لك كل التوفيق في مساعيك المستقبلية.

لكم منا عظيم الشكر و جزي الأمتنان.

نهدي هذا العمل لأرواح شهدائنا البواسل رحمهم الله ولا ننسى الذين ضحوا بأزهار شبابهم أسرانا و أسيراتنا البواسل .

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.

ABSTRACT:

This complex was chosen to be built in the city of Haifa at Caramel Street-Palestine, due to its need in that area, which in turn brings economic and living benefit since it contains shops and apartments.

Many important aspects of this project were covered, such as project description, hypothesis, goals, objectives, hypotheses and proof of results.

The building is consisted of a basement as a parking, a ground floor which is for commercial use, five stories for residential use, a small roof that contain services for the building. The height of this building is mostly 27 meters, including the eight stories. The total area of the building is 6887 square meters. The site elevation is about 450 meters AMSL (above mean sea level).

This project was designed in two steps: the first step was understanding the architectural drawings, estimating the expected loads, taking safety coefficients and modeling the structure with the computer program ETABS. The global structural codes such as ACI-318-19 and ASCE7-22 were used to make sure the work is correct according to the specifications and standards. The second step included the design of the structural elements such as slabs, beams, columns and walls using ETABS.

At the end of this project all structural elements were designed based on the specifications and standards of the codes used in this project to have a safe and economical design, and professional structural drawings were be developed using AutoCAD.

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1 INTRODUCTION

1.1 General: -

To design a project from structural aspect, a well knowledge of architectural drawings is needed, because the structural solutions depend mainly on the amount of information that is provided by the architects, and if any restriction, they may change the resultant solution, that's why identifying the architectural drawings is considered to be the first step in the design method.

Since, the architectural drawings have a huge impact on the resultant design, this chapter will mention general information about the project, which includes the location of the project, elevations, number of floors, floor heights, nature of soil and any restriction that may be due to the site geotechnical properties, which may lead to change something in the model.

After gathering all the important information, the number of solutions will decrease, and the most suitable solution according to the site geometry properties will be chosen.

So, this chapter will identify and illustrate some of the topics about the project, which include: -

1) Project description: which will describe the project in general.
2) Analysis and design principles: which will describe the assumptions about the structural elements.
3) Codes and standards: which will mention the codes and standards which will be used and applied in this project.
4) Materials: The materials which will be used in the project.
5) Loads: This section will mention the loads which will be involved in the building.
6) Load combinations: this section will describe the load combinations that will be used from the codes in this project.
7) Geotechnical investigation: which will discuss the property of the soil such as the bearing capacity and the site class of the chosen location to build the project on. By knowing the soil type, it will be easier to decide the right foundation type to carry the structure on.

1.2 Project description: -

The project is a structural analysis and design of a commercial complex, called Al-Tajamo' Commercial Building, which is located in Haifa city, near Caramel Street, the land of the site is approximately square which is perfect for the building, and it has most services near it which make the site a perfect location for this type of buildings. The site elevation is about 450 m AMSL (above mean sea level). Figures 1.2-1 and 1.2-2 show the building and it's location respectively.

Figure 1.2-1: Al-Tajamo' Commercial Building.

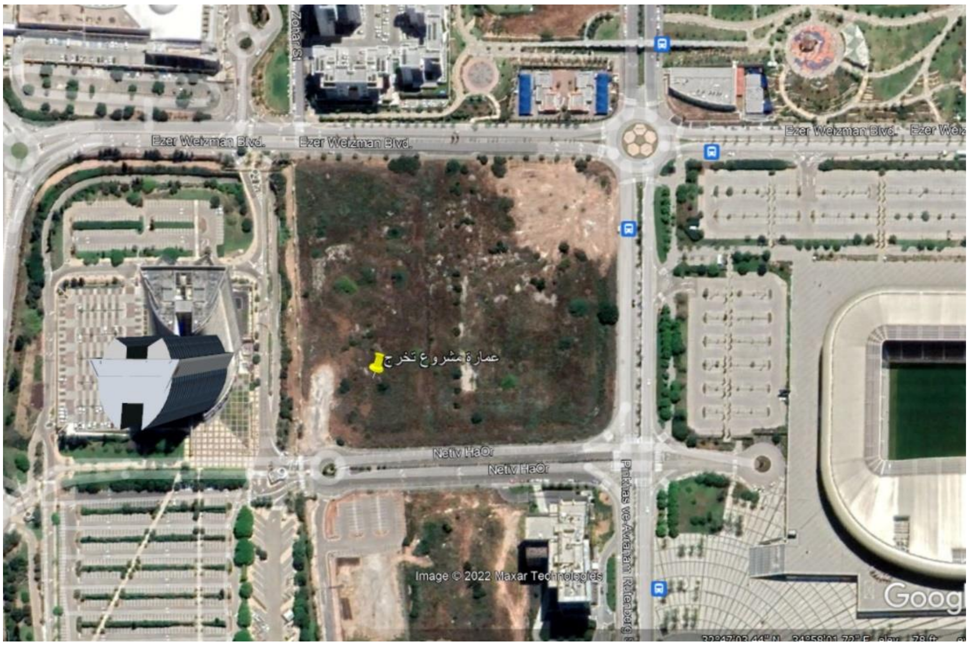

Figure 1.2-2: Site location.

The building is consisted of eight stories, which are divided into basement, five stories and a roof, the basement will be used as a parking, the six middle stories are divided into two types: the ground floor will be for commercial use and the other five floors are residential, each floor is divided into four apartments, the roof will be used for services which will help in the management of the building. The building height will be around ۲۷ m, the basement height is 3.2 m, the ground floor is 4.2, the roof is 2.5 m and the rest of the floors will be 3.2 m. Figure 1.2-3 shows the plan of the building.

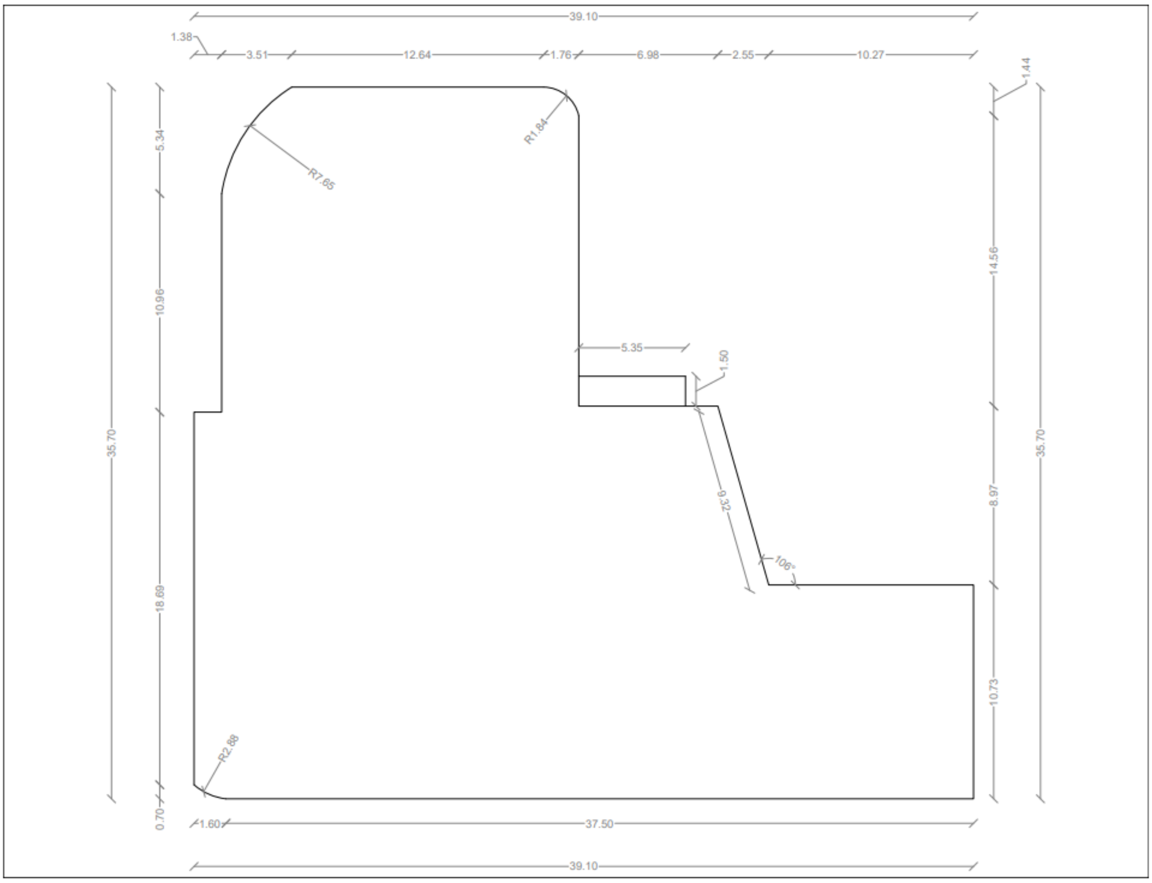

Figure 1.2-3: General outline of Al-Tajamo' Commercial Building.

The building includes interior walls which are 10 cm and 20 cm in thickness and are made of gypsum and wood, the staircase and elevator walls are 20 cm thick and will be constructed as shear wall, and the exterior walls are shear walls of 30 cm thick and made of three layers 20 cm shear wall, 7 cm reinforced concrete and 3 cm stone, with some openings, which are windows for the building. Figures 1.2-4 and 1.2-5 illustrate the walls cross-section.

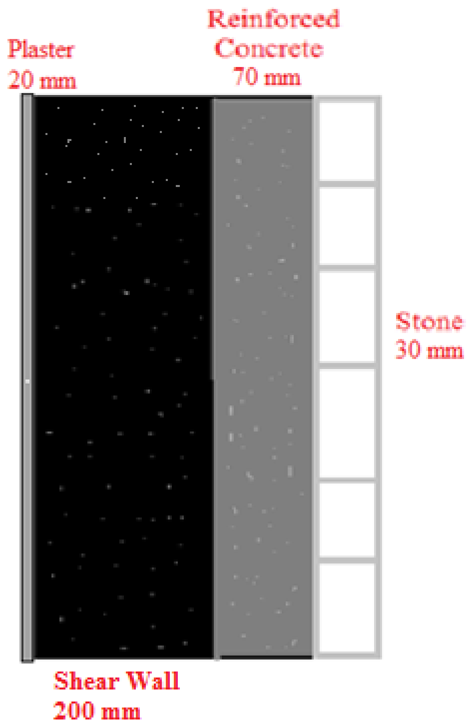

Figure 1.2-4: Exterior walls cross-section.

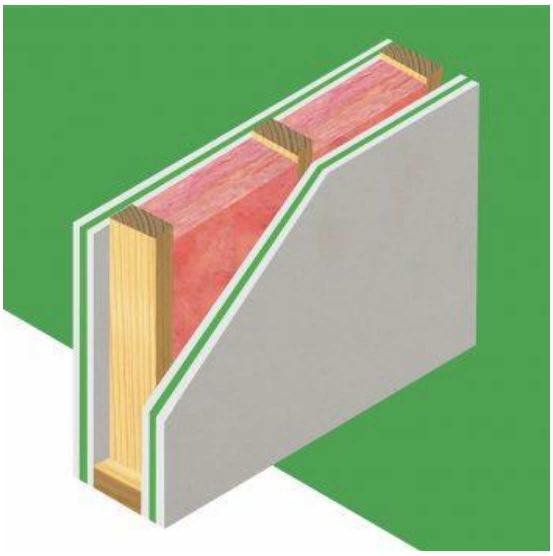

Figure 1.2-5: Interior walls cross-section.

The building total area is around 6,886 square meters, the basement is 944 square meters, the roof is 264 square meters, the ground floor is 953 square meters and the rest of the floors are 945 square meters. Table 1.2-1 summarize the floors characteristics.

Table 1.2-1: Floors characteristics

Floor	Area (m^2)	Height (m)	Use
Basement	944	3.2	Garage
Ground Floor	953	4.2	Commercial
Floor 1-5	945	3.2	Apartments
Roof	264	2.5	Services

Figures 1.2-6 to 1.2-9 show the different floors outlines and columns distribution.

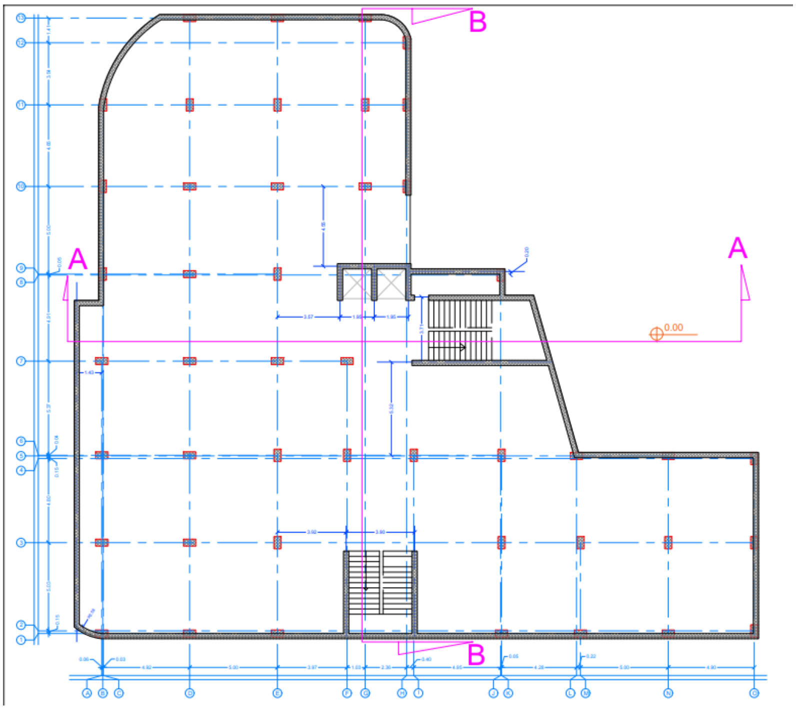

Figure 1.2-6: Basement outline and columns distribution.

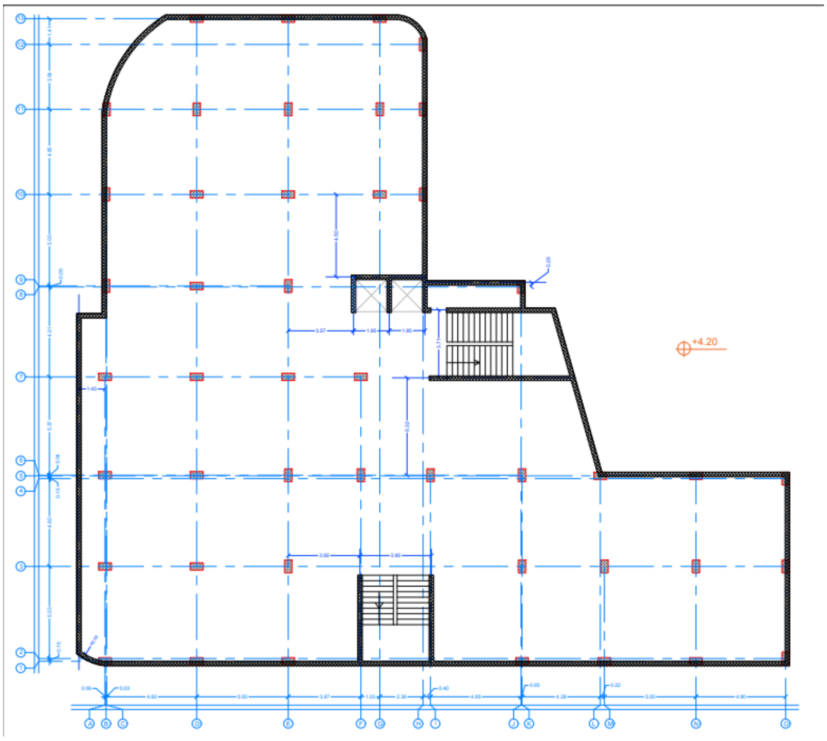

Figure 1.2-7: Ground floor outline and columns distributeon.

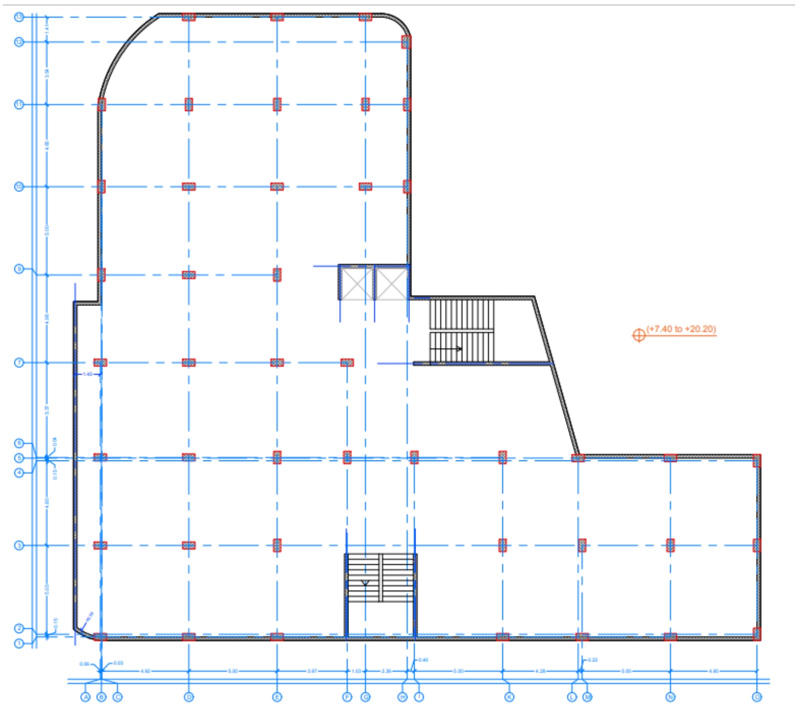

Figure 1.2-8: Residual floors outline and columns distribution.

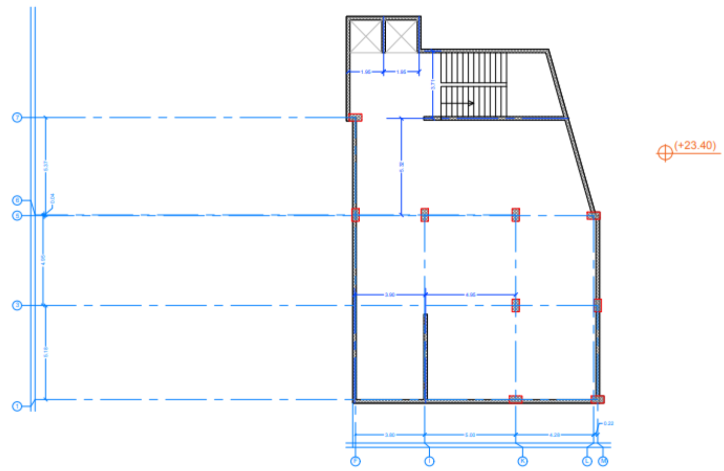

Figure 1.2-9: Roof outline and columns distribution.

1.3 Analysis and design principles:

The project was designed to be economical and safe, and the analysis and design assumptions were as follows:

- The supports were assumed to be fixed, so the foundations can resist the applied moments.
- The designed system was assumed to be a two-way flat plate.
- All slabs were two-way flat plate, which means the reinforcement are in two directions for all slabs.
- There will be beams around the ground floor slab.
- A beam will be constructed above the basement garage door.
- Partitions were not considered as structural elements, instead, they will be considered as a part of the superimposed dead load.
- The method that was used in checking the design is the direct design method, which is a method used to calculate the real bending moment diagram, and depends on the coefficients from the ACI code.
- Assuming the live load and dead load are distributed uniformly on the slabs.
- The exterior walls are shear walls (20 cm) covered by stone (7 cm) and a reinforced concrete layer (3 cm) between them.
- The interior walls are a mix of gypsum and wood for partitions and shear walls for staircases and openings.
- The foundations were assumed to be mat foundation.
- The building was represented as a three-dimensional model using ETABs.

1.4 Codes and standards: -

During the design of this building, several codes were used to make sure the design is safe and stable, these codes have a main role in controlling the material used, the dimension, the sections forms, the reinforcement details of each member and so on.

In this project four codes were used in the design phase, which are: -

- ACI-318M-19: which was used in the design of structural elements (Static design).
- ASCE-7-22 Minimum Design Loads and Associated Criteria for Buildings: which was used in the design of the earthquake in the building.
- Jordan Loads and Forces Code ,2nd Edition ,2006 : Which was used for the loads connected to the rain and snow.
- AASHTO, 8th Edition ,2017 : This code was used to identify some of the materials.

1.5 Materials: -

In this project multiple materials were used, these materials are divided into two types, materials used in structural members which includes concrete and reinforcement steel, and materials that are used in non-structural members which includes tiles, plastering, stone, wood, sand and gravel …etc.

1.5.1 Concrete: -

The concrete is one of the main materials used in the construction phase, which is a mix of aggregate, water, sand and cement, which are mixed in specific ratios to result the needed.

The concrete is known to be strong in compression which is important, but weak in tension which lead to use reinforcing steel to improve the tensile strength and give the structural element some ductility.

The concrete that was used in this project reinforcing steel is 28 MPa (B350), properties of concrete are in table 1.5-1.

Table 1.5-1: Concrete properties.

Unit Weight, γ_c	25 KN/m^3
Modulus of Elasticity, E_c	4700 $\sqrt{f}$ = 24780 MPa
Poisons Ratio, v	0.2
Compressive Strength, f_c	28 MPa
Flexure, f_r	0.62 λ $\sqrt{f}$ = 3.28 MPa
Tensile Strength, f_t	0.33 λ $\sqrt{f}$ = 1.75 MPa

1.5.2 Reinforcing Steel: -

Steel is a strong material against tension and have a high ductility compared to concrete, which is the reason why it’s used in the concrete as a reinforcement in tension zones.

The steel that was used in this project is grade 60 with a yielding stress of 420 MPa and an ultimate stress of 620 MPa.

1.5.3 Other Material: -

Different materials were used in the nonstructural members which are: -

- Gypsum and Wood: the main elements for partitions, and will be used as a light wall to prevent injuries if the wall falls during earthquake.
- Stone: stone is used as an outer cover for the external walls which gives the building aesthetic and isolation properties, such as temperature, sound, rain and fire.
- Tiles: are slabs cover which makes the floor more leveled and smoother.
- Other materials were used and all of their unit weight are in table 1.5-2.

Table 1.5-2: Materials unit weight.

Material	**Unit Weight KN/m^3**
Gypsum	8.08
Stone	26
Tiles	27
Cement Mortar	23
Sand	17
Aggregate filling	17
Concrete	25
Glass	26
Wood	6.6

1.6 Loads: -

Loads are a group of forces that affects the structure, which must be accurately calculated in order for the structure to be designed to bear these forces without failure. The nature of these loads varies mainly according to the architectural design of the building, the uses of the floors of the building, the location and environment in which the structure is located, as well as the properties of the materials used.

The loads are classified in general into gravitational loads and lateral loads. This chapter discussed each type of these loads.

1.6.1 Gravity loads:

- Dead load (D): It is the self-weight of all structural elements of the structure (reinforced concrete elements with unit weight gamma (ɣ) =25 KN/m3) and it will be calculated later using the ETABS program.
  These loads can be considered almost constant during the life of the structure.
- Superimposed Load (SD): it is considered as dead load, it results from the own weight of the non-structural elements in structure, such as backfill, tiles, cement mortar and partitions.
  Using ASCE 7-22 Table C3-1.

  The superimposed dead loads are composed of filling (120mm), cement mortar (20mm), tiles (30mm), plaster (15mm), as shown in figure 1.6-1.

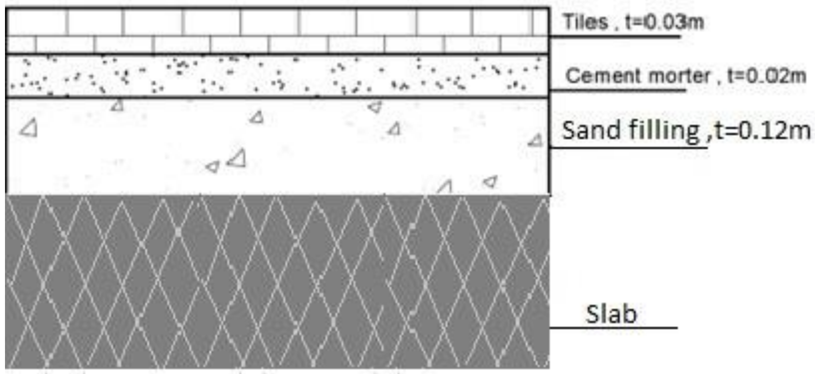

Figure 1.6-1: Slab cross-section.

Weight of non-structural elements (SD) = gamma * thickness

SD = W Tile + W Mortar+ W aggregate + W plaster

SD= (0.03*27) + (0.02* 23) + (0.12* 17) + (0.015* 23) = 3.655 KN/m²

- Load on Exterior walls:

  ⟹Thickness of shear wall with stone cladding is 0.3m, as shown in figure 1.6-2.

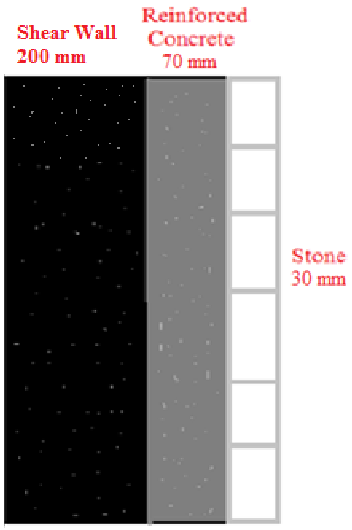

Figure 1.6-2: Exterior walls cross-section.

⟹Load (KN/m^2) for stone wall = unit weight * thickness

$$= ((0.03*\gamma_{(stone)}) + (0.07*\gamma_{(concrete)}))$$

$$= ((0.03*26) + (0.07*25)) = 2.53\ KN/m^2$$

$$=3\ KN/m^2$$

- Partitions that separate apartments that have thickness 10cm and 20 cm, partitions consist of wood and gypsum.

  Superimposed load from partitions:

= ((Length of partitions x thickness x floor height x $\gamma_{(wood)}$) + (Length of gypsum x thickness x floor height x $\gamma_{(gypsum)}$))/ Area

1) Ground Floor = ((0.1 x (15.2/0.5) x 0.05 x4.20 x6.6) +(0.15 x (87.21/0.5) x 0.1x 4.20 x 6.6) + (102.41 x 2x0.016 x 4.20 x8.08))/953 =0.193 KN/m^2
2) First and Other Story = ((0.1 x (155/0.5) x 0.05 x3.20 x6.6) +(0.15 x (51/0.5) x 0.1x 3.20 x 6.6) + (206 x 2x0.016 x 3.20 x8.08))/945 =0.25 KN/m^2

Note:

- The weight of the insulating layer is very light so it is approximately added to the calculated values.
- The thickness of the insulating layer is 18 mm.
- The layer of wood used is a frame like a column, with a width of 0.1 m, and it is repeated every 0.5 m

Table 1.6-1 shows the total super-dead loads (SD) at each floor.

Table 1.6-1: Super imposed dead load at slab.

Floor	Total SD
Ground Floor	3.85 ⟹ 4 KN/m^2
First Floor	3.91 ⟹ 4 KN/m^2
Second Floor	3.91 ⟹ 4 KN/m^2
Third Floor	3.91 ⟹ 4 KN/m^2
fourth floor	3.91 ⟹ 4 KN/m^2
Fifth Floor	3.91 ⟹ 4 KN/m^2
Roof Floor	2.5 KN/m^2

❖ Live Loads (L): The expected load that the structure will carry, such as furniture, people who use the structure, and vehicles. These loads differ from environmental loads, structural and non-structural loads.

The value of live load cannot be accurately determined, but the value of the live loads on any part of the structure is determined according to the function of this part based on the code ASCE 7-22 ,Table 4.3-1standard for the minimum design loads. Table 1.6-2 illustrate the live load in each floor.

Table 1.6-2: Live load at each floor.

Floor	Uses for	Live Load
Basement 1	Garages for Passenger vehicles only	1.92 ⟹ 3 KN/m^2
Ground Floor	Storage warehouses, Light	6 KN/m^2
First to Fifth Floors	1- Residential, Private rooms and corridors serving. 2- Public rooms and corridors serving them. 3- Balcony	1.92 ⟹ 2 KN/m^2 4.79 ⟹ 5 KN/m^2 5 KN/m^2
Roof	inactive	1 KN/m^2

- The average live loads on the first to fifth floor = ((Area of private rooms * live load) + (Area of public rooms*live load)) / Total Area

  Live load = ((116*2) + (108*3) / (224) = 2.5 KN/m^2 → 3KN/m^2

- Live load on balcony = 5 KN/m^2

❖ **Snow Loads:** According to the Jordanian code, the snow load depends mainly on a group of factors such as the height of the building above sea level, the specific weight of snow, and its value ranges from (0.1-0.4) KN/m^3, with average 0.25 KN/m^3.
From the Jordanian code, Table 3-5
The height of Haifa above sea level is 450m, so ⟹ 500>h>250
The value of the snow load (S_o) = (450-250)/800=0.25 KN/m^2
Table 1.6-3 shows the snow loads according to the height of the building above sea level.

Table 1.6-3: Snow loads.

Snow load kN/m2	Building elevation by see level h (m)
0	250 > h
(h-250)/800	500 > h > 250
(h-400)/320	1500 > h > 500

Refer to ASCE 7-٢٢ section 7.3 the minimum value of snow load

$P_f = 0.7\ C_e\ C_t\ P_g$

Where P_f: is the flat roof snow load in KN/m^2

C_e; is the Exposure Factor determined from table 7.3-1 in ASCE 7-22

C_t: is the thermal factor determined from tables 7.3-2 in ASCE 7-22

P_g: is equal to the Jordanian code (0.25 KN/m^2).

Surface roughness category is (C); because the building is located in an open terrain with scattered obstructions that have heights generally less than 30 ft (9.1 m). This category includes flat, open country and grasslands, as shown in figure 1.6-3.

26.7.2 Surface Roughness Categories A ground surface roughness within each 45 degree sector shall be determined for a distance upwind of the site, as defined in Section 26.7.3, from the categories defined in the following text, for the purpose of assigning an exposure category as defined in Section 26.7.3.

Surface Roughness B. Urban and suburban areas, wooded areas, or other terrain with numerous, closely spaced obstructions that have the size of single-family dwellings or larger.

Surface Roughness C. Open terrain with scattered obstructions that have heights generally less than 30 ft (9.1 m). This category includes flat, open country and grasslands.

Surface Roughness D. Flat, unobstructed areas and water surfaces. This category includes smooth mud flats, salt flats, and unbroken ice.

Figure 1.6-3: Surface Roughness Categories.

➢ Exposure Factor (C_e):

According to table 7.3-1 in ASCE 7-22⟹ Ce = 1.0

Table 1.6-4 shows the exposure factor (C_e) values due to surface roughness category.

Table 1.6-4: Exposure factor (C_e).

	Exposure of Roof[a]		
Surface Roughness Category	Fully Exposed[b]	Partially Exposed	Sheltered
B (see Section 26.7)	0.9	1.0	1.2
C (see Section 26.7)	0.9	1.0	1.1
D (see Section 26.7)	0.8	0.9	1.0
Above the tree line in windswept mountainous areas	0.7	0.8	N/A
In Alaska, in areas where trees do not exist within a 2 mi (3 km) radius of the site	0.7	0.8	N/A

The building is considered to be an unheated structure according to tables 7.3-2 in ASCE 7-22

So, according to tables 7.3-2 in ASCE 7-22⟹ Ct = 1.2

The following table 1.6-5 shows the values of the Thermal factor (Ct) due to the thermal condition.

Table 1.6-5: Thermal factor (C_t).

Thermal condition[a]	C_t
All structures except as indicated as follows	See Table 7.3-3
Unheated structures, open-air structures, structures kept just above freezing [40 to 50 °F (4 to 10 °C)], and other structures with cold, ventilated roofs meeting the minimum requirements of the applicable energy code	1.2
Freezer building	1.3
Continuously heated greenhouses[b] with a roof having a thermal resistance (R-value) less than 2.0 h·ft^2·°F/Btu (0.4 m^2·K/W) or a thermal transmittance (U-factor) greater than 0.5 Btu/h·ft^2·°F (2.5 W/m^2·K)	0.85

[a] These conditions shall be representative of the anticipated conditions during winters for the life of the structure.
[b] Greenhouses with a constantly maintained interior temperature of 50°F (10°C) or more, at any point 3 ft (0.9 m) above the floor level during winters and having either a maintenance attendant on duty at all times or a temperature alarm system to provide warning in the event of a heating failure.

➢ Minimum snow loads for low-slope roof:

Risk category is (III) so, according to table 7.3-4 in ASCE 7-22 ⟹ P_g = $P_{m,\ max}$ = 1.68 KN/m^2, and the Jordanian code will be used to get P_g value.

Table 1.6-6 shows the minimum snow loads for low-slope Roof P_m due to the risk category.

Table 1.6-6: Minimum snow loads for low-slope roof (P_m).

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)

Note: But according to the Jordanian code The used value will be **P_g = 0.25 KN/m^2**.

⟹The snow load on the roof = 0.7*1.0*1.2*0.25 = 0.21 KN/m^2 [According to the Jordanian code].

Note: The snow load will be ignored in the model, since the roof live load is 1.00 KN/m^2 which is higher than 0.21 KN/m^2.

1.6.2 Lateral loads:

❖ Seismic loads: As a result of vibrations and movement in the layers of the earth, it produces horizontal and vertical seismic forces, affecting the building and generating large shear forces from it.

The building is in Haifa City, Palestine, then, according to the seismic hazard map for Palestine with a probability of exceeding 2% within 50 years, the building will be under the following conditions:

- ❖ Site class: The site class is determined and depends on the properties of the soil (soil profile), according to table 20.2-1 in ASCE 7-22, Site class of Haifa is ⟹ C

  Table 1.6-7 illustrate the types of soil at site due to three variables.

Table 1.6-7: Soil classification.

Table 20.2-1. Site Classification.

Site Class	$\bar{v}_s$ Calculated Using Measured or Estimated Shear Wave Velocity Profile (ft/s)
A. Hard rock	>5,000
B. Medium hard rock	>3,000 to 5,000
BC. Soft rock	>2,100 to 3,000
C. Very dense sand or hard clay	>1,450 to 2,100
CD. Dense sand or very stiff clay	>1,000 to 1,450
D. Medium dense sand or stiff clay	>700 to 1,000
DE. Loose sand or medium stiff clay	>500 to 700
E. Very loose sand or soft clay	≥500
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.2.1

Note: For SI: 1 ft = 0.3048 m; 1 ft/s = 0.3048 m/s.

- From Palestine's seismic map:

  S_1=0.13

  S_s=0.67

Where: S_1 = the mapped spectral response acceleration parameter at long period (1sec).

S_S = the mapped spectral response acceleration parameter at short period (0.2sec).

- Site coefficients: These parameters (F_a and F_v), depend on spectral response acceleration parameter (S_1&S_s) and soil classification.

  Where: F_a = short-period site coefficient (0.2s period)

  F_v = long-period site coefficient (1.0s period).

F_a and F_v are defined according to Tables 11.4-1 and 11.4- 2 in ASCE 7-10 code. The values of F_a=1.132 and F_v =1.67, as shown in tables 1.6-8 and 1.6-9 respectively.

Table 1.6-8 & Table 1.6-9: Site coefficient (F_a & F_v).

Table 11.4-1 Site Coefficient, F_a

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				

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

Table 11.4-2 Site Coefficient, F_v

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7				

Note: Use straight-line interpolation for intermediate values of S_1.

⟹Maximum considered earthquake spectral response acceleration parameters (S_{MS} & S_{M1})

$S_{MS}= F_a * S_S = 1.132 *0.67 =0.758$

$S_{M1}= F_v* S_1= 1.67 *0.13=0.2171$

⟹Design spectral acceleration parameters (S_{DS} & S_{D1})

$S_{DS} = S_{MS} = 0.758$

$S_{D1}= S_{M1}= 0.2171$

- Risk category: According to table 1.5-1 in ASCE 7-22, risk category of the building is (III), as shown in table 1.6-10.

Table 1.6-10: Risk category of the building.

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 a 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 their quantity 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 exceeds a threshold quantity established by	IV

- ❖ Seismic Importance factor (I_e): According to table 1.5-2 in ASCE 7-22, Importance factor of the building =1.25, as shown in table 1.6-11.

Table 1.6-11: Seismic importance factors.

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

Risk Category from Table 1.5-1	Seismic Importance Factor, I_e
I	1.00
II	1.00
III	1.25
IV	1.50

- ❖ Seismic design category: Seismic design category depend on its risk category and the design spectral response acceleration parameters (S_{DS} and S_{D1}).

According to table 11.6-1 & 11.6-2 in ASCE 7-22, where: S_{DS} =0.758, S_{D1} =0.2171, Risk Category (III) ⟹ so, the Seismic Design Category is (D), as shown in the tables 1.6-12 and 1.6-13 respectively.

Table 1.6-12 & Table 1.6-13: Seismic Design Category.

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

Value of S_{DS}	Risk Category I or II or III	Risk Category 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

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

Value of S_{D1}	Risk Category I or II or III	Risk Category 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	

❖ Type of lateral forces resisting systems:

Choosing a lateral force resisting systems depends mainly on seismic design category and the height of the building according to the table 12.2-1 from ASCE 7-22, as shown in table 1.6-14.

Table 1.6-14: Seismic force-resisting system.

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^{a}	Overstrength Factor, Ω_0^{g}	Deflection Amplification Factor, C_d^{b}	Structural System Limitations Including Structural Height, h_n (ft) Limits[c] — Seismic Design Category B	C	D[d]	E[d]	F[e]
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[l,m]	14.2	6	2½	5	NL	NL	160	160	100
5. Ordinary reinforced concrete shear walls[l]	14.2	5	2½	4½	NL	NL	NP	NP	NP
6. Detailed plain concrete shear walls[l]	14.2 and	2	2½	2	NL	NP	NP	NP	NP

Continued

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The proposed system is building frame system (Special reinforced concrete shear walls).

- ❖ Response modification coefficient (R): It is known as the reduction factor of the basic shear (V).

$$V_{design} = V_{actual}/R$$

R is one of the seismic design criteria that must be considered in the non-linear performance of building structures during a strong earthquake. In fact, the response modulation factor (R) reflects the ability of the structure to dissipate energy through the inelastic behavior, response modification is affected by the height of structures and the factor R is a function of a ductility.

From ASCE 7-22 table 12.2-1, R = 6

- ❖ Over-Strength Factor (Ω): Over strength is a parameter used to quantify the difference between the required and the actual strength of material, which is depends on different factors which the most important of them is member ductility factor, which is increases as the ductility supply of the building increases.

Over strength factor, which may be defined as the ratio of maximum base shear in actual behavior to first significant yield strength in structure.

In Fact, the structures possess significant reserve strength (over strength) and capacity to dissipate energy (ductility).

From ASCE 7-22 table 12.2-1, **Ω** = 2.5

- ❖ Amplification factor for elastic deflection (C_d): is defined as the maximum nonlinear displacement during an earthquake (D_{max}), divided by the elastic displacement (D_s) calculated using reduced seismic design forces.

From ASCE 7-22 table 12.2-1, C_d = 5

- ❖ Structural period (fundamental period): The time required for one cycle of free vibration.

For seismic-resistant structures each structure has a unique natural or fundamental period of vibration.

Depend on the structural height (h_n) and building period coefficient (C_t), the approximate fundamental period (Ta), shall be determined from the following equation:

$$T_a = C_t\, h_n^{x} \qquad \text{(ASCE 7-22 equation 12.8-8)}$$

Where, the coefficients C_t and x are determined from table 12.8-2 from ASCE 7-22, as shown in table 1.6-15.

C_t=0.0488

X=0.75

T_a=0.0488×27.25$^{0.75}$=0.58 sec

Table 1.6-15: Period parameters (C_t & X).

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

The fundamental period (T_a), 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), as shown in table 1.6-16.

S_{D1} =0.2171, by linear interpolation⟹ C_u=1.49

So, the limitation of period ⟹ T_a× C_u= (0.58×1.49) =0.86 sec

Table 1.6-16: Coefficient for upper limit on calculated period.

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

❖ Computing Seismic Base Shear (V): is measured value of the maximum expected lateral force on the base of the structure due to seismic activity.

The seismic base shear in a given direction shall be determined in accordance with the following equation:

$V= C_s \times W$ (ASCE 7-22 Eq. 12.8-1)

Where;

Cs = the seismic response coefficient.

W = the effective seismic weight.

The seismic response coefficient (C_s), shall be determined in accordance with equation 12.8-2 in ASCE 7-22 .

$C_s= (S_{DS}/(R/I_e))$ (ASCE 7-22 Eq. 12.8-2)

Where, $S_{DS}=0.758$, $R=6$, $I_e=1.25 \Longrightarrow C_s=0.158$

The value of C_s, shall not exceed the following:

for $T \leq T_L$,

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)}$$

and for $T > T_L$,

$$C_s = \frac{S_{D1}T_L}{T^2\left(\frac{R}{I_e}\right)}$$

Where, T = the fundamental period of the structure (sec).

T_L = long-period transition period (sec), can be determinate from Palestine map.

Long period of Haifa (T_L) =8sec

Figure 1.6-4 shows the long period as a contour map in Palestine.

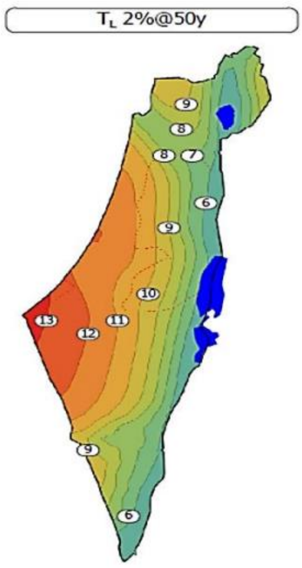


Figure 1.6-4: Long-Period Transition Period Map.

❖ Response spectrum curves (Horizontal and vertical): A response spectrum is a function of frequency or period, showing the peak response of a simple harmonic oscillator that is subjected to a transient even, response spectrum mainly depends on the period and acceleration. Figure 1.6-5 illustrate the response spectrum curve.

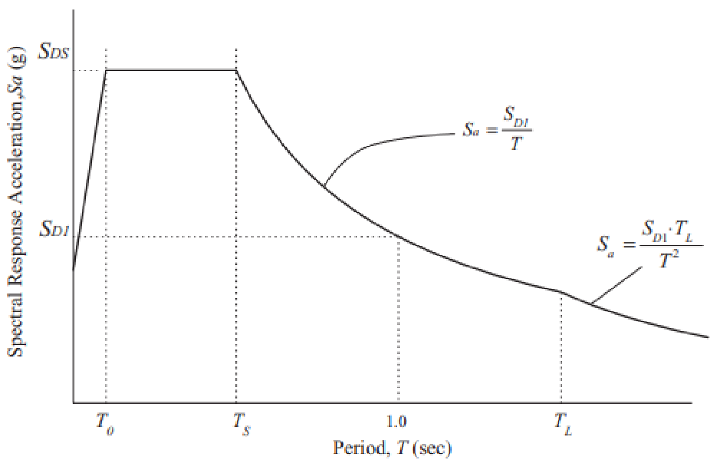


Figure 1.6-5: Two-period design response spectrum.

➢ For periods less than T_0, the design spectral response acceleration (S_a) shall be taken:

$$S_a = S_{DS} \times \left(0.4 + \left(0.6 \times \frac{T}{To}\right)\right) \qquad \text{(ASCE 7-22 equation 11.4-3)}$$

- For periods greater than or equal to T_0 and less than or equal to T_S, the design spectral response acceleration, S_a, shall be taken equal to S_{DS}.

$$S_a = S_{DS}$$

- For periods greater than T_S, and less than or equal to T_L, the design spectral response acceleration, S_a, shall be taken:

$$S_a = S_{D1}/T \qquad \text{(ASCE 7-22 equation 11.4-4)}$$

- For periods greater than T_L, S_a shall be taken:

$$S_a = (S_{D1} \times T_L)/T^2 \qquad \text{(ASCE 7-22 equation 11.4-5)}$$

Where:

S_{DS} = the design spectral response acceleration parameter at short period.

S_{D1} = the design spectral response acceleration parameter at 1-s period.

T = the fundamental period of the structure.

$T_0 = 0.2 \times \frac{SD1}{SDs}$

$T_s = \frac{SD1}{SDs}$

T_L = long-period transition period.

Table 1.6-17 shows the values of most of the needed seismic variables in this project.

Table 1.6-17: Seismic variables.

S_{D1}	S_{Ds}	T_o(sec)	T_S (sec)	T_L (sec)	T (sec)
0.2171	0.758	0.033	0.165	8	0.58

$T_s < T < T_L \implies S_a = S_{D1}/T = 0.374$ sec.

❖ Relations between response factor and over-strength, as shown in table 1.6-6.

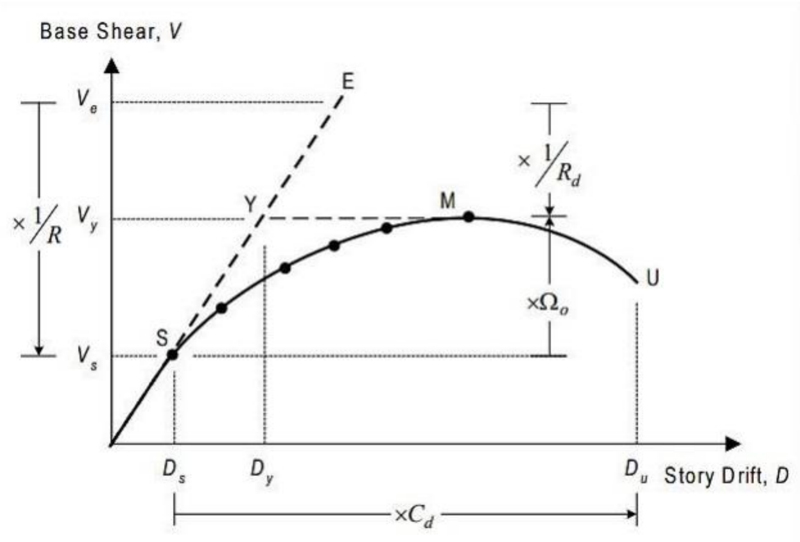


Figure 1.6-6: Response factor and over-strength.

⟹ the curve, shows the relationship between response factor and over-strength:

$$\Omega = V_y / V_s$$

$$R = V_E / V_s$$

Where:

R: Response factor.

Ω: Over strength factor.

V_y: Base shear after yielding.

V_s: Design seismic force.

V_E: Elastic design base shear.

C_d: Deflection amplification factor.

❖ Irregular structure: When the center of mass doesn't coincide with the center of stiffness, eccentricity develops in the structure. Eccentricity occurs due to the irregular arrangement of structural configuration which in turn induces torsion in the structure, different structural irregularities affect the seismic response of structures in different ways.

Structures shall be classified as having a structural irregularity based upon the criteria in this section, the irregularities are classified into two separate groups:

- Horizontal Irregularity: Structures having one or more of the irregularity types listed in Table 1.6-18 refer to:
    1. Torsional Irregularity
    2. Extreme Torsional Irregularity
    3. Reentrant Corner Irregularity
    4. Diaphragm Discontinuity Irregularity
    5. Out-of-Plane Offset Irregularity
    6. Nonparallel System Irregularity

According to ASCE 7-22 , table 12.3-1 .

Table 1.6-18: 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 Section 16.2.2	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 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	E and F 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 having 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 Section 16.2.2	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 Section 16.2.2	C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F

- Vertical Irregularity: Structures having one or more of the irregularity types listed in table 1.6-19 shall be designated as having a vertical structural irregularity.

1. Stiffness-Soft Story Irregularity
2. Stiffness-Extreme Soft Story Irregularity
3. Weight (Mass) Irregularity
4. Vertical Geometric Irregularity
5. In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity
6. Discontinuity in Lateral Strength–Weak Story Irregularity
7. Discontinuity in Lateral Strength–Extreme Weak Story Irregularity

According to ASCE 7-22 , table 12.3-2 .

Table 1.6-19: 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 elements irregularity is defined to exist where there is an in-plane offset of a vertical seismic force-resisting element resulting in overturning demands on a supporting beam, column, truss, or slab.	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	12.3.3.1 12.3.3.2 Table 12.6-1	D, E, and F B and C D, E, and F

- Soil pressure: The pressure which acts in the horizontal direction of the soil is known as lateral pressure. It is an important factor that is considered in the design of structures such as retaining walls, basements, tunnels and deep foundations.

There are three categories of lateral earth pressure and each depends upon the movement experienced by the vertical wall on which the pressure is acting

1. Active earth pressure.
2. Passive earth pressure.
3. Earth pressure at rest.

Total lateral earth pressure profiles due to static and dynamic earth pressures:

- Earth pressure at rest (Soil pressure): the lateral earth pressure exerted by the backfill on a retaining wall which is fixed in position and cannot move.

$$p_0 = K_0 \sigma_z$$

Where:

K_0: coefficient of earth pressure at rest ⟹ $K_0 = 1 - \sin Ø = 0.5$

Ø: effective soil stress friction angle, (30°) is the expected initial backfill friction angle.

σ_z: the vertical stress due to the self-weight of the soil at depth z ⟹ $\sigma_z = \gamma \times z$

z = height of the retaining wall= 3.2 m

γ: The unit weight of soil, unit weight ranges are:

- $\gamma_{dry\ soil}$ ⟹ (14 - 20) KN/m^3 ⟹ (average 17 KN/m^3)
- $\gamma_{saturated\ soil}$ ⟹ (18 – 23) KN/m^3 ⟹ (average 20 KN/m^3)
- γ_{water} ⟹ (9.81) KN/m^3 ⟹ (10 KN/m^3)

Soil pressure (P_o) = 20*0.5 *3.2 =32 KN/m^2

- Dynamic Earth Pressure: The dynamic lateral earth pressures measured directly by earth pressure cells during seismic shaking, which is assumed to be a uniform pressure.

$\Delta_P = 0.4\times k_H \times \gamma_{total} \times Z_{basement\ floor}$, as defined in Section 8.6 ASCE 41-13

$=0.4 \times 0.3032 \times 20 \times 3.2$

$=7.762$ KN/m^2

Where:

Δ_P: Earth pressure caused by seismic shaking

K_H: Horizontal seismic coefficient

K_H: assumed equal to S_{xS} /2.5$\Longrightarrow$ k_H=0.758/2.5=0.3032

S_{xs} = spectral response acceleration parameter = Fa*S_s

F_a = short-period site coefficient (0.2s period) =1.132

S_S = the mapped spectral response acceleration parameter at short period (0.2sec) =0.67

γ: backfill soil unit weight.

Assume the net surcharge load that should be applied is **10 KN/m^2**.

Total soil pressure (at z = -3.2) $\Longrightarrow$ 10+32+7.762 = 49.762 KN/m^2

Total soil pressure (at z = ٠) $\Longrightarrow$ 7.762+10 = 17.762 KN/m^2

Figure 1.6-7 illustrate the seismic stress distribution.

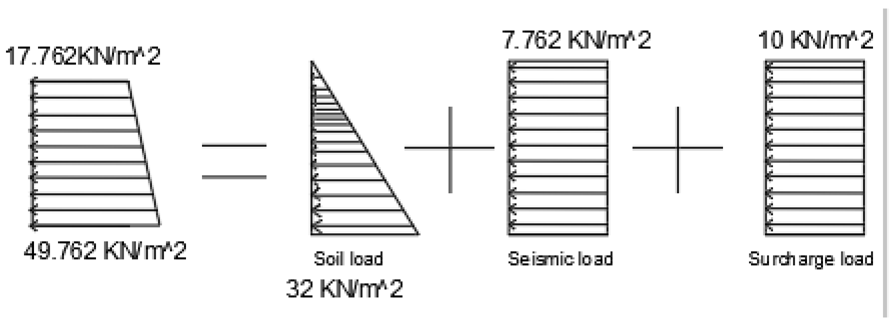


Figure 1.6-7: Seismic stress distribution.

1.7 Load combinations: -

When more than one load type is applied to the structure, a load combination is created, which is the algebraic sum of each of the load case, multiplied by a load factor in order to ensure the safety of the structure.

According to ASCE 7-22:

- Service load combinations:
  1) D
  2) D + L
  3) D + (0.6W or 0.7E)
  4) D + 0.75L + 0.75(0.7E) + 0.75S
  5) 0.6D + 0.7E

- Ultimate load combinations:
  1) 1.4D
  2) 1.2D + 1.6L + 0.5(L_r or S or R)
  3) 1.2D + 1.0E + L + 0.2S
  4) 0.9D + 1.0E
  5) 0.9 D+ 1.0 W
  6) 1.2D+1.6 (L_r or S or R) + (L or 0.5W)
  7) 1.2 D+1.0 W+L +0.5 (Lr or S or R)

Where:

D: Dead Load.

L: Live Load.

L_r: Roof Live Load.

S: Snow Load.

R: Rain Load.

W: Wind Load.

E: Earthquake Load.

- Direction of loading: Direction of loading procedures intend to address the occurrence of earthquake shaking along two principal axes of a building simultaneously.

  The directions of application of seismic forces used in the design shall be those which will produce the most critical load effects.

- for Seismic Design Category (D), use one of the following procedures:
  1. Orthogonal Combination Procedure: This method is applied, if members and their foundations are designed for 100 percent of the forces for one direction plus 30 percent of the forces for the perpendicular direction.
  2. Simultaneous Application of Orthogonal Ground Motion.

There is a possibility that the earthquake will not hit the building exactly on the X or Y axes, so these equations can be used according to section 12.4.2 in ASCE 7-22 :

→in X direction: Ex + 0.3 Ey.

→in Y direction: Ey + 0.3 Ex.

The seismic load effect (E), shall be determined in accordance with the following:

- For use in load combination 3 (1.2D + 1.0E + L + 0.2S) ⟹ Use, $E = E_h + E_v$

- For use in load combination 4 (0.9D + 1.0E) ⟹ Use, $E = E_h - E_v$

Where:

Eh = effect of horizontal seismic forces as defined in Section 12.4.2.1 ASCE 7-22.

$E_h = \rho Q_E$ (ASCE 7-22 equation 12.4-3)

ρ = redundancy factor ⟹ for seismic design category (D, E, F) ⟹ ρ=1.3

⟹ another seismic design category ⟹ ρ=1

E_v = effect of vertical seismic forces as defined in Section 12.4.2.2 ASCE 7-22.

$$E_v = 0.2 \times S_{DS} \times D = 0.152 \times D \qquad \text{(ASCE 7-22 Eq. 12.4-4a)}$$

S_{DS} = design spectral response acceleration parameter at short periods = 0.758

Seismic Load Combinations ⟹ Basic Combinations for Strength Design:

1. $(1.2 + 0.2S_{DS})D + (\rho Q_E) + (L) + (0.2 \times S)$
2. $(0.9 - 0.2S_{DS})D + \rho Q_E$

1.8 Geotechnical investigation: -

The soil is considered to be the main body that absorbs all the loads that comes from the building. So, in order to provide safety to the building, a well-known information about the soil in that site is needed concerned of the soil bearing capacity and the sound wave speed, and the occurrence of any settlement should be avoided, the seismic resisting system entirely depends on the site class and the properties of it. Therefore, a geotechnical investigation was done.

According to the results obtained from the site investigation report, the bearing capacity of the soil is **200 KN/m^2 (2000 PSF),** and the sound wave speed is **570 m/s (1870 ft/s)**.

According to the ASCE 7-22 the soil class is classified as class **C**, as shown in table 1.8-1.

Table 1.8-1: Site classification.

Site Class	$\bar{v}_s$ Calculated Using Measured or Estimated Shear Wave Velocity Profile (ft/s)
A. Hard rock	>5,000
B. Medium hard rock	>3,000 to 5,000
BC. Soft rock	>2,100 to 3,000
C. Very dense sand or hard clay	>1,450 to 2,100
CD. Dense sand or very stiff clay	>1,000 to 1,450
D. Medium dense sand or stiff clay	>700 to 1,000
DE. Loose sand or medium stiff clay	>500 to 700
E. Very loose sand or soft clay	≥500
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.2.1

Note: For SI: 1 ft = 0.3048 m; 1 ft/s = 0.3048 m/s.

2 PRELIMINARY DESIGN, PRELIMINARY DIMENSIONS OF STRUCTURAL ELEMENTS: -

2.1 General: -

This chapter will identify and illustrate some of the topics about the project, which includes:

1) Lateral and gravity forces resisting systems.
2) Slabs structural systems.
3) Preliminary slab thickness and loads.
4) Preliminary dimensions of beams.
5) Preliminary dimensions of columns.
6) Preliminary dimensions of walls.
7) Preliminary assumption of staircase.

2.2 Lateral and gravity forces resisting systems: -

The gravity force resisting system in a building is responsible for resisting the gravity loads, which will include the dead loads and the live loads that are acting on the building, which may call static loads.

Moment frames, shear walls and braced frames are lateral force-resisting systems found in structures. The three types of systems are often found in areas with high wind and/or seismic activity, like earthquakes and hurricanes. These vertical elements play the most role in keeping the structure from blowing over or collapsing.

The gravity force resisting system is **two-way flat plate solid slab with beams around the perimeter** and the lateral force resisting system is **building frame system**.

2.3 Slab structural system: -

The slab structural system used in this building is a **two-way flat plate solid slab**, which is a slab of uniform thickness directly supported on columns.

Figures 2.3-1 to 2.3-5 show the slabs layout and beams for each floor category.

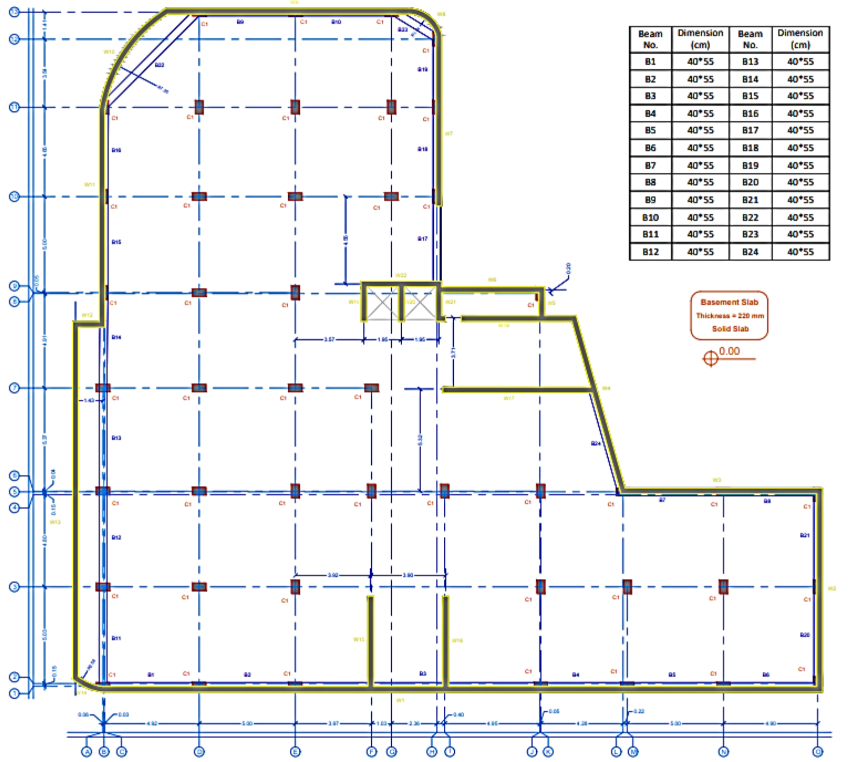
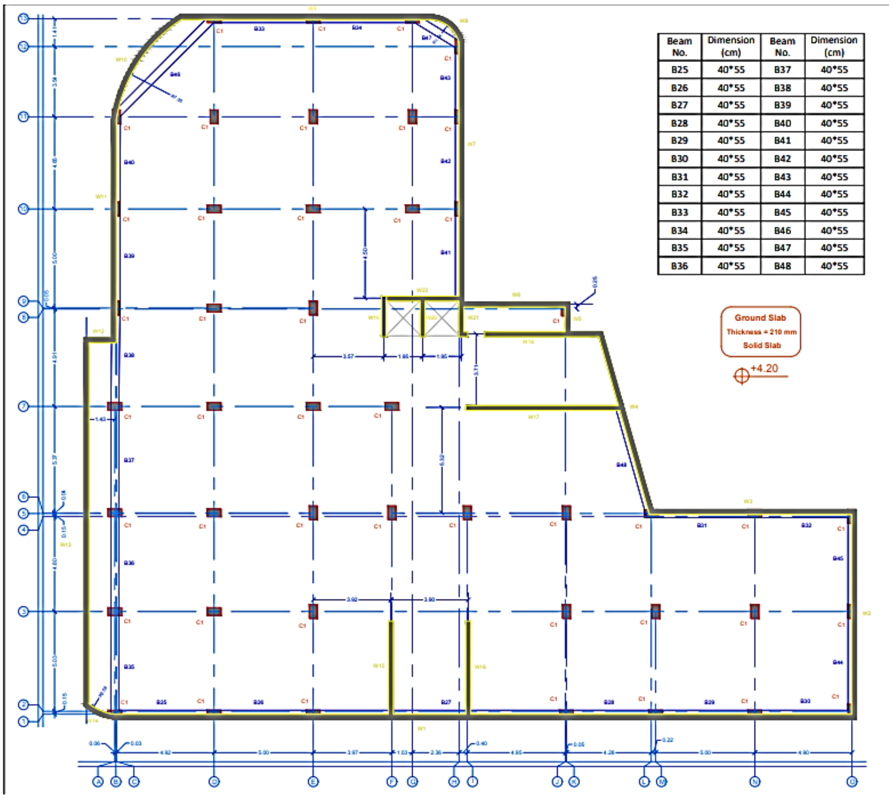
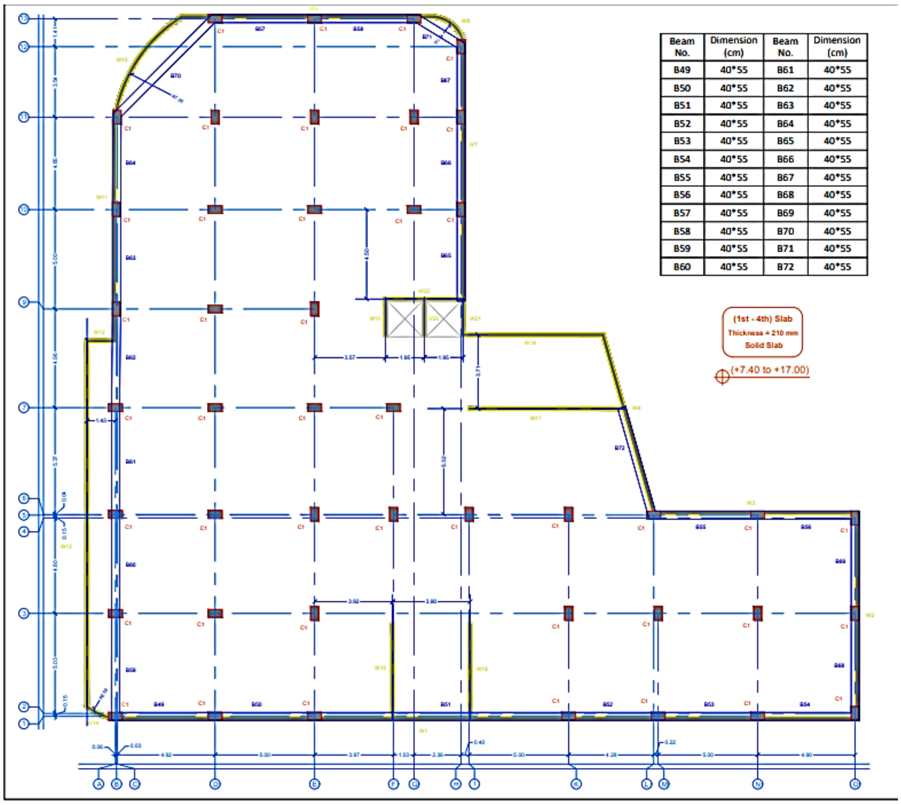


Figure 2.3-1: Basement slab layout and beams distribution.



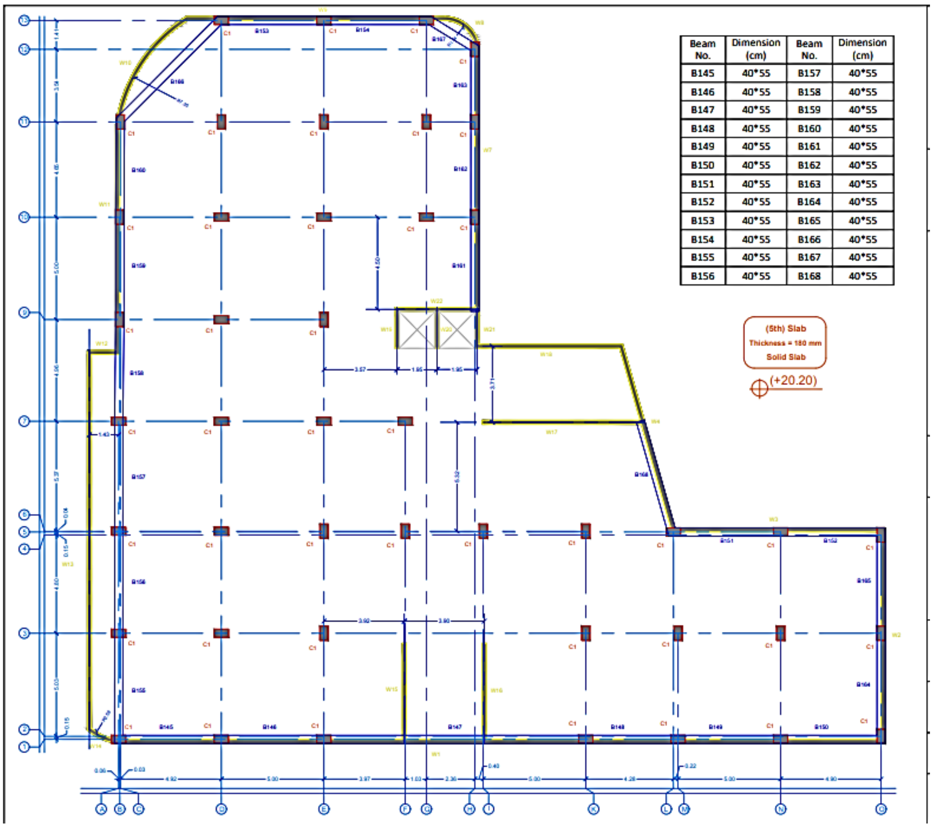
Beam No.	Dimension (cm)	Beam No.	Dimension (cm)
B25	40*55	B37	40*55
B26	40*55	B38	40*55
B27	40*55	B39	40*55
B28	40*55	B40	40*55
B29	40*55	B41	40*55
B30	40*55	B42	40*55
B31	40*55	B43	40*55
B32	40*55	B44	40*55
B33	40*55	B45	40*55
B34	40*55	B46	40*55
B35	40*55	B47	40*55
B36	40*55	B48	40*55

Figure 2.3-2: Ground slab layout and beams distribution.



Beam No.	Dimension (cm)	Beam No.	Dimension (cm)
B49	40*55	B61	40*55
B50	40*55	B62	40*55
B51	40*55	B63	40*55
B52	40*55	B64	40*55
B53	40*55	B65	40*55
B54	40*55	B66	40*55
B55	40*55	B67	40*55
B56	40*55	B68	40*55
B57	40*55	B69	40*55
B58	40*55	B70	40*55
B59	40*55	B71	40*55
B60	40*55	B72	40*55

Figure 2.3-3: Typical (1st – 4th) slab layout and beams distribution.



