An-Najah National University

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Graduation Project Report II

Structural Analysis and Design of Indonesian Rehabilitation Hospital in Nazareth city -Palestine.

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اهداء:

إلى راحة يومي ومطلع شمسي من قلبي يا متسعي ورحابة صدري إلى أمي .

إلى من تجرع الكأس فارغا ليسقيني قطرة حب إلى من كانت أنامله لنا لحظة سعادة إلى من حصد الأشواك عن دربي ليمهد لي طريق العلم إلى القلب الكبير أبي.

إلى صاحب القلب الطيب إلى صاحب النفس الأبية إلى صاحب الروح المرحة الأستاذ الفاضل إبراهيم عرمان.

إلى شركائي في الدرب، من كانوا بجواري لنُكمل خطواتنا الدراسية يدًا بيد، وخطوة بخطوة، من كانوا أهلًا للصداقة والعلم إلى أصدقاء العمر والزملاء.

إلى ابنة كنعان إلى الحضن الانتفاضة الأول إلى نابلس

إلى حارة قد أخذت اسمها من الياسمين الأبيض المنثور على جدران منازلها القديمة، إلى درة المدينة وحاملة أسرارها الى حارة الياسمين.

إلى الأحرار خلف القضبان.

إلى الشهداء كل الشهداء.

إلى فلسطين.

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 project presents the structural analysis and design process for the Indonesian Rehabilitation Hospital located in the city of Nazareth, Palestine. It consists of four floors above the ground floor and two underground floors (basements). The floors vary in use, including parking and storage rooms. Each basement floor has an area of 1853.65 m². The external walls and some of the inner walls are composed of reinforced concrete, while partition walls are made of blocks.

This project contains a sequence of steps in analysis and design taking into consideration many loads that will be applied on the structure such as gravity load (superimposed, live and snow loads), lateral loads (soil pressure, surcharge load and seismic loads), for seismic load response spectrum was used for analyzing the performance of the structure in earthquakes and equivalent static was used to comparer his result with response spectrum results and take the largest among them; to be more conservative. The structural system was chosen to be building frame systems (special reinforced concrete shear walls)for basement floors and in Y-direction all floors above ground floor, and dual system with special moment frame capable of resisting at least 25% of prescribed seismic forces in X- direction all floors above ground floor, soil type was classified as type C.

The methodology in this graduation project is to fully understand the architectural plans, to locate or to redistribute - if it is needed - the columns. Moreover, to know and assign the correct static loads according to the architectural plans and floors usages. Furthermore, to suggest different structural systems for the slabs from which to choose the best and the more convenient one, two-way solid slab with beams system was chosen. In addition, to have a preliminary dimension of the elements of the structure. After that, to apply all the data to the programs mainly on ETABS and to validate the outputs of the model through various checks such as compatibility, equilibrium, stress-strain and long- term deflection, taking into consideration the deference percentage not to exceed 10%, structural checks were also done such as horizontal and vertical irregularities, P-Delta and drift check. The objective of these checks is to make sure the serviceability, stability and strength of the structure.

The references in this project were several codes which are American Concrete Institute ACI 318-19, American Society of Civil Engineers ASCE 7-22 and American Society for Testing and Materials ASTM. This structure was represented on the Extended Three-dimensional Analysis and design of Building Systems ETABS 20 software.

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CHAPTER 1: INTRODUCTION

1.1General:

Structural analysis and design are fields of engineering that focus on the structural behavior of buildings and other structures. It involves analyzing and designing structures to ensure that they can withstand the loads and forces that they will be subjected to, such as wind, earthquakes, and other environmental factors. A structural analysis and design graduation project typically involves the design of a structure for a specific site and purpose. This may involve analysing the site conditions, determining the loads that the structure will be subjected to, and using this information to design the structure using appropriate materials and construction methods. The project may also involve creating detailed drawings and specifications for the structure. The goal of a structural analysis and design graduation project is to create a structure that is safe, functional, and aesthetically pleasing. It requires a strong understanding of engineering principles and the ability to apply these principles in a practical manner. It also requires the ability to work effectively in a team and to communicate technical information to a variety of audiences.

1.2 Project description:

Project name is " Analysis and Design of Indonesian Rehabilitation Hospital ". It is located in Nazareth – Palestine, and the coordinates of the project according to the global coordinate system WGS are 32°42′08″N, 35°17′52″E. Location elevation (site level) is +347m AMSL (above mean sea level).

The building consists of six floors divided into two parts with construction joint between them. Two floors are under the ground (B_1 & B_2) as parking usage. The other floors are used for hospital usage. Exterior retaining shear walls are being used for basement one, basement two, and the right side of the building, while interior walls are partitions. However, for the left part of the building exterior walls are curtain. The architectural plans, sections and facades are attached in Appendix A.

Table 1.1 shows the areas and heights of each floor respectively.

Table1.1 Floor areas and heights.

Floor Number	High of each floor (m)	Area of each floor (m^2)
B2	4	1853.65
B1	4	1853.65
GF	4	1387
F1	4	1308.96
F2	4	1230.05
Roof	4	1137.11

1.3 Analysis and design principles:

The analysis and design principles are as follows:

- For external walls, part of them are stone shear walls, and the other are curtain walls. They will be added as dead line load kN/m on the external beams.
- All base supports are assumed fixed in the 3D model of the structure.
- All slabs are two-way solid slabs so the reinforcement in two directions.
- Tie beams will be on ground level used as the same dimensions of this structural framing beams if needed.
- Partitions are not going to be considered as structural elements, instead, they will be considered as part of the superimposed dead load.
- Any inclined slabs (Stairs, ramps…etc.) will be modeled as a horizontal slab with the modifiers changing as needed.

1.4 Codes and standards:

This project is based on the following codes to obtain loads, load combinations, materials specifications and methods of design and analysis:

• ASCE 7-22 (American Society of Civil Engineers): for loads computations.

• IBC 2018 (International Building Code): for loads computations.

• ACI 318-19 (American Concrete Institute): for reinforced concrete design.

• ASTM 2016 (American Society for Testing and Materials): for materials specifications.

1.5 Materials:

According to ASTM 2016 code:

❖ The compressive strength (fc') of concrete should be taken from the cylindrical test, that means the load required to break a cylinder with dimensions (300mm height, 150mm diameter) that has been cured for 28 days.

❖ Tensile strength of concrete (f_t) is the ability of concrete to resist breaking or cracking under tension.

❖ Modulus of rupture of concrete (f_r) is defined as an ultimate strength pertaining to the failure of beams by flexure equal to the bending moment at rupture divided by the section modulus of the beam.

❖ Modulus of elasticity (E_c) is defined as a quantity that measures the element's resistance to being deformed elastically when a stress is applied to it.

❖ Poisson's ratio is defined as the ratio of the change in the width per unit width of a material, to the change in its length per unit length, as a result of strain.

❖ The unit weight of material (γ) is the weight per unit volume of the material and its unit is KN/m^3.

❖ The yield strength of steel (f_y) is defined as the maximum stress that the steel can withstand when it is deformed within its elastic limit.

❖ The ultimate strength of steel (f_u) is defined as the maximum stress that the steel can withstand before its failure. The materials used for structural elements are concrete and steel as follows:

❖ The reinforced concrete used for slab, beams, columns and structural walls has a compressive strength (fc')=28 MPa. In **Table 1.2** the other properties of concrete are clarified.

Table 1.2 Concrete and steel properties.

Unit weight, ɣ	25 (kN/m^3)
Compressive strength, f_c	28 Mpa (B=350)
Linear modulus of Elastisity,E_c	$4700\sqrt{fc} = 24870$ Mpa
Maximum strain,E_t	0.003
Specified compressive strength, f_{ct}	$0.33\ ɣ\ \sqrt{fc}$
Poisons ratio	0.2

The reinforcing steel is the steel bars used for resisting loads transferred by concrete structural elements. The type of steel used in all the elements of the structure is grade 60 with ultimate strength (f_u)= 620 MPa, yield strength (f_y) = 420 MPa and E_c= 200 GPa.

The materials used for non-structural elements of the project are shown with their unit weights in the **Table 1.3** .

Table1. 3 Material unit weight kN/m3.

Material	**Unit weight ɣ(KN/m^3)**
Masonry stone	26.5
Cement mortar &Plaster	23
Partition blocks	15
Filing material aggregate	18
Glass	25
Marble Tiles	27
Aluminium	26
False celling	0.2 (KN/m^2)
Sand	23
Reinforced concrete	25

1.6 Loads:

This section includes all loads expected to be exposed to the structure during its life. In this section, all load definitions and values are determined based on codes in section **1.4**.

1.6.1 Dead Load:

It is defined as the own weight of the structural elements (Slabs, walls, columns,beams, foundation). It will be calculated automatically using the ETABS software.

1.6.2 Live Load:

It is defined as the load produced by the use, occupancy, and movable objects of the building. For this project the loads are from ASCE7-22 as shown in **Table 1.4.**

Table1.4 Live Load values.

Occupancy	Live Load (KN/m^2)
Parking	3
Store 1st Floor	5
Store under 1st Floor	6
Store above 1st Floor	4
W.C	2
Audiology room	6
X-Ray room	6
Stairs	5
Gym	5
Laboratories	3
Balconies	5
Patient room	2
Office use	2.5
Kitchen	5
Pharmacy	6
Clinic room	5
Corridor at GF	5
Corridor above 1st Floor	4
Roof live Load(L_r)	1
Storage	12

1.6.3 Superimposed dead load:

It is the weight of non-structural elements (tiles, partitions, mortar, plastering, etc.…) that is distributed on the slabs, walls, ramps and staircases**.**

1. Superimposed dead load on the slabs:

The method used in calculating the superimposed dead loads on the slabs is as follows:

- ❖ First, each story contains two or more representative areas, each of which the number of areas taken depends on the different uses of the floor (residential, parking, shops…etc). It also relies on grouping non-structural elements in one place and separating them in another.
- ❖ Second, superimposed dead load (SDL) (including internal weights,cut-outs, plaster, fillers...etc) were calculated for each area.
- ❖ Third, if the areas on each floor or even on different floors have numbers close to SDL and they have the same use, then they are combined together on the largest value of SDL.
- ❖ Fourth, the SDL values for all floors shown in the table are summarized.

On the following pages, there are some computational samples for different areas, each with the area in different uses (eg: clinics, parking area...etc), as shown in **Figure 1.1**.

<u>Zone I :</u>

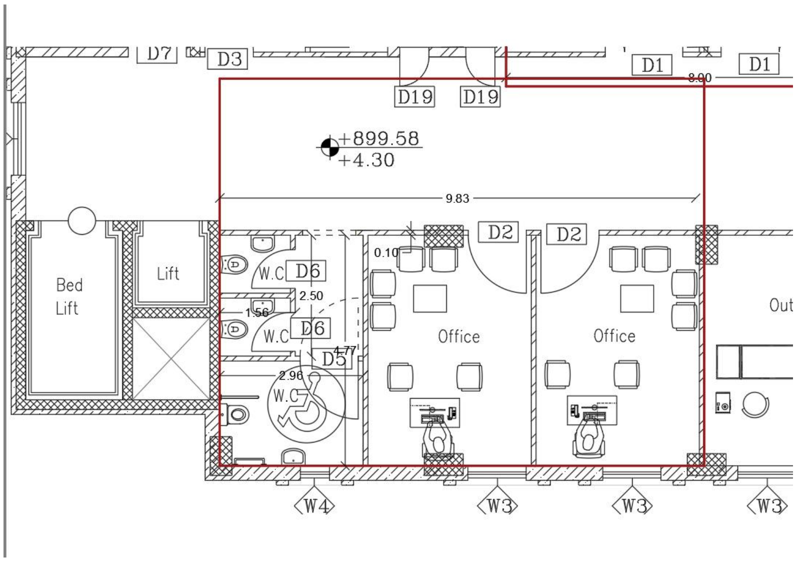


Figure1. 1 Sample 1st floor zone I for SID calculation

Sample calculation for 1st foor zone I:

Partition length = 31.16 m

Height of story = 4 m

Hight of door = 2.25m

Sum. of door length= 6.3m

Area of partitions = Total area – Door area

$= (31.16\times 4) - (6.3\times 2.25)$

$= 110.465\ m^2$

The thickness of the partition wall = 0.1 m

Super imposed load (SDL) from block partitions = $\dfrac{Volume \times (\gamma) Block}{Area}$

$= \dfrac{110.46 \times 0.1 \times 15}{80}$

$= 2.07\ KN/m^2$

Super imposed load (SDL) from plastering block partitions:

The thickness of the plastering =0.04m

$$SDL=\frac{110.46\times0.04\times23}{80}$$

$$= 1.27\ kN/m^2$$

Superimposed load (SDL) from Marble tiles:

Tile thickness = 3 cm

Concrete thickness =2.5 cm

Sand thickness = 10 cm

SDL= Tile weight +Concrete weight + Sand weight + False celling weight

$$SDL = 0.03\times27 + 0.025\times23 + 0.1\times18 + 0.2$$

$$= 3.385\ KN/m^2$$

$$\text{Total SDL} = 2.07+1.27+3.385$$

$$= 6.725\ KN/m^2$$

Table 1.5 show the average superimposed load value for each floor.

Table1. 5 Average super-imposed load values

Floor Number	Super-imposed load (KN/m^2) on the slab
Basement 2	4.1
Basement 1	4.5
Ground Floor	7
Floor 1	7.5
Floor 2	7.5
Roof	7

Super imposed load from wall weight:

1. External wall consists of glass:

SDL glass = γ_{glass} × thickness × story height

= 25 × 0.05 ×4.3

= 5.4 KN/m^2

2. External wall consists of masonry stones and reinforce concrete :

SDL masonry wall = (0.03×26.5) + (0.07× 25)

= 2.5 KN/m^2

1.6.4 Soil load:

Lateral earth pressure is the pressure that soil exerts in the horizontal direction. The lateral earth pressure is important because it affects the consolidation behaviour and strength of the soil and because it is considered in the design of geotechnical engineering structures such as retaining walls, basements, tunnels, deep foundations and braced excavations.

Soil pressure = $\gamma \times K_o \times h$ KN/m^2

Surcharge pressure = WK KN/m^2

Seismic load (Δp) = $0.4\ K_h\ \gamma_t\ H_{rw}$ KN/m^2 equation (8-30) form ASCE 41-13 .

Where:

K_o: coefficient of earth pressure at rest.

H_{rw}: height of the retaining wall = 8 m.

γ: Unit weight of soil.

It is assumed that γ= 22 KN/m^3.

Surcharge load equals to = 5 KN/m^2.

Δp: additional earth pressure caused by seismic shaking, which is assumed to be a uniform pressure.

K_h: horizontal seismic coefficient in the soil, which may be assumed equal to S_{XS} / 2.5.

γ_t: total unit weight of soil.

S_{XS}: spectral response acceleration parameter, $S_{xs} = f_a\ S_S$

fa: site factor.

Ss: mapped MCER (Maximum Considered Earthquake),5 percent damped, spectral response acceleration parameter at short period of 0.2 second.

(fa and Ss values from section1.6)

$K_h = S_{xs}/2.5$

$S_{xs} = f_a\, S_S$

$=1.12 \times 0.7$

$=0.784$

$Kh = 0.784/2.5 = 0.314$

$K_o = 1 - \sin Ø$

$= 0.15$ (as approximation value)

Calculation for soil load and load distribution are shown in **Table 1.6** and **Figure 1.2** respectively

Table1. 6 Lateral soil load value.

Soil pressure (KN/m^2)	**Surcharge pressure** (KN/m^2)	**Seismic load** (KN/m^2)	**Total lateral load** (KN/m^2)
$22\times0.15\times8$ $=26.4$	5×0.15 $= 0.75$	$0.4\times0.314\times22\times8$ $=22$	49.15

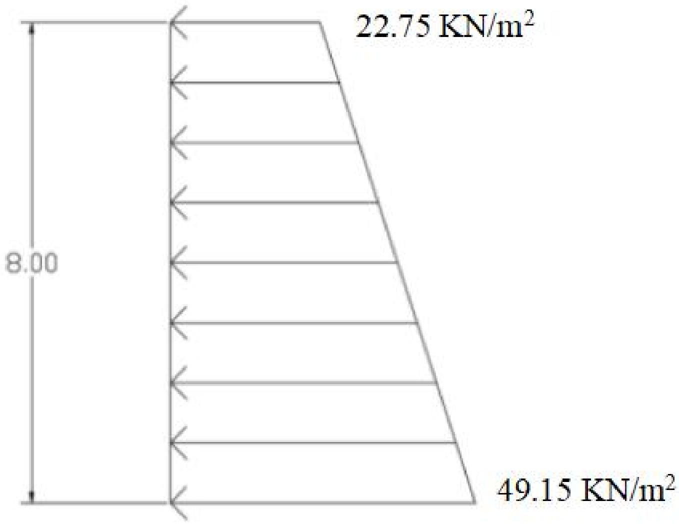


Figure1. 2 Lateral loads for zone 1.

1.6.5 Water load:

Water Hight (Hw) = 4m

Water unit weight (W)= 10 KN/m^2

$q_w = W\times H_w$

$=10\times4$

$=40$ KN/m^2

Force, P= $q_w\times H_w$

$= 0.5\times40\times4$

$=80$ KN

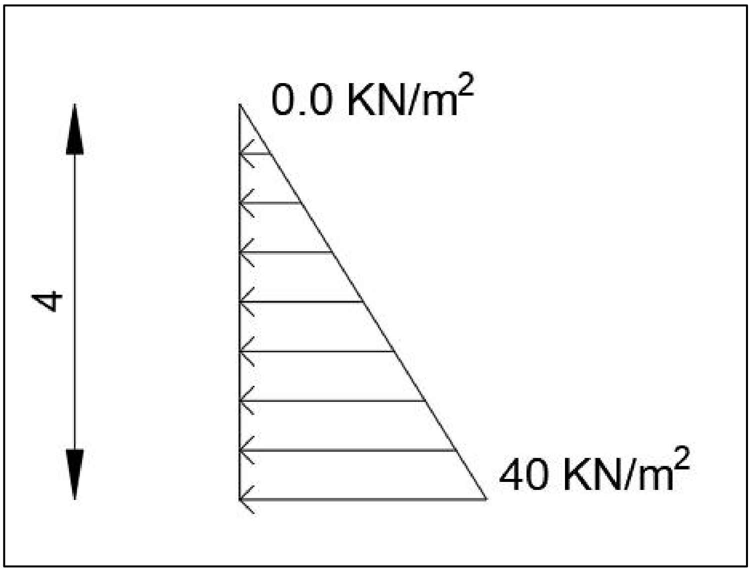


Figure1. 3 Water load

1.6.6 Snow Load

According to ASCE 7-22 the flat roof snow load, pf, is calculated using the following formula(ASCE 7-22 equation (7.3-1))

$$\mathrm{Pf} = 0.7 \times \mathrm{Ce} \times \mathrm{Ct} \times \mathrm{Pg}$$

Where:

P_f: Snow load on flat roofs kN/m^2.

C_e: Exposure factor ; ASCE7-22 table 7.3-1 .

C_t: Thermal factor ;ASCE7-22 table 7.3-2 .

P_g: Ground snow load kN/m^2.

P_m: Minimum snow load kN/m^2.

For C_e value, it depends on two major things; Exposure of roofs and surface roughness category.

Because the building in flat, open country where obstructions are < 9m tall, the surface roughness category is **C** with Partially Exposed.

According to ASCE 7-22 Table 7.3-1, C_e value is 1.0, as shown in **Table 1.7**.

Table1. 7 Exposure Factor, Ce

Table 7.3-1. 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

Table1. 8 Risk Category Of Building and Other Structures For Flood, Wind, Snow, Earthquake, and Lee Loads according to ASCE 7-22.

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.	III
Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure.	
Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where their quantity exceeds a threshold quantity established by the authority having jurisdiction and is sufficient to pose a threat to the public if released.	
Buildings and other structures designated as essential facilities.	IV
Buildings and other structures, the failure of which could pose a substantial hazard to the community.	
Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity exceeds a threshold quantity established by the authority having jurisdiction to be dangerous to the public if released and is sufficient to pose a threat to the public if released.[a]	
Buildings and other structures required to maintain the functionality of other Risk Category IV structures.	

[a]Buildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the substances is commensurate with the risk associated with that Risk Category.

Compute Min snow load on flat roof Pm using Table 7.3-4 according to ASCE 7-22, as shown in **Table 1.9**.

Table1. 9 Min. Snow Load for Low_Slope Roof (ASCE 7_22)

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

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

P_m = 1.92 KN/m^2

$P_m = P_g$

According to Table 7.3-2 in ASCE 7-22, as shown in **Table 1.10,** C_t = 1.2 .

Table1. 10 Thermal Factor,Ct(ASCE 7_22)

Table 7.3-2. 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.

$P_f = 0.7 \times 1.2 \times 1 \times 1.92$

$= 1.6$ KN/m^2

P_g : based on the Jordanian loads code table 3.5

Table1. 11 Ground snow load kN/m^2 (Pg) – Jordanian Code.

Structure elevation -h (m)	Snow load-S_o (KN/m^2)
h <250	0
250 <h < 500	(h-250)/800
h > 500	(h-400)/320

This building is +347m AMSL, so using the Jordanian Code

$$Pg = \frac{347 - 250}{800} = 0.12$$

As the value is very small, to be conservative use $P_m = P_g$, then for risk category IV which is P_g=1.92 KN/m^2.

Snow load will be the Max (Pf, Pm) = Max(1.6, 1.92) .

Snow load = 1.92 KN/m^2

But in load combinations we will select the higher of live roof load and snow load to add it in combinations, according to ASCE 7-22, Table 4.3-1 live load, roof live load Lr = 0.96kN/m^{2,} as shown in **Table 1.12**.

Table1. 12 Live Load Table – ASCE 7-22

Roofs					
Ordinary flat, pitched, and curved roofs	20 (0.96)	Yes (4.8.2)	—		4.8.1
Roof areas used for assembly purposes	100 (4.79)	No (4.7.5)			
Roof areas used for occupancies other than assembly	Same as occupancy served	Yes (4.8.3)	—		
Vegetative and landscaped roofs					
Roof areas not intended for occupancy	20 (0.96)	Yes (4.8.2)	—		
Roof areas used for assembly purposes	100 (4.79)	No (4.7.5)	—		
Roof areas used for occupancies other than assembly	Same as occupancy served	Yes (4.8.3)	—		
Awnings and canopies					
Fabric construction supported by a skeleton structure	5 (0.24)	No (4.8.2)	—		
Screen enclosure support frame	5 (0.24) based on the tributary area of the roof supported by the frame member	No (4.8.2)	—	200 (0.89)	
All other construction	20 (0.96)	Yes (4.8.2)	—		4.8.1

Roof live load (L_r) is < Snow load, therefore, we will use snow load in load combinations.

1.6.7 Seismic loads:

Buildings shall be designed to have a low probability of collapse under earthquake effects. Seismic analysis was performed following ASCE 7-22 the seismic load composes of horizontal and vertical effects that can be determined as in the following equations:

$E = E_h + E_v$

Where:

E: Seismic Load.

E_h : Horizontal seismic forces effect.

E_v: Vertical seismic forces effect.

- **Risk category:**

This structure had a risk category IV according to (Table 1.5-1 ASCE7-22) because the failure which could pose a substantial risk to human life.

Risk Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, Ice Loads from ASCE7-22 as shown in **Table 1.12**.

Table1. 13 Risk Category of Buildings and Other Structures for Wind, Tornado, Snow, Earthquake, and Ice Loads.

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

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

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

- **Importance factor (I_e):**

A factor that gives an indication of the risk to human life, health, and welfare concerning damage to property, loss of use or functionality. And it depends on risk category as in (ASCE 7-22 Table 1.5-2).

Importance Factor **(I_e) = 1.5**.

Importance Factors by Risk Category of Buildings and Other Structures for Earthquake Loads as shown in **Table 1.14**.

Table1. 14 Important Factor by Risk Category

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

➢ **Site class:**

The site class according to soil investigation and soil tests was class “C”, because the soil in the project zone was improved.

➢ **Acceleration parameters:**

To start the seismic analysis according to ASCE 7-22, the peak accelerations were obtained from Palestine standard, Amendment No.5 with values of **S_s= 0.7g and S_1=0.14g** as shown in **Figure 1.4**:

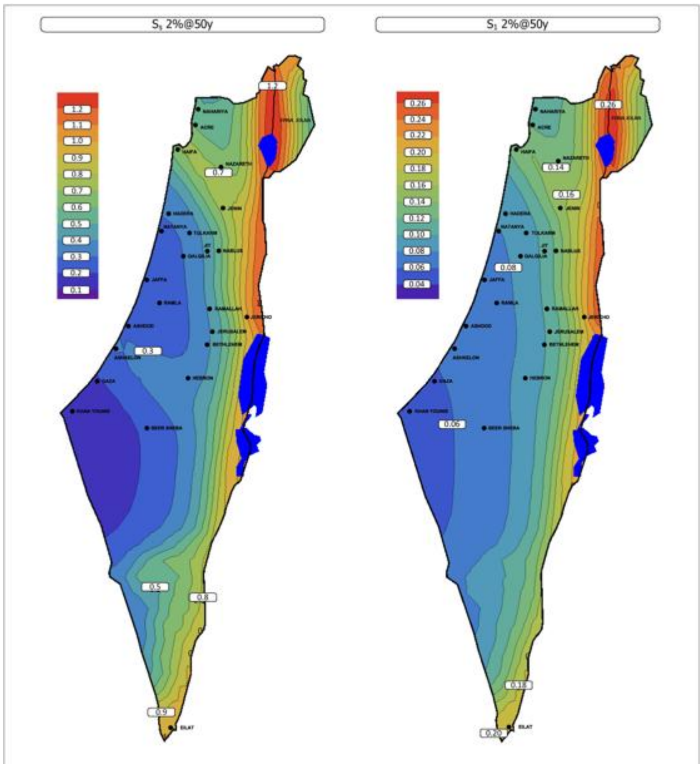


Figure1. 4 Palestinian Maps for S_s and S1 2% @ 50 years

where:

S_s: the mapped spectral response acceleration parameter at short period.

S_1: the mapped spectral response acceleration parameter at a period of 1 sec.

S_{MS}: the maximum considered earthquake spectral response acceleration at short period with 5% damping.

S_{M1}: the maximum considered earthquake spectral response acceleration at 1sec period with 5% damping.

S_{DS}: the design earthquake spectral response acceleration at short period with 5% damping.

S_{D1}: the design earthquake spectral response acceleration at 1 sec. period with 5% damping.

$S_{MS} = F_a S_S$

$S_{M1} = F_v S_1$

Where Fa and Fv are factors take into account the soil effect corresponding to acceleration at short period and 1 sec. period respectively which were determined according (**Tables 1.15** and **1.16** respectively).

Table1. 15 Fa Factors_ASCI 7_16

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \ge 1.25$
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.

Table1. 16 Fv Factors_ASCI 7_16

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
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.

$S_{DS}= S_{MS} \times 1.0$

$S_{D1}= S_{M1}\times 1.0$

According to the site class in this project:

$F_a = 1.12$

$F_v = 1.66$

$S_{MS}= 1.12 \times 0.7 = 0.784$

$S_{M1}= 1.66 \times 0.1\ 4= 0.2324$

$S_{DS} = 1 \times 0.784$

$= 0.784$

$S_{D1} = 1 \times 0.2324$

$= 0.2324$

➢ **Long-Period Transition Period (TL):**

It is the transition period separating the constant velocity and constant displacement segments of the design response spectrum. Which is taken from maps of Palestine standard based on 2% probability of exceedance in 50 years.

$T_L = 8$ sec.

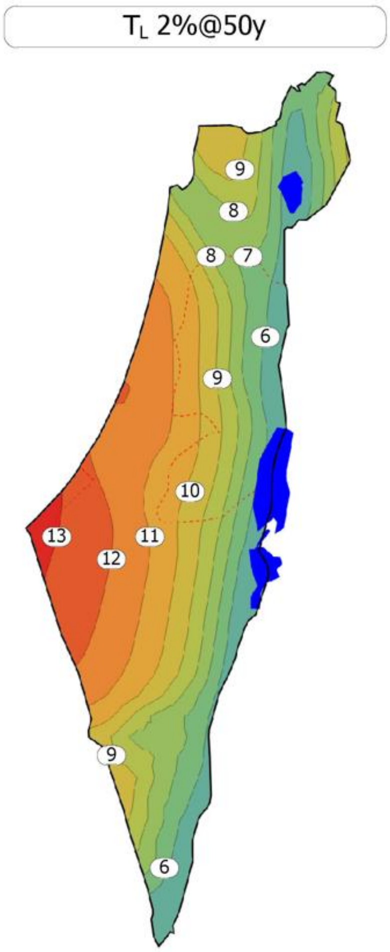


Figure1. 5 TL from Palestinian Standard.

➢ **Seismic design category:**

The seismic design category based on short-period response acceleration was determined from **Table 1.17** and it is Category D, which determined based on short-period response acceleration parameter (S_{DS}) .

Table1. 17 (ASCI-22, Table 11.6-1)

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

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

Seismic Design Category Based on 1 sec, period response acceleration parameter as shown in T**able 1.18**.

Table1. 18 (ASCI-22, Table 11.6-2)

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

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

The seismic design category is **"D"**.

➢ Selection of the structural system:

Selection and limitations, the basic lateral and vertical seismic force-resisting system shall conform to one of the types indicated in Table 12.2-1 or a combination of systems as permitted in the code. Each system is subdivided by the types of vertical elements used to resist lateral seismic forces. The structural systems used shall be in accordance with the structural system limitations and the limits on structural height, h_n, contained in Table 12.2-1. The appropriate response modification coefficient, R; overstrength factor, Ω0; and deflection amplification factor, Cd, indicated in Table 12.2-1shall be used in determining the base shear, element design forces, and design story drift.

Design Coefficients and Factors for Seismic Force –Resisting Systems as shown in **Table 1.19** and **Table 1.20**.

Table1. 19 (ASCE, Table 12.2-1).

B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames	14.1	8	2	4	NL	NL	160	160	100
2. Steel special concentrically braced frames	14.1	6	2	5	NL	NL	160	160	100
3. Steel ordinary concentrically braced frames	14.1	3¼	2	3¼	NL	NL	35^{j}	35^{j}	NP^{j}
4. Special reinforced concrete shear walls^{g,h}	14.2	6	2½	5	NL	NL	160	160	100
5. Reinforced concrete ductile coupled walls^{l}	14.2	8	2½	8	NL	NL	160	160	100
6. Ordinary reinforced concrete shear walls^{g}	14.2	5	2½	4½	NL	NL	NP	NP	NP
7. Detailed plain concrete shear walls^{g}	14.2 and 14.2.2.7	2	2½	2	NL	NP	NP	NP	NP
8. Ordinary plain concrete shear walls^{g}	14.2	1½	2½	1½	NL	NP	NP	NP	NP
9. Intermediate precast shear walls^{g}	14.2	5	2½	4½	NL	NL	40^{i}	40^{i}	40^{i}
10. Ordinary precast shear walls^{g}	14.2	4	2½	4	NL	NP	NP	NP	NP
11. Steel and concrete composite eccentrically braced frames	14.3	8	2½	4	NL	NL	160	160	100
12. Steel and concrete composite special concentrically braced frames	14.3	5	2	4½	NL	NL	160	160	100
13. Steel and concrete composite ordinary braced frames	14.3	3	2	3	NL	NL	NP	NP	NP
14. Steel and concrete composite plate shear walls	14.3	6½	2½	5½	NL	NL	160	160	100
15. Steel and concrete composite special shear walls	14.3	6	2½	5	NL	NL	160	160	100
16. Steel and concrete composite ordinary shear walls	14.3	5	2½	4½	NL	NL	NP	NP	NP
17. Special reinforced masonry shear walls	14.4	5½	2½	4	NL	NL	160	160	100

Table1. 20 (ASCI, Table 12.2-1)

C. MOMENT-RESISTING FRAME SYSTEMS									
1. Steel special moment frames	14.1 and 12.2.5.5	8	3	5½	NL	NL	NL	NL	NL
2. Steel special truss moment frames	14.1	7	3	5½	NL	NL	160	100	NP
3. Steel intermediate moment frames	12.2.5.7 and 14.1	4½	3	4	NL	NL	35^{k}	NP^{k}	NP^{k}
4. Steel ordinary moment frames	12.2.5.6 and 14.1	3½	3	3	NL	NL	NP^{l}	NP^{l}	NP^{l}
5. Special reinforced concrete moment frames^{m}	12.2.5.5 and 14.2	8	3	5½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2½	NL	NP	NP	NP	NP
8. Steel and concrete composite special moment frames	12.2.5.5 and 14.3	8	3	5½	NL	NL	NL	NL	NL
9. Steel and concrete composite intermediate moment frames	14.3	5	3	4½	NL	NL	NP	NP	NP
10. Steel and concrete composite partially restrained moment frames	14.3	6	3	5½	160	160	100	NP	NP
11. Steel and concrete composite ordinary moment frames	14.3	3	3	2½	NL	NP	NP	NP	NP
12. Cold-formed steel—special bolted moment frame^{n}	14.1	3½	3^{o}	3½	35	35	35	35	35

➢ Response Modification Factor (R)

The Response modification factor represents the ductility of the structural components considering the non-linear behaviour.

In order to determine R, Ω_o and C_d values, the seismic structural system should be selected. It was assumed as a building frame system (special reinforced concrete shear walls) .

Values of R, Ω_o and C_d were determined for the selected structural system from Table (ASCE 7-22, Table 12.2-1)

Response Modification Factor (R) = 6

Over-strength Factor (Ω_o) = 2.5

Deflection Amplification Factor (C_d) = 5

And for moment resisting frame system (special reinforced concrete moments frame) .

Response Modification Factor (R) = 8

Over-strength Factor (Ω_o) = 3

Deflection Amplification Factor (Cd) = 5.5

Where :

*** Response modification factor (R)**: is a key parameter in seismic construction design. And it use to reduction the design seismic force .R indicates the ability of a structure to dissipate energy through inelastic behavior, as demonstrated in recent building codes. The intent of the R factor is to simplify the structural design process such that only linearly elastic static analysis (i.e., the equivalent lateral force procedure) is needed for most building design.

*** Overstrength factor (Ω):** The overstrength, which is specified as member or structural capacity, is usually defined using overstrength factor, which may be defined as the ratio of maximum base shear in actual behaviour to first significant yield strength in structure. This factor came from that the real strength of a structure may be higher than its design strength because of overall design simplifications.

*** Deflection Amplification Factor:** A Deflection Amplification Factor is introduced to predict expected maximum deformations from that produced by the design seismic forces. The value of the deflection amplification factor Cd 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.

➢ **Structural period**

The approximate fundamental period (T_a), in second, shall be determined from the following equation:

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

Where:

h_n: is the hight of the building from ground floor without staircase hight.

C_t: is building period coefficient.

C_t and x are determined from (**Tables 1.21** and **1.22** in ASCE -7-22):

Table1. 21 Parameters Ct and x for moment-resisting frame.

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)*	0.8
Concrete moment-resisting frames	0.016 (0.0466)*	0.9
Steel eccentrically braced frames in accordance with Table 12.2-1, line B1 or D1	0.03 (0.0731)*	0.75
Steel buckling-restrained braced frames	0.03 (0.0731)*	0.75
All other structural systems	0.02 (0.0488)*	0.75

*SI equivalents in parentheses.

Table1. 22 Parameters Ct and x for Building frame system

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)*	0.8
Concrete moment-resisting frames	0.016 (0.0466)*	0.9
Steel eccentrically braced frames in accordance with Table 12.2-1, line B1 or D1	0.03 (0.0731)*	0.75
Steel buckling-restrained braced frames	0.03 (0.0731)*	0.75
All other structural systems	0.02 (0.0488)*	0.75

*SI equivalents in parentheses.

For moment-resisting frame system:

h = 16m (from ground level to top of the building structure excluding the staircase).

$C_t = 0.0466$

X = 0.9

$Ta = 0.0466 \times 16^{0.9} = 0.565$ sec.

For Building frame system:

$C_t = 0.0488$

X = 0.75

$T_a = 0.0488 \times 16^{0.75} = 0.39$ sec.

T from ETABS $\leq T_a C_u$

Where Cu is Coefficient for upper limit on calculated Period and can be determinate from ASCE 7-22 table 12.8-1 as shown in **Table1.23.**

Table1. 23 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

$S_{D1} = 0.2324$

By interpolation $C_u = 1.4676$

The period is limited to T_a C_u= 0.565× 1.4676= 0.83 sec when structural system is moment-resisting frame system.

The period is limited to T_a C_u= 0.3904 × 1.4676= 0.573 sec when structural system is building frame system.

➢ **Seismic Base Shear:**

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

$V = C_s W$

Where:

C_s: the seismic response coefficient.

W: the effective seismic weight.

- $C_s = (S_{DS} \times I_e) / R = (0.784 \times 1.5) / 8 = 0.147$ for moment resisting frame system.
- $C_s = (S_{DS} \times I_e) / R = (0.784 \times 1.5) / 6 = 0.196$ for building frame system.
- $C_{s,\,min} = 0.044\, S_{DS} \times I_e \geq\ 0.01 \rightarrow 0.0517\ < 0.196 \rightarrow$OK

$0.0517 < 0.147 \rightarrow$OK.

When $T \leq T_L$ use the following equation:

- $C_{s,\,max} = \dfrac{S_{D1} \times I_e}{T \times R}$

Where T_L = 8 sec according **Figure 1.5.**

- $C_{s,\,max} = \dfrac{S_{D1} \times I_e}{T \times R} = \dfrac{0.2324 \times 1.5}{0.83 \times 8} = 0.052\ < 0.147$

Use Cs = 0.052 for moment- resisting frame system.

→ V = 0.052 W

- $C_{s,\,max} = \dfrac{S_{D1} \times I_e}{T \times R} = \dfrac{0.2324 \times 1.5}{0.57 \times 6} = 0.101 < 0.196$

Use Cs = 0.101 for building- frame system.

→ V = 0.101 W

The value of 'W' will be determined later by using ETABS.

➢ **Response Spectrum curves:**

This dynamic method done by using computer program by defining a function called response spectrum depends on the period and acceleration as follow:

T: the fundamental period of the structure.

T_o: 0.2 S_{D1}/ S_{DS}

T_S: S_{D1} /S_{DS}

T_L: long-period transition period (s).

In this project:

S_{DS} = 0.784

S_{D1}= 0.2324

T = 0.565sec. (For moment-resisting frame system)

T = 0.3904 sec. (For building frame system)

T_o = 0.0592 sec.

T_S = 0.2964 sec.

T_L = 8 sec.

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

$S_a = S_{D1}/T$

For moment-resisting frame system S_a = 0.411

For building frame system S_a = 0.595

Figure 1.6 and **Figure 1.7** show response spectrum curve for moment resisting frame system and building frame system respectively.

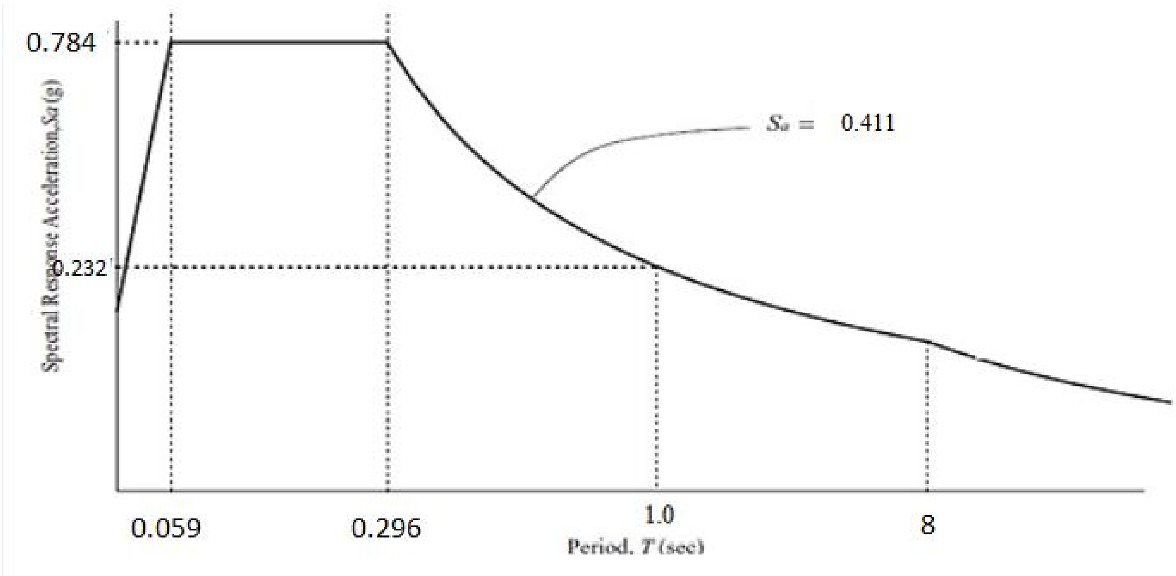


Figure1. 6 Response spectrum curve for moment resisting frame

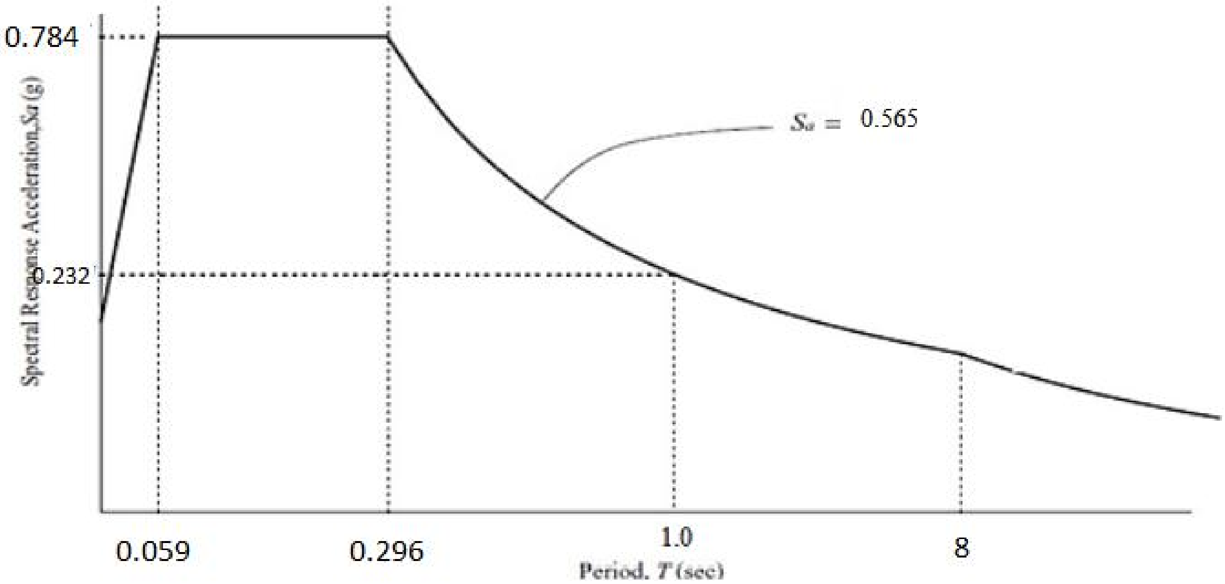


Figure1. 7 Response spectrum curve for Building frame .