Beam No.	Dimension (cm)	Beam No.	Dimension (cm)
B145	40*55	B157	40*55
B146	40*55	B158	40*55
B147	40*55	B159	40*55
B148	40*55	B160	40*55
B149	40*55	B161	40*55
B150	40*55	B162	40*55
B151	40*55	B163	40*55
B152	40*55	B164	40*55
B153	40*55	B165	40*55
B154	40*55	B166	40*55
B155	40*55	B167	40*55
B156	40*55	B168	40*55

Figure 2.3-4: Typical (5th) slab layout and beams distribution.

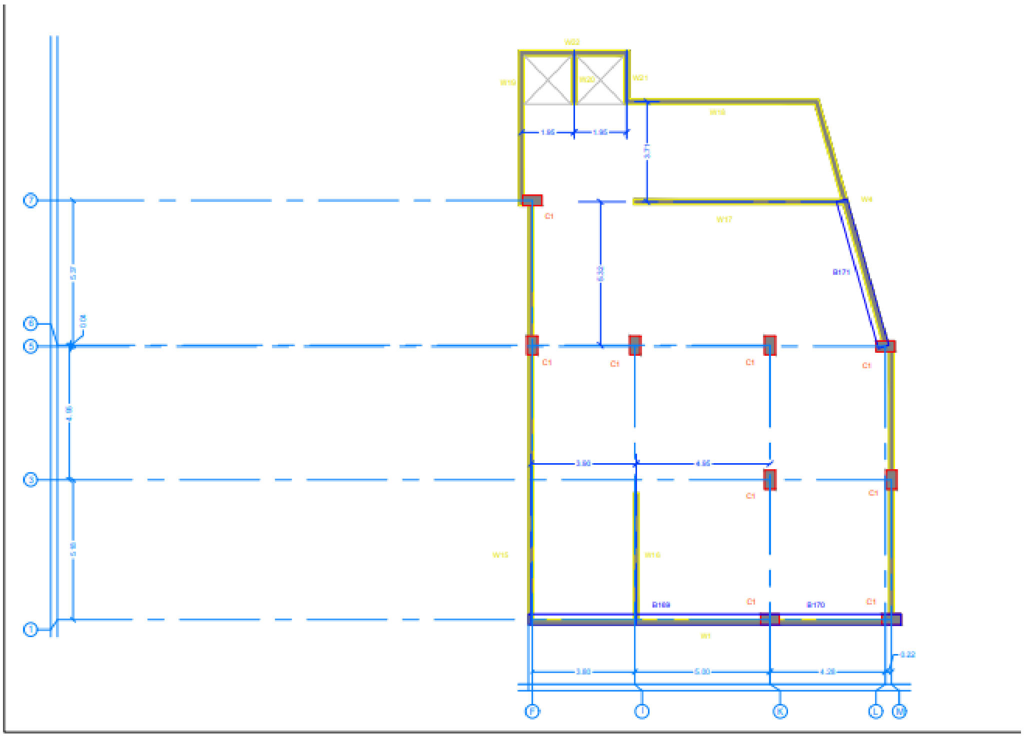


Figure 2.3-5: Roof slab layout and beams distribution.

2.4 Preliminary slab thickness and loads: -

A slab is a flat, two-dimensional planar structural component of building having a very small thickness compared to its other two dimensions. Reinforced concrete slabs are used in roofs, floors, ceilings and as the decks of bridges.

The slab used in this project is a two-way flat plate solid slab, the thickness will be taken according to the ACI 318-19 code.

The largest span in the slab is 5.37 m, so according to table 2.4-1 which is taken from ACI 318-19.

f_y, psi[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]	
40,000	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$	$\ell_n/36$	$\ell_n/40$	$\ell_n/40$
60,000	$\ell_n/30$	$\ell_n/33$	$\ell_n/33$	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$
80,000	$\ell_n/27$	$\ell_n/30$	$\ell_n/30$	$\ell_n/30$	$\ell_n/33$	$\ell_n/33$

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

The largest span is found in the panel between columns [3D,3E,4D,4E] according to ETABS gridlines, as shown in figure 2.4-1.

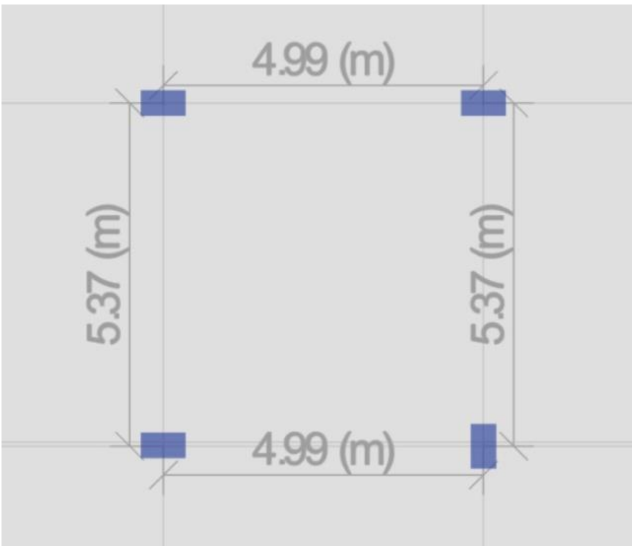


Figure 2.4-1: Slab panel.

$\frac{l}{33} = \frac{5370}{33}$ = 162.72 mm → Take t = 180 mm

As an example, for a frame strip, take a frame strip from the ground floor (Basement slab) in the Y-axis, as shown in figure 2.4-2.

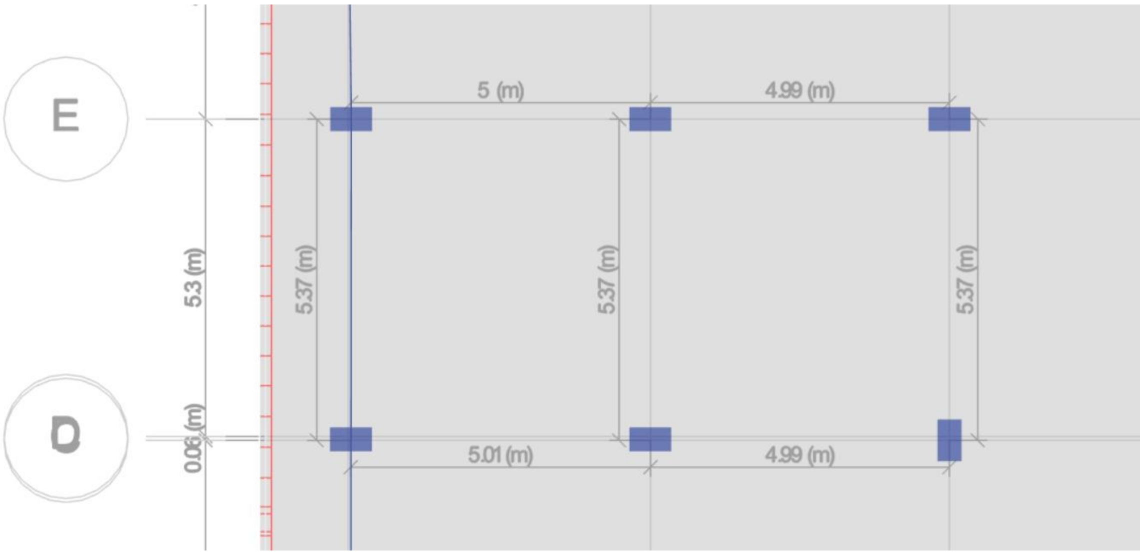


Figure 2.4-2: Frame strip.

By combination [1.2 (D.D. + S.D.) + 1.6 L.L.] → 1.2 * (0.18 * 25 + 4) + 1.6 * 6
→ W_u = 19.80 KN/m^2

$$M_o = \frac{Wu*l2*ln*ln}{8} = \frac{19.80*5.00*(5.37-0.30)*(5.37-0.30)}{8} = 318.10 \text{ KN.m}$$

By using the ACI 318-14 direct design method, distribution coefficients for end spans as shown in table 2.4-2.

Table 2.4-1: Distribution coefficients for end spans.

	Exterior edge unrestrained	Slab with beams between all supports	Slab without beams between interior supports: Without edge beam	Slab without beams between interior supports: With edge beam	Exterior edge fully restrained
Interior negative	0.75	0.70	0.70	0.70	0.65
Positive	0.63	0.57	0.52	0.50	0.35
Exterior negative	0	0.16	0.26	0.30	0.65

Since, the span is interior, take the negative coefficients equal 0.65, and the positive coefficient equals 0.35.

$M_{negative}$ = 0.65 * M_o → 0.65 * 318.10 = 206.77 KN.m

$M_{positive}$ = 0.35 * M_o → 0.35 * 318.10 = 111.34 KN.m

Table 2.4-2: Portion of interior negative M_u in column strip.

$\alpha_{f1}\ell_2/\ell_1$	ℓ_2/ℓ_1		
	0.5	**1.0**	**2.0**
0	0.75	0.75	0.75
≥1.0	0.90	0.75	0.45

Note: Linear interpolations shall be made between values shown.

By using table 2.4-3:

M_{cs} = $M_{negative}$ * 0.75 = 206.77 * 0.75 = 158.08 KN.m

M_{ms} =$M_{negative}$ - M_{cs} = 206.77 – 158.08 = 48.69 KN.m

Table 2.4-3: Portion of positive M_u in column strip.

$\alpha_{f1}\ell_2/\ell_1$	ℓ_2/ℓ_1		
	0.5	**1.0**	**2.0**
0	0.60	0.60	0.60
≥1.0	0.90	0.75	0.45

Note: Linear interpolations shall be made between values shown.

By using table 2.4-4:

M_{cs} = $M_{positive}$ * 0.60 = 111.34 * 0.60 = 66.80 KN.m

M_{ms} =$M_{positive}$ - M_{cs} = 111.34 – 66.80 = 44.54 KN.m

2.5 Preliminary dimensions of beams: -

Concrete beam is a load-bearing unit that can be used to carry both horizontal and vertical loads. Known as reinforced concrete beams or reinforced cement concrete (RCC) beams.

According to the ACI 318-19, the preliminary dimensions were be chosen according to table 2.5-1.

Table 2.5-1: Two-way beams depth.

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 = 60,000 psi. For other cases, minimum h shall be modified in accordance with 9.3.1.1.1 through 9.3.1.1.3, as appropriate.

The longest spans are between columns [3D,3E or 4D,4E] according to ETABS gridlines, with length equal to 5.37 m, which has both ends continuous.

$\frac{l}{21} = \frac{5370}{21} = 255.71\ mm$ → Take: **h = 300 mm**

And for the width let's try $\frac{l}{20} = \frac{5370}{20} = 268.5\ mm$

→ Take: **b = 300 mm** [to fit with the column's width]

2.6 Preliminary dimensions of columns: -

A reinforced concrete column is a structural member designed to carry compressive loads and moment, composed of concrete with an embedded steel to provide reinforcement.

For design purposes, the columns are separated into two categories: short columns and slender columns.

The initial dimensions of the column are determined by using the tributary area method, taking the column with the biggest tributary area, and support the most slabs. Which will be the column H-10 on the roof as shown in figure 2.6-1.

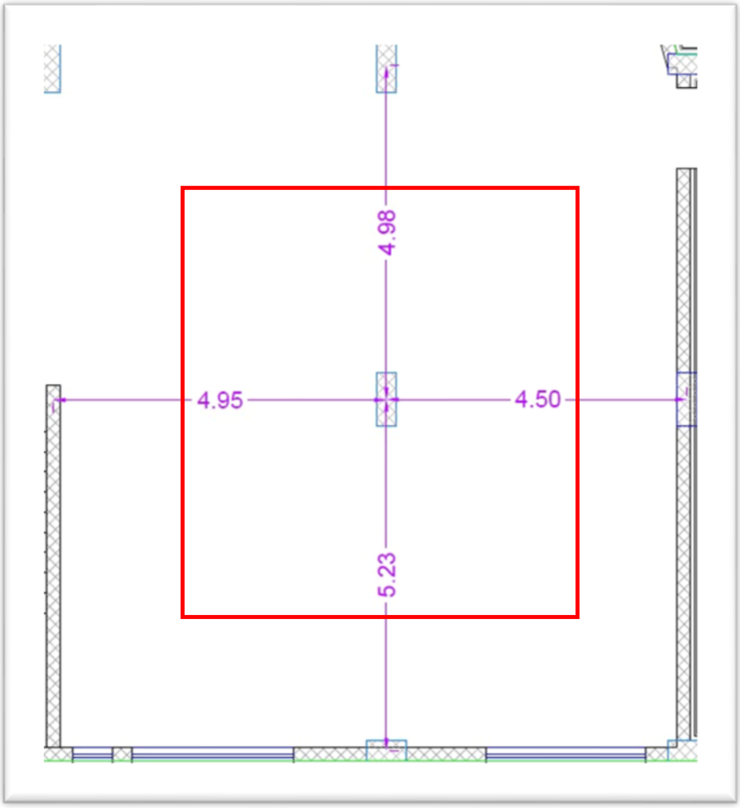


Figure 2.6-1: Column tributary area.

The tributary area = $\frac{5.23+4.98}{2} * \frac{4.50+4.95}{2}$ = **24.12 m^2**

D.L. = **Thickness of slab * ɣ**

S.D. = **4.00 KN/m^2**

The ultimate load on the ground floor = 1.2 (D.L.+S.D.) + 1.6 L.L. = 1.2 * (0.18 * 25 + 4) + 1.6 * 6 = **19.80 KN/m²**

The ultimate load on the residential floors = 1.2 (D.L.+S.D.) + 1.6 L.L. = 1.2 * (0.18 * 25 + 4) + 1.6 * 3 = **15.00 KN/m²**

The ultimate load on the roof = 1.2 (D.L.+S.D.) + 1.6 L.L. = 1.2 * (0.18 * 25 + 4) + 1.6 * 1.0 = **11.80 KN/m²**

The ultimate axial load on the column = Area * $\sum Loads$ = 24.12 * (19.80 * 1 + 15.00 * 6 + 11.80 * 1) = **2932.99 KN**

To consider the eccentricity and the column reduction factor: -

$\frac{2932.99}{0.65*0.8}$ = **5640.37KN = Pn_o**

$Pn_o = 0.85\ f'_c\ A_{Conc} + F_y\ A_s \rightarrow$ assuming $\rho = 0.02 \rightarrow A_s = 0.02\ A_g$

$5{,}640{,}370 = 0.85 * 28 * (A_g - 0.02A_g) + 420 * 0.02 * A_g \rightarrow A_g = 177{,}795.04\ mm^2$

Use **b = 300 mm** & **h = 700 mm→ A_g = 210,000.00 mm²**

2.7 Preliminary dimensions of walls: -

A shear wall is a general term for a wall that is designed and constructed to resist racking from forces such as wind using masonry, concrete, cold-formed steel, or wood framing. Shear walls significantly reduce the sway of a structure to reduce damage to the structure and its contents.

According to the ACI 318-19 the minimum thickness (t) required in the shear wall is shown in table 2.7-1.

Table 2.7-1: Wall minimum thickness, (t).

Wall type	Minimum thickness *h*		
Bearing[1]	Greater of:	4 in.	(a)
		1/25 the lesser of unsupported length and unsupported height	(b)
Nonbearing	Greater of:	4 in.	(c)
		1/30 the lesser of unsupported length and unsupported height	(d)
Exterior basement and foundation[1]	7.5 in.		(e)

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

The type of the shear wall is bearing. Hence, the required thickness is the greater of 4 inches (101.6 mm) or $\frac{4200-180}{25}$ = 160.8 mm.

→ Take: **t = 200 mm**

2.8 Assumptions of staircase: -

A staircase is a structural element, used for travelling from one story of a building to another. It is designed to cover a large vertical distance by using steps.

The staircase in ETABS will be represented as a 20 cm slab with loads, which will work as a staircase till the other project which we will design it in details.

Loads on staircase assumed slab: -

1. Live load = 5 kN/m^2
2. Super imposed dead load = 4 kN/m^2
3. Dead load.

3 THREE-DIMENSIONAL ANALYSIS AND DESIGN

3.1 General: -

This chapter will identify and illustrate some of the topics about the project, which includes: -

1) Structural modeling of the building: This section will illustrate and show the main points in structural modeling by using the software ETABS.
2) Evaluation of the preliminary design: This section will introduce and evaluate the analysis and design results for the preliminary dimensions of the elements, which may lead for some changes in the elements.
3) Verification of structural analysis: This section will make sure that all the internal forces in the structural members are nearly similar to the hand calculation.
4) Deflection computations: This section will check the slab deflection according to the code.
5) Verification of structural design.
6) Design of slabs.
7) Design of beams.
8) Design of columns.
9) Design of walls.
10) Design of footings.
11) Design of tie beams.
12) Design of stairs.
13) Design of diaphragms and collectors.
14) Analysis and design of non-structural walls.
15) Analysis and design of non-structural elements.

3.2 Structural modeling of the building:

Structural modeling is one of the prime phases of the project. It is basically a mathematical or a wireframe representation of a 3D model. It helps to visualize each corner of a virtual building structure, helps in analyzing the building and designing it.

The program which will be used in this project is ETABS, which can be defined as an engineering software product that is used in analysis and design of multi-story buildings.

Several points will be shown in this section as pictures to describe what has been used in this software to make the model as it’s. This section is detailed for educational purposes.

- Units: Figures 3.2-1 to 3.2-3 show the units used in modeling the building.

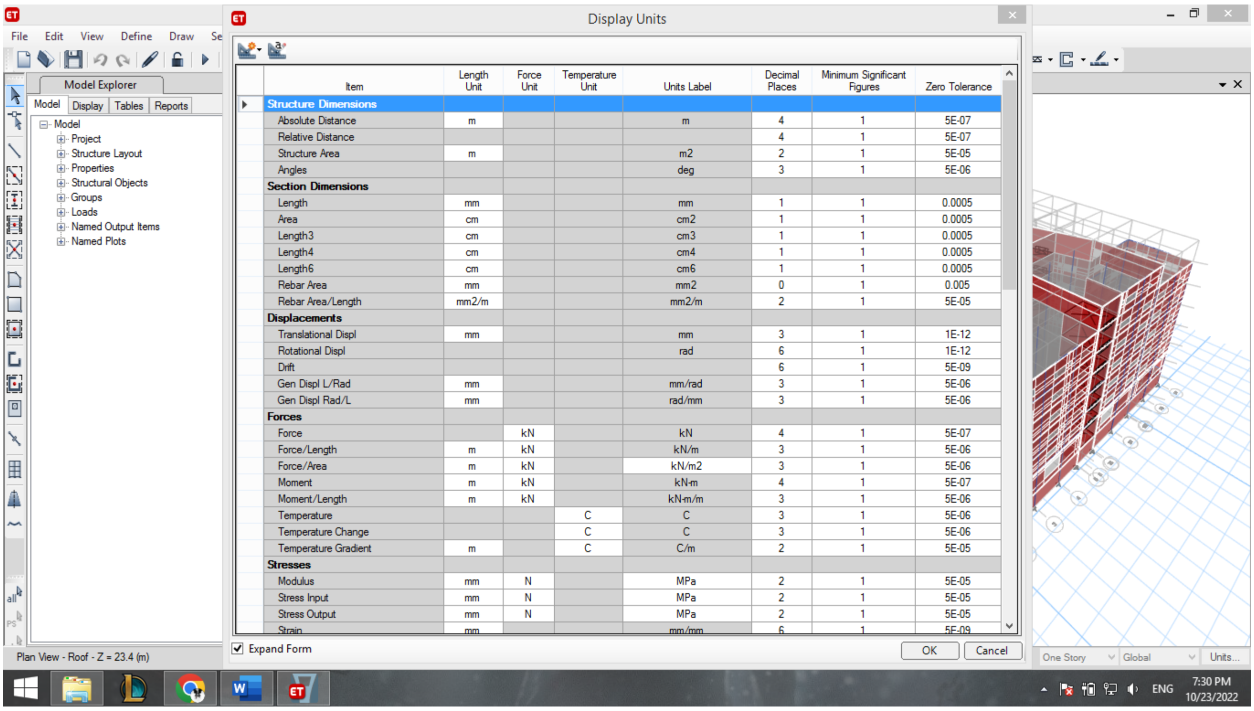
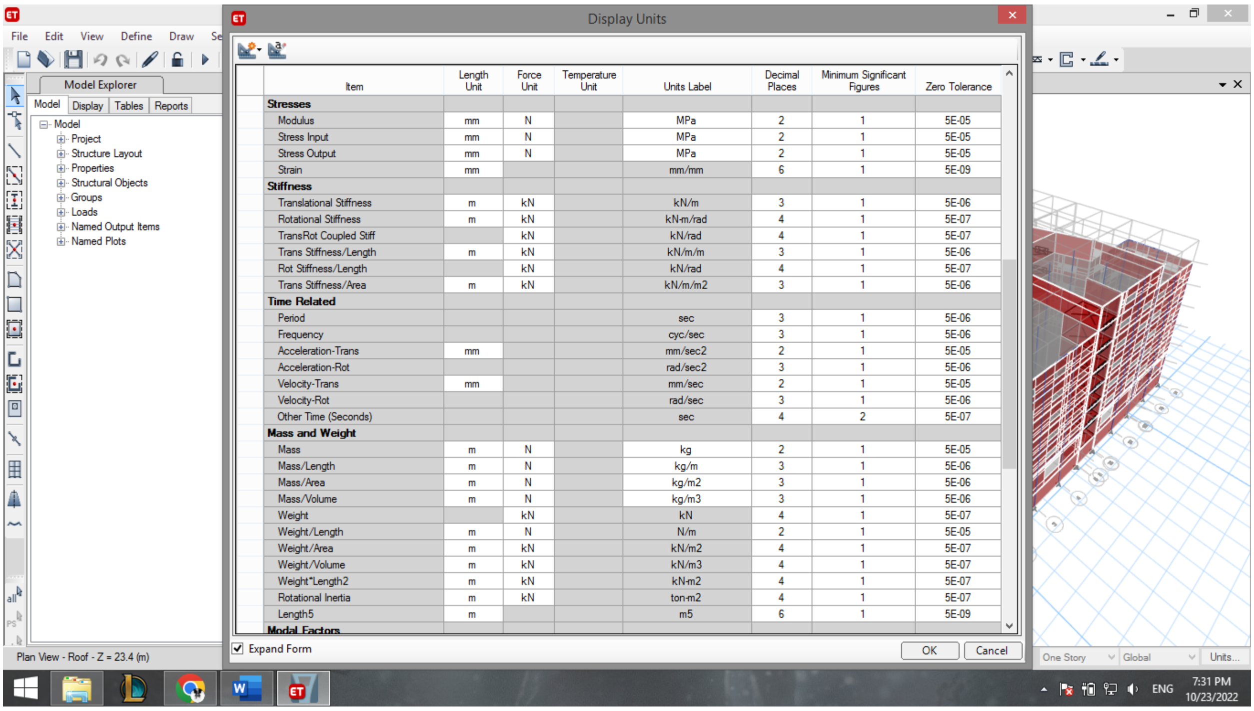


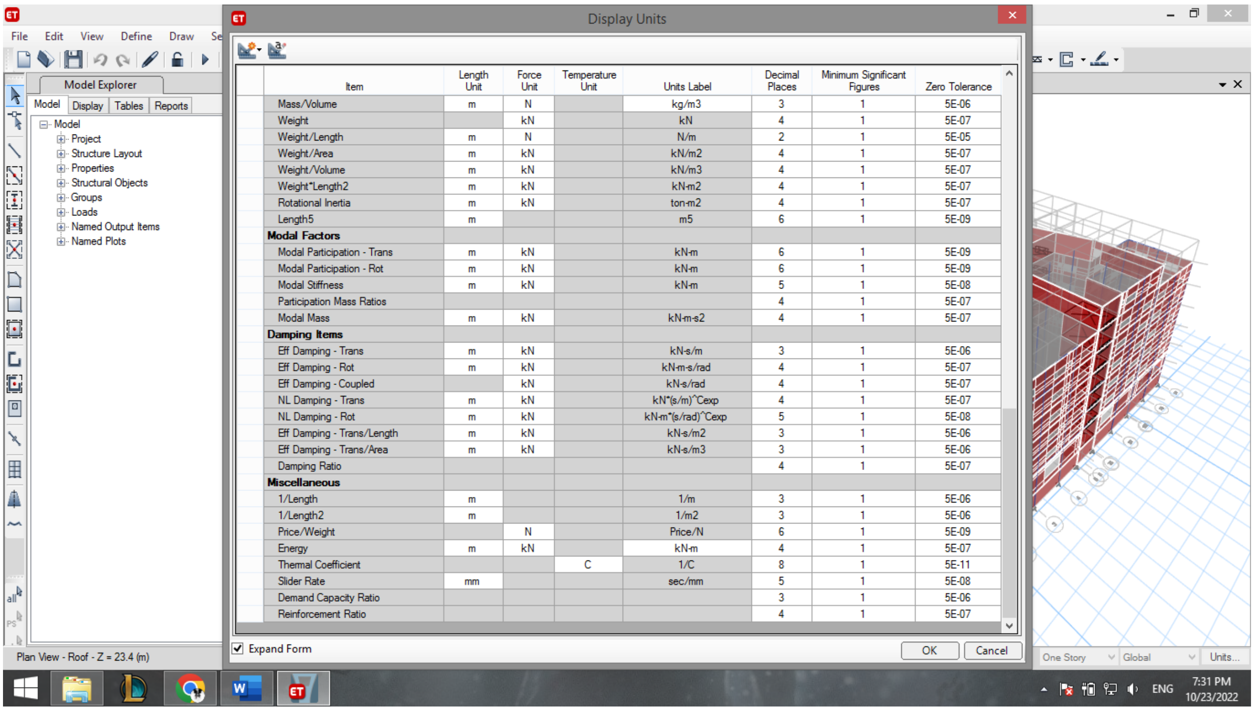
Figure 3.2-1: Program units 1.



Display Units

Item	Length Unit	Force Unit	Temperature Unit	Units Label	Decimal Places	Minimum Significant Figures	Zero Tolerance
Stresses							
Modulus	mm	N		MPa	2	1	5E-05
Stress Input	mm	N		MPa	2	1	5E-05
Stress Output	mm	N		MPa	2	1	5E-05
Strain	mm			mm/mm	6	1	5E-09
Stiffness							
Translational Stiffness	m	kN		kN/m	3	1	5E-06
Rotational Stiffness	m	kN		kN-m/rad	4	1	5E-07
TransRot Coupled Stiff		kN		kN/rad	4	1	5E-07
Trans Stiffness/Length	m	kN		kN/m/m	3	1	5E-06
Rot Stiffness/Length		kN		kN/rad	4	1	5E-07
Trans Stiffness/Area	m	kN		kN/m/m2	3	1	5E-06
Time Related							
Period				sec	3	1	5E-06
Frequency				cyc/sec	3	1	5E-06
Acceleration-Trans	mm			mm/sec2	2	1	5E-05
Acceleration-Rot				rad/sec2	3	1	5E-06
Velocity-Trans	mm			mm/sec	2	1	5E-05
Velocity-Rot				rad/sec	3	1	5E-06
Other Time (Seconds)				sec	4	2	5E-07
Mass and Weight							
Mass	m	N		kg	2	1	5E-05
Mass/Length	m	N		kg/m	3	1	5E-06
Mass/Area	m	N		kg/m2	3	1	5E-06
Mass/Volume	m	N		kg/m3	3	1	5E-06
Weight		kN		kN	4	1	5E-07
Weight/Length	m	N		N/m	2	1	5E-05
Weight/Area	m	kN		kN/m2	4	1	5E-07
Weight/Volume	m	kN		kN/m3	4	1	5E-07
Weight*Length2	m	kN		kN-m2	4	1	5E-07
Rotational Inertia	m	kN		ton-m2	4	1	5E-07
Length5	m			m5	6	1	5E-09
Modal Factors							

Figure 3.2-2: Program units 2.



Display Units

Item	Length Unit	Force Unit	Temperature Unit	Units Label	Decimal Places	Minimum Significant Figures	Zero Tolerance
Mass/Volume	m	N		kg/m3	3	1	5E-06
Weight		kN		kN	4	1	5E-07
Weight/Length	m	N		N/m	2	1	5E-05
Weight/Area	m	kN		kN/m2	4	1	5E-07
Weight/Volume	m	kN		kN/m3	4	1	5E-07
Weight*Length2	m	kN		kN-m2	4	1	5E-07
Rotational Inertia	m	kN		ton-m2	4	1	5E-07
Length5	m			m5	6	1	5E-09
Modal Factors							
Modal Participation - Trans	m	kN		kN-m	6	1	5E-09
Modal Participation - Rot	m	kN		kN-m	6	1	5E-09
Modal Stiffness	m	kN		kN-m	5	1	5E-08
Participation Mass Ratios					4	1	5E-07
Modal Mass	m	kN		kN-m-s2	4	1	5E-07
Damping Items							
Eff Damping - Trans	m	kN		kN-s/m	3	1	5E-06
Eff Damping - Rot	m	kN		kN-m-s/rad	4	1	5E-07
Eff Damping - Coupled		kN		kN-s/rad	4	1	5E-07
NL Damping - Trans	m	kN		kN*(s/m)^Cexp	4	1	5E-07
NL Damping - Rot	m	kN		kN-m*(s/rad)^Cexp	5	1	5E-08
Eff Damping - Trans/Length	m	kN		kN-s/m2	3	1	5E-06
Eff Damping - Trans/Area	m	kN		kN-s/m3	3	1	5E-06
Damping Ratio					4	1	5E-07
Miscellaneous							
1/Length	m			1/m	3	1	5E-06
1/Length2	m			1/m2	3	1	5E-06
Price/Weight		N		Price/N	6	1	5E-09
Energy	m	kN		kN-m	4	1	5E-07
Thermal Coefficient			C	1/C	8	1	5E-11
Slider Rate	mm			sec/mm	5	1	5E-08
Demand Capacity Ratio					3	1	5E-06
Reinforcement Ratio					4	1	5E-07

Figure 3.2-3: Program units 3.

- Gridlines: Figure 3.2-4 shows the gridlines used in modeling the building.

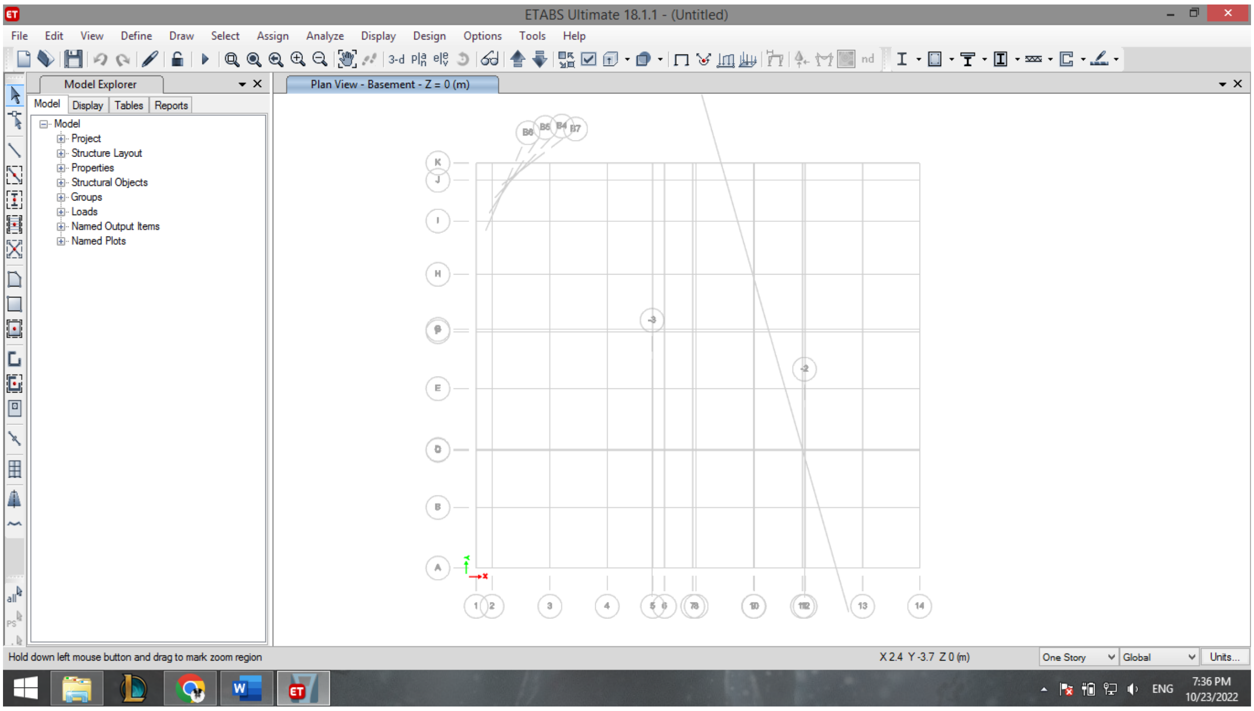


Figure 3.2-4: Gridlines.

- Materials: Figures 3.2-5 and 3.2-6 show the materials used in modeling the building.

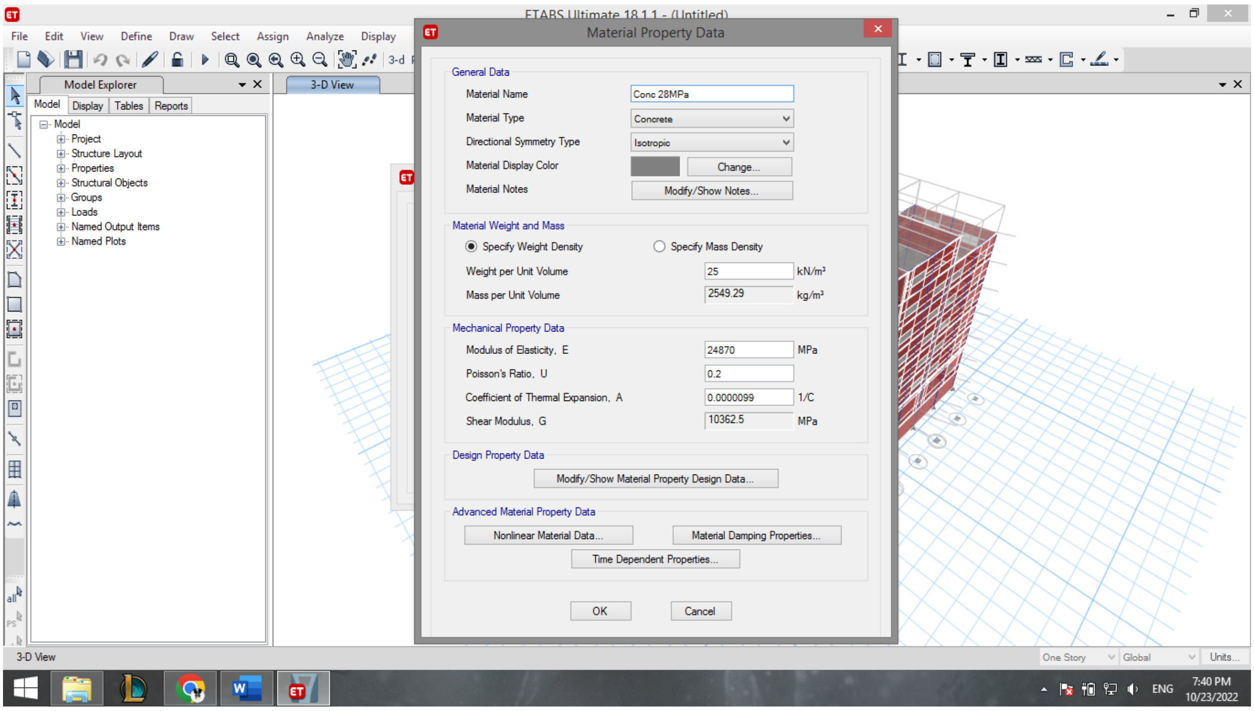


Figure 3.2-5: Concrete (Materials).

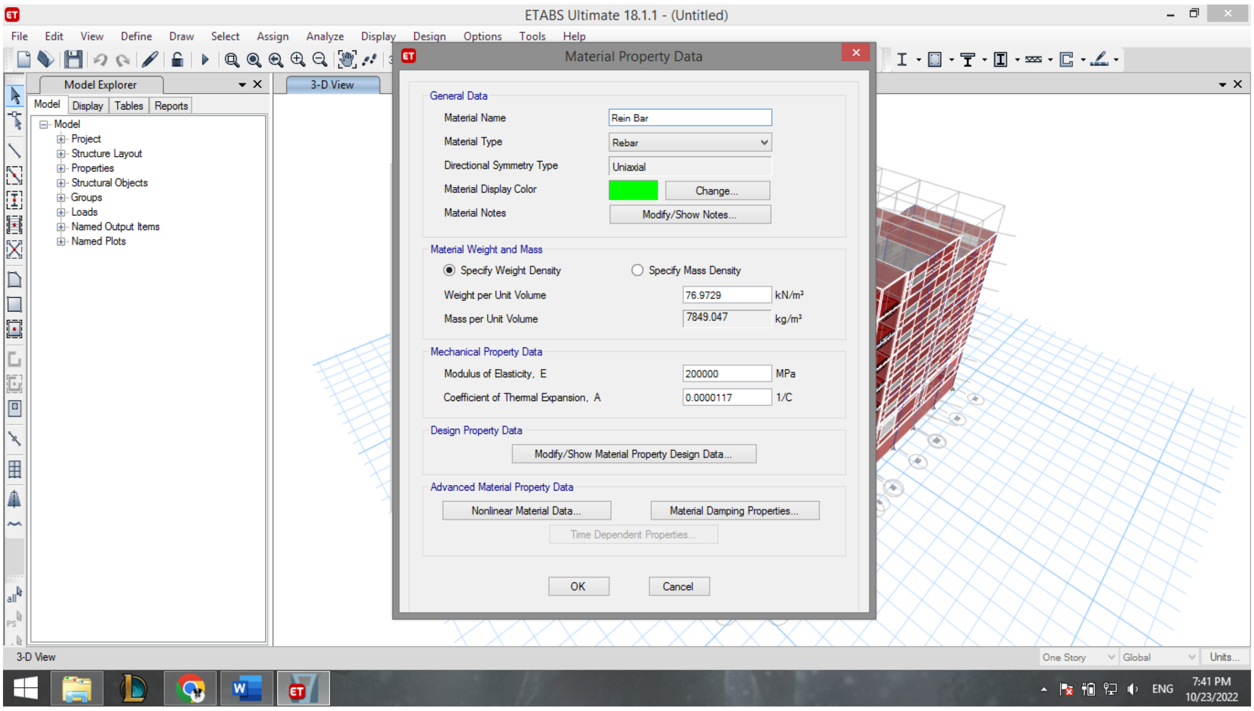


Figure 3.2-6: Steel Reinforcement (Materials).

- Frame properties: Figures 3.2-7 to 3.2-10 show the frame properties used in modeling the building.

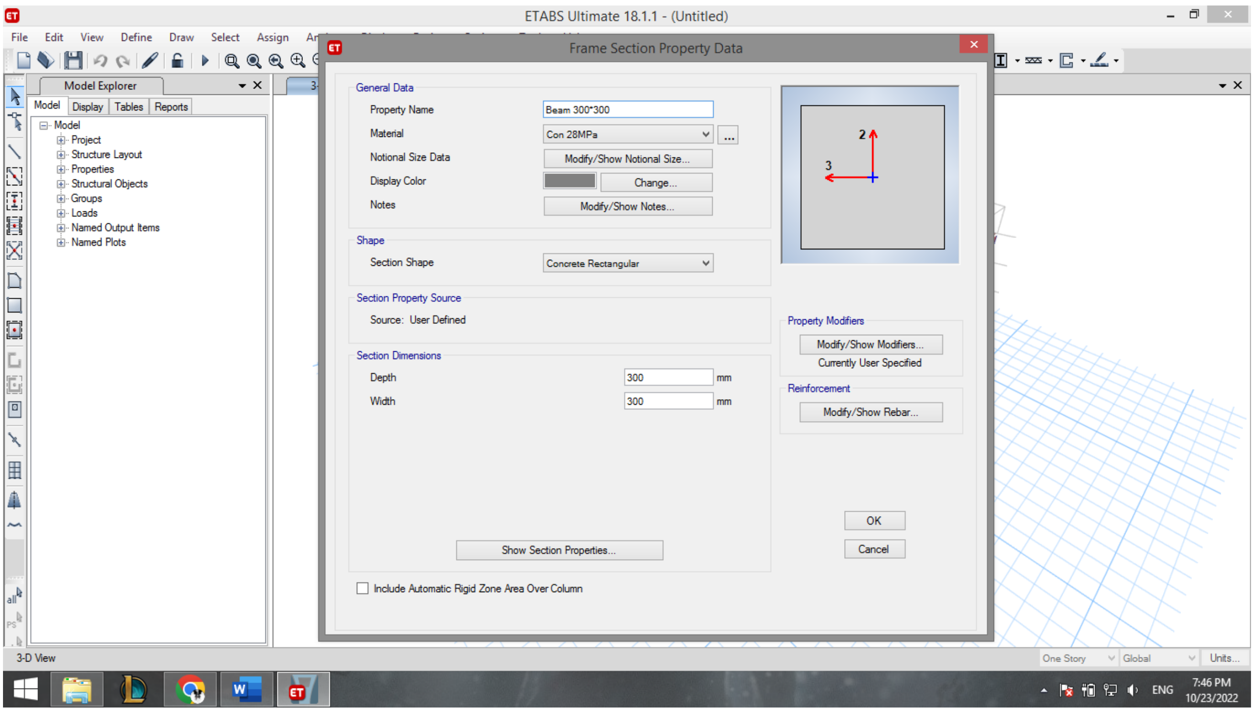


Figure 3.2-7: Beam section (Frame properties).

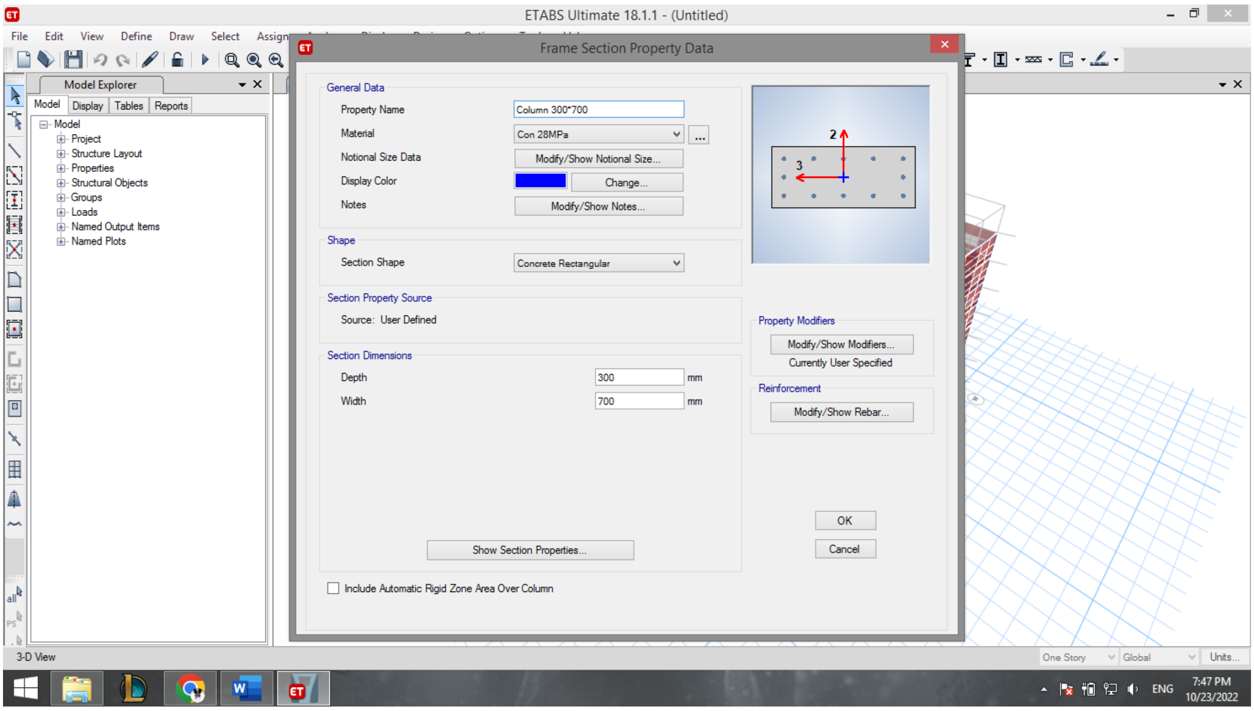


Figure 3.2-8: Column section (Frame properties).

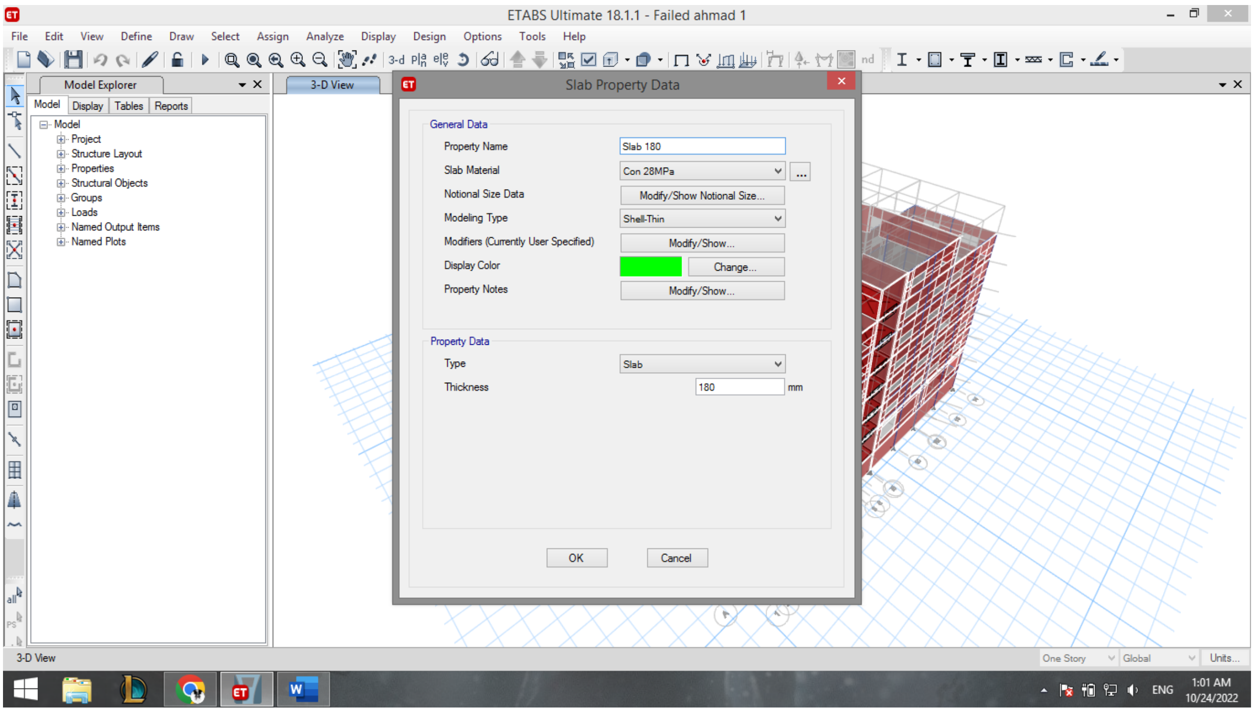


Figure 3.2-9: Slab property (Frame properties).

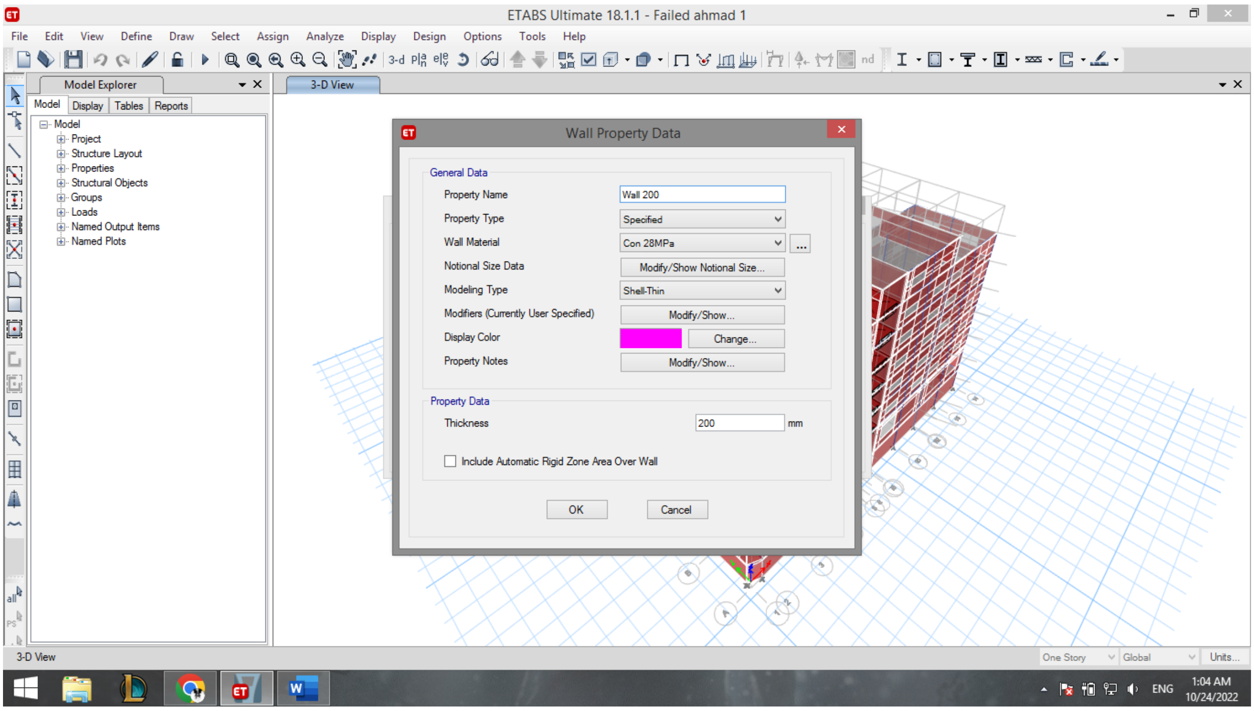


Figure 3.2-10: Wall property (Frame properties).

- Section modifiers: Figures 3.2-11 to 3.2-14 show the sections modifiers used in modeling the building.

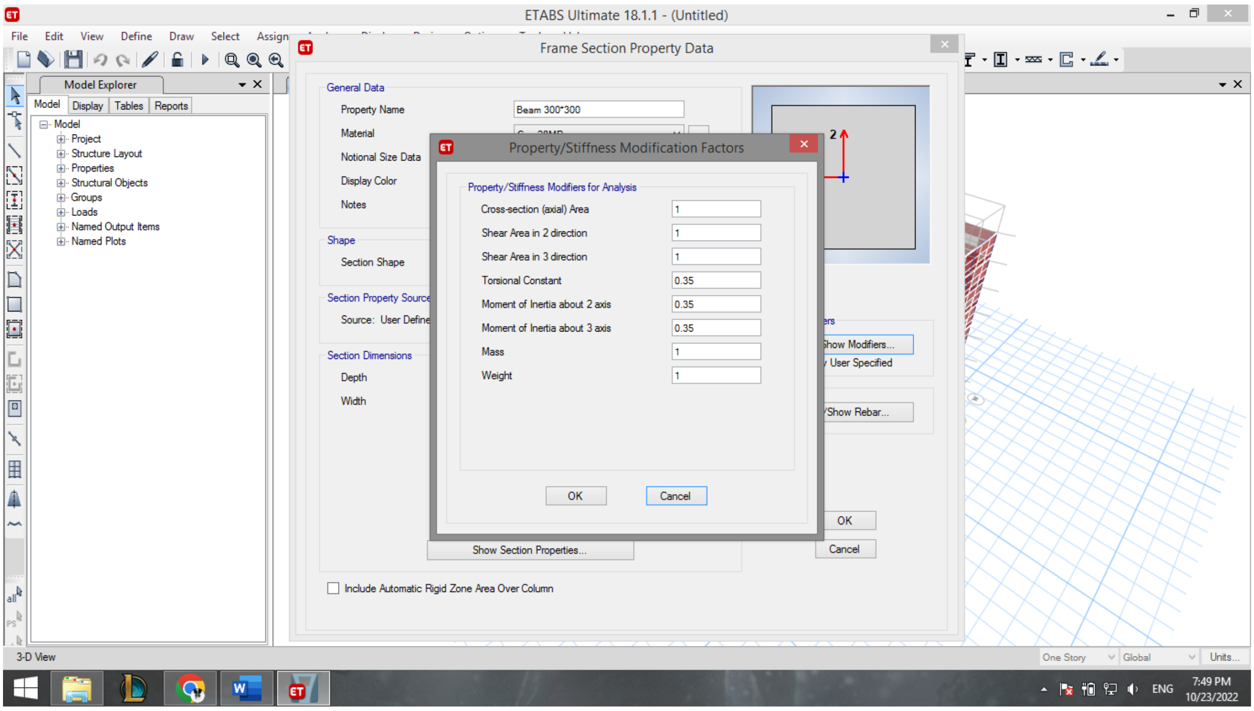


Figure 3.2-11: Beam modifiers.

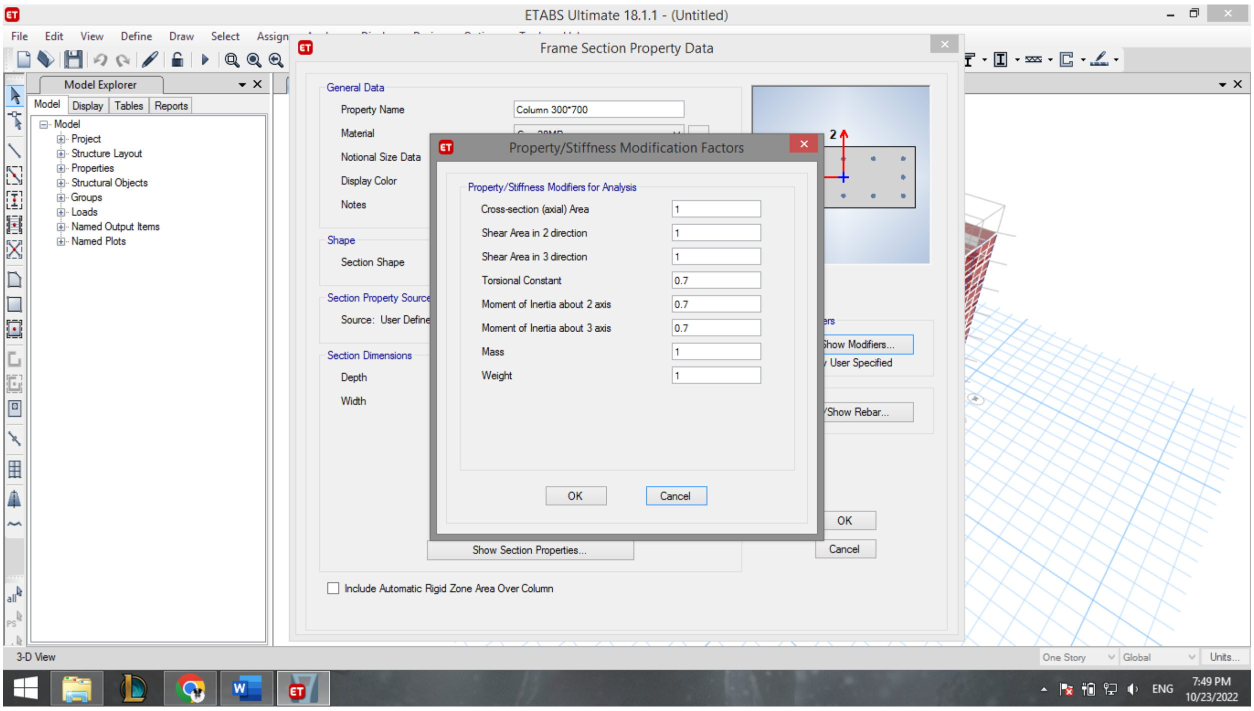


Figure 3.2-12: Column modifiers.

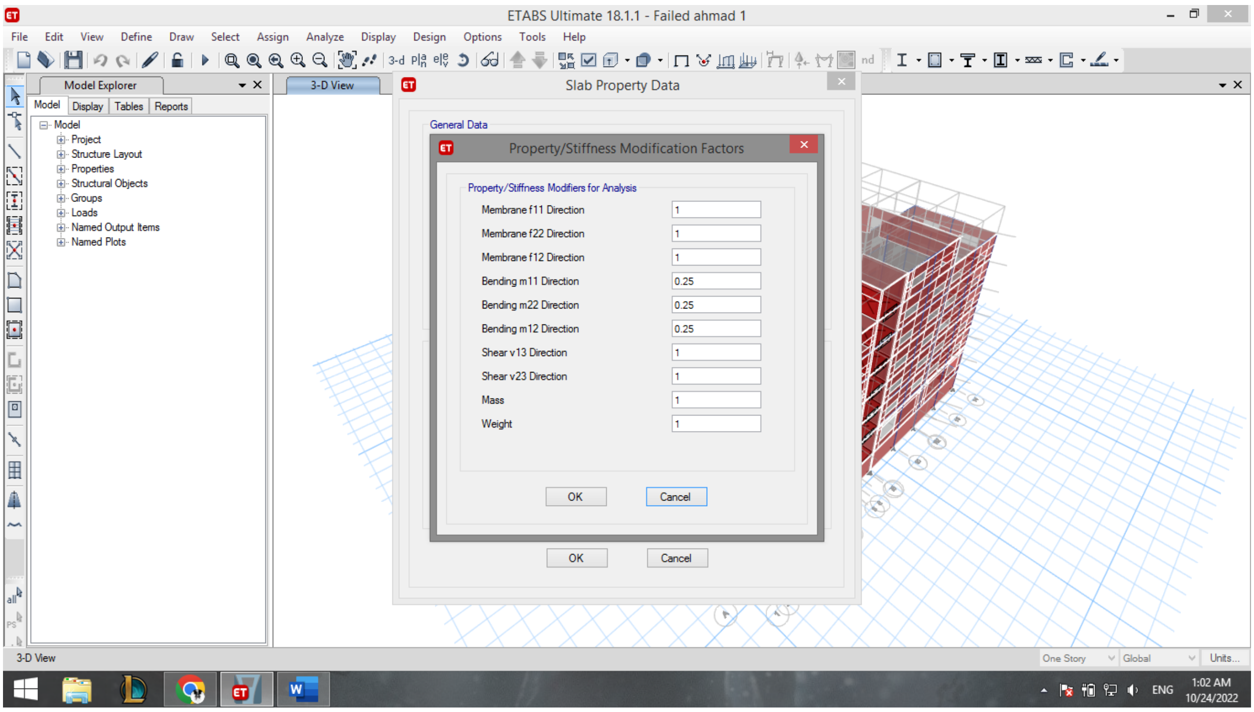


Figure 3.2-13: Slab modifiers.

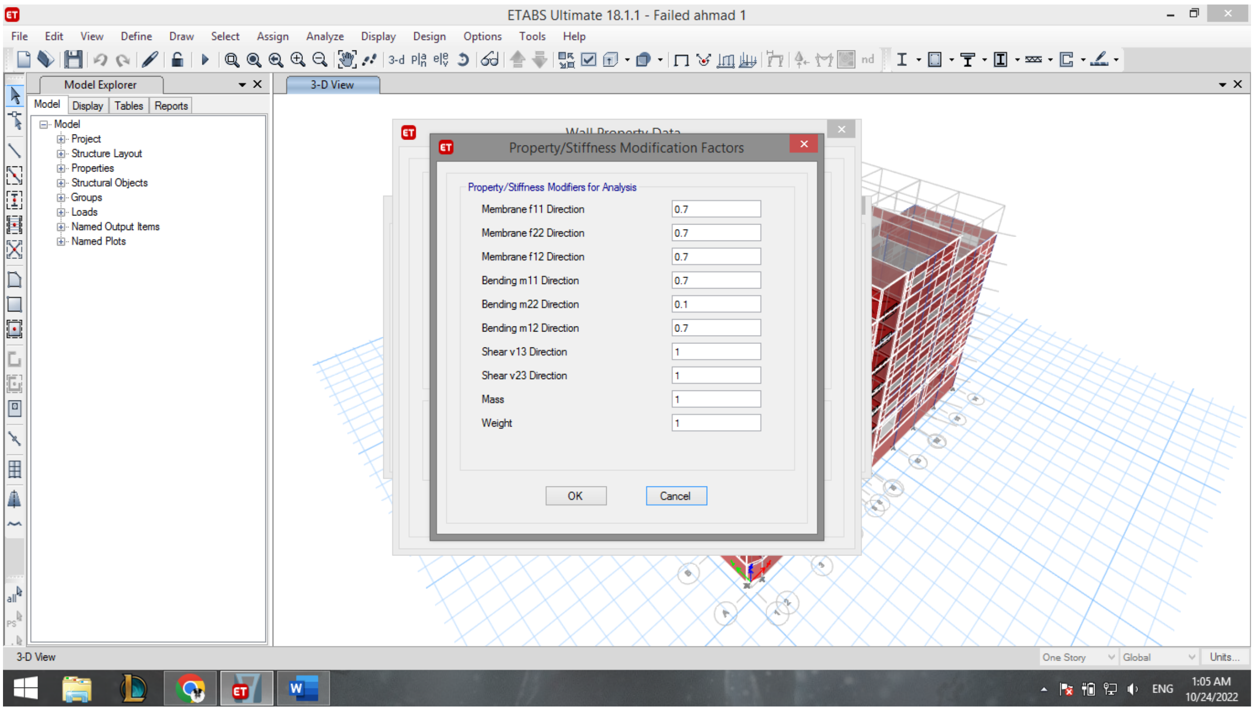


Figure 3.2-14: Wall modifiers.

- Load definitions: Figure 3.2-15 shows the load definitions used in modeling the building.

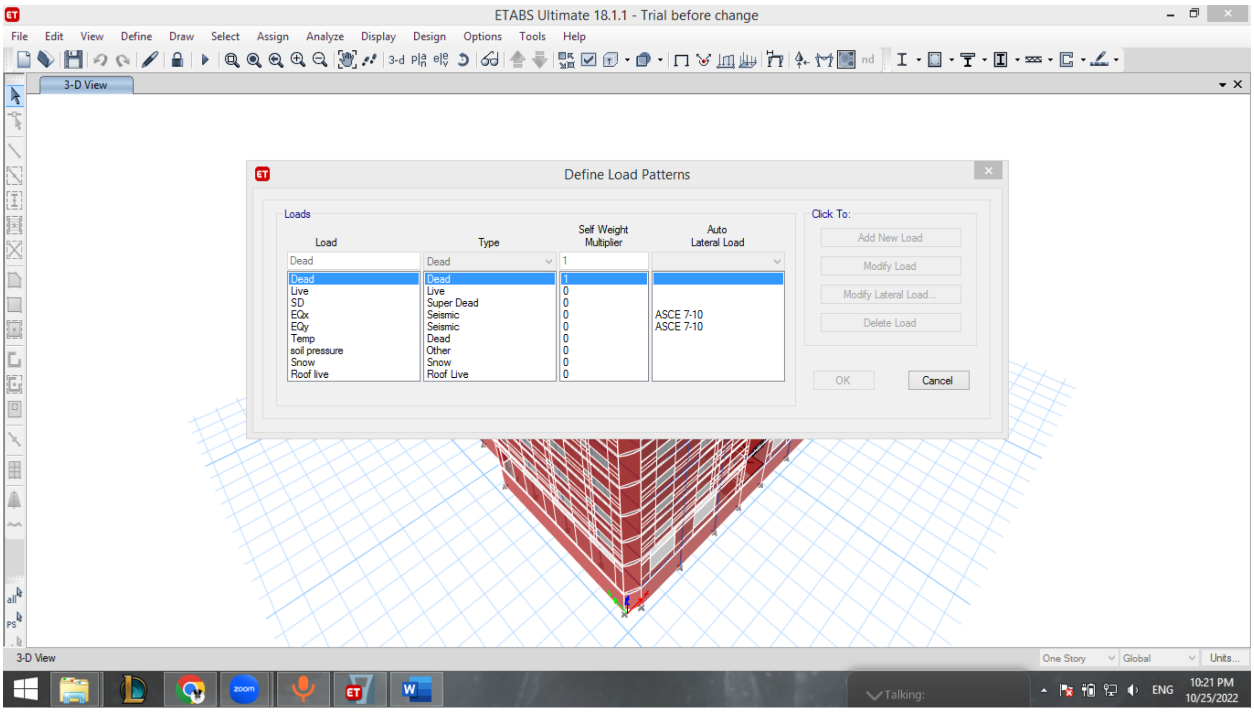


Figure 3.2-15: Load definitions.

- Load combinations: Figures 3.2-16 and 3.2-17 show the load combinations used in modeling the building.

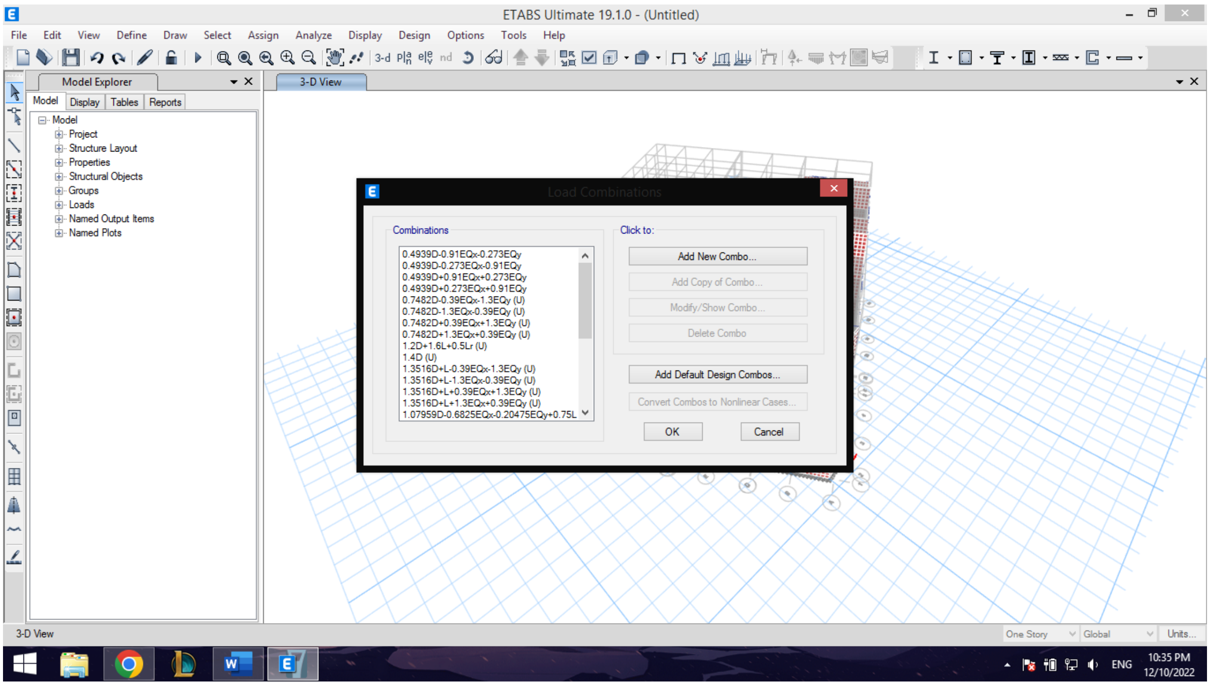


Figure 3.2-16: Load combinations 1.