1.7 Load Combinations:

A load combination results when more than one load type acts on the structure. Building codes usually specify a variety of load combinations together with load factors (weightings) for each load type in order to ensure the safety of the structure under different maximum expected loading scenarios.

Basic 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

Basic ultimate load combinations:

1. 1.4D
2. 1.2D + 1.6L + 0.5(Lr 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 (Lr or S or R)+(L or 0.5W)
7. 1.2 D+1.0 W+L +0.5 (Lr or S or R)

Symbols:

D: Dead Load.

L: Live Load.

Lr: Roof Live Load.

S: Snow Load.

R: Rain Load.

W: Wind Load.

E: Earthquake Load.

➢ **Ultimate load combinations:**

Table1. 24 Ultimate Load Combinations.

1	1.4DL+1.4SID
2	1.2DL+1.2SID+1.6LL + 0.5S
3	1.2DL+1.2SID+1.0 LL + 1.6 S
5.1	1.36 DL + 1.36 SID + 1.0 LL + 1.3 E_{qx} + 0.39 E_{qy} + 0.2S
5.2	1.36 DL + 1.36 SID + 1.0 LL + 1.3 E_{qy} + 0.39 E_{qx} + 0.2S
6	0.9DL+0.9SID
7.1	0.74 DL+ 0.74 SID + 1.3Eqx + 0.39E_{qy}
7.2	0.74 DL + 0.74 SID + 1.3Eqy + 0.39E_{qx}

Sample Calculation:

For Equation 5: 1.2DL+1.2SID+ 1.0 LL + 1.0Eq + 0.2S

-Eq= E_h + E_v for DL >=1.0.

-Eq = E_h - E_v for DL < 1.0.

-E_h=ρ× (E_{qx} + 0.3Eqy) where ρ=1.3 for Seismic Design D,E, and F.

E_h= 1.3(Eqx + 0.3Eqy) = 1.3Eqx + 0.39Eqy

-E_v = 0.2 S_{Ds} × DL, where S_{DS} =0.784

E_v = 0.2 × 0.784 × DL

E_v=0.157 DL

For Equation 5: 1.2DL+1.2SID+ 1.0 LL + 1.0Eq + 0.2S

=1.2 DL +1.2 SID + 1.0 LL + 1.0 (E_v+E_h) + 0.2S

=1.2 DL +1.2 SID + 1.0 LL + 1.0 (0.157 DL + 1.3Eqx + 0.39Eqy) + 0.2S

Equation 5.1: $\boxed{1.36 \text{ DL} + 1.36 \text{ SID} + 1.0\text{LL} + 1.3\text{Eqx} + 0.39\text{Eqy} + 0.2\text{S}}$

Equation 5.2: $\boxed{1.36 \text{ DL} + 1.36 \text{ SID} + 1.0\text{LL} + 1.3\text{Eqy} + 0.39\text{Eqx} + 0.2\text{S}}$

➢ **Service Load Combinations:**

Table1. 25 Service Load Combinations

1	1.0 DL + 1.0 SID
2	1.0 DL + 1.0 SID + 1.0 LL
3	1.0 DL + 1.0 SID + 1.0 S
4	1.0 DL +1.0 SID +0.75 LL + 0.75 S
5.1	1.1 DL + 1. 1SID + 0.91Eqx + 0.273Eqy
5.2	1.1 DL + 1.1 SID + 0.91Eqy + 0.273Eqx
6a	1.0 DL + 1.0 SID +0.75 LL + 0.75 S
6b.1	1.1 DL + 1.1 SID + 0.75 LL + 0.68 Eqx + 0.2Eqy + 0.75S
6b.2	1.1 DL + 1.1 SID + 0.75 LL + 0.68 Eqy + 0.2Eqx + 0.75S
7	0.6 DL + 0.6 SID
8.1	0.45 DL + 0.45 SID + 0.91Eqx + 0.273Eqy
8.2	0.45 DL + 0.45 SID + 0.91Eqy + 0.273Eqx

1.8 Geotechnical Report:

Soil is one of the most important parts that affect the structure, and inaccuracy of determining the bearing capacity of the soil, can cause a failure in the structure, partially or entirely. Describing soil in an engineering design system comes from identifying the soil bearing capacity which is "The ultimate load which a foundation can support", bearing capacity value is calculated from many tests and the most common test for shallow soil is the bearing capacity test.

The soil in the site consists mainly of soft rock. The subsurface investigation was necessary to determine the geotechnical conditions and the suitable solutions to ensure that the proposed building will be designed and constructed properly.

According to the soil investigation report, the top soil existing at the site is mainly soft rock with a bearing capacity of about 300 KN/m^2 and soil type "C".

In this project there is no need to dig a large distance to erect walls , so the coefficient of earth pressure at rest (K_o) is decided to be equal to 0.15 as equivalent value .

CHAPTER 2: PRELIMINARY DESIGN

2.1 General:

Each structure consists of structural elements such as slabs, columns, walls, and beams that are designed to resist lateral and gravity loads, these elements are connected to transmit these loads safely to the soil underneath. This chapter includes the preliminary dimensions of these elements according to ACI 318-19. The preliminary dimensions of the structural elements will be used in developing the building's structural modal and may be modified if they are not adequate.

2.2 Lateral and gravity forces resisting systems:

-Building Frame System with special reinforced concrete shear walls will be selected for both B1 and B2 Levels, in addition to the right side of the building.

- For the left part of the building (GF, F1, F2, and F3), the system is Moment Resisting Frame System with special reinforced concrete shear walls

-Frame system will be selected for gravity forces resisting.

2.3 Slabs Structural System:

This means that the type of slabs and beams and the dimensions of them which used in the structure.

The slab system is two-way solid slab with drop beams.

Figures below shows the distribution of beams in each floor. (More clear drawing will be in appendix)

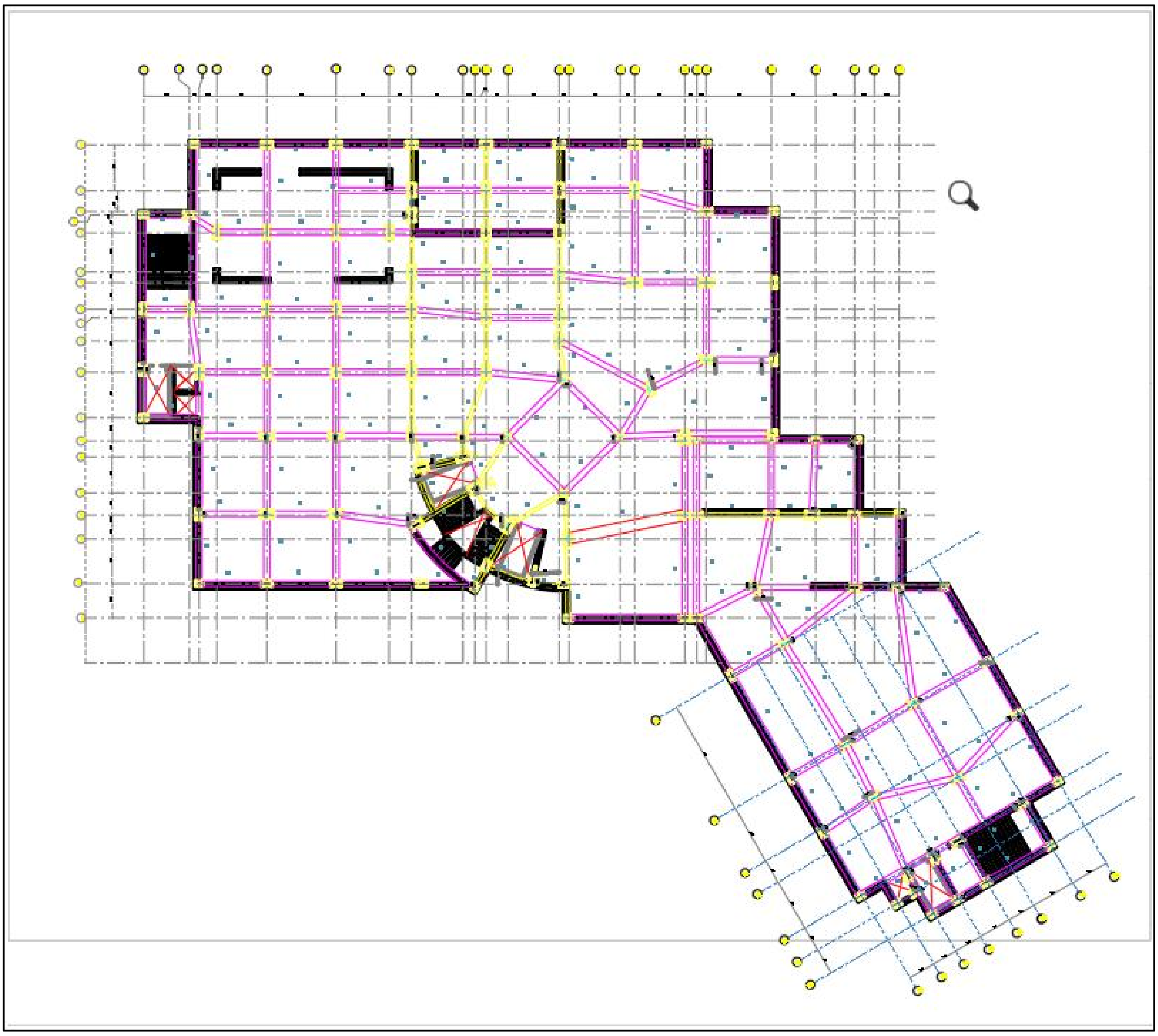


Figure 2.1 Beams distribution (basement one and basement two).

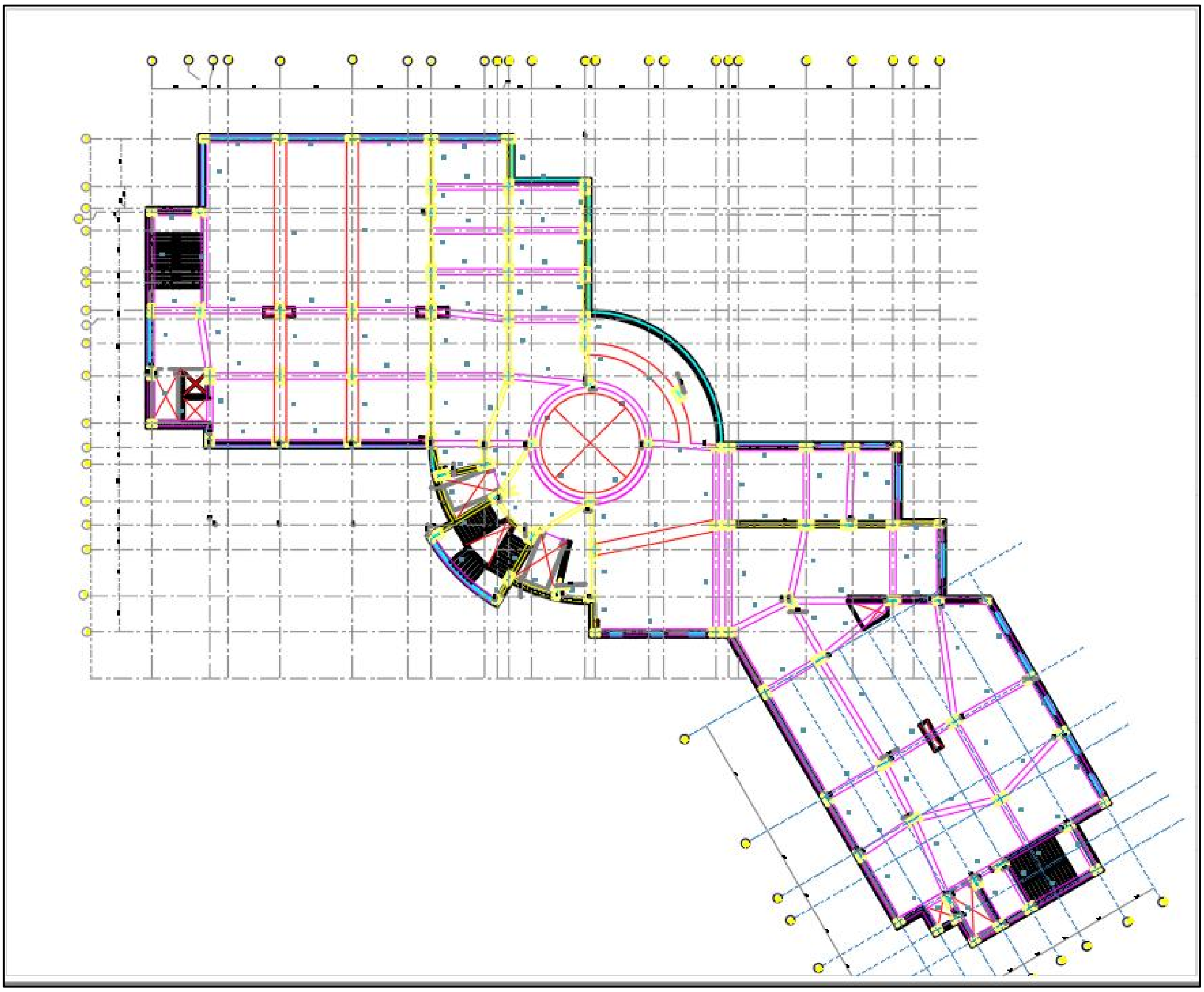


Figure 2.2 Beams distribution (ground floor).

Figure 2.3 Beams distribution (first floor).

Figure 2.4 Beams distribution (second floor).

Figure 2.5 Beams distribution (third floor).

2.4 Preliminary slab thickness and loads:

In this project it was decided that solid slab with beams between all columns is chosen.

Preliminary slab thickness will be calculate considering critical panel, which is shown in Figure 2.6.

L_n: largest panel dimension

Assume $L_{n=}$ L :

$$B = \frac{L_{n\_large}}{L_{n\_small}}$$

$$= \frac{8.49}{7.29} = 1.16 < 2$$, so the panel approximately square in shape.

The approximate calculations of the slab were based on Table 8.3.1.2 in ACI-318-19, as shown in Table 2.1.

Table 2.1 Minimum thickness of nonprestressed two-way slabs with beams between supports on all sides

α_{fm} [1]	Minimum h, mm		
$\alpha_{fm} \leq 0.2$	8.3.1.1 applies		(a)
$0.2 < \alpha_{fm} \leq 2.0$	Greater of:	$\dfrac{L_n\left(0.8+\dfrac{f_y}{1400}\right)}{36+5\beta(\alpha_{fm}-0.2)}$	(b)[1],[2]
		125	(c)
$\alpha_{fm} > 2.0$	Greater of:	$\dfrac{L_n\left(0.8+\dfrac{f_y}{1400}\right)}{36+9\beta}$	(d)
		90	(e)

Assume $\alpha f_m > 2$:

$$h_{min} = \frac{Ln \times (.8+\frac{Fy}{1400})}{36+9\beta}$$ >90 mm according to table 2.1

$$\rightarrow h_{min} = \frac{8.49\times(0.8+\frac{420}{1400})}{36+(9\times1.16)} = 0.201\ m$$, use h = 200 mm.

Check αf_m for that panel shown in **Figure 2.6**:

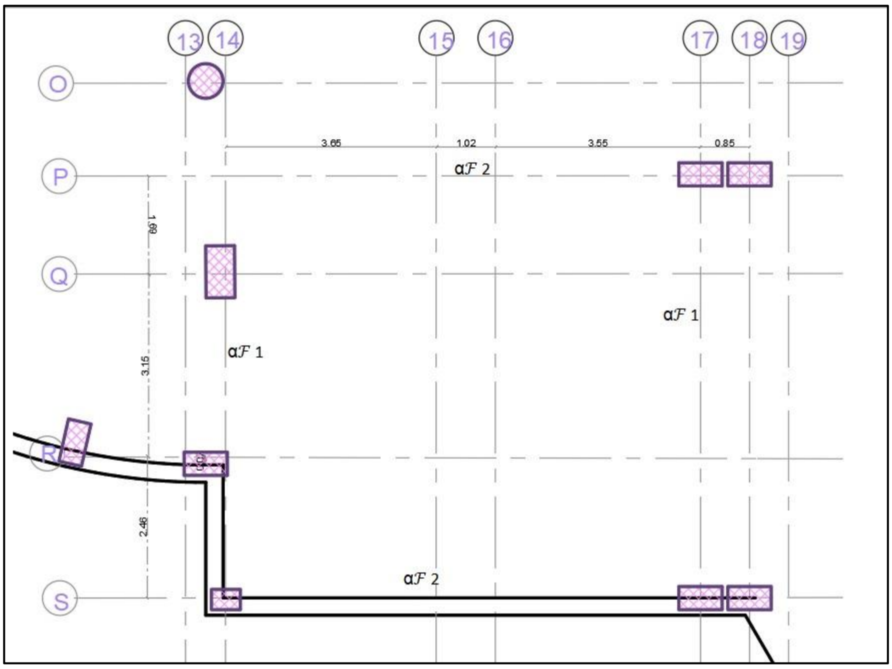


Figure 2.6 Critical panel

Beam and slab size calculation:

*** For L = 7.29 m:**

Beam thickness (h) = $\dfrac{L}{18.5} = \dfrac{7.29}{18.5} = 0.39$ m

By enlarge beam thickness thirty percent:

h = 1.3× 0.39 = 0.5 m

Beam width (b) = $\dfrac{L}{20} = \dfrac{7.29}{20} = 0.4$ m

$$I_{beam} = \frac{b \times h^3}{12} = \frac{0.4 \times 0.5^3}{12}$$

$$= 4.17 \times 10^{-3}\ m^4$$

Slab width = 4.11 m.

$$I_{slab} = \frac{b \times h^3}{12} = \frac{4.11 \times 0.2^3}{12} = 2.74 \times 10^{-3}\ m^4$$

$$\alpha_{fm(1)} = \frac{I_{beam}}{I_{slab}} = \frac{4.17 \times 10^{-3}}{2.74 \times 10^{-3}} = 1.52$$

*** For L =8.49 m:**

Beam thickness (h) = $\frac{L}{21} = \frac{8.49}{21}$ = 0.4 m

By enlarge beam thickness thirty percent:

h =0.4 × 1.3 = 0.53 m …Use h = 0.55 m

Beam width (b) == $\frac{L}{20} = \frac{8.49}{20}$ =0.42 m

Use b =0.45 m.

$I_{beam} = \frac{b \times h^3}{12} = \frac{0.45 \times 0.55^3}{12} = 6.24 \times 10^{-3}\ m^4$

Slab width =6.29 m

$I_{slab} = \frac{bh^3}{12}$

$I_{slab} = \frac{b \times h^3}{12} = \frac{6.29 \times 0.2^3}{12} = 4.39 \times 10^{-3}\ m^4$

$\alpha_{fm(2)} = \frac{I_{beam}}{I_{slab}} = \frac{6.24 \times 10^{-3}}{4.39 \times 10^{-3}} = 1.42$

Assume:

$\alpha_{fm(1)} = \alpha f_{m(3)}$

$\alpha_{fm(2)} = \alpha f_{m(4)}$

$\alpha_{f\,m} = \frac{\alpha f1 + \alpha f2 + \alpha f3 + \alpha f4}{4}$

Where:

α_{fm} = average value of α_f for all beams on edges of a panel

$\alpha_{f\,m} = \frac{2 \times 1.52 + 2 \times 1.42}{4} = 1.47 < 2$ …Not OK, use the first equation in **Table 2.1**.

$h_{min} = \frac{L_n(0.8 + \frac{f_y}{1400})}{36 + 5\beta(a_{fm} - 0.2)}$

$= 0.18$ m

Use h =0.20 m

Check for shear:

Refer to **1.6.2** and **1.6.3** the critical live load (LL) and superimposed load (SD) are:

LL = 6 kN/m^2

SD = 7.5 kN/m^2

Slab own weight (DL) = 0.20×25 = 5 kN/m^2

W_u =1.2 D +1.6 L

Where:

W_u: Ultimate load.

D : Dead load.

L: Live load.

Wu = 1.2(5+7.5) + 1.6×6

= 26 kN/m^2

The shear can be calculated considering the short direction of the largest panel.

X-direction:

$$V_u = Wu \times \frac{L}{2} - \frac{b}{2} - d$$

$$= 26 \times \left(\frac{4.5}{2} - \frac{0.4}{2} - 0.15\right) = 49.4 \text{ KN}/\text{m}$$

Where:

Wu: ultimate load in slab.

L: short panel dimension.

d: effective depth of slab.

Shear strength capacity of the slab depending on ACI 318-19 as shown in **Table2.2**.

Table 2.2 V_c for nonprestresses members

Table 22.5.5.1—V_c for nonprestressed members

Criteria	V_c		
$A_v \geq A_{v,min}$	Either of:	$\left[0.17\lambda\sqrt{f'_c} + \frac{N_u}{6A_g}\right]b_w d$	(a)
		$\left[0.66\lambda(\rho_w)^{1/3}\sqrt{f'_c} + \frac{N_u}{6A_g}\right]b_w d$	(b)
$A_v < A_{v,min}$	$\left[0.66\lambda_s\lambda(\rho_w)^{1/3}\sqrt{f'_c} + \frac{N_u}{6A_g}\right]b_w d$		(c)

Notes:
1. Axial load, N_u, is positive for compression and negative for tension.
2. V_c shall not be taken less than zero.

Assume that $A_v < A_{v,min}$

$$\phi v_c = \left[0.66 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \sqrt{f'_c} + \frac{N_u}{6A_g}\right] b_w d$$

Where:

Ø V_c: shear strength capacity of the slab .

Ø: Shear reduction factor = 0.75

λ: 1 for normal weight concrete.

λ_s: factor used to modify shear strength based on the effects of member depth, commonly referred to as the size effect factor.

f'_c: concrete compressive strength = 28 MPa

b_w: Web width of the slab section = 1 m

A_g: area gross.

N_u: factored axial force normal to cross section occurring simultaneously with *Vu* or *Tu*; to be taken as positive for compression and negative for tension, N

$\rho_w = 0.0018 \times \frac{h}{d}$

$$\phi v_c = 0.75 \times \left[0.66 \times 1 \times 1 \times \left(0.0018 \times \frac{200}{150}\right)^{1/3} \sqrt{28} + zero\right] \times 1000 \times 150$$

= 52.6 KN/m

$ØV_c < V_u$ → Not OK for shear.

Try h= 250mm for the larger panels

$V_u = Wu \times \frac{L}{2} - \frac{b}{2} - d$

$= 26 \times \left(\frac{4.5}{2} - \frac{0.4}{2} - 0.15\right) = 82$ KN /m

D= 210 mm

$ØV_c$= 68.6 KN/m

$ØV_c < V_u$ → Not OK for shear.

Try h = 300 mm

d =260 mm

Vu = 79

$ØV_c$ = 86.88 KN > 79 KN → OK

Using the same procedure for all floors, slab preliminary thickness in all floor are equal to 300 mm.

*** Preliminary ramp thickness:**

The main ramp for the building starts from basement one up to the entrance of basement two, which is located between grid lines 19, C, K and 20.

Ramps are going to be designed as one-way solid slab and will be performed on ETABS as horizontal slabs, however, the modifiers for flexural stiffness will be very small to neglect their stiffness as diaphragms.

Preliminary ramp thickness was calculated according to ACI-318-19 Table 7.3.1.1. as shown in **Table 2.3.**

Table 2.3 Minimum thickness of solid non-prestressed one- way slab

CODE

Table 7.3.1.1—Minimum thickness of solid nonprestressed one-way slabs

Support condition	Minimum h[1]
Simply supported	$\ell/20$
One end continuous	$\ell/24$
Both ends continuous	$\ell/28$
Cantilever	$\ell/10$

[1]Expression applicable for normalweight concrete and f_y = 420 MPa. For other cases, minimum h shall be modified in accordance with 7.3.1.1.1 through 7.3.1.1.3, as appropriate.

Minimum thickness = L/24 = 4.5/24 = 0.1875m, try h= 200 mm

2.5 Preliminary dimensions of beams:

Beam dimension was calculated according to ACI 318-19, table 9.3.1.1 as shown in **Table 2.4.**

Table 2.4 Minimum depth of nonprestressed beam

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

Sample calculation:

*** Beam thickness:**

The critical span has a length equals to 11.50 m "One end continuous" therefore,

$h = \frac{L_n}{18.5}$ based on **Table 2.3**.

$h = \frac{11.5}{18.5}$

$= 0.62m$

Enlarge thickness by thirty presents → $h = 0.62 \times 1.3$

$= 0.80$ m

*** Beam Width(b):**

$$\text{Beam width(b)} = \text{Max.} \begin{cases} \frac{L}{20} \\ \frac{h}{2} \\ 0.20 \end{cases}$$

Where:

L: span length

h: beam thickness

$$b = \text{Max.} \begin{cases} \frac{11.5}{20} \\ \frac{h}{2} \\ 0.20 \end{cases}$$

$$= \text{Max.} \begin{cases} \frac{11.5}{20} \\ \frac{h}{2} \\ 0.20 \end{cases}$$

$$= \text{Max.} \begin{cases} 0.575 \\ 0.4 \\ 0.20 \end{cases}$$

$= 0.575$ m

Use $b = 0.6$ m

But because of the drop is to large, the width and thickness will be reversed, then b=800mm and h=600mm.

Table 2.5 include preliminary dimension of beams based clear span between columns.

Table 2.5 Preliminary dimension beam

Beam number	Height (mm)	Width (mm)
B1	600	250
B2	600	400
B3	600	800

2.6 Preliminary dimensions of columns:

Axial forces in columns caused by gravity loads are used in the computation of the preliminary dimensions of walls and columns, using the tributary area method. The Tributary area of a column is defined as the loaded area surrounding a column and that directly contributes to the applied loads on that column.

Sample calculation:

The initial dimensions of column 99 will be calculated as sample calculation,its located at intersection of 6` and F` grid lines as shown in **Figure 2.7**.

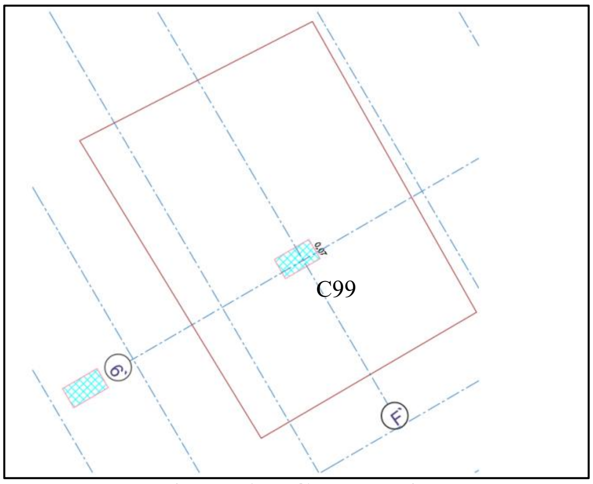


Figure 2.7 Column site

Tributary area= 44 m^2

Number of stories= 6

*** Dead Load:**

$$\text{Dead load (from drop beams)} = \text{Beam lenght} \times \text{Beam width} \times \text{Drop thickness} \times \gamma_{\text{concrete}} \times \text{Numper of Stories}$$

$$= 13 \times 0.4 \times 0.3 \times 25 \times 6$$

$$= 234 \text{ KN}$$

$$\text{Dead load (from slab weight)} = \text{Tributary area} \times \text{Slab thickness} \times \gamma_{\text{concrete}} \times \text{numper of Stories}$$

$$= 44 \times 0.3 \times 25 \times 6$$

$$= 1980 \text{ KN}$$

*** Superimposed load:**

$$\text{Superimposed load} = \text{Tributary area} \times \text{SD in story} \times \text{numper of Stories}$$

$$= 44 \times [2 \times 4.5 + 4 \times 7.5]$$

= 1716 KN

*** Live load:**

$$\text{Live load} = \text{Tributary area} \times \text{LL in story} \times \text{numper of Stories}$$

$$= 44 \times [2 \times 3 + 4 \times 5]$$

= 1144 KN

*** Ultimate Load:**

$P_u = 1.2D + 1.6L$

Pu = 1.2 × (234+1980+1716) + 1.6 ×1144

= 6546.4 KN

Assume 1% steel ratio, the column is short, and no moment.

$$\phi P_n = \phi \lambda (0.85 f_c (A_g - A_s) + F_y A_s)$$

Where:

Ø: Reduction capacity factor and equals to 0.65 for tied column.

P_n: Nominal axial compressive strength of column in kN.

λ: Reduction factor to consider minimum eccentricity and equals 0.8 for tied column.

A_g: Gross area of the concrete section in mm^2.

A_s: Total area of steel reinforcement inside column in mm^2.

F_y: Specified yield strength for non-prestressed reinforcement in (MPa).

f'_c: Specified compressive strength of concrete, cylinder at 28 days in (MPa).

$A_s = \rho \times Ag$

ρ: steel ratio and equal to 1% of column area.

$$\phi P_n = 6546.4 \times 1000 = 0.65 \times 0.8 (0.85 \times 28 (A_g - 0.01 A_g) + 420 \times (0.01 A_g))$$

$A_g = 453\,470\ mm^2$

Try column 900 mm x 500 mm

Table 2.6 show all column preliminary dimensions.

Table 2.6 Columns dimension

Column Number	Type	Dimension
C9,C10,C11,C12,C15,C16,C17,C18,C19,C20,C21,C22,C25MC26,C27,C28C29, C32,C33,C34,C35,C36,C37,C38,C40,C41,C42,C43,C44,C46,C47,C51,C52,C53, C54,C64,C65,C66,C69,C70,C71,C72,C73,C88,C89,C96,C98,C99,C102,C103, C106,C107,C108	R	500x 900
C2,C3,C4,C5,C6,C7,C14,C23,C30,C31,C39,C48,C50,C58,C59,C60,C61,C62, C68,C74,C75,C78,C81,C82,C83,C85,C86,C87,C90,C93,C94,C95,C97,C100, C101,C104,C109,C112,C114,C115.	R	400x 750
C1,C8,C13,C24,C49,C63,C79,C80,C84,C91,C92,C105,C110,C111,C113,C116.	R	350x 500
C45,C55,C56,C67.	C	600

Where:

R: rectangular.

C: Circular.

2.7 Preliminary dimensions of walls:

Two types of walls in this structure will be used:

1. Basement retaining walls.
2. Elevators and staircase walls. (Shear walls).

From architectural plans, elevators and staircase walls have an initial thickness equal to 200 mm,in contrast, retaining walls have an initial thickness equal to 300 mm.

CHAPTER 3: THREE-DIMENSIONAL ANALYSIS AND DESIGN FOR THE LEFT PART OF THE HOSPITAL BUILDING

3.1 General:

This chapter discusses the three-dimensional analysis and design of the project. The structure is modeled using ETABS finite element tools, the suggested dimensions were checked and evaluated, in order to assure all previously mentioned assumptions, and changes on the dimensions of the elements was done in case the previously assumed dimensions were not sufficient for the 3D model. Mesh is automatically generated in slabs, and manually assigned in walls. Analysis results of the program are checked to verify the results., which are: compatibility check, equilibrium check and stress-strain check. Moreover, design checks were performed to ensure the stability of the structure, structural irregularities check and P – delta effect.

3.2 Structural Modeling of the Building:

This section illustrates the main definitions of structural section property of columns, beams, slabs, and walls. In additional, load patterns and load combinations are introduced. This section is detailed for educational purpose.

Figures 3.1 is showing the 3D – structure.

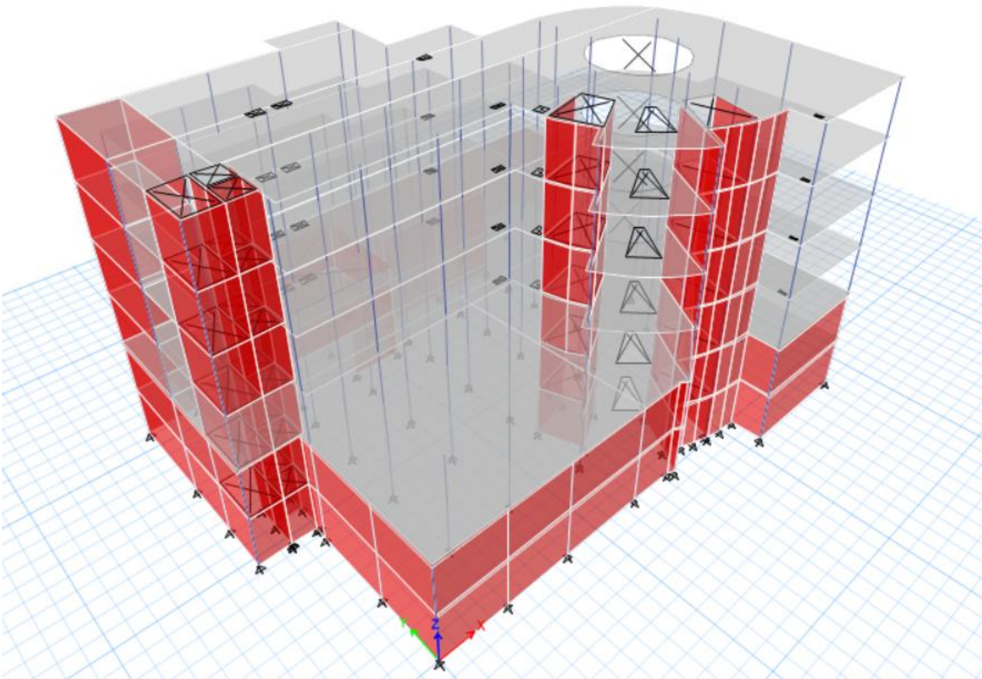


Figure3. 1: 3D - structural modal to left part

3.2.1 Units:

SI unites were used, which are meter (m) for length, kilo newton (KN) for force and Celsius (C) for temperature.

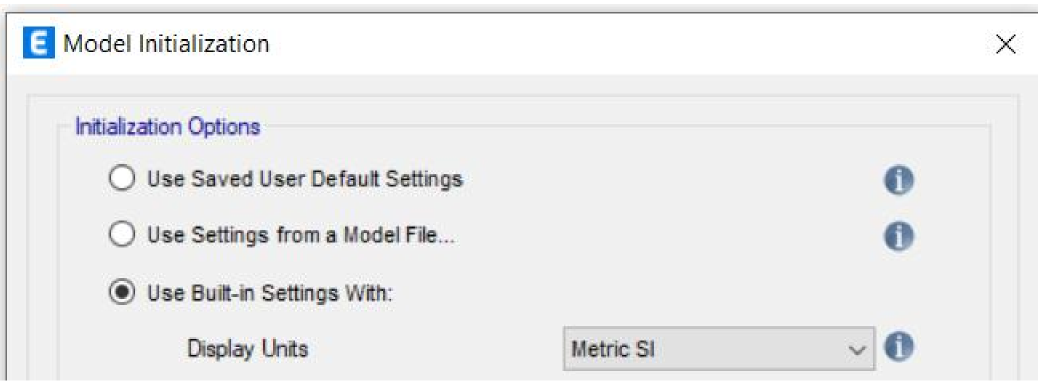


Figure3. 2 Display units in ETABS

3.2.2 Material:

Material used for slabs, beams, column and wall have compressive strength of 28 Mpa, properties are shown in **Figure 3.3.**

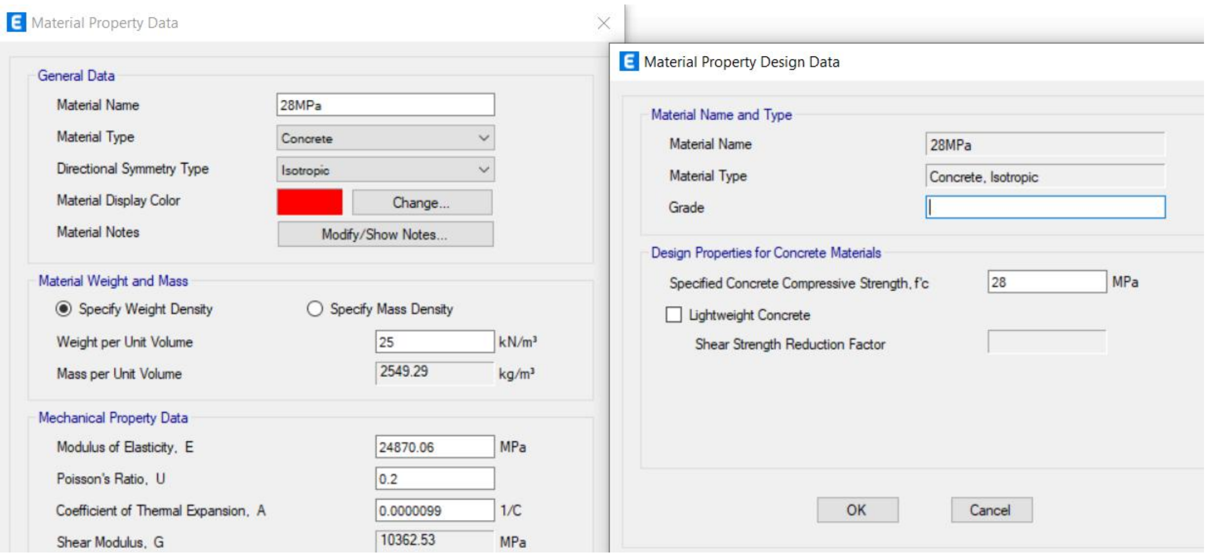


Figure3. 3 slab, beams, column and wall properties.

3.2.3 Section properties:

Frame elements:

Columns and beams are defined with preliminary dimensions mentioned in sections 2.5 and 2.6 respectively.

I. Columns:

The definition for the circular and rectangular columns are shown in **Figures 3.4** and **3.5** respectively.

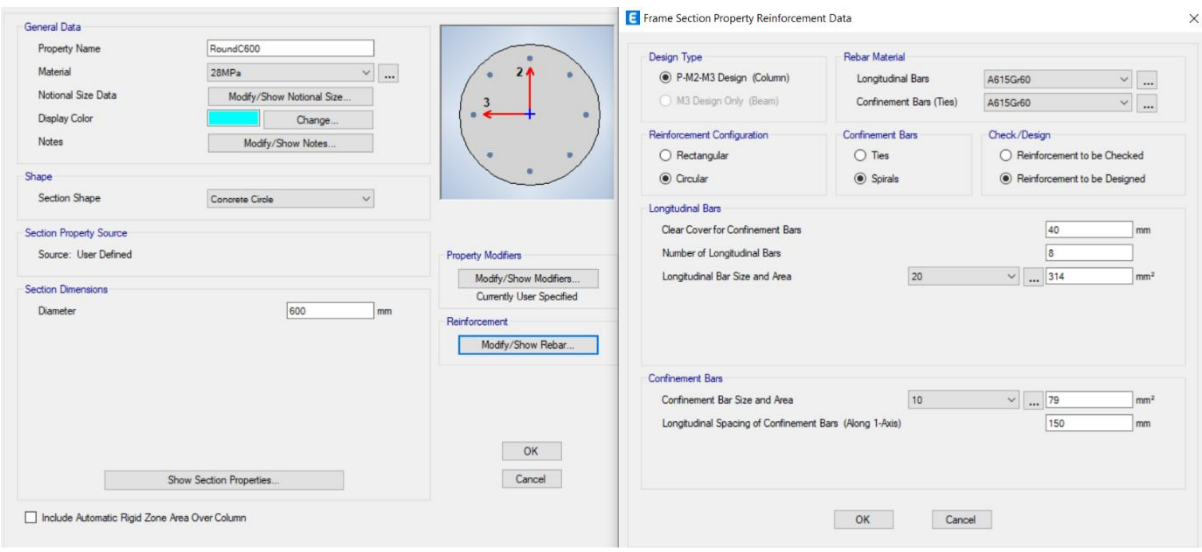


Figure3. 4 Definition of circular column section in ETABS.