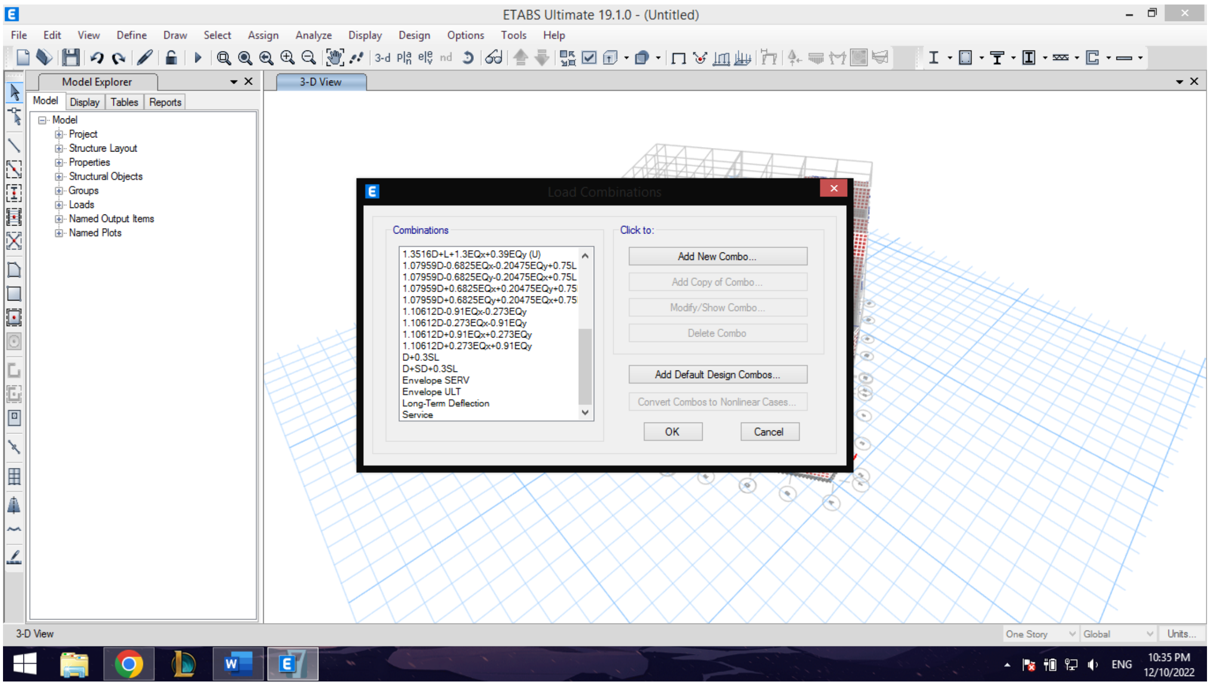


Figure 3.2-17: Load combinations 2.

- Load cases: Figure 3.2-18 shows the load cases used in modeling the building.

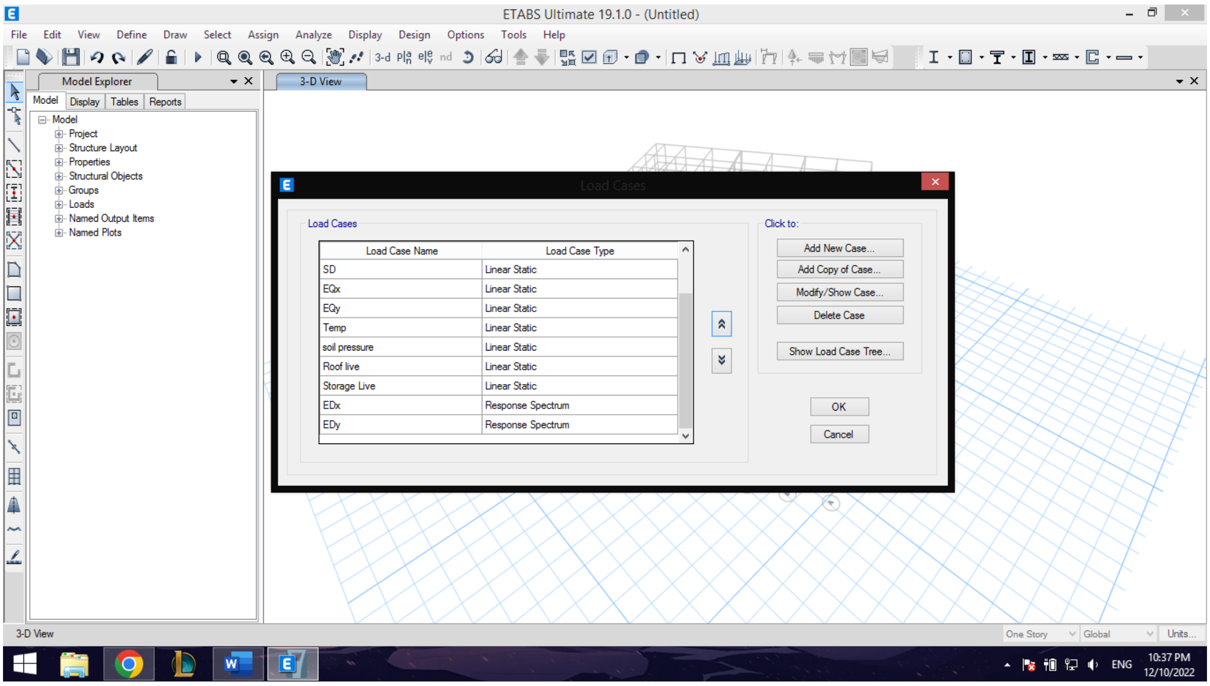


Figure 3.2-18: Load cases.

- Loads assignments: Figures 3.2-19 to 3.2-23 show the load cases used in modeling the building.

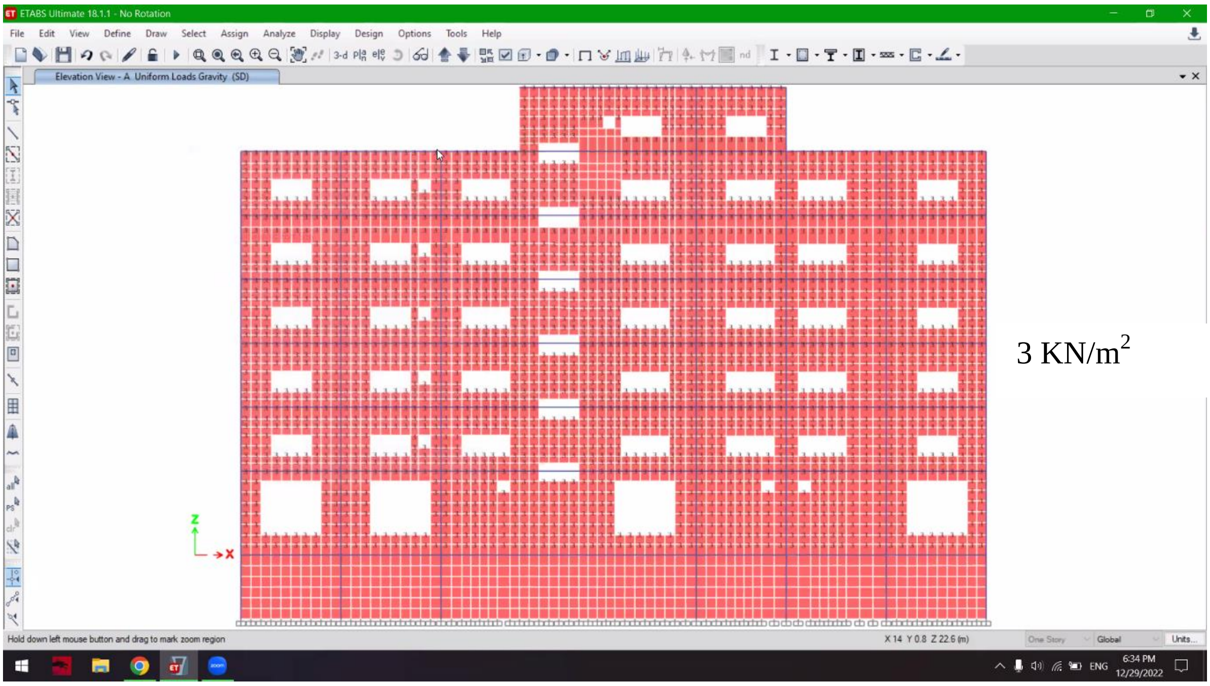


Figure 3.2-19: SD on walls.

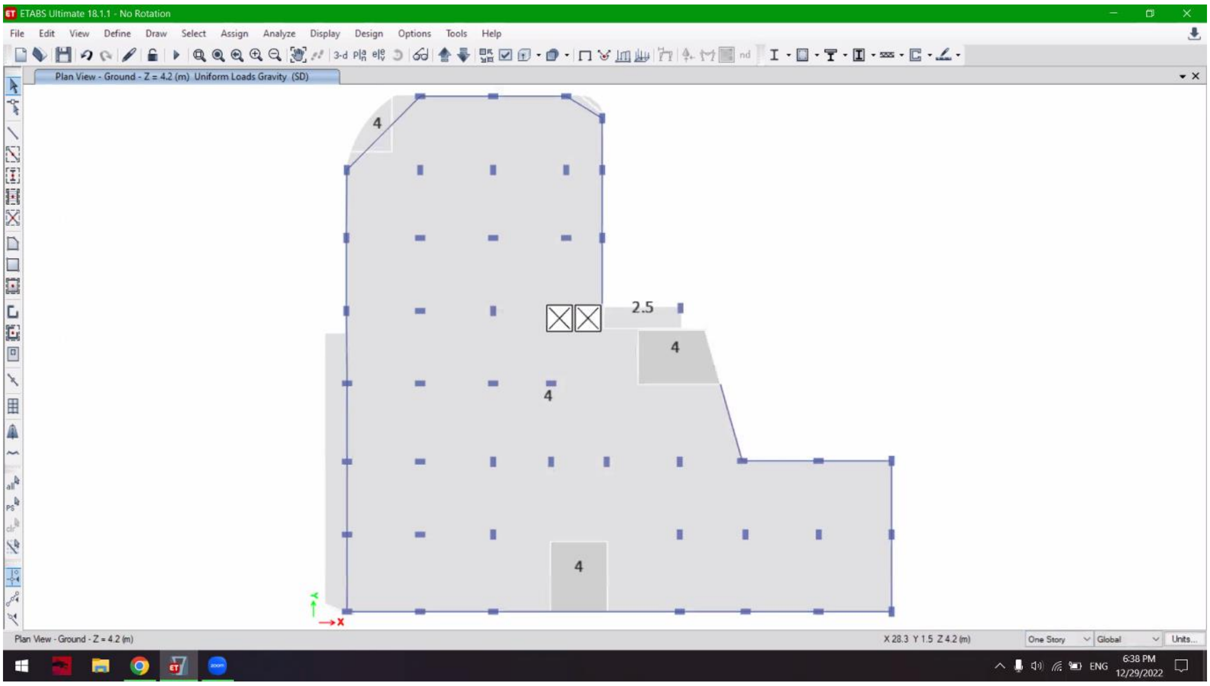


Figure 3.2-20: SD on 1st floor.

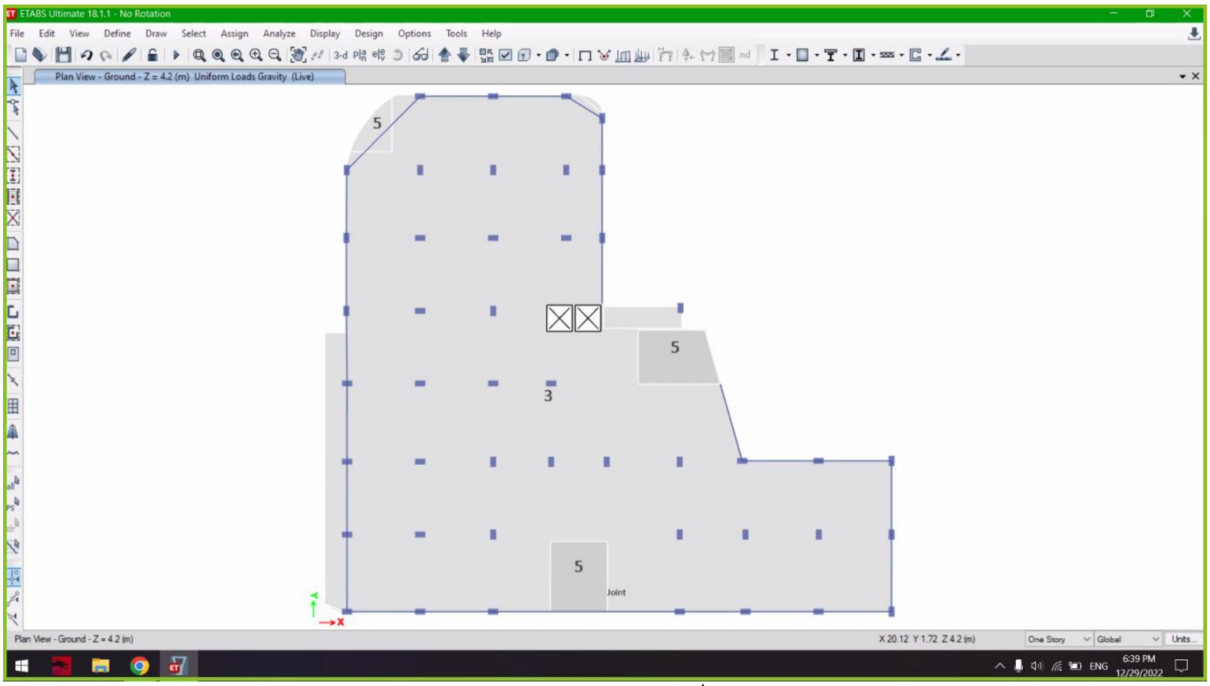


Figure 3.2-21: LL on 1st floor.

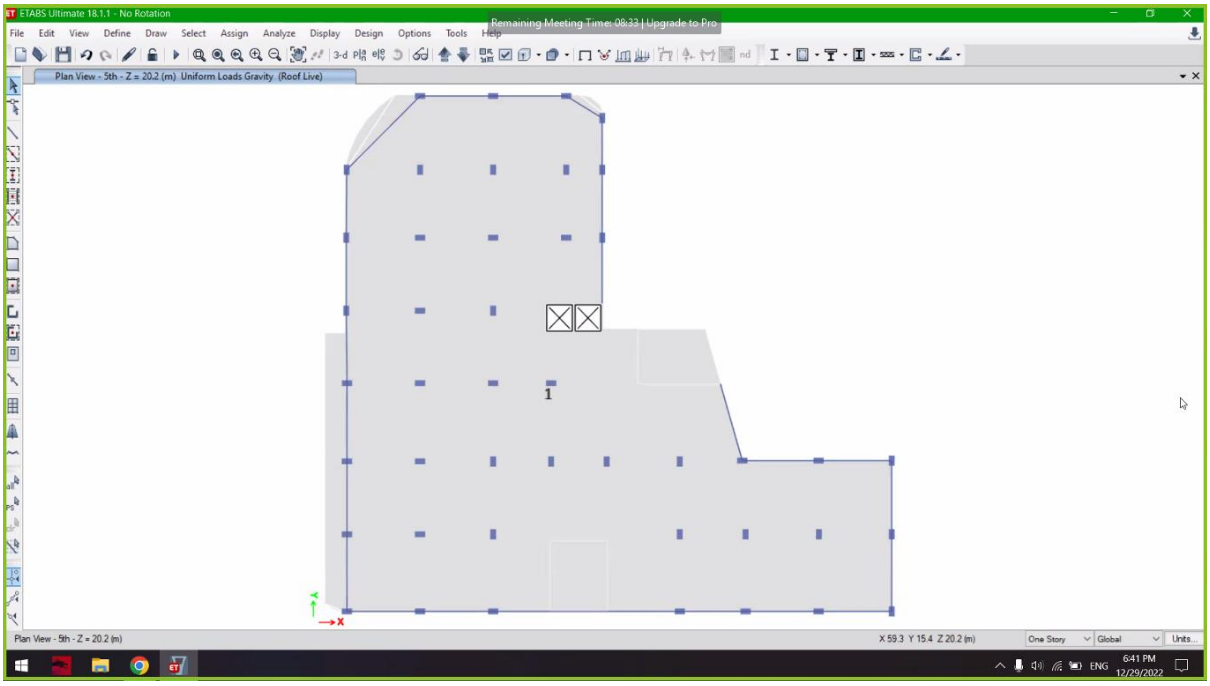


Figure 3.2-22: Roof load on the roof floor.

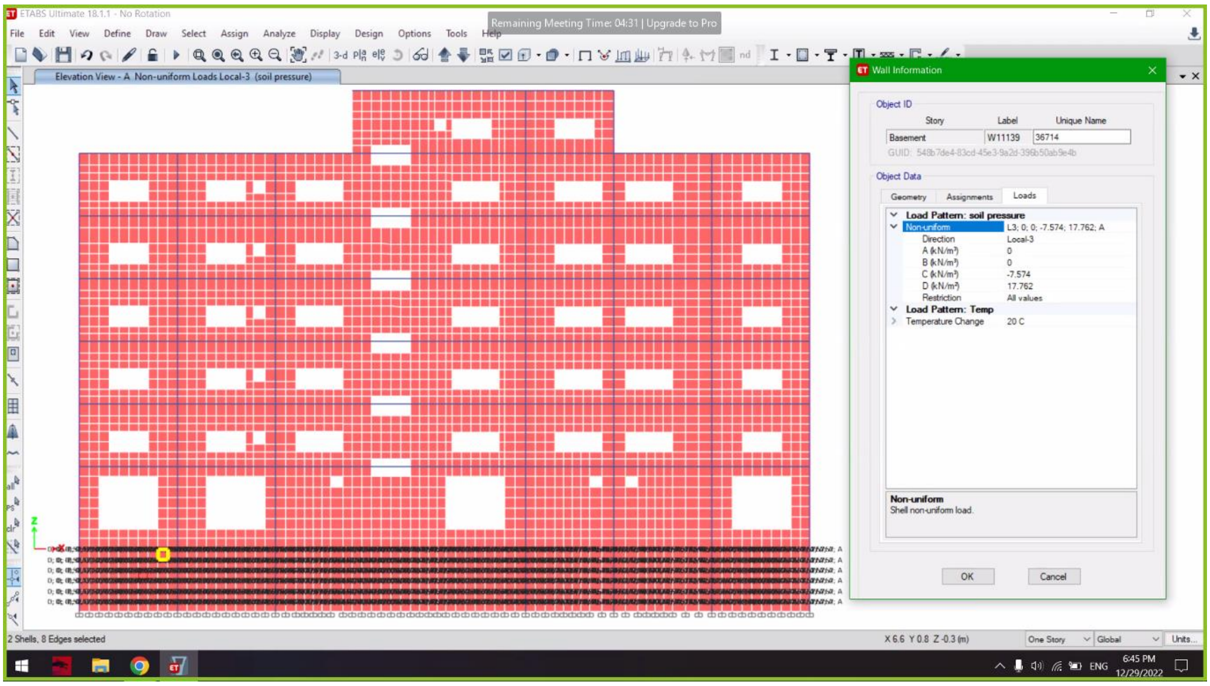


Figure 3.2-23: Soil pressure on Basement.

- Masses: Figure 3.2-24 shows the masses used in modeling the building.

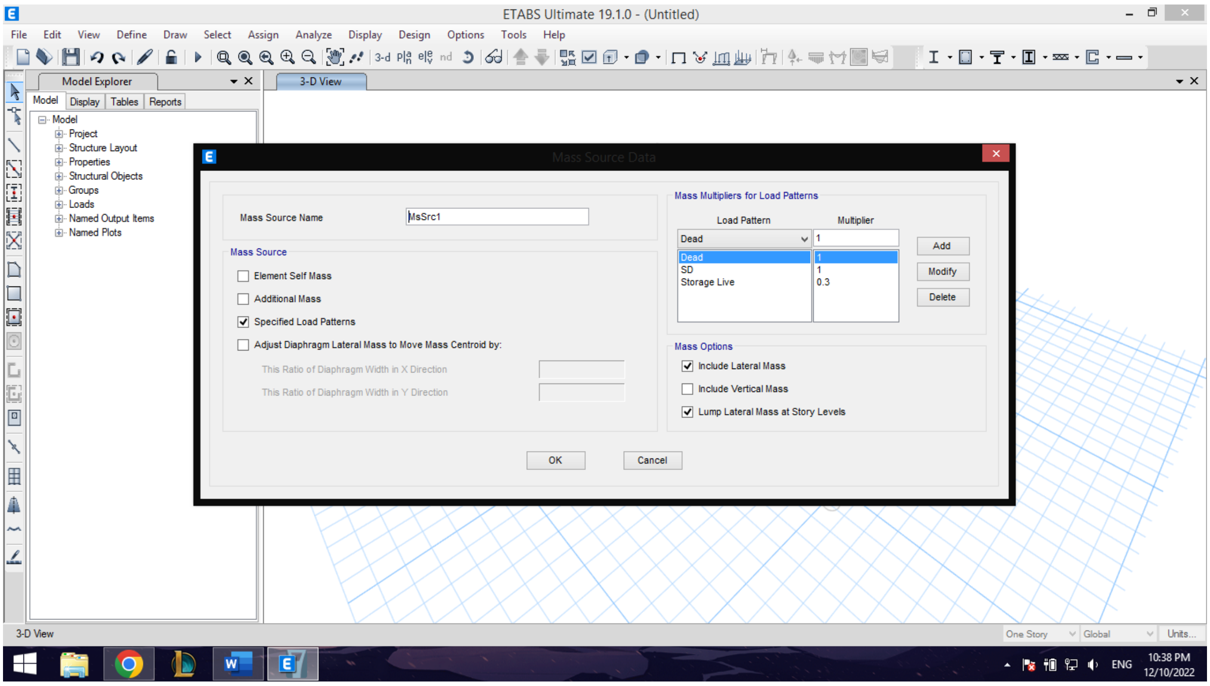


Figure 3.2-24: Mass Source.

- Functions: Figure 3.2-25 shows the function used in modeling the building.

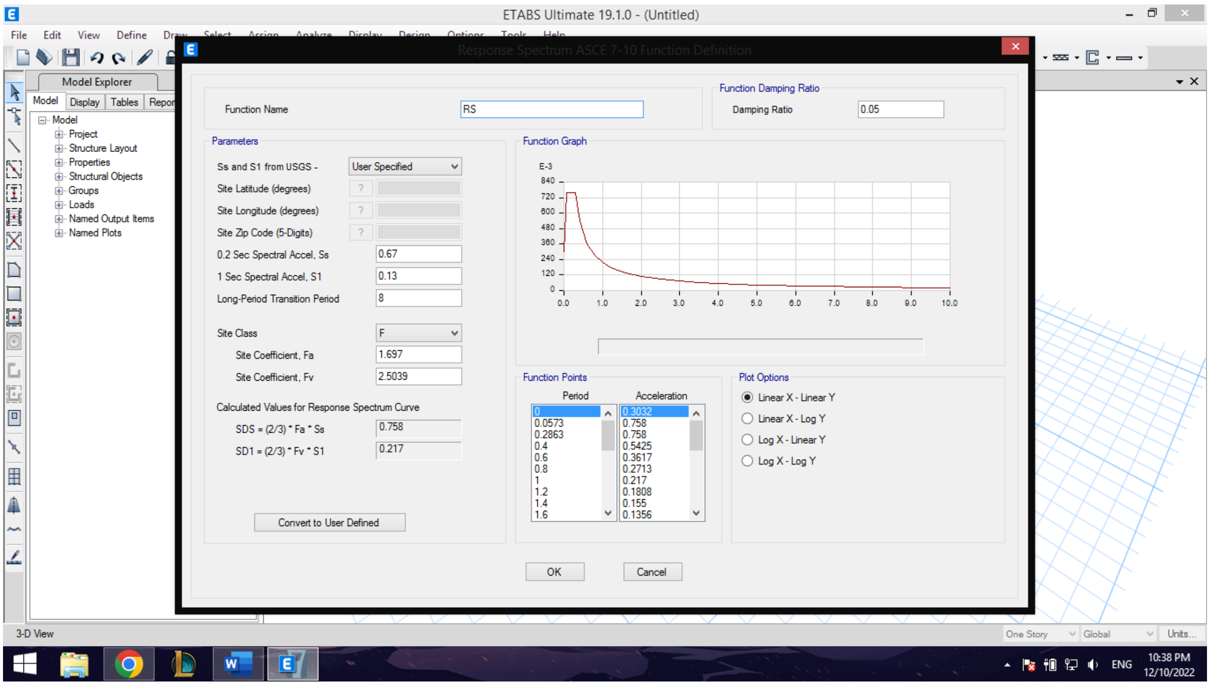


Figure 3.2-25: Response spectrum function.

- Supports: Figures 3.2-26 and 3.2-27 show the supports (Fixed) used in modeling the building.

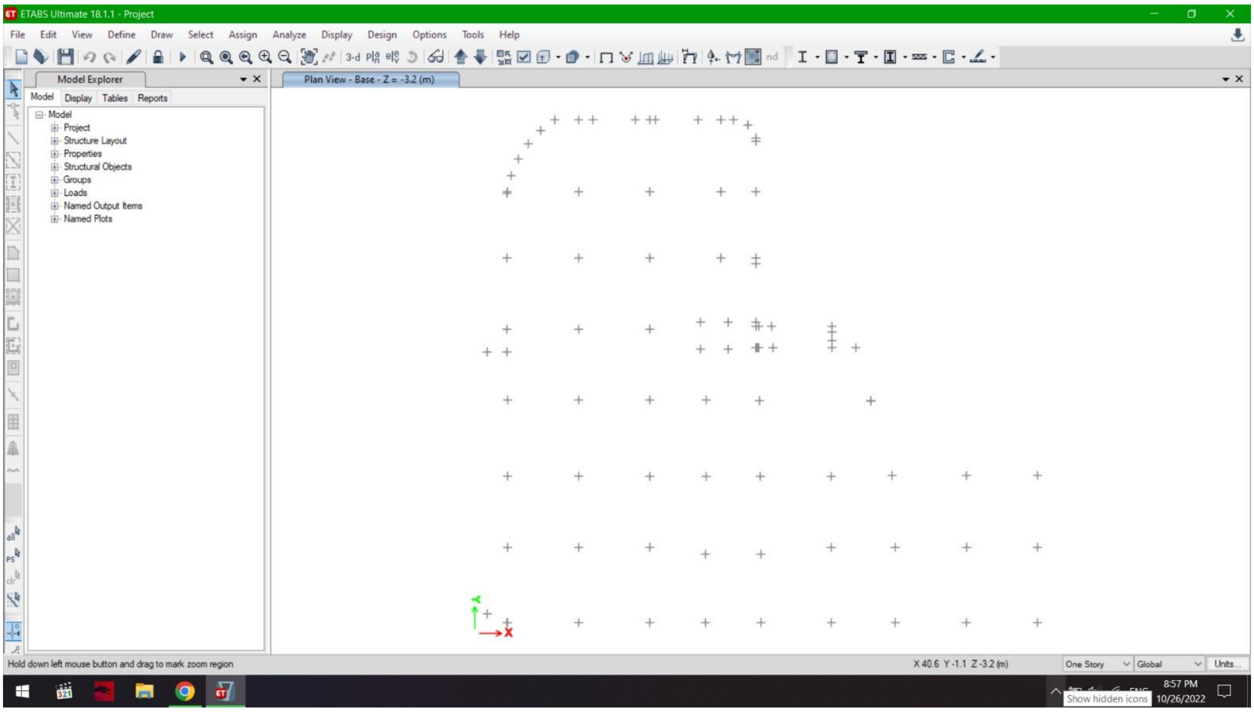


Figure 3.2-26: Supports layout.

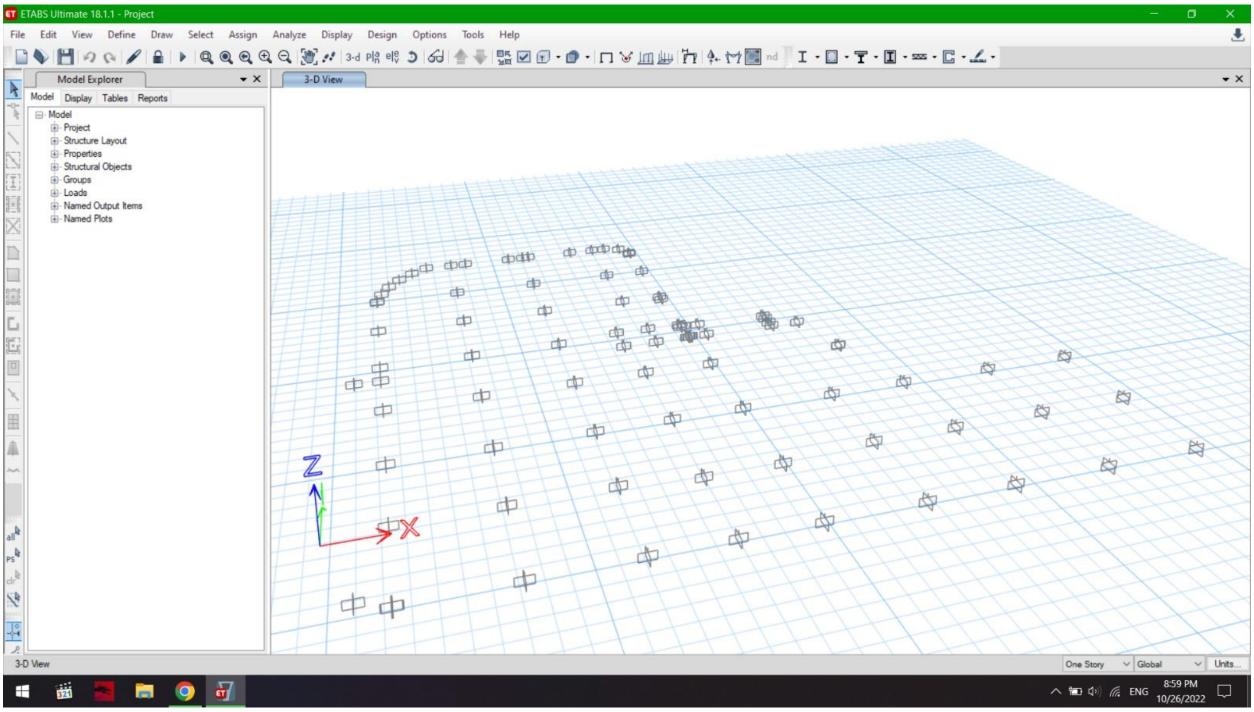


Figure 3.2-27: Supports 3D layout.

- Codes: Figures 3.2-28 to 3.2-30 show the codes used in modeling the building.

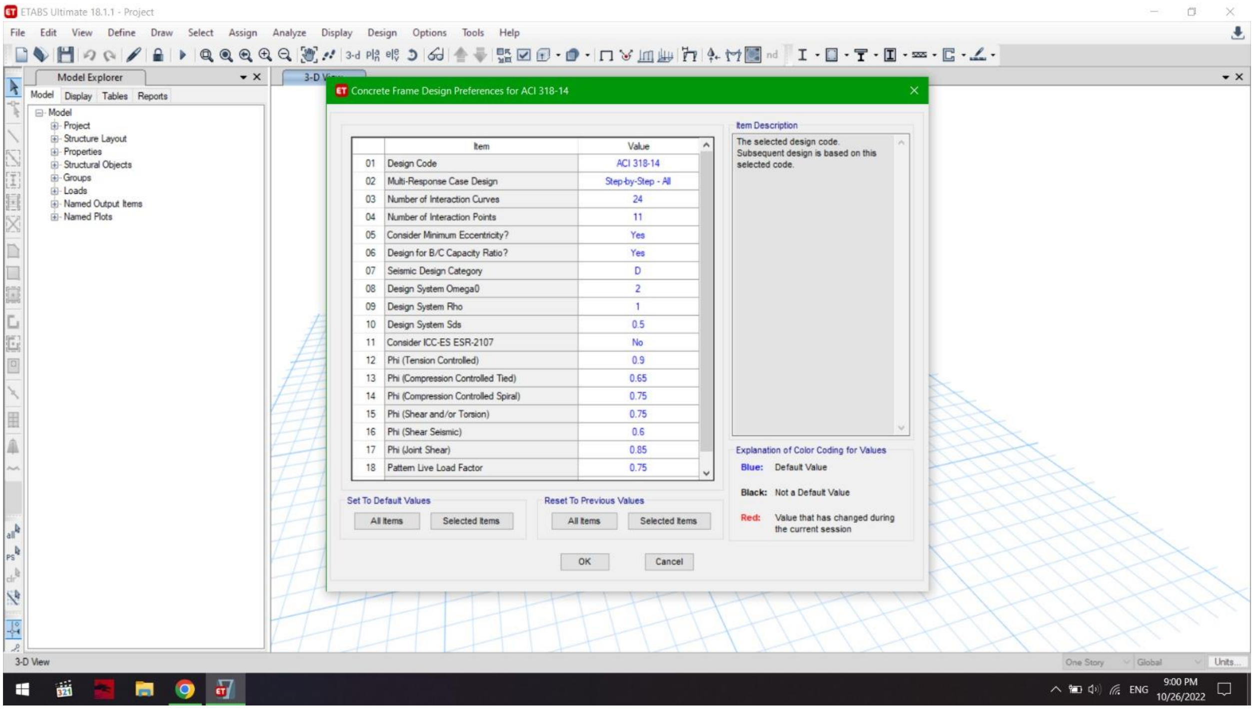


Figure 3.2-28: Frame design code.

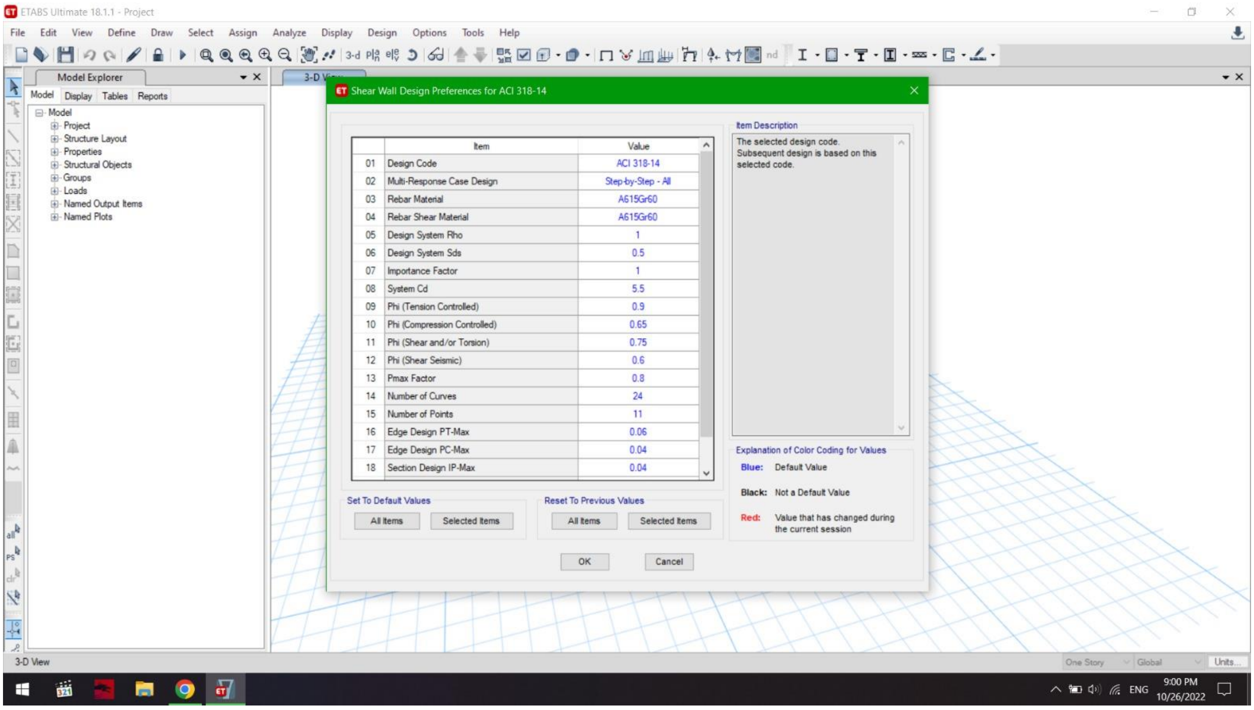


Figure 3.2-29: Wall design code.

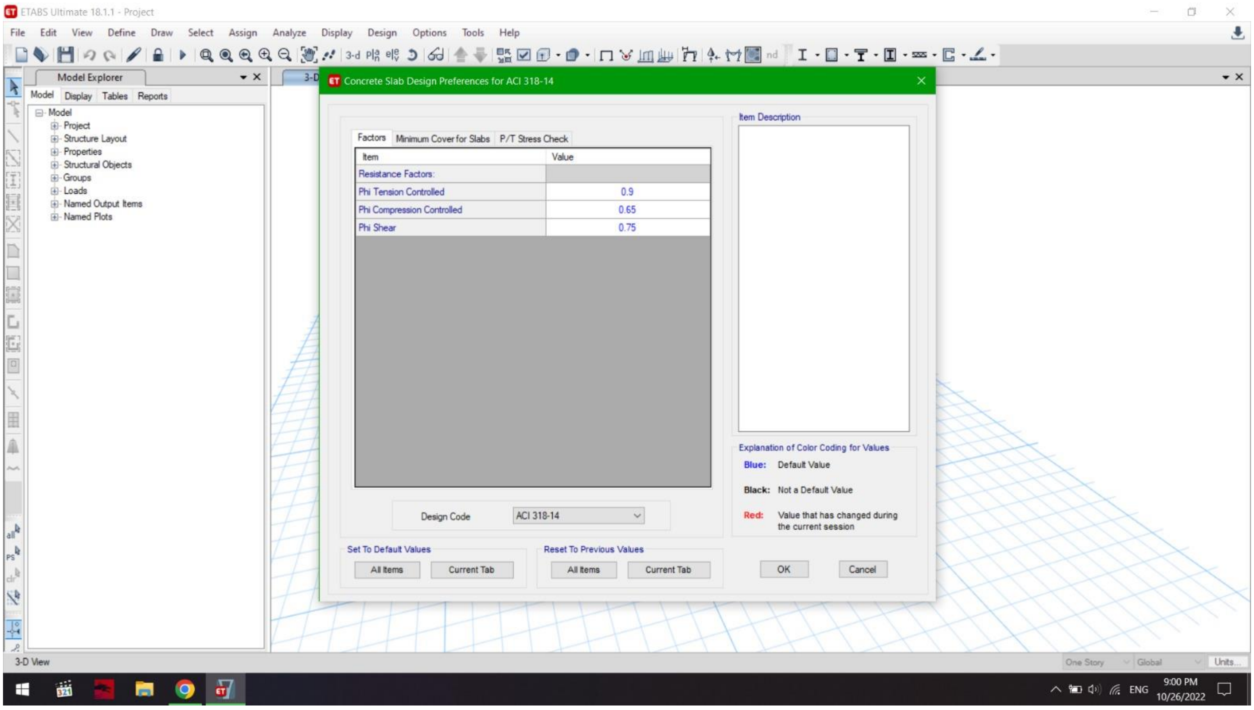


Figure 3.2-28: Slab design code.

3.3 Evaluation of the preliminary design: -

The model was made according to the figures in the last section, which shown every detail needed. The design check was made and some failures in the columns and beams were found. Hence, the dimension got changed to eliminate the failures.

First: With shear walls around the building:

- Beams: -

  Previous dimensions 300 mm depth * 300 mm width for all beams.

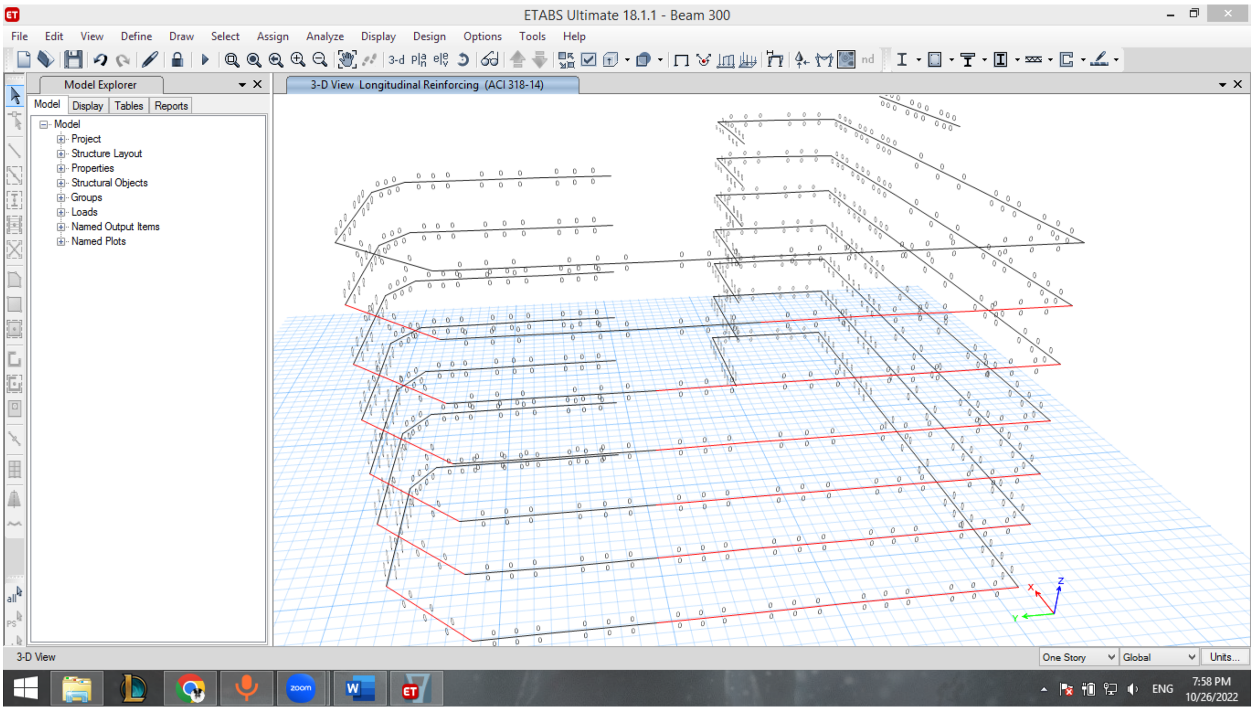


Figure 3.3-1: Beams 30*30.

Error: O/S #45 Shear stress due to shear force and torsion together exceeds maximum allowed.

New dimensions 400 mm depth * 400 mm width for all beams.

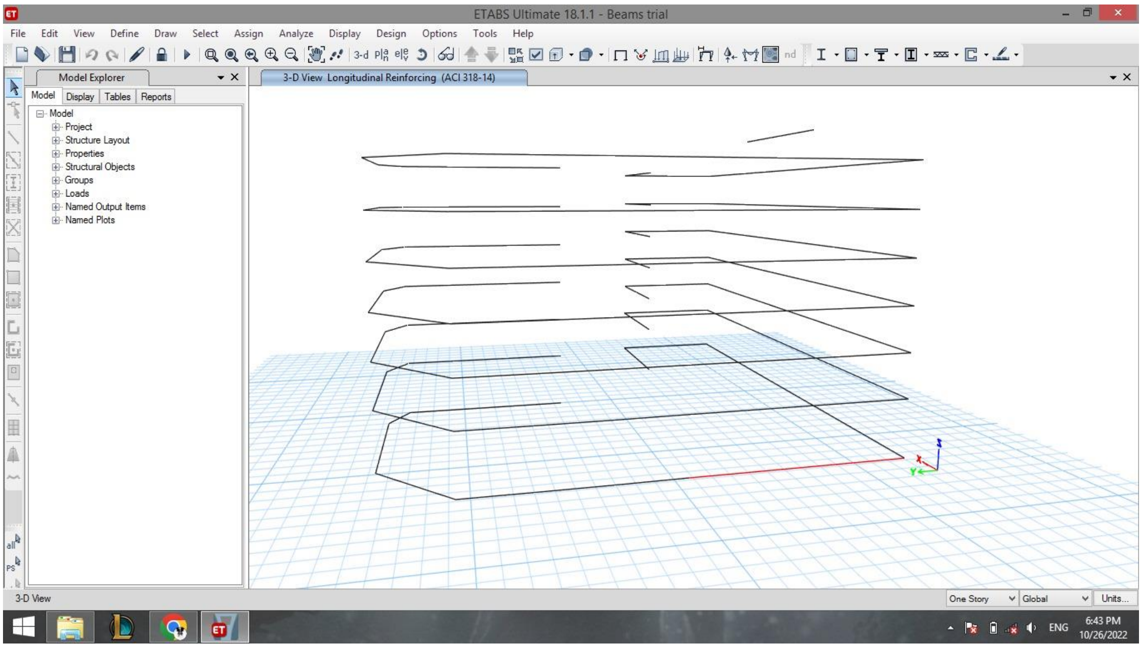


Figure 3.3-2: Beams new dimensions.

Error: O/S #45 Shear stress due to shear force and torsion together exceeds maximum allowed.

Last new dimensions 400 mm depth * 400 mm width for all beams, and 500 mm depth * 400 mm width for the beams that failed in figure 3.??

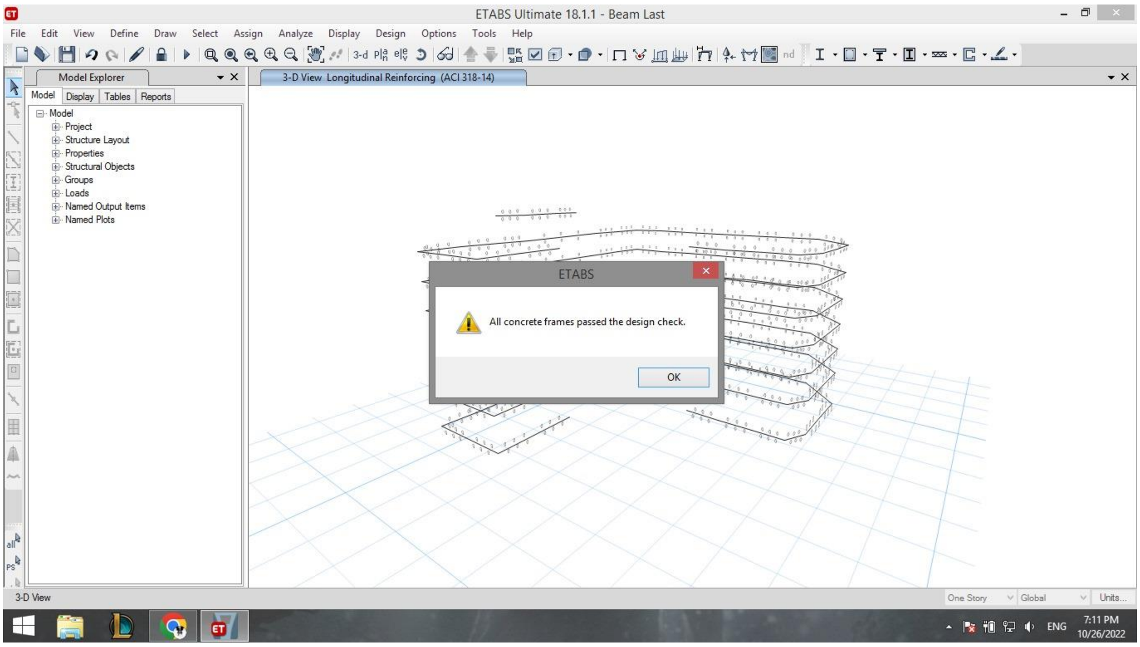


Figure 3.3-3: Beams 40*40 & 50*40.

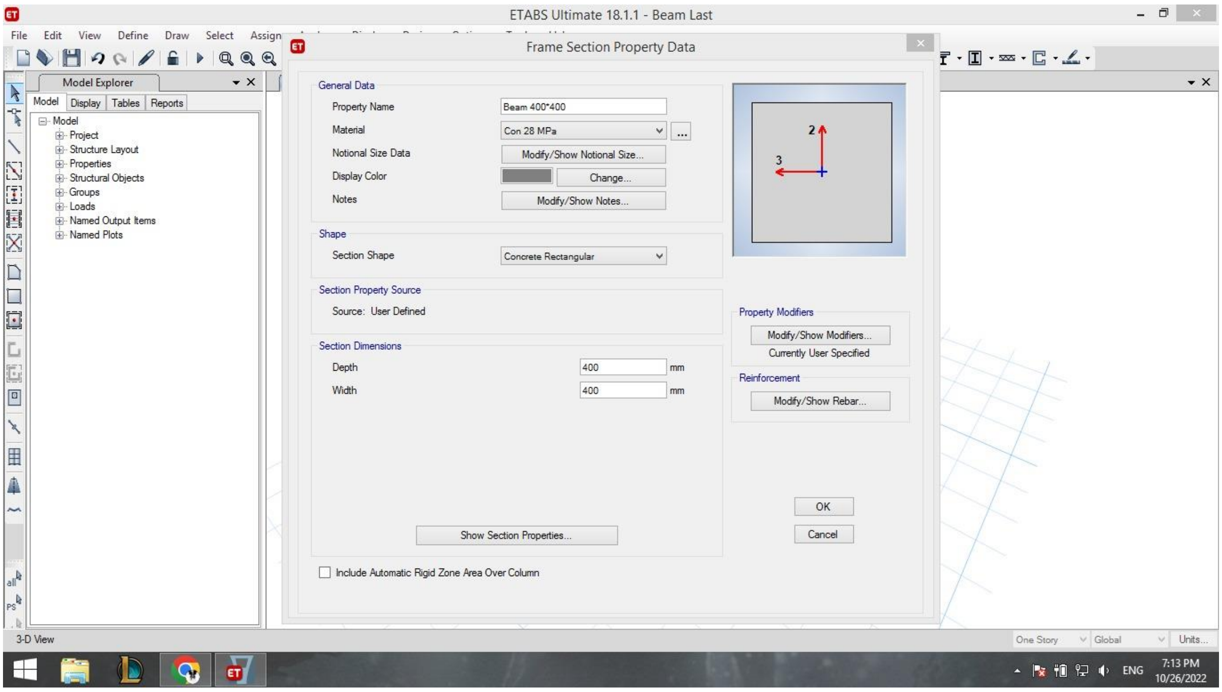


Figure 3.3-4: Beams 40*40.

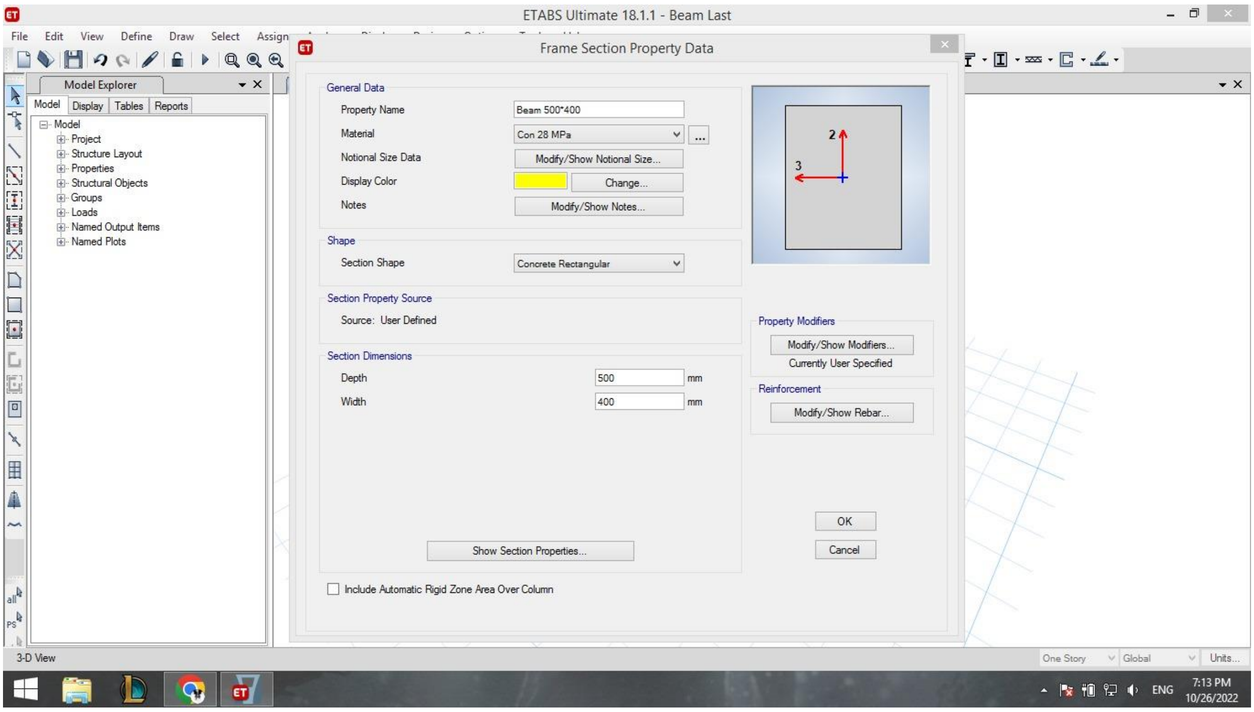


Figure 3.3-5: Beams 50*40.

- Columns: -

  Previous dimensions 300 mm depth * 700 mm width for all columns.

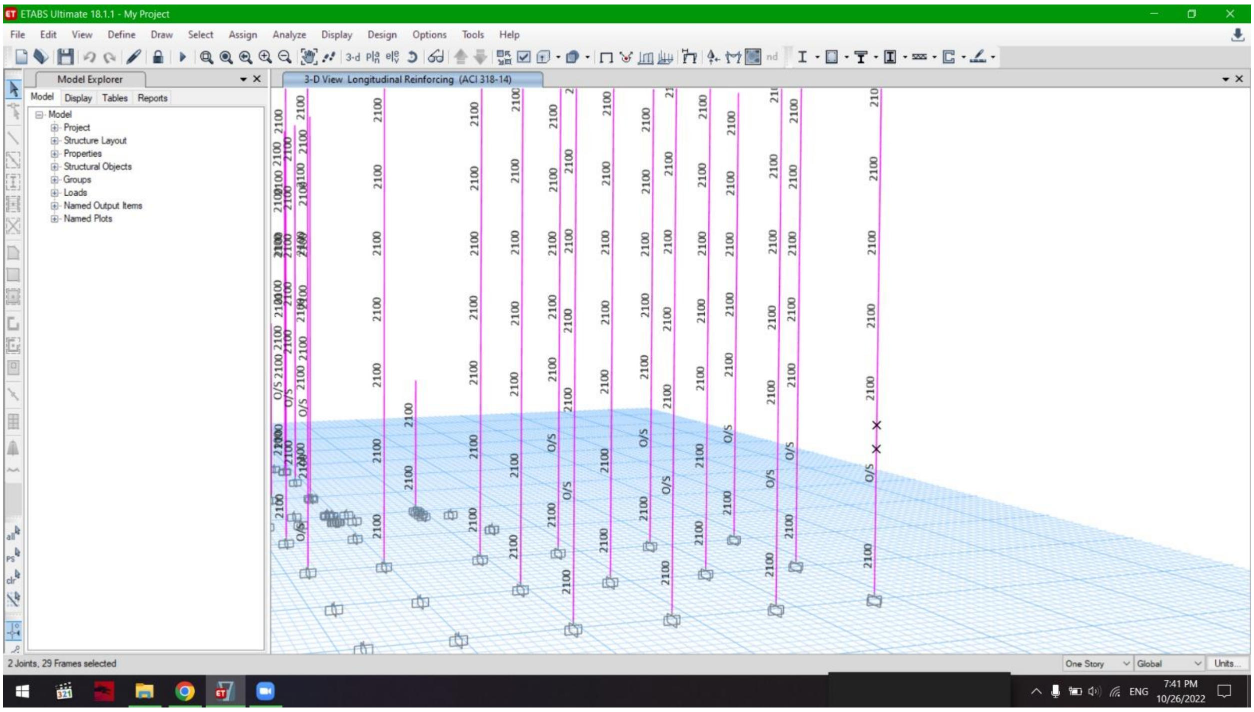


Figure 3.3-6: Columns previous dimensions.

Error: Warning #20 The δ_{ns} factor exceeds the limit.

New dimensions 400 mm depth * 700 mm width for all columns.

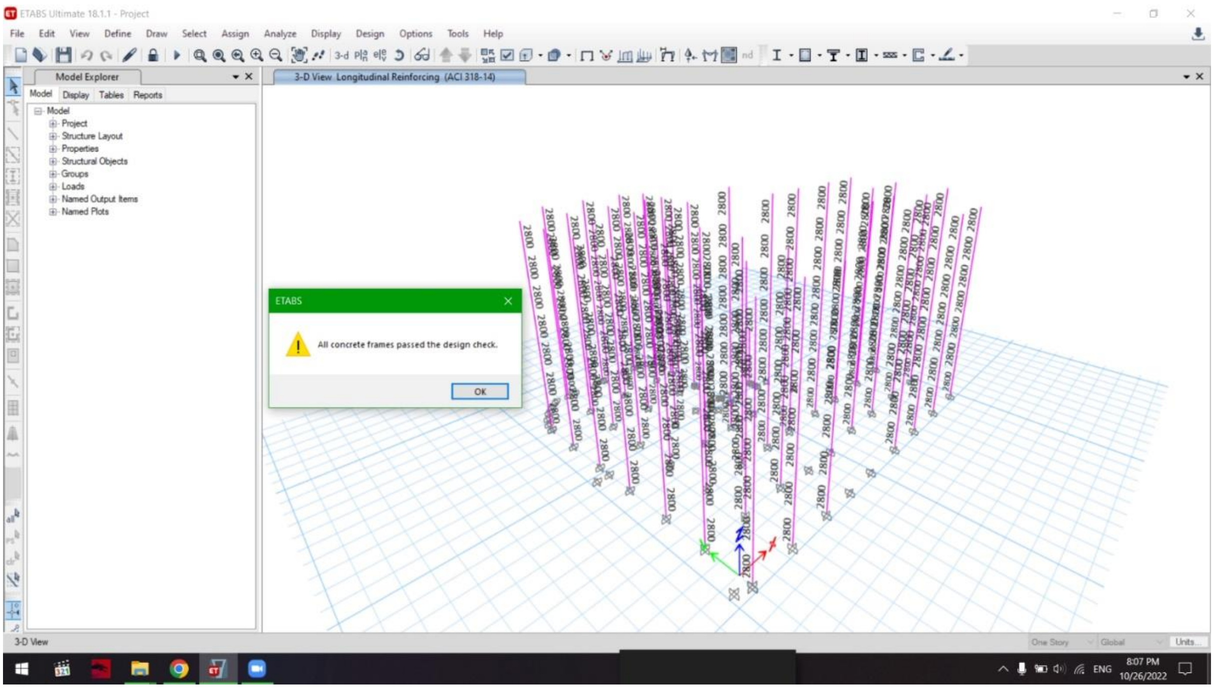


Figure 3.3-7: Columns 40*70.

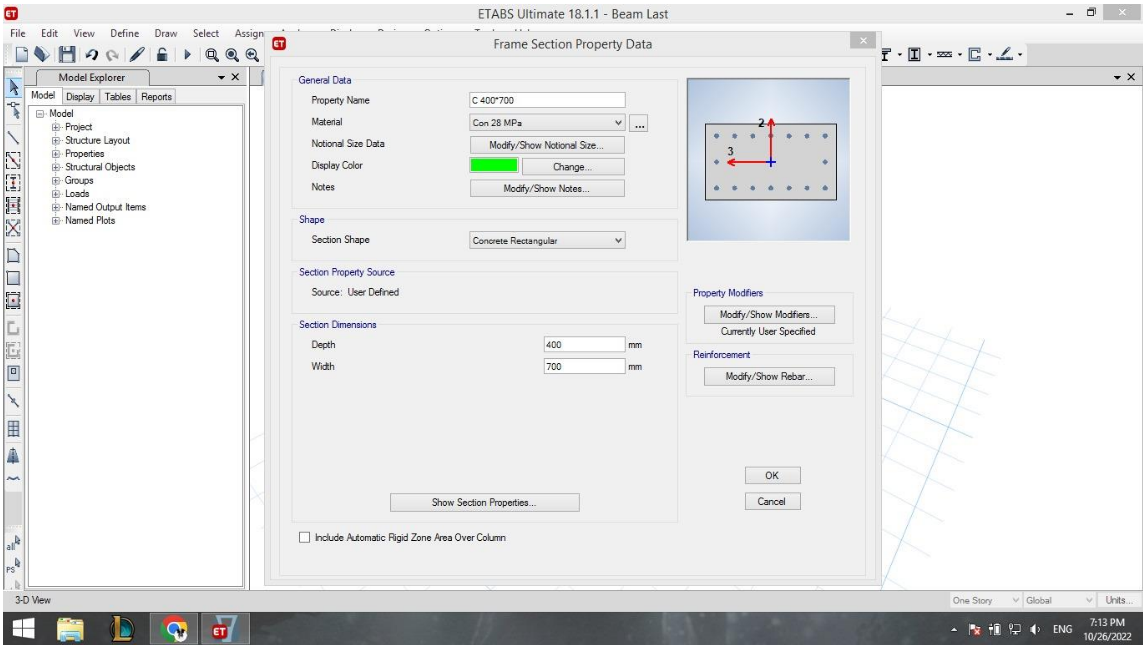


Figure 3.3-8: Columns 40*70 detailed.

Second: Without shear walls around the building:

- Beams: -

  Previous dimensions 400 mm depth * 400 mm width for all beams, and some are 500 mm depth * 400 mm width for all beams

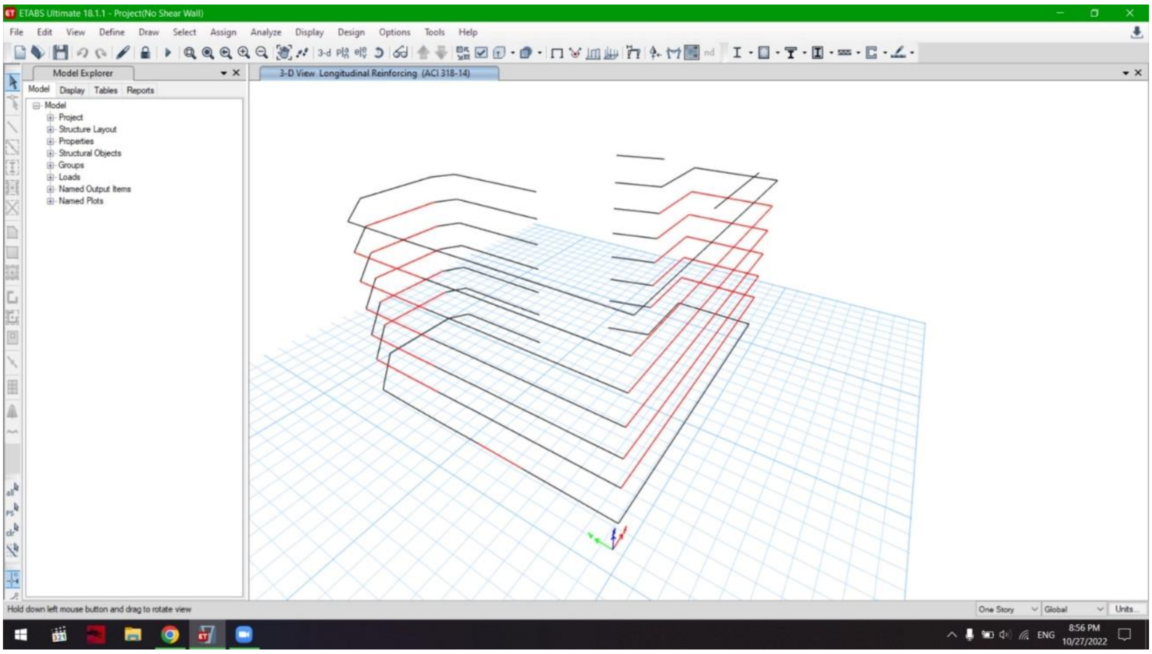


Figure 3.3-9: Beams 40*40 & 50*40.

Error: O/S #45 Shear stress due to shear force and torsion together exceeds maximum allowed.

New dimensions 550 mm depth * 400 mm width for all beams.

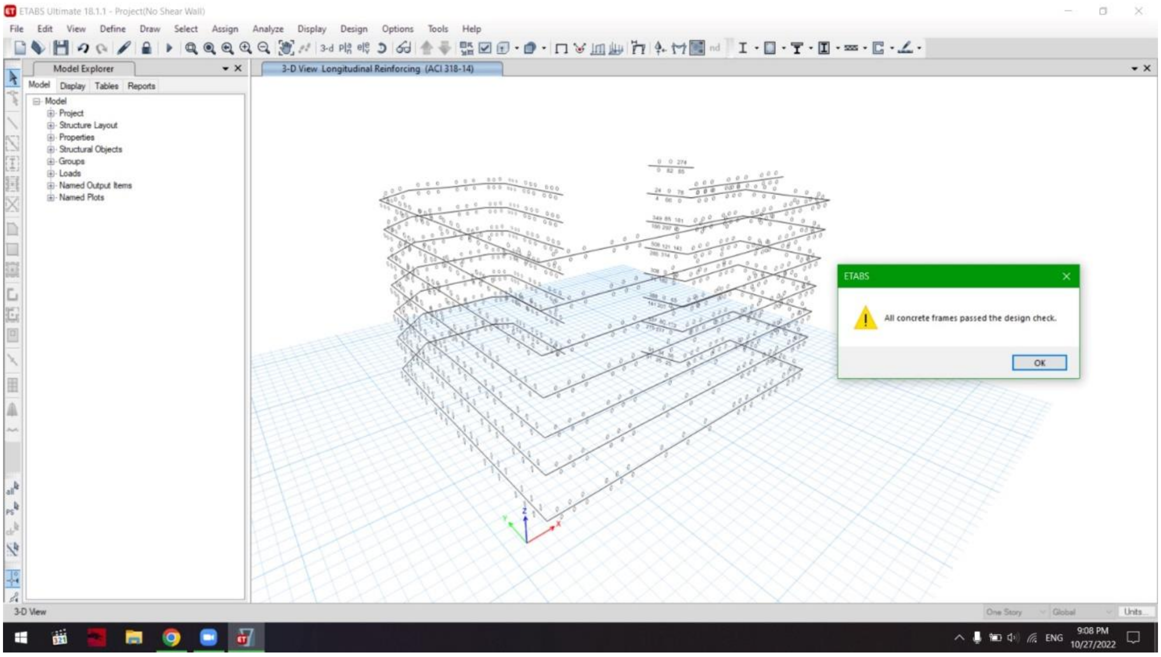


Figure 3.3-10: Beams 55*40.