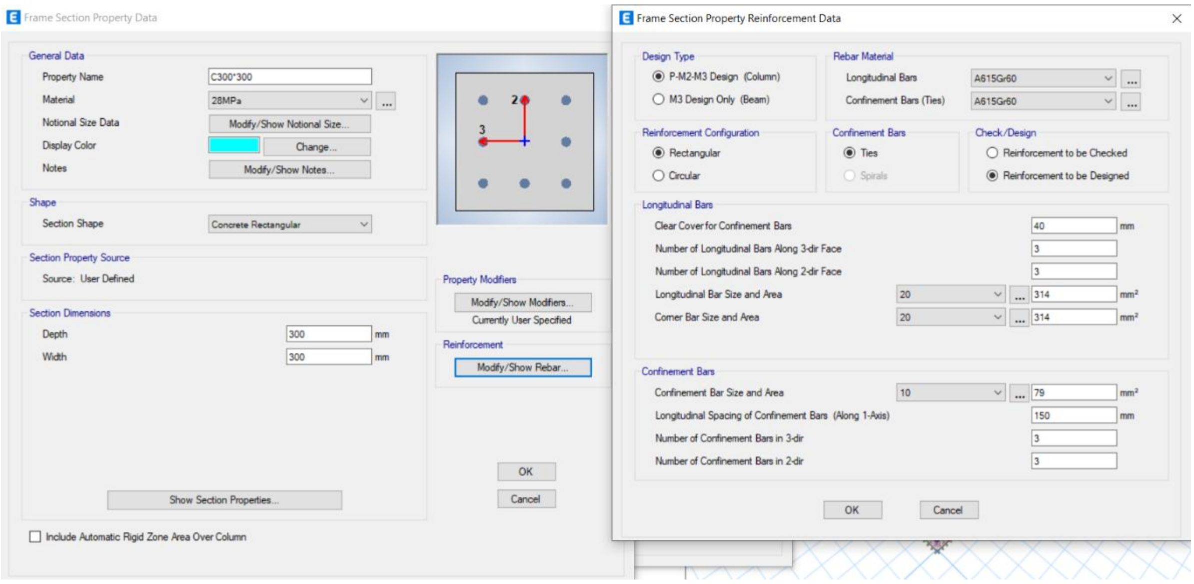


Figure3. 5 Definition of rectangular column section in ETABS

Beams:

Beam's definitions and properties are shown in **Figure 3.6**

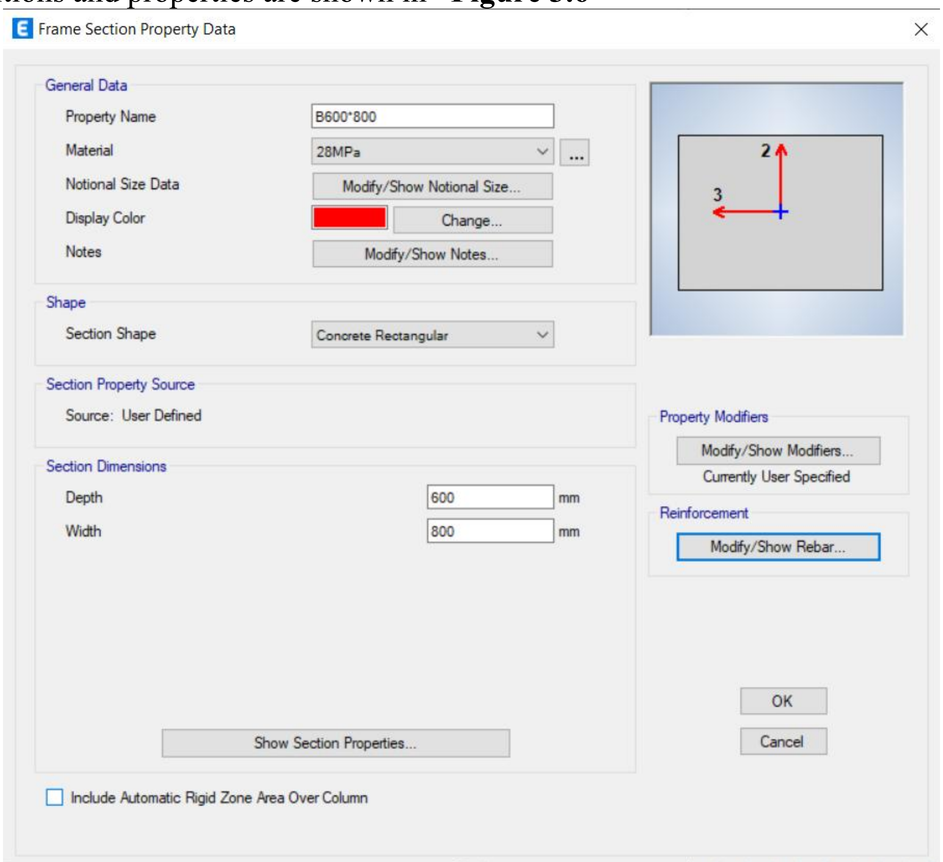


Figure3. 6 Definition of rectangular beam section in ETABS.

Shells:

I. Slabs are defined with preliminary dimensions that are mentioned in section 2.4, horizontal slabs, stairs and ramp properties are shown in **Figure 3.7**, **Figure 3.8** and **3.9** respectively.

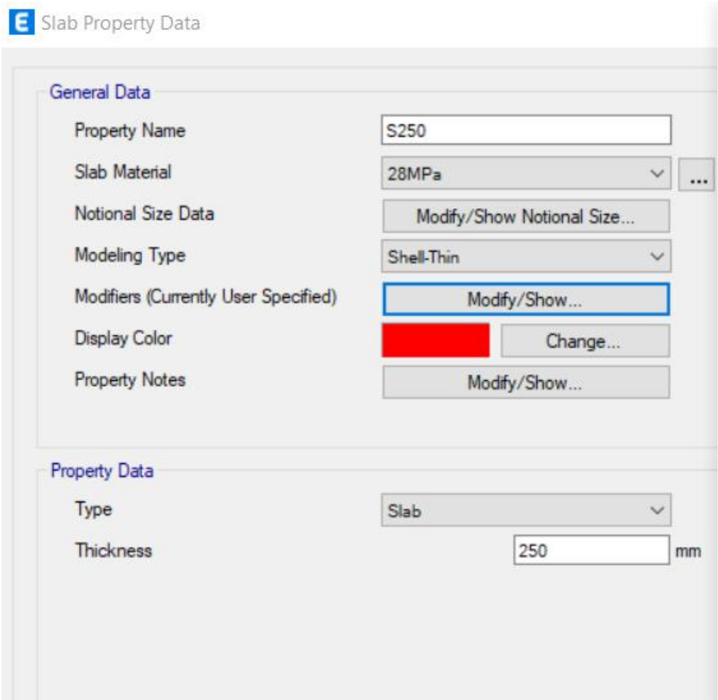


Figure3. 7 : Definition of horizontal slab section in ETABS.

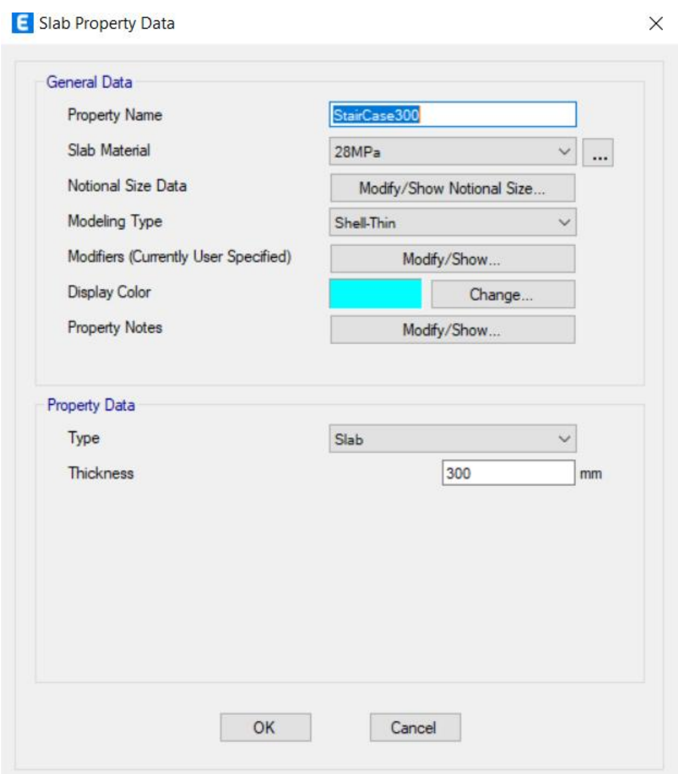


Figure3. 8 : Definition of horizontal slab section in ETABS.

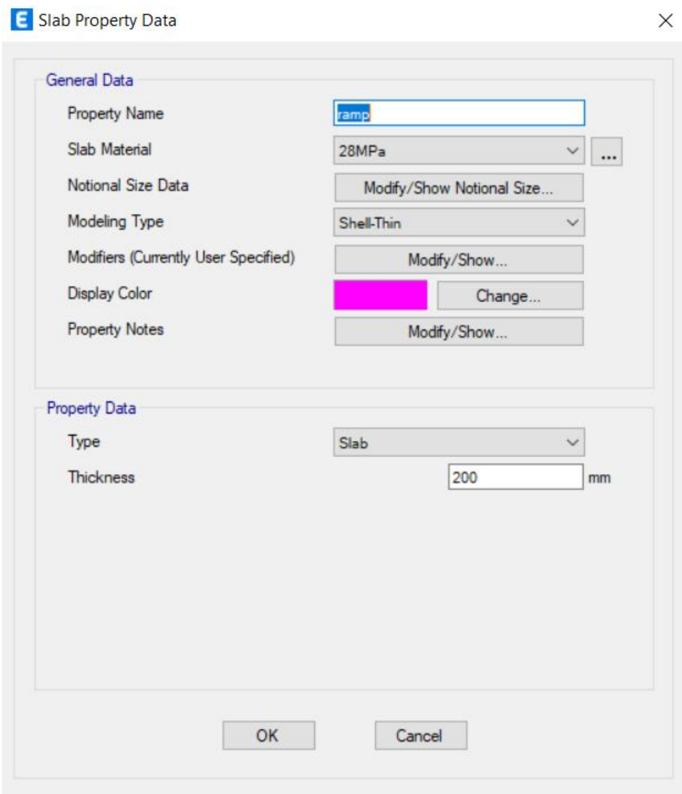


Figure3. 9 :Definition of Ramp slab section in ETABS.

II. Shear walls:

Shear walls properties shown in **Figure 3.10**.

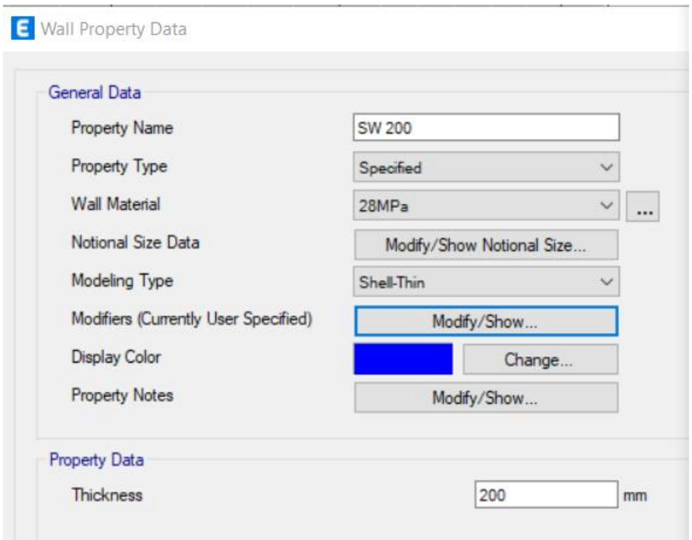


Figure3. 10: Definition of shear wall section in ETABS.

3.2.4 Section Modifiers:

To assure that the design assumptions are very similar to the real behavior of structural elements, certain modifiers are assigned in the definition of the elements, which are:

- **Slab modifiers**: figures 3.11 and 3.12 shows the used modifiers for slabs.

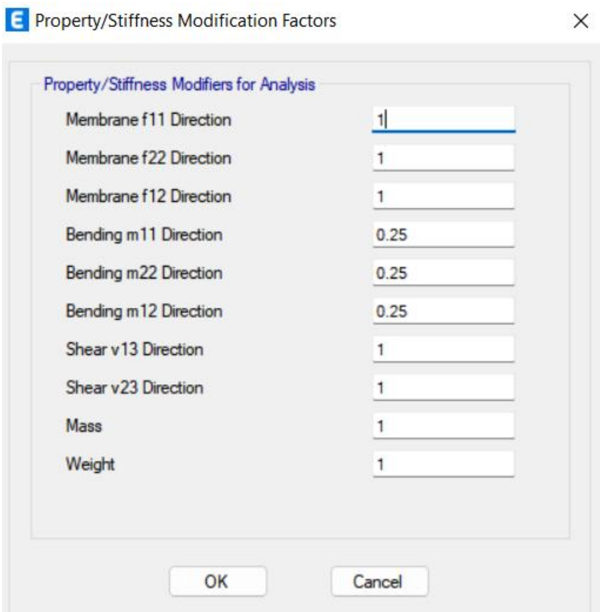


Figure3. 11Modification factor for floors slab section in ETABS.

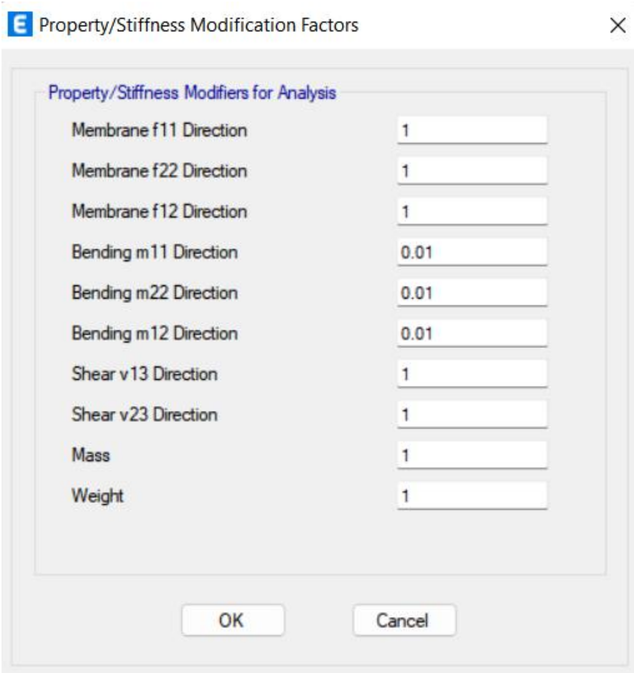


Figure3. 12: Modification factor for Staircase slab section in ETABS.

-Beam modifiers:

Figures 3.13 shows the used modifiers for beams.

Mass and weight modifiers = Thickness of beam drop part / Total thickness of beam

= (600-300)/600

= 0.5

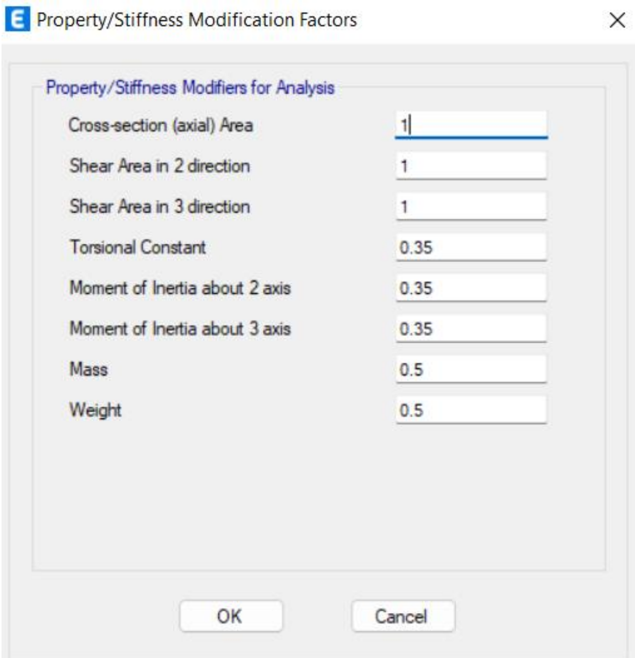


Figure3. 13 :Modification factors for beam section in ETABS.

- Column modifiers:

Figure 3.14 bellow shows the used modifiers for any column section.

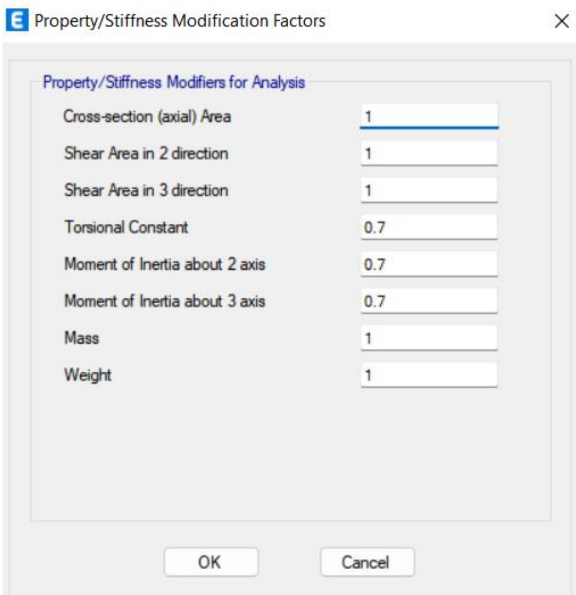


Figure3. 14 :Modification factors for column section in ETABS.

- **Wall modifiers:**

Shear walls have membrane and bending modifiers equal to 0.7 as shown in **Figure 3.15,** assuming uncracked sections.

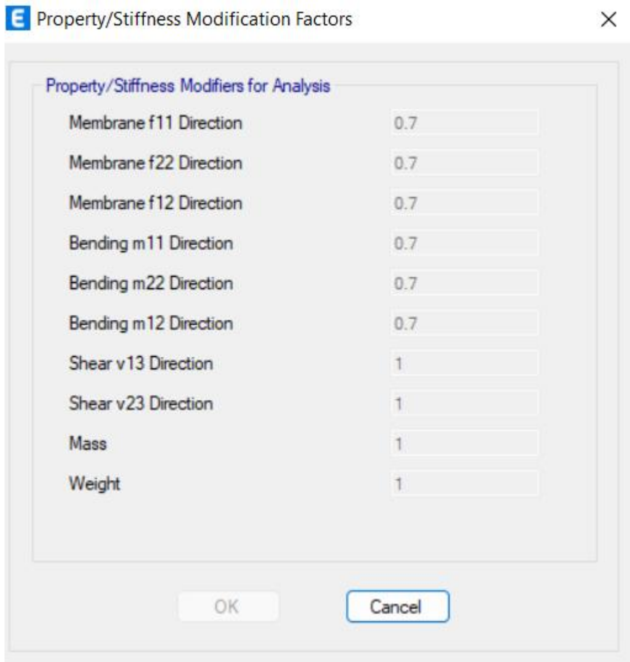


Figure3. 15 : Modification factor for walls section in ETABS.

3.2.5 Load definitions:

The process of defining all expected loads that will affect the structure. Load patterns are used to define these loads.

Figure 3.16 shows these load definitions according to section 1.6.

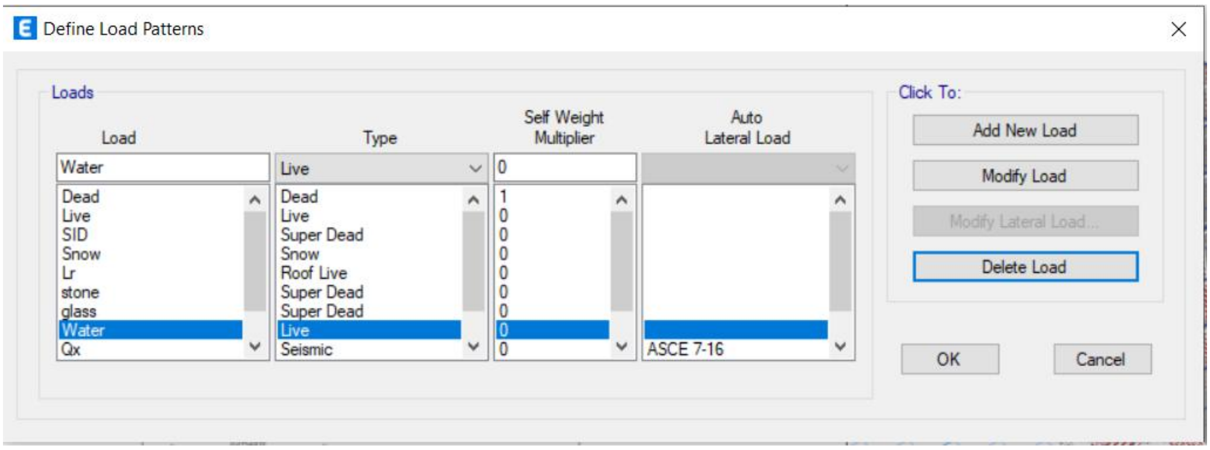


Figure3. 16: Definition of load patterns in ETABS.

3.2.7 Load cases:

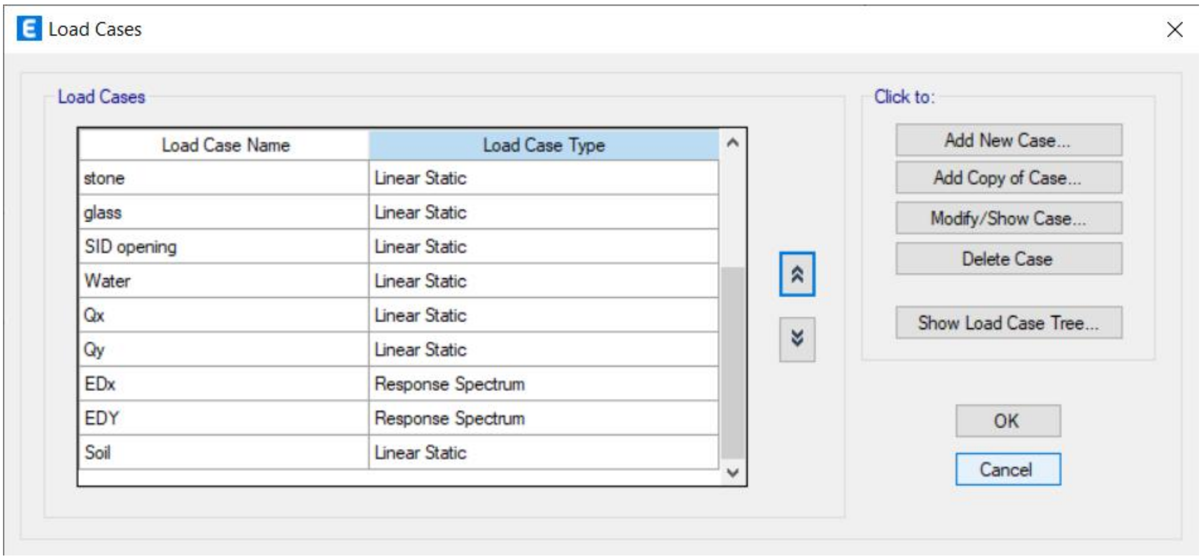


Figure3. 17: Definition of load cases in ETABS.

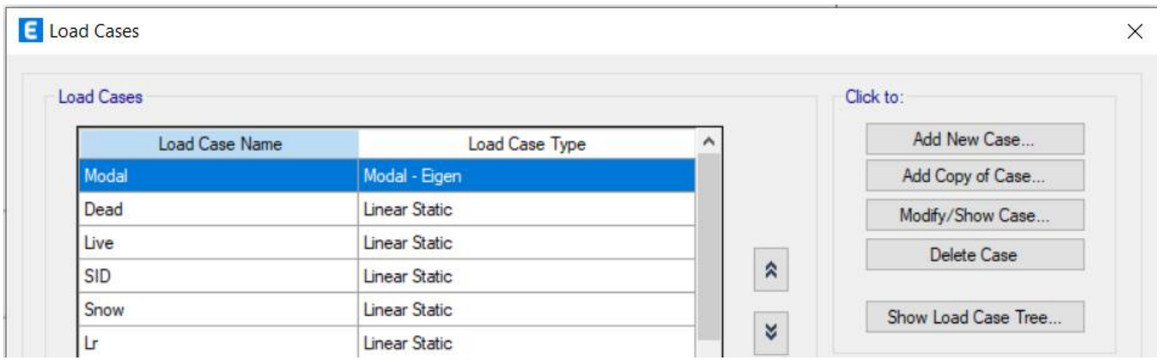


Figure3. 18: Definition of load cases in ETABS.

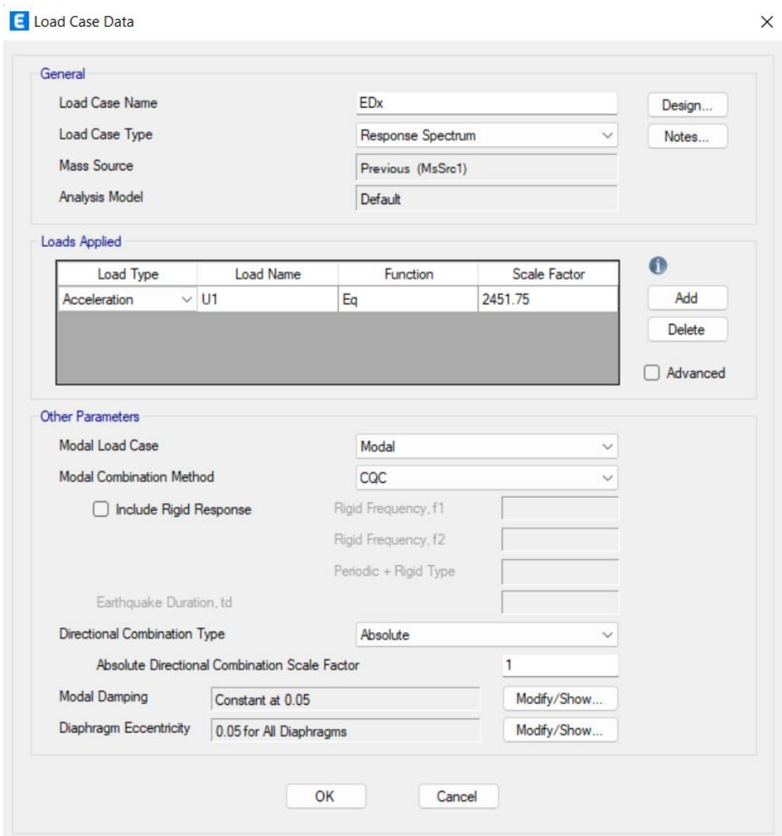


Figure3. 19 Definition of response spectrum load cases in ETABS.

***Scale factor** = I×g/R = 1.5×9807/6 = 2451.75

3.2.6 Load combination:

Figure 3.20 and **Figure 3.21** shows that load combinations used for the analysis and design process:

Where:

- **S:**is refer to service combination.

- **U:** is refer ultimate combination.

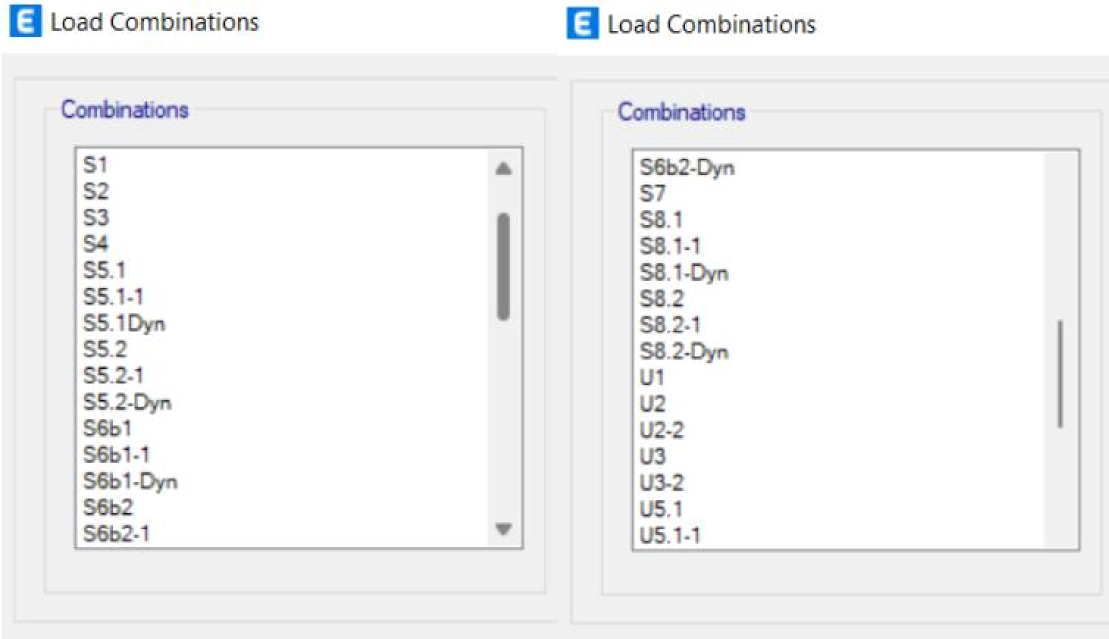


Figure3. 20: Definition of load combination in ETABS.

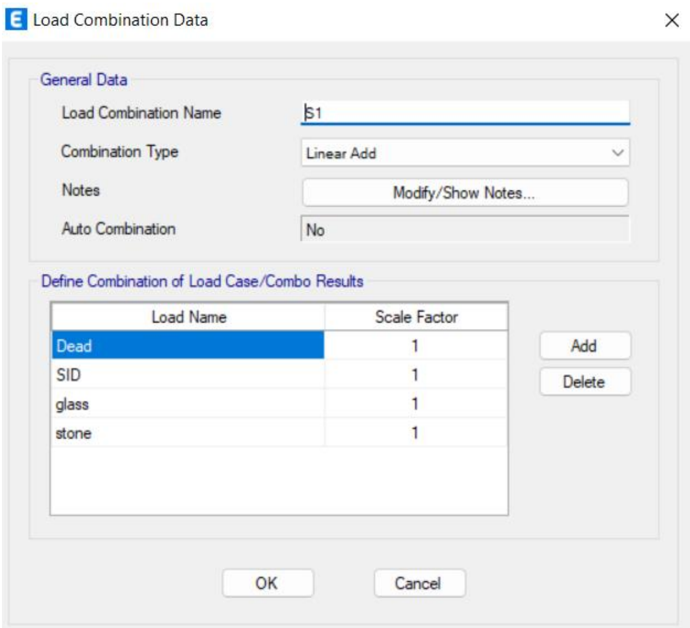


Figure3. 21:Sample of service load combination definition in ETABS.

3.2.7 Loads assignments:

Refer to section 1.6, **Figures 3.22, 3.23, 3.24,3.25,3.26** and **3.27** shows the load assignment at each floor.

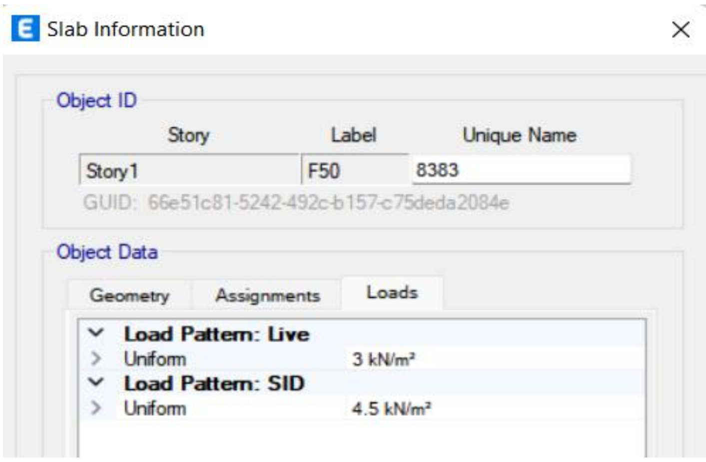


Figure3. 22: Load assignment for slab basement 1& 2 in ETABS.

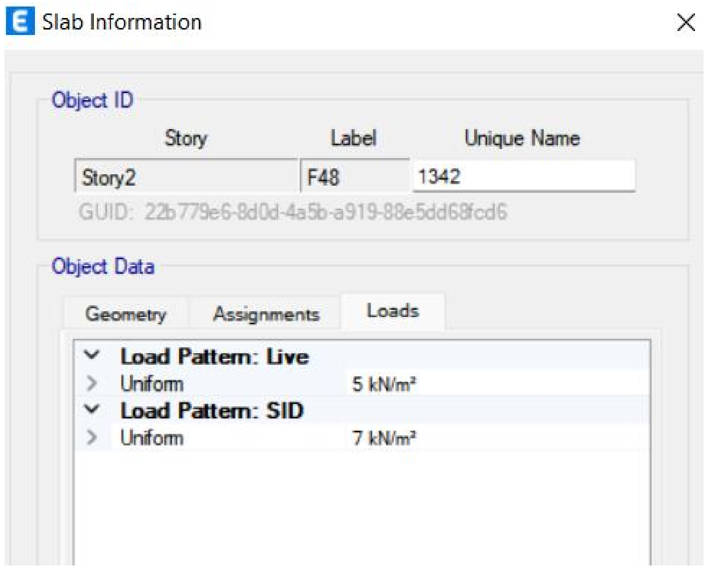


Figure3. 23:Load assignment for slab ground floor in ETABS.

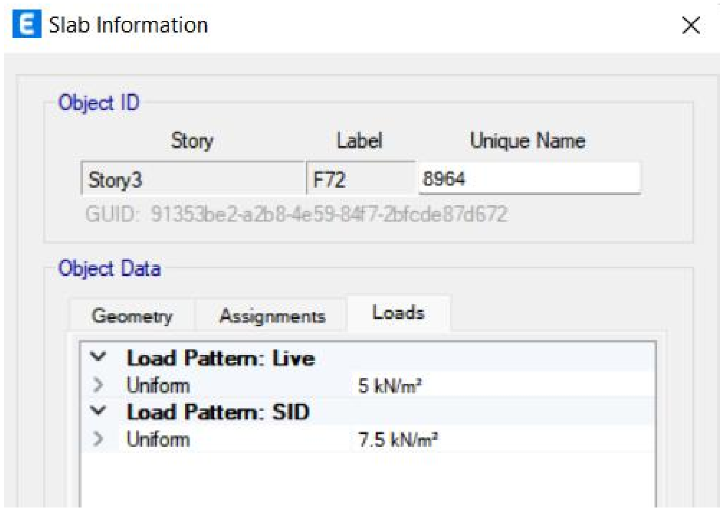


Figure3. 24: Load assignment for first slab floor in ETABS.

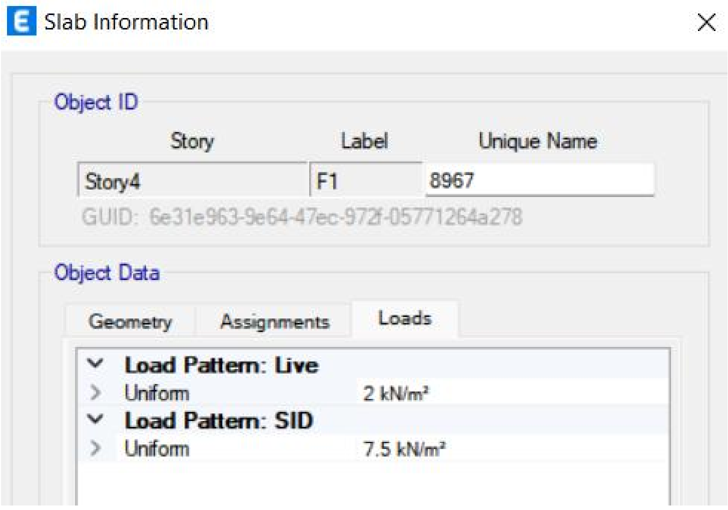


Figure3. 25: Load assignment for slab in second floor in ETABS.

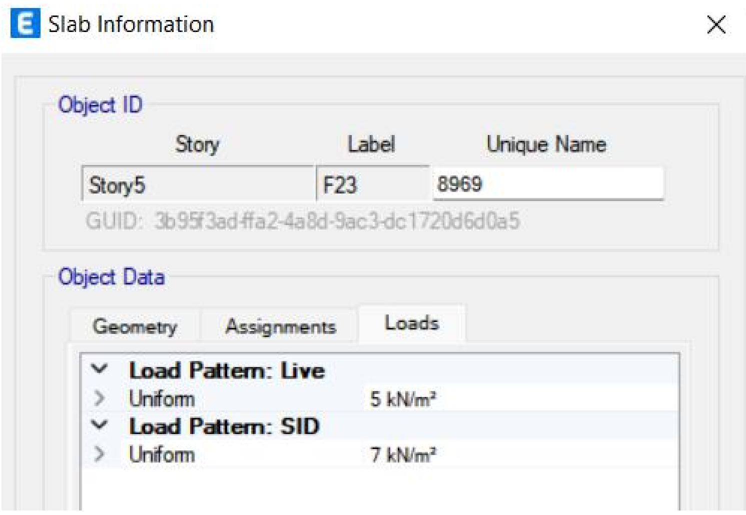


Figure3. 26: Load assignment for slab in third floor

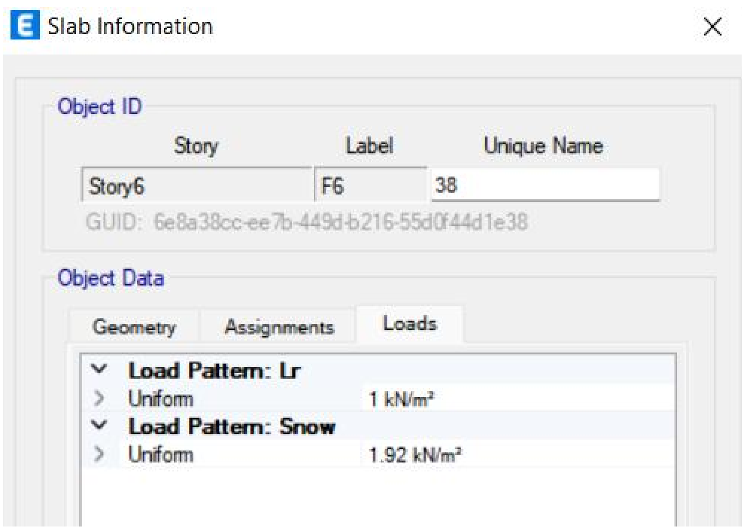


Figure3. 27:Load assignment for slab in roof in ETABS.

3.2.8 Masses:

The definition of mass source used to consider the superimposed dead load and percentage of live load in base shear calculations. **Figure 3.28** shows mass source definition.

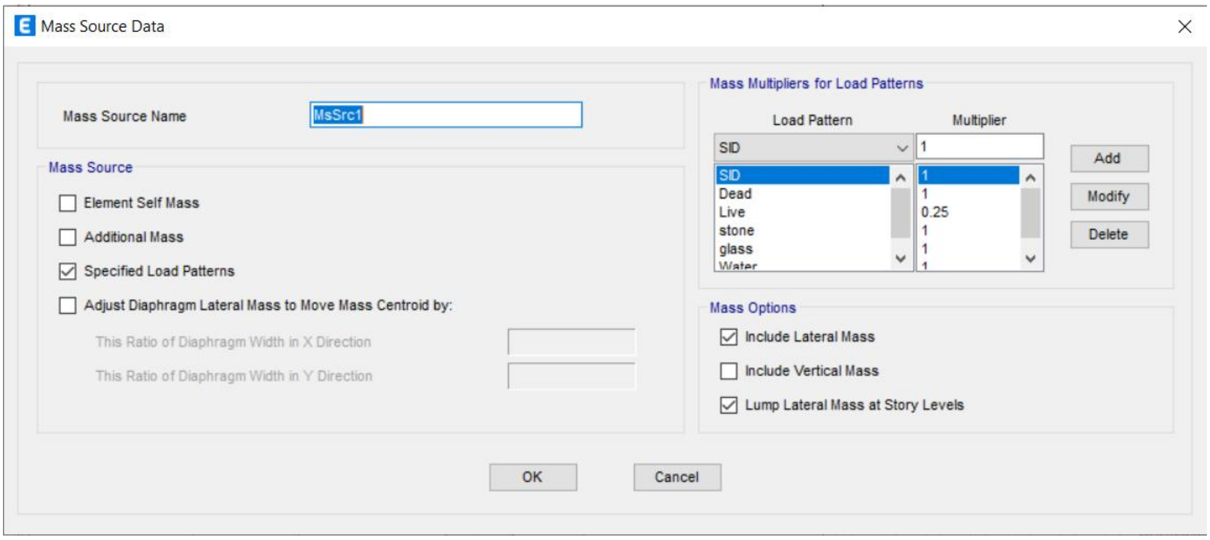


Figure3. 28: Definition of mass source data in ETABS.

3.2.9 Functions:

This defined function is a response spectrum function used to define the response curve.

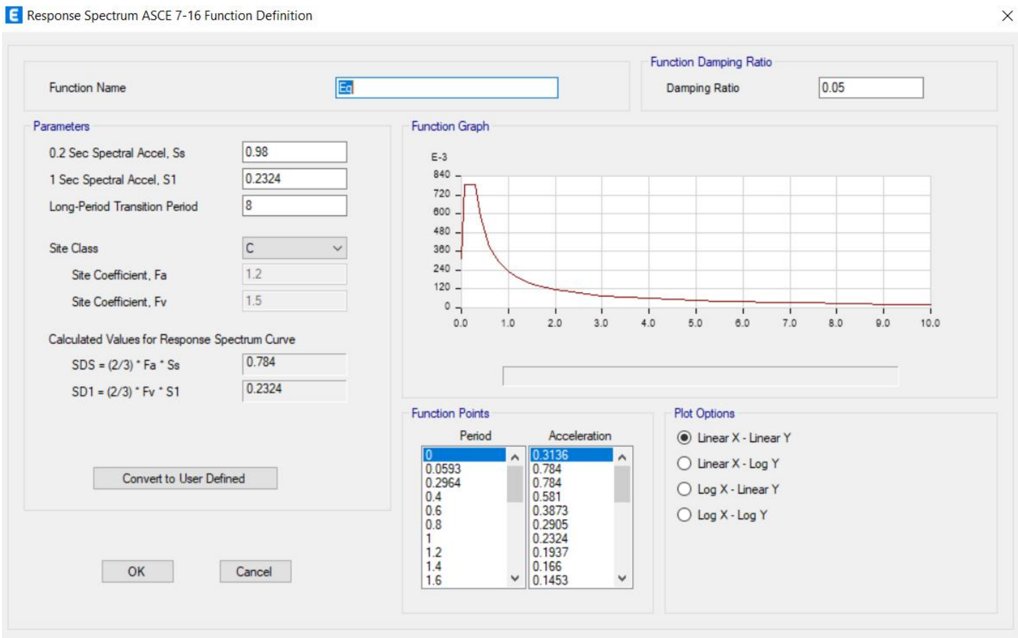


Figure3. 29:Definition of response spectrum function in ETABS.

3.2.10 Design Specification:

The code used for the design is defined and seismic factors such as: Ω, ρ and Sds.

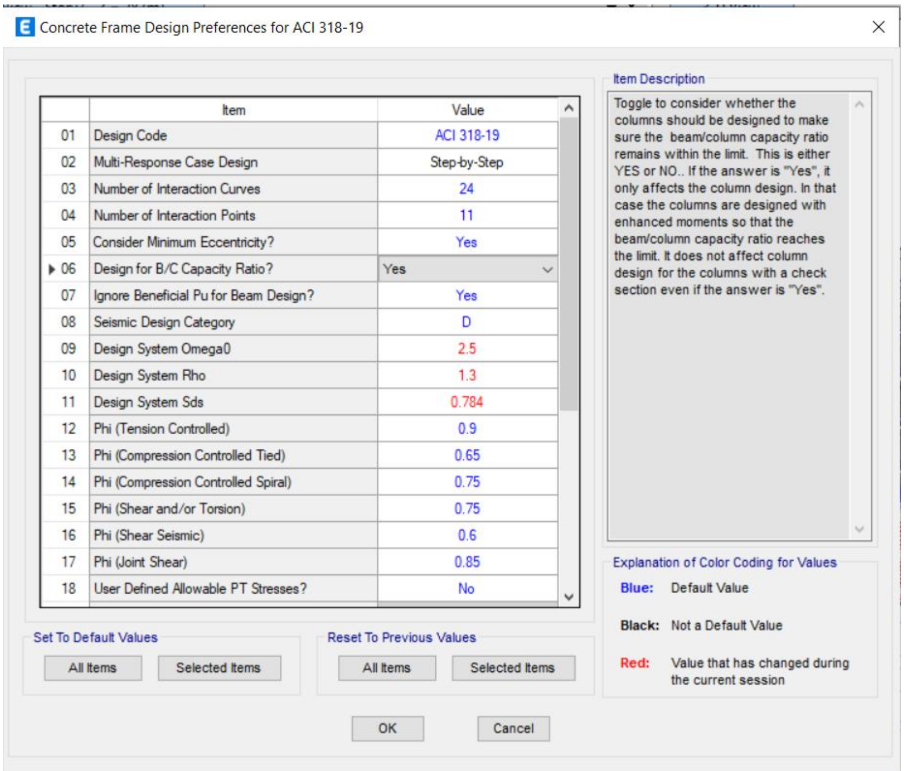


Figure3. 30:Concrete frame design preferences for ACI 318-19 in ETABS.