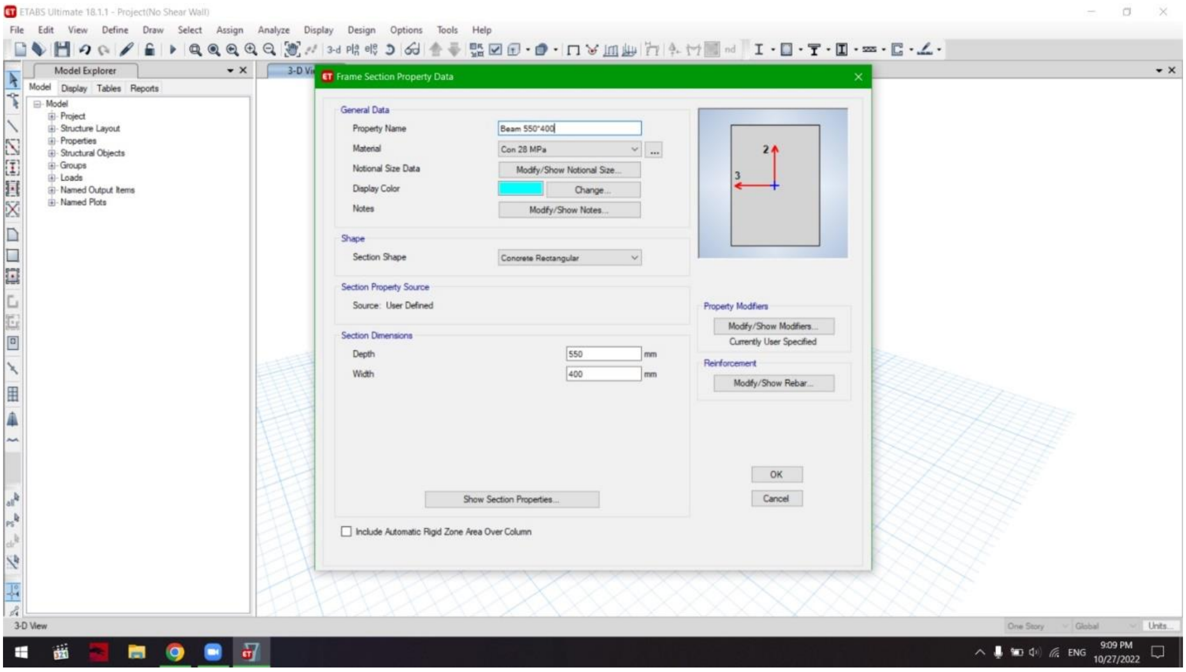


Figure 3.3-11: Beams 55*40.

- Columns: -

Previous dimensions 400 mm depth * 700 mm width for all columns. All passed.

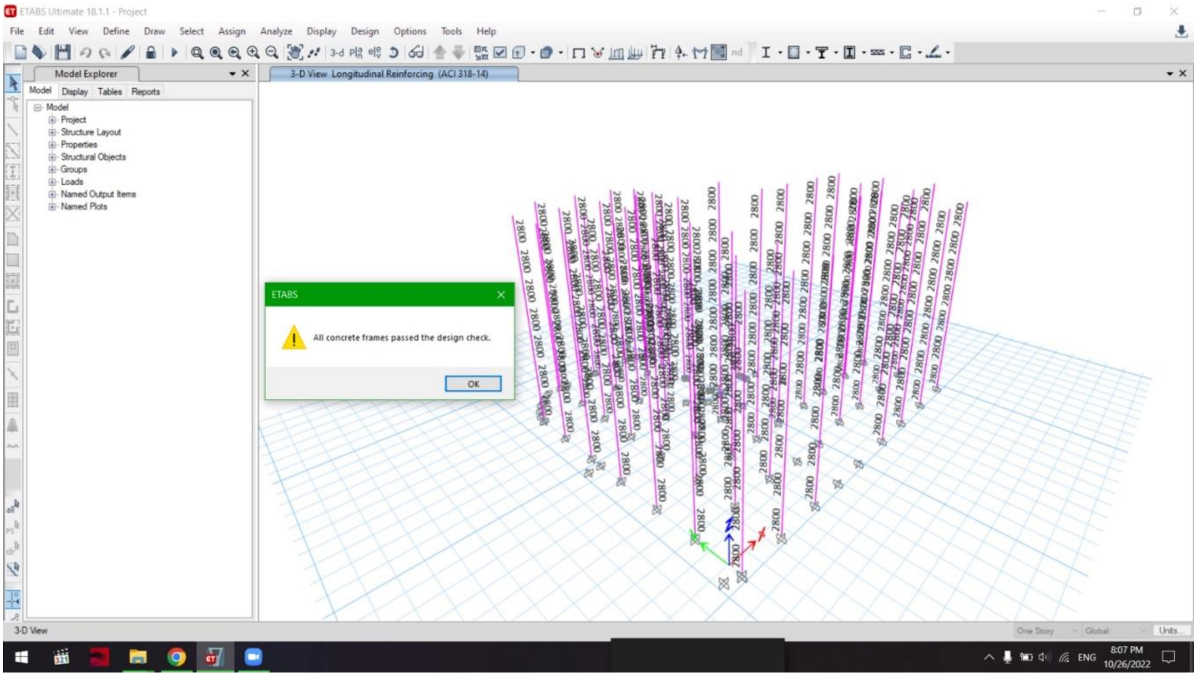


Figure 3.3-12: Columns 40*70.

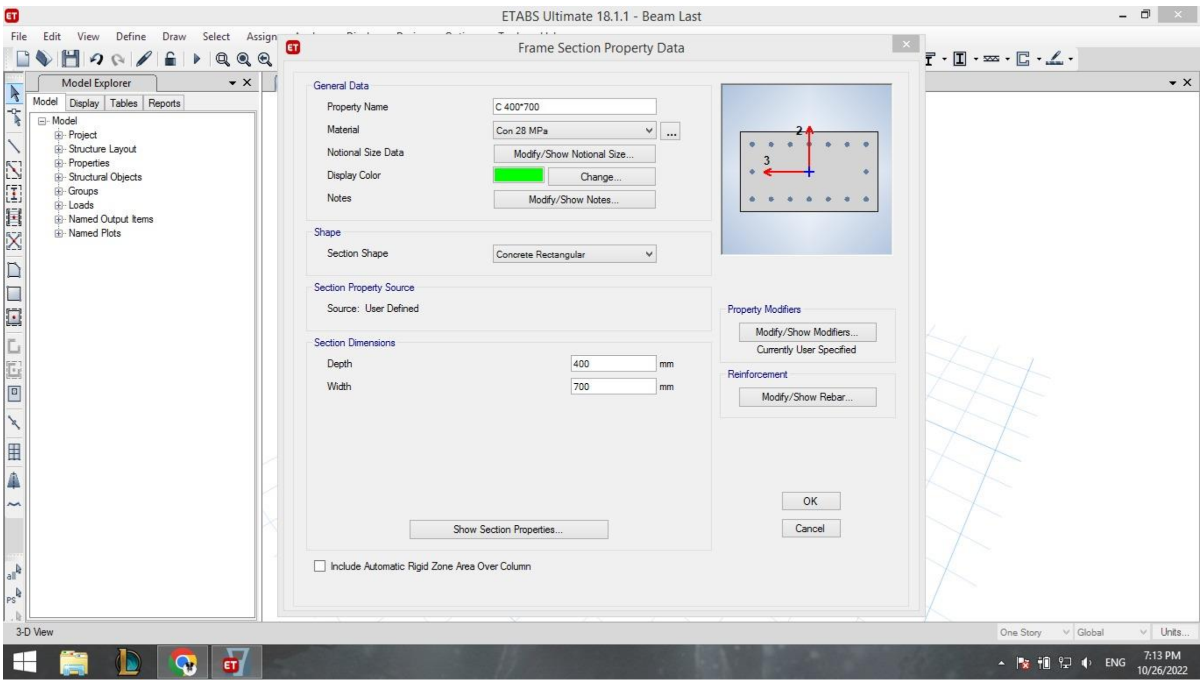


Figure 3.3-13: Columns 40*70 detailed.

3.4 Verification of structural analysis: -

3.4.1 Compatibility of the structure:

Compatibility is related to the shape of a structure, this includes deformations, location of reaction points and the way that a structure is allowed to bend and deform.

This check is performed to make sure that all the structural elements are connected. So, when the building undergoes a certain deformation, all the members close to each other which are connected will deform together.

The check can be performed by running the model, then starting the animation to check if there is any disconnection. Figure 3.4-1 shows the structure under the deformation of its own weight.

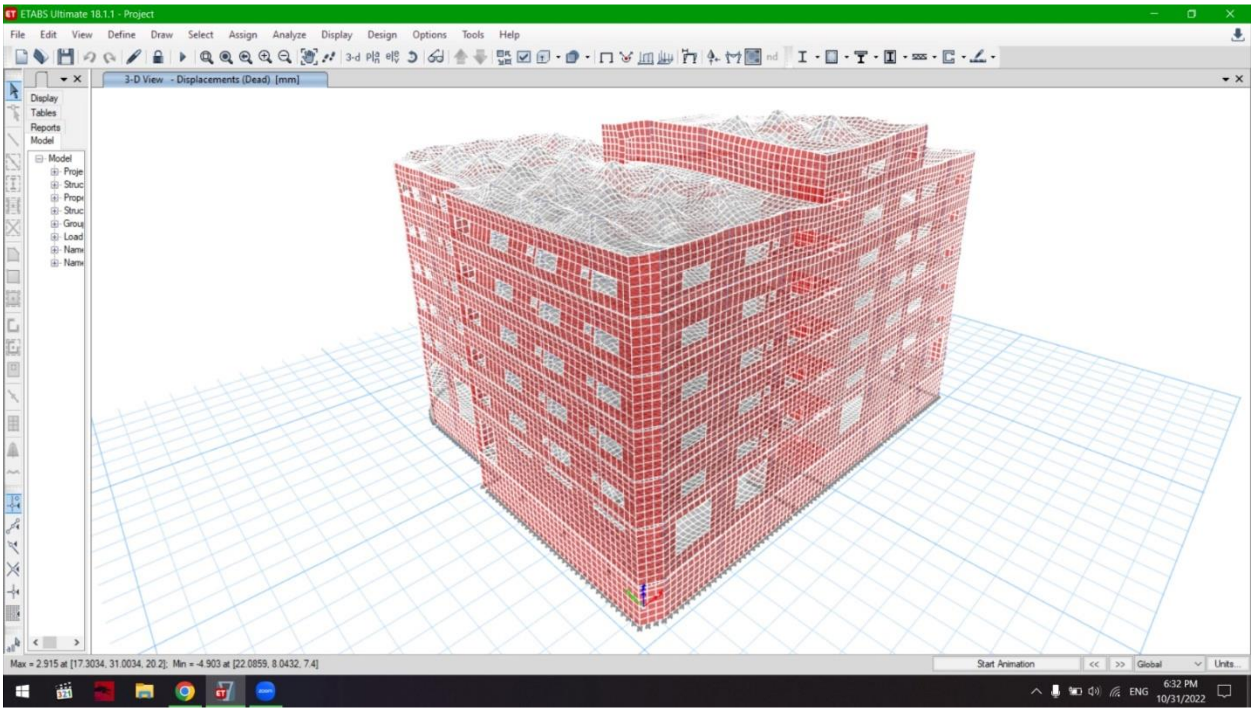


Figure 3.4-1: Structure in 3D view.

According to figure 3.4-1, the structure is stable and compatible, and there is no discontinuity between continues members.

3.4.2 Gravity loads: -

Gravity loads are the vertical forces that act on a structure. The weight of the structure, human occupancy and snow are all types of loads that needs to have a complete load path to the ground.
Every gravity load will be calculated manually and compared to the ETABS results, the allowable difference between them can be within ± 5%.

1) Dead load: a constant load in a structure that is due to the weight of the members and the supported structure.
   It can be calculated such as:
   Dead Load = ∑ (The volume of the element * $Ɣ_{Material}$)
   - Columns:
     Since the columns have a single section and material, take the total length of all columns (1065.80 m).
     Columns D.L. = (Length * Width * Depth * Ɣ)
     → Columns D.L. = (1065.80 * 0.70 * 0.40 * 25.00) = **7460.60 KN**

   - Beams:
     Since the beams have a single section and material, take the total length of all beams (902.45 m).
     Beams D.L. = (Length * Width * Depth * Ɣ)
     → Beams D.L. = (902.45 * 0.40 * 0.55 * 25.00) = **4963.48 KN**

   - Walls:
     Since the walls have a single section and material, take the total area of the walls (3587.50 m^2).
     Walls D.L. = (Area * Depth * Ɣ)
     → Walls D.L. = (3587.50 * 0.20 * 25.00) = **17937.50 KN**
   - Slabs:
     Since the slabs have different sections, take areas according to the depth of the slab. Table 3.4-1 shows the values of dead load on each slab.

Table 3.4-1: Slab dead load.

Slab	**Area (m2)**	Depth	D.D.
Basement	891.39	0.22	4902.65
Stairs	37.67	0.20	188.35
Ground	879.60	0.21	4617.90
Elec. Roof	8.03	0.21	42.16
Balcony	5.41	0.21	28.40
Stairs	37.67	0.20	188.35
1st	879.60	0.21	4617.90
Balcony	5.41	0.21	28.40
Stairs	37.67	0.20	188.35
2nd	879.60	0.21	4617.90
Balcony	5.41	0.21	28.40
Stairs	37.67	0.20	188.35
3rd	879.60	0.21	4617.90
Balcony	5.41	0.21	28.40
Stairs	37.67	0.20	188.35
4th	879.60	0.21	4617.90
Balcony	5.41	0.21	28.40
Stairs	37.67	0.20	188.35
5th	883.36	0.18	3975.12
Stairs	37.67	0.20	188.35
Roof	258.93	0.18	1165.19
Total	**6730.45**		**34635.07**

Dead load = ∑ Dead loads = **64996.65 KN**

ETABS dead load = **65513.25 KN**

Difference % = $\frac{|Manual-Etabs|}{Manual} * 100$, as shown in table 3.4-2.

Table 3.4-2: Dead load error %.

Results	Manual	ETABS	Error%
D.D.	64996.65	65513.25	**0.79**

2) Super imposed dead load: the load of non-structural elements on the structure after the structural elements have been casted.
   It can be calculated such as:
   Super imposed dead load = ∑ (The areas subjected to S.D. * S.D. per meter square)

   The stone side area was measured by counting the shells of the shear wall, which had a S.D. on it, and multiply it by the average area of the shells (10346), shells area = 0.25 m^2.

3) Live load: the load of any moving or non-stationary elements.
   It can be calculated such as:
   Live Load = ∑ (The areas subjected to live loads * L.L. per meter square)

   Table 3.4-3 shows the numbers used in calculating L.L. and S.D., and the errors are shown in table 3.4-4.

Table 3.4-3 : Live and super imposed dead loads.

Slab	Area (m2)	L.L. (KN/m2)	L.L. (KN)	S.D. (KN/m2)	S.D. KN
Basement	891.39	6.00	5348.34	4.00	3565.56
Stairs	37.67	5.00	188.35	4.00	150.68
Ground	879.60	3.00	2638.80	4.00	3518.40
Elec. Roof	8.03	0.00	0.00	2.50	20.08
Balcony	5.41	5.00	27.05	4.00	21.64
Stairs	37.67	5.00	188.35	4.00	150.68
1st	879.60	3.00	2638.80	4.00	3518.40
Balcony	5.41	5.00	27.05	4.00	21.64
Stairs	37.67	5.00	188.35	4.00	150.68
2nd	879.60	3.00	2638.80	4.00	3518.40
Balcony	5.41	5.00	27.05	4.00	21.64
Stairs	37.67	5.00	188.35	4.00	150.68
3rd	879.60	3.00	2638.80	4.00	3518.40
Balcony	5.41	5.00	27.05	4.00	21.64
Stairs	37.67	5.00	188.35	4.00	150.68
4th	879.60	3.00	2638.80	4.00	3518.40
Balcony	5.41	5.00	27.05	4.00	21.64
Stairs	37.67	5.00	188.35	4.00	150.68
5th	883.36	1.00	883.36	2.50	2208.40
Stairs	37.67	5.00	188.35	4.00	150.68
Roof	258.93	1.00	258.93	2.50	647.33
Stone	2586.50	-	-	3.00	7759.50
	Total	**21138.33**		Total	**32955.82**

Table 3.4-4: Live and super imposed dead errors %.

Results	Manual	ETABS	Error%
S.D.	32955.82	33526.20	**1.70**
L.L.	21138.33	21203.02	**0.31**

3.4.3 Soil loads: -

It's the summation of the lateral static earth pressure and the dynamic (seismic) earth pressure on the walls in the basements.
The soil loads will be calculated manually and compared to the ETABS results, the allowable difference between them should be within ± 5%. Figure 3.4-2 shows the soil pressure values on the basement walls.

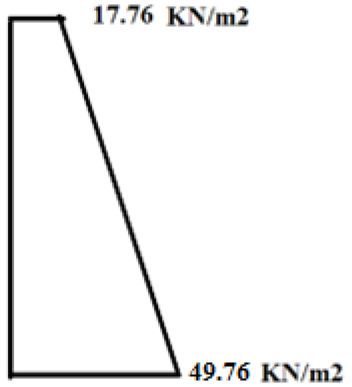


Figure 3.4-2: Soil pressure.

Taking 1 m length of the wall, the resultant force of the pressure: $\frac{49.76+17.76}{2} * 3.20 = 108.03$ KN/m_{length}

Fx = ∑ (Resultant * The length alongside or parallel to the X-axis)

Fy = ∑ (Resultant * The length alongside or parallel to the Y-axis)

The differences are shown in table 3.4-5.

Table 3.4-5 : Soil loads differences %.

Direction	Manual	ETABS	%Difference
X	1728.48	1728.91	**0.03**
Y	2285.91	2286.56	**0.03**

3.4.4 Verification of internal forces: -

Verification of internal forces are done to see if the local loads would transfer probably, and it can be an extra test to check the continuity. The live load will be used to check these members, Live load = 3.00 KN/m^2

1. **Columns**: taking a column sample on the fourth floor as shown in figure 3.4-3, and compare the results from manual calculations and ETABS.

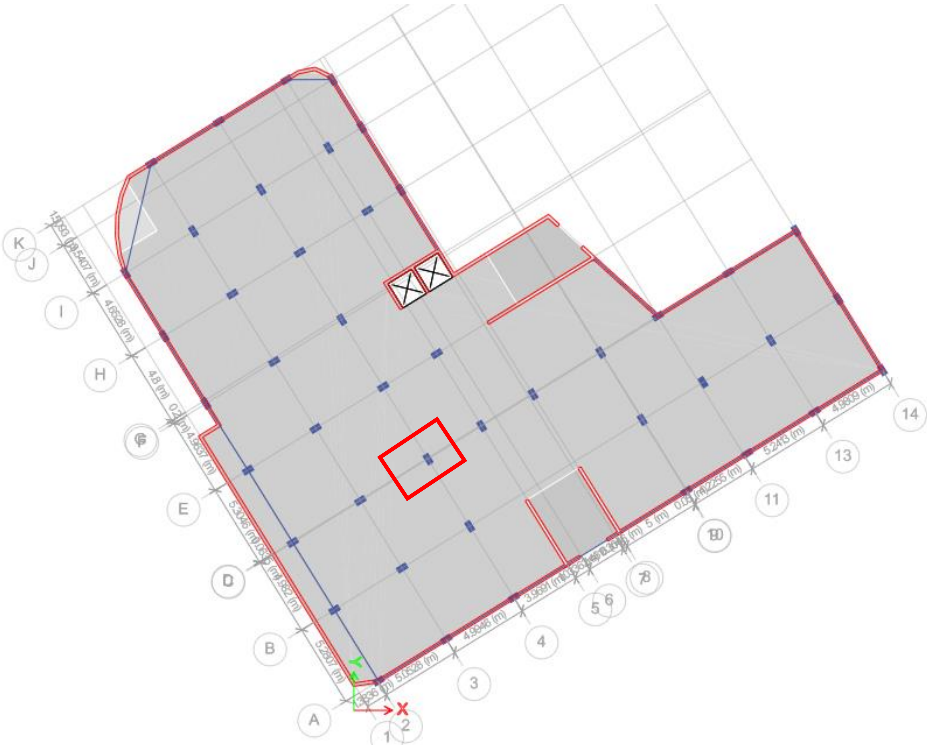


Figure 3.4-2: Column sample.

Attribute area = $\frac{5.35+5}{2} * \frac{5+5}{2}$ = 25.88 m^2

Axial force on the columns = 3.00 * 25.88 = **77.63 KN**

ETABS result as shown in the figure 3.4-4.

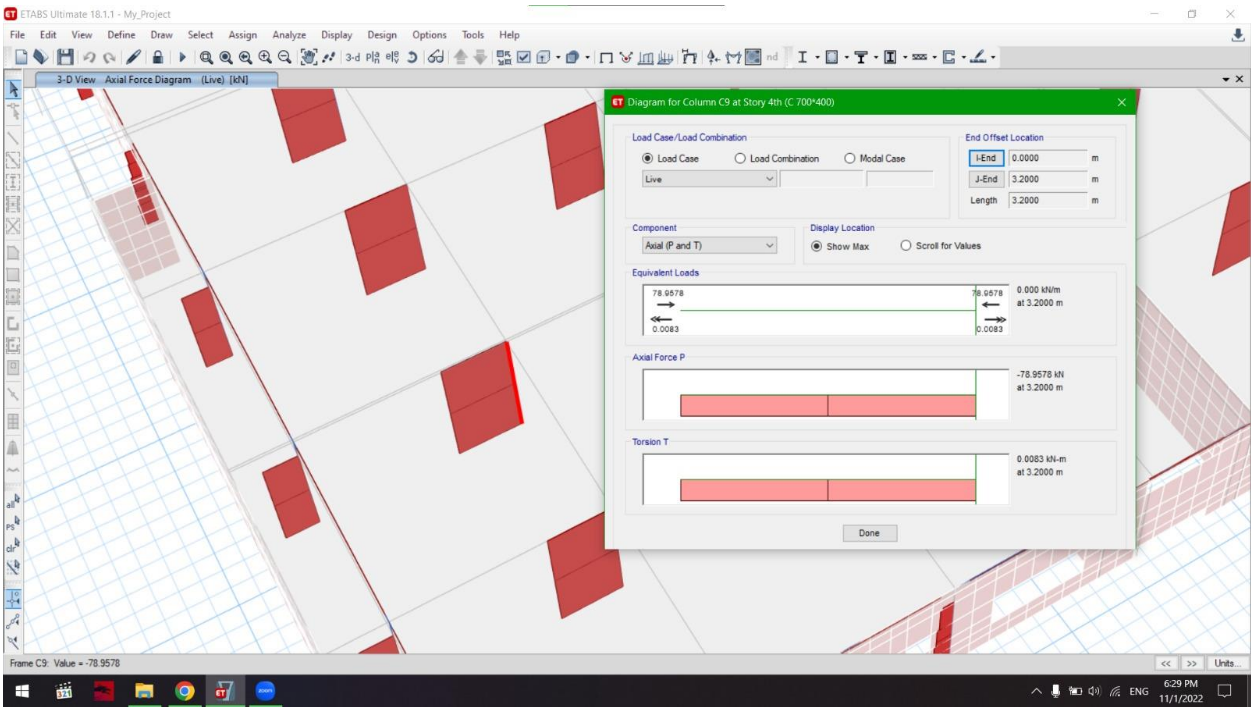


Figure 3.4-3: ETABS Column.

The error in column results is shown in table 3.4-6.

Table 3.4-6: Column load error %.

Column	Manual	ETABS	%Error
D3	77.63	78.97	**1.70**

2. **Slab**: taking a frame strip sample in the X- direction on the fourth floor, and compare the results from manual calculations and ETABS. Figures 3.4-5 to 3.4-8 show the needed values to do manual calculations.

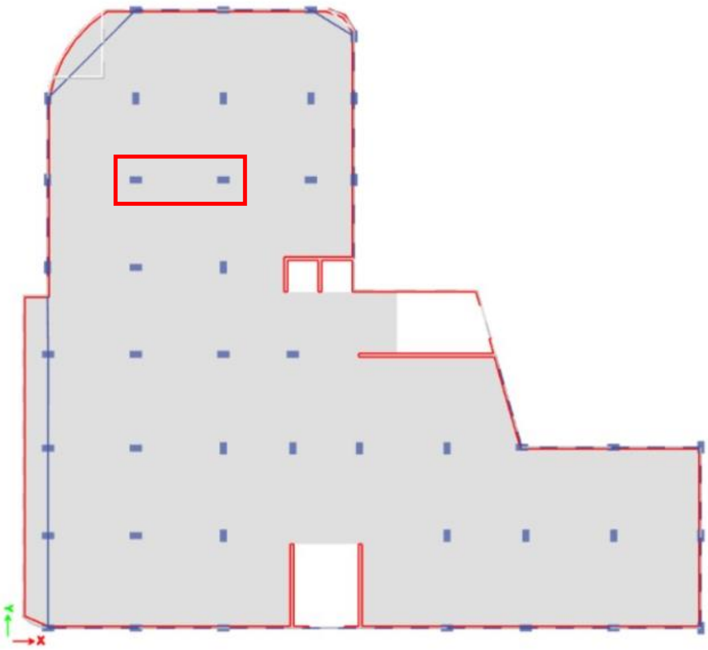


Figure 3.4-4: Frame strip sample.

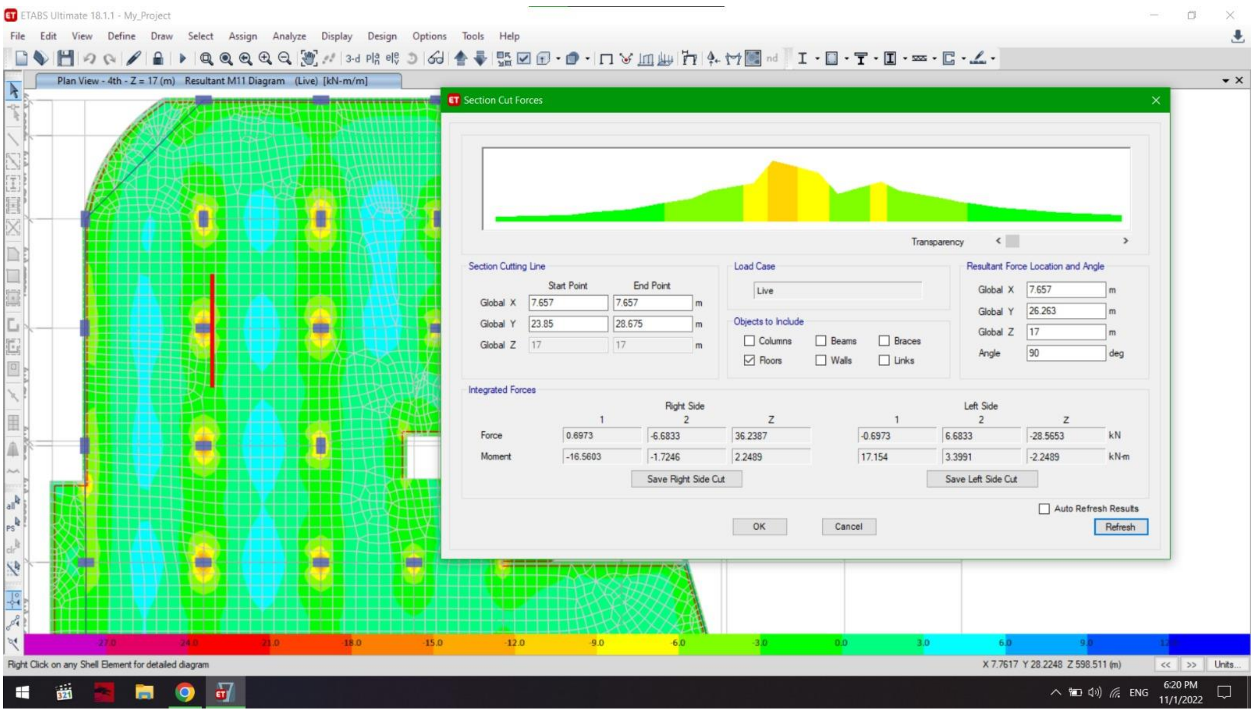


Figure 3.4-5: Strip left end.

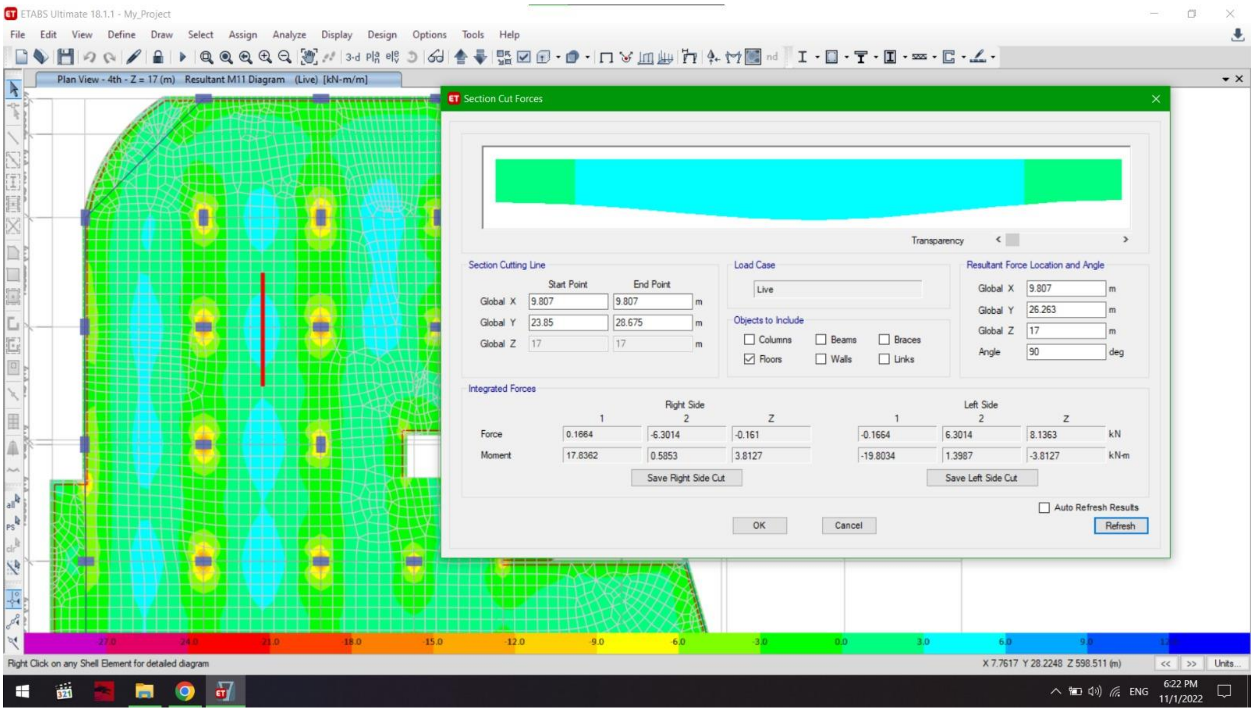


Figure 3.4-6: Strip middle.

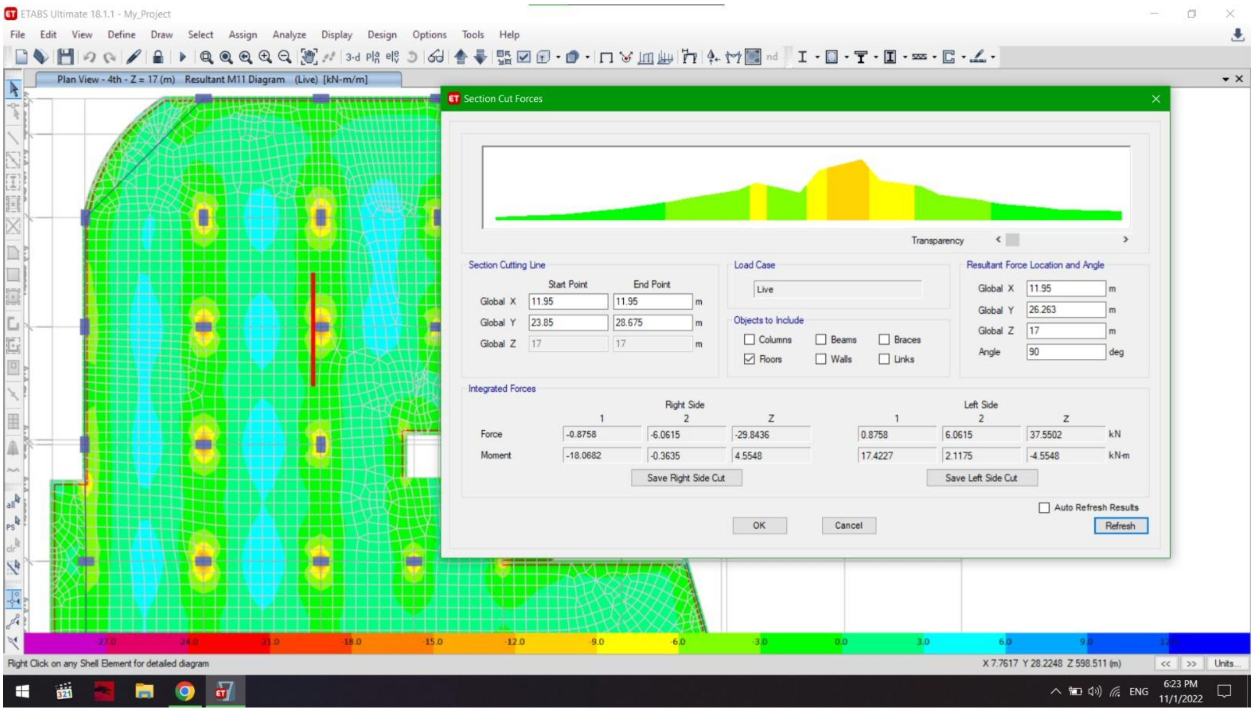


Figure 3.4-7: Strip right end.

$\mathbf{L_1}$ = 5.00 m, $\mathbf{L_n}$ = 4.30 m, $\mathbf{L_2} = \frac{5+4.65}{2}$ = 4.83 m

$\mathbf{M_0} = \frac{ql* L2*Ln\hat{}2}{8}$ = **33.46 KN.m** [Considering the span as a simply supported for simplicity]

$\mathbf{M_{ETABS}} = \frac{16.56+17.47}{2} + 18.82$ = **35.84 KN.m**

Error % $= \frac{|33.46-35.84|}{33.46} * 100$ = **7.11%**

Allowable difference should be within range ±10%.

3. **Beam**: taking a beam on the fourth floor connecting the columns B and C, as shown in the figures 3.4-9 to 3.4-12.

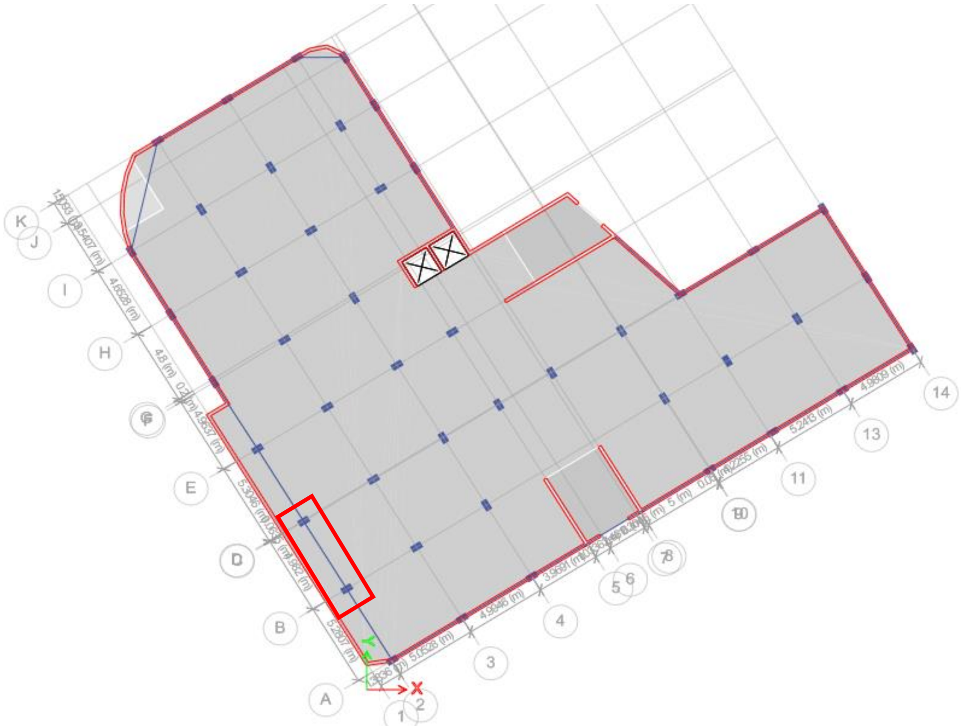


Figure 3.4-8: Beam sample.

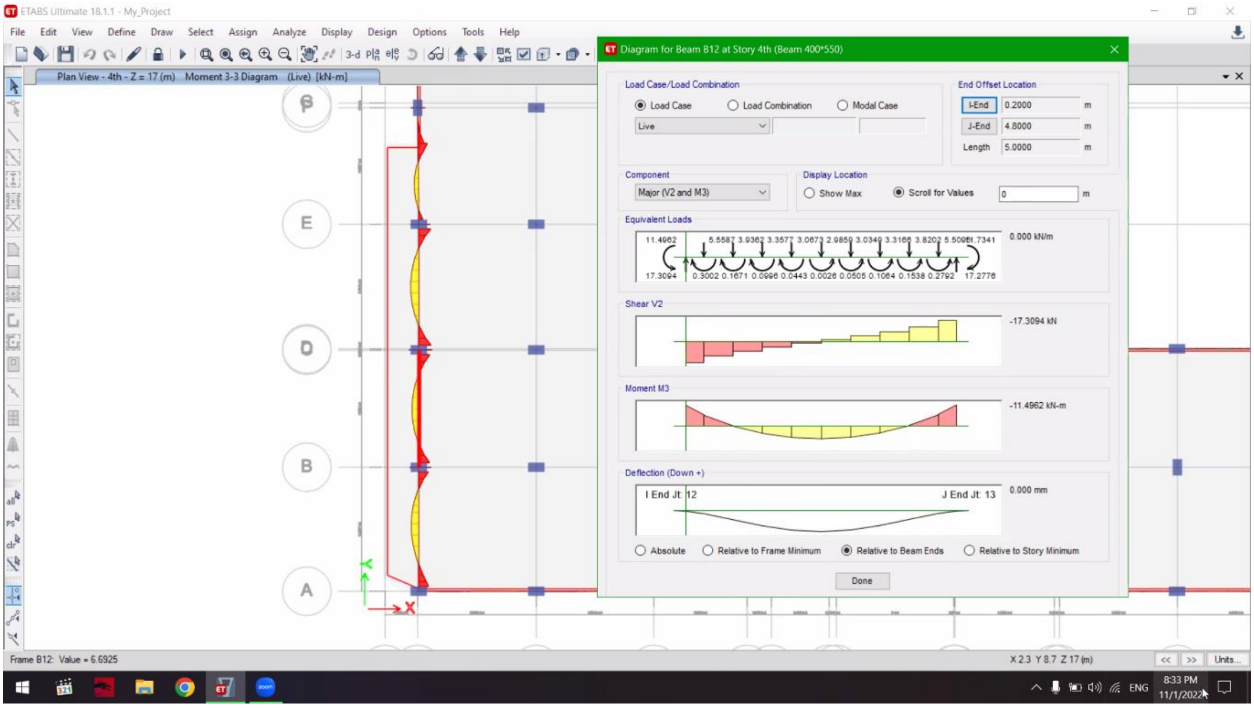


Figure 3.4-9: Moment left end.

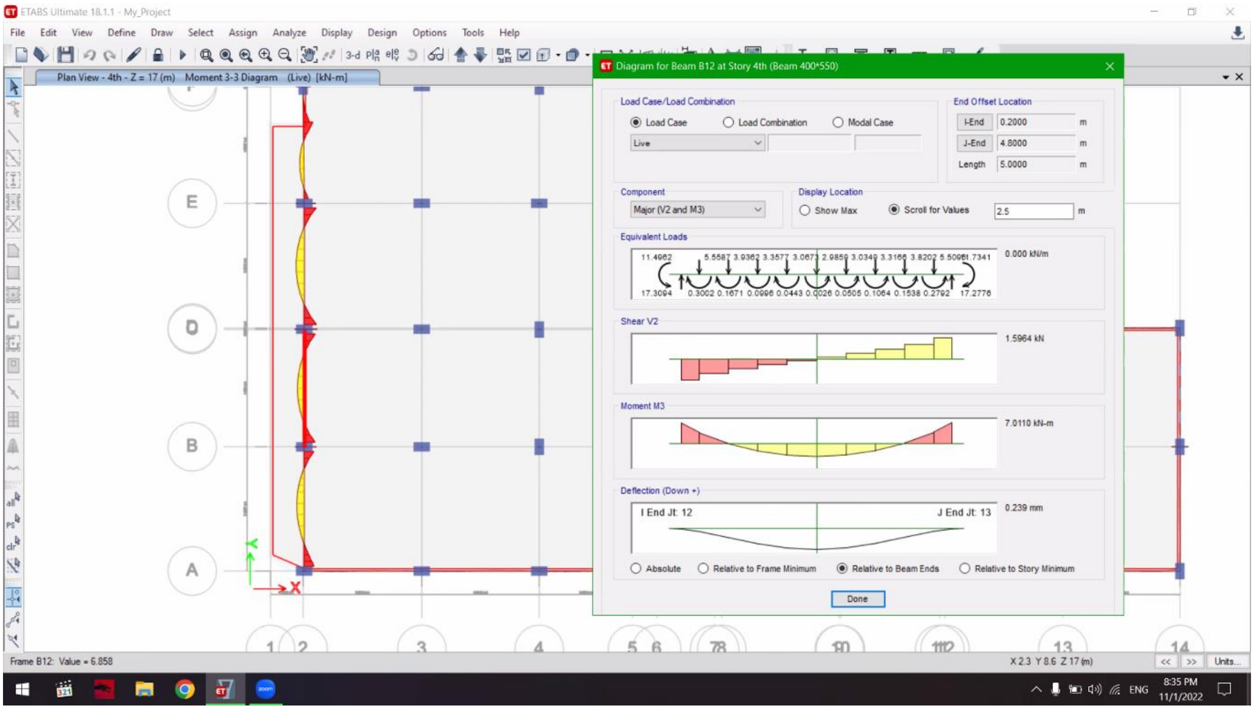


Figure 3.4-10: Moment middle.

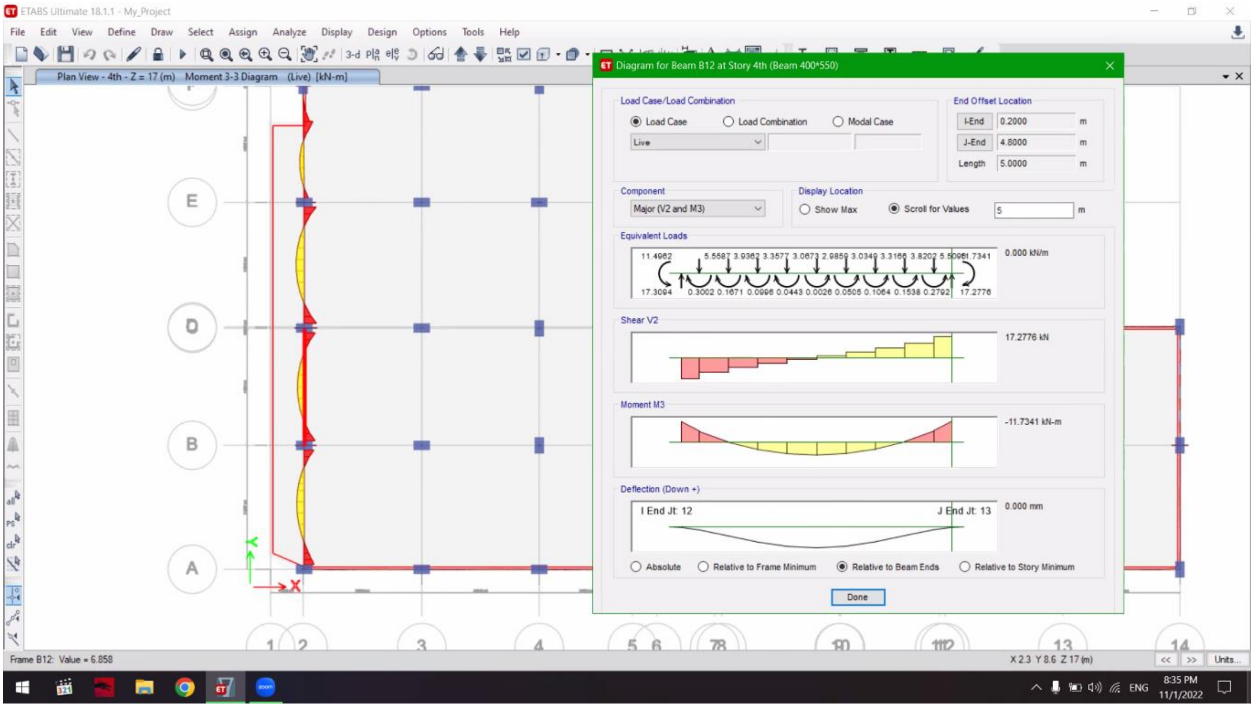


Figure 3.4-11: Moment right end.

$\mathbf{L_1}$ = 5.00 m, $\mathbf{L_n}$ = 4.60 m, $\mathbf{L_2}$ = 2.50 m

assuming the beam will take the half the right panel and nothing from the left.

$\mathbf{M_0} = \frac{ql * L2 * Ln\hat{}2}{8}$ = **19.84 KN.m** [Considering the beam a simply supported for simplicity]

$\mathbf{M_{ETABS}} = \frac{11.50+11.73}{2} + 7.01$ = **18.63 KN.m**

Difference % $= \frac{|19.84-18.63|}{19.84} * 100$ = **6.10%**

Allowable difference can be within range ±10%.

4. **Walls**: taking an area in the basement next to the beam A 12-13, as shown in the figures 3.4-13 and 3.4-14.

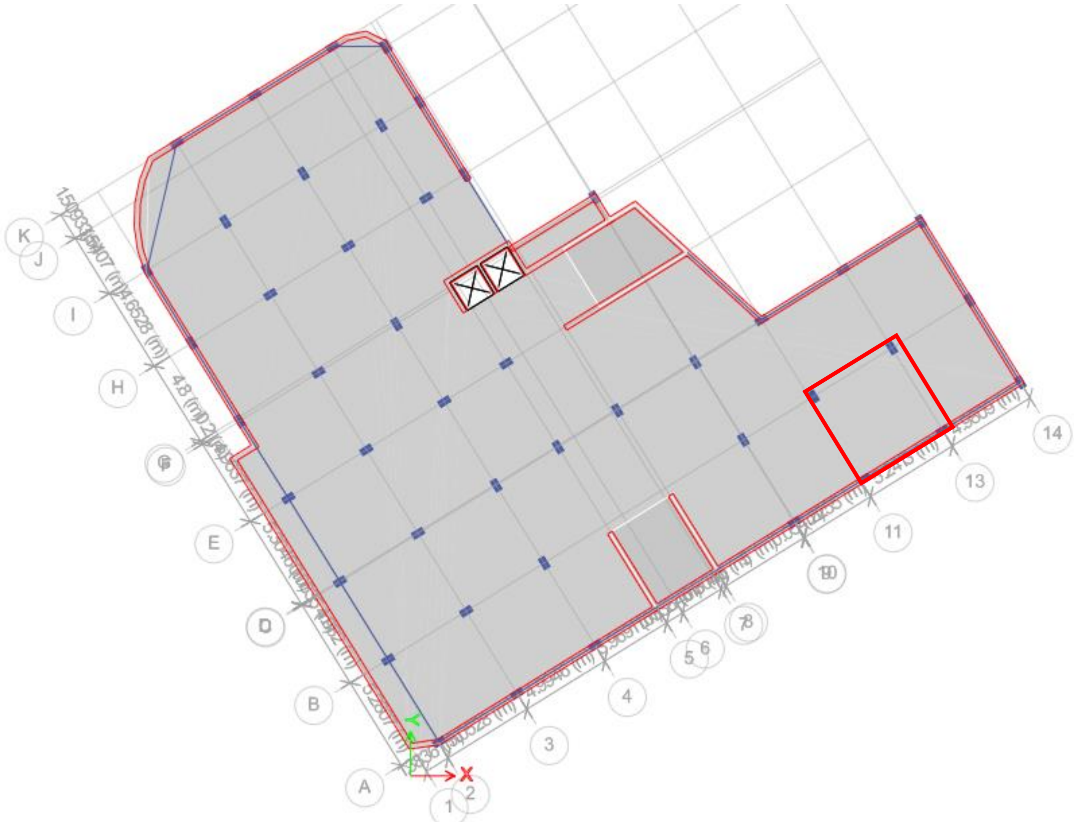


Figure 3.4-12: Shear wall sample.

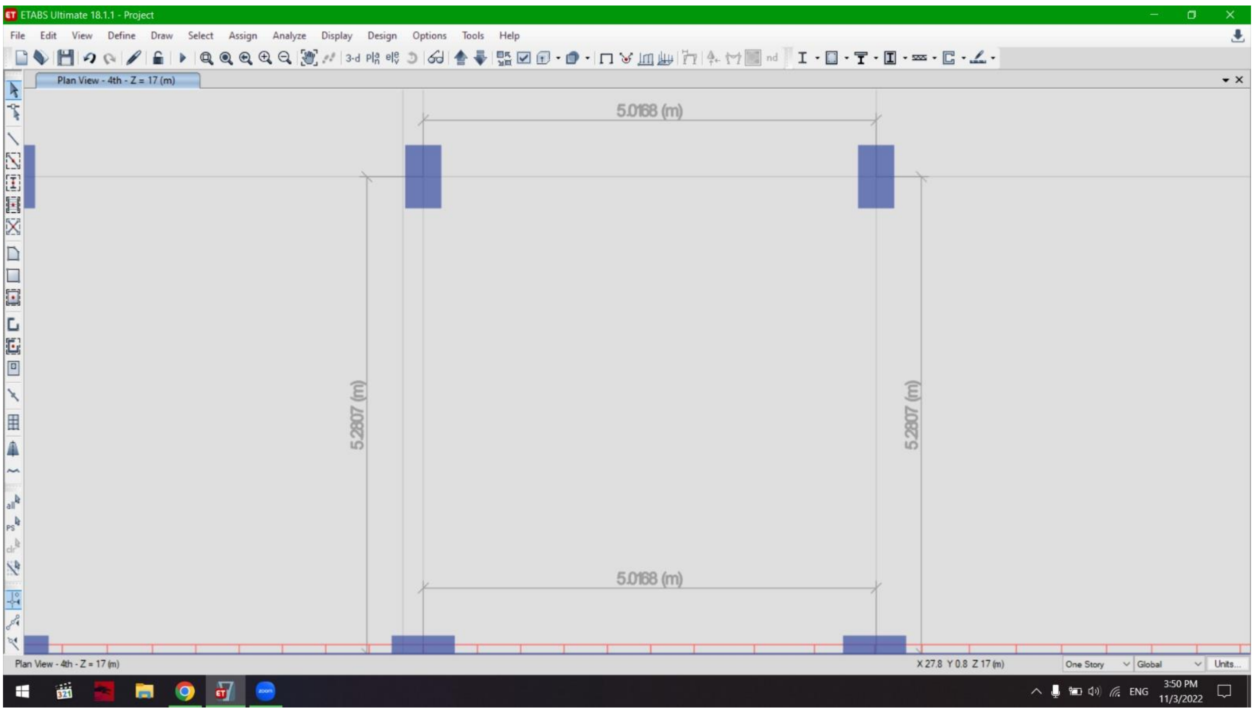


Figure 3.4-13: Shear wall panel sample.

Panel area = 5.28 * 5.02 = 26.51 m^2

Attribute area = 26.51 * X

Since the shear wall will take more than half the loading area, then assuming the shear segment will take X = 65% of the panel area → Axial load = 0.65 * 26.51 * 6 = **103.39 kN.**

ETABS result as shown in the figures 3.4-15 to 3.4-17.

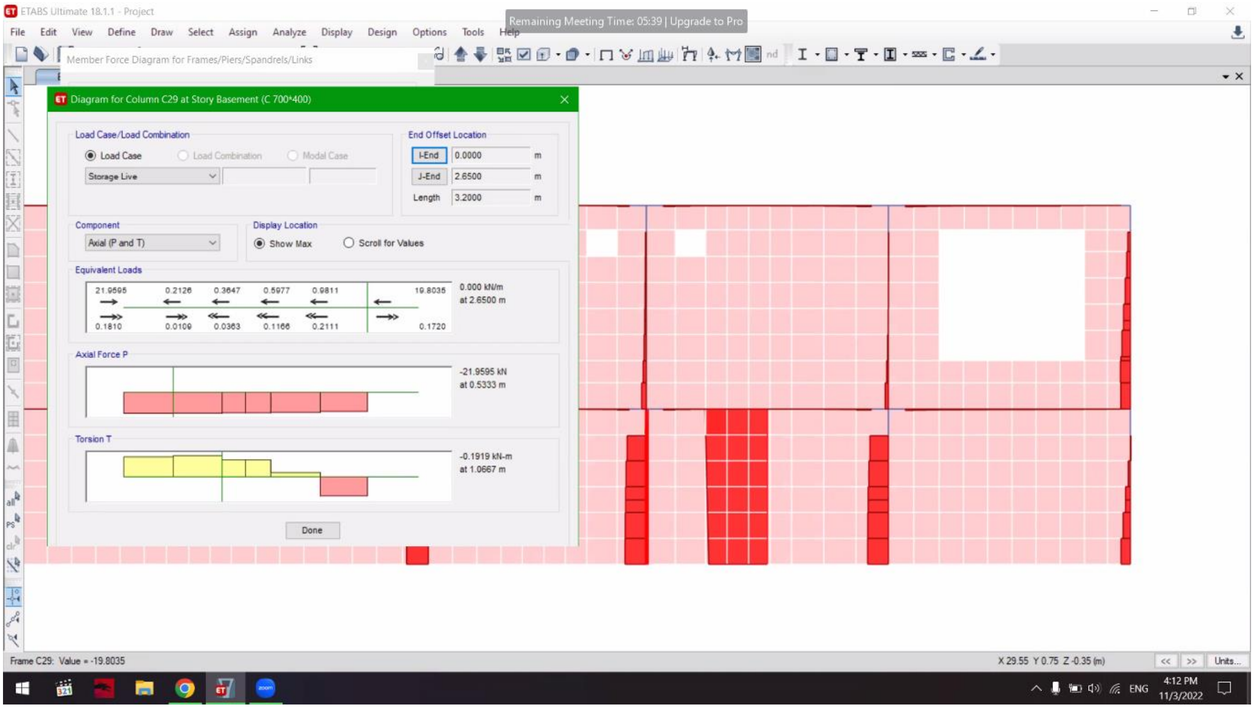


Figure 3.4-14: Left column from the shear wall segment.

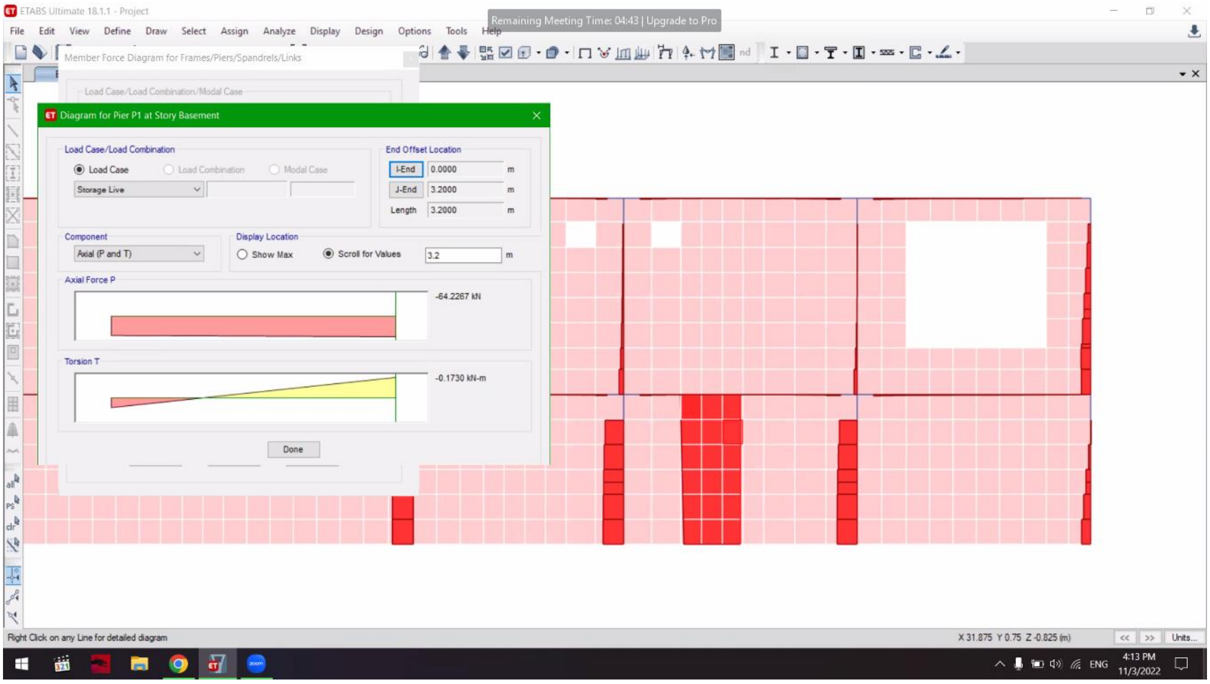


Figure 3.4-15: Shear wall between the two columns.

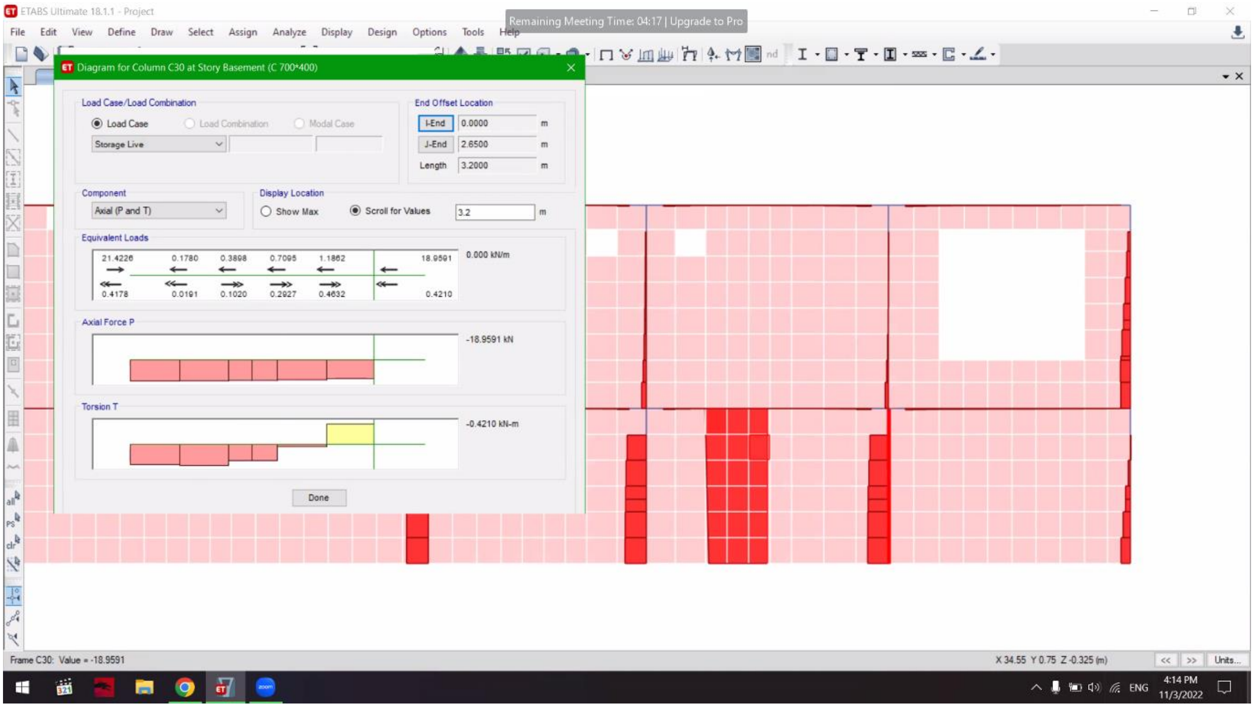


Figure 3.4-16:Right column from the shear wall segment.

Left column axial force = 19.8 kN

Shear wall axial force = 64 kN

Right column axial force = 18.96 kN

Total axial force = 102.76 kN

Difference % = $\frac{|103.39-102.76|}{103.39}$ * 100 = **0.61%**

Allowable difference can be within range ±10%.

3.4.5 Seismic forces:

1.Analysis method:

Three basic methods are available for analyzing the responses of a structure subjected to seismic forces, which are:

- Static analysis (Equivalent lateral force method).
- Response spectrum analysis.
- Time history analysis.

The analysis methods used in this project are response spectrum analysis and equivalent lateral force method.

- **Check equilibrium: Base shear**

- <u>From the equivalent force method:</u>

The equivalent static lateral force method is a simplified technique to substitute the effect of dynamic loading of an expected earthquake by a static force distributed laterally on a structure for design purposes.

From ETABS:

Table 3.4-7: Base reaction by equivalent method.

Base Reactions

Units: As Noted Hidden Columns: No Sort: None

Filter: ([Output Case] = 'EQx' OR [Output Case] = 'EQy')

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
EQx	LinStatic	Step By Step	1	-15577.2717	-3.706E-06	0.0001	0.0026	-263929.7667	333522.3936
EQx	LinStatic	Step By Step	2	-15577.2717	-3.839E-06	0.0001	0.0028	-263929.7666	362954.5867
EQx	LinStatic	Step By Step	3	-15577.2717	-3.573E-06	0.0001	0.0023	-263929.7668	304090.2006
EQy	LinStatic	Step By Step	1	-4.044E-06	-13892.8086	2.884E-05	235389.4713	0.0004	-95363.5556
EQy	LinStatic	Step By Step	2	-6.584E-06	-13892.8086	2.041E-05	235389.4711	0.0002	-127743.7555
EQy	LinStatic	Step By Step	3	-1.504E-06	-13892.8086	3.727E-05	235389.4716	0.0005	-62983.3558

- ✓ E_{Qx}= 15,577.27 KN
- ✓ E_{Qy}= 13,892.8 KN

From manual calculation:

$V=C_s*W$ (ASCE 7-22 equation 12.8-1)

Where W= D+SD+0.3SL

D: Dead load.

SD: Superimposed Dead load.

SL: Storage live load.

Table 3.4-8: Total load from (D+SD+0.3SL).

Base Reactions

Units: As Noted Hidden Columns: No Sort: None

Filter: ([Output Case] = 'D+0.3SL')

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
D+0.3SL	Combination			-4.707E-06	8.707E-07	101716.0558	2188981.9853	-676770.5874	0.0001

W=101,716.1 KN

$Cs_1=\frac{SDs*Ie}{R}$ (ASCE 7-22 equation 12.8-3)

$T \le T_L \quad Cs_2=\frac{SD1*Ie}{R*T}$ (ASCE 7-22 equation 12.8-4)

$Cs_{3\ min} = 0.044 S_{DS} I_e \ge 0.01$ (ASCE 7-22 equation 12.8-6)

Table 3.4-9: Base shear.

SDs	0.758
SD1	0.2171
R	6
Ie	1.25
W	101716.1
Ta	0.58
cu	1.483

Axis	T from etabs	T limit	T to be use	Cs1	Cs2	Cs3	Cs final	V	Vetabs	Error
x	0.276	0.860	0.276	0.158	0.164	0.042	0.158	16062.667	15577.300	3.022
y	0.322	0.860	0.322	0.158	0.140	0.042	0.140	14287.374	13892.800	2.762

V_x=0.158*101,716.1 = 16,062.67 KN

V_y=0.140*101,716.1 = 14,287.4 KN

Difference percentage $E_{Qx} = \frac{16062.7-15577.3}{16062.7} * 100 = 3\ \% < 5\%$ ………… OK

Difference percentage $E_{Qy} = \frac{14287.4-13982.8}{14287.4} * 100 = 2.8\ \% < 5\%$ …………OK

- <u>Check base shear by response spectrum curve (dynamic method):</u>

The dynamic seismic values should be equal or greater to the value of earthquakes in the static method, but the values were not equal on ETABS, so the value of the scale factor for each of the dynamic earthquake forces in X and Y directions must be changed until their values are equal or greater to the static results.

Figures 3.4-18 to 3.4-21 show the scale factor for earthquake dynamic loads before and after the changes: -

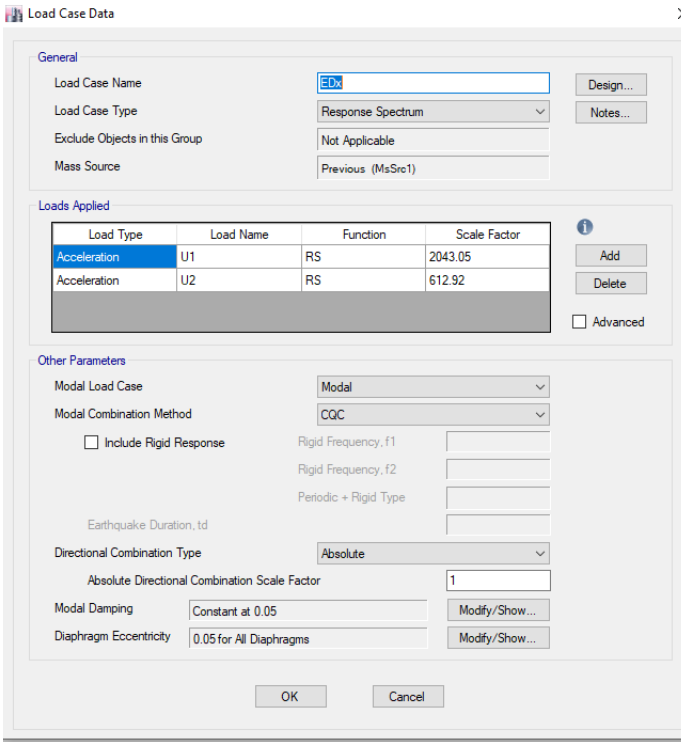


Figure 3.4-17: Definition of EDx before modification.

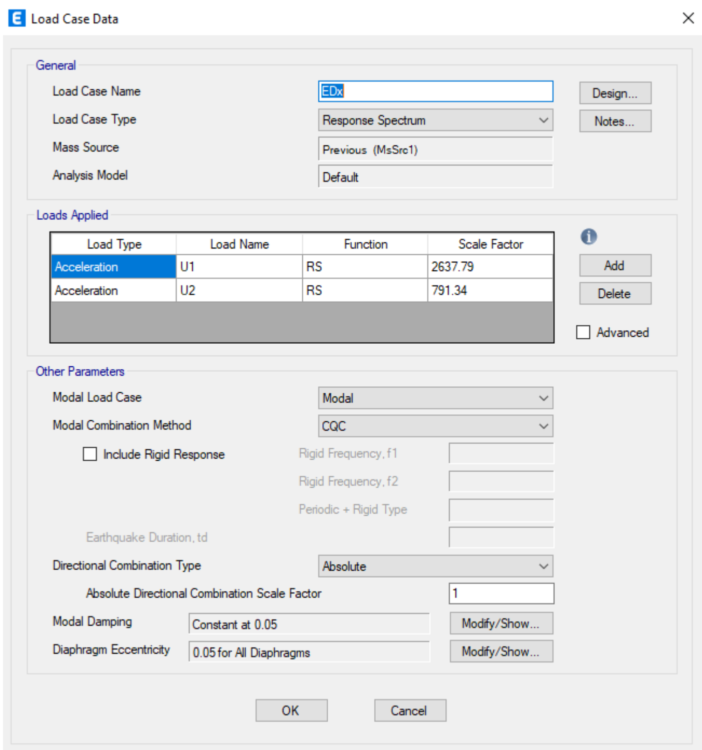


Figure 3.4-18: Definition of EDx after modification.

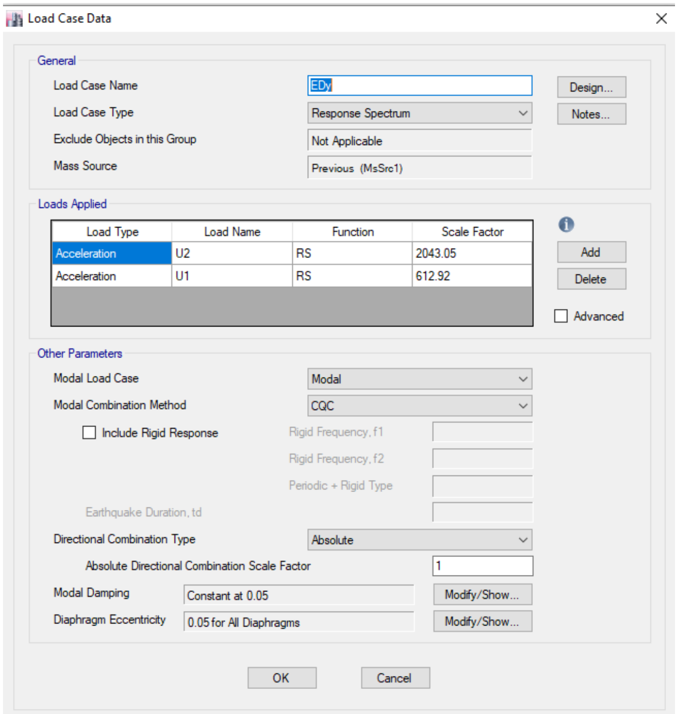


Figure 3.4-19: Definition of EDy before modification.

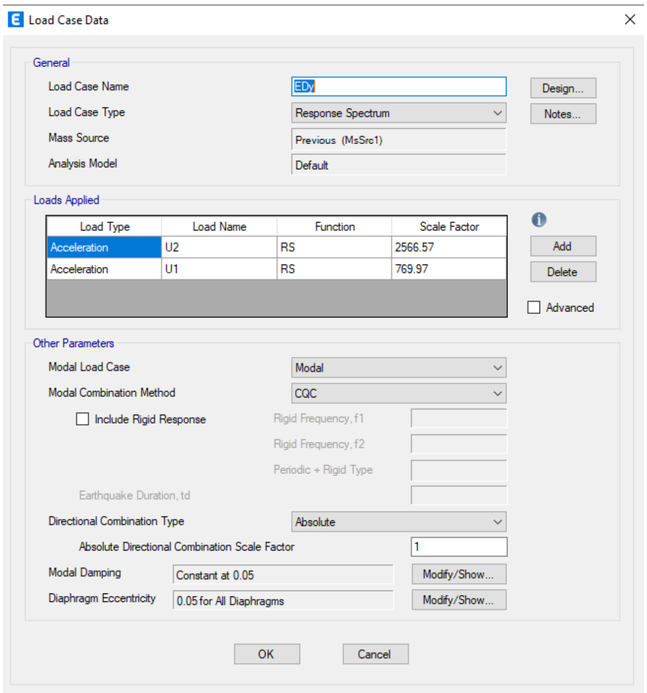


Figure 3.4-20: Definition of EDy after modification.

- Check base shear from the response spectrum method:

In X-direction:

- From ETABS due to $E_{QX} \Longrightarrow F_x$ = 15,576.606 KN
- From ETABS due to $E_{DX} \Longrightarrow F_x$ =15,576.58 KN
- From manual $\Longrightarrow V_x$ = 16,062.67 KN

Difference percentage between (Manual V_x & E_{DX}) = (16062.67-15576.58 /16062.67)

= 3 % < 5% $\Longrightarrow$ Ok

In Y-direction:

- From ETABS due to $E_{Qy} \Longrightarrow F_y$ = 13,892.2149 KN
- From ETABS due to $E_{Dy} \Longrightarrow F_y$ =13,892.19 KN
- From manual $\Longrightarrow V_y$= 14,287.4 KN

Difference percentage between (Manual V_y & E_{Dy}) = (14287.4−13892.19/14287.4)

= 2.7 % < 5% $\Longrightarrow$ Ok

Table 3.4-10 shows the base shear value in X and Y directions, according to the dynamic and static methods after changing the scaling factor.

Table 3.4-10: Base shear value F_x & F_y.

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	Step Type	Step Number	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
EQx	LinStatic	Step By Step	1	-15576.606	1.957E-06	4.952E-05	0.0016	-263917.5773	333506.4448
EQx	LinStatic	Step By Step	2	-15576.606	2.188E-06	0.0001	0.0019	-263917.5772	373528.2004
EQx	LinStatic	Step By Step	3	-15576.606	1.725E-06	4.295E-05	0.0014	-263917.5775	293484.6892
EQy	LinStatic	Step By Step	1	-4.225E-06	-13892.2149	1.532E-05	235378.5995	0.0002	-95365.9893
EQy	LinStatic	Step By Step	2	-1.017E-05	-13892.2149	6.012E-06	235378.5992	-2.778E-05	-152346.5256
EQy	LinStatic	Step By Step	3	1.719E-06	-13892.2149	2.462E-05	235378.5998	0.0004	-38385.4531
EDx	LinRespSpec	Max		15576.5802	4887.2757	0.0001	77429.214	260693.8226	443146.9018
EDy	LinRespSpec	Max		5134.4647	13892.1969	4.082E-05	234111.9173	82926.1086	268397.8151

- **Story forces**

  The induced lateral seismic force at any level shall be determined from the following equations:

$$F_x = C_{vx} V$$ (ASCE 7-22 equation 12.8-12)

$$C_{vx} = \frac{w_x h_x^k}{\sum_{n}^{i=1} w_i h_i^k}$$ (ASCE 7-22 equation 12.8-13)

Where:

C_{vx}: vertical distribution factor

V: total design lateral force or shear at the base of the structure (kN)

W_i and W_x: the portion of the total effective seismic weight of the structure (W) located or assigned to Level i or x .

h_i and h_x: the height (m) from the base to Level i or x

k: an exponent related to the structure period as follows:

- for structures having a period of 0.5 s or less, k = 1
- for structures having a period of 2.5 s or more, k = 2

X direction (E_{Qx})

T_x= 0.275 sec ……………. K=1

Table 3.4-11: Story forces in X direction.

Manual						
Floor	Height (m) h_i	W_i	$W_i*(h_i^k)$	C_{vx}	F_x	V_x
Roof	26.6	2791.07	74242.46	0.0566	880.70	880.70
5th	23.4	10367.5	242599.5	0.1848	2877.82	3758.51
4th	20.2	13609.9	274920	0.2094	3261.22	7019.73
3rd	17	13609.9	231368.3	0.1763	2744.59	9764.32
2nd	13.8	13609.9	187816.6	0.1431	2227.96	11992.28
1st	10.6	13609.9	144264.9	0.1099	1711.33	13703.61
Ground	7.4	14391	106493.4	0.0811	1263.27	14966.88
Basement	3.2	15925.2	50960.64	0.0388	604.52	15571.40
		97914.37	1312666	1	15571.40	

Table 3.4-12: Story forces in X direction form ETABS.

Story Response

Story	Elevation m	Location	X-Dir kN	Y-Dir kN
Roof	23.4	Top	921.7435	0
5th	20.2	Top	2867.8617	0
4th	17	Top	3244.7159	0
3rd	13.8	Top	2731.812	0
2nd	10.6	Top	2217.4677	0
1st	7.4	Top	1703.2723	0
Ground	4.2	Top	1258.5789	0
Basement	0	Top	607.9509	0
Base	-3.2	Top	0	0

For example; for the first floor:

Difference % = $\frac{1711.33-1703.27}{1711.33} * 100$ = 0.47% < 5% …………. OK

Table 3.4-13: Difference percentage in each floor in X-direction.

Floor	Difference %
Roof	4.66
5th	0.35
4th	0.51
3rd	0.47
2nd	0.47
1st	0.47
Ground	0.37
Basement	0.57

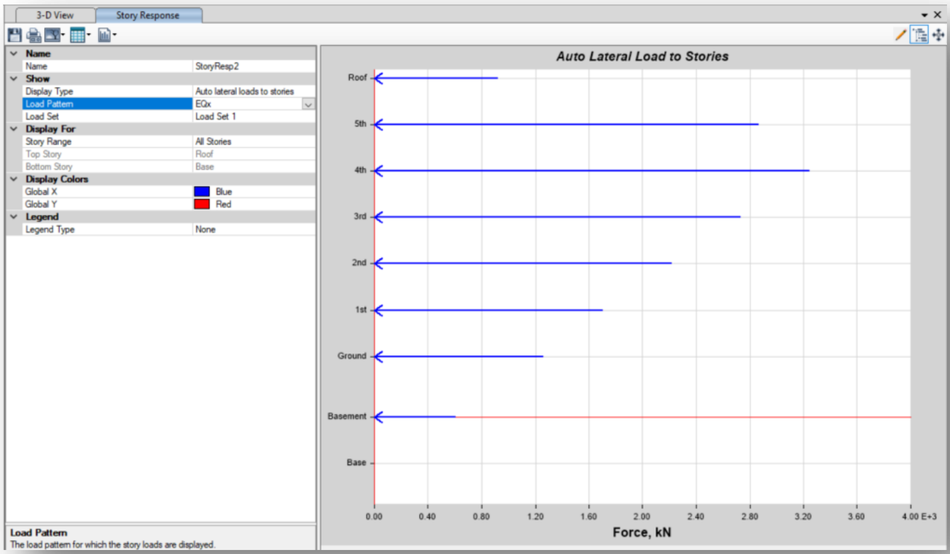


Figure 3.4-21: Auto lateral load to stories in X-direction.

Y direction (E_{Qy})

T_x= 0.322 sec ……………. K=1

Table 3.4-14: Story forces in Y direction.

Manual						
Floor	Height (m) hi	Wi	Wi*(hi^k)	Cvx	Fy	Vy
Roof	26.6	2791.07	74242.46	0.0566	783.02	783.02
5th	23.4	10367.5	242599.5	0.1848	2558.66	3341.68
4th	20.2	13609.9	274920	0.2094	2899.54	6241.22
3rd	17	13609.9	231368.3	0.1763	2440.20	8681.42
2nd	13.8	13609.9	187816.6	0.1431	1980.87	10662.29
1st	10.6	13609.9	144264.9	0.1099	1521.54	12183.83
Ground	7.4	14391	106493.4	0.0811	1123.17	13307.00
Basement	3.2	15925.2	50960.64	0.0388	537.47	13844.47
		97914.37	1312666	1	13844.47	

Table 3.4-15: Story forces in Y direction form ETABS.

Story Response

Story	Elevation m	Location	X-Dir kN	Y-Dir kN
Roof	23.4	Top	0	819.5169
5th	20.2	Top	0	2549.7994
4th	17	Top	0	2884.8584
3rd	13.8	Top	0	2428.8384
2nd	10.6	Top	0	1971.5378
1st	7.4	Top	0	1514.3696
Ground	4.2	Top	0	1118.9953
Basement	0	Top	0	540.5257
Base	-3.2	Top	0	0

For example:

Difference percentage= $\frac{1521.54-1514.37}{1521.54} * 100 = 0.47\ \% < 5\%$ ………… OK

Table 3.4-16: Difference percentage in each floor in Y-direction

Floor	Difference %
Roof	4.66
5th	0.35
4th	0.51
3rd	0.47
2nd	0.47
1st	0.47
Ground	0.37
Basement	0.57

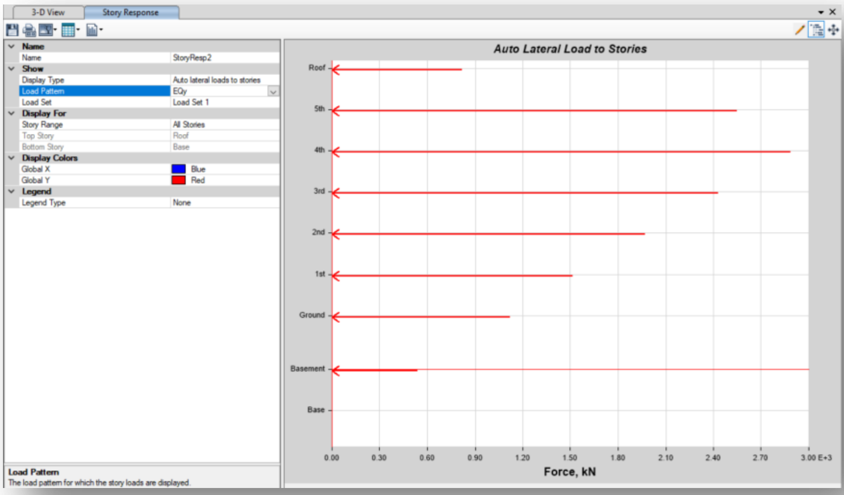


Figure 3.4-22: Auto lateral load to stories in Y-direction.

- Period check:

The periods from ETABS can be taken if they are less than the limits from the ASCE 7-22

The limitation of period ⟹ $T_a \times C_u = (0.58 \times 1.4829) = 0.86$ sec

From ETABS:

T_x =0.276 sec ⟹ $T_X < T_a$ ⟹ ok

T_y = 0.322 sec ⟹ $T_Y < T_a$ ⟹ ok

Table 3.4-17: Time period.

Modal Participating Mass Ratios | Base Reactions

1 of 12 Reload Apply

Case	Mode	Period sec	UX	UY
Modal	1	0.322	0.0003	0.7507
Modal	2	0.276	0.7489	0.0005
Modal	3	0.206	0.0323	0.0034
Modal	4	0.109	0.0013	0.1222
Modal	5	0.099	0.0991	0.0068
Modal	6	0.092	0.0072	0.0002
Modal	7	0.074	0.0036	0.0056
Modal	8	0.067	0.0153	0.0168
Modal	9	0.064	0.0161	0.0233
Modal	10	0.059	0.01	0.0006
Modal	11	0.055	0.0061	0.0017
Modal	12	0.053	0.0043	0.0105

- **Type of lateral force structural system: -**

  To determine the resistance of the system, the horizontal forces should be inserted, then the readings of the horizontal forces from walls and columns will be compared with the reading from walls only, the percentage of them must be greater than 98%.

X direction (E_{Qx}):

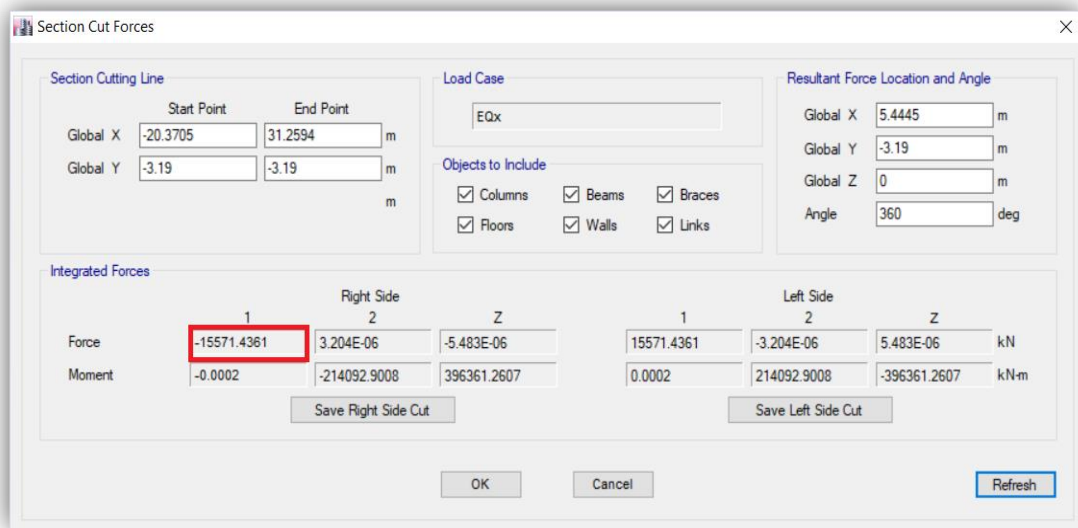


Figure 3.4-23: Total vertical reactions in X direction.

Total V_x = 15571.43 KN

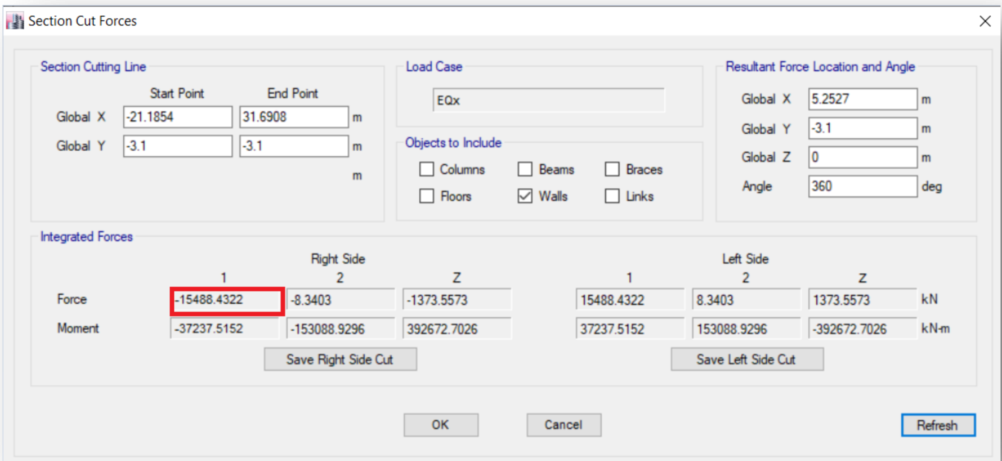


Figure 3.4-24: Wall vertical reactions in X direction.

From only walls $\Longrightarrow V_x$=15488.4

Percentage of load resisting by walls = $\frac{15488.4}{15571.4} * 100 = 99.4$ %

So, the system is shear wall resisting system in the x direction (E_{Qx}).

Y-direction (E_{Qy}) :

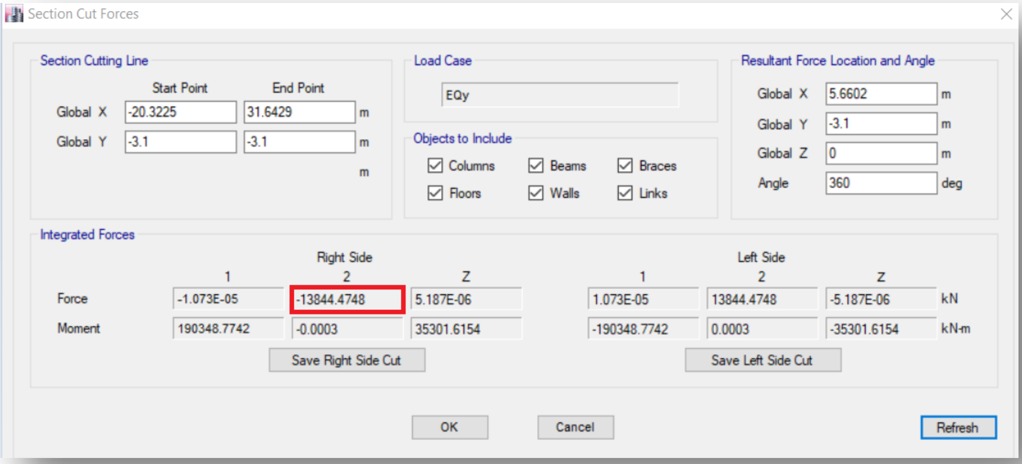


Figure 3.4-25: Total vertical reactions in Y direction.

Total V_y= 13844.5 KN

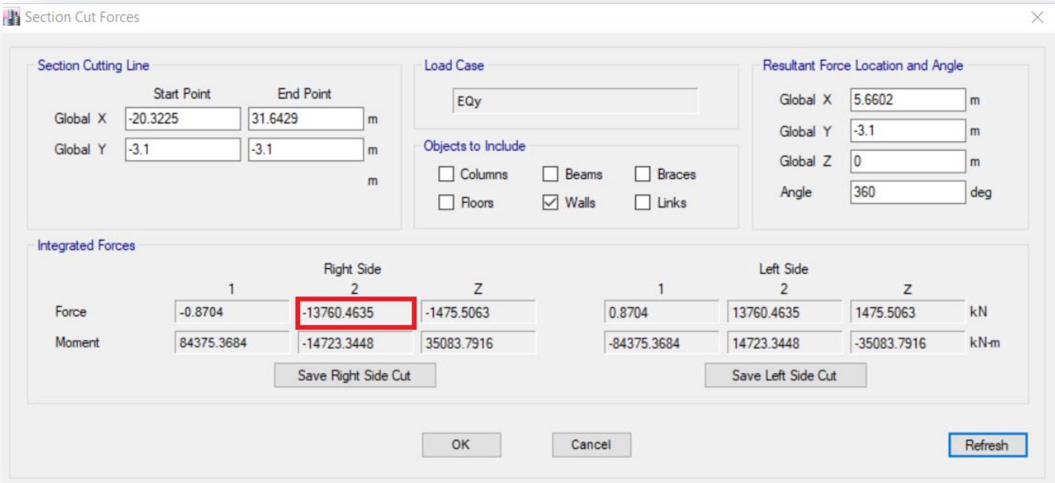


Figure 3.4-26: Wall vertical reactions in Y direction.

From only walls $\Longrightarrow$ V_y=13760.5 KN

Percentage of load resisting by walls = $\frac{13760.5}{13844.5} * 100 = 99.3$ %

So, the system is shear wall resisting system in the y direction (E_{Qy}).

➢ Check mass participation ratio for modal analysis:

Number of modes in analysis shall be conducted to determine the natural modes of vibration for the structure. The analysis shall include a sufficient number of modes to obtain a combined modal mass participation of 100% of the structure's mass.

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.

Table 3.4-18: Mass participation ratio.

Case	Mode	Sum UX	Sum UY
Modal	1	0.0003	0.7507
Modal	2	0.7492	0.7511
Modal	3	0.7816	0.7545
Modal	4	0.7829	0.8767
Modal	5	0.882	0.8836
Modal	6	0.8892	0.8838
Modal	7	0.8928	0.8895
Modal	8	0.908	0.9062
Modal	9	0.9241	0.9296
Modal	10	0.9341	0.9301
Modal	11	0.9402	0.9318
Modal	12	0.9445	0.9423

⟹The structure needed 12 modes in each direction X and Y, to collect mass participation ratio more than 90%.

➢ Check horizontal irregularity:

Accidental Torsion Irregularities: where diaphragms are not flexible, accidental torsion shall be applied to all structures for determination if a horizontal irregularity exists.

Torsional Irregularity: the maximum story drift including accidental torsion 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 Ratio (TIR): A Torsional Irregularity Ratio shall be calculated for each story and for each accidental torsion case:

TIR= $\Delta_{max} / \Delta_{avg}$

where Δ_{max} is the maximum story drift at the building's edge subjected to lateral forces using the equivalent lateral force procedure with the application of accidental torsion and Ax = 1.0; and Δ_{avg} is the average of the story drifts at the two opposing edges of the building.

Amplification factor of Accidental Torsion (A_x) = $(\Delta_{max} / 1.2\Delta_{avg})^2$

Where:

- Δ_{max} is the maximum displacement at level x, computed assuming Ax = 1
- Δ_{avg} is the average of the displacements at the extreme points of the structure at level x, computed assuming Ax = 1.
- For the purpose of computing Δ_{max} and Δ_{avg}, it shall be permitted to assume the diaphragm is rigid.
- The torsional amplification factor (Ax), shall not be less than 1 and is not required to exceed 3.0

In X-direction:

Table 3.4-19: Diaphragm maximum and average drift according to EQx.

Diaphragm Max Over Avg Drifts

File Edit Format-Filter-Sort Select Options

Units: As Noted Hidden Columns: No Sort: None Diaphragm Max Over Avg Drifts

Filter: ([Output Case] = 'EQx') AND ([Item] = 'Diaph D1 X')

Story	Output Case	Case Type	Step Type	Step Number	Item	Max Drift	Avg Drift	Ratio
Roof	EQx	LinStatic	Step By Step	1	Diaph D1 X	0.000165	0.000125	1.321
Roof	EQx	LinStatic	Step By Step	2	Diaph D1 X	0.00017	0.000126	1.353
Roof	EQx	LinStatic	Step By Step	3	Diaph D1 X	0.000161	0.000123	1.303
5th	EQx	LinStatic	Step By Step	1	Diaph D1 X	0.000161	0.00012	1.342
5th	EQx	LinStatic	Step By Step	2	Diaph D1 X	0.000171	0.000122	1.402
5th	EQx	LinStatic	Step By Step	3	Diaph D1 X	0.000152	0.000119	1.281
4th	EQx	LinStatic	Step By Step	1	Diaph D1 X	0.000201	0.000162	1.238
4th	EQx	LinStatic	Step By Step	2	Diaph D1 X	0.000213	0.000163	1.312
4th	EQx	LinStatic	Step By Step	3	Diaph D1 X	0.00019	0.000163	1.168
3rd	EQx	LinStatic	Step By Step	1	Diaph D1 X	0.000245	0.000205	1.195
3rd	EQx	LinStatic	Step By Step	2	Diaph D1 X	0.000267	0.000208	1.286
3rd	EQx	LinStatic	Step By Step	3	Diaph D1 X	0.00023	0.000206	1.118
2nd	EQx	LinStatic	Step By Step	1	Diaph D1 X	0.000278	0.000237	1.177
2nd	EQx	LinStatic	Step By Step	2	Diaph D1 X	0.000305	0.00024	1.272
2nd	EQx	LinStatic	Step By Step	3	Diaph D1 X	0.000262	0.000238	1.099
1st	EQx	LinStatic	Step By Step	1	Diaph D1 X	0.000313	0.000266	1.176
1st	EQx	LinStatic	Step By Step	2	Diaph D1 X	0.000324	0.000262	1.24
1st	EQx	LinStatic	Step By Step	3	Diaph D1 X	0.000301	0.00027	1.113
Ground	EQx	LinStatic	Step By Step	1	Diaph D1 X	0.000278	0.000245	1.137
Ground	EQx	LinStatic	Step By Step	2	Diaph D1 X	0.000274	0.000246	1.113
Ground	EQx	LinStatic	Step By Step	3	Diaph D1 X	0.000299	0.000248	1.203
Basement	EQx	LinStatic	Step By Step	1	Diaph D1 X	0.000194	0.000152	1.277
Basement	EQx	LinStatic	Step By Step	2	Diaph D1 X	0.00021	0.000154	1.359
Basement	EQx	LinStatic	Step By Step	3	Diaph D1 X	0.000181	0.00015	1.207

The max ratio between Δ_{max} and Δ_{avg} = 1.402 → Ax = $(1.402/1.2)^2$ = 1.365

Then, the eccentricity in X direction = 0.05 *1.365 = 0.068

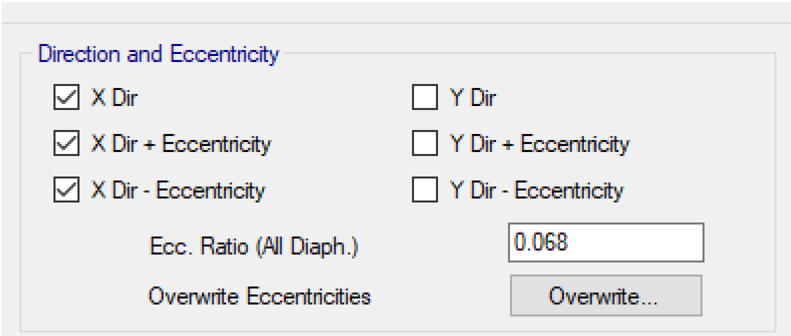


Figure 3.4-27: Eccentricities ratio in X direction for equivalent analysis.

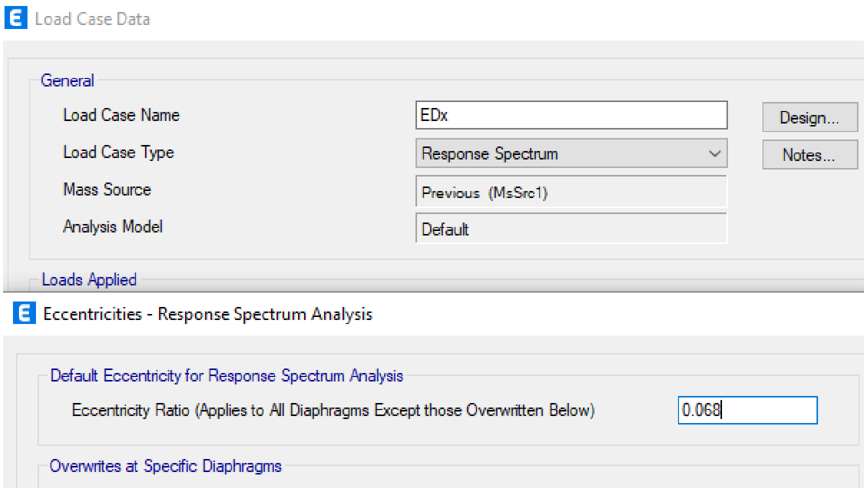


Figure 3.4-28: Eccentricities ratio in X direction for response spectrum analysis.

In Y-direction:

Table 3.4-20: Diaphragm maximum and average drift according to EQy.

Diaphragm Max Over Avg Drifts

File Edit Format-Filter-Sort Select Options

Units: As Noted Hidden Columns: No Sort: None Diaphragm Max Over Avg Drifts

Filter: ([Output Case] = 'EQy') AND ([Item] = 'Diaph D1 Y')

Story	Output Case	Case Type	Step Type	Step Number	Item	Max Drift	Avg Drift	Ratio
Roof	EQy	LinStatic	Step By Step	1	Diaph D1 Y	0.000159	0.000102	1.567
Roof	EQy	LinStatic	Step By Step	2	Diaph D1 Y	0.000158	0.000103	1.543
Roof	EQy	LinStatic	Step By Step	3	Diaph D1 Y	0.000161	0.000101	1.594
5th	EQy	LinStatic	Step By Step	1	Diaph D1 Y	0.000239	0.000197	1.215
5th	EQy	LinStatic	Step By Step	2	Diaph D1 Y	0.000239	0.000194	1.232
5th	EQy	LinStatic	Step By Step	3	Diaph D1 Y	0.000243	0.000191	1.271
4th	EQy	LinStatic	Step By Step	1	Diaph D1 Y	0.000269	0.000243	1.11
4th	EQy	LinStatic	Step By Step	2	Diaph D1 Y	0.000281	0.000248	1.133
4th	EQy	LinStatic	Step By Step	3	Diaph D1 Y	0.000269	0.000231	1.167
3rd	EQy	LinStatic	Step By Step	1	Diaph D1 Y	0.000313	0.00029	1.079
3rd	EQy	LinStatic	Step By Step	2	Diaph D1 Y	0.000327	0.00029	1.127
3rd	EQy	LinStatic	Step By Step	3	Diaph D1 Y	0.00031	0.000273	1.135
2nd	EQy	LinStatic	Step By Step	1	Diaph D1 Y	0.000343	0.000321	1.069
2nd	EQy	LinStatic	Step By Step	2	Diaph D1 Y	0.000361	0.000319	1.13
2nd	EQy	LinStatic	Step By Step	3	Diaph D1 Y	0.000348	0.000305	1.139
1st	EQy	LinStatic	Step By Step	1	Diaph D1 Y	0.000385	0.000335	1.147
1st	EQy	LinStatic	Step By Step	2	Diaph D1 Y	0.000389	0.000327	1.189
1st	EQy	LinStatic	Step By Step	3	Diaph D1 Y	0.000425	0.000355	1.196
Ground	EQy	LinStatic	Step By Step	1	Diaph D1 Y	0.00031	0.000248	1.249
Ground	EQy	LinStatic	Step By Step	2	Diaph D1 Y	0.000352	0.000261	1.348
Ground	EQy	LinStatic	Step By Step	3	Diaph D1 Y	0.000334	0.000268	1.244
Basement	EQy	LinStatic	Step By Step	1	Diaph D1 Y	0.000216	0.000166	1.303
Basement	EQy	LinStatic	Step By Step	2	Diaph D1 Y	0.000219	0.000161	1.359
Basement	EQy	LinStatic	Step By Step	3	Diaph D1 Y	0.000213	0.000168	1.273

The max ratio between Δ_{max} and Δ_{avg} = 1.594→ Ax = $(1.594/1.2)^2$ = 1.764

Then, the eccentricity in Y direction = 0.05 *1.764 = 0.088

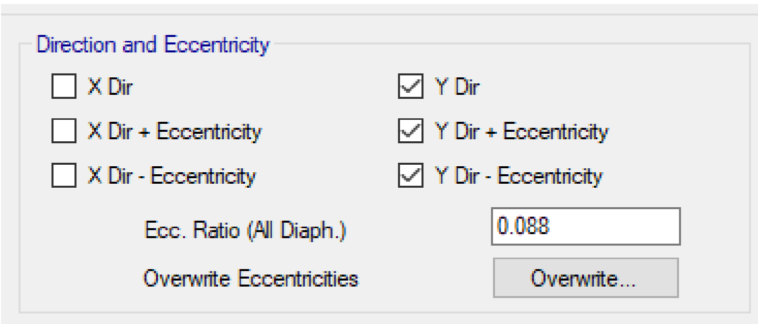


Figure 3.4-29: Eccentricities ratio in Y direction for equivalent analysis.

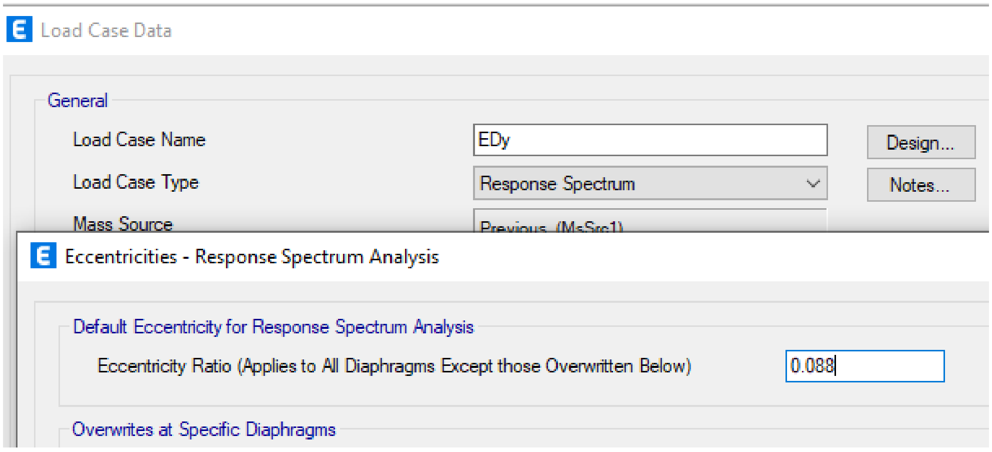


Figure 3.4-30: Eccentricities ratio in Y direction for response spectrum analysis.

- Vertical Irregularity:

1. 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, where there are at least three stories above, less than 80% of the average stiffness of the three stories above.
2. 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, where there are at least three stories above, less than 70% of the average stiffness of the three stories above.

In X-direction:

Table 3.4-21: Story stiffness according to EQx.

Story	Output case	Stiffness in X	Check(lateral stiffness is less than 70% of that in the story above)	Check Extreme(lateral stiffness is less than 60% of that in the story above)
Basement	Eqx	32126010.49	OK	OK
Ground	Eqx	16057736.97	OK	OK
1st	Eqx	16115349.1	OK	OK
2nd	Eqx	14376068.22	OK	OK
3rd	Eqx	13863869.97	OK	OK
4th	Eqx	11919730.16	OK	OK
5th	Eqx	8648593.542	OK	OK
roof	Eqx	2300389.1	OK	OK

In Y-direction:

Table 3.4-22: Story stiffness according to EQy.

Story	Output case	Stiffness in Y	Check(lateral stiffness is less than 70% of that in the story above)	Check Extreme(lateral stiffness is less than 60% of that in the story above)
Basement	Eqy	26205373	OK	OK
Ground	Eqy	13010599.88	OK	OK
1st	Eqy	11212618.1	OK	OK
2nd	Eqy	9657138.157	OK	OK
3rd	Eqy	8942712.684	OK	OK
4th	Eqy	7477884.201	OK	OK
5th	Eqy	5052166.295	OK	OK
roof	Eqy	1935759.728	OK	OK

3. 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 3.4-23: story weight check.

Story	Weight of story	check status with respect to story above
Basement	122064.5355	ok
Ground	102671.8666	ok
1st	84199.2437	ok
2nd	67412.5465	ok
3rd	50625.8493	ok
4th	33839.1512	ok
5th	17046.8403	ok
Roof	4186.2684	ok

- Check type of diaphragm (slab) rigid or flexible :

Diaphragms of concrete slabs or concrete-filled metal deck with span-to-depth ratios of 3 or less in structures that do not have a Type 2, 3, 4, or 5 horizontal structural irregularity are permitted to be idealized as rigid, and if the ratio $\frac{\Delta max}{\Delta avg}$ is less than 2, then the diaphragm (slab) is classified as rigid.

Table 3.4-24: Story maximum over average drift according to EQx.

Story Max Over Avg Drifts

File Edit Format-Filter-Sort Select Options

Units: As Noted Hidden Columns: Yes Sort: None

Filter: ([Output Case] = 'EQx') AND ([Direction] = 'X')

Story	Output Case	Direction	Max Drift mm	Avg Drift mm	Ratio
Roof	EQx	X	0.529	0.401	1.321
Roof	EQx	X	0.55	0.403	1.367
Roof	EQx	X	0.508	0.391	1.299
5th	EQx	X	0.516	0.385	1.342
5th	EQx	X	0.558	0.392	1.423
5th	EQx	X	0.478	0.379	1.261
4th	EQx	X	0.643	0.519	1.238
4th	EQx	X	0.703	0.524	1.343
4th	EQx	X	0.597	0.522	1.143
3rd	EQx	X	0.783	0.655	1.195
3rd	EQx	X	0.883	0.669	1.319
3rd	EQx	X	0.725	0.663	1.094
2nd	EQx	X	0.891	0.757	1.177
2nd	EQx	X	1.01	0.773	1.306
2nd	EQx	X	0.825	0.768	1.075
1st	EQx	X	1	0.851	1.176
1st	EQx	X	1.052	0.832	1.264
1st	EQx	X	0.949	0.869	1.092
Ground	EQx	X	1.168	0.977	1.195
Ground	EQx	X	1.148	0.993	1.156
Ground	EQx	X	1.292	1.014	1.274
Basement	EQx	X	0.619	0.485	1.277
Basement	EQx	X	0.69	0.496	1.389
Basement	EQx	X	0.567	0.479	1.184

Table 3.4-25: Story maximum over average drift according to EQy

Story Max Over Avg Drifts

File Edit Format-Filter-Sort Select Options

Units: As Noted Hidden Columns: Yes Sort: None

Filter: ([Output Case] = 'EQy') AND ([Direction] = 'Y')

Story	Output Case	Direction	Max Drift mm	Avg Drift mm	Ratio
Roof	EQy	Y	0.51	0.326	1.567
Roof	EQy	Y	0.504	0.33	1.528
Roof	EQy	Y	0.517	0.32	1.615
5th	EQy	Y	0.766	0.63	1.215
5th	EQy	Y	0.788	0.621	1.27
5th	EQy	Y	0.787	0.598	1.316
4th	EQy	Y	0.862	0.776	1.11
4th	EQy	Y	0.927	0.782	1.186
4th	EQy	Y	0.875	0.717	1.221
3rd	EQy	Y	1	0.927	1.079
3rd	EQy	Y	1.083	0.916	1.183
3rd	EQy	Y	1.042	0.861	1.21
2nd	EQy	Y	1.099	1.027	1.069
2nd	EQy	Y	1.204	1.019	1.181
2nd	EQy	Y	1.187	0.97	1.224
1st	EQy	Y	1.27	1.093	1.162
1st	EQy	Y	1.324	1.059	1.25
1st	EQy	Y	1.509	1.161	1.3
Ground	EQy	Y	1.303	1.024	1.272
Ground	EQy	Y	1.613	1.121	1.438
Ground	EQy	Y	1.479	1.17	1.264
Basement	EQy	Y	0.691	0.53	1.303
Basement	EQy	Y	0.706	0.503	1.403
Basement	EQy	Y	0.676	0.509	1.329

In each x and y direction, the ratio is less than 2, so the diaphragm is defined as rigid.

As the diaphragm is rigid, the lateral forces in the floor are transferred to the vertical elements based on their stiffnesses.

- P-Delta Effects: P-delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects are not required to be considered where the stability coefficient θ, as determined by the following equation is equal to or less than 0.1:

$$\theta = \frac{Px/hsx}{Vx/\Delta xe} \qquad \text{(ASCE 7-22 equation 12.8-18)}$$

Where:

- θ: Stability coefficient
- P_x = Total vertical design load at and above level x (where computing P_x, no individual load factor need exceed 1.0)
- V_x/Δ_{xe} = Story stiffness at level x, calculated as the ratio of the seismic design shear, V_x, divided by the corresponding elastic story drift, Δ_{xe}
- h_{sx} = Story height below level x

The stability coefficient (θ) shall not exceed θ_{max}, determined as follows:

$$\theta_{max} = (0.5 / \beta * C_d) \leq 0.25 \qquad \text{(ASCE 7-22 equation 12.8-18)}$$

Where:

C_d: Deflection amplification factor.

β: is the ratio of shear demand to design shear capacity for the story between levels (x) and (x – 1).

The value of β is permitted to be conservatively taken as 1.0, and shall not be taken less than ($1.25/\Omega_0$).

$\theta_{max} = (0.5 / 1*5) = 0.1 \leq 0.25$

The stability coefficient (θ) $\Longrightarrow$ $0.1 < \theta < \theta_{max}$

In X-direction:

P_x represented this load combination (p-delta in x): D+SD+L+ L_{storge} +L_{roof} +EQ_x

Table 3.4-26: Effect of p-delta in X direction

TABLE: Story Forces								
Story	**hight**	**Output Case**	**Case Type**	**Step Type**	**Location**	**P**	**Stiff X**	**Ø**
						kN	kN/m	(Hight/P)/Stiffness
Roof	2.5	P-delta in X	Combination	Max	Bottom	4186.2684	2289468.675	0.000731396
5th	3.5	P-delta in X	Combination	Max	Bottom	17046.8403	8164440.157	0.000596554
4th	3.5	P-delta in X	Combination	Max	Bottom	33839.1512	11480660.35	0.00084214
3rd	3.5	P-delta in X	Combination	Max	Bottom	50625.8493	13400814.23	0.001079377
2nd	3.5	P-delta in X	Combination	Max	Bottom	67412.5465	14267213.66	0.001349999
1st	3.5	P-delta in X	Combination	Max	Bottom	84199.2438	16474668.07	0.001460237
Ground	4.5	P-delta in X	Combination	Max	Bottom	102671.867	15819620.5	0.001442258
Basement	3.5	P-delta in X	Combination	Max	Bottom	122064.536	31381340.11	0.001111348

All values of θ in X direction are less than 0.1 → neglect effect of P-delta.

In Y-direction:

P_y represented this load combination (p-delta in y): D+SD+L+ L_{storge} +L_{roof} +EQ

Table 3.4-27: Effect of p-delta in Y direction.

TABLE: Story Forces								
Story	**hight**	**Output Case**	**Case Type**	**Step Type**	**Location**	**P**	**Stiff Y**	**Ø**
						kN	kN/m	(Hight/P)/Stiffness
Roof	2.5	P-delta in Y	Combination	Max	Bottom	4186.2684	1911920.648	0.000875825
5th	3.5	P-delta in Y	Combination	Max	Bottom	17046.8403	4990075.95	0.000976042
4th	3.5	P-delta in Y	Combination	Max	Bottom	33839.1512	7383240.083	0.001309497
3rd	3.5	P-delta in Y	Combination	Max	Bottom	50625.8493	9000980.214	0.001606995
2nd	3.5	P-delta in Y	Combination	Max	Bottom	67412.5465	10041139.05	0.001918182
1st	3.5	P-delta in Y	Combination	Max	Bottom	84199.2437	11545332.81	0.002083693
Ground	4.5	P-delta in Y	Combination	Max	Bottom	102671.867	11903126.18	0.001916805
Basement	3.5	P-delta in Y	Combination	Max	Bottom	122064.536	27622145.96	0.001262595

All values of θ in X direction are less than 0.1 → neglect effect of P-delta

- Story drift (Δ): is the lateral displacement of a floor relative to the floor below, and the story drift ratio is the story drift divided by the story height.

  The design story drift (Δ) shall not exceed the allowable story drift Δ_a as obtained from Table 12.12-1 in ASCE7-22 as shown in table 3.4-28.

Table 3.4-28: Allowable story drift Δ_a.

Table 12.12-1. Allowable Story Drift, Δ_a.

Structure	Risk Category I or II	Risk Category III	Risk Category 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, and ceilings that have been designed to accommodate the drifts associated with the Design Earthquake Displacements	$0.025h_{sx}$ [a]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures [b]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

In this project, $\Delta_a = 0.015\ h_{sx}$, where h_{sx} is the story height below level x.

In X-direction:

Table 3.4-29: Story drift in X direction according to equivalent method (EQx).

TABLE: Story Drifts				
Story	Output Case	Direction	Elastic drift	Inelastic drift (Cd*elastic drift)/Ie
Roof	EQx	X	0.000172	0.000688
5th	EQx	X	0.000174	0.000696
4th	EQx	X	0.00022	0.00088
3rd	EQx	X	0.000276	0.001104
2nd	EQx	X	0.000316	0.001264
1st	EQx	X	0.000329	0.001316
Ground	EQx	X	0.000273	0.001092
Basement	EQx	X	0.000216	0.000864

✓ All values of drift ratio in x direction less than 0.015 → story drift ok

In Y-direction:

Table 3.4-30: Story drift in Y direction according to equivalent method (EQy).

TABLE: Story Drifts				
Story	Output Case	Direction	Elastic drift	Inelastic drift (Cd*elastic drift)/Ie
Roof	EQx	Y	0.000157	0.000628
5th	EQx	Y	0.000246	0.000984
4th	EQx	Y	0.00029	0.00116
3rd	EQx	Y	0.000339	0.001356
2nd	EQx	Y	0.000376	0.001504
1st	EQx	Y	0.000414	0.001656
Ground	EQx	Y	0.000384	0.001536
Basement	EQx	Y	0.000221	0.000884

- ✓ All values of drift ratio in y direction are less than 0.015 → story drift ok

3.5 Deflection computations: -

The deflection is the degree to which a part of a structural element is displaced under a load (because it deforms). It may refer to an angle or a distance.

It's a serviceability requirement that must meet the limits according to ACI 318-19. In this project every span between columns in the slab should not exceed the limit of the permitted vertical deflection.

In deflection computations third of the live loads (Roof, storage and live in typical) are considered as sustained loads, the rest of it is considered non-sustained.

Table 3.5-1: Deflection limitations.

Table 24.2.2—Maximum permissible calculated deflections

Member	Condition		Deflection to be considered	Deflection limitation
Flat roofs	Not supporting or attached to nonstructural elements likely to be damaged by large deflections		Immediate deflection due to maximum of L_r, S, and R	$\ell/180$[1]
Floors	Immediate deflection due to L		$\ell/360$	
Roof or floors	Supporting or attached to nonstructural elements	Likely to be damaged by large deflections	That part of the total deflection occurring after attachment of nonstructural elements, which is the sum of the time-dependent deflection due to all sustained loads and the immediate deflection due to any additional live load[2]	$\ell/480$[3]
		Not likely to be damaged by large deflections		$\ell/240$[4]

The structure is considered to be attached to nonstructural elements (Stone and gypsum partations) which are unlikely to be damaged by lagre deflections. So, according to table 3.5-1, the deflection limitation should be used is **L/240**.

The deflection which will be computed be the long-term deflection (Time dependant deflection due to sustained loads and immediate deflection due to unsustained loads).

The long-term deflection load combination used is:

The total long-term deflection, Δ_{LT} is given by:

$$\Delta_{LT} = \Delta_L + \lambda_\infty \Delta_D + \lambda_t \Delta_{LS}$$

where:

Δ_L: immediate live load deflection (The deflection due to the un-sustained part of the live load is considered in this equation).

Δ_D : immediate dead load deflection.

Δ_{LS} : immediate sustained live load deflection.

λ_Δ: multiplier for additional deflection due to long-term effects, and it is given by:

$$\lambda_\Delta = \frac{\xi}{1 + 50\rho'}$$

where ρ' (compression steel ratio) shall be the value at midspan for simple and continuous spans, and at support for cantilevers.

It shall be permitted to assume ξ, the time-dependent factor for sustained loads, to be equal to:

5 years or more 2.0

12 months.............................. 1.4

6 months............................... 1.2

3 months................................ 1.0

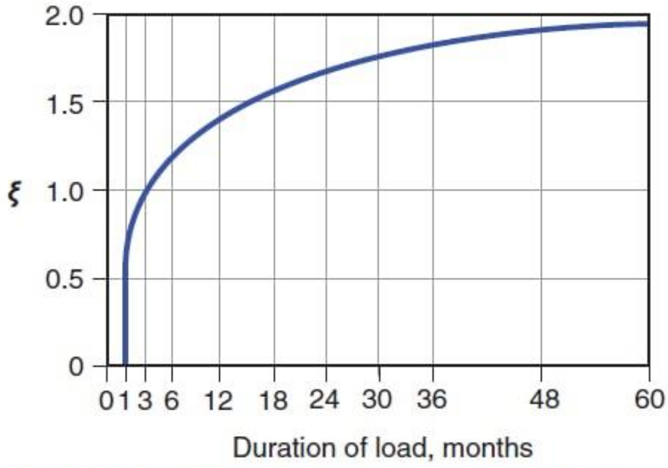


Fig. R24.2.4.1—Multipliers for time-dependent deflections.

Figure 3.5-1: ζ values (ACI 318-22 fig. R24.2.4.1).

The structure is expected to exist for more than 60 months, therefore ζ will equal 2.

By taking ρ equal to ~~zero~~ as a worst case, the multiplier for the additional deflection due to the long-term effect will equal ζ which is 2. So, the resultant load combination will be:

$\Delta_{LT} = \Delta_L + \lambda_\infty \, \Delta_D + \lambda_t \, \Delta_{LS} \rightarrow \Delta_{LT} = \Delta_L + 2\,\Delta_D + 2\,\Delta_{LS}$

Δ_L = 2/3 Live loads (Roof + Storage + Typical)

Δ_{LS} = 1/3 Live loads (Roof + Storage + Typical)

Δ_D = Dead loads + Super imposed dead loads

Δ_{LT} = 2 (D.D. + S.D.) + 1.333 L.L. (Roof, storage and live load in typical)

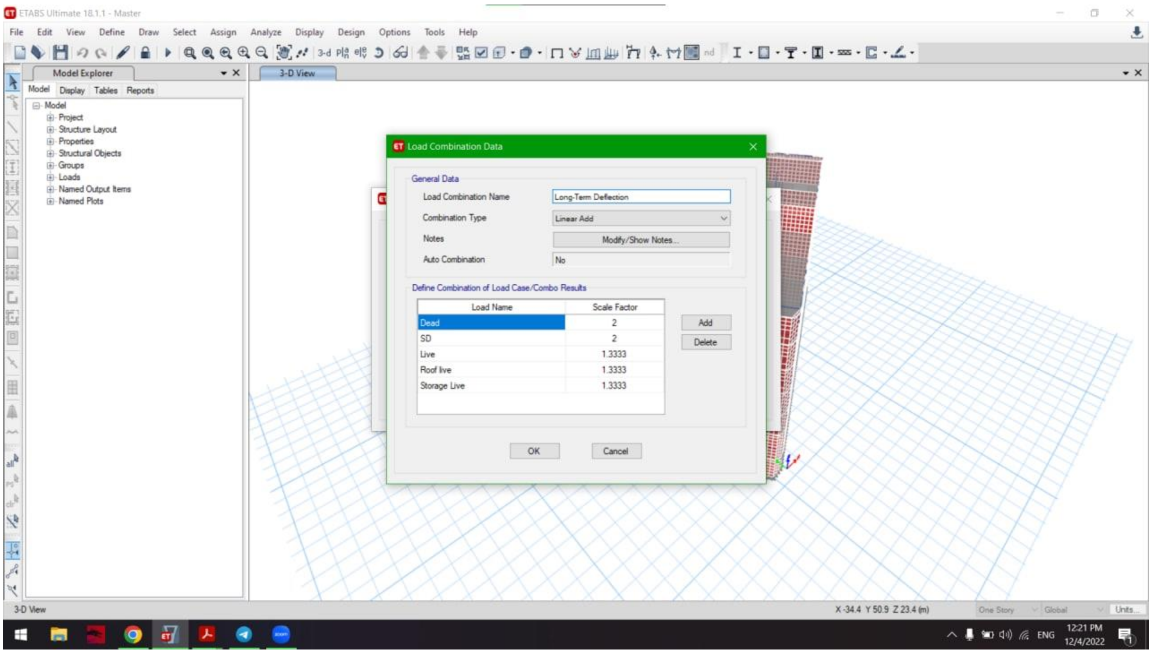


Figure 3.5-2: Long-term deflection.

The relative deflection will be compared with the required limitation which is L/240.

Long-term deflections in basement slab (since it has the highest deflection values) according to ETABS, two samples were taken A and B because they have the maximum values, as shown in figure 3.5-3 to 3.5-8.

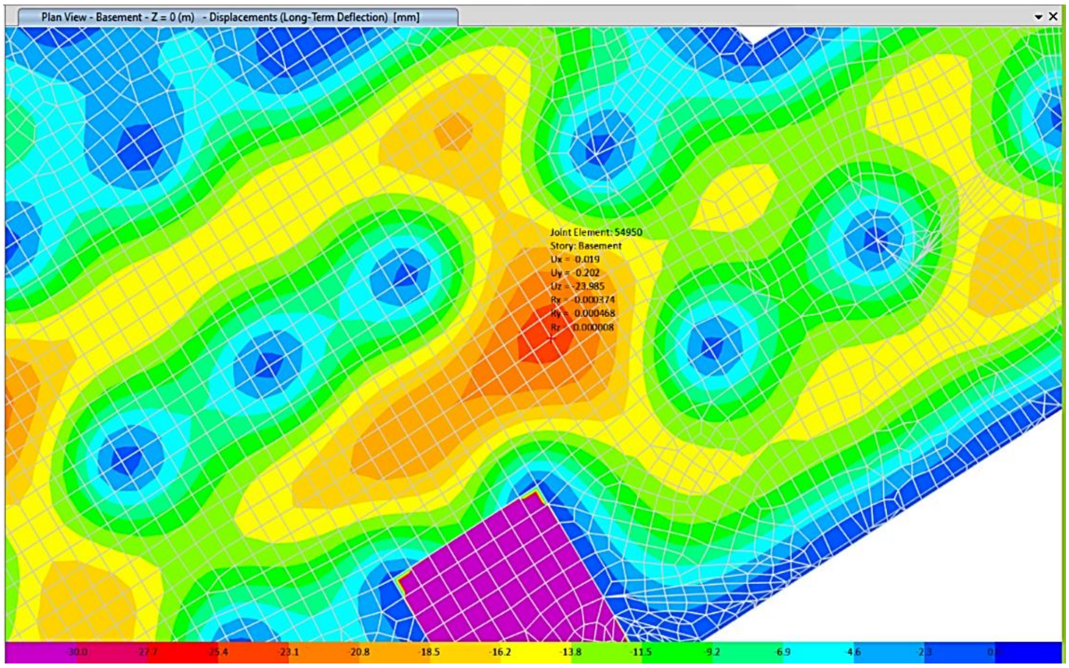


Figure 3.5-3: Long-term deflection A.1.

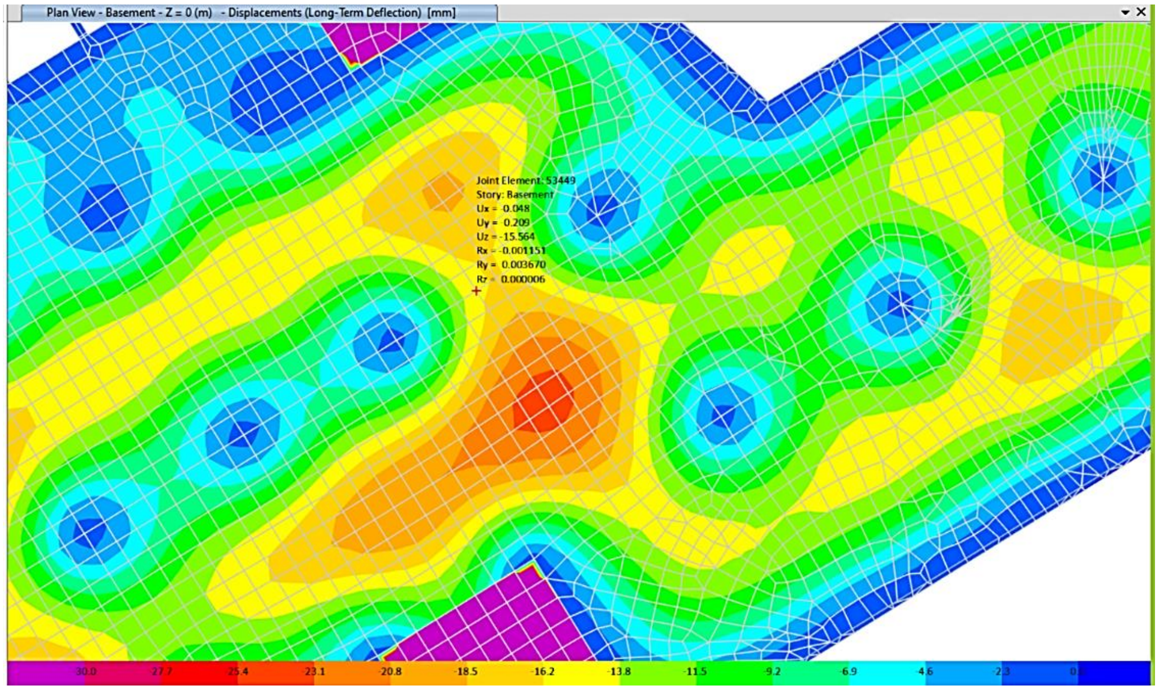


Figure 3.5-4 :Long-term deflection A.2.

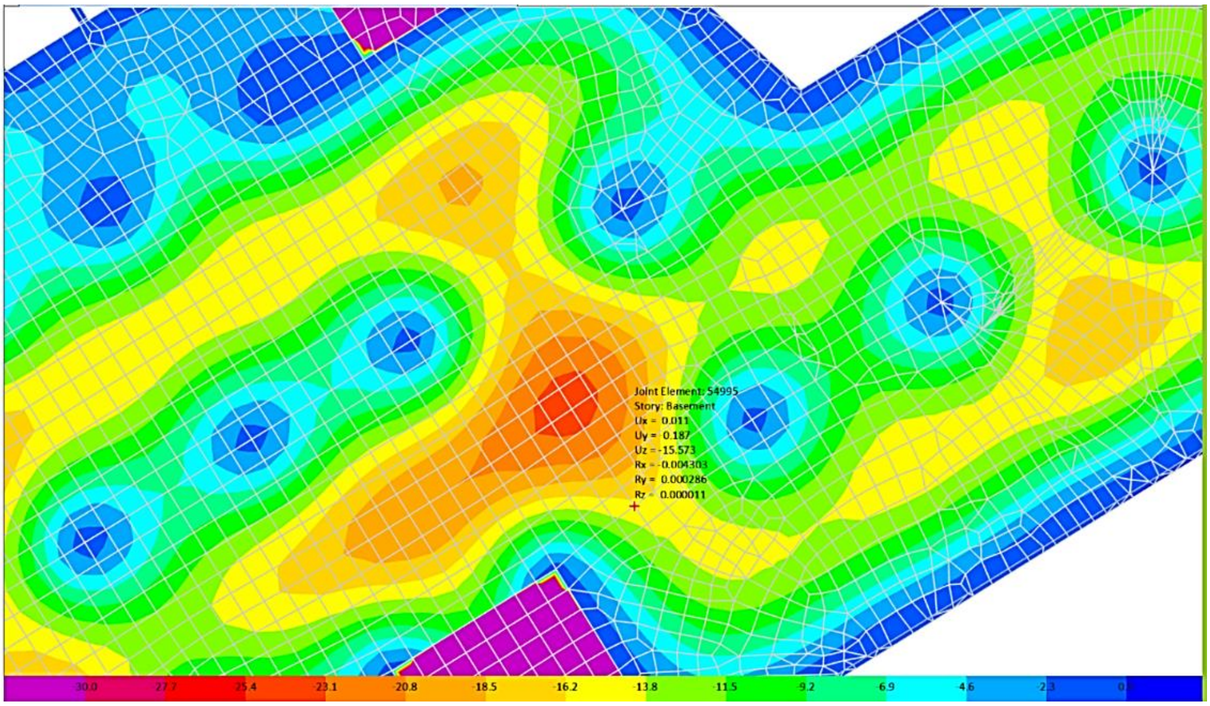


Figure 3.5-5: Long-term deflection A.3.

The relative deflection = $23.98 - \frac{15.56+15.57}{2}$ = 8.42 mm, The limitation = $\frac{5000}{240}$ = 20.83 mm

8.42 < 20.83 **OK.**

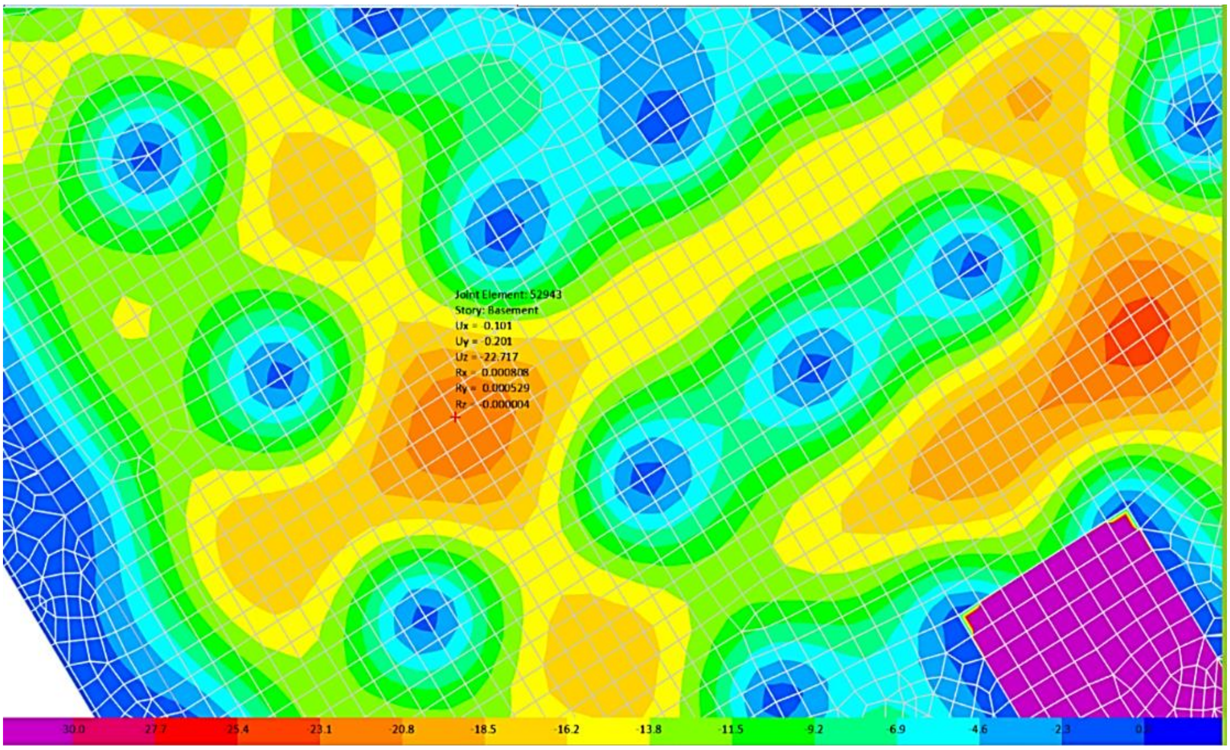


Figure 3.5-6: Long-term deflection B.1.

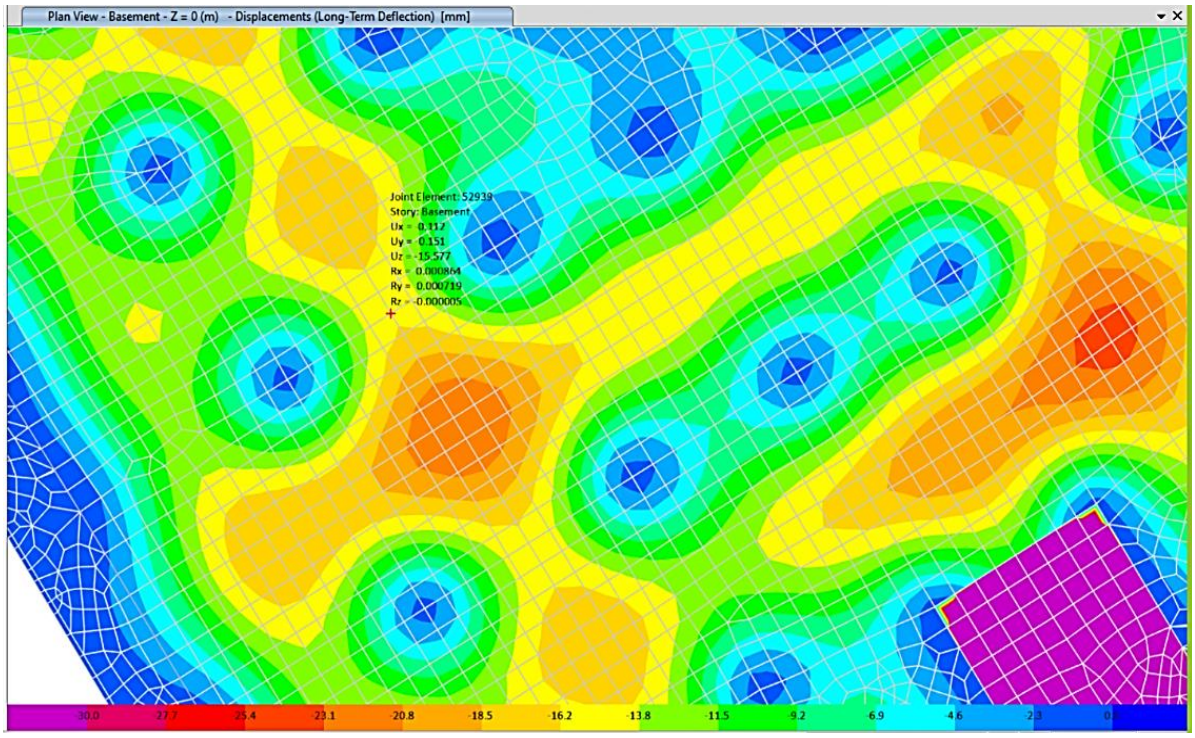


Figure 3.5-7: Long-term deflection B.2.