3.2.11 Supports:

Supports restrains were assigned to be fixed as have been assumed to resist the forces and moment in all directions.

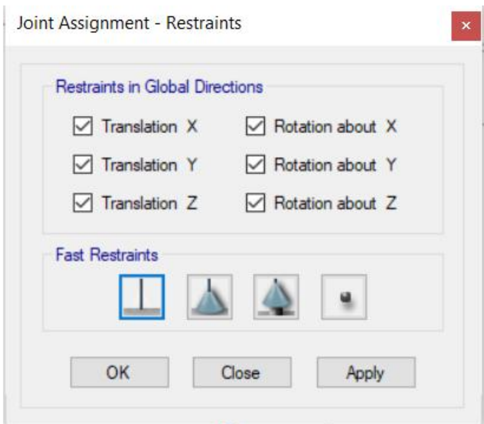


Figure3. 31Restrain type in ETABS.

3. 2.12 Codes:

The used codes were previously discussed in section 1.4, **Figure3.32** shows the selected codes used in the model definition.

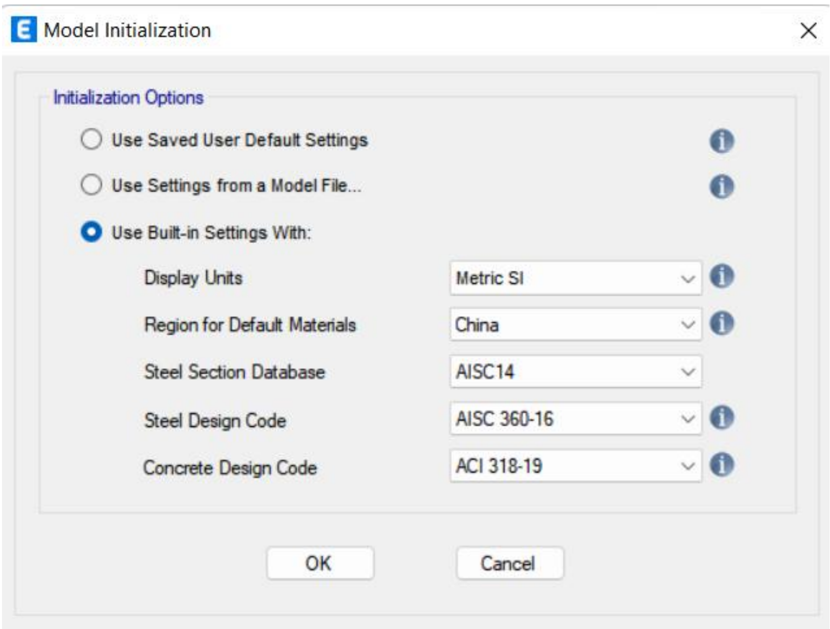


Figure3. 32: Codes used in ETABS.

3.3 Evaluation of the preliminary design for Left Model:

3.3.1 Check Dimensions of Beams:

This check is done by designing frames in the structure and then checking that there is no failure due to shear and torsion.

If the beams light up red so one of the two warning messages will appear:

1. Shear stress due to shear force and torsion together exceeds maximum allowed, or
2. Reinforcing required exceeds maximum allowed.

To come over the first problem, make the torsional constant modifier near to zero, if it is still red then the reason of failure is the shear and the solution of it is enlarging the sections. There are a lot of beams that light up red during the design and the reasons explained in detail Mention the section .

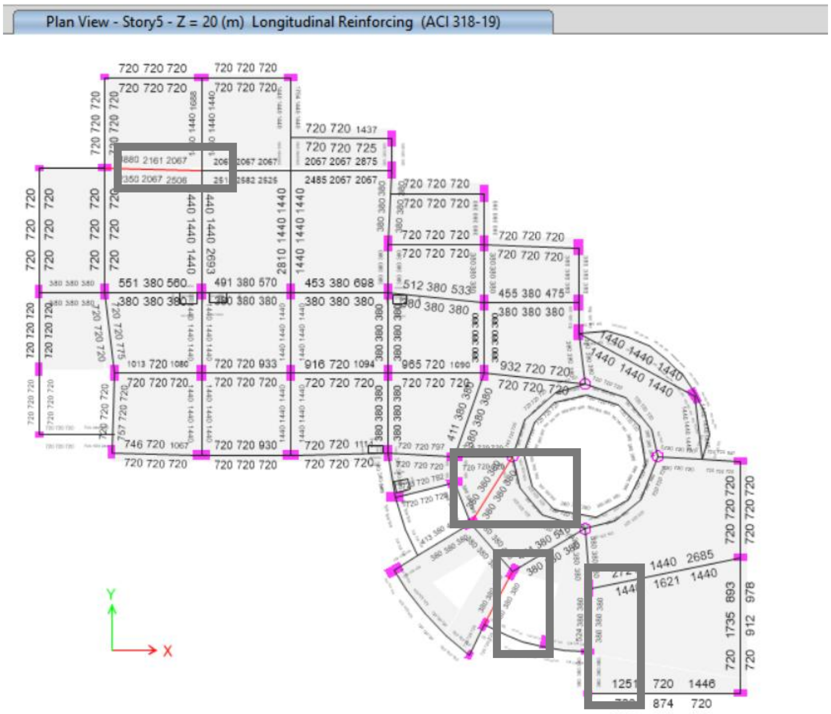


Figure3. 33Beams that light up red in ETABS due to shear stress.

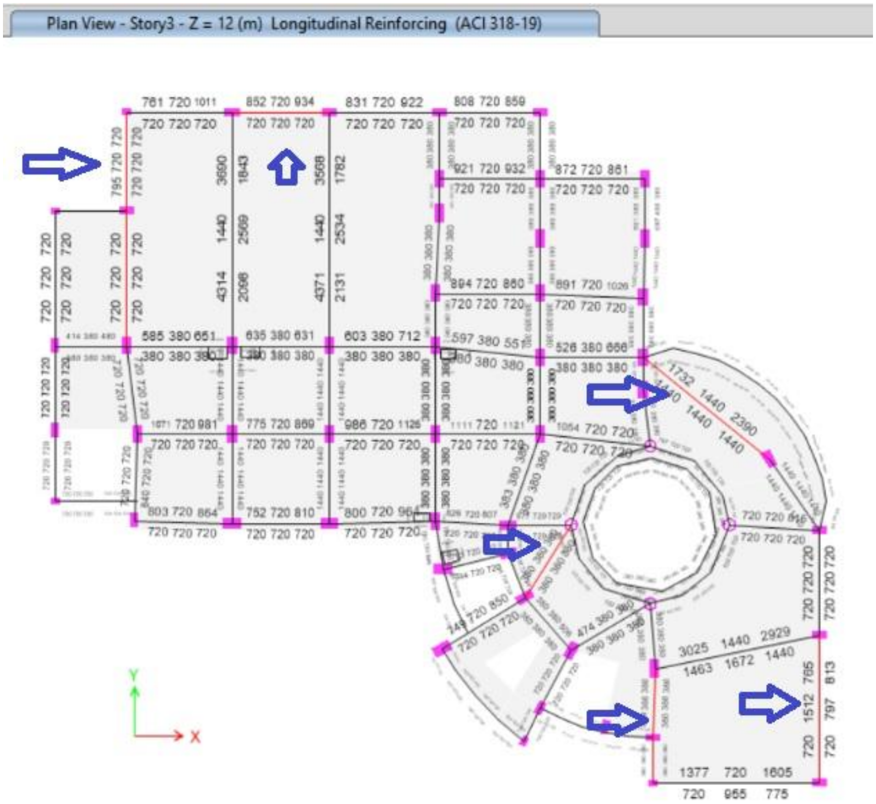


Figure3. 34 Beams that light up red in ETABS due to shear stress.

As a result of the failure in the share stress, the torsion modifier will be changed as shown in **Figure 3.35** below.

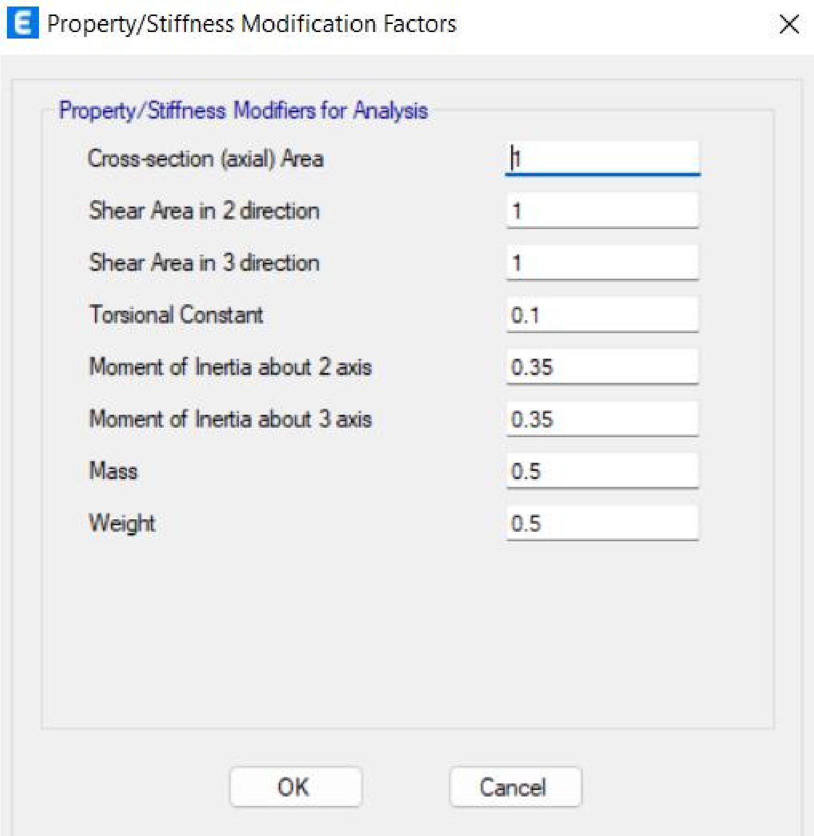


Figure3. 35New beams modifier in ETABS; decreasing the torsion modifier.

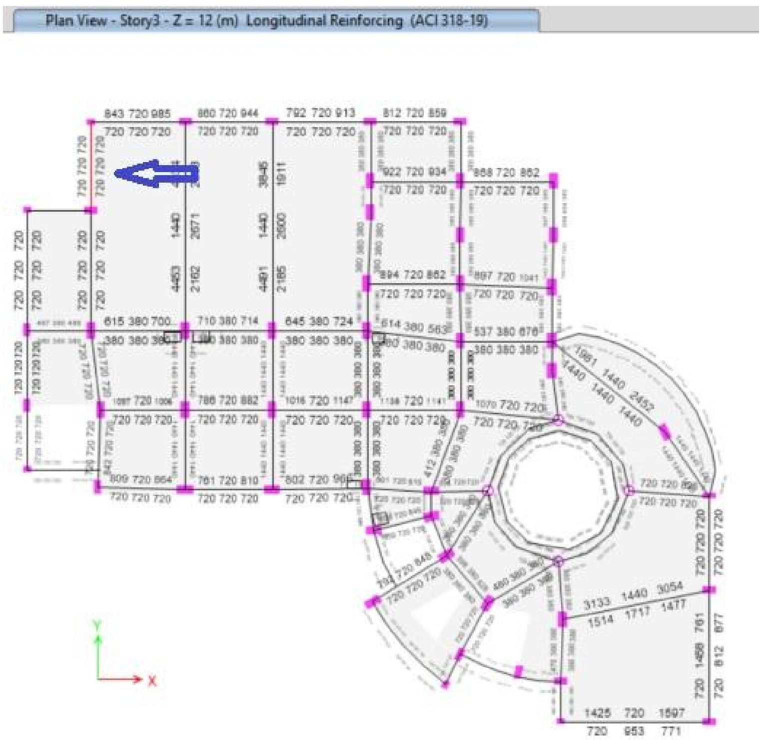


Figure3. 36 Beams that light up red in ETABS due to exceeds maximum reinforcement.

To solve this problem the beam dimension was increased from 400×600 to 800×600.

3.3.2 Check dimensions of columns:

This check is done by designing frames in the structure and then checking the rebar percentage within the limitation of maximum and minimum.

If the columns light up red so one of the three warning messages will appear:

1. Reinforcing required exceeds maximum allowed, to come over this problem, enlarge the section of column.

2. Column factored axial load exceeds Euler force, to come over this problem, enlarge the section of the column.

3. Strong column weak beam is not satisfied (this indicates that the framing type used is special), to bypass this problem, also enlarge the section of the column.

Some columns were highlighted with the red color during the design and the reasons explained in detail above in the beginning of this section.

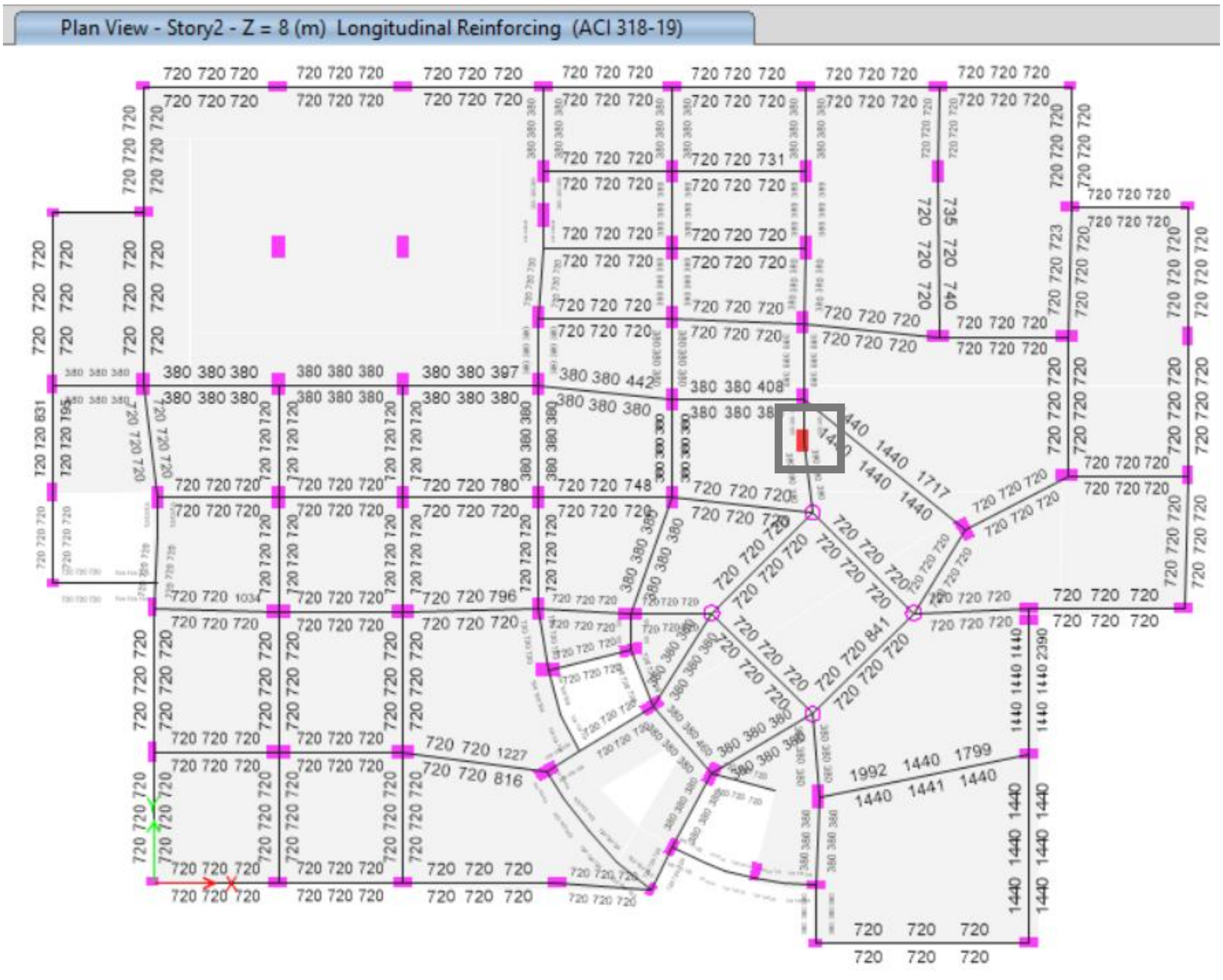


Figure3. 37 columns that light up red in ETABS.

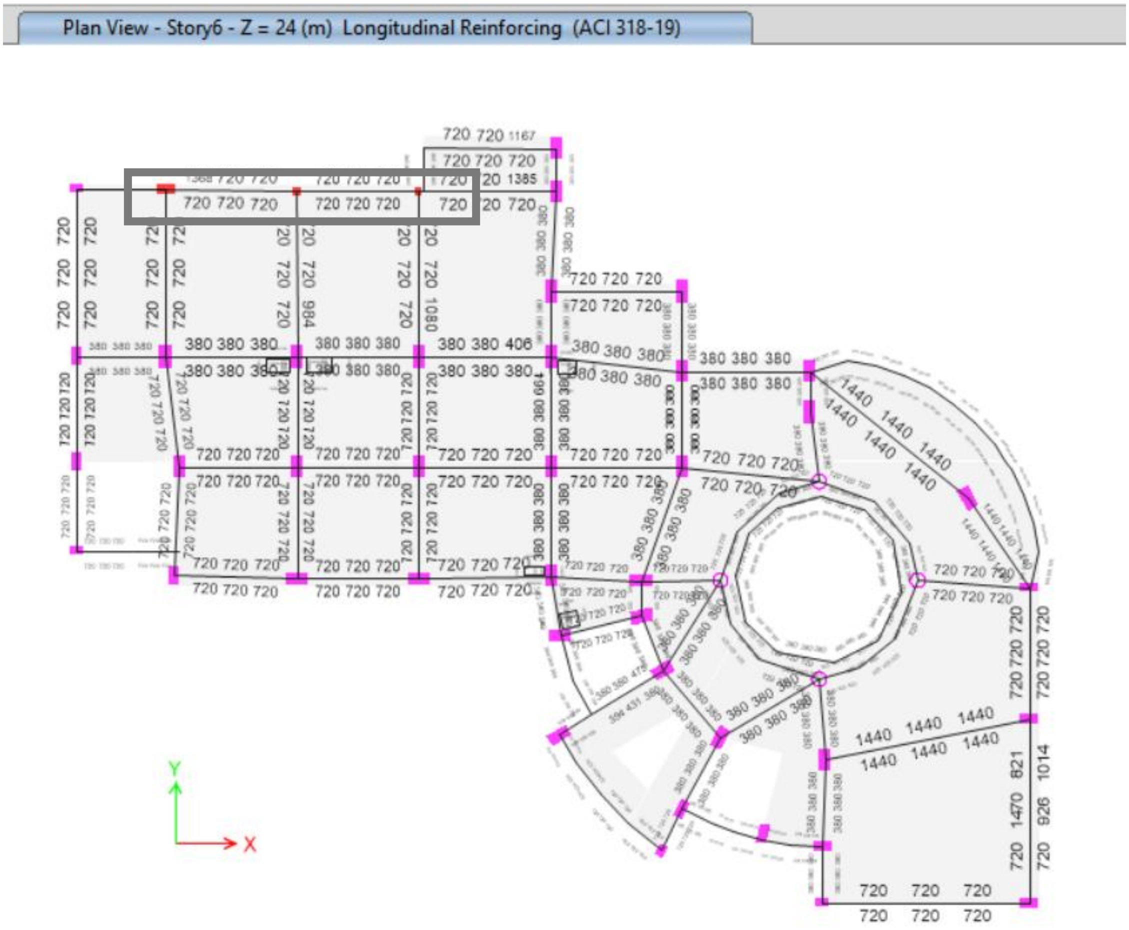


Figure3. 38columns that light up red in ETABS.

Table3. 1:Columns dimension.

Column Number	Type	Dimension
C9,C10,C11,C12,C15,C16,C17,C18,C19,C20,C21,C22,C25MC26,C27,C28C29, C32,C33,C34,C35,C36,C37,C40,C41,C42,C43,C44,C46,C47,C51,C52,C53,C54, C64,C65,C66,C69,C70,C71,C72,C73,C88,C89,C96,C98,C99,C102,C103, **C106,C107,C108**	R	500x 900
C2,C3,C4,C5,C6,C7,C14,C23,C30,C31,C39,C48,C50,C58,C59,C60,C61,C62, C68,C74,C75,C78,C81,C82,C83,C85,C86,C87,C90,C93,C94,C95,C97,C100, C101,C104,C109,C112,C114,C115.	R	400x 750
C1,C8,C13,C24,C49,C63,C79,C80,C84,C91,C92,C105,C110,C111,C113,C116.	R	350x 500
C38	R	600x900
C45,C55,C56,C67.	C	600

As a result of this failure, the dimensions of the columns were changed as follows:

3.3.3 Check Slab thickness:

By checking the deflection of the slab in section **3.5.1,** the slab thickness is adequate, and there is no need to increase the thickness.

Note:

Slab thickness in basement one and ground floor can be reduced to 250 mm, but to simplify hand calculation, it decided to take all slab thickness equal to 300 mm.

3.3.4 Evaluation of design:

No red frame member.

Figure3. 39 : verify all members in ETABS.

So, all dimensions are adequate and are based on codes standards.

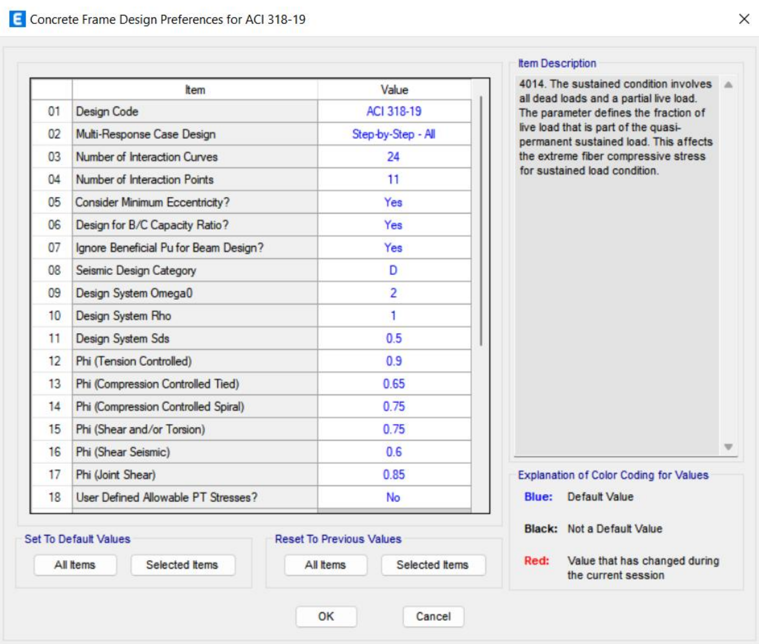


Figure3. 40 design preferences.

3.4 Verification of structural analysis:

3.4.1 Compatibility of the structure:

A general mesh of approximate maximum size 0.6 m was used for floors to insure compatibility.

All the elements in the model moves as a one group, this means that all the elements are connected to each other and there are no problems with modeling unit.

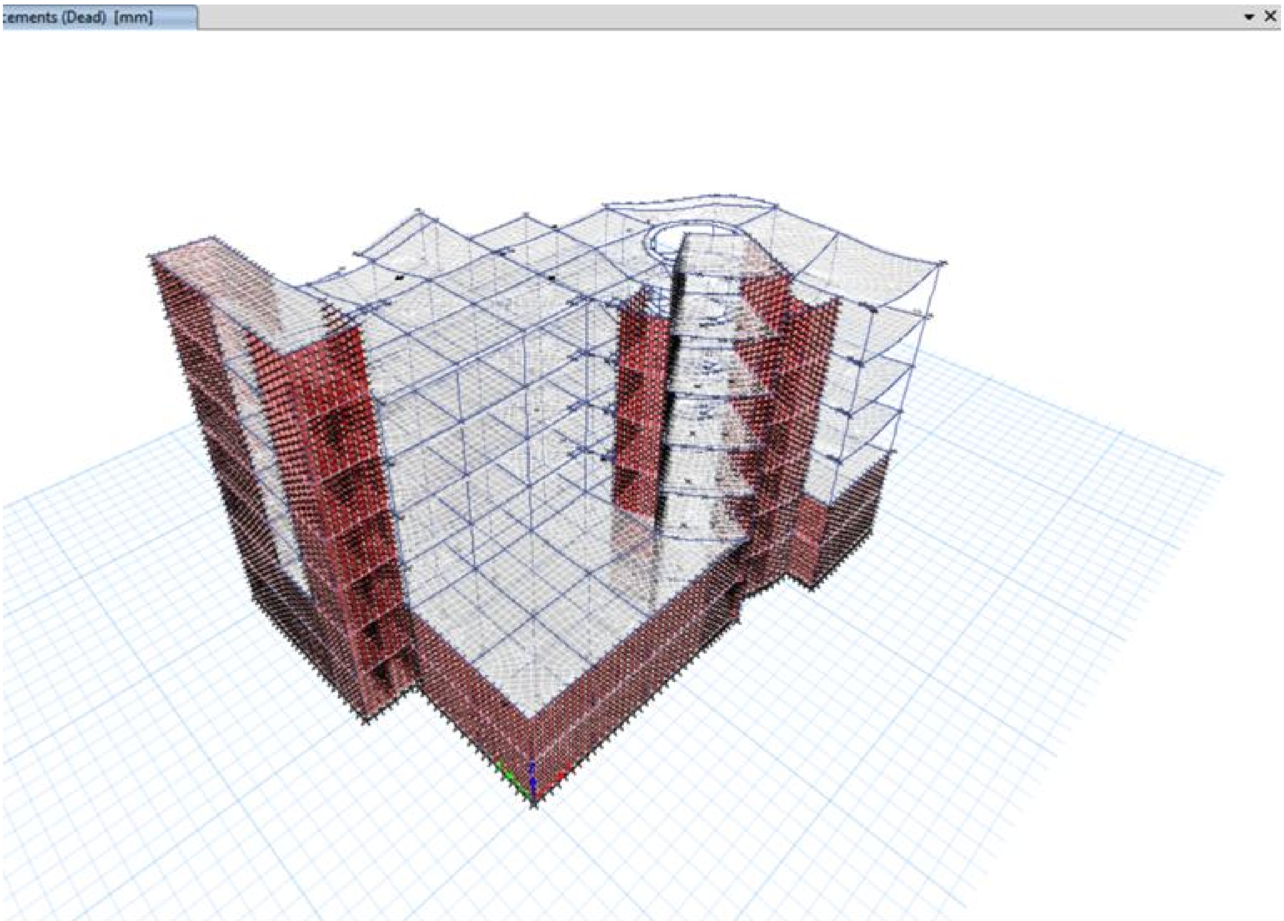


Figure3. 41 compatibility check

3.4.2 Gravity loads:

3.4.2.1 Check equilibrium for dead loads:

Calculations of own weight for structural elements (slab, beam, column, and wall) by hand:

Table3. 2: Slabs dead load.

Floor	Area (m^2)	Depth (m)	Total weight (KN)
Basement 2	1196.02	0.3	8970.15
Ramp	51.24	0.3	384.3
Basement 1	1250.45	0.3	9378.375
Ground floor, GF	791.8	0.3	5938.5
First Floor, F1	742.36	0.3	5567.7
Second Floor, F2	700.99	0.3	5257.425
Roof	642.72	0.3	4820.4
Stairs	77.19	0.3	578.925
Sum			40895.775

Table3. 3:Columns dead load.

Group	Depth(m)	width(m)	Area (m^2)	Total length (m)	Total weight (kN)
C350*500	0.35	0.5	0.175	28	122.5
C400*750	0.4	0.75	0.3	152	1140
C500*350	0.5	0.35	0.175	124	542.5
C500*900	0.5	0.9	0.45	632	7110
C750*400	0.75	0.4	0.3	228	1710
C900*500	0.9	0.5	0.45	184	2070
CR 600	Diameter =600mm		0.282	96	676.8
C600*900	0.6	0.9	0.54	8	108
C500*750	0.5	0.75	0.375	32	300
C400*300	0.4	0.3	0.12	8	24
Sum.					13803.8

Table3. 4:beam dead load.

Group	Depth(m)	Width(m)	Area(m^2)	Total length(m)	Weight modifiers	Total weight (kN)
B250*600	0.25	0.6	0.15	549.59	0.5	1030.48
B600*400	0.6	0.4	0.24	1948.941	0.5	5846.823
B600*800	0.6	0.8	0.48	214.981	0.5	1289.886
B glass	0	0.6	0	181.3677	0	0
Sum.						8167.189

Table3. 5:wall dead load.

Wall section	Area (m^2)	Depth (m)	Total weight (kN)
Wall 200	1687.28	0.2	8036.4
Wall 300	1632.5	0.3	12243.75
Sum.			20280.15

shear wall dead load.

Total dead load = 83146.914

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
Dead	LinStatic			0	0	83501.588	1374519.9709	-1325106	0

Figure3. 42 Total dead load from ETABS

Dead load from ETABS = 83501.588 KN

Difference percentage = $\frac{83146.914-83501.588}{83146.94} = 0.43\% < 5\%$ OK

3.4.2.2 Check equilibrium for superimposed dead loads:

Calculation By hand:

Table3. 6 :Superimposed dead load, SD.

Floor	Area(m^2)	SD(KN/m^2)	Total weight (KN)
Basement 1	1,150.55	4.5	5,177.475
Ground Floor, GF	1,113.1	7	7,791.7
First Floor, F1	747.33	7.5	5,674.17
Second Floor, F2	694.75	7.5	5,210.625
Third Floor, F3	657.69	7	4,603.83
Stair	271.86	4	1,087.44
Sum.			29,545.24

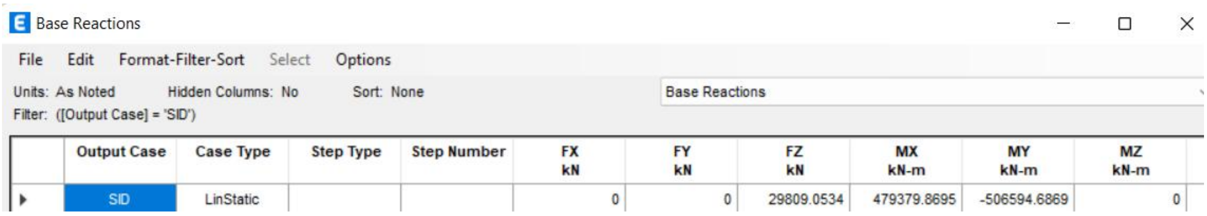


Figure3. 43 Total superimposed dead load from ETABS.

Difference percentage= $\frac{29{,}545.24 - 29809}{29{,}545.24} \times 100\%$

$= 0.9\ \% \quad < 5\ \%$…………OK.

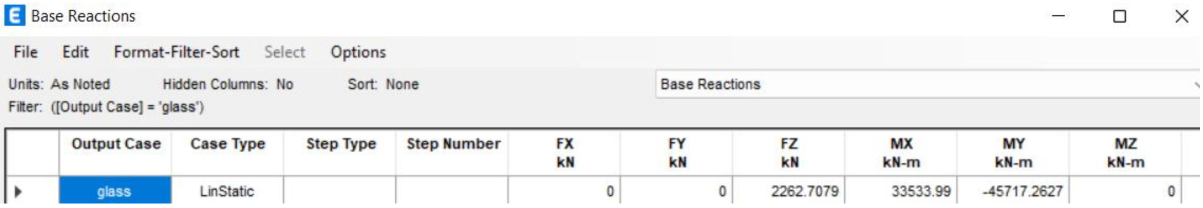
- **Check equilibrium for glass wall loads:**

Calculations of glass wall by hand:

Table3. 7: Glass wall load.

Floor	Total weight (KN)
Basement 1	0
Ground Floor, GF	0
First Floor, F1	728.21
Second Floor, F2	729.36
Third Floor, F3	671.61
Roof	115.57
Sum.	2244.75

By ETABS:



Base Reactions

File | Edit | Format-Filter-Sort | Select | Options

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

Filter: ([Output Case] = 'glass')

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
glass	LinStatic			0	0	2262.7079	33533.99	-45717.2627	0

Figure3. 44 Total glass wall load from ETABS.

Difference percentage= $\frac{2244.75 - 2262.7}{2244.75} \times 100\%$

$= 0.8\% \quad < 5\%$…………OK

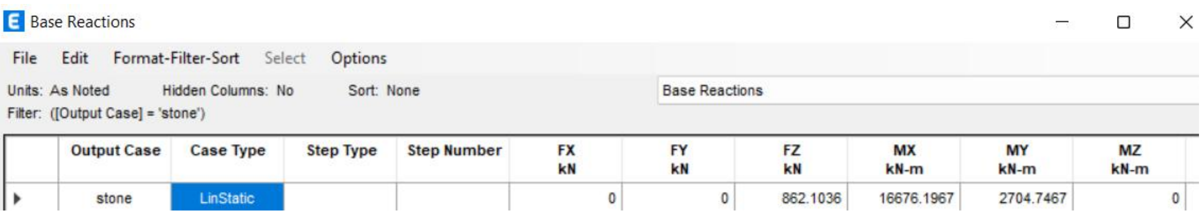
- **Check equilibrium for stone loads :**

Calculations of stone wall by hand:

Table3. 8: Stone walls load.

Floor	Wall Area(m^2)	Total weight (KN)
Basement 1	72.62	0
Ground Floor, GF	72.62	181.55
First Floor, F1	72.62	181.55
Second Floor, F2	72.62	181.55
Third Floor, F3	72.62	181.55
Roof	54.51	136.275
Sum.		862.475

Stone load by hand = 862.475 KN.



Base Reactions

File | Edit | Format-Filter-Sort | Select | Options

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

Filter: ([Output Case] = 'stone')

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
stone	LinStatic			0	0	862.1036	16676.1967	2704.7467	0

Figure3. 45 Total Stone load from ETABS

Stone load by ETABS = 862.1 KN

Difference percentage= $\frac{862.475 - 862.1}{862.475} \times 100\%$

$= 0.04\% \quad < 5\%$…………OK.

3.4.2.3 Check equilibrium for live loads:

Calculations of live loads by hand:

Table3. 9 :Live load calculation, LL.

Floor	Area(m^2)	LL(KN/m^2)	Total weight (KN)
Basement 1	367	12	4404
	783.56	3	2520.46
Ground Floor, GF	1,113.10	5	5565.5
First Floor, F1	747.33	5	3736.65
Second Floor, F2	127.79	4	511.16
	566.95	2	1133.62
Third Floor, F3	462.86	5	2314.3
	194.83	3	584.49
Stair	271.86	5	1359.3
Sum.			22129.5

The total manual calculated live load (live and roof) of the structure equals 22129.5 KN.

Base Reactions

File Edit Format-Filter-Sort Select Options

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

Filter: ([Output Case] = 'Live')

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
Live	LinStatic			0	0	22044.2746	384084.4123	-382029.4185	0

Figure3. 46 :Live load from ETABS.

Live load from ETABS:

Difference percentage= $\frac{22129.5 - 22044.27}{22129.5} \times 100\%$

$= 0.4 \;< 5\%$…………OK

3.4.2.4 Check equilibrium for snow loads:

Table 3.11 shows the manual calculations of snow load.

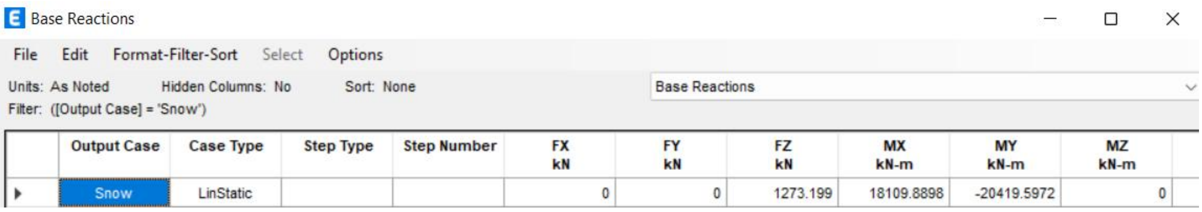
Calculations of snow load by hand:

Table3. 10: Snow load.

Floor	Area (m^2)	LL (KN\m^2)	Total weight (KN)
Third floor, F3 ROOF	584.12	1.92	1121.51
Level 27	80.95	1.92	155.425
Sum	6		1276.934

The total manual calculated snow load of the structure equals 1276.934 KN.

While the total snow load from the program equals 1277 KN.


Base Reactions
File Edit Format-Filter-Sort Select Options
Units: As Noted Hidden Columns: No Sort: None
Filter: ([Output Case] = 'Snow')
Base Reactions

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
Snow	LinStatic			0	0	1273.199	18109.8898	-20419.5972	0

Figure3. 47 : Snow load from ETABS.

Difference percentage= $\frac{1276.934 - 1277}{1276.934} \times 100\%$

$= \text{zero} < 5\%$…………OK

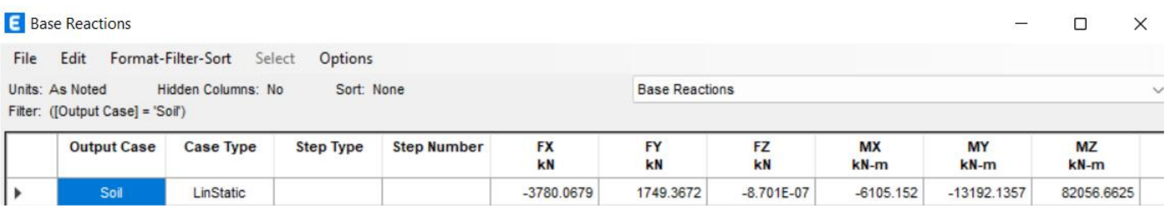
3.4.3 Check equilibrium for soil loads:

Calculations of soil load by hand:

Table3. 11: soil load.

Direction	X_Dir.	Y_Dir.	Z_Dir.
Sum. of load (KN)	3700	1750	0

By ETABS:


Base Reactions
File Edit Format-Filter-Sort Select Options
Units: As Noted Hidden Columns: No Sort: None
Filter: ([Output Case] = 'Soil')
Base Reactions

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
Soil	LinStatic			-3780.0679	1749.3672	-8.701E-07	-6105.152	-13192.1357	82056.6625

Figure3. 48 :Total soil load from ETABS.

Difference percentage(X_Dir.) $= \frac{3700-3780}{3700} \times 100\%$

$= 2.16\% \quad < 5\%$............OK.

Difference percentage(Y_Dir.) $= \frac{1750-1749.4}{1750} \times 100\%$

$= 0.06\% \quad < 5\%$............OK.

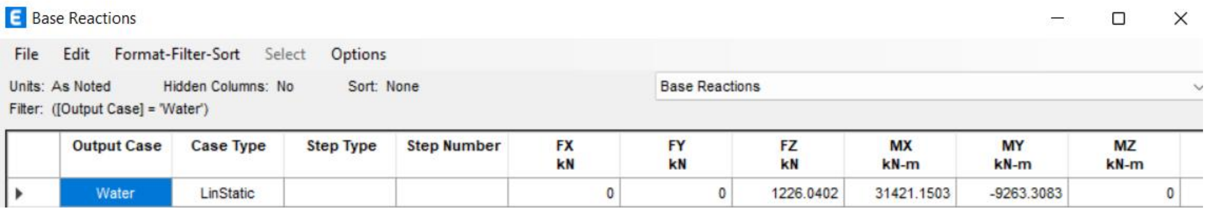
3.4.4 Check equilibrium for water loads:

Calculations of water load by hand:

Table3. 12: water load.

Direction	X_Dir.	Y_Dir.	Z_Dir.
Sum. of load (KN)	0	0	1226.03

By ETABS:


Base Reactions
File Edit Format-Filter-Sort Select Options
Units: As Noted Hidden Columns: No Sort: None
Filter: ([Output Case] = 'Water')
Base Reactions

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
Water	LinStatic			0	0	1226.0402	31421.1503	-9263.3083	0

Figure3. 49 :Total water load from ETABS.

Difference percentage= $\frac{1226.03 - 1226.0301}{1226.03} \times 100\%$

$=$ zero ....OK.

3.4.5 Seismic forces:

3.4.5.1 Analysis method:

The structural analysis required consist of one of the types permitted in Table 12.6-1, based on the structures seismic design category, structural system, dynamic properties, and regularity.

Table 12.6-1 Permitted Analytical Procedures

Seismic Design Category	Structural Characteristics	Equivalent Lateral Force Procedure, Section 12.8[a]	Modal Response Spectrum Analysis, Section 12.9.1, or Linear Response History Analysis, Section 12.9.2[a]	Nonlinear Response History Procedures, Chapter 16[a]
B, C	All structures	P	P	P
D, E, F	Risk Category I or II buildings not exceeding two stories above the base	P	P	P
	Structures of light-frame construction	P	P	P
	Structures with no structural irregularities and not exceeding 160 ft (48.8 m) in structural height	P	P	P
	Structures exceeding 160 ft (48.8 m) in structural height with no structural irregularities and with $T < 3.5T_s$	P	P	P
	Structures not exceeding 160 ft (48.8 m) in structural height and having only horizontal irregularities of Type 2, 3, 4, or 5 in Table 12.3-1 or vertical irregularities of Type 4, 5a, or 5b in Table 12.3-2	P	P	P
	All other structures	NP	P	P

[a]P: Permitted; NP: Not Permitted; $T_s = S_{D1}/S_{DS}$.

Figure3. 50 Permitted analytical procedures, ASCE7-10.

The Permitted analytical procedures is response spectrum and seismic response history.

The analysis method used in this project is response spectrum analysis using equivalent lateral force method to base shear check.

3.4.5.2 Check equilibrium: base shear:

The values of dynamic earthquakes forces must be approximately equal to the value of earthquakes forces in the static method. However, the values were not equal, which led to change the scale factor for each of the dynamic earthquake forces in X and Y.

Modification factor in X direction = 1.39

Modification factor in Y direction = 1.78

The figures below show the scale factor for earthquake loads (equivalent and dynamic) before and after changes.

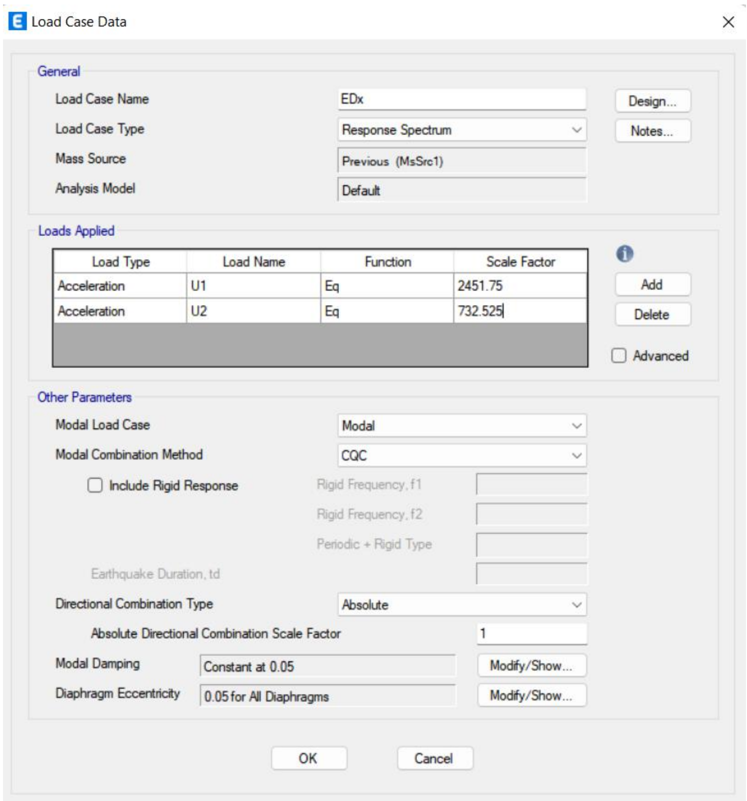


Figure3. 51 Definition of EDx before modification in ETABS.

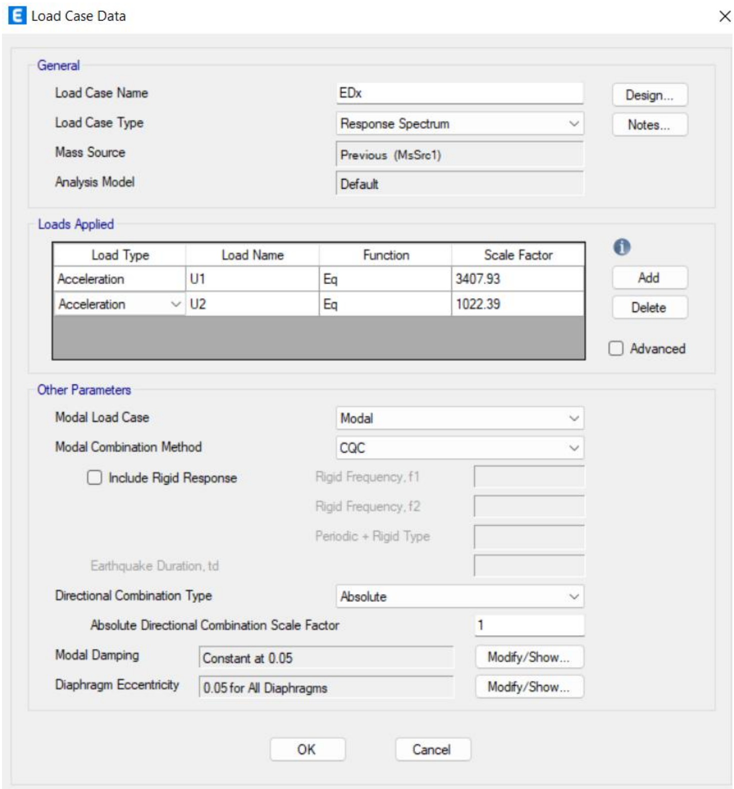


Figure3. 52 Definition of EDx after modification in ETABS.

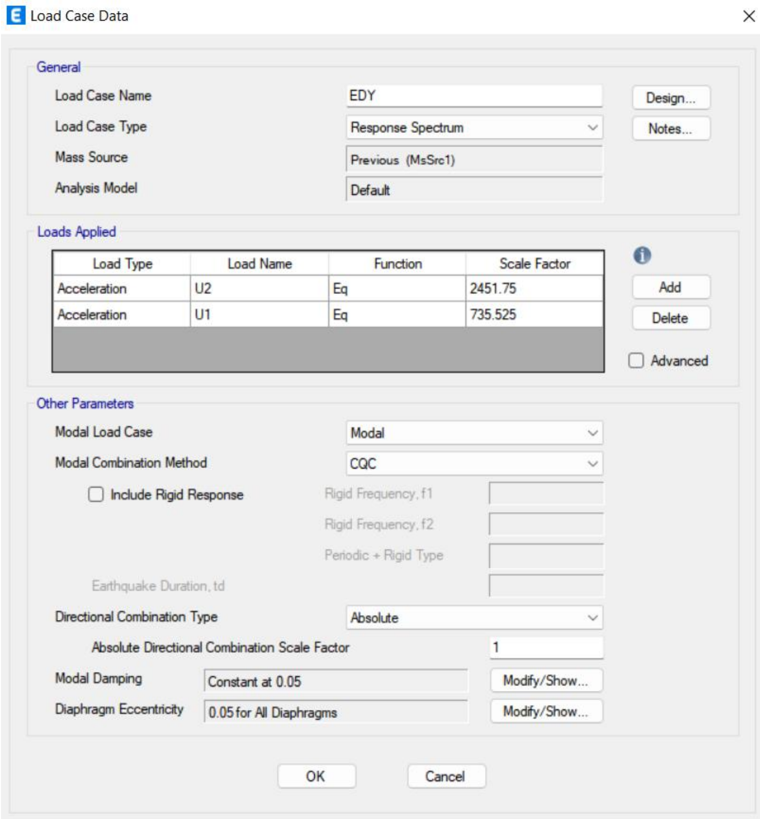


Figure3. 53 Definition of EDy before modification in ETABS.

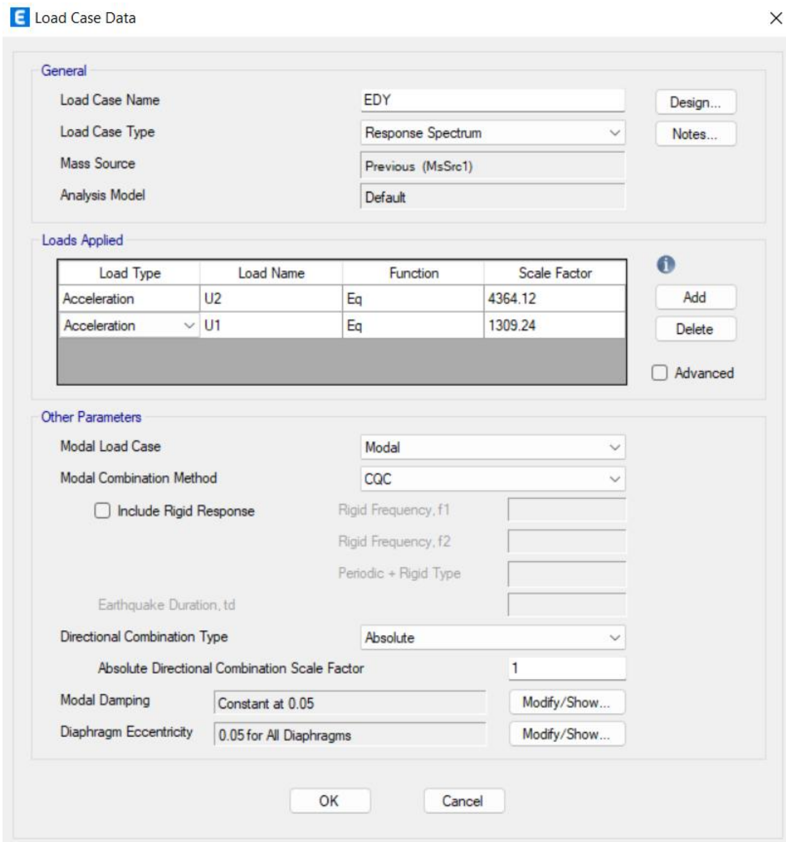


Figure3. 54 Definition of EDy after modification in ETABS.

Refer to section **1.6** the base shear can defined as:

V = 0.101 W

Not: the value of Cs changed in Y direction from 0.101 to 0.2045 in Y direction due to changed period in Y direction.

Tx = 0.637 sec.

Ty = 0.284 sec.

Base Reactions

Units: As Noted Hidden Columns: No Sort: None

Filter: ([Output Case] = 'Base_shaer')

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
Base_shaer	Combination			0	0	115819.8083	1910109.0659	-1863825	5.358E-07

Figure3. 55:Total loads from Base shear combination.

From ETABS Fz = 115819.8 KN

$V_x = 0.101 \times 115819.8 = 11{,}697.8$ KN

$V_y = 0.2045 \times 115819.8 = 23{,}685$ KN

Check base shear from equivalent force method:

Base Reactions

Units: As Noted Hidden Columns: No Sort: None

Filter: ([Output Case] = 'Qx' OR [Output Case] = 'Qy')

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
Qx	LinStatic	Step By Step	1	-11216.4384	0	-4.705E-05	-0.0009	-173892.8643	181983.216
Qx	LinStatic	Step By Step	2	-11216.4384	0	-4.782E-05	-0.0009	-173892.8643	199970.3993
Qx	LinStatic	Step By Step	3	-11216.4384	0	-4.628E-05	-0.0009	-173892.8644	163996.0327
Qy	LinStatic	Step By Step	1	0	-21681.5106	1.871E-05	333659.7262	-0.0005	-326913.4429
Qy	LinStatic	Step By Step	2	0	-21681.5106	2.04E-05	333659.7263	-0.0006	-368337.1198
Qy	LinStatic	Step By Step	3	0	-21681.5106	1.702E-05	333659.7262	-0.0005	-285489.766

Figure3. 56:Base reaction.

From ETABS F_x due to E_{QX} =11216.44 KN

$$\text{Difference percentage} = \frac{11{,}697.8 - 11216.44}{11{,}697.8} \times 100\%$$

$$= 4.11\% < 10\% \rightarrow \text{Ok.}$$

From ETABS F_Y due to E_{QY} =21681.5 KN

$$\text{Difference percentage} = \frac{23{,}685 - 21681.5}{23{,}685} \times 100\%$$

$$= 8.45\% < 10\% \rightarrow \text{Ok.}$$

Check base shear from response spectrum method:

Base Reactions

File Edit Format-Filter-Sort Select Options

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

Filter: ([Output Case] = 'EDx' OR [Output Case] = 'EDY')

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
EDx	LinRespSpec	Max		11254.9781	5968.5815	4.108E-05	90323.9165	134537.9866	251198.3442
EDY	LinRespSpec	Max		5449.7693	21724.2408	3.362E-05	350058.2079	61525.9125	400912.329

Figure3. 57:Base reactions in ETABS.

From ETABS F_x due to E_{DX} = 11255 KN

$$\text{Difference percentage} = \frac{11{,}697.8 - 11255}{11{,}697.8} \times 100\%$$

$$= 3.78\ \% < 10\% \rightarrow \text{Ok.}$$

From ETABS F_Y due to E_{DY} =21724.24 kN

$$\text{Difference percentage} = \frac{23{,}685 - 21724.24}{23{,}685} \times 100\%$$

$$= 8.28\ \% < 10\% \rightarrow \text{Ok.}$$

3.4.5.3 Story forces:

The lateral seismic force (kN) induced 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 that have a period of 0.5 s or less, k = 1.

for structures that have a period of 2.5 s or more, k = 2.

and for structures that have a period between 0.5 and 2.5 s, k shall be 2 or shall be determined by linear interpolation between 1 and 2.

K_x= 1.04

K_y= 1

By hand calculation:

In X direction:

V_x = 12384.4 KN

Kx = 1.04

Table3. 13:Story forces in X direction.

Story	K	H(m)	Weight (KN)	$W \times H^k$	C_{Vx}	Qx(KN)	F(X) Force (KN)
story 7	1.04	28	2097.34	67098.56	0.040789	11892.0479	505.14
story 6	1.04	24	10173.3	277255	0.168544	11892.0479	2087.31
story 5	1.04	20	16268.77	366799.6	0.222978	11892.0479	2761.448
story 4	1.04	16	17124.2	306121.5	0.186092	11892.0479	2304.637
story 3	1.04	12	18743.4	248425.2	0.151018	11892.0479	1870.26
story 2	1.04	8	30100.9	261693.6	0.159084	11892.0479	1970.16
story 1	1.04	4	28109.8	118850.26	0.07219	11892.0479	894.03
sum				1646243.7	1		12392.98

From ETABS:

Table3. 14:Story forces in X direction form ETABS compared to manual values.

story	output case	Vx (KN)	F(X)(KN)	F(X) Force (KN)
Story 7	Qx	332.77	332.778	505.14
Story 6	Qx	2355.27	2022.5	2087.31
Story 5	Qx	5169.54	2814.27	2761.448
Story 4	Qx	7516.61	2347.07	2304.637
Story 3	Qx	9430.08	1913.47	1870.26
Story 2	Qx	11281.87	1851.79	1970.16
Story 1	Qx	12224.77	942.9	894.03

By hand calculation:

In Y direction:

VY = 24173.66 KN

Ky = 1

Table3. 15: Story forces in Y direction.

story	K	H	Weight (KN)	$W \times H^k$	Cvx	Qy(KN)	F(Y) force (KN)
story 7	1	28	2097.34	58725.52	0.03969974	23015.945	937.287
story 6	1	24	10173.25	244158	0.16505616	23015.9451	3965.445
story 5	1	20	16268.88	325377.6	0.21996239	23015.9451	5284.85
story 4	1	16	17124.15	273986.4	0.18522081	23015.9451	4451.39
story 3	1	12	18743.35	224920.1	0.15205094	23015.9451	3674.30
story 2	1	8	30100.9	240807.2	0.16279095	23015.9451	3845.28
story 1	1	4	28109.8	112439.2	0.07521901	23015.9451	1807.28
sum				1480414	1		24033

From ETABS:

Table3. 16: Story forces in Y direction form ETABS compared to manual values.

story	output case	Vy(KN)	Fy(KN)	F(Y) force (KN)
story 7	Qy	626.97	626.977	937.287
story 6	Qy	4455.47	3828.5	3965.445
story 5	Qy	9823.29	5367.82	5284.85
story 4	Qy	14336.57	4513.28	4451.39
story 3	Qy	18054.97	3718.4	3653.1
story 2	Qy	27107.07	3652.1	3845.28
story 1	Qy	23614.47	1907.4	1807.28

3.4.5.4 Period:

Refer to section 1.6.7, T_{limit} = Ta Cu = 0.573 sec.

From ETABS:

Table3. 17:Time period from ETABS model analysis.

Case	Mode	Period sec	UX	UY
Modal	1	0.649	0.4612	0
Modal	2	0.325	0.0042	0.0607
Modal	3	0.284	0.0003	0.4841

Tx = 0.659 sec.

Ty = 0.284 sec.

Tx > Ta Cu, so T_a C_u was used in Qx definition.

Ty < Ta Cu, period check OK.

3.4.5.5 Type of lateral force structural system

To determine the load resistance system, a horizontal load must be assigned in a certain amount on all floors. The reaction was noted and the percentage was calculated and compared. Table 3.18 shows the typed of lateral forces resisting systems used in the structure.

Table3. 18:Type of lateral force structural system.

Percentage of Loads Resisting by Walls	Type
Less than 25%	Moment Resisting Frame System
25% to 75%	Dual System
More than 75%	Shear wall Resisting System

To find the resisting system, applied a lateral force in X and Y direction and determine the portion of the force that resisted by the shear wall.

For X Dir.

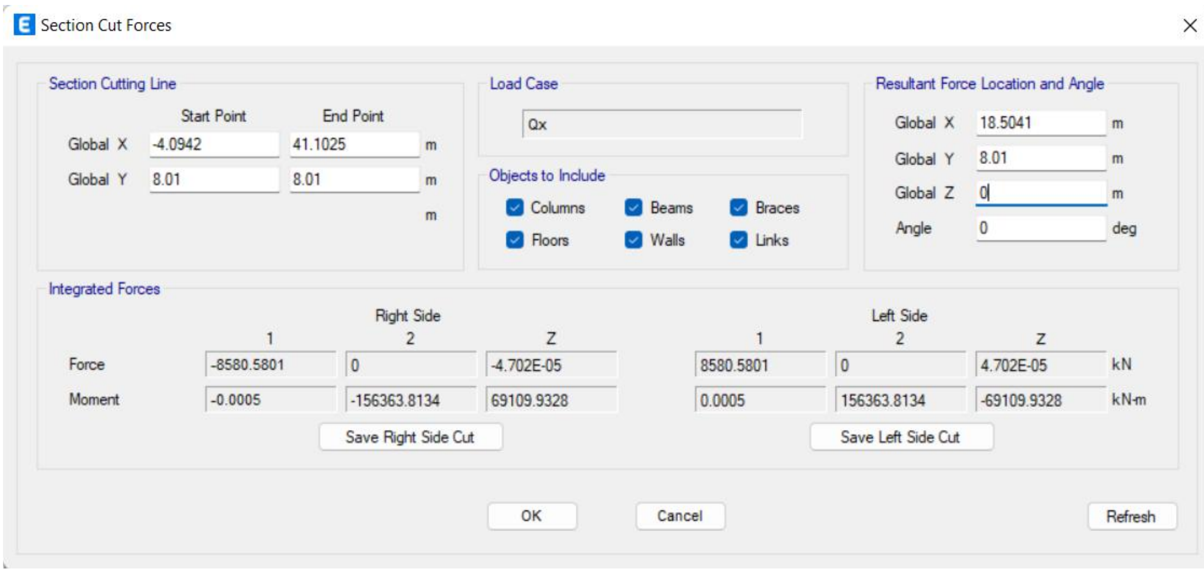


Figure3. 58 Total vertical reactions in X direction

Total Reactions =8580.58 KN

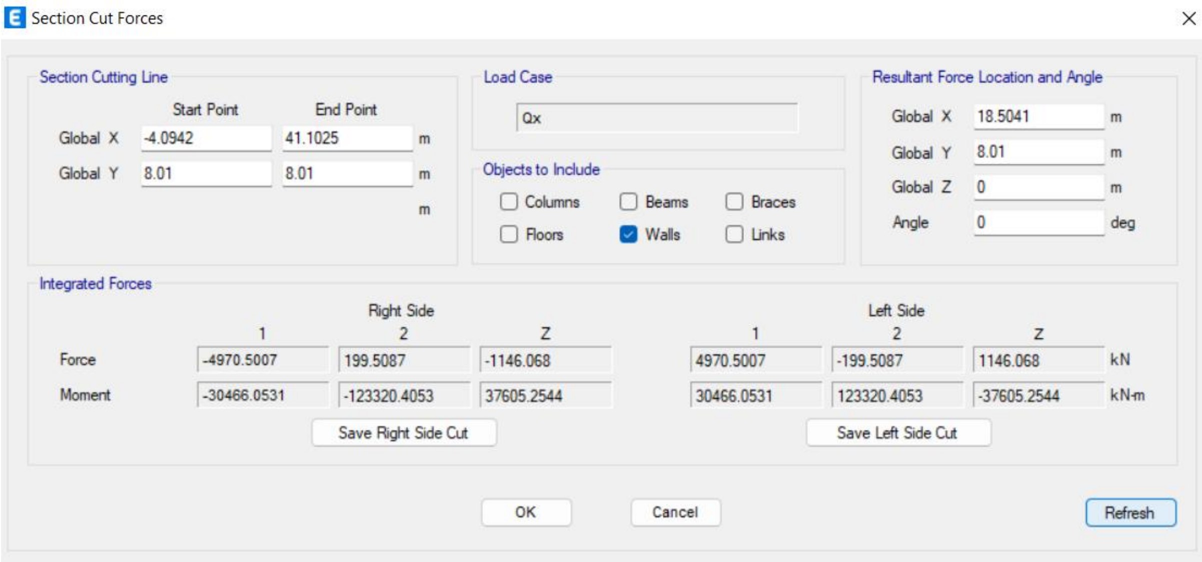


Figure3. 59 Walls vertical reactions in X direction

Wall Reactions =4970.5 KN

Percentage of load resisting by walls = (4970.5 /8580.58) *100 % = 58 %

Dual System in the X direction

For Y Dir.

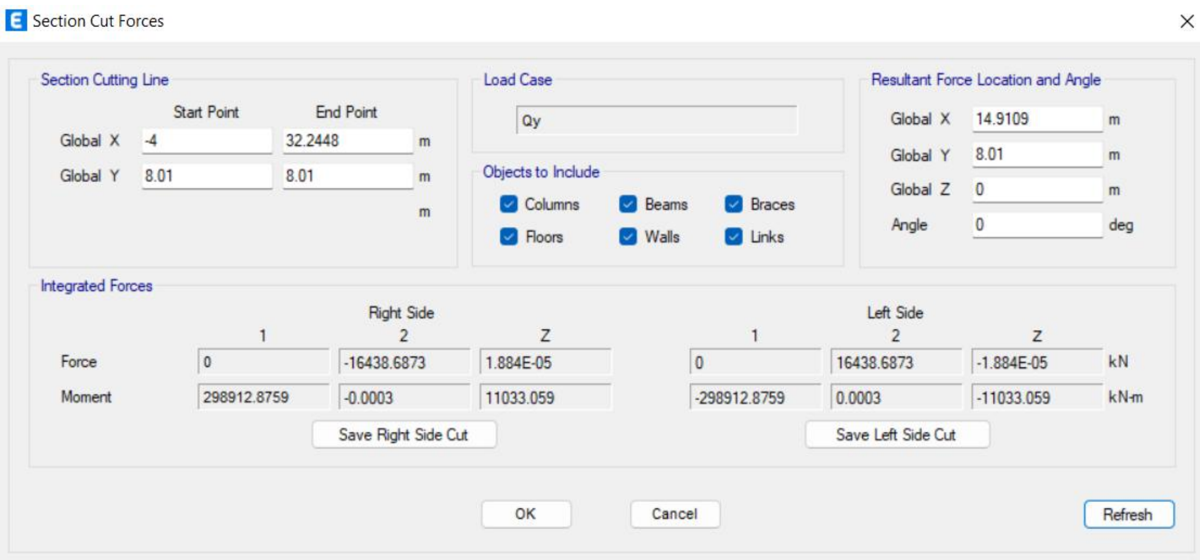


Figure3. 60 Total vertical reactions in Y direction

Total Reactions = 16438.7 KN

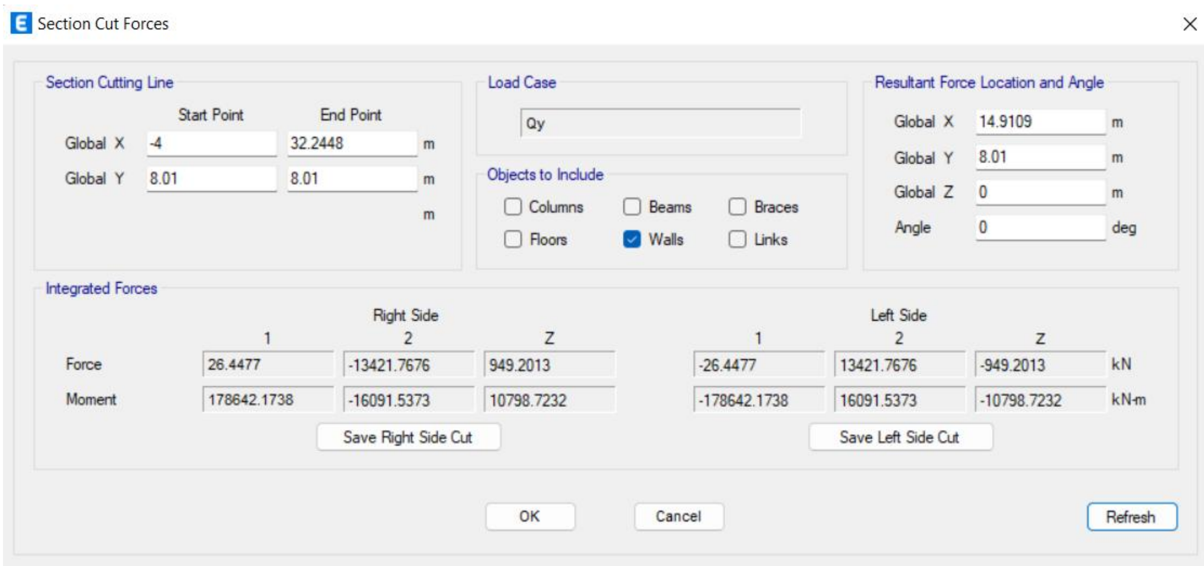


Figure3. 61 Walls vertical reactions in Y direction

Wall Reactions = 13421.76 KN

Percentage of load resisting by walls = (13421.76/16438.7) *100 % = 81.65 %

Shear wall Resisting System in the Y direction.

3.4.5.6 Mass participation ratio for modal analysis:

The effective mass participation factor represents the percentage of the system mass that participates in a particular mode. It provides a measure of the energy contained within each resonant mode. A mode with a large mass participation factor is usually a significant contributor to the dynamic response of a system.

Table3. 19:Mass participation ratio.

Case	Mode	SumUX	SumUY	SumRZ
Modal	1	0.4612	0	0.0046
Modal	2	0.4654	0.0607	0.3476
Modal	3	0.4657	0.5348	0.4192
Modal	4	0.5683	0.5348	0.4242
Modal	5	0.5765	0.5372	0.4252
Modal	6	0.5765	0.5374	0.4261
Modal	7	0.6039	0.5409	0.4296
Modal	8	0.6042	0.5578	0.4369
Modal	9	0.6045	0.625	0.46
Modal	10	0.6067	0.6278	0.5085

Modal	11	0.6366	0.63	0.5253
Modal	12	0.6408	0.7917	0.5299
Modal	13	0.6409	0.8011	0.5318
Modal	14	0.6685	0.8038	0.556
Modal	15	0.6835	0.8108	0.5566
Modal	16	0.8989	0.8108	0.5681
Modal	17	0.899	0.811	0.5942
Modal	18	0.9174	0.8388	0.6188
Modal	19	0.9185	0.839	0.627
Modal	20	0.9188	0.8437	0.6497
Modal	21	0.921	0.8753	0.6606
Modal	22	0.921	0.8855	0.6612
Modal	23	0.921	0.8889	0.6661
Modal	24	0.9232	0.8966	0.6972
Modal	25	0.9245	0.9132	0.6997
Modal	26	0.9245	0.9144	0.7322
Modal	27	0.9263	0.926	0.7654
Modal	28	0.9272	0.9329	0.8322
Modal	29	0.9324	0.933	0.8668
Modal	30	0.9324	0.9339	0.8876
Modal	31	0.957	0.9345	0.8876
Modal	32	0.9573	0.9348	0.8935
Modal	33	0.9616	0.9365	0.8971
Modal	34	0.962	0.9368	0.8972
Modal	35	0.962	0.938	0.9127

The results:

1. The structure needed 18 modes in X-direction to collect mass participation ratio more than 90%.

2. The structure needed 25 modes in Y-direction to collect mass participation ratio

more than 90%.

3. The structure needed 35 modes in RZ-direction to collect mass participation ratio

more than 90%.

3.4.6 Structural irregularity:

3.4.6.1 Horizontal irregularity:

There are two types of structural irregularities, horizontal and vertical irregularities. The existence of irregularity can affect the forces upon which the structure is exposed to. Therefore, it is important to have these irregularities checked, taking into consideration the seismic design category, project seismic design category is D, which determine what type will be applicable to be checked. The intent of this process is to see the effect on the structure in order to achieve and ensure safety and serviceability of the structure.

Horizontal irregularities are shown in Table3. 20.

Type	Horizontal irregularities description	Commentary
1	Torsional Irregularity: Torsional irregularity, defined to exist where either: • More than 75% of any story's lateral strength below the diaphragm is provided at or on one side of the center of mass, or • The Torsional Irregularity Ratio (TIR) exceeds 1.2.	Exists, the ratio exceeded 1.2
2	Reentrant Corner Irregularity: Reentrant corner irregularity, defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 20% of the plan dimension of the structure in the given direction.	Dose not exist.
3	Diaphragm Discontinuity Irregularity: Diaphragm discontinuity irregularity, defined to exist where there is a diaphragm with an abrupt discontinuity or variation in stiffness, including one that has a cut-out or open area greater than 25% of the gross enclosed diaphragm area, or a change in effective diaphragm stiffness of more than 50% from one story to the next.	Exists
4	Out-of-Plane Offset Irregularity: Out-of-plane offset irregularity, 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.	Dose not exist.
5	Non-parallel System Irregularity: Non-parallel system irregularity, defined to exist where vertical lateral force-resisting elements are not parallel to the major orthogonal axes of the seismic force-resisting system	Exists

Torsional Irregularity:

Torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with Ax= 1.0, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure, as shown in **Figure 3.62.**

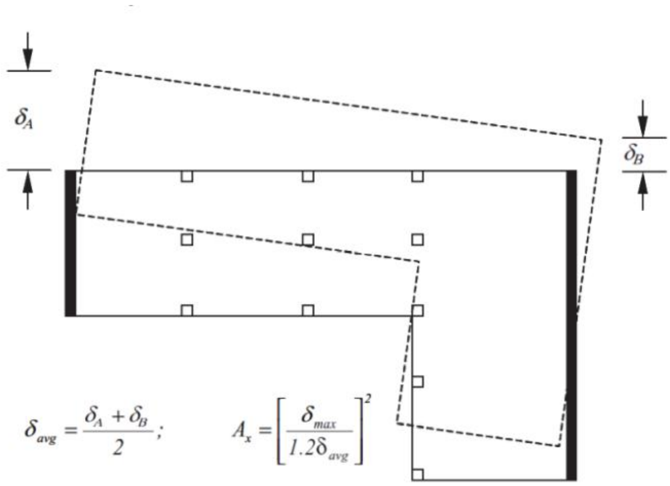


Figure3. 62: Torsional irregularity.

Where:

δ_{max} = maximum displacement at level x computed assuming. Ax = 1.

δ_{avg} = average of the displacements at the extreme points of the structure at level x computed.

The torsional amplification factor Ax shall not be less than 1 and is not required to exceed 3.0.

Table3. 21: Torsion irregularity in X-direction.

Story	Output Case	Max Drift	Avg Drift	Ratio	Ax	Result
Stair	Qx	0.001206	0.001138	1.06	0.78	Regular
Stair	Qx	0.000815	0.000813	1.003	0.70	Regular
Roof	Qx	0.001238	0.001108	1.117	0.87	Regular
F3	Qx	0.001571	0.001459	1.077	0.81	Regular
F2	Qx	0.001832	0.00166	1.104	0.85	Regular
F1	Qx	0.001731	0.001365	1.268	1.12	Regular
Gf	Qx	0.000447	0.000238	1.881	2.46	Irregular
B1	Qx	0.000113	7.70E-05	1.469	1.50	Irregular

The Eccentricity will be normalized to the largest value of A_x.

Eccentricity = 0.05 × A_x.

$= 0.05 \times 2.46 = 0.123$

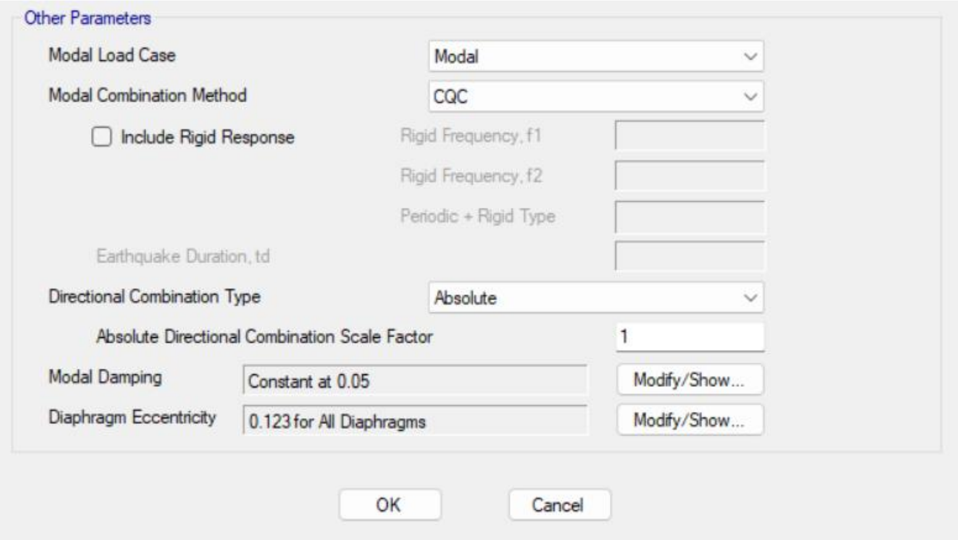


Figure3. 63 Eccentricities in X direction for response spectrum analysis

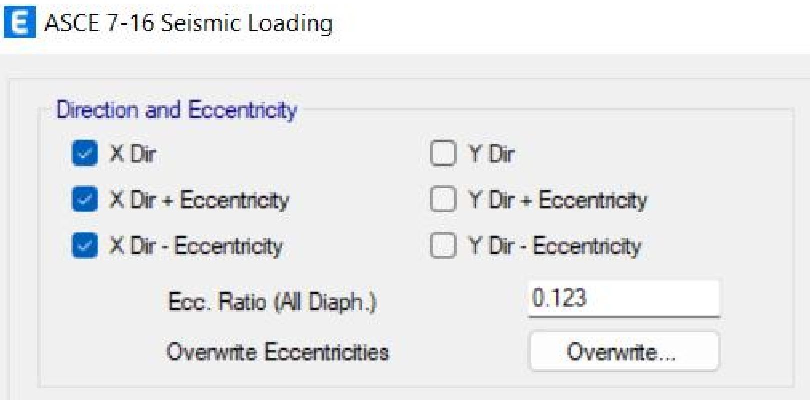


Figure3. 64 :Eccentricities in X direction for equivalent static analysis.

In Y direction:

Table3. 22: Torsion irregularity in Y-direction.

Story	Output Case	Max Drift	Avg Drift	Ratio	Ax	Result
Stair	Qy	0.000464	0.000447	1.04	0.75	Regular
Stair	Qy	0.000933	0.000768	1.215	1.03	Regular
Roof	Qy	0.000645	0.000553	1.166	0.94	Regular
F3	Qy	0.000644	0.000567	1.134	0.89	Regular
F2	Qy	0.000614	0.000523	1.174	0.96	Regular
F1	Qy	0.000555	0.000426	1.301	1.18	Regular
Gf	Qy	0.000265	0.000162	1.632	1.85	Irregular
B1	Qy	0.000151	0.000109	1.388	1.34	Irregular

The Eccentricity will be normalized to the largest value of A_x.

Eccentricity = $0.05 \times A_x$.

$= 0.05 \times 1.85$

$= 0.0925$

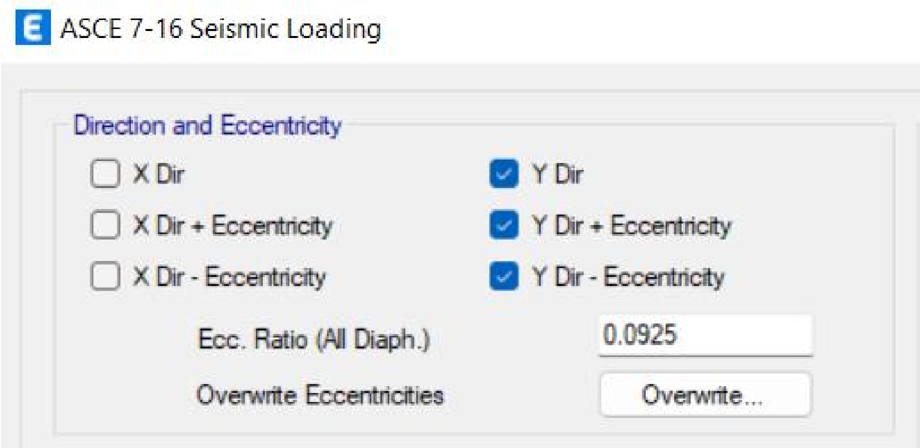


Figure3. 65 Eccentricities in Y direction for equivalent static analysis.

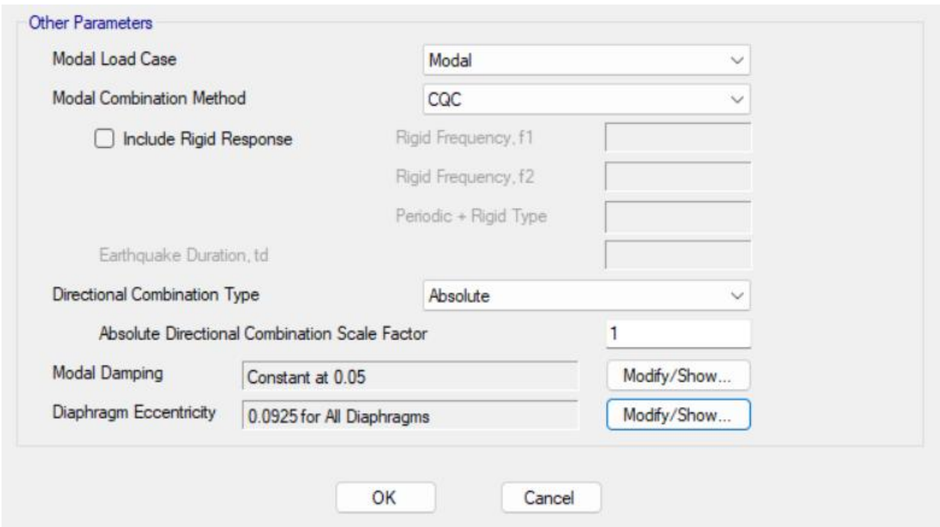


Figure3. 66: Eccentricities in Y direction for response spectrum analysis.

Diaphragm Discontinuity Irregularity :

Diaphragm discontinuity irregularity, defined to exist where there is a diaphragm with an abrupt discontinuity or variation in stiffness, including one that has a cut-out or open area greater than 25% of the gross enclosed diaphragm area, or a change in effective diaphragm stiffness of more than 50% from one story to the next.

In this building the open area is less than 25% of the gross diaphragm area but the chance in effective diaphragm stiffness more than 50% from story to the next.

To simplify the diaphragm stiffness calculation, it was assumed that there is an load effects on each diaphragm in the building in a global-X and Y directions, and the maximum resulting deformation values are read.

Sample calculation:

Assume load equals = 15 KN/m^2 in global-Y direction.

For B_1 floor the area = 1247.26 m^2.

And from ETABS deformation U_y = 0.464 mm

$$\text{Stiffness} = \frac{area \times \text{load}}{deformation} = \frac{1247.26 \times 15}{0.000464}$$

$$= 40320905.17$$

Table **3.23** shows effective diaphragm stiffness for each story in X and Y directions:

Table3. 23: effective diaphragm stiffness.

Story	**Stiffness in X KN/m**	**Stiffness in Y KN/m**
B_1	22166.943×10^3	40320.905×10^3
GF	22419.11×10^3	22166.943×10^3
F_1	2116.19×10^3	14575.13×10^3
F_2	1951.78×10^3	15387×10^3
F_3	1588.545×10^3	1744.3×10^3
Roof	1785.33×10^3	11615×10^3
Stairs	158.5×10^3	1564×10^3

As the table shows, there are changes in the effective stiffness values greater than 50%, then according of **ASCE-7-22** in section **12.3.3.5** the design forces shall be increased 25% at each diaphragm level where the irregularity occurs for the following elements of the seismic force-resisting system: Connections of diaphragms to vertical elements, and collectors and their connections, including connections to vertical elements of the seismic force-resisting system.

3.4.6.2 Vertical irregularity:

Table3. 24: Vertical irregularities description.

Type	Description	Commentary
1a.	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.	–
1b.	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.	Only applicable for Seismic design categories E and F.
2.	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.	–
3.	In-Plane Discontinuity in Vertical Lateral Force Resisting Element Irregularity is defined to exist where there is an in-plane offset of a vertical seismic force-resisting element resulting in overturning demands on supporting structural elements.	Does not exist
4a.	Discontinuity in Lateral Strength–Weak Story Irregularity is defined to exist where the story lateral strength is less than that in the story above. The story lateral strength is the total lateral strength of all seismic force-resisting system elements resisting the story shear for the direction under consideration.	Only applicable for Seismic design categories E and F.
4b.	Discontinuity in Lateral Strength–Extreme Weak Story Irregularity is defined to exist where the story lateral strength is less than 65% of that in the story above. The story lateral strength is the total lateral strength of all seismic force-resisting system elements resisting the story shear for the direction under consideration.	Does not exist

3.4.7 Effect of P-delta.

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 0.10:

$$\theta = \frac{P_x / h_{sx}}{V_x / \Delta_{xe}}$$

(ASCE 7-22 equation (12.8-8))

Where:

θ: Stability coefficient.

P_x: Total un-factored vertical design load at and above level x.

Δ_{xe}: Design story drift.

V_x: Seismic shear force.

h_x: story height below level x.

In X direction:

Table3. 25 :Effect of p-delta in X direction.

Story	P(KN)	Stiff X(kN/m)	hxs(m)	Theta=P/(h×S)
Stair	1034.2288	44889.638	3	0.008
Roof	11632.0099	365832.453	4	0.008
F3	29457.4112	719747.334	4	0.010
F2	47253.2329	1042661.336	4	0.011
F1	67868.9116	2318134.797	4	0.007
Gf	101928.8963	8431674.057	4	0.003
B1	133579.0543	21312265.52	4	0.002

The values of θ in X direction less than 0.1neglect effect of P-delta.

In Y direction:

Table3. 26:Effect of p-delta in Y direction.

Story	P(KN)	Stiff Y(kN/m)	hxs(m)	Theta=P/(h×S)
Stair	1024.7511	90467.42	3	0.004
Roof	11632.0099	1572317.64	4	0.002
F3	29457.4112	3709096.238	4	0.002
F2	47253.2329	6664598.372	4	0.002
F1	67868.9117	11463249.01	4	0.001
Gf	101928.8964	26983383.77	4	0.001
B1	133579.0544	49692228.28	4	0.001

The values of θ in Y direction less than 0.1 neglect effect of P-delta.