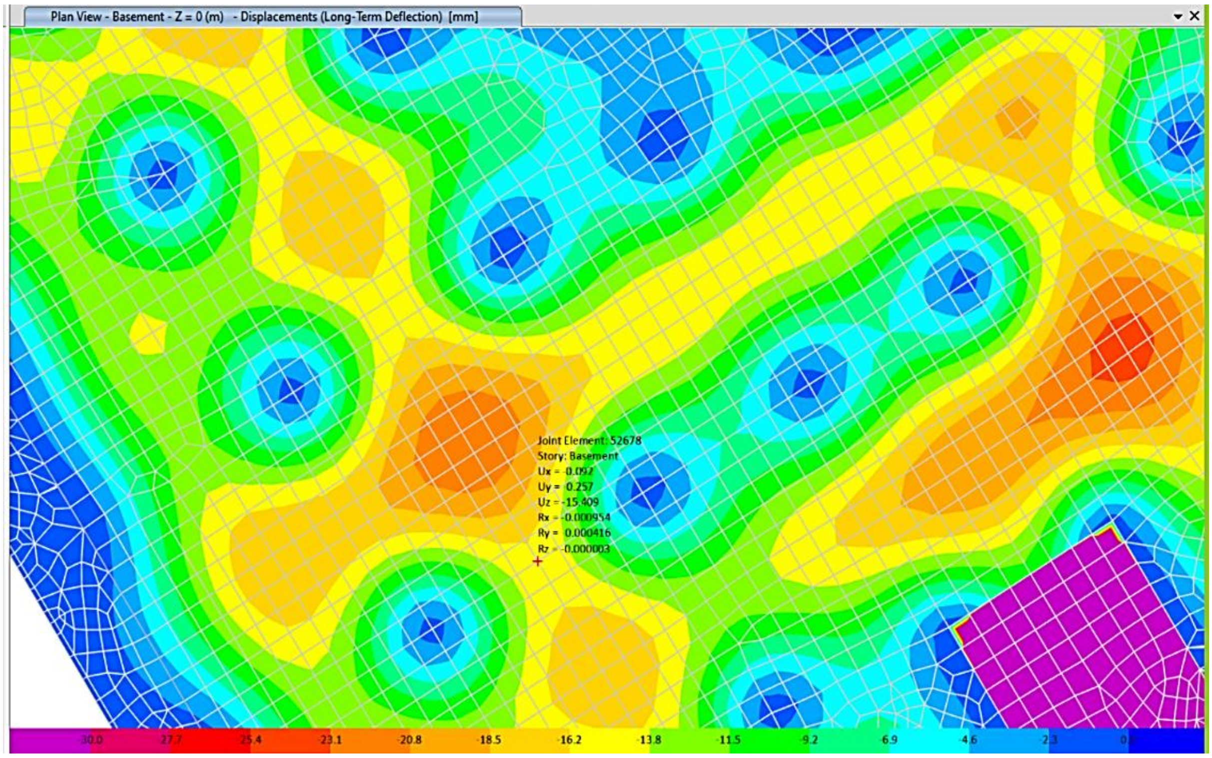


Figure 3.5-8: Long-term deflection B.3.

The relative deflection = $22.72 - \frac{15.58+15.41}{2}$ = 7.23 mm, The limitation = $\frac{5350}{240}$ = 22.29 mm

8.42 < 22.29 **OK.**

3.6 Verification of structural design:

Before obtaining the steel reinforcement results from the used program (ETABS), some checks must be made to various samples of the structural elements such as; beams, columns and walls, to make sure the program calculations and methods are according to the used code (ACI318-19).

The combinations used in these samples will be included in the ultimate envelopes (gravity and seismic), as shown in figures 3.6-1 and 3.6-2.

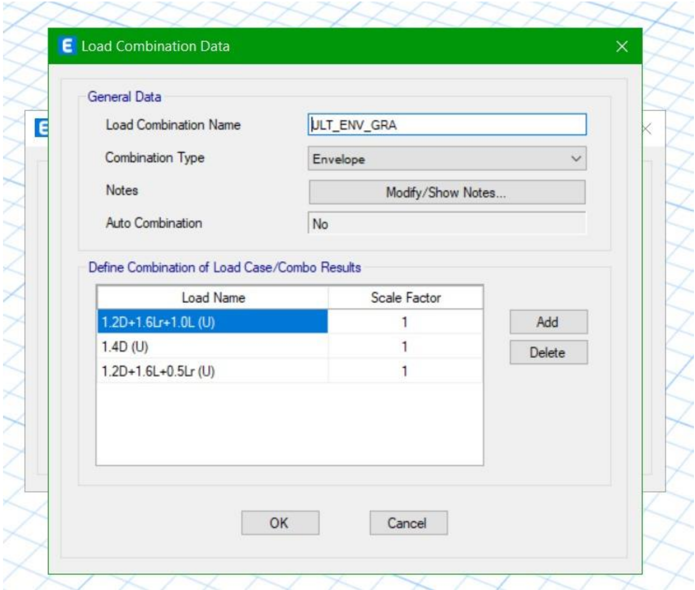


Figure 3.6-1:Ultimate Envelope (Gravity).

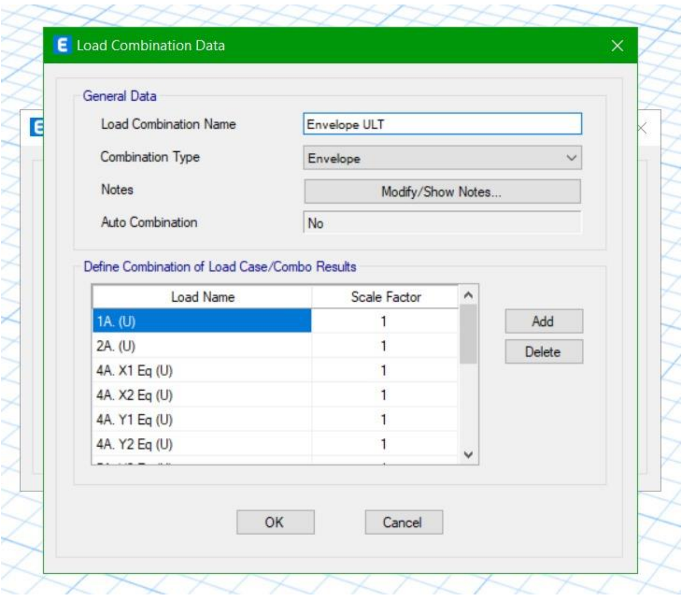


Figure 3.6-2:Ultimate Envelope (Seismic).

3.6.1 Beams: -

The 2nd model will be used for these checks (The gravity model).

Beam B13 from the first floor will be taken as a sample, as shown in figure 4.6-3.

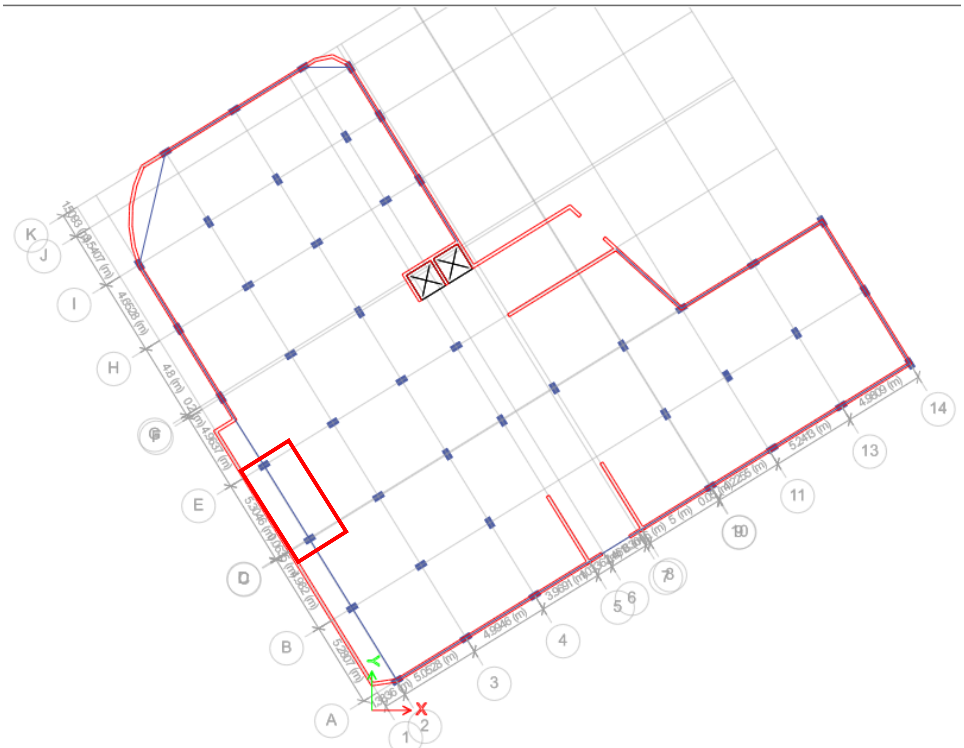


Figure 3.6-3:Beam B13.

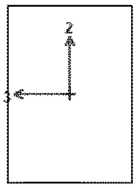
b_w = 400 mm, h = 550 mm, d = 490 mm

The top-left of the beam 13 will be taken to do the checks (I-end).

Figure 3.6-4 shows the design data for the beam.

ETABS Concrete Frame Design

ACI 318-19 Beam Section Design (Summary)



Beam Element Details

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	LLRF	Type
1st	B13	197	Beam 400*550 (Tcr)	ULT_{ENV}	200	5353.4	1	Sway Special

Section Properties

b (mm)	h (mm)	b_f (mm)	d_s (mm)	d_{ct} (mm)	d_{cb} (mm)
400	550	400	0	60	60

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}
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)	-109.3142	-1.8672	609	0	644	644
Bottom (-2 Axis)	54.6571	-1.8672	0	302	644	644

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
148.4472	129.1772	19.27	0	126.75

Torsion Force and Torsion Reinforcement for Torsion, T_u

T_u kN-m	ϕT_{th} kN-m	ϕT_{cr} kN-m	Area A_0 cm²	Perimeter, p_h mm	Rebar A_t /s mm²/m	Rebar A_l mm²
14.4291	8.3801	33.5202	1219.3	1544.4	190.71	874

Model 2_(Gravity).EDB Page 1 of 1 2/26/2023

Figure 3.6-4:Beam B13 design sheet.

- Flexural steel: The longitudinal steel that resist the moment $M_{3\text{-}3}$. Table 3.6-1 shows the ultimate moment and steel rebar results on I-end on beam B13.

Table 3.6-1:Ultimate moment and steel rebar results on I-end on beam B13.

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)	-109.3142	-1.8672	609	0	644	644
Bottom (-2 Axis)	54.6571	-1.8672	0	302	644	644

P_u = 1.87 kN (Tension)

M_u = 109.31 kN.m

$\rho = \frac{0.85*fc'}{fy}\left(1-\sqrt{1-\frac{2.61*Mu}{b_w*d^2*fc'}}\right)$

ρ = 0.00309 → $A_{s\ required} = \rho\ b_w\ d$ → $A_{s\ required}$ = 605.64 mm^2

$A_{s\ Total} = A_{s\ required} + \frac{Pu}{2\ \emptyset\ fy}$

$\emptyset$ = 0.9 → $A_{s\ Total}$ = 608.11

$A_{s\ ETABS}$ = 609.00 mm^2

Diff % = 0.15 %

$A_{s\ minimum}$ = Max of ($\frac{1.4}{fy}$, $\frac{0.25\sqrt{f'_c}}{fy}$) b_w d → 653.33 … 643.51 (without any approximation)

$A_{s\ ETABS\ minimum}$ = 644.00 mm^2

Diff % = 0.08 %

$A_{s\ minimum} > A_{s\ Total\ required}$; take $A_{s\ minimum}$

- Shear steel: The transverse longitudinal steel that resist the shear forces. Table 3.6-2 shows the ultimate shear and steel transverse results on I-end on beam B13.

Table 3.6-2:Ultimate shear and steel transverse results on I-end on beam B13.

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
148.4472	129.1772	19.27	0	126.75

V_u = 148.45 kN

Φ $V_{c\ ETABS}$ = 129.18 kN

Φ $V_{s\ ETABS}$ = 19.27 kN

$V_{u\ by\ end\ moment}$ = V_p = 0

$V_c = \frac{1}{6} * \lambda * \sqrt{fc'} * b_w * d \rightarrow V_c = 172.86$

$V_s = \frac{Vu}{\Phi} - Vc$

Φ = 0.75

V_s = 25.07 kN

$V_{s\ ETABS}$ = 25.69 kN

Diff % = 2.47

$\frac{Av}{S} = \frac{Vs}{fyt*d} \rightarrow \frac{Av}{S}$ = 124.83 mm^2 /m

$\frac{Av}{S}$ ETABS = 126.75 mm^2 /m

Diff % = 1.54 %

- Torsion steel: The transverse and longitudinal steel that resist the torsion forces. Table 3.6-3 shows the ultimate torsion acting on I-end of beam B13.

Table 3.6-3:Ultimate torsion and steel transverse results on J-end on beam B13.

Torsion Force and Torsion Reinforcement for Torsion, T_u

T_u kN-m	ϕT_{th} kN-m	ϕT_{cr} kN-m	Area A_o cm²	Perimeter, p_h mm	Rebar A_t /s mm²/m	Rebar A_l mm²
14.4291	8.3801	33.5202	1219.3	1544.4	190.71	874

Figure 3.6-5 describe the details needed for measuring the parameter needed in torsion calculations.

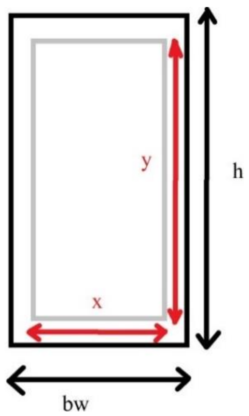


Figure 3.6-5:Beam transverse detailing.

Assuming Φ 10 is used in transverse steel:

$x = b_w - 2*(\text{clear cover} + \frac{ds}{2}) \rightarrow x = 400 - 2*(40+5) \rightarrow x = 310$ mm

$y = h - 2*(\text{clear cover} + \frac{ds}{2}) \rightarrow y = 550 - 2*(40+5) \rightarrow y =$

460 mm

$A_{cp} = b_w * h = 220000$ mm^2

$A_{oh} = x * y = 142600$ mm^2

$A_o = 0.85\ A_{oh} = 121210\ mm^2$

$P_{cp} = 2*b_w + 2*h = 1900\ mm$

$P_h = 2*x + 2*y = 1540\ mm$

Figure 3.6-6 shows the axial compression acting on beam B13.

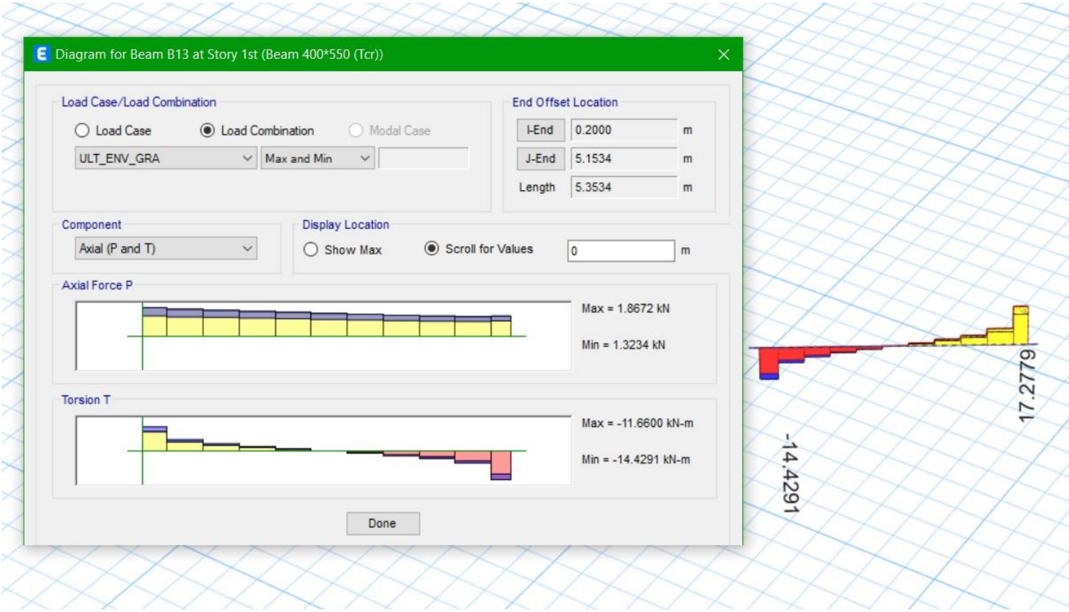


Figure 3.6-6:Axial compression force on beam B13.

Table 3.6-4 shows the equation needed to calculate threshold torsion for solid cross sections, according to ACI 318-19.

Table 3.6-4:Threshold torsion for solid cross sections Tth.

Type of member	T_{th}	
Nonprestressed member	$\frac{1}{12}\lambda\sqrt{f'_c}\frac{A_{cp}^2}{P_{cp}}$	(a)
Prestressed member	$\frac{1}{12}\lambda\sqrt{f'_c}\frac{A_{cp}^2}{P_{cp}}\sqrt{1+\frac{f_{pc}}{0.33\lambda\sqrt{f'_c}}}$	(b)
Nonprestressed member subjected to axial force	$\frac{1}{12}\lambda\sqrt{f'_c}\frac{A_{cp}^2}{P_{cp}}\sqrt{1+\frac{N_u}{0.33A_g\lambda\sqrt{f'_c}}}$	(c)

Φ T_{th} = 8.4 kN.m

Φ $T_{th\ ETABS}$ = 8.38 kN.m

Diff % = 0.24 %

T_u = 14.43 kN.m > Φ T_{th} → Then needed to design for torsion.

$\frac{At}{s} = \frac{Tu}{\Phi * 2 * Ao * fyt} \rightarrow \frac{At}{s}$ = 188.97 mm^2 /m

$\frac{At}{s}$ ETABS = 190.71 mm^2 /m

Diff % = 0.92 %

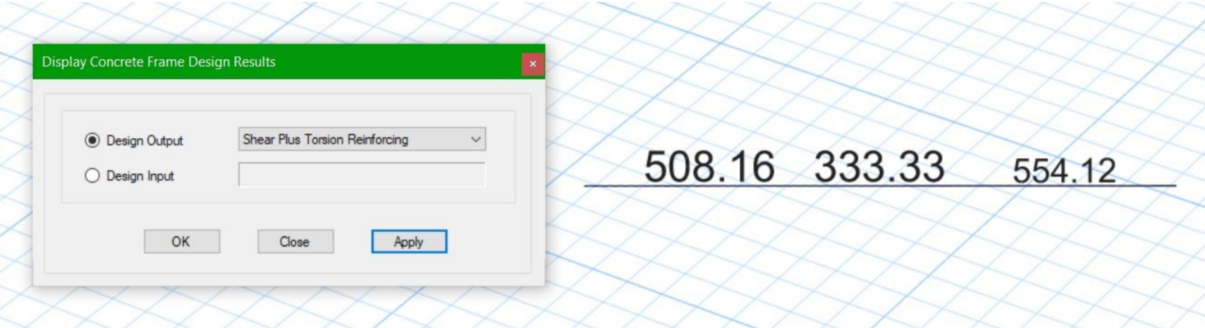


Figure 3.6-7:Shear plus torsion transverse reinforcement in beam B13.

$$\frac{Av+t}{s} = \frac{Av}{s} + 2 * \frac{At}{s}$$

$\frac{Av}{s}$ = 124.83 mm^2 /m

$\frac{At}{s}$ = 188.97 mm^2 /m

$\frac{Av+t}{s}$ = 502.77 mm^2 /m

$\frac{Av+t}{s}$ ETABS = 508.16 mm^2 /m

Diff % = 1.07 %

$\frac{Av}{s}$ minimum = Max of ($\frac{0.062 * \sqrt{f'c} * bw}{fyt}$, $\frac{0.35\ bw}{fyt}$) → 333.33 mm^2 /m

$\frac{Av+t}{s}$ = 502.77 > 333.33 mm^2 /m → then use the steel measured.

For the longitudinal steel to resist torsion:

$A_l = \frac{At}{s} \, Ph \, \frac{fyt}{fy}$

$\frac{At}{s}$ = 188.97 mm^2 /m

A_l = 291.01 mm^2

$A_{l\ minimum} = \frac{5\sqrt{f'c}}{12\ fy} \, Acp - \frac{At}{s} \, Ph \, \frac{fyt}{fy}$ = 863.88 mm^2/m

$A_{l\ minimum} > A_{l\ required}$; take $A_{l\ minimum}$

$A_{l\ ETABS}$ = 874.00 mm^2

Diff % = 1.17 %

Figure 3.6-8 shows the total longitudinal steel at I-end in beam B13.

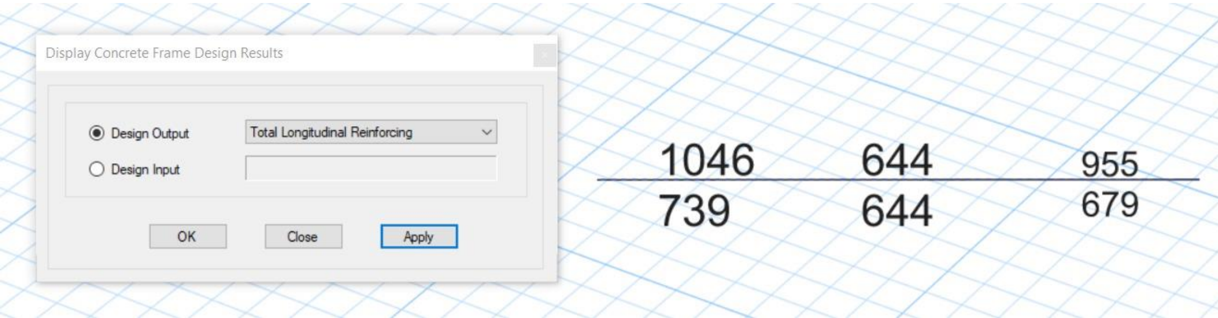


Figure 3.6-8:The total longitudinal steel at I-end in beam B13.

$A_{s\ flexural}$ = 608.11 mm^2

A_l = 863.88 mm^2

Total $A_s = A_{s\ flexural} + \frac{Al}{2}$ = 1040.05 mm^2

$A_{s\ ETABS}$ = 1046.00 mm^2

Diff % = 0.57 % ✓

3.6.2 Columns:-

Use the following column for the check, which is located on the basement floor, as shown in figure 3.6-9.

This column has a length of 3.20 m and the dimensions for the cross-section are 700 mm by 400 mm.

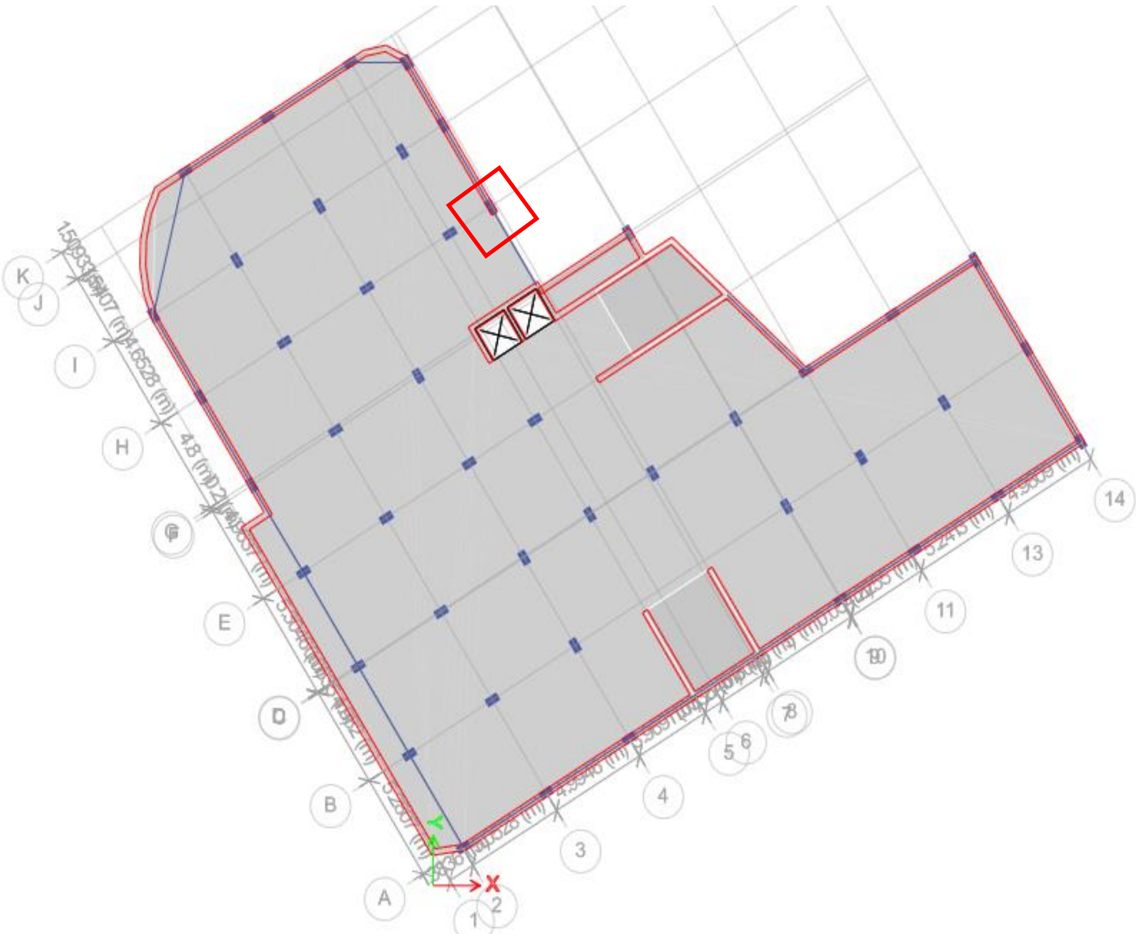


Figure 3.6-9:The column selected for the design check.

The internal forces are as follows :

Pu = 786.12kN

Mu_2 = 126.41 kN.m

Mu_3 = 33.48 kN.m

Table 3.6-5 shows the values of P_u, M_{u2} and M_{u3} for the design.

Figures 3.6-10 and 3.6-11 show the Interaction diagrams for this column.

Table 3.6-5:The value of Pu, Mu2 and Mu3 for the design.

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 % %
-786.12	-126.4197	33.4838	28.489	21.4139	3566	1.27

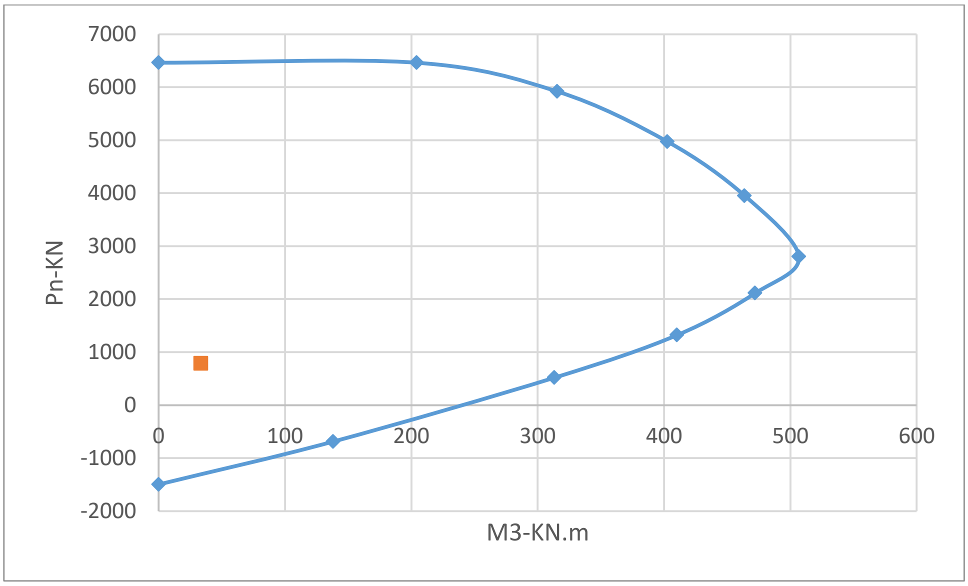


Figure 3.6-10:Interaction diagram (P and M3).

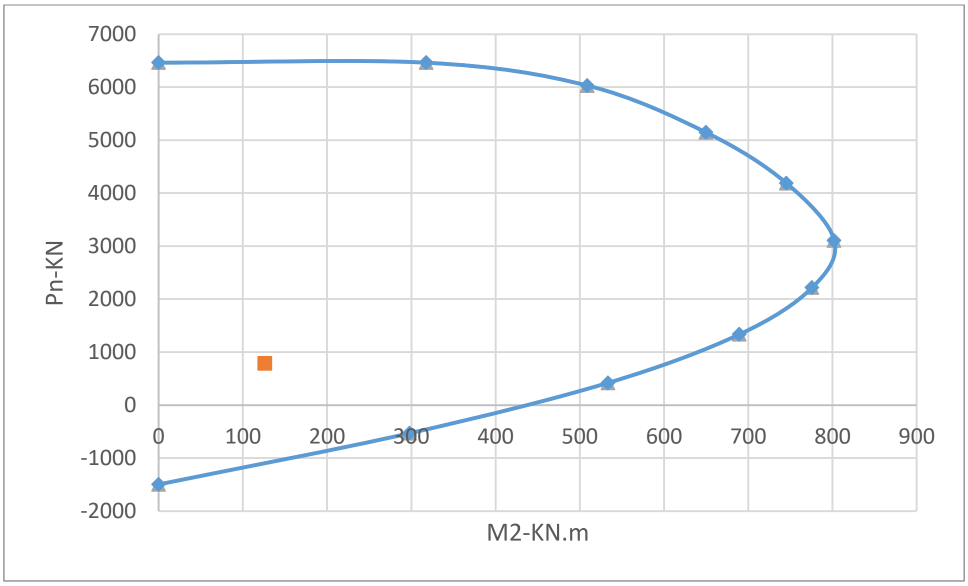


Figure 3.6-11:Interaction diagram (P and M2).

- Dimensional limits:

Columns shall satisfy the following:

a) Shortest cross-sectional dimension should be at least 300mm.

400mm > 300 mm OK

b) The ratio of the shortest cross-sectional dimension to the perpendicular dimension shall be at least 0.4.

$\rightarrow \frac{400}{700} = 0.57 > 0.4$ OK

- Minimum flexural strength of columns:

  This equation must be achieved:

$$\sum Mnc \geq (6/5) \sum Mnb \qquad \text{(ACI318-19 equation 18.7.3.2)}$$

Figure 3.6-12 illustrate the chosen column.

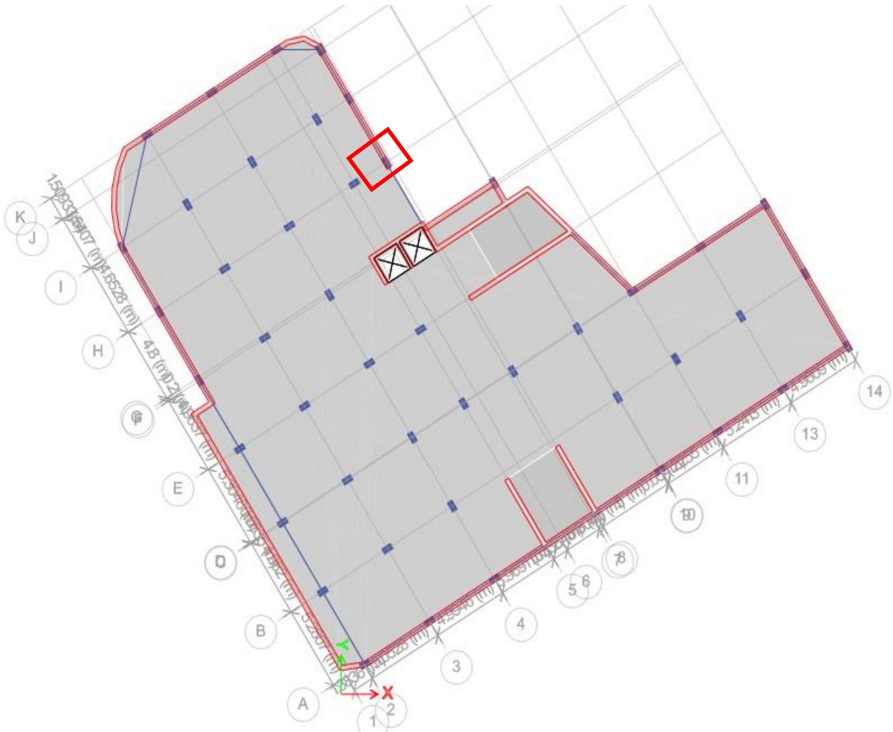


Figure 3.6-12:The column selected for the design check.

ΣM_B: Beam flexural.

As = 644 mm^2

$Mn = A_s * F_y \left(d - \frac{As\ Fy}{1.7\ Fc'\ b} \right)$

$Mn = 644 * 420 * \left(490 - \frac{644*420}{1.7*28*400} \right)$

Mn = 128.69 KN.m

Direction 1 → 128.69 + 128.69 = 257.38 KN.m

Direction 2 → 128.69 + 128.69 = 257.38 KN.m

Basement floor → Pu = 262.83 kN from **Figure 3.6-14,** Mn = 460 kN.m

Ground floor → Pu = 146.8 kN from **Figure 3.6-13,** Mn = 352 kN.m

ΣMc = 352 + 460 = 812 kN.m

$\frac{812}{257.38} = 3.154 \geq 1.2$ OK

Capacity ratio = $\frac{1.2}{3.154}$ = 0.380

Capacity ratio $_{ETABS}$ =0.386

Diff % = 1.57 % OK

Figures 3.6-13 and 3.6-14 show the Interaction diagrams for this column.

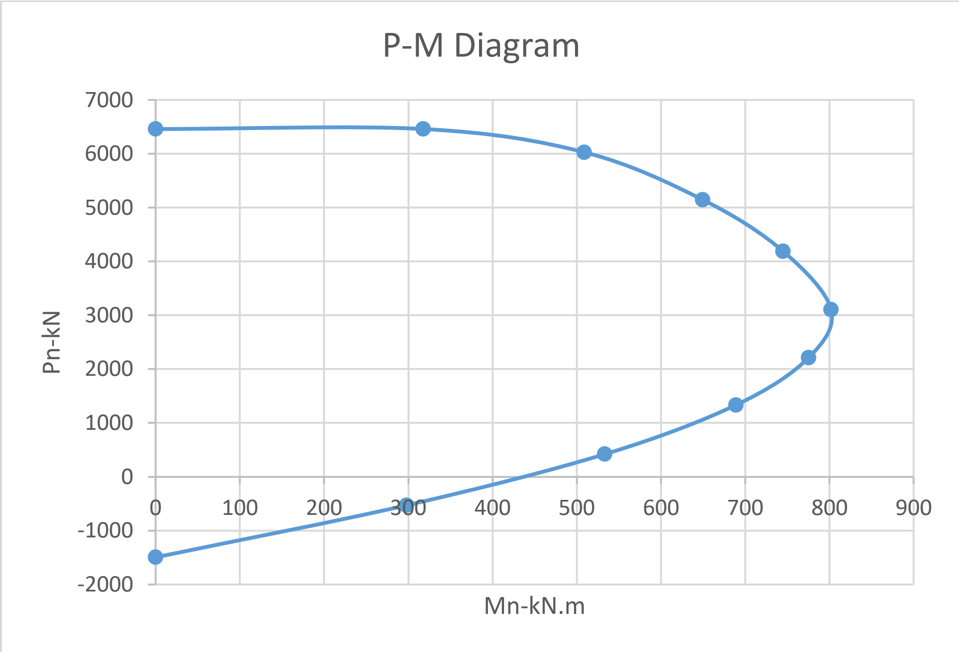


Figure 3.6-13:Interaction diagram (P-M) for column in the ground floor.

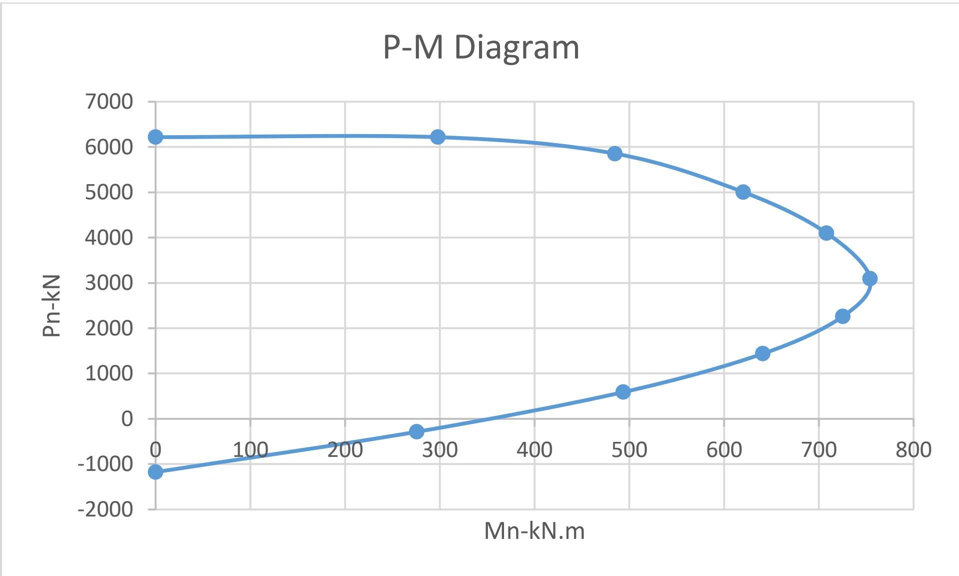


Figure 3.6-14:Interaction diagram (P-M) for column in the basement floor.

- Shear Design:

  Table 3.6-6 shows the results of shear design.

Table 3.6-6:Result of shear design.

	Rebar A_v /s mm²/m	Design V_u kN	Design P_u kN	Design M_u kN-m	ϕV_c kN	ϕV_s kN	ϕV_n kN
Major Shear(V2)	574.56	18.1202	-786.1201	33.4838	59.0116	49.5184	108.53
Minor Shear(V3)	1118.8	244.3045	-786.1201	58.8252	63.3005	181.004	244.3045

ΣM_{Pr} : Beam Flexure.

As = 644 mm^2

M_{Pr} = As * 1.25 * Fy (d - $\frac{As*1.25*Fy}{1.7\, Fc'\, b}$)

M_{Pr} = 644 * 1.25 * 420 * (490 - $\frac{644*1.25*420}{1.7*28*400}$)

M_{Pr} = 159.6 KN.m

Direction 1 → ΣM_{Pr} = 319.3 KN.m

Direction 2 → ΣM_{Pr} = 319.3 KN.m

Ve = $\frac{\Sigma MPr,b}{h\ column}$ = $\frac{319.3}{3.2}$ = 99.78 KN.

Maximum ultimate shear force is 244.30 KN, which is larger than Ve = 99.78 KN, So the value 244.30 is used for shear design for $\frac{Av}{s}$.

Figures 3.6-15 and 3.6-16 show the values of shear and axial loads on the column.

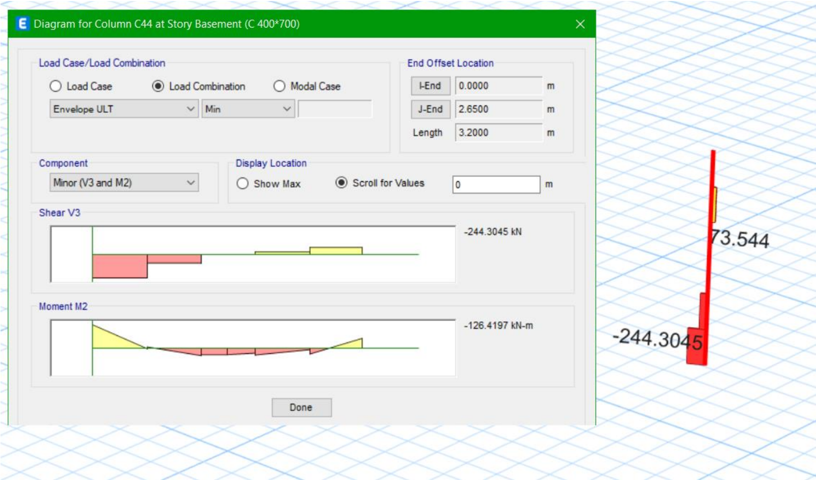


Figure 3.6-15:The value of shear force Vu2.

Check if Vc = 0:

→ Vu >Ve , So this point is not achieved.

→ Pu = 786.12 KN > $\frac{700*400*28}{20000}$, So this point is not achieved.

So Vc is not equal to zero.

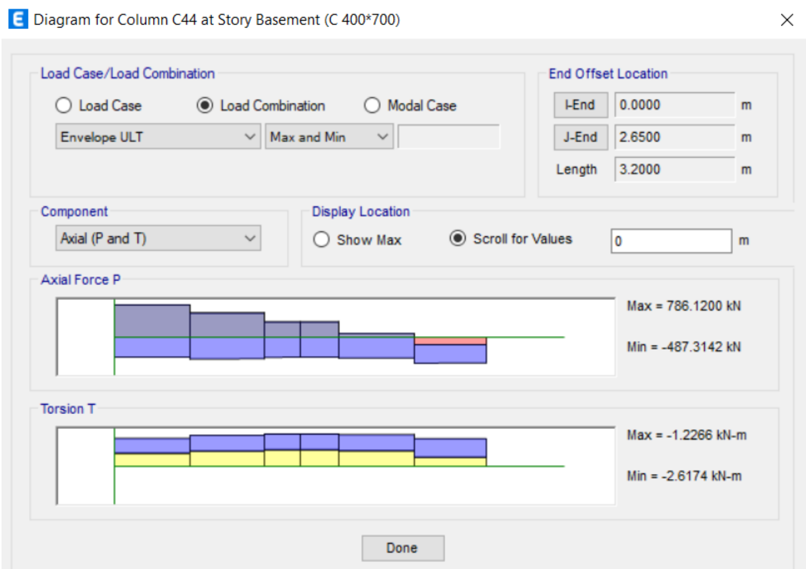


Figure 3.6-16:The value of axial force Pu.

$\frac{Nu}{6*Ag} \leq 0.05$ Fc' → $\frac{786.12*1000}{6*(700*400)} = 0.4679 < 1.4$

Use 0.4679.

Vc = (0.17 * 1 *$\sqrt{28}$ -0.4679) * 400 * 640 /1000

Vc = 110.50 KN.

Vc_{Max} = 0.42*1*$\sqrt{28}$ * 400 * 640/1000 = 568.90 KN.

At the bottom, Pu = 786.12 kN (Tension) from **Figure 3.6-14** → Mn = 75 kN.m

At the top, Pu = 628.87 kN (Compression) from **Figure 3.6-14** → Mn = 470 kN.m

Ve = $\frac{75+470}{3.2}$ =170.31 > 110.5 So, use reduction factor $\phi = 0.6$

$\frac{Vu}{\phi} = \frac{244.3}{0.6}$ = 407.17 kN

Vs=$\frac{Vu}{\phi}$ -Vc =407.17-110.5= 296.67 KN

$(\frac{Av}{s})$=1103.68 mm^2/m

$(\frac{Av}{s})_{min}$ =0.062*$\sqrt{28}$ *700*1000/420 = 546 mm^2/m

$(\frac{Av}{s})_{min}$= 0.35*700*1000/420 = 583.33 mm^2/m

$\left(\frac{Av}{s}\right) > (\frac{Av}{s})min$ so, take $\left(\frac{Av}{s}\right)$ = 1103.68 mm^2/m

$(\frac{Av}{s})_{Etabs}$ =1118.8 mm^2/m

Diff % =1.37 < 10% …. OK

3.6.3 Walls: -

The 1st model will be used for these checks (Full seismic model with shear modifiers on the columns).

1) Pier wall:

The bottom of the wall at elevation Elev. E1 will be taken as a sample, as shown in figure 3.6-17.

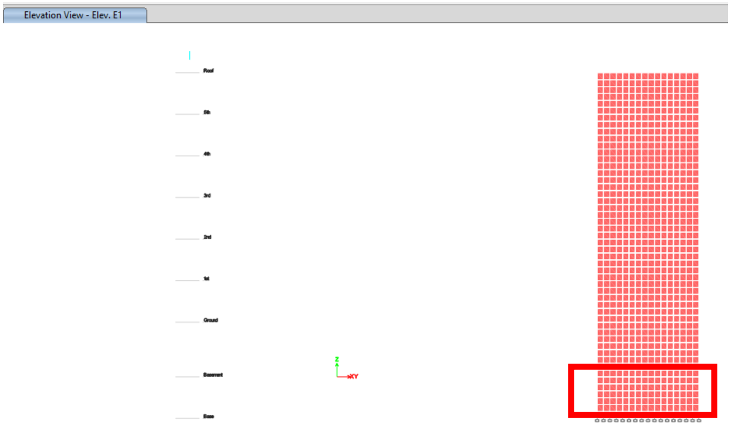


Figure 3.6-17:Wall of elevation Elev. E1.

l_w = 7830.50 mm, h_w = 3200 mm, b_w = 300 mm.

The **bottom-left** of it will be taken.

Figure 3.6-18 shows the pier wall design sheet that will be used in the checks.

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
Basement	E1P1	11763.5	26484	7830.5	300	0.465

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	8433.6	24424.6	15093.4	28543.4	7830.5	300
Bottom	Leg 1	8433.6	24424.6	15093.4	28543.4	7830.5	300

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	5873	0.0025	0.0021	Envelope ULT	7770.7458	-73.9123	-7892.8126	2349165
Bottom	10514	0.0045	0.0021	Envelope ULT	-472.9587	10.9677	-10372.8614	2349165

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	1062.8	Envelope ULT	1084.9614	4119.347	3335.8254	1238.606	3335.8254
Bottom	Leg 1	782.65	Envelope ULT	-472.9587	7007.4931	2783.0064	1238.606	2783.0064

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	878.6	Envelope ULT	7770.7458	-7892.8126	5.88	5.6	1661.7	1740.1
Top–Right	Leg 1	878.6	Envelope ULT	7770.7458	4119.347	4.65	5.6	1661.7	1740.1
Bottom–Left	Leg 1	781.4	Envelope ULT	7179.2562	-10372.8614	6.44	5.6	1562.8	1740.1
Botttom–Right	Leg 1	781.4	Envelope ULT	7179.2562	7007.4931	5.34	5.6	1562.8	1740.1

Figure 3.6-18:Pier wall design sheet.

- **Flexural check**: The longitudinal steel that resist the moment M_{3-3} and P_u. Table 3.6-7 shows the ultimate forces and stresses results on Elev. E1 wall.

Table 3.6-7:ultimate forces result on Elev. E1 wall.

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	5873	0.0025	0.0021	Envelope ULT	7770.7458	-73.9123	-7892.8126	2349165
Bottom	10514	0.0045	0.0021	Envelope ULT	-472.9587	10.9677	-10372.8614	2349165

Figure 3.6-19 shows the distribution of the vertical rebar along the cross-section of the wall.

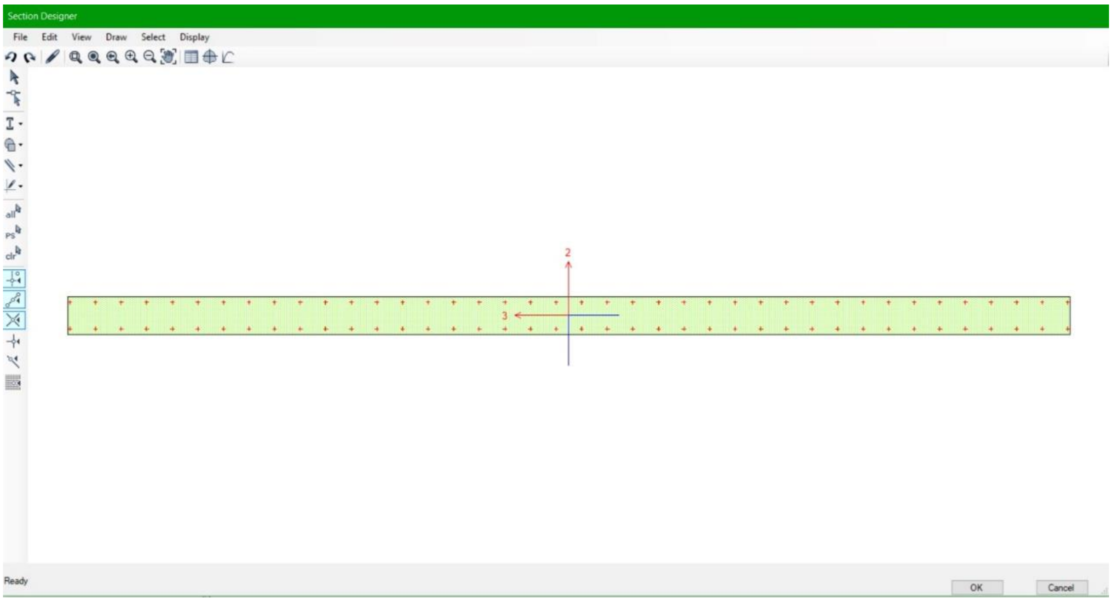


Figure 3.6-19:Wall of elevation Elev. E1.

$A_{s\ ETABS}$ = 10514 mm^2; this steel is distributed all over the gross area using two meshes.

To get the steel for every meter of the wall for one mesh, by using Φ14, the steel used will be 1Φ14/ 200mm.

Spacing between bars = $\frac{7830.5}{\frac{10514}{2(\pi*14^2)/4}}$ = 229.30 mm → use 200 mm.

The interaction diagram for this pier wall is shown in figure 3.6-20.

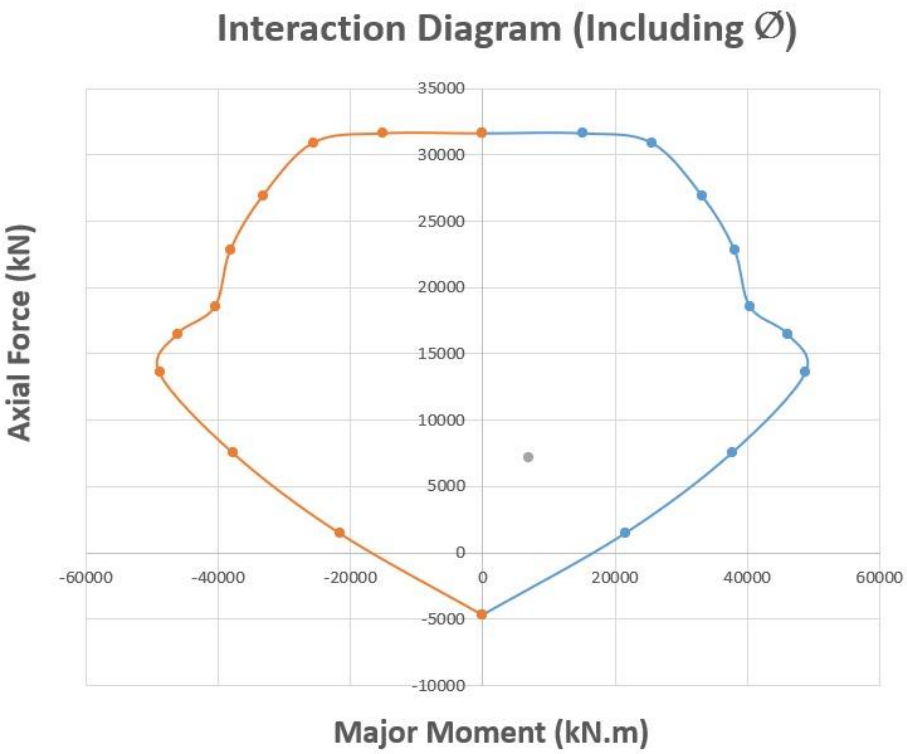


Figure 3.6-20:Interaction diagram of pier wall.

Point (7007.49 kN.m , 7179.26 kN)

The point that represents the ultimate loads on this wall for flexural falls inside the curve. ✓

- **Shear strength check**: The transverse longitudinal reinforcement steel that resist the shear force. Table 3.6-8 shows the ultimate shear and steel transverse results on Elev. E1 wall.

Table 3.6-8:Ultimate shear and steel transverse results on Elev.1 wall.

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	1062.8	Envelope ULT	1084.9614	4119.347	3335.8254	1238.606	3335.8254
Bottom	Leg 1	782.65	Envelope ULT	-472.9587	7007.4931	2783.0064	1238.606	2783.0064

V_u = 2783.01 kN

Φ $V_{n\ ETABS}$ = 2783.01 kN

Φ $V_{c\ ETABS}$ = 1238.61 kN

Φ V_n = Φ ($\alpha_c\ \lambda\ \sqrt{f'c}$ + $\rho_t\ f_{yt}$) A_{cv}

Φ = 0.60

α_c = 0.25; since $h_w \backslash l_w \leq 1.5$

λ = 1.0; since the concrete used is a normal weight

$A_{v\ horizontal}$ = 782.65 mm^2 /m

$\rho_t = \frac{Av\ horizontal}{b*s2} = \frac{782.65}{300*1000}$ = 0.0026

$A_{cv} = l_w * b_w$ = 2349150 mm^2

Φ V_n = 0.6 * (0.25 * 1 * $\sqrt{28}$ + 0.0026 * 420) * 2349150 = 3403.74 kN

V_n = 5672.90 kN

$V_{n\ max} = 0.66\sqrt{f'c}\ A_{cv}$ = 8204.15 kN > 5672.90 ✓

Φ V_n Diff % = 18.24 %

Φ V_c = Φ $\alpha_c\ \lambda\ \sqrt{f'c}\ A_{cv}$ → Φ V_c = 1864.58 kN

Diff % = 33.57 %

Note: **the differences in values were high; because ETABS takes the full height of the wall since the pier is assigned for the whole wall, so the α_c will be taken as 0.17 by the ETABS.**

If you take α_c = 0.17, the difference in Φ V_n will be 0.86%, and the difference in Φ V_c will be 2.30%.

- **Boundary element check**: Check if there is a need to use a boundary element for the wall. Table 3.6-9 shows the ultimate forces and stresses results on Elev. E1 wall.

Table 3.6-9;Ultimate forces and stresses results on Elev. E1 wall.

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	878.6	Envelope ULT	7770.7458	-7892.8126	5.88	5.6	1661.7	1740.1
Top–Right	Leg 1	878.6	Envelope ULT	7770.7458	4119.347	4.65	5.6	1661.7	1740.1
Bottom–Left	Leg 1	781.4	Envelope ULT	7179.2562	-10372.8614	6.44	5.6	1562.8	1740.1
Botttom–Right	Leg 1	781.4	Envelope ULT	7179.2562	7007.4931	5.34	5.6	1562.8	1740.1

$\sigma_u = \frac{Nu}{A} \pm \frac{Mu\,x}{Iy} \pm \frac{Mu\,y}{Ix}$

$P_u = N_u = 7179.26$ kN

$M_{u\,2\text{-}2} = 48.56$ kN.m

$M_{u\,3\text{-}3} = 10372.86$ kN.m

X = 3915.25 mm

Y = 150.00 mm

A = 2349150.00 mm^2

$I_x = 1.20\ x10^{13}\ mm^4$

$I_y = 1.76\ x10^{10}\ mm^4$

$\sigma_u = \frac{7179.26*10^3}{2349150.00} + \frac{48.56*150.00*10^6}{1.76*10^{10}} + \frac{10372.86*3915.25*10^6}{1.20*10^{13}} = 6.85$ MPa

$\sigma_{u\ ETABS} = 6.44$ MPa

Stress Diff % = 5.99 %

Note: **the difference in stress value is due the $M_{u\,2\text{-}2}$ which the ETABS didn't consider in the calculations. Hand calculation difference for the stress without taking the $M_{u\,2\text{-}2}$ equals 0.00%.**

$\sigma_{limit} = 0.2\ f'c = 5.60$ MPa

$\sigma_u > \sigma_{limit} \rightarrow$ Boundary element is needed.

$C_{depth} = (\frac{\alpha + \omega}{0.85\ \beta1 + 2\omega}) l_w$

$\alpha = \frac{Pu}{b * lw * f'c} = 0.1092$

$\omega = \rho_l \frac{fy}{f'c} = 0.0675$

$\rho_l = 0.0045$

$\beta_1 = 0.85$; since f'c = 28 MPa

c_{depth} = 1613.59 mm

$c_{depth\ ETABS}$ = 1562.80 mm

Diff % = 3.15 %

2) Spandrel wall:

The spandrel at the middle bottom of the wall in elevation A, that is going through the first-floor slab, will be taken as a sample, figure 3.6-21 shows the chosen spandrel.

Figure 3.6-21:Wall of elevation A.

b_w = 300 mm, h_w = 2200 mm, d = 1980 mm, L_n = 2400 mm

The **top-right** will be taken of it.

Figure 3.6-22 shows the spandrel design sheet that will be used in the checks.

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
1st	AS34	17734.5	11857.2	2200	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	1300	0.3	Envelope ULT	-911.3452
Right	1599	0.36	Envelope ULT	-1112.0834

Spandrel Flexural Design—Bottom Reinforcement

Station Location	Reinf Area mm²	Reinf Percentage	Reinf Combo	Moment, M_u kN-m
Left	1363	0.31	Envelope ULT	957.7696
Right	1300	0.3	Envelope ULT	789.0934

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	1322.29	500	Envelope ULT	865.6669	205.8977	659.7693	865.6669
Right	1611.64	500	Envelope ULT	1009.0378	204.8952	804.1426	1009.0378

Spandrel Shear Design—Diagonal Reinforcement

Station Location	A_{diag} mm²	Shear Combo	V_u kN	V_{ulimit} kN	L/H Ratio	Seismic Design	Diag Reinf Mandatory
Left	2050	Envelope ULT	865.6669	695.9751	1.091	Yes	Yes
Right	2390	Envelope ULT	1009.0378	695.9751	1.091	Yes	Yes

Figure 3.6-22:Spandrel wall design sheet.

Flexural strength check: The longitudinal steel that resist the moment $M_{3\text{-}3}$. Table 3.6-10 shows the ultimate forces and stresses results on the spandrel.

Table 3.6-10:Ultimate forces and stresses results on the spandrel.

Spandrel Flexural Design—Top Reinforcement

Station Location	Reinf Area mm²	Reinf Percentage	Reinf Combo	Moment, M_u kN-m
Left	1300	0.3	Envelope ULT	-911.3452
Right	1599	0.36	Envelope ULT	-1112.0834

P_u = 43.30 kN (Tension)

$V_{u\ 2\text{-}2}$ = 1009.04 kN

$V_{u\ 3\text{-}3}$ = 10.97 kN

Torsion = 18.08 kN

$M_{u\ 2\text{-}2}$ = 2.20 kN.m

$M_{u\ 3\text{-}3}$ = 1112.08 kN.m

$$\rho = \frac{0.85\, f'c}{fy} * (1 - \sqrt{1 - \frac{2.61\, Mu}{bd^2 f'c}})$$

$\rho = 0.0039 \rightarrow A_s = \rho\ b\ d \rightarrow A_{s\ flexural} = 1544.4\ mm^2$

$A_{s\ total} = A_{s\ flexural} + \frac{Pu}{2\ \Phi\ fy}$

$\Phi = 0.90 \rightarrow A_{s\ total} = 1601.68\ mm^2$

$A_{s\ ETABS} = 1599.00\ mm^2$

Diff % = 0.17 %

$L_n/h = 1.09 < 2$

$V_u = 1009.04 > [0.33\ \lambda \sqrt{f'c}\ A_{cw}] = 768.33$

So, it shall be reinforced with two intersecting groups of diagonally bars symmetrical about the midspan of this spandrel.

- **Shear strength check**: The transverse longitudinal reinforcement steel that resist the shear force. Table 3.6-11 shows the ultimate shear and steel transverse results on the spandrel.

Table 3.6-11:Ultimate shear and steel transverse results on the spandrel.

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	1322.29	500	Envelope ULT	865.6669	205.8977	659.7693	865.6669
Right	1611.64	500	Envelope ULT	1009.0378	204.8952	804.1426	1009.0378

P_u = 43.30 kN (Tension)

V_u = 1009.04 kN

$\Phi\ V_{n\ ETABS}$ = 1009.04 kN

$\Phi\ V_{c\ ETABS}$ = 204.90 kN

$\Phi\ V_{s\ ETABS}$ = 804.14 kN

$V_c = \alpha_c\ \lambda \sqrt{f'c}\ A_{cv}$

$\alpha_c = 0.17\ (1+\frac{Nu}{3.45\ Ag}) > 0$; since there is tension on the spandrel

α_c = 0.165

λ = 1; since the concrete is normal weight.

$A_{cv} = d * b_w$ = 396000 mm^2

V_c = 345.75 kN

$\Phi\ V_c$ = 207.45 kN

Diff % = 1.23 %

$V_s = \frac{Vu}{\Phi} - V_{c\ ETABS} \rightarrow \frac{1009.04}{0.6} - 341.50 = 1340.23$ kN

$\frac{Av}{S} = \frac{Vs}{fyt*d} \rightarrow \frac{Av}{S}$ = 1611.63 mm^2 /m

$(\frac{Av}{S})_{ETABS}$ = 1611.64 mm^2 /m

Diff % = 0.00 %

Table 3.6-12 shows the ultimate shear and diagonal reinforcement steel results on the spandrel.

Table 3.6-12:Ultimate shear and diagonal reinforcement steel results on the spandrel.

Spandrel Shear Design—Diagonal Reinforcement

Station Location	A_{diag} mm²	Shear Combo	V_u kN	V_{uLimit} kN	L/H Ratio	Seismic Design	Diag Reinf Mandatory
Left	2050	Envelope ULT	865.6669	695.9751	1.091	Yes	Yes
Right	2390	Envelope ULT	1009.0378	695.9751	1.091	Yes	Yes

V_u = 1009.04 kN

$A_{vd} = \frac{Vu}{2 * \Phi s * fy * \sin\alpha}$

$\sin\alpha = \frac{0.8\ hs}{\sqrt{Ls^2 + (0.8hs)^2}} \rightarrow \frac{0.8*2200}{\sqrt{2400^2 + (0.8*2200)^2}} = 0.591$

$A_{vd} = \frac{1009.04*10^3}{2 * 0.85 * 420 * 0.591} = 2391.24\ mm^2$

$A_{vd\ ETABS}$ = 2390.00 mm^2

Diff % = 0.05 %

3.6.4 Footing: -

The allowable stress on the soil is q = 200 kN/m^2

Figure 3.6-23 shows the mat foundation.

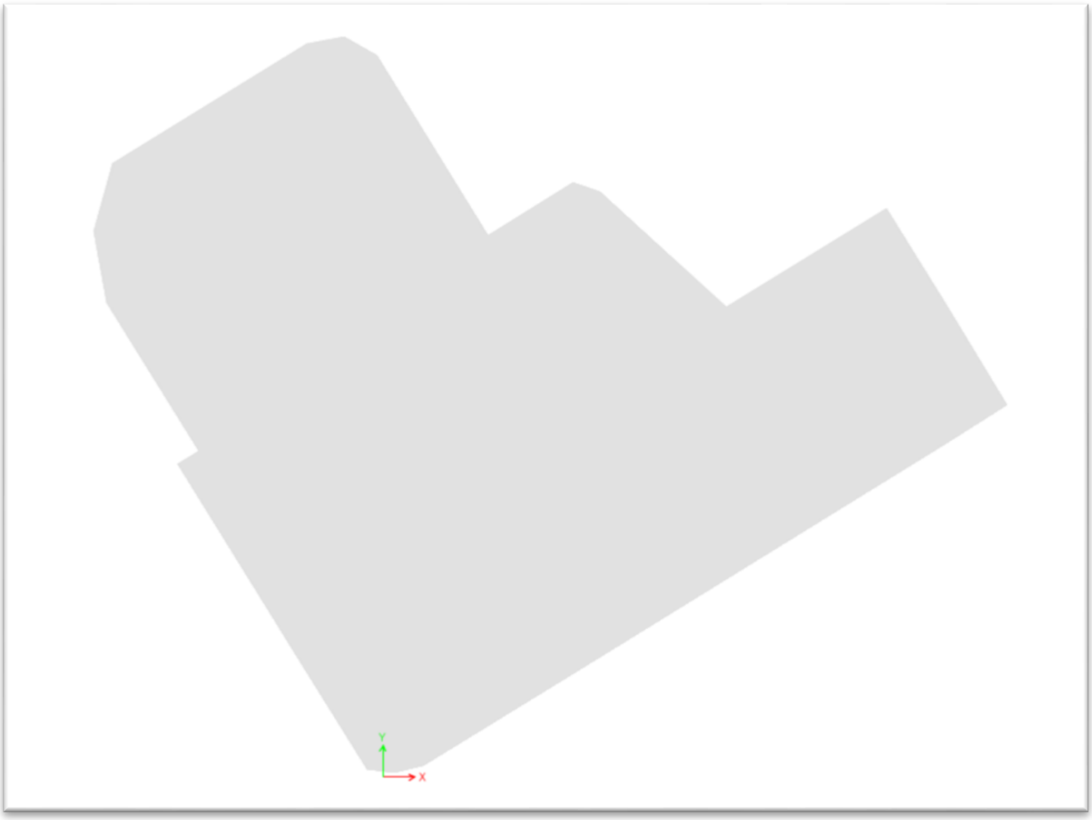


Figure 3.6-23:Mat Foundation Layout.

The foundation thickness is 800 mm.

- Check foundation for soil pressure, figure 3.6-24 shows the soil pressure on the mat foundation.

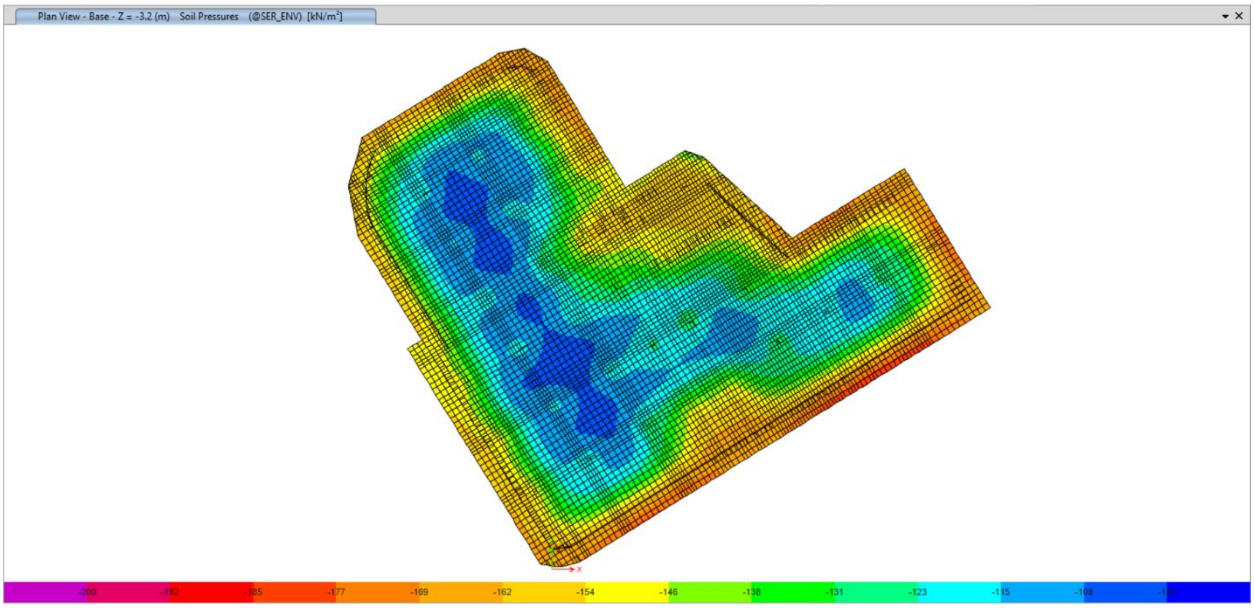


Figure 3.6-24:Soil Pressure on the mat foundation.

The soil pressure on the foundation is less than 200 kN/m^2, so it's **OK**.

- Check the wide beam shear using no stirrups for it, taking b_w = 1000 mm.

$\emptyset\ V_c = 0.75 * 0.66\ \lambda_s\ \lambda\ \rho^{1/3} \sqrt{f'c}\ b_w\ d$

$\emptyset\ V_c = 0.75 * 0.66 * 1 * 1 * 0.002^{1/3} * \sqrt{28} * 1000 * (800\text{-}50) = 247.50$ kN

Figures 3.6-25 and 3.6-26 show the shear on the foundation using a section cuts.