3.4.8 Verification of internal forces:

3.4.8.1 Slabs:

Slabs in Y direction:

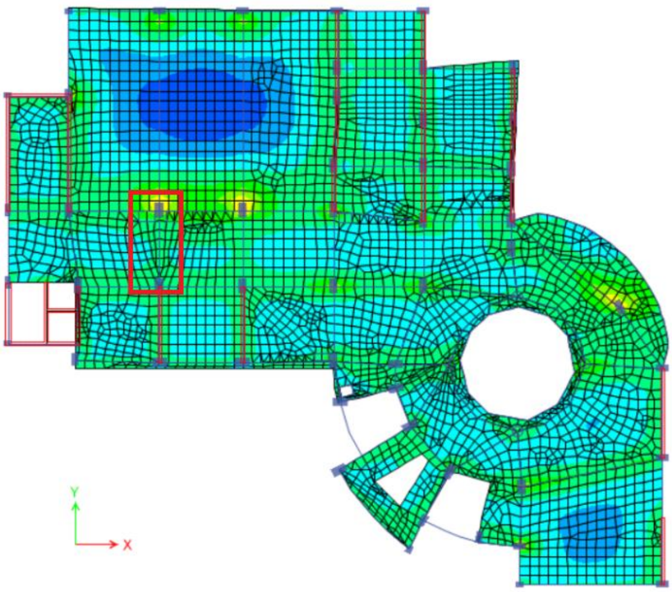


Figure3. 67 Strip in Y direction.

Hand calculations:

Slab own weight = 0.3 × 25

= 7.5 KN/m^2

SID = 7.5 KN/m^2

Live load = 5 KN/m^2

Wu = 1.2 D+1.2 SID +1.6 L

= 1.2 × 7.5 + 1.2× 7.5 + 1.6× 5

= 26 KN/m^2

$q_u = W_u\ L_2 = 26 \times 5.14 = 133.64$ KN/m

$$M_o = \frac{q_u\ L_1^{\ 2}}{8} = \frac{133.64 \times 4.48^2}{8} = 335.3 \text{ KN.m}$$

$$M_b = \frac{W_u\ L_1^{\ 2}}{8} = \frac{0.3 \times 0.8 \times 25 \times 1.2 \times 4.48^2}{8} = 18 \text{ KN.m}$$

Sum. of M = 335.3+18 = 353.3 KN.m

By ETABS:

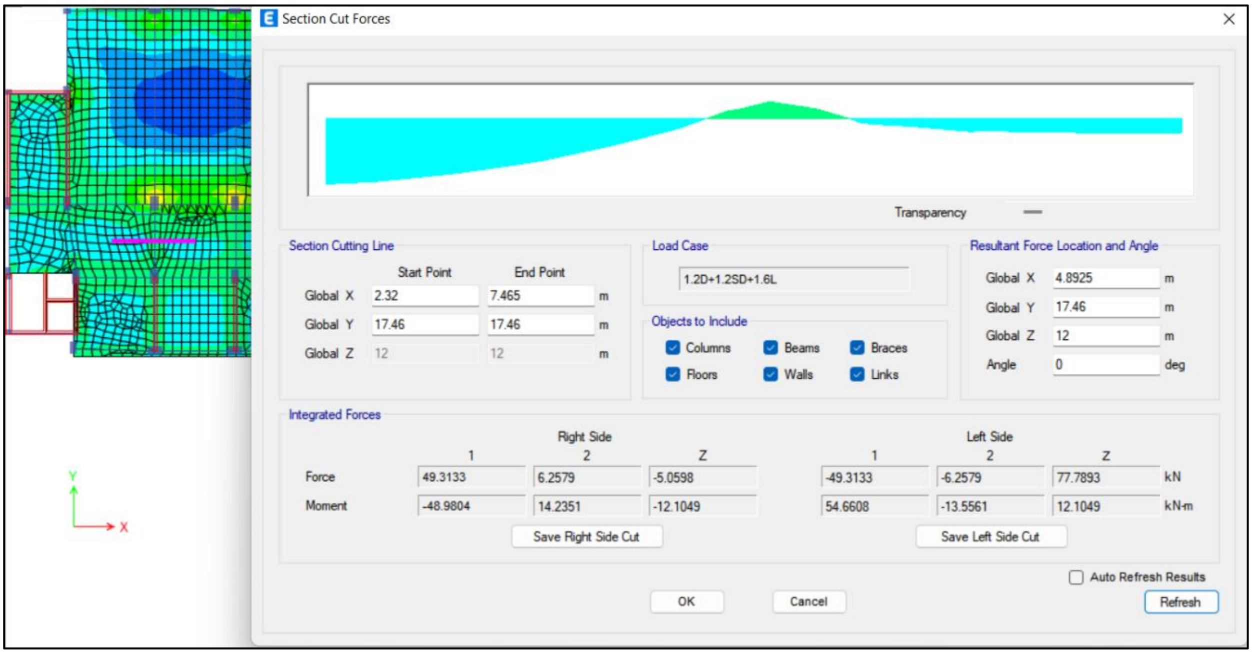
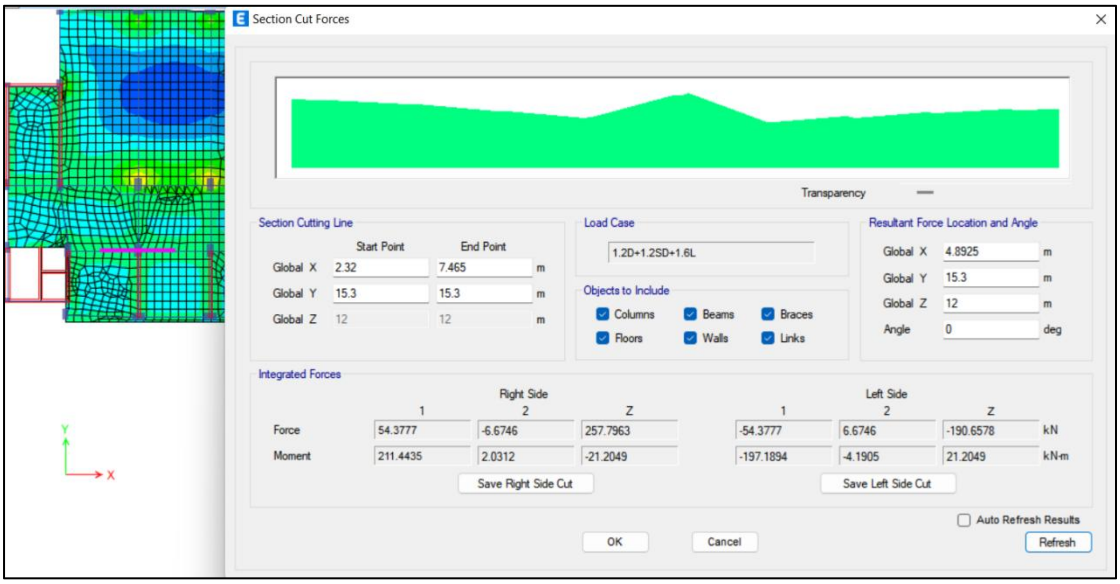


Figure3. 68 Moment 2-2 By ETABS.



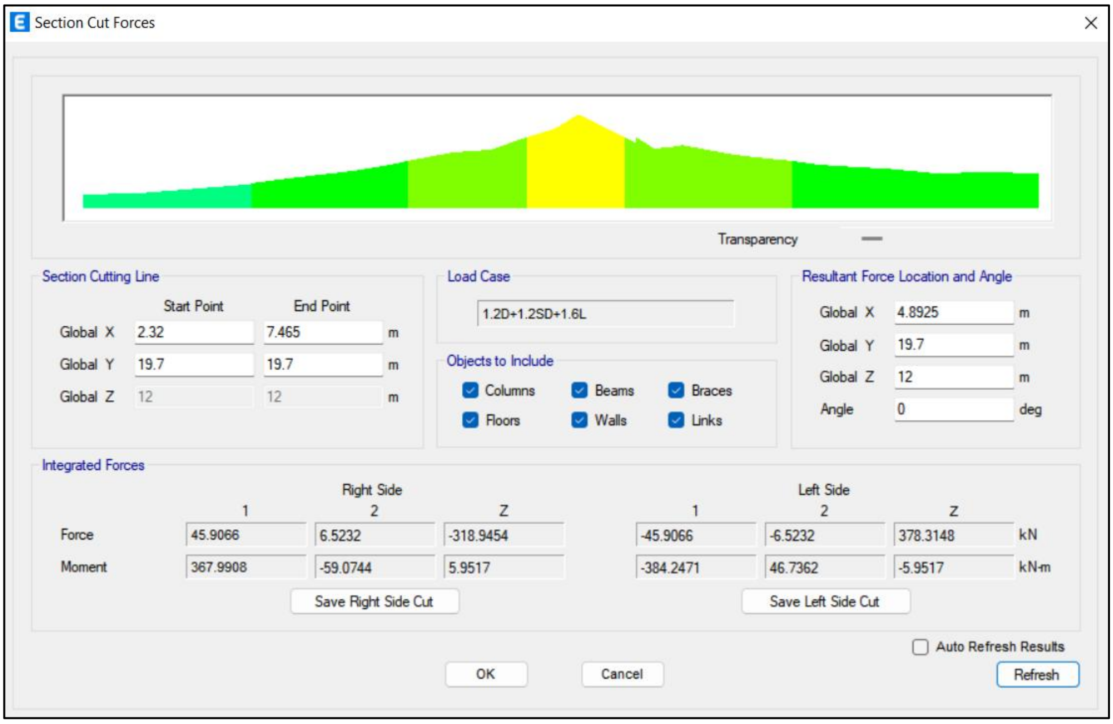


Figure3. 69: Moment 2-2 By ETABS.

$$M_o = \frac{368 + 211.44}{2} + 49$$

$$= 338.7$$

$$\text{Difference percentage} = \frac{353.3 - 338.7}{353.3} \times 100\%$$

$$= 4.13\%$$

Slabs in X direction:

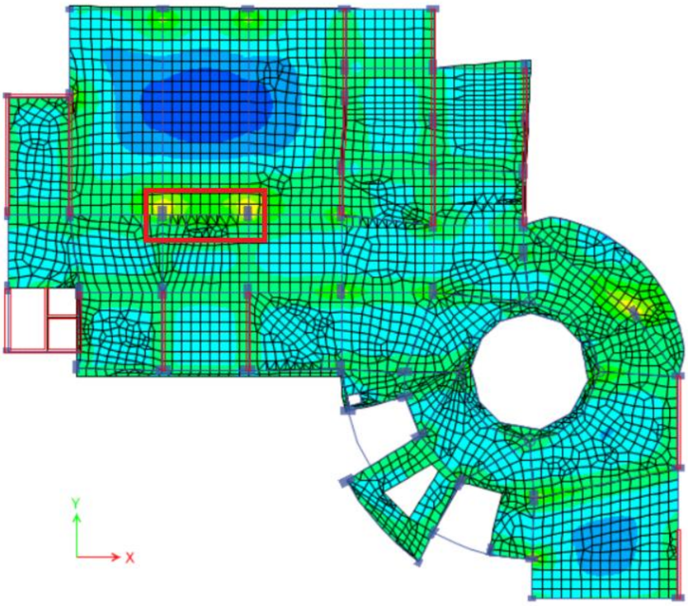


Figure3. 70 slab to be check.

$$q_u = W_u\ L_2 \quad = 26 \times\ 8.14 = 211.64 \text{ KN/m}$$

$$M_o = \frac{q_u\ L_1^{\ 2}}{8} = \frac{211.64 \times 4.93^2}{8} = 643 \text{ KN.m}$$

$$M_b = \frac{W_u\ L_1^{\ 2}}{8} = \frac{0.3 \times 0.25 \times 25 \times 1.2 \times 4.93^2}{8} = 6.8 \text{ KN.m}$$

Sum. of M = 643+ 6.8 = 649.8 KN.m

By ETABS:

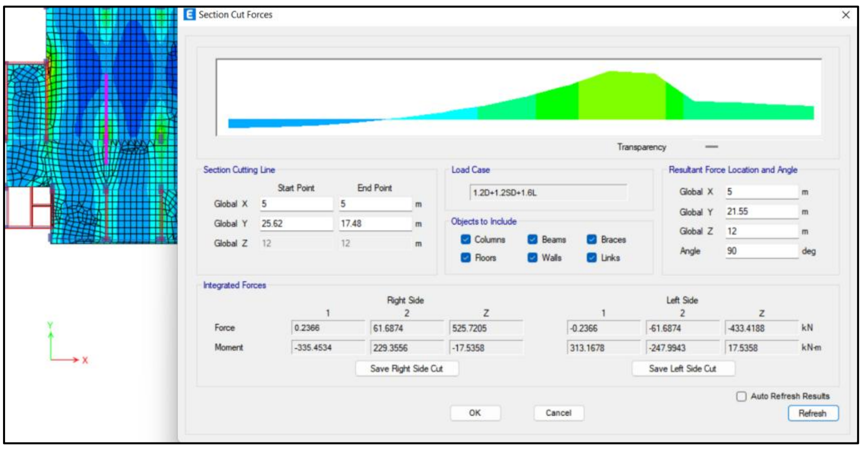


Figure3. 71 Moment 1-1 by ETABS.

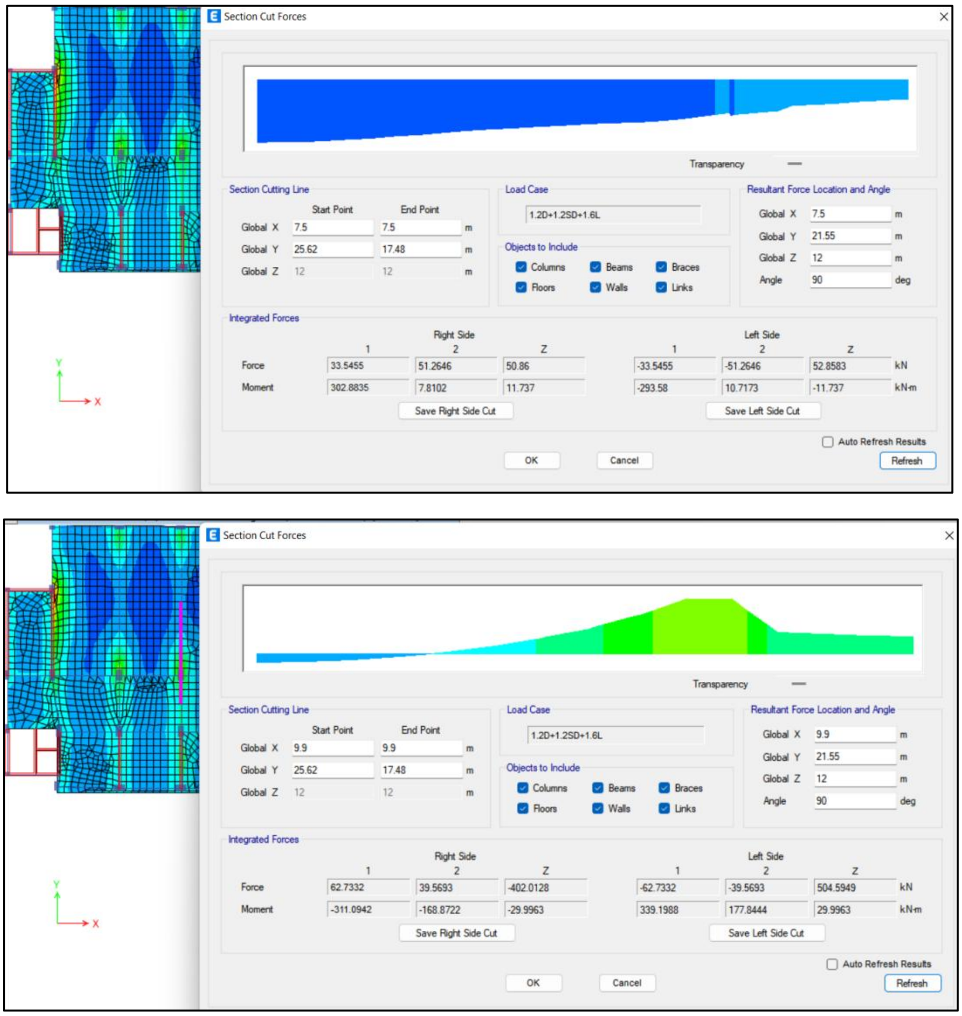


Figure3. 72: Moment 1-1 by ETABS.

$$M_o = \frac{335.5 + 311}{2} + 303$$

$$= 626.25 \text{ KN.m}$$

$$\text{Difference percentage} = \frac{649.8 - 626.5}{649.8} * 100\%$$

$$= 3.6\%$$

3.4.8.2 Beams:

In Y_Dir.

Hand calculation:

Negative factored moment = $0.65 \times M_o$

= 0.65×353.3

= 229.65 KN.m

Positive factored moment = $0.35 \times M_o$

= 0.35×353.3

= 123.65 KN.m

$I_b = 14.4 \times 10^{-3}$
$I_s = 11.1 \times 10^{-3}$
$\alpha = I_b/I_s$
$= 1.3$
$L_2 = 4.93$ m
$L_1 = 4.48$ m
$L_2/L_1 = 1.1$
$\alpha\, L_2/L_1 = 1.43$

Calculation of the coefficients is done by using the following Tables.

Table3. 27:ACI 318-14 Table 8.10.5.1-Portion of interior negative M, in column strip

$\alpha_f\, L_2/L_1$	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

Table3. 28:ACI 318-14 Table 8.10.5.5-Portion of positive M, in column strip

$\alpha_f\, L_2/L_1$	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

Table3. 29:ACI 318-14 Table 8.10.5.7.1-Portion of column strip Mu in beams

$\alpha_f L_2/L_1$	**Distribution Coefficient**
0.0	0.0
≥ 1.0	0.85

Negative Column strip = $0.72 \times 229.65 = 165.35$ KN.m

Positive Column strip = $0.72 \times 123.65 = 89$ KN.m

Negative moment at beam = 0.85×283 = 240.6 KN.m

Positive moment at beam =0.85×89 = 75.7 KN.m

Figure3. 73 moments for beam in the Y direction.

From ETABS:

M1- = 77.8 KN.m, M + = 47 KN.m

$$\text{Difference percentage } (M_-) = \frac{240.6 - 77.8}{240.6} \times 100\%$$

$$= 67.66\ \%$$

$$\text{Difference percentage } (M_+) = \frac{75.7 - 47}{75.7} \times 100\%$$

$$= 38\ \%$$

In X_Dir.

Hand calculation:

Negative factored moment = $0.65\times M_o$

$= 0.65\times649.8$

$=422.4$ KN.m

Positive factored moment = $0.35\times M_o$

$= 0.35\times649.8$

$= 227.4$ KN.m

$I_b = 4.5 \times10^{-3}$

$I_s = 18.3 \times10^{-3}$

$\alpha = I_b/I_s$

$= 0.245$

$L_2 = 8.14$ m

$L_1 = 4.9$ m

$L_2/L_1 = 1.66$

$\alpha\ L_2/L_1 = 0.41$

Negative Column strip = 0.67×422.4 = 283 KN.m

Positive Column strip = 0.58×227.4 = 132 KN.m

Negative moment at beam = 0.35×283 = 99 KN.m

Positive moment at beam =0.35×132 = 46.2 KN.m

Figure3. 74 moments for beam in the X direction.

From ETABS:

M1- = 25.6, M + =15.4

$$\text{Difference percentage } (M_{-}) = \frac{99 - 25.6}{99} \times 100\%$$

$$= 74.14\%$$

$$\text{Difference percentage } (M_{+}) = \frac{46.2 - 15.4}{46.2} \times 100\%$$

$$= 66.7\%$$

3.4.8.3 Columns:

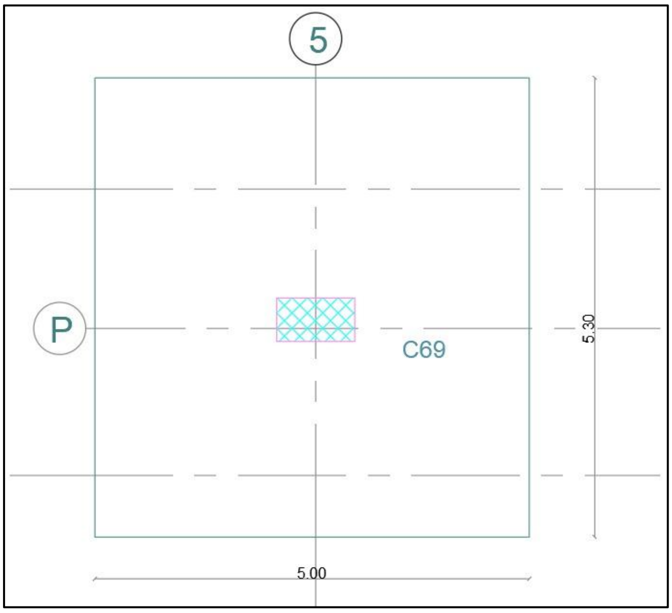


Figure3. 75 column site .

Column 69 is appeared in story basement one and ground floor

Tributary area = 26.5 m^2

Column section: C 500 × 900 mm

Column self-weight = Area × Column length × Unit weight of reinforced concrete

= 0.5 × 0.9 × 2 × 4 × 25

= 90 KN.

Beam weight = Area × Beam length × Unit weight of reinforced concrete

= 0.4 × 0.4 ×10.335× 25 ×2

= 82.56 KN.

Slab weight = Tributary area × Thickness of slab × Unit weight of reinforced concrete

= 26.6 × 0.20 × 25 × 2

= 266.64 KN

Total dead load = (397.5+82.56+90)

= 439.2 KN.

Live load = Tributary area × Live load

= 26.5 × (5+3)

= 209.6

Superimposed dead load = Tributary area × SID

$= 26.5 \times (4.5 + 7)$

$= 304.75$

Results from ETABS are subject to ultimate load combination: 1.2 D+1.2SID + 1.6L.
Manual Ultimate base reaction at column 69 is = $1.2\times(304.75+439.2)+1.6\times(209.6)$

$= 1228.1$ KN

Ultimate base reaction at column 69 from ETABS in figure **3.76:**

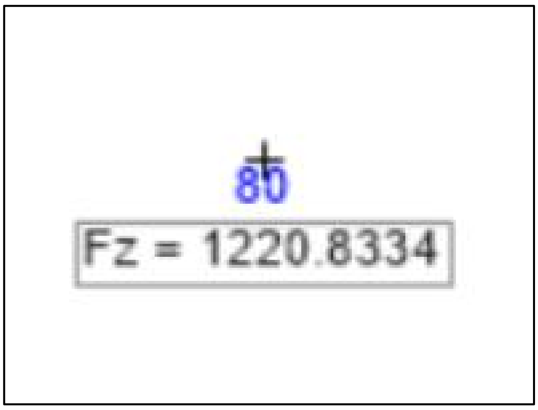


Figure3. 76 Column reaction by ETABS.

Difference percentage= $\frac{1228.1-1220.83}{1228.1} \times 100\%$

$= 0.6\ \%$

3.4.8.4 Walls:

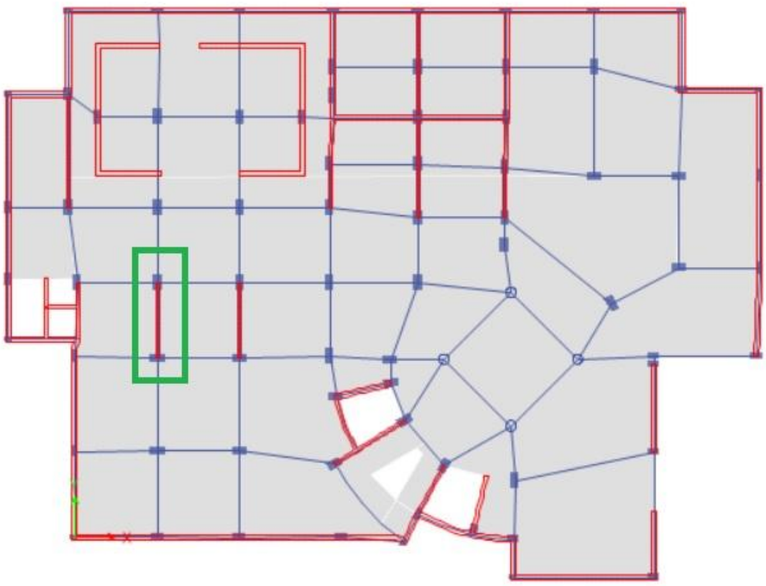


Figure3. 77 Wall to be check.

Calculation by hand:

Tributary area equal to 47.42 m^2 for B1and GF, and equal to 34.52 m^2 for F1, F2, F3 and Roof.

Sum. of beams length equal to 19.5 m for B1and GF, and equal to 16.66 m for F1, F2, F3 and Roof.

Dead Load:

Slab weight:

Table3. 30: Slab weight.

Floor	Tributary area(m2)	Slab thickness (m)	Slab weight (KN)
B1	47.42	0.2	237.1
GF	47.42	0.2	237.1
F1	34.52	0.2	237.1
F2	34.52	0.2	237.1
F3	34.52	0.2	237.1
Roof	34.52	0.2	237.1
Sum.			1422.6

Beams weight:

Table3. 31: Beams weight.

Beam type	Length	Drop thickness	Width	Weight (KN)
B2	79	0.3	0.4	237
B3	27.32	0.3	0.8	163.92
Sum.				400.92

Wall own weight:

Table3. 32 :Wall own weight.

Wall length (m)	Hight(m)	Thickness	Floor number	Weight
4.5	4	0.2	6	540

Columns weight:

Table3. 33: Columns weight.

Column dimension	Floor number	Weight
0.5×0.9	6	540

Sum. of dead load = Slab weight + Beams weight + Wall own weight + Columns weight

= 1422.6 + 401 + 541.2 + 540

= 2904.8 KN

Superimposed dead load:

Table3. 34: Superimposed dead load.

Floor	Tributary area(m2)	SD	Weight
B1	47.42	4.5	213.39
GF	47.42	7	331.94
F1	34.52	7.5	258.9
F2	34.52	7.5	258.9
F3	34.52	7	241.64
Roof	34.52	0	0
Sum.			1304.77

Live load:

Table3. 35: Live load.

Floor	Tributary area(m2)	LL	Weight
B1	47.42	3	142.26
GF	47.42	5	237.1
F1	34.52	5	172.6
F2	11.1	4	44.4
F2	23.42	2	46.84
F3	34.52	5	172.6
Roof	34.52	0	0
Sum.			815.8

1.2D + 1.2SD + 1.6 L $= 1.2 \times 2904.8 + 1.2 \times 1304.77 + 1.6 \times 815.8$

$= 6356.76$ KN

By ETABS:

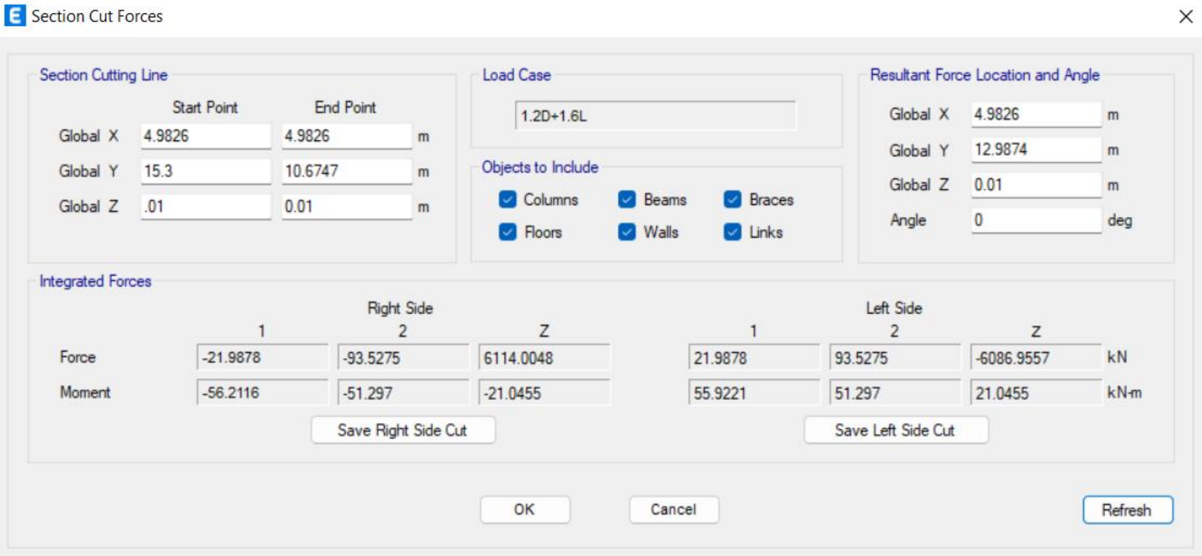


Figure3. 78: Reaction by ETABS.

Axial Load = 6114 KN

$$\text{Difference percentage} = \frac{6356.76 - 6114}{6356.76} \times 100\%$$

$= 3.8\ \%$

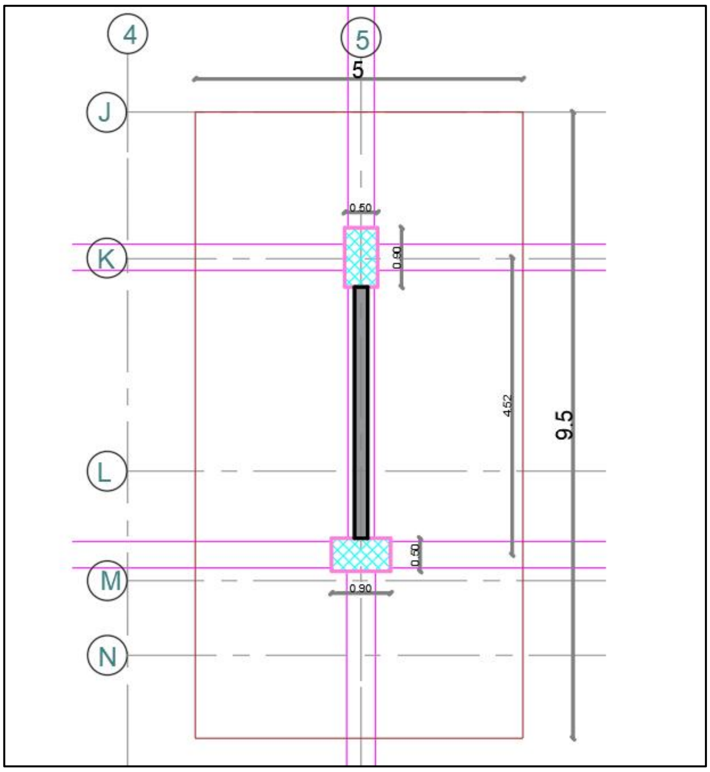


Figure3. 79: Tributary area for the wall.

3.5 Deflection computations:

3.5.1 Deflection check:

Deflection can be described as the degree to which an element of structure changes (deformed), this can happen under loading of the element which causes either displacement or rotation or both, there are two types when considering the deflection check:

1. Immediate deflection due to live loads in roofs and floors that not supporting or attached to non-structural elements likely to be damaged by large deflections.

2. Long term deflection in the floors and roofs that supporting or attached to non-structural elements either likely to be damaged by large deflections or not.

The long-term deflection was computed according to the ACI 318-19 code using the following equation: -

$$\Delta LT = \Delta L + \lambda_{\infty} \Delta D + \lambda_t \Delta L$$

Where: -

ΔLT: Total long-term deflection.

ΔL: Immediate deflection due to live load.

ΔD: Immediate deflection due to dead load.

ΔLs: Immediate deflection due to sustained live load.

λ: Multiplier for additional deflection due to long-term effects. And was computed by the following equation: -

$$\lambda = \frac{\varepsilon}{1+50\rho}$$

ρ`: Compression steel ratio.

ξ: Time-dependent factor for sustained loads and for five years or more, has a value equal to 2.

Allowable deflections:

The minimum thickness mentioned in ACI 318-19 code tables 7.3.1.1 and 9.3.1.1 shall apply for one way construction or attached to partitions or other construction likely to be damaged by large deflections, unless calculated deflection indicates that a slab with less thickness can be used for more economical design.

Table 3.30 (ACI 318-19 Table 24.2.2) shows the allowable deflections in beams and slabs.

Table3. 36: (ACI 318-19 Table 24.2.2) Maximum permissible calculated deflection.

Member	Condition		Deflection to be conside ed	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 load combination used to check the deflection is:

Long term = 2 Dead load + 2 Super imposed loads + 1.5 Live Load.

Long term allowable deflection is:

$$\Delta_{all} = \frac{L}{240}$$

This procedure was done to all slabs to check the deflection, If the deflection is larger than the allowable, two option are going to be made either to increase the depth of the slab or to add beams one sample calculation is shown in this section and it was done to story F2 (Z = 16 m) in ETABS.

Readings are taken such that there are 6 reading taken from the sides and one in the middle. Each direction will be checked individually. The location and the direction of these reading are shown in Figure**3.80.**

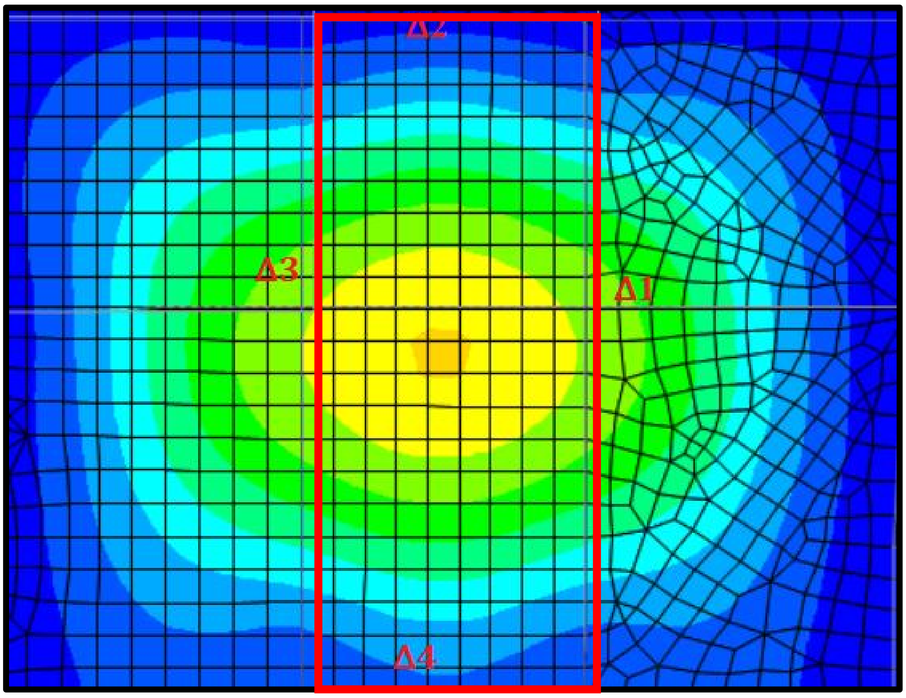


Figure3. 80 Deformed shape of slab in first floor.

long-term deflections for slab systems on the sides of the panels:

In X direction:

* Δ2:

long term deflection allowable deflection =L/240 = (4.95/240) *1000=20.6 mm.

long-term deflections for slab systems at the middle = 12.3mm

long-term deflections for slab systems at the right side = 9 mm

long-term deflections for slab systems at the left side = 9.5 mm

Relative long-term deflection = $12.3 - \frac{9+9.5}{2}$ = 3.05 mm < 20.6mm it is ok.

* Δ4:

long term deflection allowable deflection =L/240 = (4.95/240) *1000=20.6 mm.

long-term deflections for slab systems at the middle = 13.35mm

long-term deflections for slab systems at the right side = 11.162mm

long-term deflections for slab systems at the left side = 11.568mm

Relative long-term deflection = $13.35 - \frac{11.162+11.568}{2}$ = 1.98 mm < 20.6mm it is ok.

In Y direction:

* Δ1:

long term deflection allowable deflection =L/240 = (12 /240) *1000= 50 mm.

long-term deflections for slab systems at the middle = 58.46mm long-term deflections for slab systems at the top = 9mm

long-term deflections for slab systems at the bottom = 9.5 mm

Relative long-term deflection = $58.46 - \frac{9+9.5}{2}$ = 49.21 < 50 it is ok.

* Δ3:

long term deflection allowable deflection =L/240 = (12 /240) *1000= 50 mm.

long-term deflections for slab systems at the middle = 13.35 mm

long-term deflections for slab systems at the top = 11.56 mm

long-term deflections for slab systems at the bottom =9.5 mm

Relative long-term deflection = $13.35 - \frac{9.5+11.56}{2}$ = 2.82 mm < 50 it is ok.

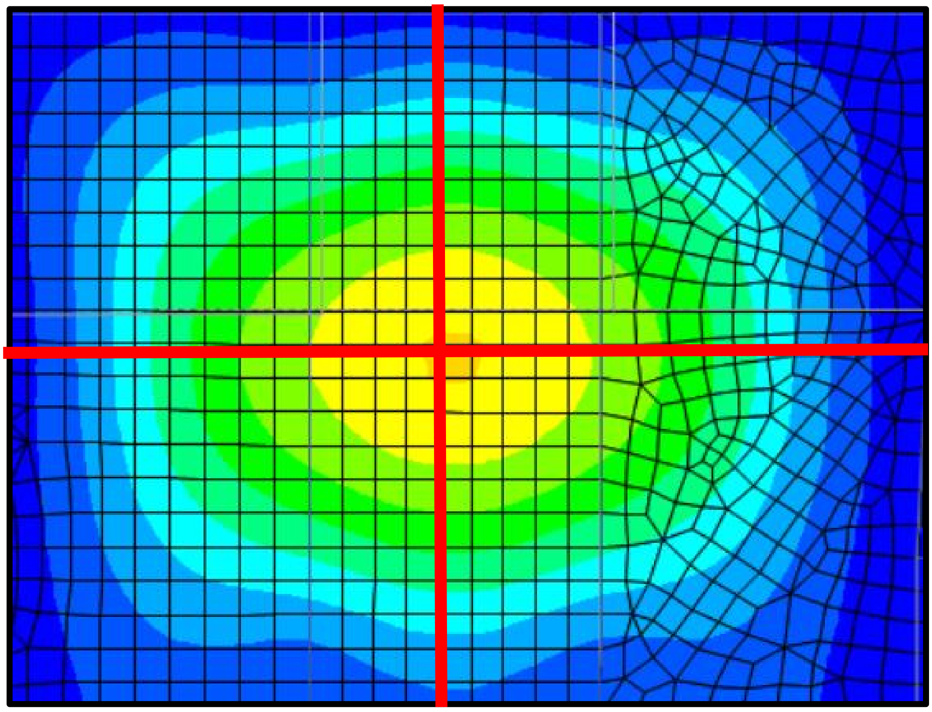


Figure3. 81 Deformed shape of slab

long-term deflections for slab systems on the center of panels:

In X direction:

long term deflection allowable deflection =L/240 = (4.95/240) *1000=20.6 mm.

long-term deflections for slab systems at the middle = 64.5mm

long-term deflections for slab systems at the right side = 59 mm

long-term deflections for slab systems at the left side = 64.5 mm

Relative long-term deflection = $64.5 - \frac{58.46+59}{2}$ = 5.77 mm < 20.6mm   it is ok.

In Y direction:

long term deflection allowable deflection =L/240 = (12 /240) *1000= 50 mm.

long-term deflections for slab systems at the middle = 64.5mm

long-term deflections for slab systems at the top = 12.3mm

long-term deflections for slab systems at the bottom = 16.8 mm

Relative long-term deflection = $64.5 - \frac{12.3+16.8}{2}$ = 49.95 < 50 it is OK.

3.5.2 Story drift, elastic and plastic deflections

From ETABS:

In X direction

Table3. 37: Story drift by ETABS.

Story	Case output	Story drift(mm)
Stair	Qx	23.28
Roof	Qx	16.92
F3	Qx	20.48
F2	Qx	22.25
F1	Qx	20..14
Gf	Qx	2.43
B1	Qx	1.09

In Y direction:

Table3. 38:Story drift by ETABS.

Story	Case output	Story drift (mm)
Stair	Qy	19.043
Roof	Qy	7.95
F3	Qy	8.28
F2	Qy	6.69
F1	Qy	6.21
Gf	Qy	2.8
B1	Qy	2.04

The allowable story drift , from table 12.12-1 In ASCE-7-22 are shown in **Figure 3.82.**

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	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

Figure3. 82: allowable story drift.

allowable Story drift = 0.01×4000

= 40mm

It is noted that all story drift values in X and Y directions are less than 40 mm.

3.5.3 Width of separation joints between building parts

Sometimes different structures are combined within one building, these two structures will behave very differently in an earthquake, which can lead to the transfer of damaging impact forces between the two structures and cause either damage or collapse, so using separation joints allow each structure to behave independently and avoid impacts from the other structure. Joints between the two structures should be filled with elastic materials and all separation joints should be wide enough to accommodate differences in lateral movement between the two structures.

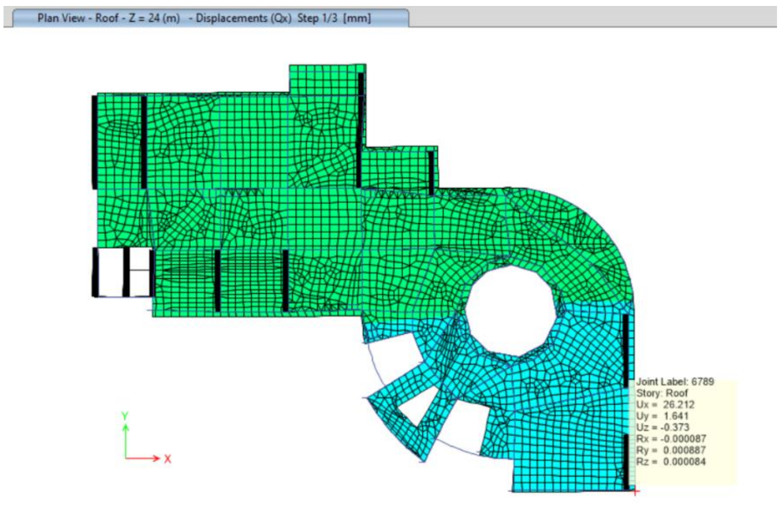


Figure3. 83Slab displacements from earthquake x in Left Part.

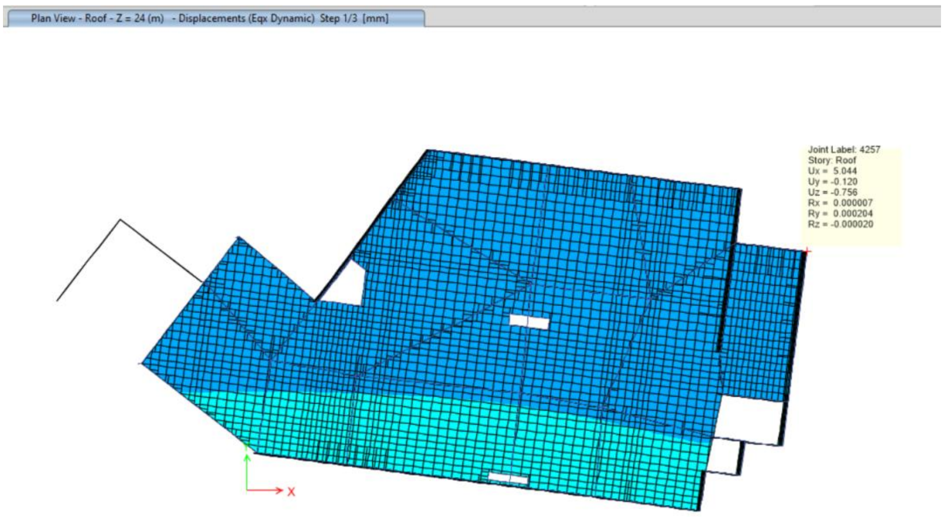


Figure3. 84Slab displacements from earthquake x in Right Part

Displacement in Left Part = 26.2 mm

Displacement in Right Part = 5 mm

$$\text{Resultant displacement} = \left[\sqrt{(\text{Displacement in Left Part})^2 + (\text{Displacement in Right Part})^2}\right]\frac{C_d}{I}$$

$$= \left[\sqrt{(26.2)^2 + (5)^2}\right]\frac{5}{1.5}$$

$$= [26.6]\frac{5}{1.5}$$

$$= 89 \text{ mm.}$$

3.6 Verification of structural design

3.6.1 Beam design verification :

Select the following beam (shown in **Figure 3.85**) for flexure design check, which is located in the second floor, this beam has a length of 15.8 m and dimensions for the cross section are 1100 mm in width and 600 mm in depth (d=540 mm) .

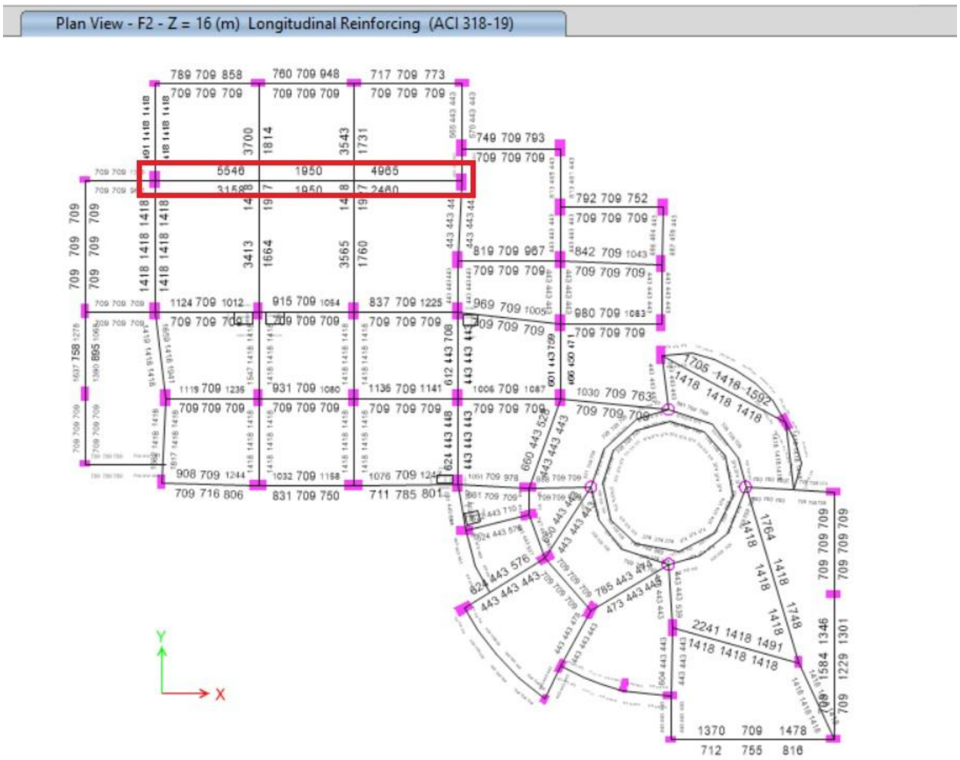


Figure3. 85:The beam selected for the flexure design check .

3.6.1.1 Flexure Design Verification:

Figure 3.86 shows ETABS result for selected beam.

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)	-419.9549	0	2124	0	1950	2124
Bottom (-2 Axis)	209.9774	0	0	1045	1950	1950

Figure3. 86:The negative ultimate moment for the design

The area of steel for this beam due to the ultimate negative moment = 2124 mm^2.

Mu =419.95 KN.m

Cover =60 mm

d=600-60 =540 mm

b=1100 mm

$$\rho = \frac{0.85\, f_c'}{f_y}\left(1-\sqrt{1-\frac{2.61 M_u}{b\, d^2\, f_c'}}\right)$$
$$= 0.00357$$

$\rho_{min} = 0.00333$

$\rho_{max} = 0.025$ (ACI 318-19, section 18.6.3.1)

$\rho_{min} < \rho < \rho_{max}$ ….OK.

$As = \rho b d$

$= 2120.6\ \text{mm}^2$

Difference percentage between the longitudinal reinforcement from the manual and from ETABS:

Difference percentage= $\frac{2120.6-2124}{2120.60} \times 100\%$ =0.16% <5% …OK.

3.6.1.2 Shear Design Verification:

Use the following beam (shown in **Figure 3.87**) for shear and torsion design check, which is located in the second floor, this beam has a length of 5.35 m and the dimensions for the cross section are 400 mm in width and 600 mm in depth (d=540 mm).

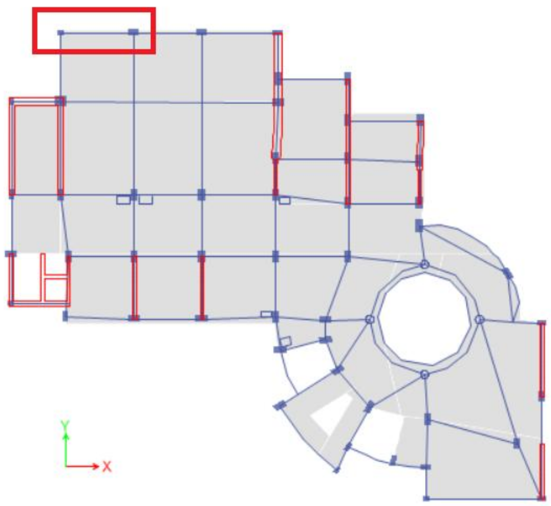


Figure3. 87:The beam selected for the shear and torsion design check .

Ultimate shear value is shown in **Figure 3.88**.

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 mm^2/m
145.9852	142.3585	3.6267	85.1202	21.65

Figure3. 88:The beam selected for the shear and torsion design check .

V_u = 145.98 kN

$$V_c = (\frac{1}{6}\sqrt{f_c'} + \frac{N_u}{6A_g})b_w d$$

$$= (\frac{1}{6}\sqrt{28} - \frac{28.8\times1000}{6\times400\times600})\times400\times\frac{540}{1000}$$

$$= 189.98 \text{ KN}$$

$$\phi V_c = 0.75\times189.98$$

$$= 142.4 \text{ KN}$$

$$\phi V_s = V_u - \phi V_c$$

$$= 145.98 - 142.4$$

$$= 3.6 \text{ KN}$$

V_p calculation:

Figure 3.89 shows the area of steel at the top and bottom of the beam which is shown above.

709	709	752
709	709	709

Figure3. 89 Area of steel

$$M_{pr} = \phi A_s\times1.25\times f_y(d - \frac{a}{2})$$

$$a = \frac{A_s\times1.25\times f_y}{0.85 f_c b}$$

A_{s1}=767 mm^2

A_{s2}= 709 mm^2

b =400 mm

a_1=42.3 … M_{pr1}=208.9 KN.m

a_2=39 mm … M_{pr2}= 193.72 KN.m

$$V_p = \frac{\sum M_{pr}}{L_n}$$
$$= \frac{208.9 + 193.72}{4.7}$$
$$= 85.2 \text{ KN}$$

$$\frac{A_v}{S} = \frac{V_s}{f_{yt} d}$$
$$= \frac{3.6 \times 1000 / 0.75}{420 \times 540} \times 10^6$$
$$= 21.16 \text{ mm}^2/\text{mm}$$

3.6.1.3 Torsion Design Verification:

The ultimate torsion (T_u) for the design = 21.82 KN.m as shown in **Figure 3.90** .

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
21.8272	9.6237	38.4949	1351.5	1644.4	260.26	847

Figure3. 90: ultimate torsion for the design .

$$\phi T_{th} = \phi 0.083 \lambda \sqrt{f'_c} \frac{A_{cp}}{P_{cp}}$$

Where:

A_{cp} = area enclosed by outermost perimeter of concrete cross section.

P_{cp} = outside perimeter of concrete cross section.

$$\phi T_{th} = \phi 0.083 \lambda \sqrt{f'_c} \frac{A_{cp}}{P_{cp}}$$
$$= 0.75 \times 0.083 \times 1 \times \sqrt{28} \times \frac{600 \times 400}{2(600 + 400)}$$
$$= 9.486 \text{ KN.m}$$

$$\frac{A_t}{S} = \frac{T_n}{2A_o f_{yt}}$$

Where:

A_t/S = area of stirrups required for torsion .

A_o = gross area enclosed by shear flow path.

$= 0.85\ X_o\ Y_o$

X_o =400- 89

= 311

Y_o= 600-89

= 511

A_o= 135082.85 mm^2

$T_n = T_u / Ø$

= 21.8/0.75

= 29 KN.m

$A_t/S = 29 \times 10^6 /(2 \times 13508285 \times 420)$

= 0.256 mm^2/mm

=256 mm^2/m

A_t/S from ETABS equal to 260.26 mm^2/m.

Difference percentage = 1.4% , This difference is due to the beam being subjected to an axial load.

$$A_l = \frac{A_t}{S} P_h \frac{f_{yt}}{f_y}$$

$$A_{l,\min} = \frac{5\sqrt{f_c'}A_{cp}}{12 f_y} - \frac{A_t}{S} P_h \frac{f_{yt}}{f_y}, \text{Where } \frac{A_t}{S} \geq 0.175 \frac{b_w}{f_y}$$

Where :

A_L : longitudinal reinforcement for torsion .

P_h : perimeter of centerline of outermost closed transverse reinforcement .

$P_h = 2(X_o+Y_o)$

$= 2(311+511)$

$= 1644$ mm

$-A_L = 0.256 \times 1644$

$= 420.86$ mm

$$- A_{L,min} = (5 \times \sqrt{28 \times 600 \times 400} / 12 \times 420) - 0.256 \times 1 \times 1644$$
$$= 838.9 \text{ mm}^2$$

$A_{L,min} > A_L$

$A_L = 838.9$ mm^2

A_L from ETABS equal to 847 mm^2.

Difference percentage = 0.9% , This difference is due to the beam being subjected to an axial load .

3.6.2 Column design verification :

Select the column shown in **Figure 3.91** for the check, which is located on the third floor.

This column has a length of 4 m and the dimensions for the cross section are 600 mm and 900 mm.

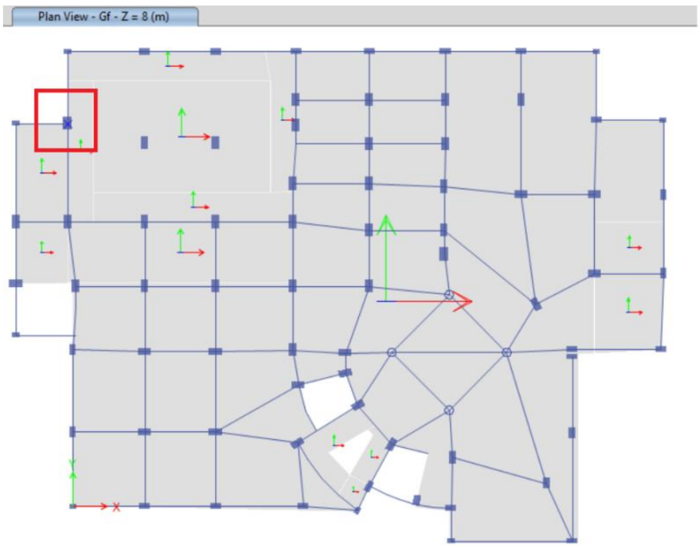


Figure3. 91: The column selected for design check.

From ETABS :

P_u = 4629 kN

$M_{u,2}$ = 581.2 kN.m

$M_{u,3}$ = 150.5 kN.m

The values of Pu, Mu,2 and Mu,3 is shown in **Figure 3.92.**

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 % %
-4628.8904	581.2067	-150.4615	195.5243	153.8643	16154	2.99

Figure3. 92 The values of Pu, Mu,2 and Mu,3 for the design

Figure 3.93 shows interaction diagram between $ØP_n$ and $ØM_{n,2}$, to check the column capacity for them.

Figure 3.94 shows interaction diagram between $ØP_n$ and $ØM_{n,3}$, to check the column capacity for them.

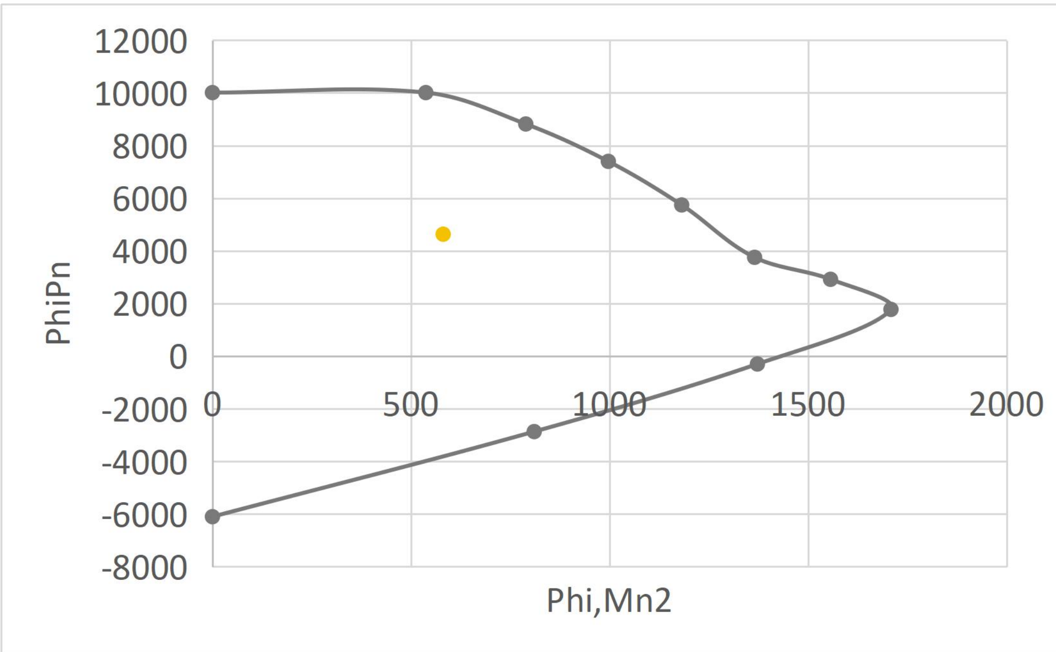


Figure3. 93 Interaction Diagram (P and M2)

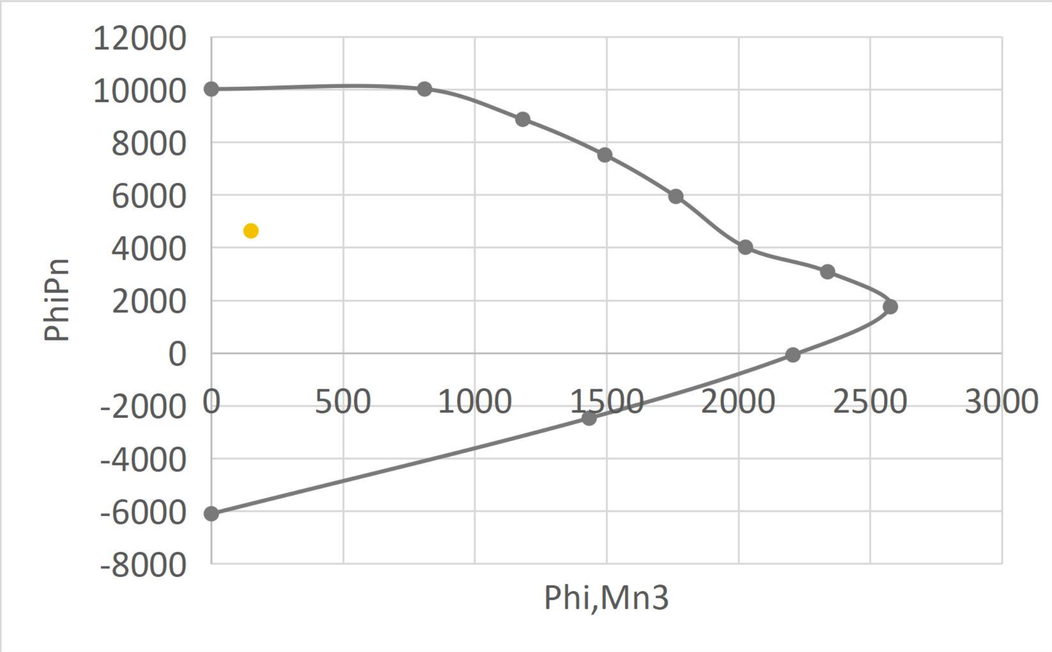


Figure3. 94 Interaction Diagram (P and M3)

Bressler's Reciprocal load method shall be applied to check the column section for the biaxial moments .

$$\frac{1}{\phi Pn} = \frac{1}{\phi Pn_3} + \frac{1}{\phi Pn_2} - \frac{1}{\phi Pn_o}$$

Where:

$ØP_n$: Axial load capacity for Mu,3 and Mu,2.

$ØP_{n,3}$: Axial load capacity for Mu,3.

$ØP_{n,2}$: Axial load capacity for Mu,2.

$ØP_{n,o}$: Axial load capacity without moment.

$ØP_{n,3}$ =10011 kN (from **Figure 3.93)**

$ØP_{n,2}$ =10011 kN (from **Figure 3.94**)

$ØP_{n,o}$ = Ø ƛ (0.85 f'c (Ag-As) + As fy)

$ØP_{n,o} = 0.65 \times 0.8 \times (0.85 \times 28 \times (540000 - 16154) + 16154 \times 420)$

$ØP_{n,o}$ = 10011 kN

Then, $ØP_n$ = 10011 kN > 4629 , it is Ok .

3.6.3 Wall design verification:

Use the following wall for check which is located in the basement one floor, as shown in **Figure 3.95**, This wall has a height of 4 m and the dimensions for the cross section are 8424 mm and 300mm thickness.

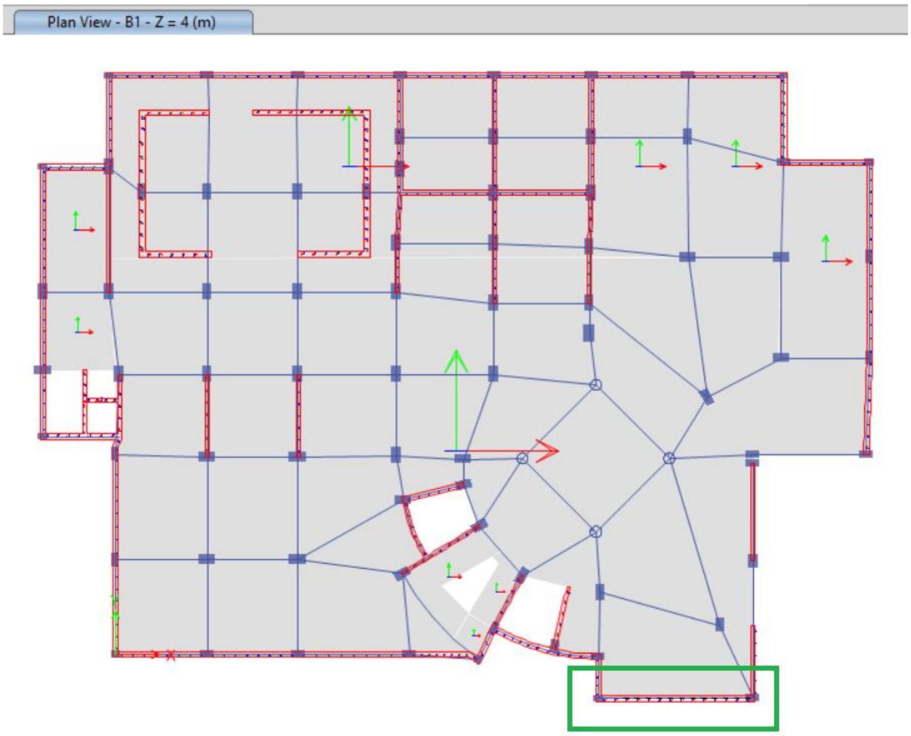


Figure3. 95: The wall selected for design check.

From ETABS :

$P_u = 510$ KN

$M_{u2} = 373.8$ KN.m

$M_{u3} = 2296.11$ KN.m

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	8032	0.0032	0.0022	U5.2Dyn	510.0487	373.8298	2296.1117	2527146
Bottom	9544	0.0038	0.0022	U5.2Dyn	476.7578	407.2932	3029.966	2527146

Figure3. 96 The values of Pu, Mu,2 and Mu,3 for the design

Figure 3.97 shows interaction diagram between $ØP_n$ and $ØM_{n,2}$, to check the wall capacity for them.

Figure 3.98 shows interaction diagram between ØP$_n$ and ØM$_{n,3}$, to check the wall capacity for them.

As = 8032 mm^2

$$\text{Number of bars} = \frac{\text{Length of wall}}{\text{Spacing between bars}} = 2 \times \frac{8424 - (50 \times 2)}{250} = 68, 34 \text{ bar each side}$$

$$\text{Area of each bar} = \frac{\text{Total Rebar Area}}{\text{Number of bars}} = \frac{8032}{68} = 118 \text{ mm}^2, use\ 72 \text{ bars, 36 in each side}$$

$$= \frac{8032}{72} = 111.56 \text{ mm}^2 \rightarrow \text{Use } \phi 12 \text{ mm}$$

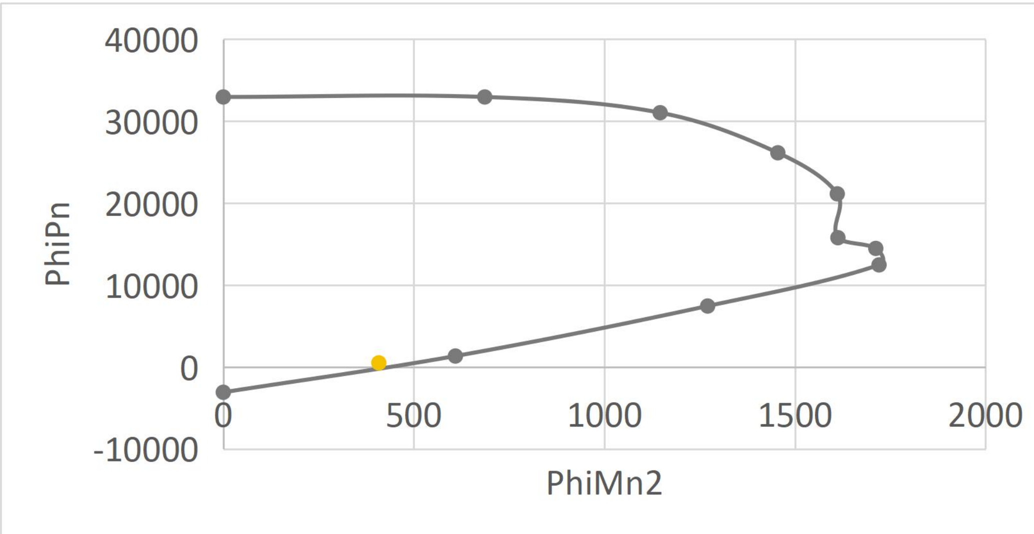


Figure3. 97 Interaction Diagram (P and M2)

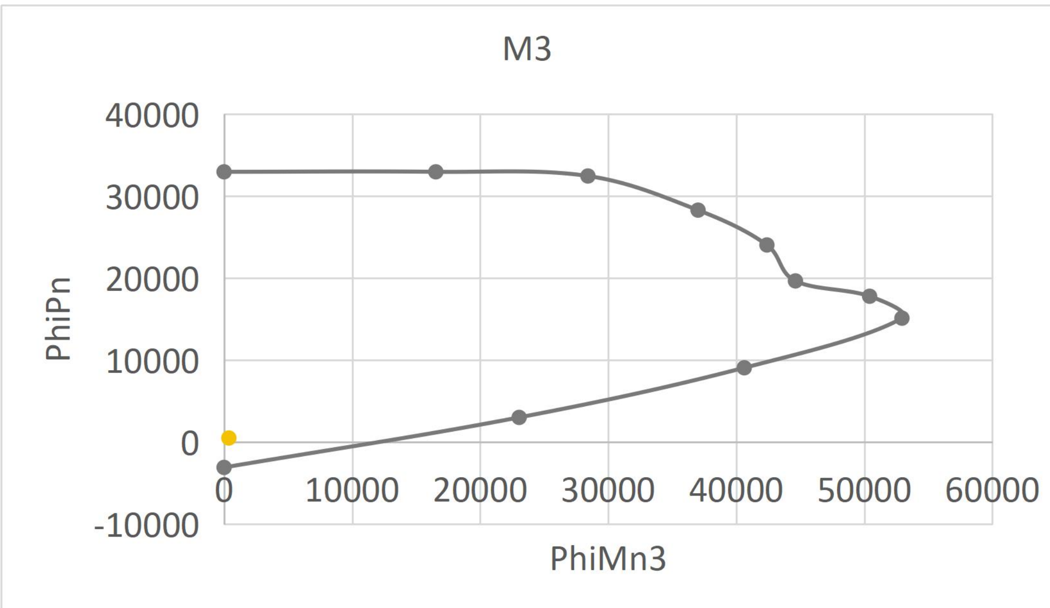


Figure3. 98 Interaction Diagram (P and M3)

Bressler's Reciprocal load method shall be applied to check the column section for the biaxial moments .

$$\frac{1}{\phi Pn} = \frac{1}{\phi Pn_3} + \frac{1}{\phi Pn_2} - \frac{1}{\phi Pn_o}$$

Where:

ØPn: Axial load capacity for Mu,3 and Mu,2.

ØPn,3: Axial load capacity for Mu,3.

ØPn,2: Axial load capacity for Mu,2.

ØPn, o: Axial load capacity without moment.

ØPn,3 =32953 kN (from **Figure 3.97**)

ØPn,2 =32953 kN (from **Figure 3.98**)

ØPn,o = Ø ƛ (0.85 f'c (Ag-As) + As fy)

ØPn,o = 0.65 × 0.8 × (0.85 × 28 × (227200– 8032) + 8032 × 420)

ØPn,o = 32931 kN

Then, ØPn = 48580 kN > 510 , it is Ok .

Figure 3.99 shows shear design result by ETABS.

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	750	U5.1	1567.1542	1921.251	1387.6914	1998.6712	3590.7732
Bottom	Leg 1	750	U5.1	1920.4767	3994.0866	1414.271	1998.6712	3590.7732

Figure3. 99 shear design result for selected beam in ETABS.

$$\begin{aligned} V_c &= \alpha\gamma\sqrt{f'_c}\, b\, L_w \\ &= 0.24 \times 1 \times \sqrt{28} \times 300 \times \frac{8384}{1000} \\ &= 3327.29 \text{ KN} \end{aligned}$$

$$\begin{aligned} \phi V_c &= 0.75 \times 3327.29 \\ &= 1996.37 \text{ KN} \end{aligned}$$

ØVc By ETABS equal to 1998.6 KN.

Different percentage = 0.11%

$$V_n = (\alpha_c \lambda \sqrt{f'_c} + \rho_l f_{yt}) A_{cv} \qquad (18.10.4.1)$$

where:

$\rho_l = 0.0025$

Ø = 0.6

$\alpha_c = 0.25$ for $h_w/L_w \leq 1.5$

$\alpha_c = 0.17$ for $h_w/L_w \geq 2.0$

h_w: Hight of the wall and equal 8m.

ℓ_w: Length of the wall and equal 8.4 m .

h_w/ℓ_w=0.95 <1.5 $\rightarrow \alpha c$ =0.25

$$V_n = (0.25 \times 1 \times \sqrt{28} + 0.0025 \times 420) \times 300 \times \frac{8384}{1000}$$
$$= 5968.25 \text{ KN}$$
$$\emptyset V_n = 0.6\ V_n$$

$$= 3580.95$$

ØV_n by ETABS equal to 3590.77 KN.

Different percentage = 0.2%

Boundary element check :

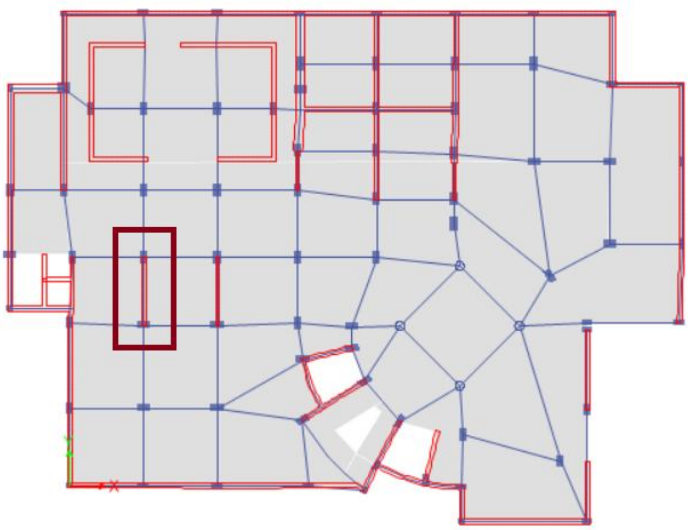


Figure3. 100 : The wall selected for Boundary element check.

As shown in the figure for bottom left :

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	425.4	U5.2Dyn	3208.4597	-1871.9187	5.04	5.6	850.9	1003.6
Top–Right	Leg 1	425.4	U5.2Dyn	3208.4597	1727.398	4.87	5.6	850.9	1003.6
Bottom–Left	Leg 1	448.4	U5.2Dyn	3436.8328	-1906.1149	5.29	5.6	896.8	1003.6
Botttom–Right	Leg 1	448.4	U5.2Dyn	3436.8328	1796.5618	5.16	5.6	896.8	1003.6

Figure3. 101 ETABS result for design.

For bottom Left side:

P_u = 3436.83 KN

M_u = 1906.11 KN.m

C_{depth} = 896.8 mm in ETABS.

C_{limit} = 1003.6 mm in ETABS.

$$\sigma = \frac{P_u}{A} \pm \frac{M_u C}{I}$$

Where:

P_u = ultimate axial force on wall.

Mu = ultimate bending moment on wall.

A = gross area of wall thickness.

I = gross moment of inertia of wall section.

C = Distance from the neutral axis of a flexural member to the fiber of maximum compressive strain, in. (mm)

Length of wall = 4516.4 mm.

Thickness of wall = 250 mm.

C = 0.5 × L =0.5 × 4516.4 = 2258.2 mm.

I=(1/12) × 250 ×4516.4^3 = 1.91 × 10^{12} mm^4.

$$\sigma = \frac{P_u}{A} \pm \frac{M_u C}{I} = \frac{3436.83 \times 1000}{250 \times 4516.4} + \frac{1906.11 \times 10^6 \times 2258.2}{1.91 \times 10^{12}} = 3.043 + 2.25$$

$= 5.29$ same value in ETABS

$$\sigma_{limit} = 0.2f'c$$
$$= 0.2 \times 28$$
$$= 5.6$$

The design displacement, δ_u is given by :

$$\delta_u = \delta_e \frac{C_d}{I}$$

Where:

Cd = deflection amplification factor.

I = important factor.

$$C_{limit} = \frac{L_w}{600(1.5\,\delta_u / h_{wcs})}$$

$$\delta_u / h_{wcs} \geq 0.005$$

displacement δ_e, by ETABS equal to 13.14 mm.

$$\delta_u = 13.14 \times \frac{5}{1.5} = 0.0438 \text{ mm}$$
$$\delta_u / h_{wcs} = 0.0438 / 4 = 0.0014 < 0.005$$
take $\delta_u / h_{wcs} = 0.005$

$$C_{limit} = \frac{L_w}{600(1.5\,\delta_u / h_{wcs})} = \frac{4516.4}{600 \times (1.5 \times 0.005)} = 1003.6$$

C_{limit} by ETABS equal to 1003.6 mm.

Different percentage = zero.

$$C_{depth} = (\frac{\alpha + \omega}{0.85\beta + 2\omega})L_w$$

$$\alpha = \frac{P_u}{b\,L_w\,f_c'}$$

$$\omega = \frac{\rho_l f_y}{f_c'}$$

Where:

P_u = ultimate axial compression force on wall.

b = thickness of wall.

L_w= length of wall.

ρl = ratio of vertical steel in the wall section.

Where :

Pu = ultimate axial compression force on wall.

b = thickness of wall.

Lw = length of wall.

ρ_1 = ratio of vertical steel in the wall section.

β_1 = factor relating the depth of the neutral axis and the depth of the compression block, which given by :

$$\beta_1 = 0.85 \qquad \text{for} \quad 17\text{ MPa} \le f'_c \le 28\text{MPa}$$

$$\beta_1 = 0.85 - 0.05\frac{f'_c - 28}{7} \qquad \text{for} \quad 28\text{ MPa} < f'_c < 56\text{MPa}$$

$$\beta_1 = 0.65 \qquad \text{for} \quad f'_c \ge 56\text{ MPa}$$

For this wall:

$$\alpha = \frac{P_u}{b\,L_w\,f'_c} = \frac{3436.83 \times 1000}{250 \times 416.4 \times 28} = 0.108$$

$$\omega = \frac{\rho_1 f_y}{f'_c} = \frac{0.0025 \times 420}{28} = 0.0375$$

$$\beta = 0.85$$

$$C_{depth} = \left(\frac{\alpha + \omega}{0.85\beta + 2\omega}\right)L_w$$

$$= \left(\frac{0.108 + 0.0375}{0.85 \times 0.85 + 2 \times 0.0375}\right) \times 4516.4$$

$$= 824\text{ mm.}$$

Cdepth by ETABS equal to 896.8mm.

Different percentage = 8.8%

3.7 Design of slabs:

Check for shear:

$$\phi V_c = \frac{\varphi}{6}\lambda\sqrt{f'_c}\, b_w d$$

$$= \frac{0.75}{6}\times 1\times\sqrt{28}\times 1000\times\frac{200}{1000}$$

= 132.28 KN for slab with 250mm thickness.

= 165.4 KN for slab with 300mm thickness.

Vu in basement 1 = 85 KN < ΦV_c , then it's OK.

Vu in ground floor = 83 KN <ΦV_c , then it's OK.

Vu in the first floor = 124 KN <ΦV_c , then it's OK

Vu in the second floor = 130 KN <ΦV_c , then it's OK.

Vu in the third floor = KN <ΦV_c, then it's OK.

∴ No Need for shear reinforcement.

• Splicing Lengths:

$$L_s(\phi 12) = 1.3\left(\frac{f_y}{2.1\sqrt{f'_c}}\right)d_b = 1.3\times\frac{420}{2.1\times\sqrt{28}}\times 12 = 590\text{ mm.}$$

$$L_s(\phi 14) = 1.3\left(\frac{f_y}{2.1\sqrt{f'_c}}\right)d_b = 1.3\times\frac{420}{2.1\times\sqrt{28}}\times 14 = 690\text{ mm.}$$

$$L_s(\phi 16) = 1.3\left(\frac{f_y}{2.1\sqrt{f'_c}}\right)d_b = 1.3\times\frac{420}{2.1\times\sqrt{28}}\times 16 = 780\text{ mm.}$$

Note: that the reinforcement of the slab is a mesh reinforcement. The bottom steel is splicing at the edges and the top steel is splicing at the middle of span.

Roof slab:

*** For 300mm:**

Top- Dir.1

A_{smax} in Roof (300mm slab) = 1150 mm^2

$A_{s,min}$= 0.0018×1000×300 =540 mm^2

Use 1Φ14/100mm

Bottom Dir.1

As,max =1415 mm^2

$A_{s,min}$= 0.0018×1000×250 =450 mm^2

$A_{s,min}$= 0.0018×1000×300 =540 mm^2

Use 1Φ14/100mm

Table3. 39:Reinforcement for the slabs.

Floor	Bottom steel in direction 1	Top steel in direction 1	Bottom steel in direction 2	Top steel in direction 2
Basement one and two	Ø 12 / 250 mm.	Ø 12 / 250 mm.	Ø 12 / 250 mm.	Ø 12 / 250 mm.
GF-F3	Ø 14 / 100 mm.	Ø 14 / 100 mm.	Ø 14 / 100 mm.	Ø 14 / 100 mm.

General reinforcement for the slab shown in Figure **3.102**

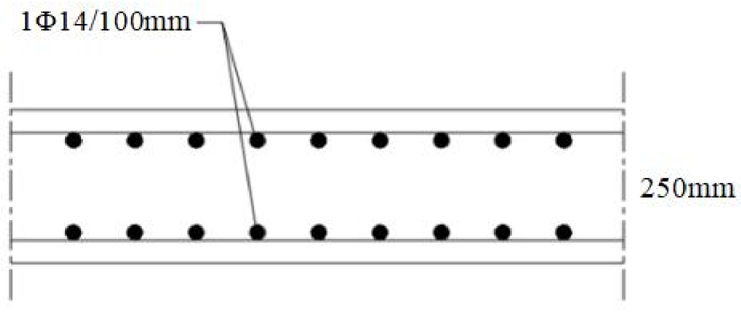


Figure3. 102 General reinforcement for the slab.

3.8 Design of beams:

According to ACI 318-14, there is a set of specifications and conditions that must apply to beams in order to be designated as special beams They will all be mentioned in this section .

❖ **Dimensions limits :**

- Clear span ℓn shall be at least 4d .
- Width b_w shall be at least the lesser of 0.3h and 250 mm.
- Projection of the beam width beyond the width of the supporting column on each side shall not exceed the lesser of c_2 and $0.75c_1$.

Where :

- C_1: dimension of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined.
- C_2 :dimension of rectangular or equivalent rectangular column, capital, or bracket measured in the direction perpendicular to c1.

❖ **Longitudinal reinforcement :**

Beams shall have at least two continuous bars at both top and bottom faces. At any section, for top as well as for bottom reinforcement and the reinforcement ratio ρ shall not exceed 0.025 for Grade 420 reinforcement and 0.02 for Grade 550 reinforcement .

❖ **Splicing :**

Lap splices shall not be used in these locations :

- Within the joints.
- Within a distance of twice the beam depth from the face of the joint.
- Within a distance of twice the beam depth from critical sections where flexural yielding is likely to occur as a result of lateral displacements beyond the elastic range of behavior.

❖ **Transverse reinforcement:**

The first hoop shall be located not more than 50 mm from the face of a supporting column.

Hoops shall be provided in the following regions of a beam:

- Over a length equal to twice the beam depth measured from the face of the supporting column toward mid span, at both ends of the beam.
- Over lengths equal to twice the beam depth on both sides of a section where flexural yielding is likely to occur as a result of lateral displacements beyond the elastic range of behavior.

Spacing of the hoops shall not exceed the least of:

- $\frac{d}{4}$.
- 150 mm.
- $6d_b$, Six times the diameter of the smallest primary flexural reinforcing bars .

➢ Where hoops are not required, stirrups with seismic hooks at both ends shall be spaced at a distance not more than d/2 throughout the length of the beam.

➢ Maximum spacing between bars restrained by legs of crossties or hoops =350mm.

➢ Maximum spacing between bars not restrained by legs of hoops =150 mm.

➢ Extension of hoops = $6d_b > 75$ mm .

❖ **development length :**

When clear spacing of bars being developed or lap spliced at least 2db and clear cover at least d_b :

- When clear spacing of bars at least d_b and clear cover at least d_b :

*For bars less than 19 mm:

$$L_{dt} = \left(\frac{f_y(\psi_t \psi_e \psi_g)}{2.1\lambda\sqrt{f'_c}} \right) d_b = \left(\frac{f_y \times 1 \times 1 \times 1}{2.1 \times 1 \times \sqrt{f'_c}} \right) d_b = \left(\frac{f_y}{2.1\sqrt{f'_c}} \right) d_b$$

*For bars equals or larger than 22 mm:

$$L_{dt} = \left(\frac{f_y(\psi_t \psi_e \psi_g)}{1.7\lambda\sqrt{f'_c}} \right) d_b = \left(\frac{f_y \times 1 \times 1 \times 1}{1.7 \times 1 \times \sqrt{f'_c}} \right) d_b = \left(\frac{f_y}{1.7\sqrt{f'_c}} \right) d_b$$

- Other cases:

*For bars less than 19 mm:

$$L_{dt} = \left(\frac{f_y(\psi_t \psi_e \psi_g)}{1.4\lambda\sqrt{f'_c}} \right) d_b = \left(\frac{f_y \times 1 \times 1 \times 1}{1.4 \times 1 \times \sqrt{f'_c}} \right) d_b = \left(\frac{f_y}{1.4\sqrt{f'_c}} \right) d_b$$

*For bars equals or larger than 22 mm:

$$L_{dt} = \left(\frac{f_y(\psi_t \psi_e \psi_g)}{1.1\lambda\sqrt{f'_c}} \right) d_b = \left(\frac{f_y \times 1 \times 1}{1.1 \times 1 \times \sqrt{f'_c}} \right) d_b = \left(\frac{f_y}{1.1\sqrt{f'_c}} \right) d_b$$

- Development length ℓd for deformed bar shall be ≥ 300 mm.
- Development lengths should be increased by 30% for top steel bars when d ≥ 300 mm.

❖ **Flexure:**

b = 400 mm

h = 600 mm

d = 540 mm

Span 1 :

Return to Figure **3.103** :

As Bottom = 995mm^2

As Top1 = 1542 mm^2

As Top2 = 1487 mm^2

$A_{s,\,min}$ = 0.00333×400×540 = 712.8 mm^2

$\rho_{max.singly}$ = 0.025 for special reinforcement.