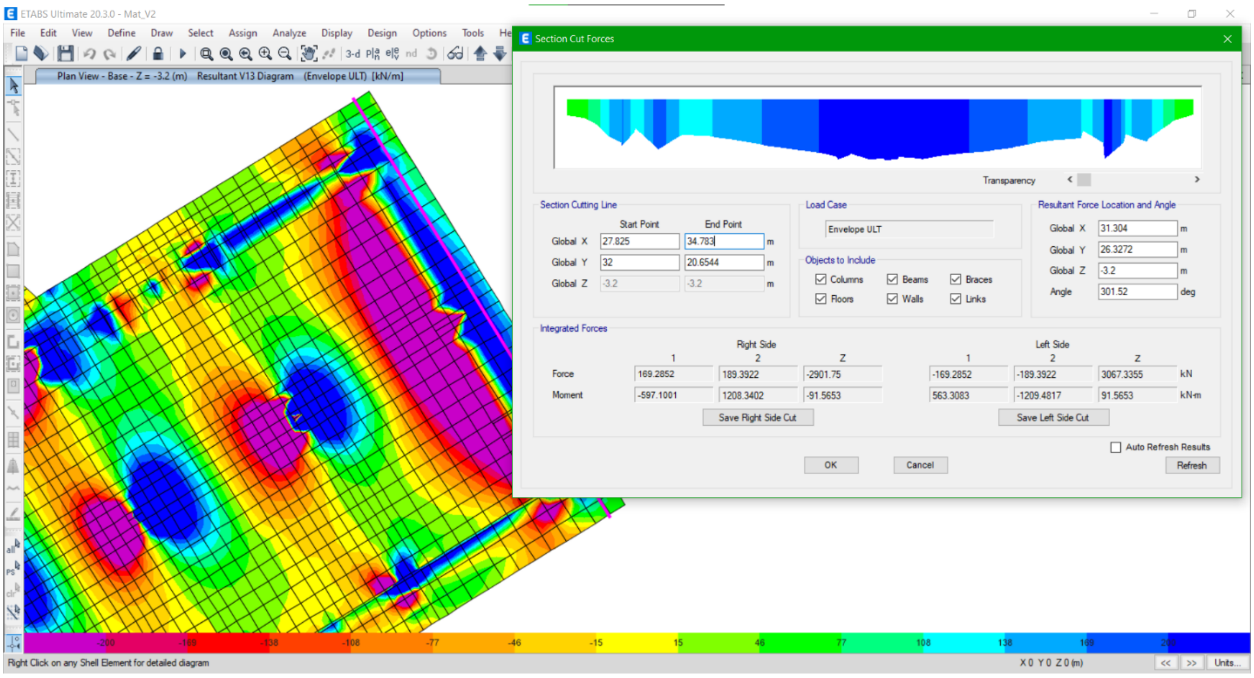


Figure 3.6-25:Wide beam shear check V13.

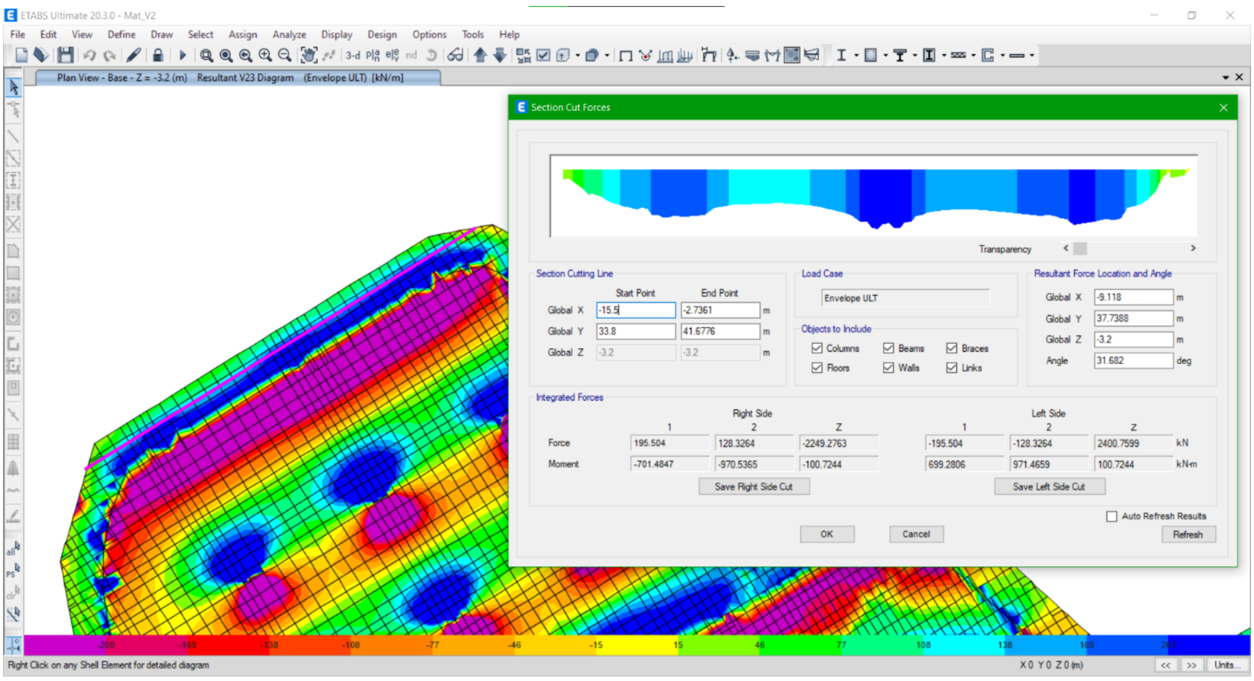


Figure 3.6-26:Wide beam shear check V23.

By $V_{13} = \frac{2901.75+3067.34}{2} \div 13.30$ **(the length of the section cut, since we took the b_w = 1000mm)**

$\rightarrow V_{13} = 224.25$ kN $< \emptyset V_c = 247.50$

By $V_{23} = \frac{2249.28+2400.76}{2} \div 15.00$ **(the length of the section cut, since we took the b_w = 1000mm)**

$\rightarrow V_{23} = 155.00$ kN $< \emptyset\ V_c = 247.50$

The shears V_{13} and V_{23} are less than $\Phi\emptyset\ V_c = 247.50$.

- Check for punching:

Figure 3.6-36 shows the punching shear on the foundation.

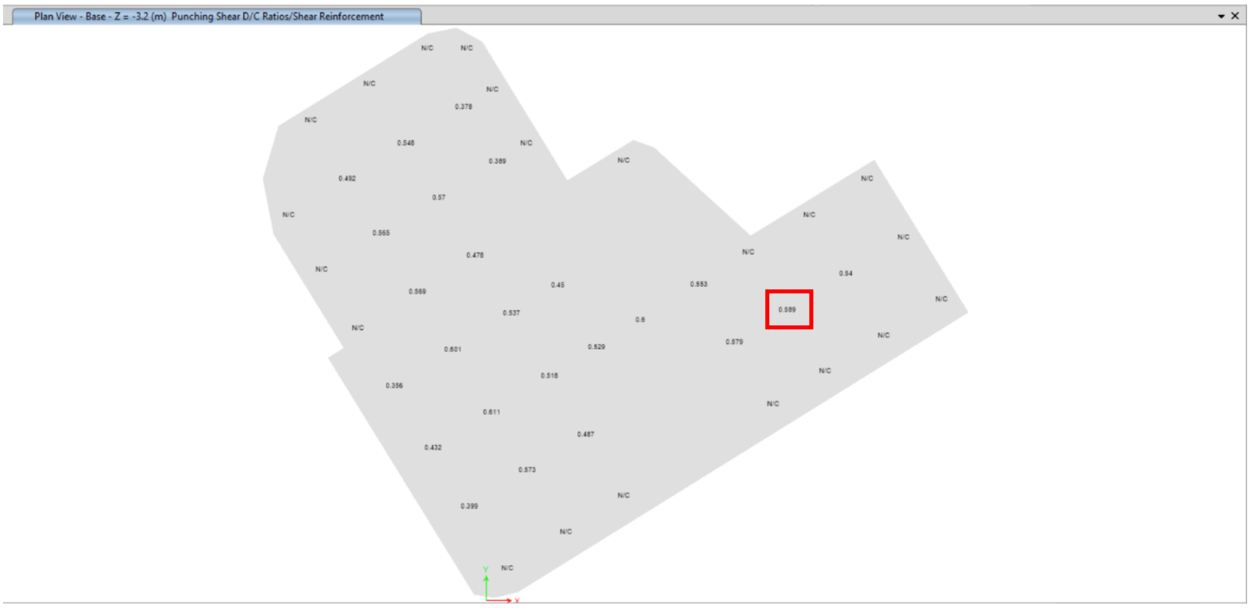


Figure 3.6-27:Punching on the mat foundation.

The punching values are less than 0.9, so it’s **OK**.

Using the ACI 318-19 formulas to compute the shear-moment transfer (punching shear) stresses using the following column loads at the connection with slab for this column.

Figure 3.6-28 shows the data sheet for punching shear design for column in ground floor.

Geometric Properties

Combination = Envelope ULT

Story = Base

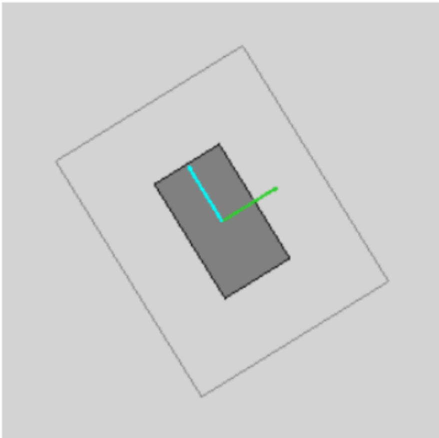
Point Label = 73

Column Shape = Rectangular

Column Location = Interior

Global X-Coordinate = 21.9735 m

Global Y-Coordinate = 20.6878 m



Column Punching Check

Avg. Eff. Slab Thickness = 744 mm

Eff. Punching Perimeter = 5176 mm

Cover = 56 mm

Conc. Comp. Strength = 28 MPa

Reinforcement Ratio = 0

Section Inertia I22 = 135983721.1 cm^4

Section Inertia I33 = 96718555.6 cm^4

Section Inertia I23 = 0 cm^4

Gamma $_{v2}$ = 0.428243

Gamma $_{v3}$ = 0.372406

Moment Mu2 = -19.6757 kN-m

Moment Mu3= -31.5838 kN-m

Shear Force = -2946.2066 kN

Unbalanced Moment Mu2 = -8.426 kN-m

Unbalanced Moment Mu3= -11.762 kN-m

Max Design Shear Stress = 0.78 MPa

Conc. Shear Stress Capacity = 1.32 MPa

Punching Shear Ratio = 0.59

Figure 3.6-28:Data sheet for punching shear design for column in ground story.

P_u = 2946.21 KN

M_x = 19.68 KN.m

M_y = 31.58 KN.m

The punching shear stress resistance of the slab (∅ V_c) should be larger than V_u → Demand/Capacity ratios for punching shear < 0.9

V_c shall be calculated in accordance with Table 22.6.5.2 in ACI 318-19. As shown in table 3.6-13.

Table 3.6-13:Vc formulas for two-way slab.

Table 22.6.5.2—v_c for two-way members without shear reinforcement

v_c		
Least of (a), (b), and (c):	$0.33\lambda_s\lambda\sqrt{f_c'}$	(a)
	$\left(0.17+\frac{0.33}{\beta}\right)\lambda_s\lambda\sqrt{f_c'}$	(b)
	$\left(0.17+\frac{0.083\alpha_s d}{b_o}\right)\lambda_s\lambda\sqrt{f_c'}$	(c)

b_0: the perimeter length of the critical zone = 2 (700+744) +2(400+744) =5176 mm.

d: the effective depth of the slab = 744 mm.

Ɓ = ratio of $\frac{lond\ side}{short\ side}$ of column = $\frac{700}{400}$ = 1.75

$\dot{\alpha}_s$: factor describes the location of the column= 40 for interior column

λ_s: factor used to modify shear strength on the effects of member depth

for (Foundations) → λ_s =1

→∅ *V_c =0.75 * 0.33 * 1 * 1 * $\sqrt{28}$ = 1.31 MPa

→ Vu = $\frac{Pu}{A}$ + (γ_{v1}*M_{ux}*Y'/ J_{cx}) + (γ_{v2}*M_{uy}*X' / J_{cy}) = 0.77 Mpa

Where:

J_{cx} = 135983721.1 cm^4

J_{cy} = 96718555.6 cm^4

X' = (700/2) +(744/2) = 722 mm

Y' = (400/2) +(744/2) = 572 mm

γ_x = 1- (1 / (1 + (2 /3) * (b_x/b_y) ^0.5)) = 0.43

γ_y = 1- (1 / (1 + (2 /3) * (b_y/b_x) ^0.5)) = 0.37

b_x = 700 + 744 = 1444 mm, by = 400 + 744= 1144 mm

b_o = 1444*2 + 1144*2 = 5176 mm

D/C ratio for punching by hand = 0.77/1.31 = 0.59 < 0.9 → Ok for punching shear.

D/C ratio for punching from ETABS = 0.59 < 0.9 → Ok for punching shear.

Diff % = 0.00%

3.7 Design of Slabs: -

- Check punching shear:

Demand/Capacity ratios for punching shear in slab should be less than 0.9

1) From ETABS: all slabs have ratios less than 0.9 for punching shear. OK

Figure 3.7-1 shows the demand/capacity ratios for punching shear in slab in ground floor.

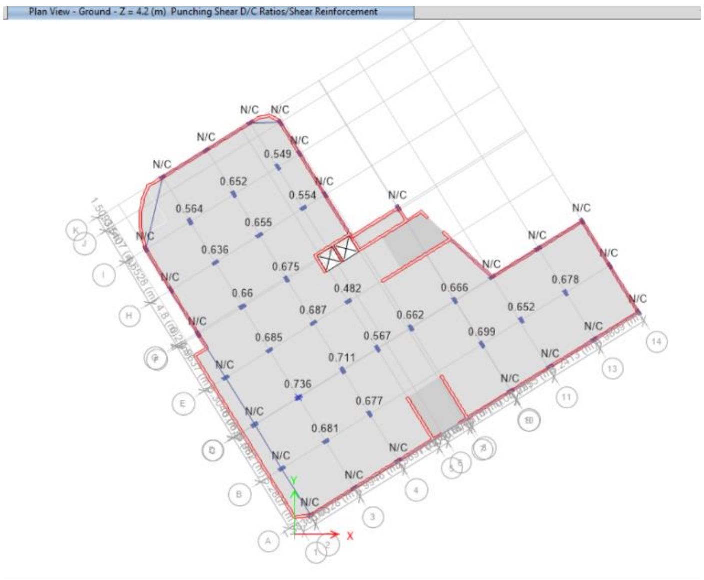


Figure 3.7-1:Demand/Capacity ratios for punching shear in slab in ground floor.

2) Manual check:

A certain column in ground floor will be taken as a sample, as shown in figure 3.7-2.

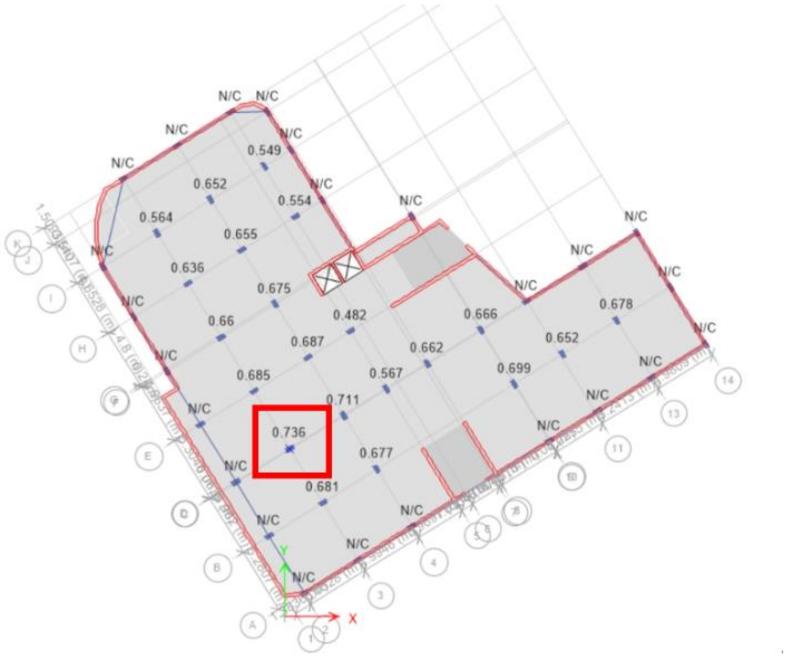


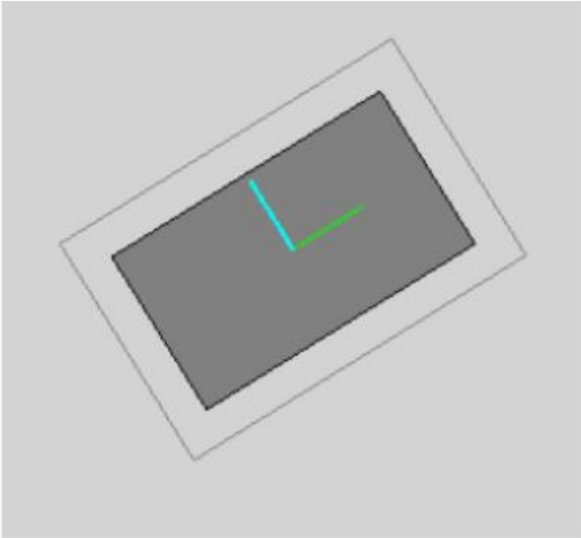
Figure 3.7-2:The column selected for the punching check.

Thickness of slab =210 mm

Column dimension =700*400 mm

Use the ACI 318-19 formulas to compute the shear-moment transfer (punching shear) stresses using the following column loads at the connection with slab for this column.

Figure 3.7-3 shows the data sheet for punching shear design for column in ground floor.



Column Punching Check

Avg. Eff. Slab Thickness = 164 mm

Eff. Punching Perimeter = 2856 mm

Cover = 46 mm

Conc. Comp. Strength = 28 MPa

Reinforcement Ratio = 0

Section Inertia I22 = 2785486.2 cm^4

Section Inertia I33 = 5278837.2 cm^4

Section Inertia I23 = 0 cm^4

Gamma $_{v2}$ = 0.350072

Gamma $_{v3}$ = 0.452096

Moment Mu2 = 16.1209 kN-m

Moment Mu3= 4.0588 kN-m

Shear Force = 420.7904 kN

Unbalanced Moment Mu2 = 5.6435 kN-m

Unbalanced Moment Mu3= 1.8349 kN-m

Max Design Shear Stress = 0.97 MPa

Conc. Shear Stress Capacity = 1.32 MPa

Punching Shear Ratio = 0.74

Figure 3.7-3:Data sheet for punching shear design for column in ground story.

Pu=420.79 KN

Mx=16.12 KN.m

My=4 KN.m

The punching shear stress resistance of the slab (V_c) should larger than V_u → Demand/Capacity ratios for punching shear < 0.9

According to ACI 318-19, for non-prestressed slabs where $\Delta_x / h_{xx} < 0.005 \rightarrow$ no need shear reinforcement.

Figure 3.7-4 shows the criteria of the ACI code concerning the shear reinforcement for slabs.

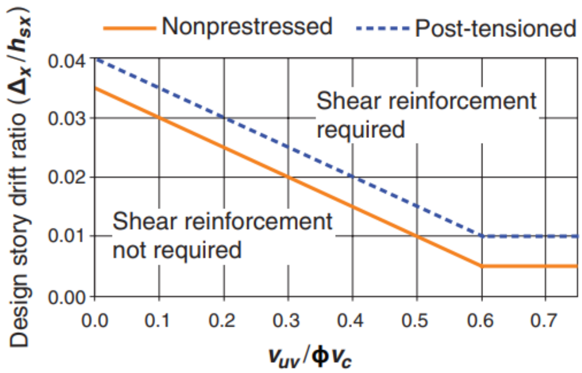


Fig. R18.14.5.1—Illustration of the criteria of 18.14.5.1.

Figure 3.7-4:Criteria of the ACI code concerning the shear reinforcement for slabs.

All values Δ_x / h_{xx} in our project were less than 0.005, then V_c shall be calculated in accordance with Table 22.6.5.2 in ACI 318-19. As shown in table 3.7-1.

Table 3.7-1:Vc formulas for two-way slab.

Table 22.6.5.2—v_c for two-way members without shear reinforcement

v_c		
Least of (a), (b), and (c):	$0.33\lambda_s\lambda\sqrt{f_c'}$	(a)
	$\left(0.17+\frac{0.33}{\beta}\right)\lambda_s\lambda\sqrt{f_c'}$	(b)
	$\left(0.17+\frac{0.083\alpha_s d}{b_o}\right)\lambda_s\lambda\sqrt{f_c'}$	(c)

b_0: the perimeter length of the critical zone = 2 (700+164) +2(400+164)   =2856mm.

d: the effective depth of the slab = 164 mm.

Ƀ = ratio of $\frac{lond\ side}{short\ side}$ of column = $\frac{700}{400}$=1.75

$\dot{\alpha}_s$: factor describes the location of the column= 40 for interior column

λ_s: factor used to modify shear strength on the effects of member depth

for (d < 250mm) → λ_s =1

→ $\emptyset$ *V_c =0.75*0.33*1*1*$\sqrt{28}$ =1.31 MPa

→Vu = $\frac{Pu}{A}$ + (γ_{v1}*M_{ux}*Y'/ J_{cx}) + (γ_{v2}*M_{uy}*X' / J_{cy}) =0.984 Mpa

Where:

J_{cx} =2785486.2 cm^4

J_{cy} =5278837.2 cm^4

X' = (700/2) +(164/2) = 432 mm

Y' = (400/2) +(164/2) = 282mm

γ_x = 1- (1 / (1 + (2 /3) * (b_x/b_y) ^0.5)) = 0.45

γ_y = 1- (1 / (1 + (2 /3) * (b_y/b_x) ^0.5)) = 0.35

b_x = 700+164 = 864mm, by=400+164=564mm

D/C ratio for punching by hand = 0.984/1.31 =0.75 < 0.9 → Ok for punching shear.

D/C ratio for punching from ETABS =0.74 < 0.9 → Ok for punching shear.

Diff % = ((0.75 -0.74)/0.74) *100% =1.35%

- Wide beam shear:

According to ACI 318-19, the shear strength capacity of the slab is given by:

$\emptyset$*V_c =0.75*0.66*((λ_s * λ * ρ_w $^{1/3}$*$\sqrt{F'c}$ +$\frac{Nu}{6\times Ag}$)) b_w*d

The ground slab with thickness 210 mm, will be taken as a sample.

Let ρ_w = 0.0018$\frac{h}{d}$ =0.0018*$\frac{210}{180}$=0.0021

Nu : axial force = 200 KN/m

A_g = 1000*210 =210000 mm^2

$$\emptyset * V_c = \frac{0.75\left(\left(0.66*1*0.0021^{\frac{1}{3}}*\sqrt{28}\right)-\left(200*\frac{1000}{6*210000}\right)\right)*1000*180}{1000} = 39KN$$

From ETABS, figures 3.7-5 to 3.7-8 show the ultimate shear and axial force in the slab.

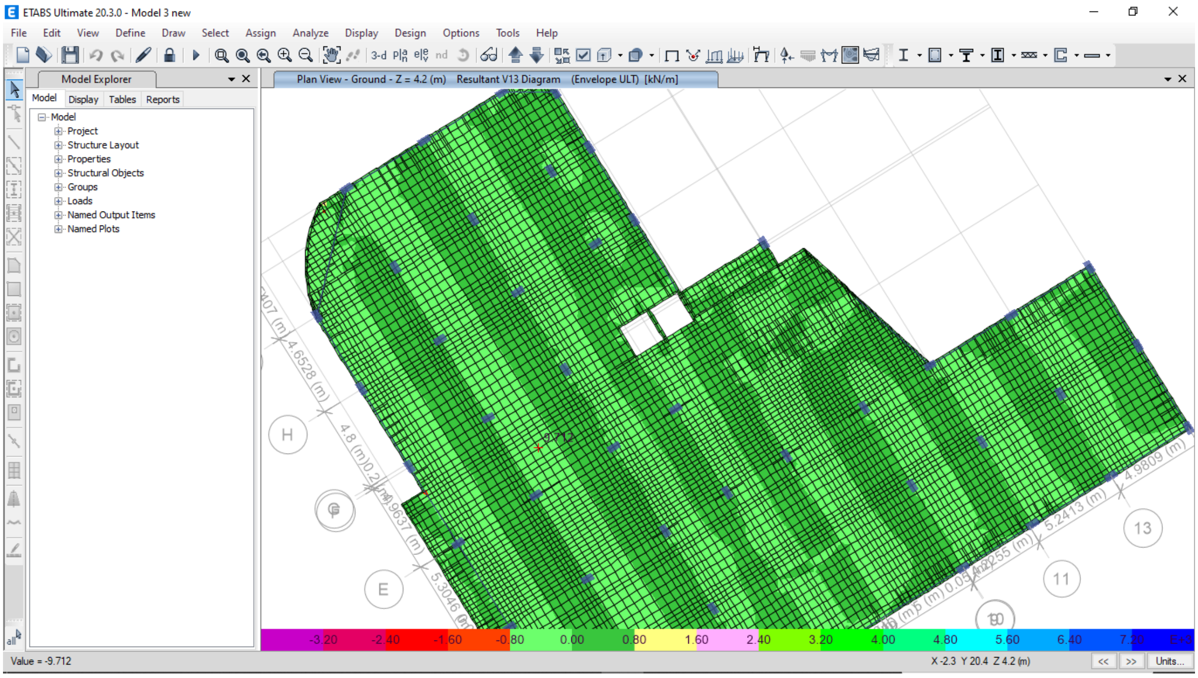


Figure 3.7-5:Ultimate shear force in the slab in x-direction, V13.

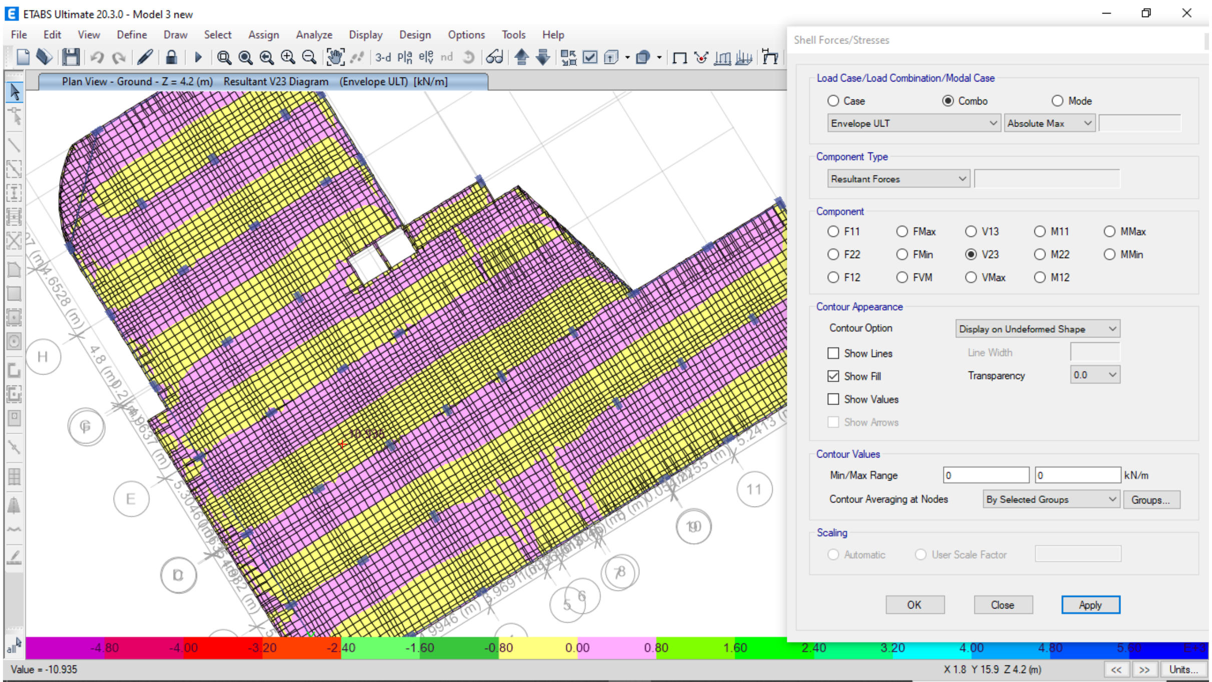


Figure 3.7-6:Ultimate shear force in the slab in y-direction, V23.

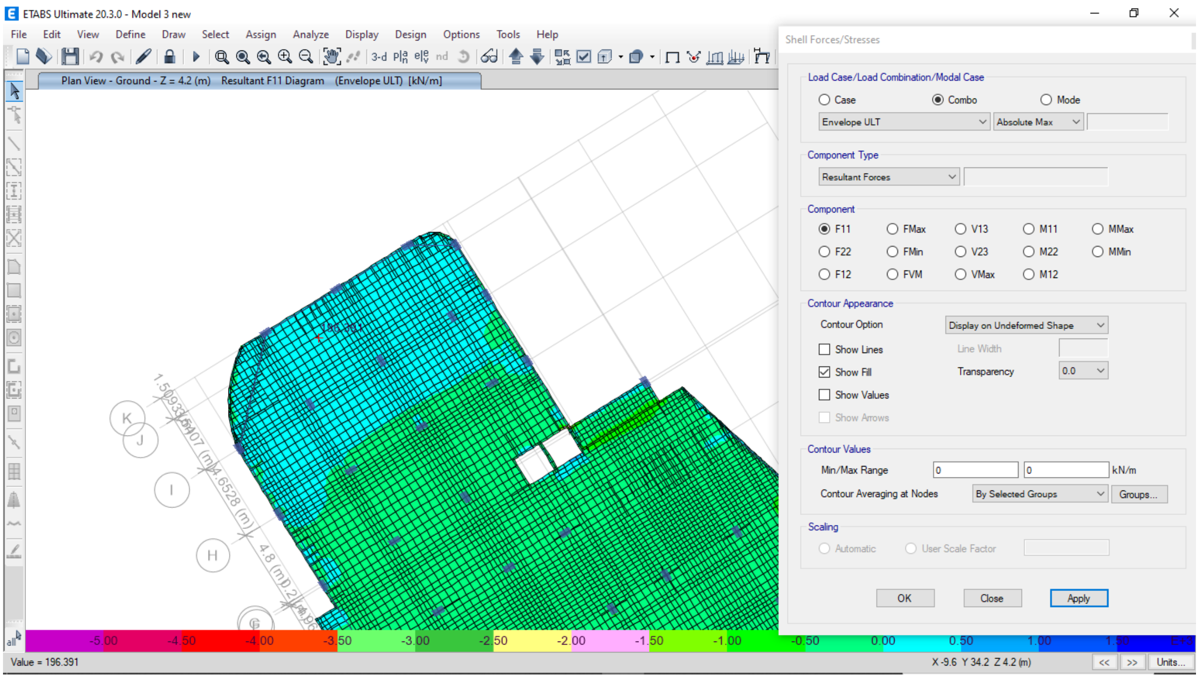


Figure 3.7-7:Axial force in the slab in X-direction, F11.

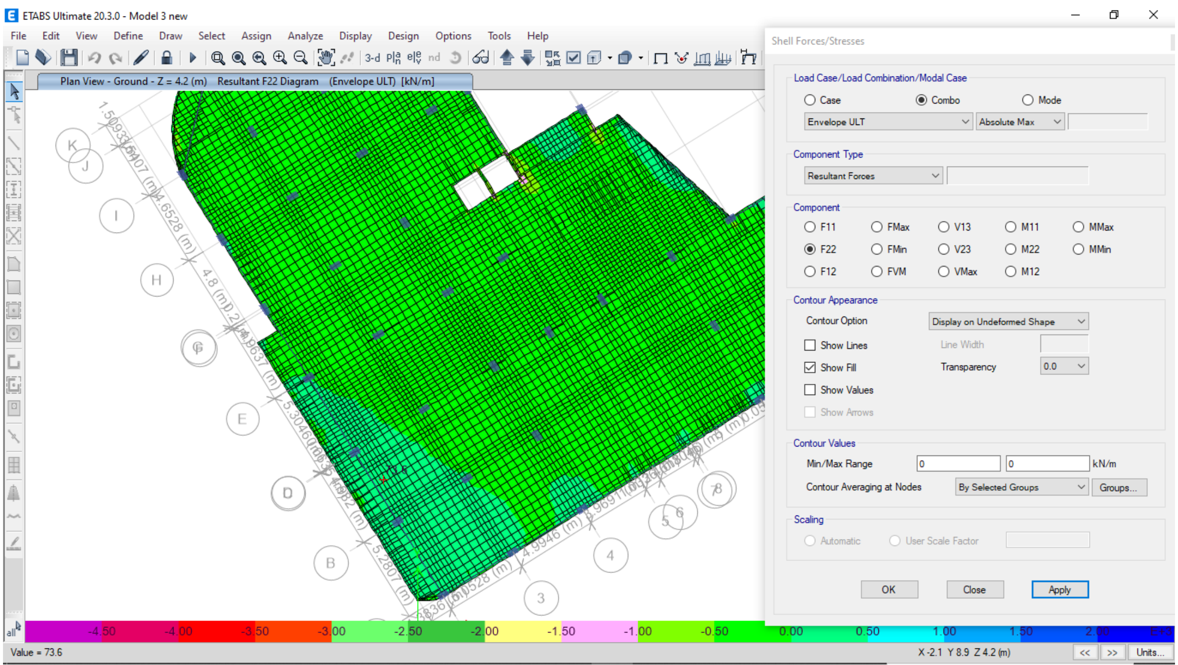


Figure 3.7-8:Axial force in the slab in Y-direction, F22.

$V_u < \emptyset * V_c$

All values of V_u less than $\emptyset * V_c \rightarrow$ The slab needs no shear reinforcement.

Table 3.7-2, shows the ultimate shear forces and shear strength capacity for each slab.

Table 3.7-2:Shear strength capacity and ultimate shear forces.

No. of floor	$N_{u\ max}$	$\emptyset*V_c$	$V_{u\ max}$	OK/Not ok
Basement (h=220mm)	140 KN	48.5 KN	15 KN	OK
Ground (h=210mm)	200 KN	39 KN	12 KN	OK
First floor (h=210mm)	80 KN	52 KN	15 KN	OK
2^{nd} floor (h=210mm)	75 KN	52.34 KN	20 KN	OK
3^{rd} floor (h=210mm)	110 KN	48.6 KN	20 KN	OK
4^{th} floor (h=210mm)	80 KN	52 KN	15 KN	OK
5^{th} floor (h=180mm)	90 KN	41.5 KN	15 KN	OK
Roof floor (h=180mm)	160 KN	34 KN	20 KN	OK

- Design of slabs for flexure:

Maximum bending moment (M11) in ground floor = 23 KN.m/m, as shown in figure 3.7-9.

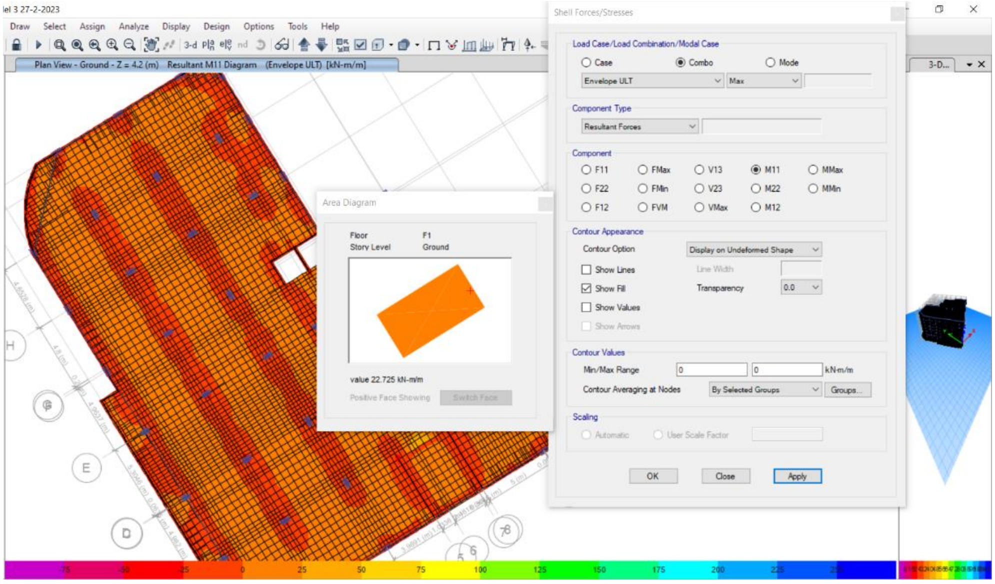


Figure 3.7-9:Max ultimate bending moment M11 in ground floor.

$$\rho = \frac{0.85f'_c}{f_y}\left(1 - \sqrt{1 - \frac{2.61M_u}{bd^2 f'_c}}\right)$$

Bottom steel:

ρ =0.0019, b = 1000, h=210, clear cover = 30

As = ρ b d = 0.0019 × 1000 ×180= 342 mm^2.

As min= ρ min × b × d = 0.0018 × 1000 ×210 = 378 mm^2.

As < $A_{s\ min}$ → Use $A_{s\ min}$

378 mm^2 →1Ø 12 / 250 mm.

Reinforcement for the slabs is shown in Table 3.7-3.

Table 3.7-3:Reinforcement for the slabs.

No. Floor	Bottom steel in direction 1	Top steel in direction 1	Bottom steel in direction 2	Top steel in direction 2
Basement	Ø 12 / 250 mm.	Ø 12 / 250 mm.	Ø 12 / 250 mm.	Ø 12 / 250 mm.
Ground	Ø 12 / 250 mm.	Ø 12 / 250 mm.	Ø 12 / 250 mm.	Ø 12 / 250 mm.
1st	Ø 12 / 250 mm.	Ø 12 / 250 mm.	Ø 12 / 250 mm.	Ø 12 / 250 mm.
2nd	Ø 12 / 250 mm.	Ø 12 / 250 mm.	Ø 12 / 250 mm.	Ø 12 / 250 mm.
3rd	Ø 12 / 250 mm.	Ø 12 / 250 mm.	Ø 12 / 250 mm.	Ø 12 / 250 mm.
4th	Ø 12 / 250 mm.	Ø 12 / 250 mm.	Ø 12 / 250 mm.	Ø 12 / 250 mm.
5th	Ø 10 / 200 mm.	Ø 10 / 200 mm.	Ø 10 / 200 mm.	Ø 10 / 200 mm.
Roof	Ø 10 / 200 mm.	Ø 10 / 200 mm.	Ø 10 / 200 mm.	Ø 10 / 200 mm.

- **Note:** Additional top reinforcement is used around columns and illustrated in detailed slab reinforcement drawings.

- Detailing and requirements:
- The development length in tension, L_d is given by:

$$L_d \geq (0.48 * F_y * d_b / \sqrt{F'c}\,) \geq 300\text{mm} \rightarrow \text{for } d_b < 20\text{mm}$$

$$L_d \geq (0.59 * F_y * d_b / \sqrt{F'c}\,) \geq 300\text{mm} \rightarrow \text{for } d_b > 20\text{mm}$$

- In tension splicing, use 1.3*L_d:
  - $L_s (Ø12) = 1.3 * (0.48 * 420 * 12 / \sqrt{28}) = 594.34$ mm
  - $L_s (Ø10) = 1.3 * (0.48 * 420 * 10 / \sqrt{28}) = 495.28$ mm

3.8 **Design of Beams: -**

In this section, a sample frame as shown in Figure 3.8.1 in the basement, will be designed and detailed according to ACI 318-19 special moment frame requirements. Design results will be taken from ETABS after it was checked previously in section 3.6.1, the higher design result from the two models (full model and frames only model) will be used.

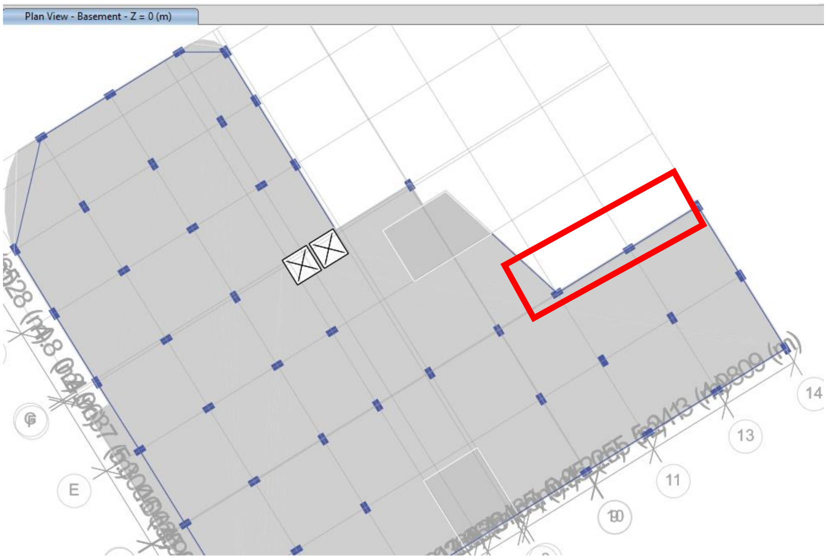


Figure 3.8-1:The beams selected for the design.

The frame includes B7 and B8 is taken as a sample for special beam details. Figures 3.8.2, 3.8.3 and 3.8.4 show the total longitudinal reinforcement, the shear plus torsion reinforcement and the torsional longitudinal reinforcement respectively.

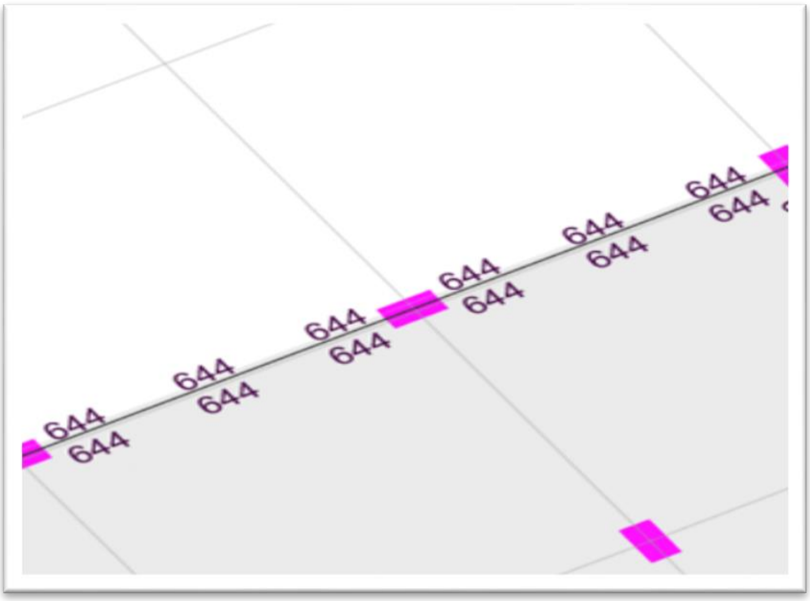


Figure 3.8-2:Total longitudinal reinforcement for beam B7 & B8 in ETABS.

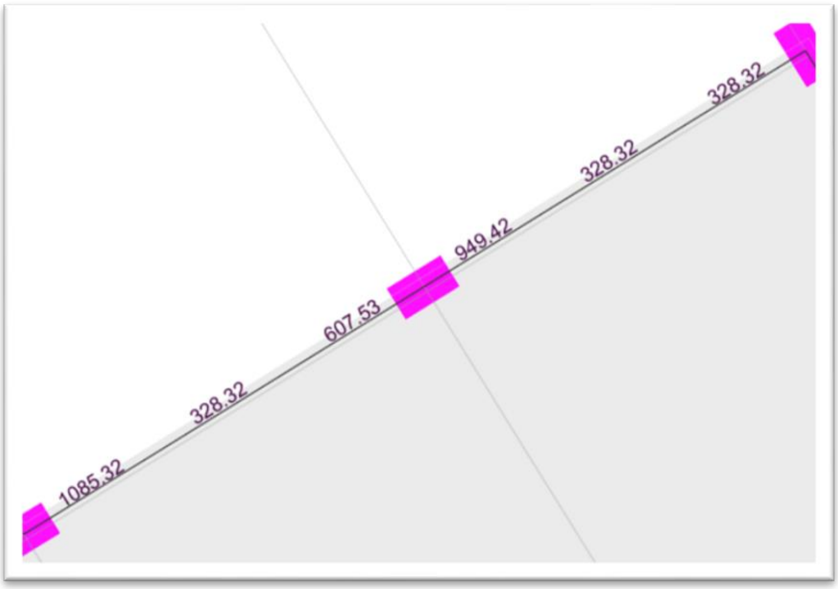


Figure 3.8-3:Shear plus torsion reinforcement for beam B7 & B8 in ETABS.

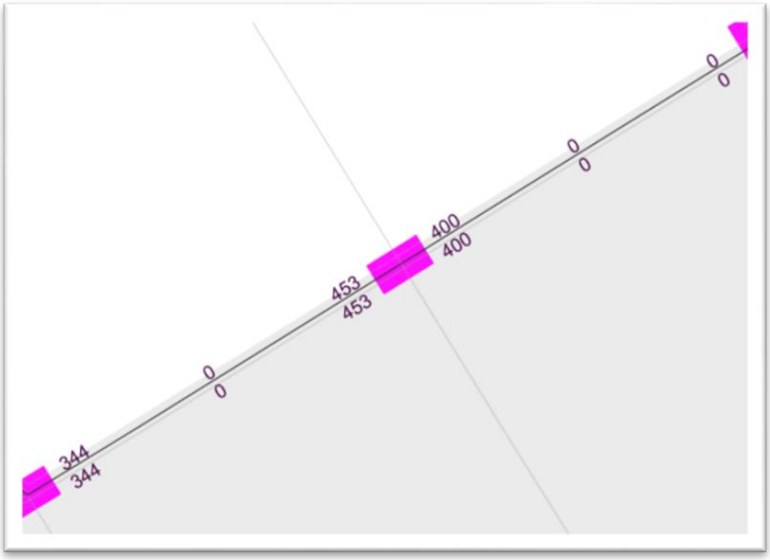


Figure 3.8-4:Torsional longitudinal reinforcement for beam B7 & B8 in ETABS.

❖ Longitudinal reinforcement:

Beam width, b= 400mm.
Beam thickness, h= 550mm.

Bottom Steel in the first and second spans: As = 644 mm → use 6Ø12.
Top Steel in the first and second spans: As = 644 mm → use 6Ø12.

- Development Lengths: L_{dt} = (0.48∗420 *12)/√28 = 457 mm.
  Splice length, L_s = 1.3* L_{dt} = 594 mm.
  L_s in top steel =1.3*1.3* L_{dt}= 772.33 mm.

❖ Transverse reinforcement: the transverse longitudinal steel that resist the shear forces.

Assume stirrup Ø12:
A_v = 2*113 = 226 mm^2
S = 226/1.085 = 208 mm

Hoops and stirrups:

- In distance 2h= 2(0.55) =1.1m:

spacing shall be the minimum of:

1. d/ 4 = 490/ 4 = 122mm
2. 150mm
3. 6*d_b = 6(12) = 72mm → take s = 70mm.

- At splicing zones:

spacing shall be the minimum of:

1. d/ 4 = 490/ 4 = 122mm
2. 100mm → take s = 100mm.

- In other places: s = d/2 = 490 /2 = 200mm.

❖ At middle we put one-third of torsional reinforcement.

A_s middle = 453/3 = 151 mm^2 → use 2 Ø10

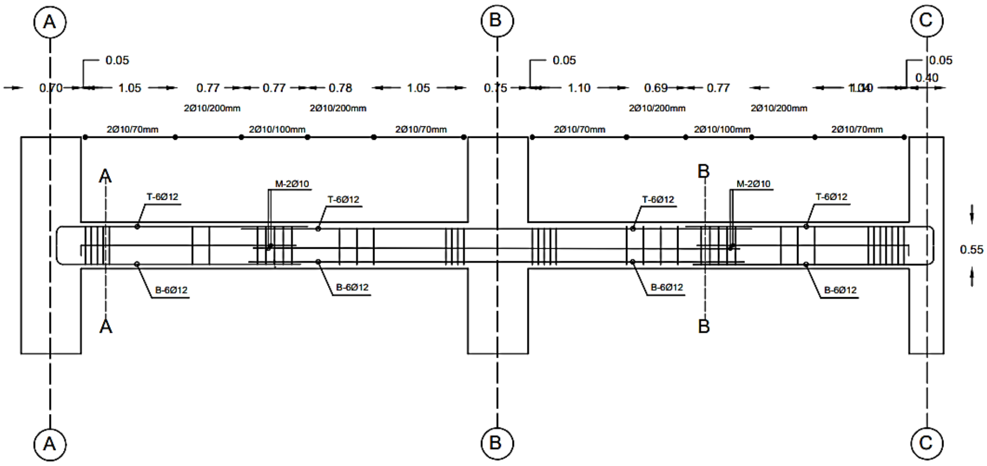


Figure 3.8-5:Longitudinal section of beams, B7 & B8.

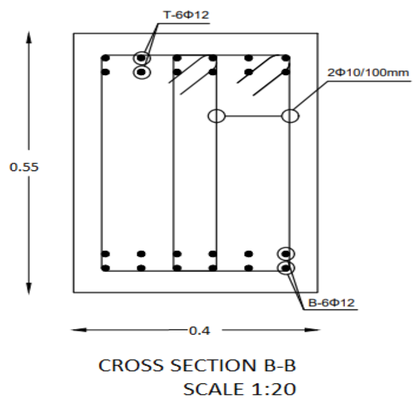


Figure 3.8-6:Cross section of beam.

3.9 **Design of Column:**

In this section, a sample column as shown in **Figure 3.9.1** in the basement, will be designed and detailed according to ACI 318-19 special moment frame requirements. Design results will be taken from ETABS after it was checked previously in section 3.6.2, the higher design result from the two models (full model and frames only model) will be used.

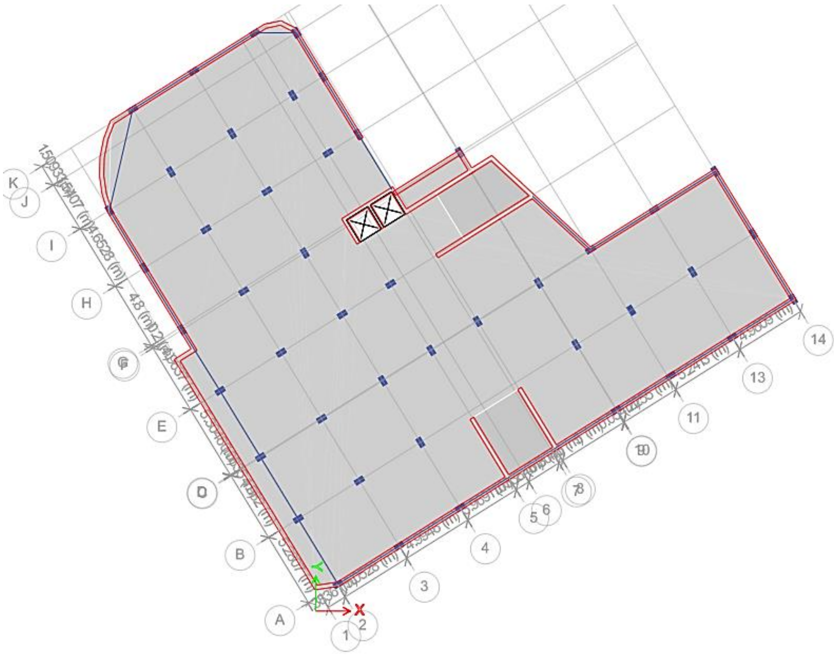


Figure 3.9-1:The column selected for the design.

❖ Longitudinal reinforcement:

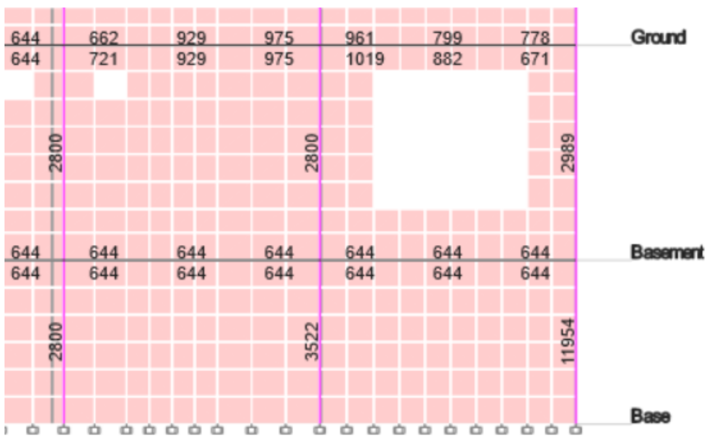


Figure 3.9-2:Longitudinal reinforcement for columns of special moment frames.

longitudinal reinforcement As = 11954 $mm^2 \rightarrow 26Ø25$

Steel ratio = $\frac{26*\frac{25^2}{4}\pi}{700*400}$ = 0.045 < 0.06   OK.

- ❖ Transverse reinforcement:
  Transverse reinforcement is required in columns to confine the core concrete, provide lateral support to longitudinal reinforcement.

Pu = 4061.92 > 2352 KN

h_x = ($\frac{700-2*40-2*12-25}{9}$) = 63.44 mm

h_x =63.44< 200 → Ok.

Smax = min :

- 0.25 * 400 = 100 mm
- 6 * 25 = 150 mm
- So = 100 + ($\frac{350-63.44}{3}$) = 195.52 mm > 150 mm , so So =150 mm.

Smax = 100 mm

Kf = $\frac{28}{175}$ + 0.6 = 0.75 < 1 , so Kf = 1

Kn = $\frac{26}{26-2}$ = 1.0833

A_{ch} = (700 – 80) (400 –80)= 198.4*10^3 mm^2

Ag = 700 * 400 = 280000 mm^2

From **figure 3.9.3,** Pu = 4062.92 > 2352 kN ➔ $Ash/\ sbc$ = maximum of (a), (b), and (c):

a) 0.3 ($\frac{280000}{198.4*103}$ - 1) $\frac{28}{420}$ = 8.22*10^{-3}

b) 0.09 * $\frac{28}{420}$ = 6 * 10^{-3}

c) 0.2 * 1 * 1.0833* $\frac{4062*10^3}{420*198.4*10^3}$ = 0.01056

largest value for $\frac{Ash}{Sbc}$ = 0.01056

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)

Figure 3.9-3:Transverse reinforcement for columns of special moment frames.

For long direction:

A_{sh} = 100*(400 – 40*2) * 0.01056 = 337.96 mm^2

$\frac{337.96}{\frac{\pi}{4}12^2} \rightarrow$ 2.99 = 3 Leg, .

For short direction

A_{sh} = 100 * (700 – 40*2)* 0.01056= 654.72 mm^2

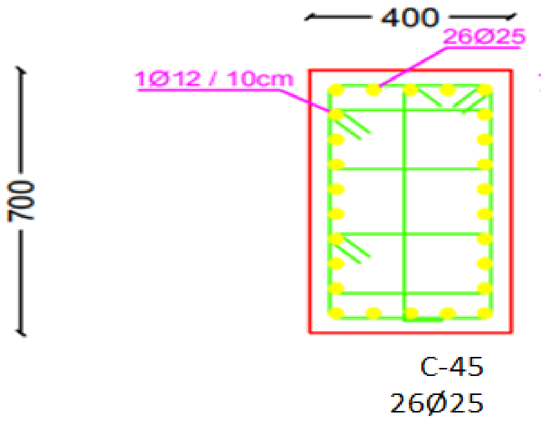


Figure 3.9-4:Cross section for column C45 in basement.

3.10 **Design of Walls:**

In this section, a wall samples (pier and spandrel) as shown in **Figure 3.10.1** in the 1st floor on the southern elevation, will be designed and detailed according to ACI 318-19. Design results will be taken from ETABS after it was checked previously in section 3.6.4, the first model (walls carrying more than 98% of the seismic forces) will be used.

- **Pier Sample:**

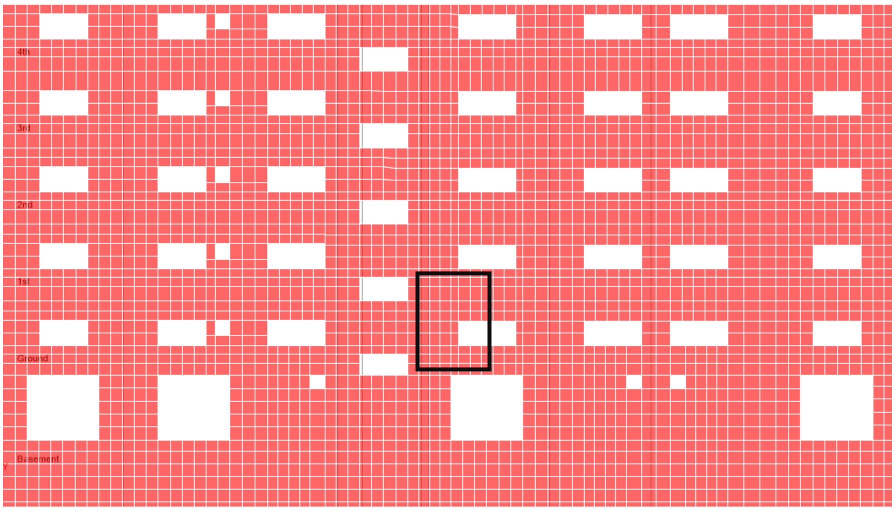


Figure 3.10-1:The Pier wall selected for the design.

Figure 3.10.2 shows the pier design details sheet.

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
1st	AP7	16833	11299.6	2120	200	0.688

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	15931.5	10742.1	17734.5	11857.2	2120	200
Bottom	Leg 1	15931.5	10742.1	17734.5	11857.2	2120	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	1060	0.0025	0.0024	Envelope ULT	1250.8993	1.2316	-201.3376	424001
Bottom	1060	0.0025	0.0024	Envelope ULT	1523.5359	-10.5074	-248.9275	424001

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	764.94	Envelope ULT	123.3408	46.7338	632.219	223.5562	632.219
Bottom	Leg 1	769.92	Envelope ULT	65.5903	149.0144	634.8785	223.5562	634.8785

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	204.6	Envelope ULT	1250.8993	-201.3376	4.29	5.6	409.2	471.1
Top–Right	Leg 1	Not Required	Envelope ULT	1250.8993	46.7338	3.26	5.6		
Bottom–Left	Leg 1	266	Envelope ULT	1523.5359	-248.9275	5.25	5.6	478	471.1
Botttom–Right	Leg 1	266	Envelope ULT	1523.5359	149.0144	4.59	5.6	478	471.1

Figure 3.10-2:Pier design details sheet.

- ❖ Longitudinal and transverse reinforcements:

  Wall length, L = 2120 mm.

  Wall thickness, b = 200 mm.

On both faces of the wall, the steel will be distributed.

Vertical steel in each face: As = 1050 mm^2 → use 1Ø12 / 300 mm.

Horizontal steel in each face: As = 769.92 mm^2 / m → use 1Ø12 / 300 mm.

Splice length for the vertical steel: $L_s = 1.3 * \frac{0.48 * fy * db\ least}{\sqrt{28}}$

$\rightarrow 1.3 * (0.48 * 420 * 10) / \sqrt{28} = 500$ mm.

❖ Boundary element reinforcement:

At the bottom from right or left of the pier wall the length of the boundary element equals 266 mm → take 300 mm.

S_{max} = min of [$\frac{1}{3}$ * least boundary element dimension,6 * d_b] → S_{max} = 66 mm

h_x = 133.33 mm

$A_{sh} \geq 0.09\ s\ b_c \frac{f'c}{fy}$

$\rightarrow A_{sh\ Y} = 0.09 * 60 * 160 * \frac{28}{420} = 57.60\ mm^2$ → Y legs Ø 10 / 60 mm

$\rightarrow A_{sh\ X} = 0.09 * 60 * 280 * \frac{28}{420} = 100.80\ mm^2$ → X legs Ø 10 / 60 mm

Then in **Y** direction use **2 legs** to close the hoop, and **X** direction use **3 legs** according to h_x. Figure 3.10-3 shows the elevation section of this pier, and figure 3.10-4 shows the cross-section of it.

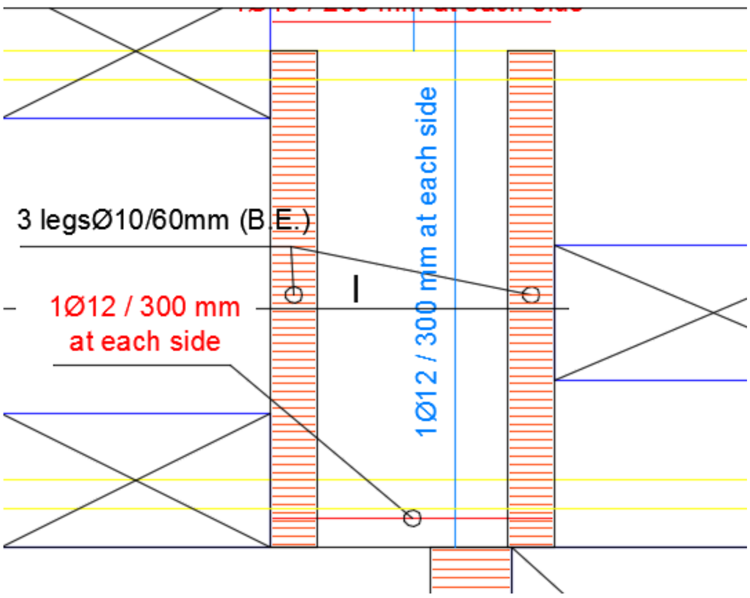


Figure 3.10-3:Pier elevation section of the wall.

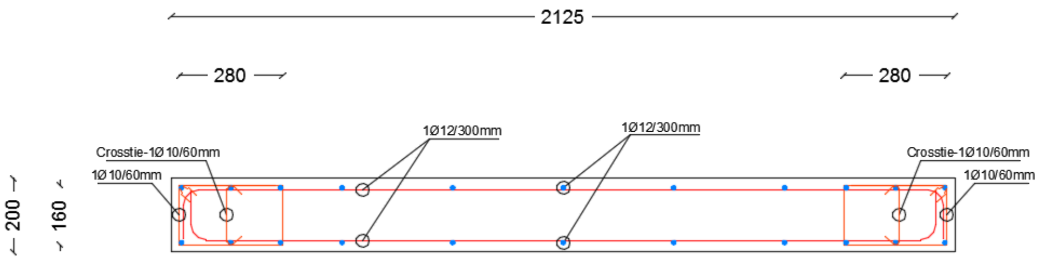


Figure 3.10-4:Cross-section of the pier.

- **Spandrel sample:**

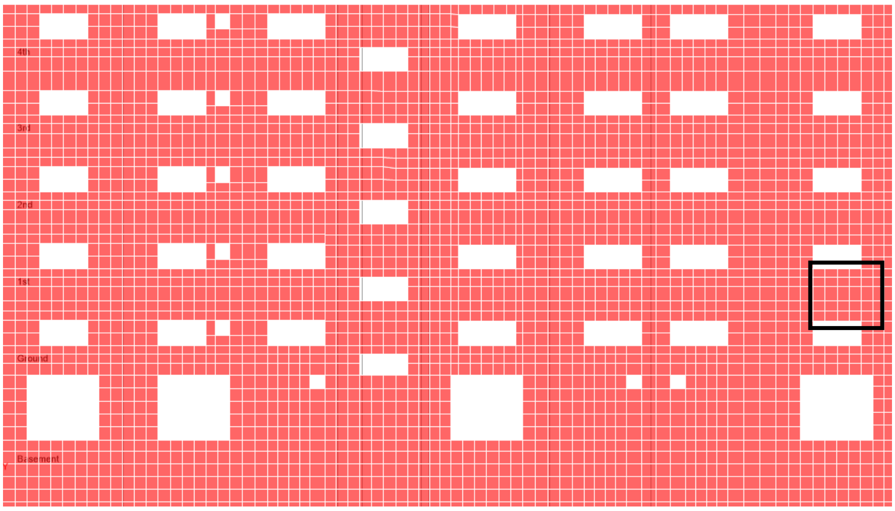


Figure 3.10-5:The spandrel wall selected for the design.

Figure 3.10.6 shows the pier design details sheet.

ACI 318-19 Spandrel Design

Spandrel Details

Story ID	Spandrel ID	Centroid X (mm)	Centroid Y (mm)	Depth (mm)	Width (mm)	LLRF
1st	AS53	30338.8	19652.4	2200	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	1389	0.32	Envelope ULT	-883.2453
Right	1300	0.3	Envelope ULT	-881.5199

Spandrel Flexural Design—Bottom Reinforcement

Station Location	Reinf Area mm²	Reinf Percentage	Reinf Combo	Moment, M_u kN-m
Left	1395	0.32	Envelope ULT	887.0215
Right	1300	0.3	Envelope ULT	794.5133

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	1391.78	500	Envelope ULT	891.9517	197.508	694.4437	891.9517
Right	1527.17	500	Envelope ULT	968.1594	206.1628	761.9967	968.1594

Spandrel Shear Design—Diagonal Reinforcement

Station Location	A_{diag} mm²	Shear Combo	V_u kN	V_{uLimit} kN	L/H Ratio	Seismic Design	Diag Reinf Mandatory
Left	1891	Envelope ULT	891.9517	695.9751	0.909	Yes	Yes
Right	2053	Envelope ULT	968.1594	695.9751	0.909	Yes	Yes

Figure 3.10-6:Pier design details sheet.

❖ Longitudinal and transverse reinforcements:

Wall height, h = 2200 mm.

Wall effective depth, d = 1980 mm.

Wall thickness, b = 200 mm.

On both faces of the wall, the shear steel will be distributed.

Horizontal Top steel: As = 1389 mm^2 → use 6Ø18.

Horizontal Bottom steel: As = 1395 mm^2 → use 6Ø18.

Horizontal middle steel: As = 500 mm^2 / m → use 1Ø10 / 300 mm.

Vertical steel: As = 1527.17 mm^2 / m → use 1Ø14 / 200 mm.

❖ Diagonal reinforcement:

The diagonal will be distributed into two groups, that intersect at the middle, and will have a development length.

Diagonal steel group 1: As = 2053 mm^2 → use 6Ø22.

Diagonal steel group 2: As = 2053 mm^2 → use 6Ø22.

$S_{max} = 6 * d_b \rightarrow S_{max} = 132$ mm.

$h_x = 350$ mm

$A_{sh} \geq 0.3\ s\ b_c\ (\frac{Ag}{Ach} - 1)\ \frac{f'c}{fy}$

$\rightarrow A_{sh\ Y} = 0.3 * 100 * 150 * (\frac{140*190}{100*150} - 1)\ \frac{28}{420}$ = 278 mm^2 → Y legs Ø 10 / 100 mm

$\rightarrow A_{sh\ X} = 0.3 * 100 * 100 * (\frac{140*190}{100*150} - 1)\ \frac{28}{420}$ = 155 mm^2 → X legs Ø 10 / 100 mm

Development length for diagonal: $L_{dt} = \frac{0.59*fy*db\ least}{\sqrt{28}}$

→ (0.59 * 420 * 22) / √28 = 1100 mm.

Then in **Y** direction use **3 legs** to satisfy the needed area of steel, and **X** direction use **2 legs** to close the hoop. Figure 3.10-7 shows the elevation section of this spandrel, and figure 3.10-8 shows the cross-section of it.

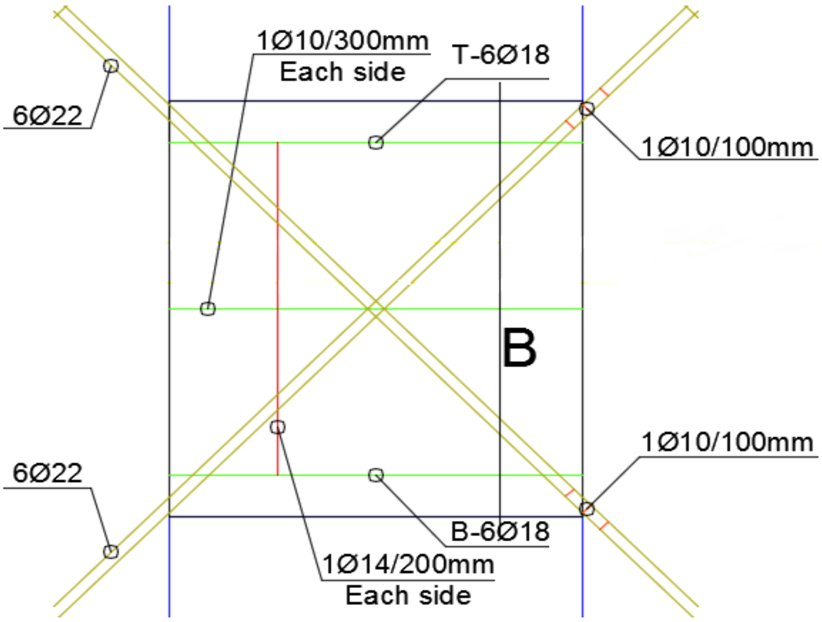


Figure 3.10-7:Spandrel elevation section of the wall.

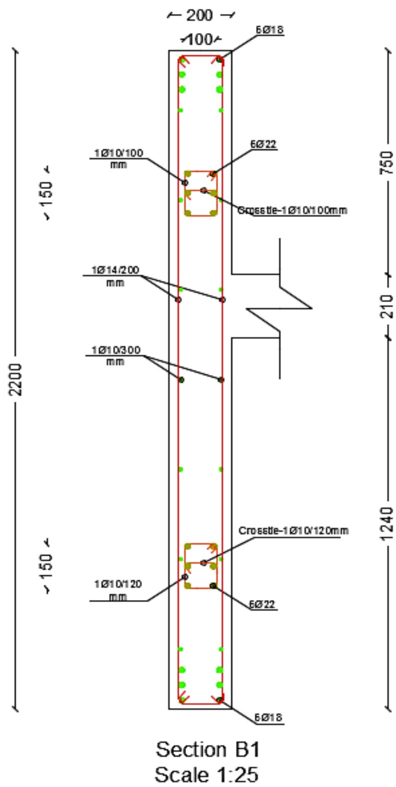


Figure 3.10-8:Cross-section of the spandrel.

3.11 **Design of footing: -**

In this project, the design of a footing must be computerized in a three-dimensional (ETABS) model in order to obtain the results of the design from the computer program after performing verifications and checks.

3.11.1 **Footing design verification**

- Moment in direction 1 = 132 KN.m, as shown in Figure 3.11-1.

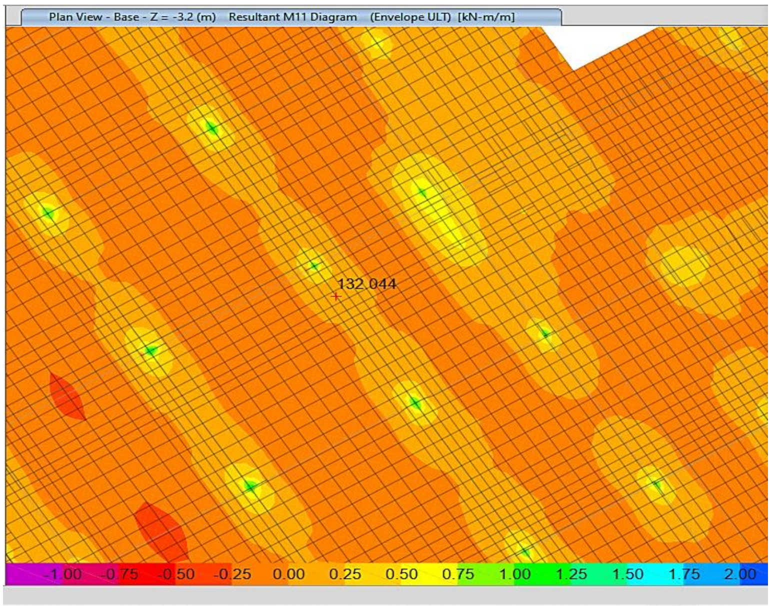


Figure 3.11-1:Moment in direction 1.

$$\rho = \frac{0.85f'_c}{f_y}\left(1-\sqrt{1-\frac{2.61M_u}{bd^2f'_c}}\right)$$

$= \frac{(0.85 * 28)}{420}\ (1 - \sqrt{1-\frac{2.61*132*10^6}{28*1000*750^2}}\ ) = 0.0006$

As = ρ b h = 0.0006 × 1000 × 750 = 450 mm^2/ m

$As_{min} = \rho_{min} \times b \times h = 0.0018 \times 1000 \times 800 = 1440\ mm^2$

As < As min, use As min = 1440 mm^2.

- Area of steel (As) in direction 1 and in bottom face = 1440 mm^2/m at the same point, as shown in figure 3.11.2.

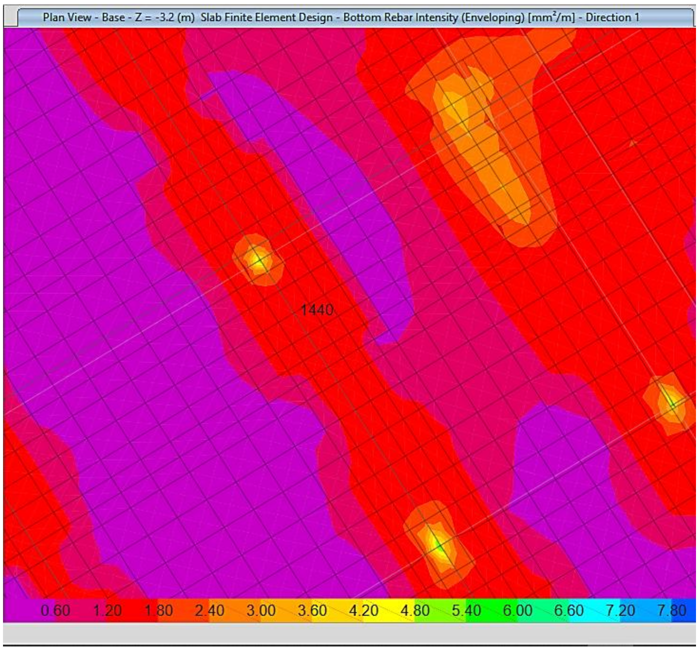


Figure 3.11-2:Area of steel in direction 1.

3.11.2 **Flexural Design:**

For mat foundation:

By using finite element method, reinforcement for both X and Y directions, have a minimum reinforcement with additional steel in some areas.

- Minimum area of steel = 0.0018 * 1000 * 800 = 1440 mm^2
- 1440mm^2 → Use Ø20 / 200mm.

X-direction:

Top reinforcement = Ø20 / 200mm

Bottom reinforcement = Ø20 / 200mm

Y-direction:

Top reinforcement = Ø20 / 200mm

Bottom reinforcement = Ø 20 / 200mm

As an example of using finite element method, reinforcement for both X and Y directions are shown in Figures 3.11.3, 3.11.4, 3.11.5 and 3.11.6.

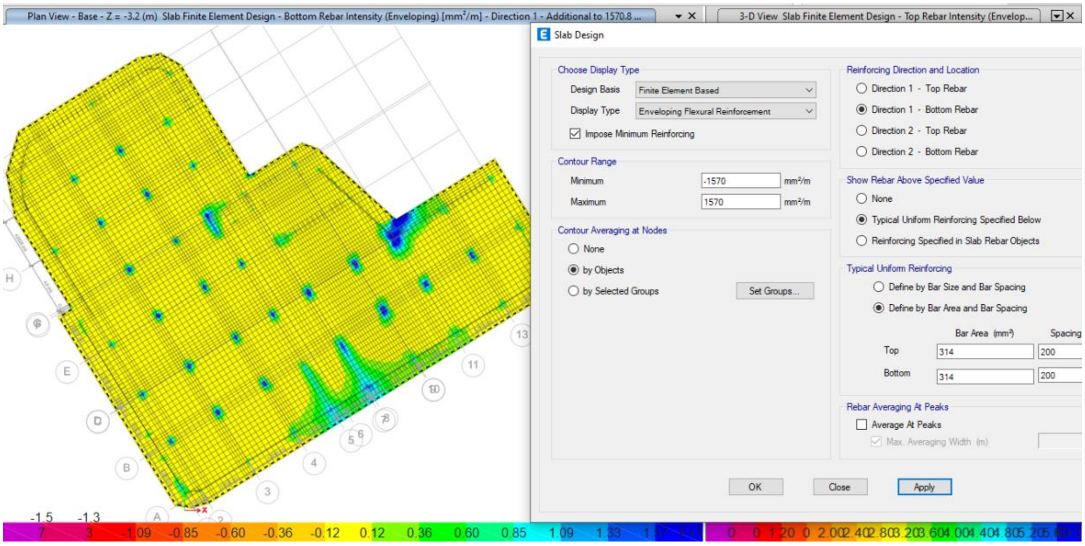


Figure 3.11-3:Bottom steel in direction 1.

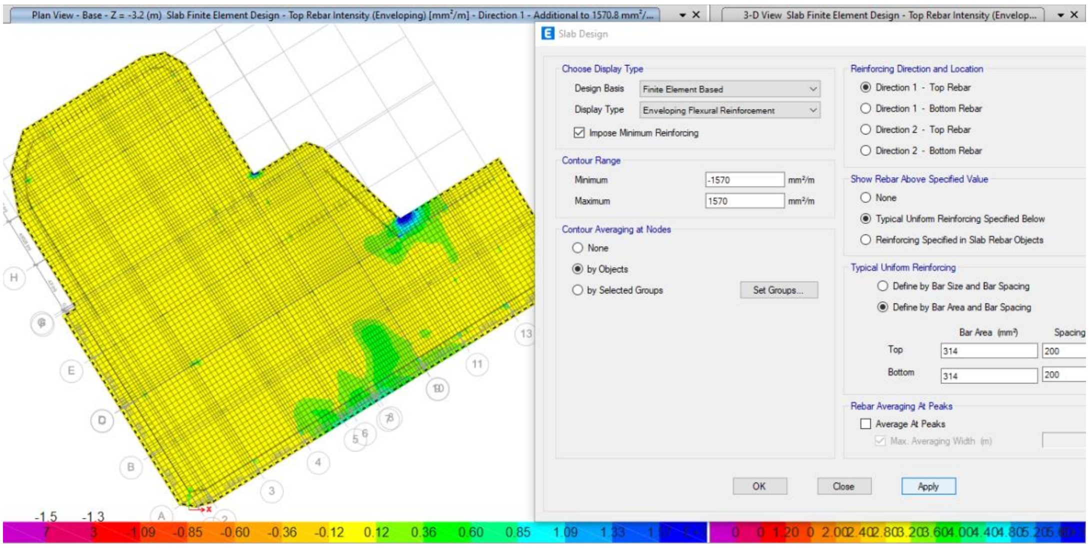


Figure 3.11-4:Top steel in direction 1.

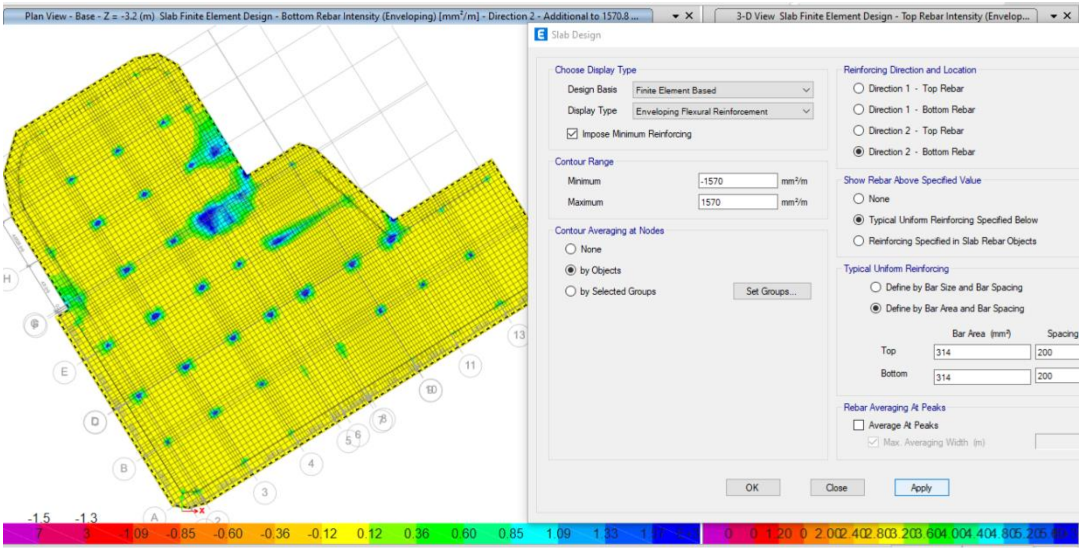


Figure 3.11-5:Top steel in direction 2.

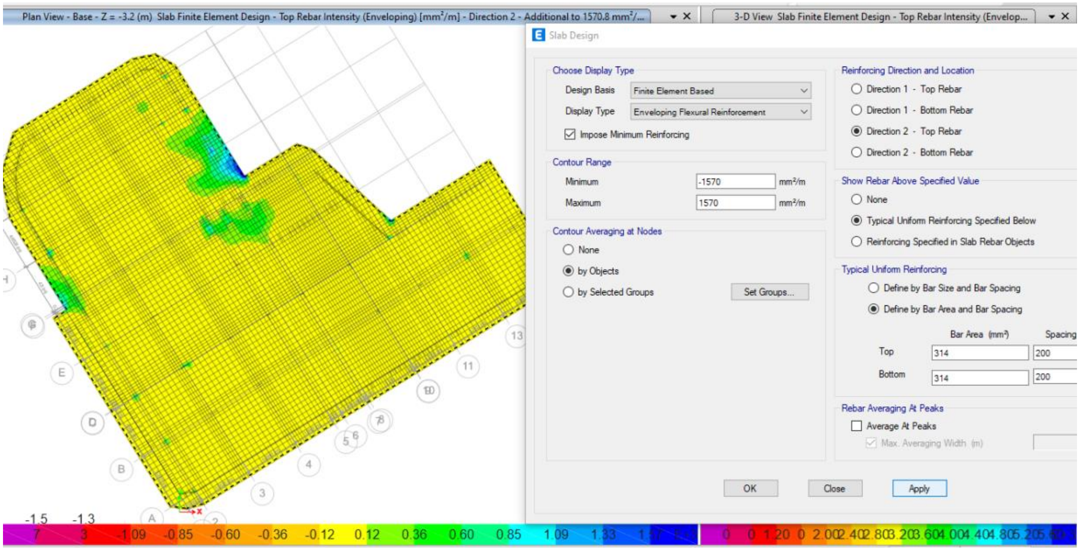


Figure 3.11-6:Bottom steel in direction 2.

- **Note:** Additional top and bottom reinforcement are used around some areas and illustrated in detailed mat reinforcement drawings.

- The development length in tension, L_d is given by:

$$L_d \geq (0.48*F_y*d_b/\sqrt{F'c}) \geq 300mm \rightarrow \text{for } d_b < 20mm$$

$$L_d \geq (0.59*F_y*d_b/\sqrt{F'c}) \geq 300mm \rightarrow \text{for } d_b \geq 20mm$$

❖ For top bars and d ≥ 300mm, increase these values by 30%.

❖ In tension splicing, use 1.3*L_d:

- $Ls_{\text{top bar}}$ (Ø 20) = 1.3*1.3 *(0.59∗420 *20 / √28) = 1600 mm
- $Ls_{\text{bottom bar}}$ (Ø 20) = 1.3*(0.59∗420 *20 / √28) = 1250 mm.

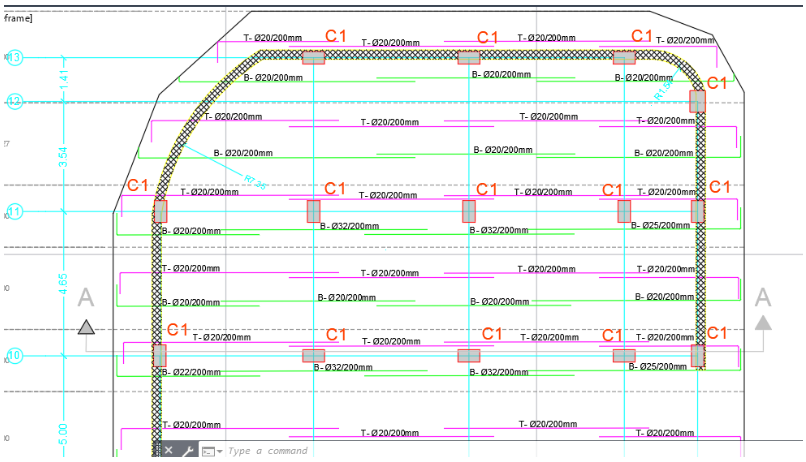


Figure 3.11-7:Footing details in x-direction.

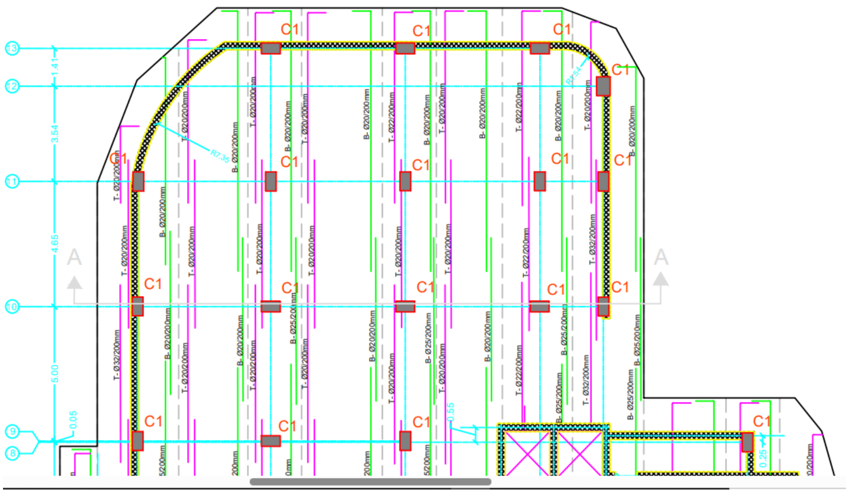
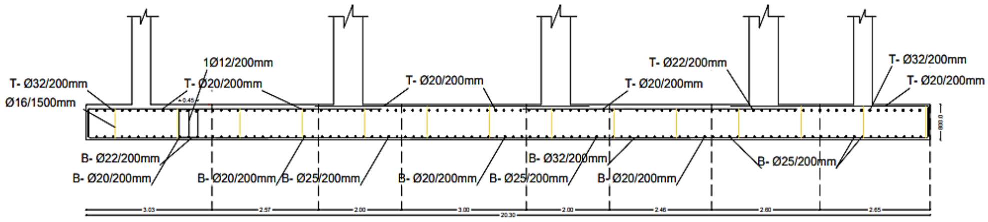


Figure 3.11-8:Footing details in y-direction.



Cross Section A-A
Scale 1:50

Figure 3.11-9:Cross section in mat foundation.

3.12 Design of Stairs :-

In this section, a staircase as shown in Figure 3.12.1 in the 3rd and 4th floors, will be designed and detailed, the staircases were modeled in special models to simplify the process and to make it clearer.

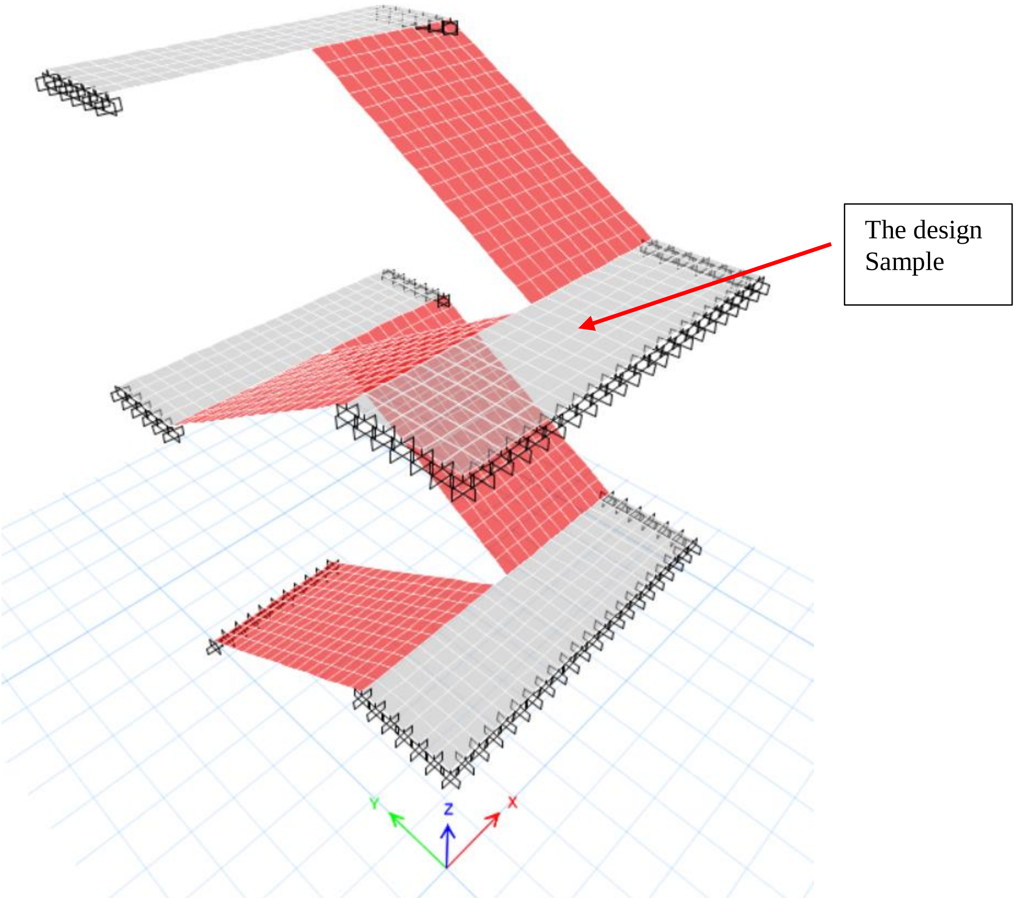


Figure 3.12-1:The staircase selected for the design.

The wall has been represented as fix supports, as shown in figure 3.12.1.

Figure 3.12.2 shows the layout of the staircase.

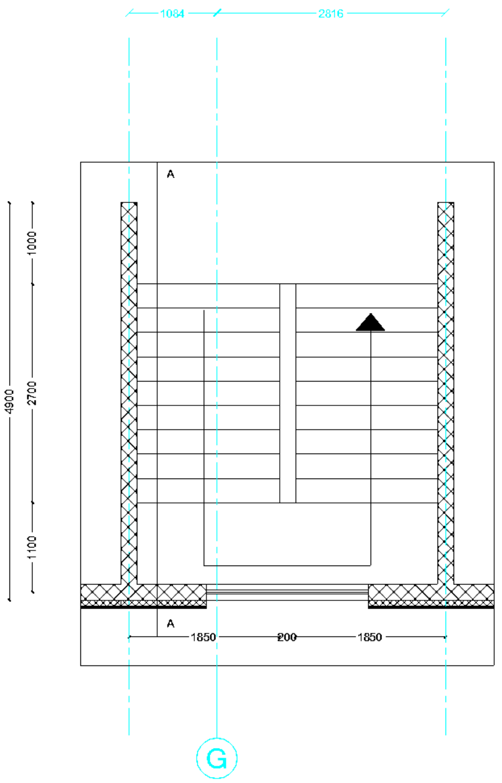


Figure 3.12-2:Staircase layout.

Analysis checks: -

- Compatibility check is shown in figure 3.12.3.

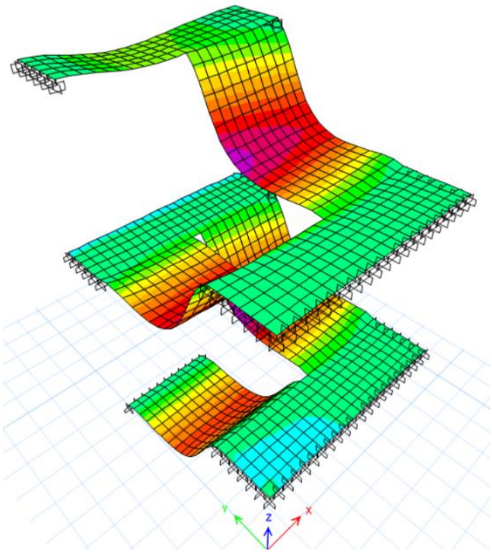


Figure 3.12-3:Compatibility check.

- Equilibrium check is shown in tables 3.12.1 and 3.12.2, which represent ETABS results and manual calculation respectively.

Table 3.12-1:Manual calculation.

Output Case	Case Type	FX kN	FY kN	FZ kN
Dead	LinStatic	0	0	168.9926
Live	LinStatic	0	0	198.0234
SD	LinStatic	0	0	158.4187

Table 3.12-2:Manual calculation.

Part	Area	D.L.	S.D.	L.L.
	m2	kN	kN	kN
Middle Landings	8.58	42.90	34.32	42.90
Story Landings	7.80	39.00	31.20	39.00
Flights	23.24	87.14	92.94	116.18
Totals	**39.62**	**169.04**	**158.46**	**198.08**

- Internal forces check: the loads are too small, so after doing the checks the internal forces were too close.
- Deflection check: the allowable deflection is L/240 = 13.08 mm

  The highest deflection in this staircase is shown in figure 3.12.4, which is 3.341 mm < 13.08 mm. **OK**

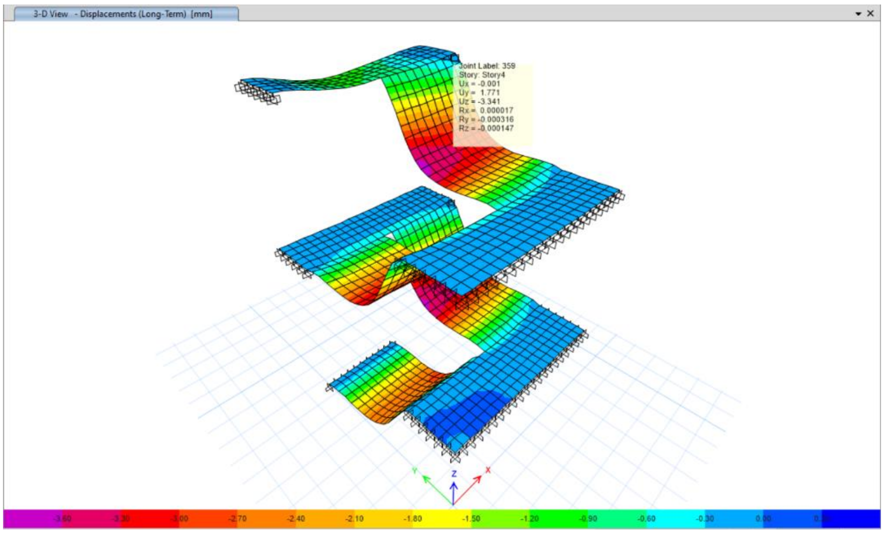


Figure 3.12-4:Staircase deflection.

❖ Design:

Thickness, h = 200 mm.

Effective depth, d = 160 mm.

Flexural:

By using the shrinkage steel (A_s = 0.0018 b h) which is 360 mm^2, which can resist an ultimate moment equal to ($\varnothing M_n$ = 21.34 kN.m) which is larger than the values on figure 3.12.5.

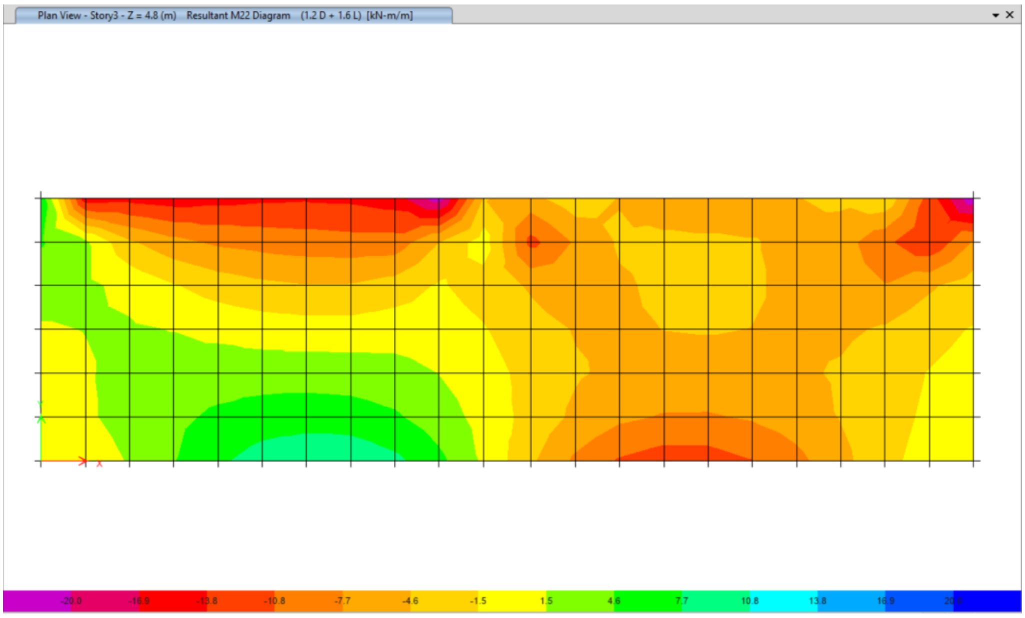


Figure 3.12-5:Ultimate moment values.

Top and bottom Steel: As = 360 mm → use 1Ø12 / 250 mm.

Development Lengths: $L_{dt} = (0.48*420 *12)/\sqrt{28}$ = 457 mm ≈ 500 mm.

Shear:

$\varnothing V_c$ = 54.92 kN (without shear reinforcement)

Figure 3.13.6 shows the shear values.

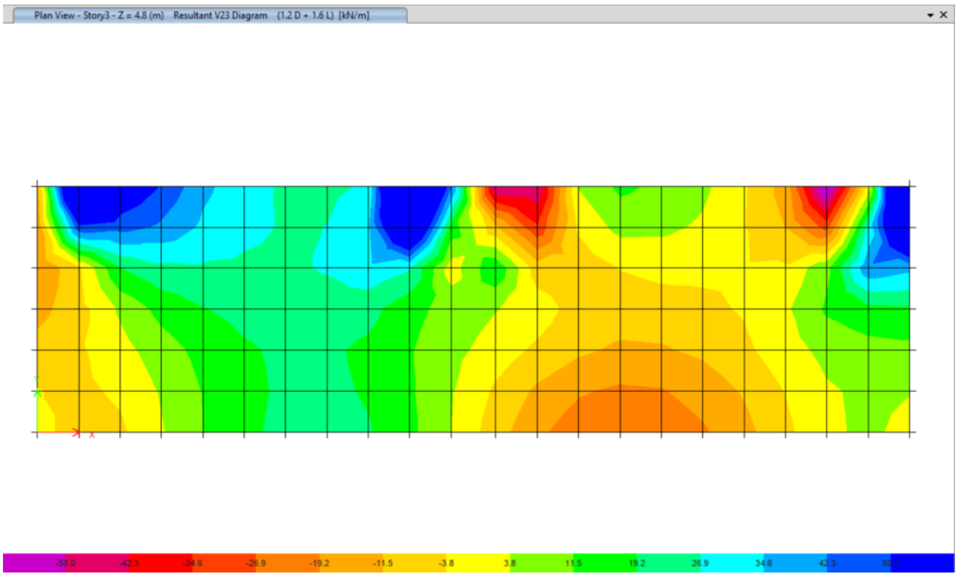


Figure 3.12-6:Shear values.

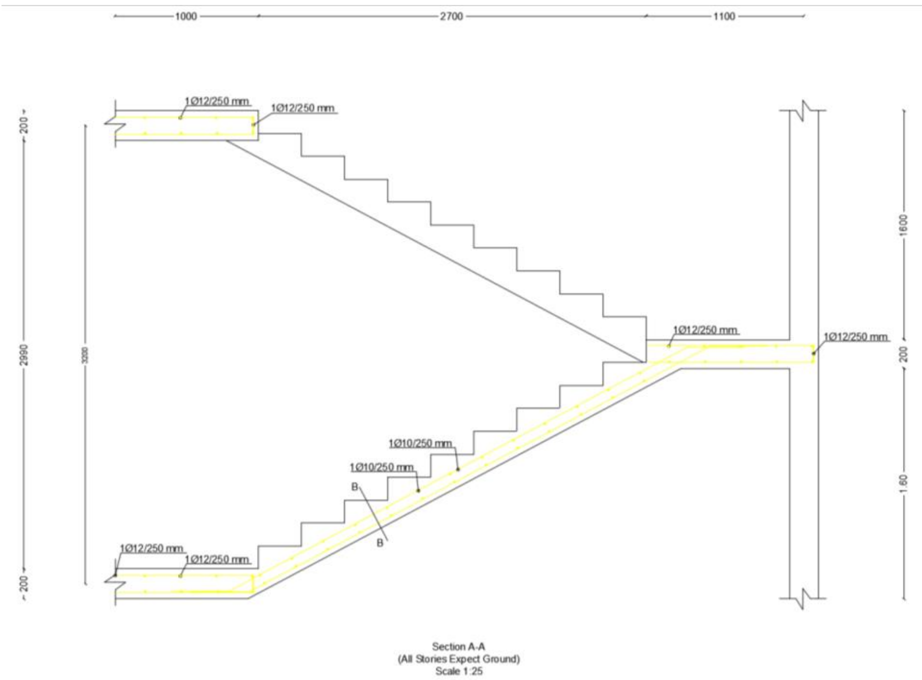


Figure 3.12-7:Staircase detailing.

3.13 Diaphragms and Collectors:

The slab system in a structural building act as a deep beam in the horizontal direction, transmitting lateral loads to the vertically resisting elements of a structure (such as shear walls or frames).

3.12.1 **Design of Diaphragms*:***

In this section, the ground story will be designed and detailed for diaphragm as shown in Figure 3.13-1, A model consisting ground story only can be analyzed with uniformly distributed load (Diaphragm Forces will be discussed later) , then take the maximum shears and moments in the slab

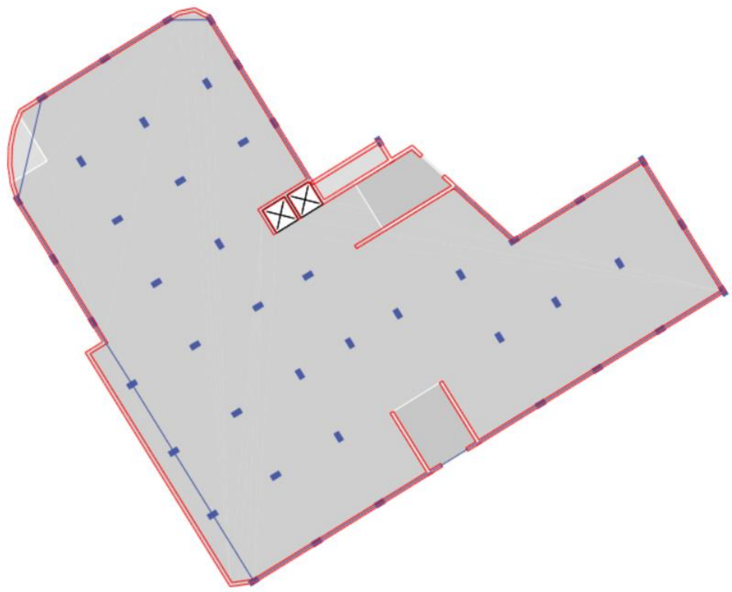


Figure 3.13-1:The ground story selected for the collector's design.

❖ **Diaphragm Design Forces**:

The diaphragm forces can be computed in accordance with ASCE 7-22 Equation (12.10-1) as follows:

$F_{px} = \sum_{i=x}^{n}(f_i / w_i)$ *Wpx (ASCE 7-22 Eq. 12.10-1)

Where:

F_{px} = Diaphragm design force at level x .

F_i = Design force applied to level i .

W_i = Weight tributary to level i .

W_{px} = Weight tributary to the diaphragm at level x.

The force shall not be less than:

$$F_{px} = 0.2 S_{DS} I_e W_{px} \qquad \text{(ASCE 7-22 Eq. 12.10-2)}$$

And the force shall not exceed:

$$F_{px} = 0.4 S_{DS} I_e W_{px} \qquad \text{(ASCE 7-22 Eq. 12.10-3)}$$

The story horizontal forces were computed from the load case Ed; dynamic seismic force in X and Y-direction and the story weights were computed from the load combination: 1.0 DEAD + 1.0 SD + 0.3 Storage live. Table 3.13-1 shows the story horizontal seismic forces in X and Y-direction and the story weights:

Table 3.13-1:The story weight and seismic force.

Story	Weight kN	Seismic Fdx kN	Seismic Fdy kN
Roof	3927.3694	1027.5833	1014.1199
5th	11747.789	3015.7035	2830.308
4th	13817.2485	3380.5943	3006.1504
3rd	13811.6402	2800.9388	2427.0571
2nd	13811.6382	2223.9947	1942.5416
1st	13811.6381	1678.4289	1454.809
Ground	16310.1851	1213.3795	1041.0898
Basement	16640.9598	474.0121	404.275

For ground story:

$= ((F_{ground} + F_{1st} + F_{2nd}+F_{3rd}+F_{4th}+F_{5th}+F_{roof})/ (W_{ground} + W_{1st} + W_{2nd} +W_{3rd}+W_{4th}+W_{5th} +W_{roof})) *W_{ground}$

in X-direction:

- $F_{ground} = \frac{1213.38+1678.429+2224+2801+3380.6+3015.7+1027.6}{16310.18+13811.638+13811.638+13811.638+13811.638+11747.789+3927.37} *16310.18$ =2868.128 KN
- $F_{minimum}= 0.2S_{DS}I_eW_{ground}$ =0.2*0.758*1.25*16310.18=3090.78 KN
- $F_{maximum}= 0.4S_{DS}I_eW_{ground}$ =0.4*0.758*1.25*16310.18=6181.56 KN

$Fx_{ground} < F_{minimum}$ → Take $F_{minimum}$ =3090.78 KN

in Y-direction:

- $F_{ground} = \frac{1041.1+1454.81+1942.54+2427+3006.15+2830.31+1014.12}{16310.18+13811.638+13811.638+13811.638+13811.638+11747.789+3927.37} *16310.18$ =2564.4 KN
- $F_{minimum}= 0.2S_{DS}I_eW_{ground}$ =0.2*0.758*1.25*16310.18=3090.78 KN
- $F_{maximum}= 0.4S_{DS}I_eW_{ground}$ =0.4*0.758*1.25*16310.18=6181.56 KN

$Fy_{ground} < F_{minimum}$ → Take $F_{minimum}$ =3090.78 KN

The force F_p can be applied at the slab as uniformly distributed load in X and Y-direction that equals F_P / A_{slab}.

F_x= 3090.78/928.98= 3.3 KN/m^2

F_y= 3090.78/928.98= 3.3 KN/m^2

The direction local axis 2 will be designed for diaphragm by the force F_Y in the positive direction. The moments and shear forces in the diaphragm can be obtained by doing section cuts in Y-direction along the slab and take the maximum values.

For shear design:

Figure 3.13-2 shows the maximum shear force by applying F_Y in positive direction.

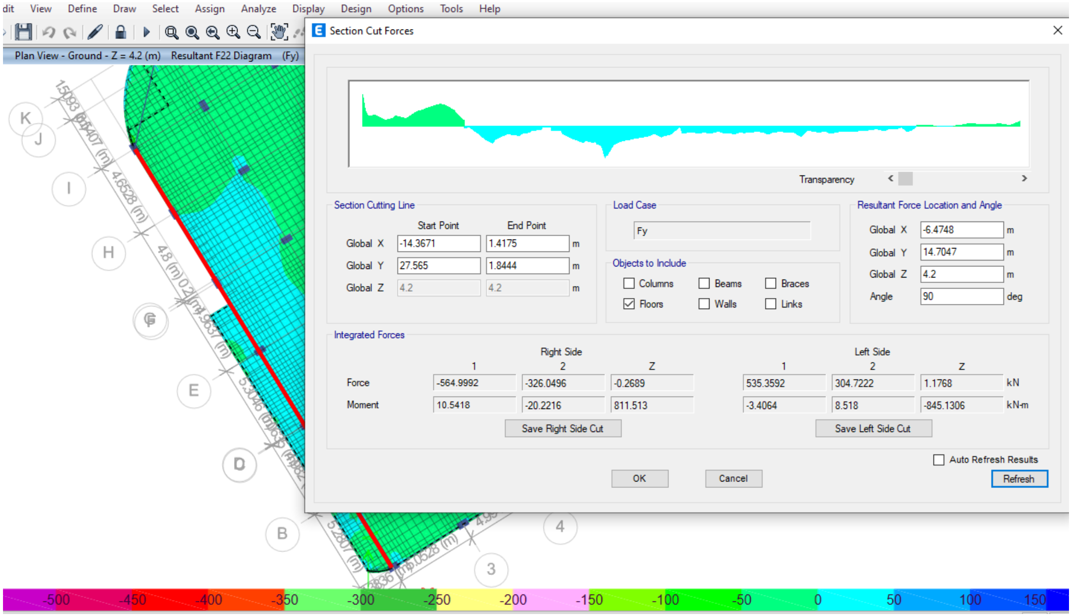


Figure 3.13-2:Maximum shear force in Y-direction.

- The maximum shear force in y-direction (Vu) = 565 kN.
- The shear strength of the diaphragms is given by assuming there is no shear reinforcement:

  $\emptyset V_n = \emptyset\ A_{cv}\,(0.17 * \lambda\sqrt{F'c} + \rho_t\ F_y)$

  $0.6*35400*210*0.17*1*\sqrt{28}\ /1000 = 4012.377KN$

  $\rightarrow \emptyset V_n = 4012.377$ KN >> V_u=565KN → there is no need for shear reinforcement.

For flexural design:

Figure 3.13-2 shows the maximum bending moment by applying F_Y in positive direction.

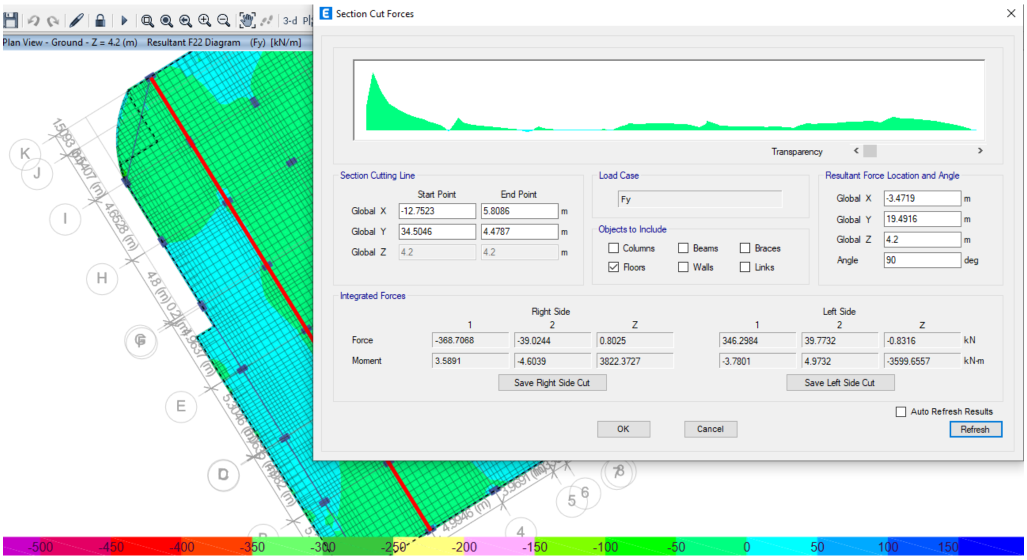


Figure 3.13-3:Maximum bending moment in Y-direction.

- The maximum binding moment in y-direction (Mu) = 3822.37 kN.
  - F= $\frac{Mu}{d} = \frac{3822.37}{35.4-(2*0.5)}$ =111.115 KN
  - $A_s = \frac{F}{\emptyset * Fy} = \frac{111.115*1000}{0.9*420}$ =294 mm^2

This area of steel can be provided in the beams in x-direction. take B25 and B26 as a sample in the X- direction on the ground floor as shown below in figure 3.13-4.

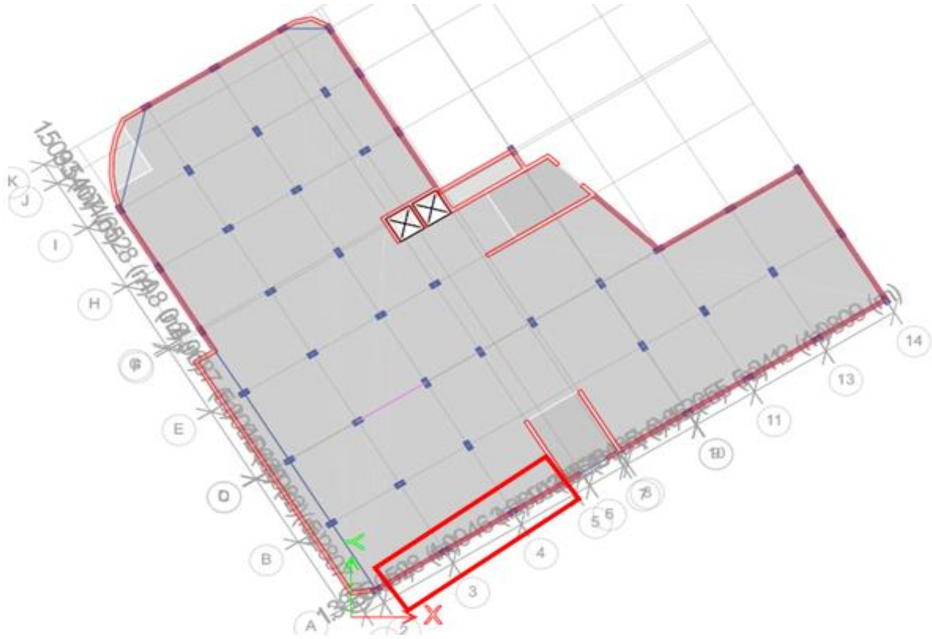


Figure 3.13-4:Beams sample.

Figures 3.13-5 and 3.13-6 show the beams in the X-direction before and after adding the diaphragm steel, respectively.

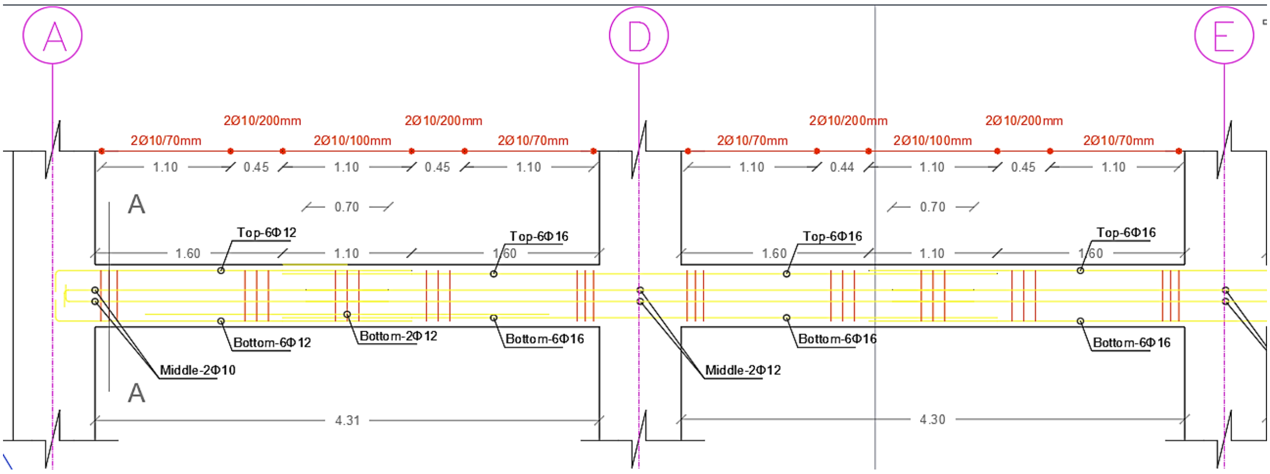


Figure 3.13-5:Beams detailing before adding the diaphragm steel.

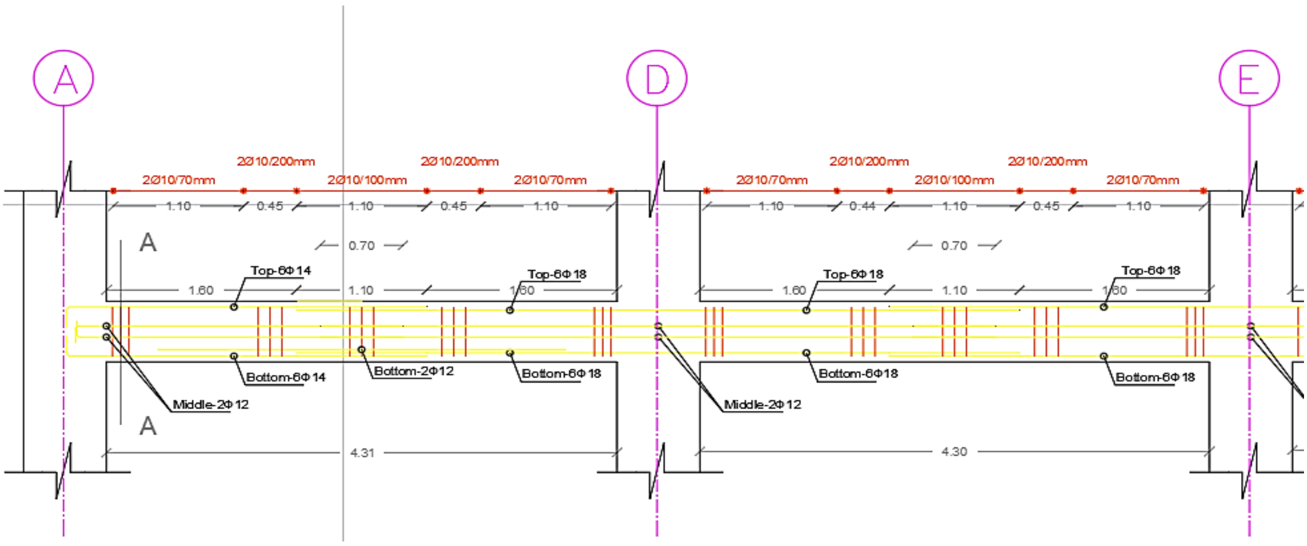


Figure 3.13-6:Beams detailing after adding the diaphragm steel.

3.13.2 Design of Collectors:

In this section, the ground story will be designed and detailed for collectors, the story was modeled in special model to simplify the process and to make it clearer, and to be able to enter the needed loads.

The loads used in the design of collector are obtained by multiplying the overstrength factor (Ω = 2.5) with the forces calculated in the diaphragm section (from the normal equation).

$$F_x = \frac{1213.38+1678.429+2224+2801+3380.6+3015.7+1027.6}{16310.18+13811.638+13811.638+13811.638+13811.638+11747.789+3927.37} * 16310.18 = 2868.13 \text{ kN}$$

F_x per m^2 = (2868.13/928.98) * 2.5 = 7.72 kN/m^2

$$F_y = \frac{1041.1+1454.81+1942.54+2427+3006.15+2830.31+1014.12}{16310.18+13811.638+13811.638+13811.638+13811.638+11747.789+3927.37} * 16310.18 = 2564.4 \text{ kN}$$

F_y per m^2 = (2564.4/928.98) * 2.5 = 6.90 kN/m^2

The direction local axis 2 will be designed for collectors by the force F_Y in the positive direction.

Figure 3.13-7 shows the F_{22} by applying the F_Y in positive direction.

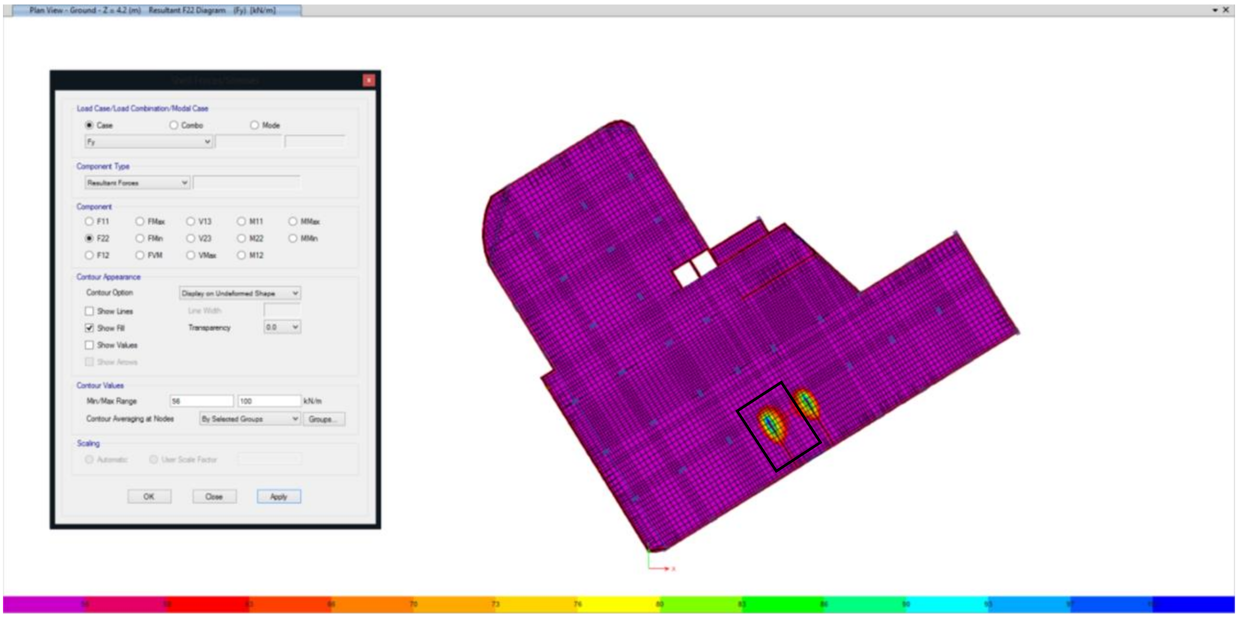


Figure 3.13-7:F22 by +FY.

The highest internal axial force (tension) F_{22} value on this area is 111.64 kN, as shown in figure 3.13-8.

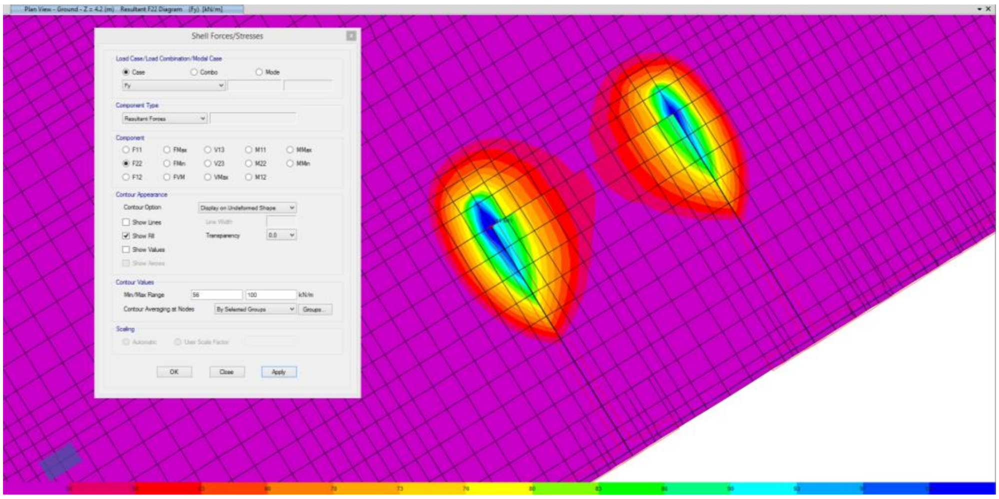


Figure 3.13-8:Highest value of F22.

$A_s = \frac{Force}{\text{Ø}*Fy} = \frac{111.64*1000}{0.9*420}$ = 295.34 mm^2 / m for 5.00 m

Note: in the slab the steel used is 1Ø12 / 250 mm and the needed steel was smaller, then use steel of length 5.00 m, 1Ø14 / 250 mm spread along 3.50 m.

By adding the A_s to the top mesh on the slabs on AutoCAD, the result is as shown in figure 3.13-9

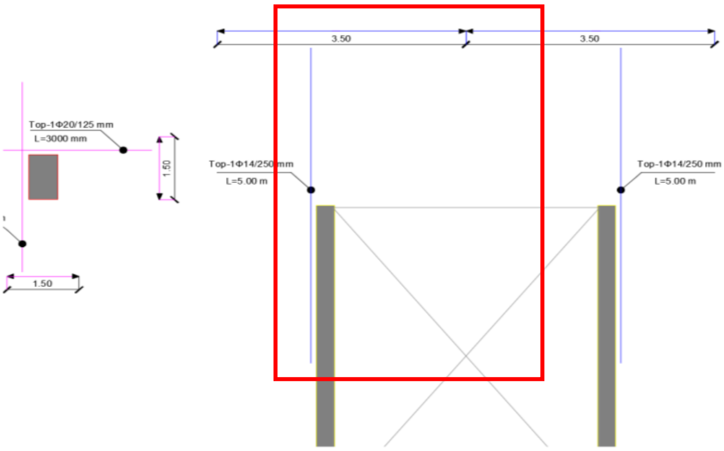


Figure 3.13-9:Collector Drawing.

3.14 Non-structural elements: -

The elements of this project are mainly divided into two types, structural and non-structural elements, the structural elements play an important role in carrying different types of loads of the structure and transfer it to the base, these elements must be designed carefully according to the specified codes, because any failure that occurs in any of them means that there will be a change in the path of loads in the system, and thus cause other structural elements to hold additional loads that may exceed its design limits, these element are beams, columns, walls, slabs and footings, and all of them had been designed in detail in the previous sections, the non-structural elements work as decorative objects which do not help in carrying any load from the structure.

Non-structural elements were considered as additional loads applied to the structural elements during the modeling process.

These elements must be designed to be able to hold its position in the structure when it deforms due to vertical or lateral loads.

Examples on the non-structural elements:

1- Stones
2- Gypsum and wood partitions
3- Aluminum and glass curtains
4- Doors and windows

3.14.1 **Stones: -**

Stones work as an insulating layer and aesthetic view to the building which cover the exposed part of the shear walls, these stones are brittle and may fall off of the structure during an earthquake, and cause damage to the nearby areas, so the connections between them must be designed to make sure they stay in their position during any event.

Generally, the stones are subjected to the same types of loads that affect the structural elements, vertical and lateral loads.

The dimensions of the stone unit that will be used in calculation are 80cm * 25cm * 3cm

- Vertical loads: mainly composed of its own weight and the vertical component of the earthquake.

F_v = volume * γ_{stone} + 0.2 * S_{DS} * volume * γ_{stone}

F_v = (0.8 * 0.25 * 0.03) * 26 + 0.2 * 0.758 * (0.8 * 0.25 * 0.03) * 26

F_v = 0.18 kN

- Lateral loads: due to the horizontal component of the earthquake and it's calculated according to the following equation:

$F_h = 0.4 * S_{DS} * I_p * W_p * \frac{Hf}{R\mu} * \frac{CAR}{Rpo}$

S_{DS}: Spectral acceleration, short period = 0.758

I_p: Component Importance Factor and its equal 1.5 if any of the following condition apply:

1- The component is required to function for life-safety purposes after an earthquake, including fire protection sprinkler systems and egress stairways.
2- The component conveys, supports, or otherwise contains toxic, highly 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.

3- The component is in or supported by a Risk Category IV structure or permanently connected by mechanical or electrical systems to a Risk Category IV structure, and the component is required for the continued operation of a structure designated an Essential Facility, or its failure would impair the continued operation of a structure designated an Essential Facility.

4- The component conveys, supports, or otherwise contains hazardous substances and is attached to a structure or portion thereof classified by the Authority Having Jurisdiction as a hazardous occupancy.

Since the stone does not fall in any of the 4 conditions, I_p will be equal 1

W_p: weight of the stone unit sample = 0.8 * 0.25 * 0.03 * 26 = 0.156 kN

H_f: Factor for force amplification as a function of height in the structure.

$H_f = 1+2.5 * \frac{z}{h}$

Where:

h: height of the building from the base.

z: height of the stone unit sample, assume the sample is the last unit at the top (z = h).

$H_f = 1 + 2.5 * \frac{26.6}{26.6} = 3.5$

R_μ: Structure ductility reduction factor.

$R_\mu = \sqrt{\frac{1.1*R}{Ie*\ \Omega o}} \geq 1.3$

Where:

R: Response modification factor = 6

I_e: Importance Factor = 1.25

Ω_o: Overstrength factor = 2.5

$R_\mu = \sqrt{\frac{1.1*6}{1.25*2.5}} = 1.453$

C_{AR}: Component resonance ductility factor that converts the peak floor or ground acceleration into the peak component acceleration.

R_{po}: Component strength factor.

Both R_μ and R_{po} are determined according to table 3.14-1, since the stone is considered as an architectural component.

Table 3.14-1:Coefficients for Architectural Component.

Architectural Component	C_{AR} Supported at or below grade plane	C_{AR} Supported above grade plane by a structure	R_{po}	Ω_{op}[a]
Interior nonstructural walls and partitions[b]				
Light frame ≤ 9 ft (2.74 m) in height	1	1	1.5	2
Light frame > 9 ft (2.74 m) in height	1.4	1.4	1.5	2
Reinforced masonry	1	1	1.5	2
All other walls and partitions	2.2	2.8	1.5	1.5
Cantilever elements (unbraced or braced to structural frame below its center of mass)				
Parapets and cantilever interior nonstructural walls	1.8	2.2	1.5	1.75
Chimneys where laterally braced or supported by the structural frame	1.8	2.2	1.5	1.75
Cantilever elements (braced to structural frame above its center of mass)				
Parapets	1	1	1.5	2
Chimneys	1	1	1.5	2
Exterior nonstructural walls[b]	1	1	1.5	2
Exterior nonstructural wall elements and connections[b]				
Wall element	1	1	1.5	2
Body of wall panel connections	1	1	1.5	2
Fasteners of the connecting system	2.2	2.8	1.5	1
Veneer				
Limited-deformability elements and attachments	1	1	1.5	2
Low-deformability elements and attachments	1	1	1.5	2

Since the stone sample is located above the grade plane, $C_{AR} = 2.8$ & $R_{po} = 1.5$

So, $F_h = 0.4 * 0.758 * 1 * 0.156 * \frac{3.5}{1.453} * \frac{2.8}{1.5} = 0.213$ kN

$F_{h, max} = 1.6 * S_{DS} * I_p * W_p$

$F_{h, max} = 1.6 * 0.758 * 1 * 0.156 = 0.189$ kN

$F_{h, min} = 0.3 * S_{DS} * I_p * W_p$

$F_{h, min} = 0.3 * 0.758 * 1 * 0.156 = 0.035$ kN

Since $F_h \geq F_{h, max} \rightarrow F_h = F_{h, max} = 0.189$ kN

The stones are attached to a 7 cm layer of concrete cement which act as an adhesive layer between the shear walls and the stone, but in order to make sure that the stone is fixed to the wall, each stone unit will be supported by 4 hooks engraved inside of it and the hooks are attached to the steel mesh inside the concrete cement layer, Figure 3.14-1 illustrate the connection.

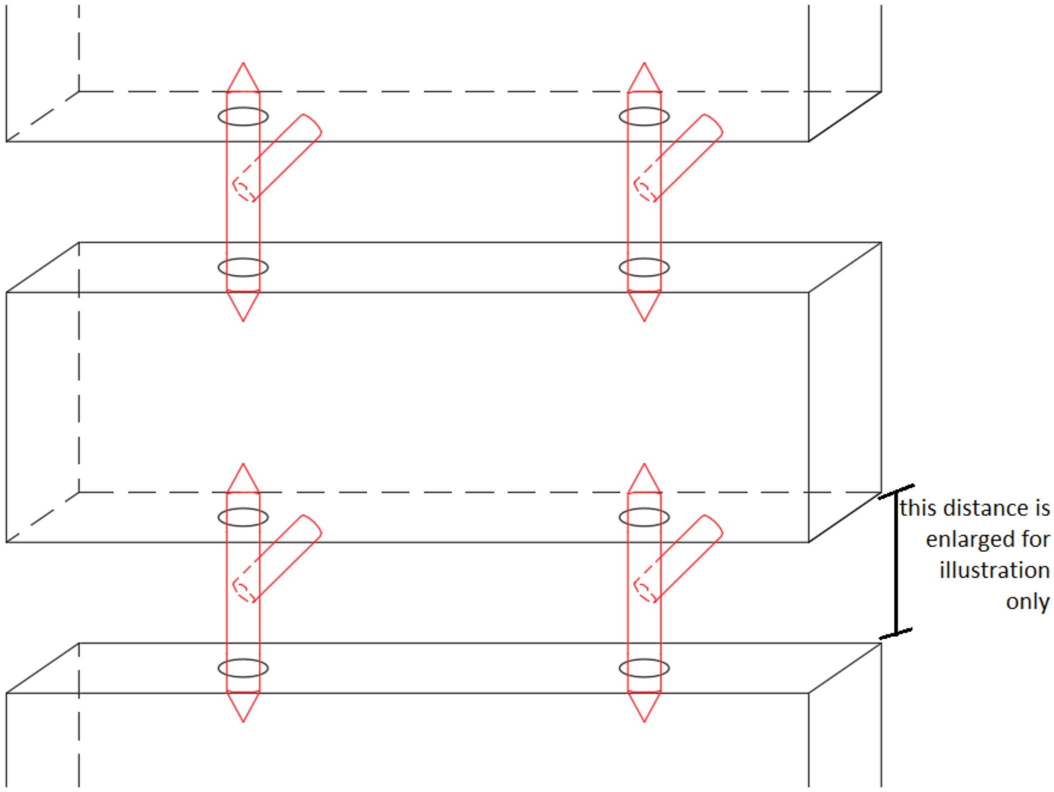


Figure 3.14-1:Connection parts

The vertical load will be distributed between the two bottom legs of each unit, but the lateral load will be distributed between the four legs.

So, every leg will be subjected to a vertical load (compression) equal to 0.18/2 = 0.09 kN.

And a horizontal load (shear) equal to 0.189/4 = 0.047 kN.

3.14.2 **Others: -**

Gypsum and wood partitions have been discussed in the introduction chapter, and they have been used instead of the traditional brick wall because of their light weight, so during the earthquake even if they fall, they won't cause any severe damage to the people inside the building.

Aluminum and glass curtain have been used as an exterior wall to cover the sides of service room at the roof, they were used instead of gypsum and wood because they can resist rainwater, unlike gypsum because it can function better only while its dry.

Doors will be composed of lightweight wood to prevent any injures if they fall during an earthquake.

Windows will be composed of laminated glass which is a type of glass that performs very well in seismic applications because the plastic interlayer holds most of the glass fragments following cracking, which significantly reduces the potential of glass falling to the ground.

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