$A_{s,\,max}$ = 0.025*400*540 = 5400 mm^2

1542 799 1487

1043 995 1026

Figure3. 103 As Bottom and As Top2 in Beam B1.

❖ **Shear :**

$\frac{A_v}{S}$ = 1235.5 mm^2/m from ETABS as shown in Figure **3.104** :

1235.49 328.32 392.83

Figure3. 104 Av/S in Beam B1.

❖ **Longitudinal Reinforcement:**

Bottom Steel:

As =995mm^2 $\rightarrow$ use 4Ø18.

Top1 Steel:

As = 1542 mm^2 $\rightarrow$ use 6Ø18.

Top2 Steel:

As = 1487 mm^2 $\rightarrow$ use 6Ø18.

❖ **The development length:**

for d_b = 18 mm , $L_{dt} = \left(\frac{f_y}{2.1\sqrt{f'_c}}\right)d_b$

$$L_{dt} = \left[\frac{420}{2.1\times\sqrt{28}}\right]\times 18 = 680\text{mm}$$

L_{dt} in top steel = 1.3×1.3×680=1150mm .

❖ **Transverse reinforcement :**

Assume one stirrup Ø12

A_v = 2×113 = 226 mm^2

S = 226/(1235.5 /1000)= 183 mm

spacing between hoops :

≤ 540/4=135mm

≤150 mm

≤6*18= 108 mm.

Select (s) = 100 mm over a length equal to twice depth of beam (2h)or fourth length of the beam(L_n/4) which is larger.

2h =2*600=1200 mm

Ln/4 =6930/4 =1432.5mm

Use 1Ø12 each 100mm from 1750 mm from the joint.

In other places select (S) 250 mm.

Spacing between top bars = (400−(2*60+18))/5= 52.4 mm <150 mm

Spacing between two legs = 400-(2*60) = 280 <350 mm

❖ Extension of hoops = 110 mm

Figure **3.105** shows Beam $B_{11}(F_2)$ with longitudinal and transverse reinforcement.

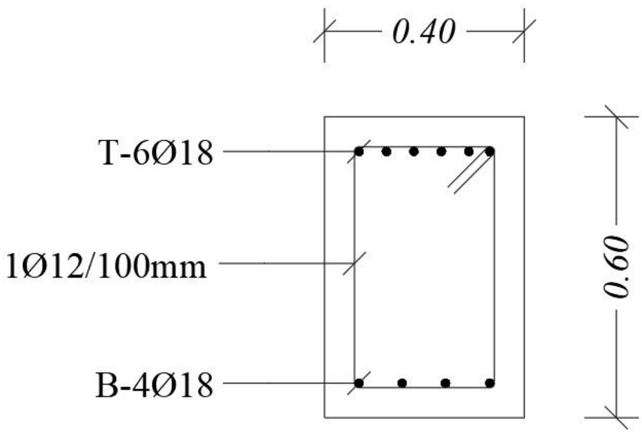


Figure3. 105 :Cross section in beam .

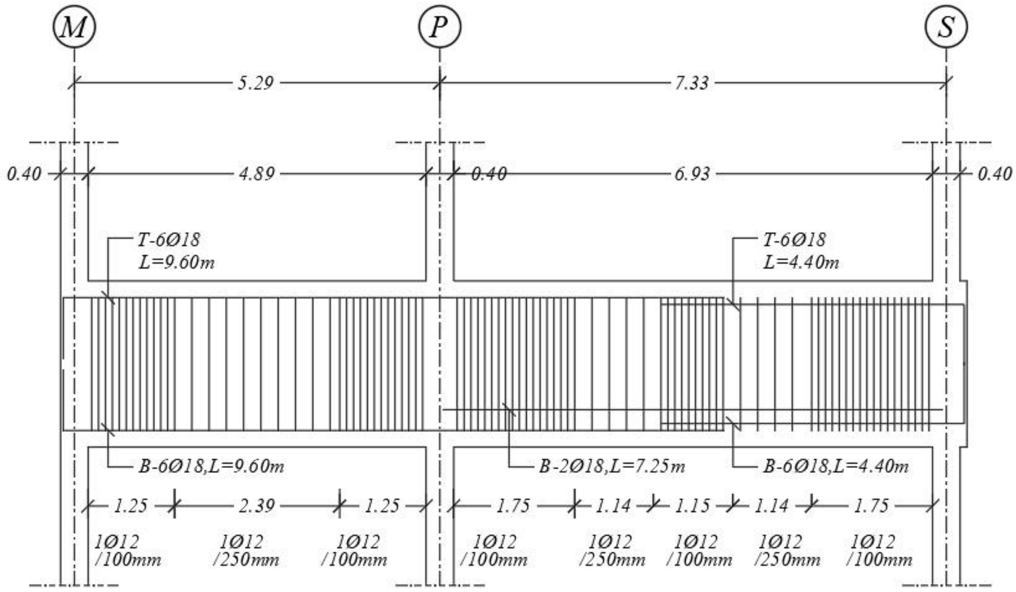


Figure3. 106:Longitudinal section in beam.

3.9 Design of columns:

• **The requirements for the special moment resisting frames according to ACI 318-19:**

❖ **Dimensional limits :**

Columns shall satisfy the following conditions:

- The least dimension of column cross section ≥ 300 mm.
- The ratio of the shortest cross-sectional dimension to the perpendicular dimension shall be ≥ 0.4 .

❖ **Minimum flexural strength of columns:**

The flexural strengths of the columns shall satisfy:

$\sum M_{nc} \geq (6/5)\sum M_{nb}$

$\sum M_{nc}$: is sum of nominal flexural strengths of columns framing into the joint.

$\sum M_{nb}$: is sum of nominal flexural strengths of the beams framing into the joint.

- ❖ **Longitudinal reinforcement :**

- Area of longitudinal reinforcement, A_{st}, shall be at least $0.01A_g$ and shall not exceed $0.06A_g$.
- In columns with circular hoops, there shall be at least six longitudinal bars.

- ❖ **Transverse reinforcement :**

Transverse reinforcement is accomplished by using rectilinear hoops or circular ones, but it must satisfy the following conditions to be as lateral support for longitudinal bars :

- Every hoop corner that supports a longitude bar has to make angle not exceeding 135 degrees .
- The spacing between any unsupported bar and supported one must be ≤ 150mm.
- The clear distance between any leg of the hoops and the another (hx) must be ≤ 350 mm.

➢ If $P_u > 0.3A_g f_c'$ or $f_c' > 70$ MP a columns with rectilinear hoops, every bar around the perimeter of the core of the column-cross section shall have lateral hook ,and the value of h_x shall not exceed 200 mm.

➢ P_u shall be the largest value in compression consistent with factored load combinations including E.

➢ Transverse reinforcement shall extend over a length ℓo from each joint face and on both sides of any section where flexural yielding is likely to occur .

ℓo = max of :

- The depth of the column at the joint face or at the section where flexural yielding is likely to occur .
- One-sixth of the clear span of the column .
- 450 mm .

- ❖ **Transverse reinforcement spacing:**

Spacing of transverse reinforcement shall not exceed the smallest of:

- One-fourth of the minimum column dimension .
- For Grade 420, $6d_b$ of the smallest longitudinal bar .

- so, as calculated by:

$$S_0 = 100 + \left[\frac{350 - h_x}{3}\right]$$

*h_x value of so shall not exceed 150 mm and need not be taken less than 100 mm .

❖ **Amount of transverse reinforcement:**

Table 18.7.5.4—Transverse reinforcement for columns of special moment frames

Transverse reinforcement	Conditions	Applicable expressions	
A_{sh}/sb_c for rectilinear hoop	$P_u \le 0.3A_g f_c'$ and $f_c' \le 70$ MPa	Greater of (a) and (b)	$0.3\left(\frac{A_g}{A_{ch}}-1\right)\frac{f_c'}{f_{yt}}$ (a)
	$P_u > 0.3A_g f_c'$ or $f_c' > 70$ MPa	Greatest of (a), (b), and (c)	$0.09\frac{f_c'}{f_{yt}}$ (b)
			$0.2k_f k_n \frac{P_u}{f_{yt}A_{ch}}$ (c)
ρ_s for spiral or circular hoop	$P_u \le 0.3A_g f_c'$ and $f_c' \le 70$ MPa	Greater of (d) and (e)	$0.45\left(\frac{A_g}{A_{ch}}-1\right)\frac{f_c'}{f_{yt}}$ (d)
	$P_u > 0.3A_g f_c'$ or $f_c' > 70$ MPa	Greatest of (d), (e), and (f)	$0.12\frac{f_c'}{f_{yt}}$ (e)
			$0.35k_f \frac{P_u}{f_{yt}A_{ch}}$ (f)

Where:

- A_g = gross area of concrete section, for a hollow section, Ag is the area of the concrete only and does not include the area of the void(s).
- A_{ch} = cross-sectional area of a member measured to the outside edges of transverse reinforcement .
- A_{sh} = total cross-sectional area of transverse reinforcement, including crossties, within spacing s and perpendicular to dimension bc .
- b_c = cross-sectional dimension of member core measured to the outside edges of the transverse reinforcement composing area A_{sh}.

- $n\ell$ = number of longitudinal bars around the perimeter of a column core with rectilinear hoops that are laterally supported by the corner of hoops or by seismic hooks .
- k_f = concrete strength factor .
- k_n = confinement effectiveness factor .
- h_x = maximum center-to-center spacing of longitudinal bars laterally supported by corners of crossties or hoop legs around the perimeter of the column.

➢ Bars should splice when needed at mid height of the column with special transverse reinforcement spacing:

S = min of:

- 6 of smallest bar diameter .
- 150 mm.

❖ Minimum flexural strength of columns checks:

A sample calculations was done on column C33 instoryB1 and story GF.

The flexural strengths of the columns shall satisfy:

$$\sum M_{nc} \geq \frac{6}{5}\sum M_{nb}$$

M_n beam:

$$M_n = A_s F_y (d - \frac{a}{2})$$

$$a = \frac{A_s F_y}{0.85\, f_c'\, b}$$

$\sum M_{n1} = \sum M_{n2} = 2\times709\times420\times(540\text{-}31.28/2) = 312.28$ KN.m

$(6/5)\sum M_{nb} = 1.2\times312.736$

$= 374.736$ KN.m

M_n of Column:

P_u on B1=4437.5KN

P_u on GF= 4161.5 KN

Interaction diagram for both is shown below:

Mn(B1) =1675 KN.m

Mn(GF) =1600 KN.m

∑Mnc =1600+1675=3572KN.m

→ ∑Mnc >(6/5)∑Mnb, OK

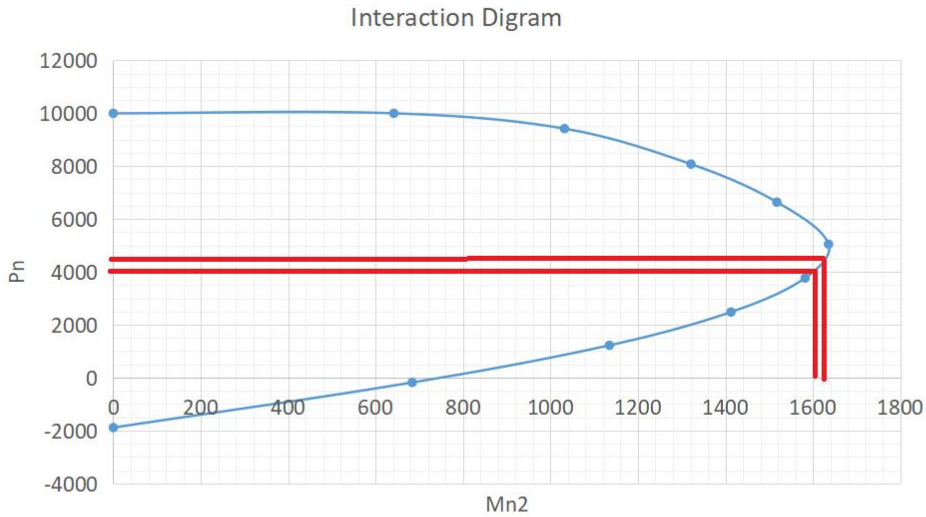


Figure3. 107:Interaction diagram.

❖ A sample of flexure, shear design calculations was done on column C80 in story B1 . Other columns' design results were calculated and drawn on detailed drawings.

From ETABS :

Column Element Details

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	LLRF	Type
B1	C142	322	500*350 B	U7.2Dyn	3400	4000	1	Sway Special

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
350	500	60	27.3

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
24870.06	28	1	420	420

Design Code Parameters

Φ_T	Φ_{CTied}	$\Phi_{CSpiral}$	Φ_{Vns}	Φ_{Vs}	Φ_{Vjoint}	Ω_0
0.9	0.65	0.75	0.75	0.6	0.85	2.5

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

Design P_u kN	Design M_{u2} kN-m	Design M_{u3} kN-m	Minimum M2 kN-m	Minimum M3 kN-m	Rebar Area mm^2	Rebar % %
-168.0378	84.338	1.1504	4.3253	5.0815	2252	1.29

Axial Force and Biaxial Moment Factors

	C_m Factor Unitless	δ_{ns} Factor Unitless	δ_s Factor Unitless	K Factor Unitless	Length mm
Major Bend(M3)	1	1	1	1	3400
Minor Bend(M2)	1	1	1	1	3400

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

	Shear V_u kN	Shear ϕV_c kN	Shear ϕV_s kN	Shear ϕV_p kN	Rebar A_v /s mm^2/m
Major, V_{u2}	48.4432	83.0126	39.8174	48.4427	287.28
Minor, V_{u3}	48.4463	78.1614	37.4905	48.4427	410.4

Figure3. 108 ETABS result for design.

$$P_{u,limit} = 0.3A_g f'_c$$
$$= 0.3 \times 500 \times 350 \times 28/1000$$
$$= 1470 \text{ KN}$$

P_u =168 KN < $P_{u,limit}$

Assume using 10 bar around the perimeter:

h_x=(500 − 2×40 − 2 ×12 − 32)/3 = 121.3 mm < 200mm,OK

❖ **Flexure reinforcement:**

A_S from ETABS =2252 mm^2 , Use 12Ø18

- ❖ **Transverse reinforcement:**

Table 18.7.5.4—Transverse reinforcement for columns of special moment frames

Transverse reinforcement	Conditions		Applicable expressions
A_{sh}/sb_c for rectilinear hoop	$P_u \le 0.3A_g f_c'$ and $f_c' \le 70$ MPa	Greater of (a) and (b)	$0.3\left(\frac{A_g}{A_{ch}}-1\right)\frac{f_c'}{f_{yt}}$ (a)
	$P_u > 0.3A_g f_c'$ or $f_c' > 70$ MPa	Greatest of (a), (b), and (c)	$0.09\frac{f_c'}{f_{yt}}$ (b) $0.2k_f k_n \frac{P_u}{f_{yt}A_{ch}}$ (c)
ρ_s for spiral or circular hoop	$P_u \le 0.3A_g f_c'$ and $f_c' \le 70$ MPa	Greater of (d) and (e)	$0.45\left(\frac{A_g}{A_{ch}}-1\right)\frac{f_c'}{f_{yt}}$ (d)
	$P_u > 0.3A_g f_c'$ or $f_c' > 70$ MPa	Greatest of (d), (e), and (f)	$0.12\frac{f_c'}{f_{yt}}$ (e) $0.35k_f \frac{P_u}{f_{yt}A_{ch}}$ (f)

$P_u < P_{u,limit}$

$A_g = 500\times350 = 17500 \text{ mm}^2$

$A_{ch} = 420\times270 = 113400 \text{ mm}^2$

$$A_{sh}/sb_c = Max\begin{pmatrix} 0.3\times(\frac{175000}{113400}-1)\times\frac{28}{420} = 0.011 \\ 0.09\times\frac{28}{420} = 6\times10^{-3} \end{pmatrix} = 0.011$$

$Ash = 0.25\times \pi \times 12^2 \times 4$

$= 452.4 \text{ mm}^2$

$A_{sh}/sb_c = 452.4/s \times 378 = 0.011$

S= 108.8 mm

Use s=100 mm

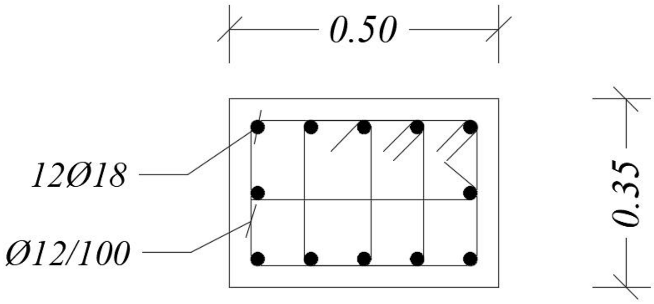


Figure3. 109 Column 80 cross section.

3.10 Design of walls:

All shear walls are special reinforced shear walls , and have a varies thickness.

Other walls design results were calculated and drawn on detailed drawings.

According to ACI 318-19 There is a set of specifications and conditions that must apply to walls in order to be designated as special wall They will all be mentioned in this section .

❖ **Dimensions limits :**

Shear wall thickness ≥ l/16

≥ 250 mm

❖ **Shear strength :**

- Walls shall have distributed shear reinforcement,If h_w/ℓ_w does not exceed 2.0, reinforcement ratio ρ_ℓ shall be at least the reinforcement ratio ρ_t which equals 0.0025 .
- At least two curtains of reinforcement shall be used in a wall if $V_u > 0.17\lambda\sqrt{f_c}\ A_{cv}$ or $h_w/\ell_w \geq 2.0$, in which hw and ℓ_w refer to height and length of entire wall, respectively.
- For all vertical wall segments sharing a common lateral force, V_n shall not be taken greater than $0.66\sqrt{f_c}\ A_{cv}$.
- For any one of the individual vertical wall segments, V_n shall not be taken greater than $0.83\sqrt{f_c}\ A_{cw}$.
- **Longitudinal reinforcement:**
- The distributed web reinforcement ratios, ρ_l and ρ_t, for structural walls shall be at least 0.0025.

- Spacing between bars must be ≤ 450 mm.
- No cut off bars .

Where :

ρ_l :ratio of area of distributed longitudinal reinforcement to gross concrete area perpendicular to that reinforcement.

ρ_t : ratio of area of distributed transverse reinforcement to gross concrete area perpendicular to that reinforcement.

❖ **Splicing:**

Reinforcement in structural walls shall be developed or spliced for fy in tension in :

- longitudinal reinforcement shall extend at least 3.6 m above the point at which it is no longer required to resist flexure but need not extend more than ℓd above the next floor level, Except at the top of a wall.
- At locations where yielding of longitudinal reinforcement is likely to occur as a result of lateral displacements, development lengths of longitudinal reinforcement shall be 1.25 times the values calculated for fy in tension.

$L_{splice} = L_{Developed} \times 1.3 \times 1.55$

❖ **Boundary elements of special structural walls:**

- shall have special boundary elements at boundaries and edges around openings of structural walls where the maximum extreme fiber compressive stress, corresponding to load combinations including earthquake effects E, exceeds $0.2f_c'$.

$\sigma c \geq 0.2f_c'$, need Boundary elements .

- The special boundary element shall be permitted to be discontinued where the calculated compressive stress is less than $0.15f_c'$.

$\sigma c \leq 0.15f_c'$, no need Boundary elements .

- ❖ **Dimensions limits for Boundary elements** :

- The boundary element shall extend horizontally from the extreme compression fiber a distance at least the greater of $c - 0.1\ell_w$ and c/2 .
- Width of the flexural compression zone, b, shall be at least $h_u/16$.
- If the ratio $C/L_w > 3/8$, the width of the boundary zone must be at least 300 mm .

Where :

C: is the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with δ_u.

L_w : Length of vertical wall .

h_u : Clear height of vertical wall.

- ❖ **The boundary element transverse reinforcement:**

- Every hoop corner that supports a longitude bar has to make angle not exceeding 135 degrees .
- The spacing between any unsupported bar and supported one must be ≤ 150mm .
- Transverse reinforcement shall be arranged such that the spacing hx between laterally supported longitudinal bars around the perimeter of the boundary element shall not exceed the lesser of 350 mm and two-thirds of the boundary element thickness .

hx ≤ 350 mm

hx ≤ 2/3 × thickness of wall

Where :

hx : clear distance between any leg of the hoops and the another .

- The length of a hoop leg shall not exceed two times the boundary element thickness, and adjacent hoops shall overlap at least the lesser of 150 mm and two-thirds the boundary element thickness.

- The boundary element transverse reinforcement at the wall base shall extend into the support at least ℓd .
- Where the special boundary element terminates on a footing, mat, or pile cap, special boundary element transverse reinforcement shall extend at least 300 mm into the footing .

❖ **The boundary element transverse spacing limit:**

Spacing of transverse reinforcement shall not exceed the least of:

- One-third of the wall dimension.
- For Grade 420, 6db of the smallest longitudinal bar .
- S_o, as calculated by:

$$S_o = 100 + \frac{350-hx}{3} \qquad hx \leq 350$$

The value of s_o from shall not exceed 150 mm and need not be taken less than 100 mm.

❖ **Amount of transverse reinforcement:**

The amount of transverse reinforcement shall be in accordance with Table () :

Table 18.10.6.4(g)—Transverse reinforcement for special boundary elements

Transverse reinforcement	Applicable expressions		
A_{sh}/sb_c for rectilinear hoop	Greater of	$0.3\left(\frac{A_g}{A_{ch}}-1\right)\frac{f'_c}{f_{yt}}$	(a)
		$0.09\frac{f'_c}{f_{yt}}$	(b)
ρ_s for spiral or circular hoop	Greater of	$0.45\left(\frac{A_g}{A_{ch}}-1\right)\frac{f'_c}{f_{yt}}$	(c)
		$0.12\frac{f'_c}{f_{yt}}$	(d)

Where:

- Ag = gross area of concrete section, for a hollow section, Ag is the area of the concrete only and does not include the area of the void(s).

- Ach = cross-sectional area of a member measured to the outside edges of transverse reinforcement
- Ash = total cross-sectional area of transverse reinforcement, including crossties, within spacing s and perpendicular to dimension bc.
- bc = cross-sectional dimension of member core measured to the outside edges of the transverse reinforcement composing area Ash.

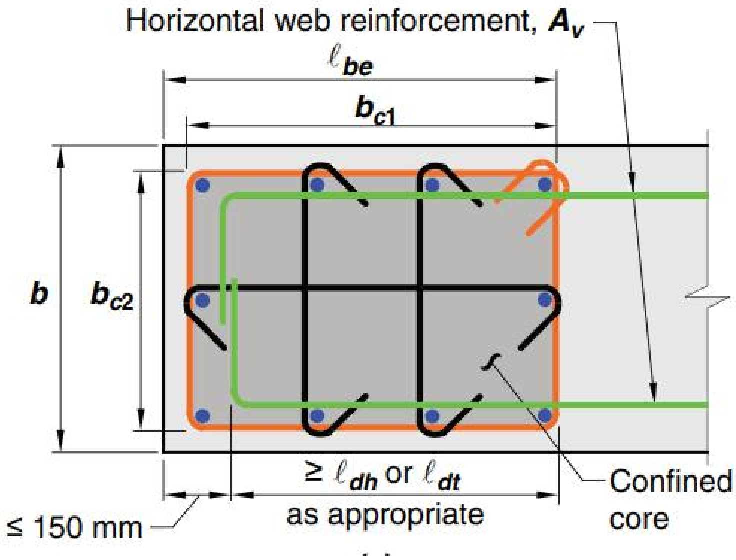


Figure3. 110: Illustration of transverse reinforcement in walls.

❖ **The development length :**

When Clear spacing of bars being developed or lap spliced at least 2db and clear cover at least d_b :

- $\dfrac{f_y}{2.1\times\sqrt{f'_c}}$ for bars less than 19 mm.
- $\dfrac{f_y}{1.7\times\sqrt{f'_c}}$ for bars equal or larger than 22 mm.

Other cases:

- $\dfrac{f_y}{1.4\times\sqrt{f'_c}}$ for bars less than 19 mm.
- $\dfrac{f_y}{1.1\times\sqrt{f'_c}}$ for bars equal or larger than 22 mm.

❖ **Longitudinal Reinforcement**:

A sample of flexure, shear design calculations was done on shear wall (W_9) - Basement 1 .

Flexural Design for P_u, M_{u2} and M_{u3}

Station Location	Required Rebar Area (mm^2)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P_u kN	M_{u2} kN-m	M_{u3} kN-m	Pier A_g mm^2
Top	3318	0.0025	0.0026	U16	1509.6976	42.7647	-244.1926	1327285
Bottom	3318	0.0025	0.0026	U16	1586.9201	-17.7231	-196.174	1327285

Shear Design

Station Location	ID	Rebar mm^2/m	Shear Combo	P_u kN	M_u kN-m	V_u kN	ϕV_c kN	ϕV_n kN
Top	Leg 1	625	U5.2Dyn	2090.4558	3861.4085	1015.9199	1049.7238	1885.9131
Bottom	Leg 1	625	U5.2Dyn	2256.8264	3793.0652	1016.0557	1049.7238	1885.9131

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	413.9	U5.2Dyn	2907.6766	-4945.1951	6.4	5.6	827.8	1179.8
Top–Right	Leg 1	413.9	U5.2Dyn	2907.6766	3861.4085	5.48	5.6	827.8	1179.8
Bottom–Left	Leg 1	424.3	U5.2Dyn	3010.881	-4255.3329	5.89	5.6	848.5	1179.8
Botttom–Right	Leg 1	424.3	U5.2Dyn	3010.881	3793.0652	5.5	5.6	848.5	1179.8

Figure3. 111 :ETABS result for design.

As from ETABS for Flexural design =3318 mm^2, Use Ø10/250 mm.

As from ETABS for shear design = 625 mm^2, Use Ø10/250 mm.

❖ Reinforcement for boundary element:

Top-Left = 413.9 mm

Top-Right = 413.9 mm

Bottom-Left = 424.3 mm

Bottom - Right = 424.3 mm

Use boundary element= 450 mm

❖ **Development Lengths:**

- $\frac{f_y}{2.1\times\sqrt{f_c'}} = \frac{420}{2.1\times\sqrt{28}} = 377.9$ mm, use $L_{dt} = 400$ mm.

❖ **Transverse reinforcement :**

hx= (250- 2×40 - 2×12 - 10)/3

$= 45.5$ mm < 350

$< \frac{2}{3} \times 250$

S_{max} min of :

- $0.333 \times 250 = 83.4$ mm .
- $6 \times Ø$ smallest longitudinal bars $= 6 \times 10 = 60$ mm .
- $S_o = 100$.

$A_{sh} = \pi \times 0.25 \times d^2 \times n = \pi \times 0.25 \times 10^2 \times 4 = 314.2$ mm^2

$b_c = 250-100 = 150$ mm

$A_g = 250 \times 5300 = 1325 \times 10^3$ mm^2 .

$A_{ch} = 200 \times 5250 = 1050 \times 10^3$ mm^2 .

A_{sh} /S b_c = max of :

- $0.3 \times (\frac{A_g}{A_{ch}} - 1) \times \frac{f'_c}{f_y} = 0.3 \times (\frac{1325}{1050} - 1) \times \frac{28}{420} = 0.0053$
- $\frac{0.09\, f'_c}{f_y} = \frac{0.09 \times 28}{420} = 6 \times 10^{-3}$

A_{sh} /S b_c $= 6 \times 10^{-3}$

$S = 314.2/(6 \times 10^{-3} \times 150) = 350$ mm$> S_{max}$=60mm.

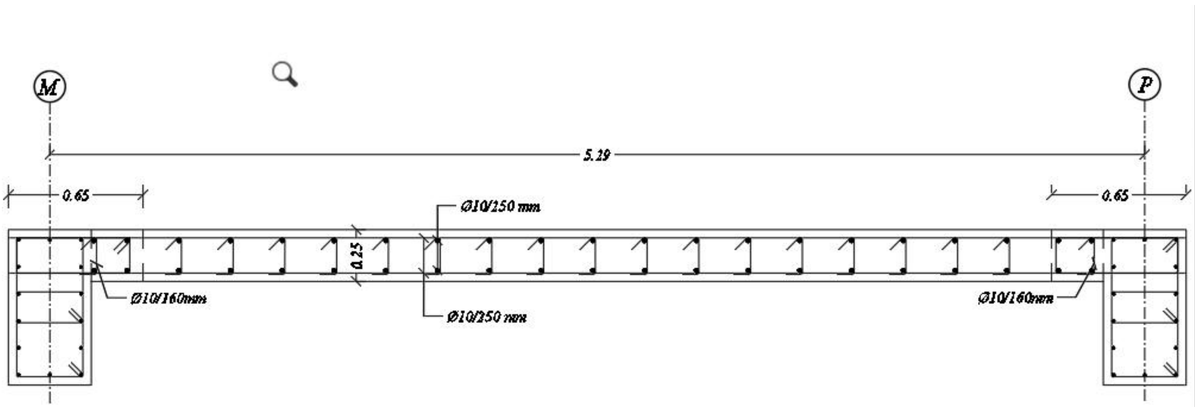


Figure3. 112 Wall 12 cross section.

3.11 Design of Footing:

This section includes the analysis and the design of the foundation system as a whole including the piles and matt foundation section using CSI SAFE 20.3 software.

3.11.1 Modelling:

Units: main three units used are meter (m) for length, kilo newton (KN) for force and Celsius (C) for temperature.

Materials: it was discussed in section 1.5, but the definition of these materials on software program is shown in Figure3.

- Concrete:

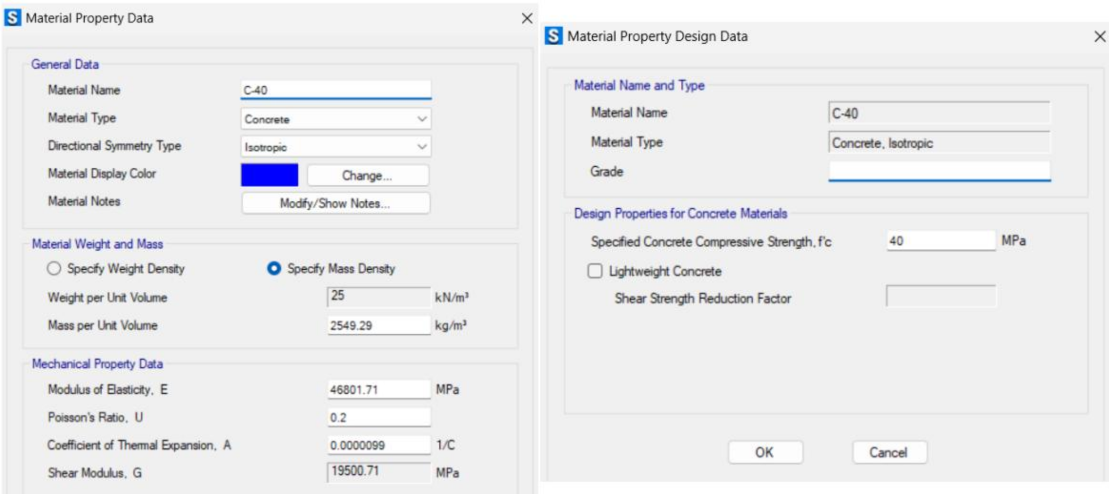


Figure3. 113: Concrete40 MPa definition in SAFE

Steel:

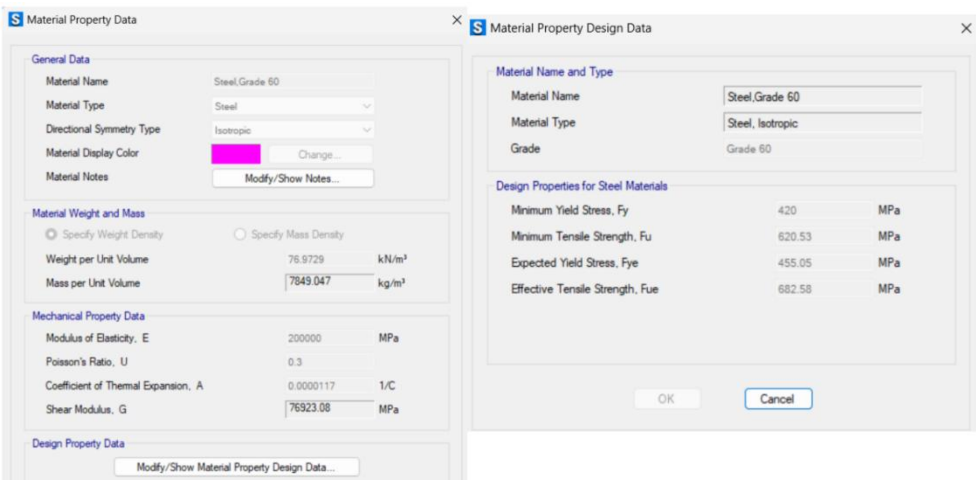


Figure3. 114 :Steel definition in SAFE.

Footing properties: the definition on software program will be as shown in **Figure3.114.**

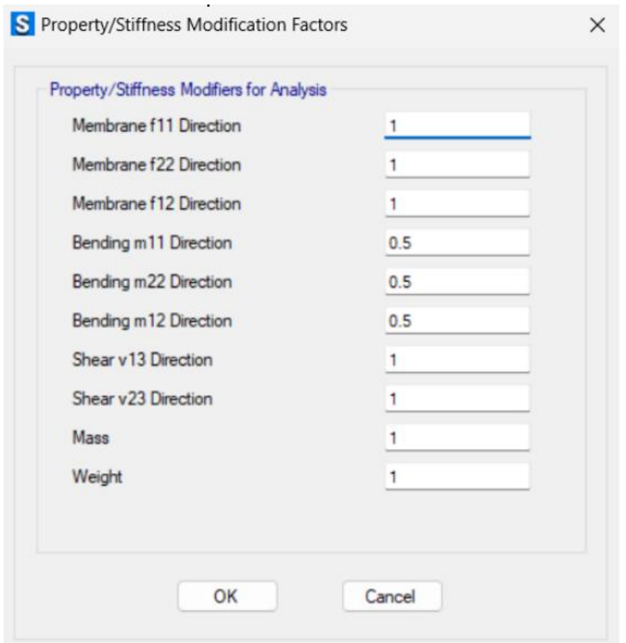


Figure3. 115 :footing modifiers in safe .

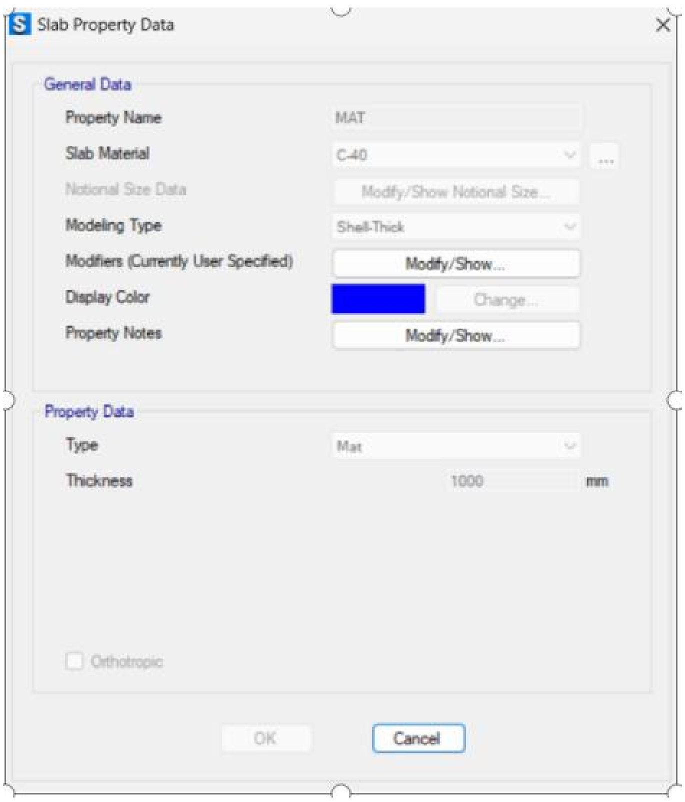


Figure3. 116 :footing definition in SAFE.

Soil properties:

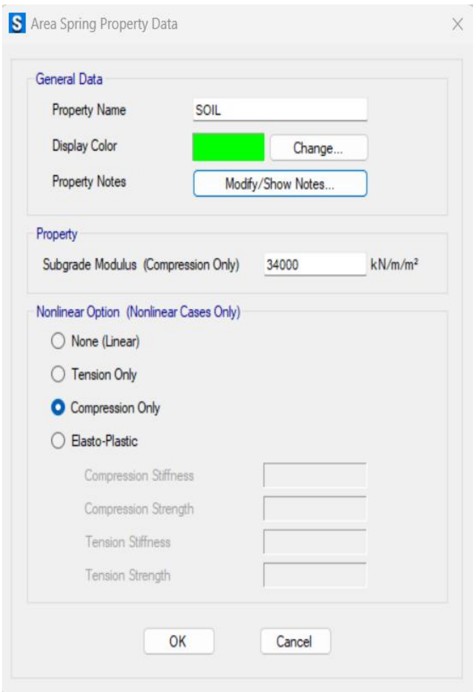


Figure3. 117 :Soil definition in SAFE.

Try mat thickness = 1000 mm

3.11.1 Check wide beam shear(One-way shear):

Maximum shear (at distance d from face of wall) 3430.4= KN/m, as shown in **Figure3.118**.

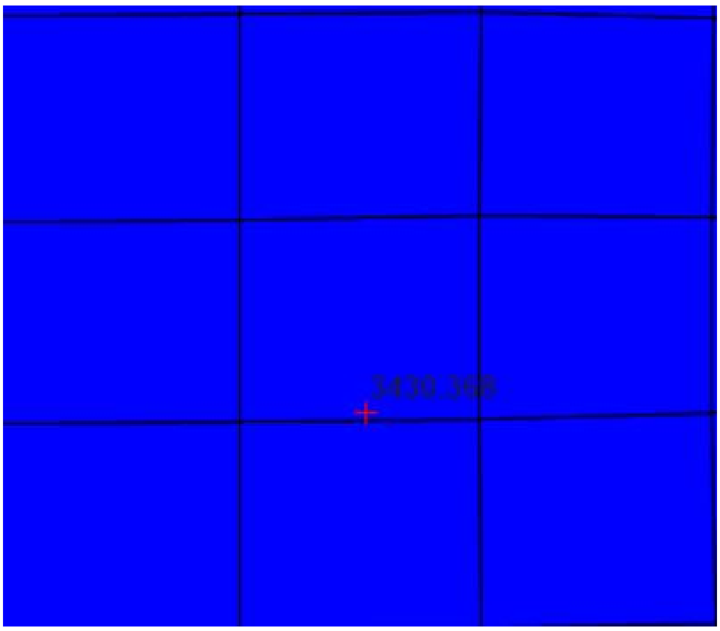


Figure3. 118 :One-way shear in footing

$$\phi V_c = \phi(0.17\lambda\sqrt{f'_c} + \frac{Nu}{6A_g})b_w d$$

Where:

$\emptyset = 0.75$

$\lambda = 1$

f'_c: specified compressive strength of concrete, MPa(its equal to 40 MPa)

b_w :web width = 1000 mm

d :effective depth = 900 mm

Nu: factored axial force normal to cross section occurring simultaneously with *Vu* or *Tu*; to be taken as positive for compression and negative for tension, N

$\emptyset V_c$=725 KN/m

Vu=3430.4 < $5\emptyset V_C$=3625, OK

$V_u > \emptyset V_c$ (there is a need for shear reinforcement)

Shear reinforcement is needed in blue zones, where V_u larger than $\emptyset V_c$, **Figure3.119 shows shear forces in x direction from SAFE**…, expected zones around columns.

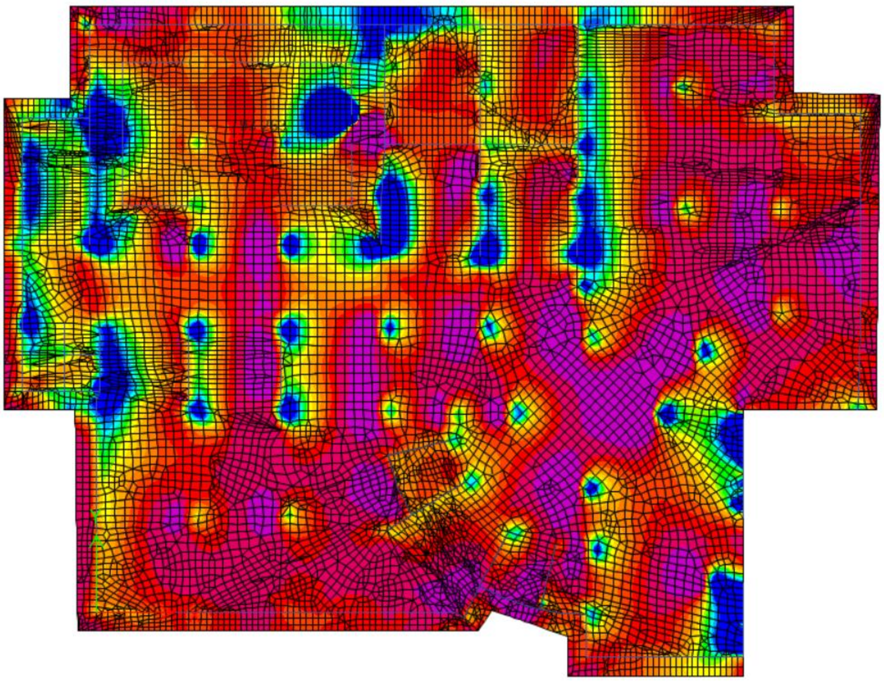


Figure3. 119 :shear forces in x direction .

Sample of calculations:

$V_n = V_c + V_s$

3430.4= 725/0.75+Vs

3430.4= 967.65 +Vs

Vs = 2463 KN

$$0.33\sqrt{f_c'}\, b_w d = 0.33\sqrt{40} \times 1000 \times \frac{900}{1000}$$

$$= 1878.4.9$$

$$Vs > 0.33\sqrt{f_c'}\, b_w d$$

$$\rightarrow S_{max} \leq \begin{bmatrix} d/4 = 175mm & \text{Along length} \\ \text{d/2} = 350 & \text{Across width} \\ 300 & \end{bmatrix}$$

$$\frac{Av}{S} = \frac{Vs}{f_{yt} d}$$

$$= \frac{2463 \times 1000}{420 \times 900}$$

$$= 6.5\ \text{mm}^2/\text{mm}$$

Use 1Ø32/100mm.

3.11.2 Check punching shear (Two-way shear):

Maximum punching shear =0.46, as shown in **Figure3.120**.

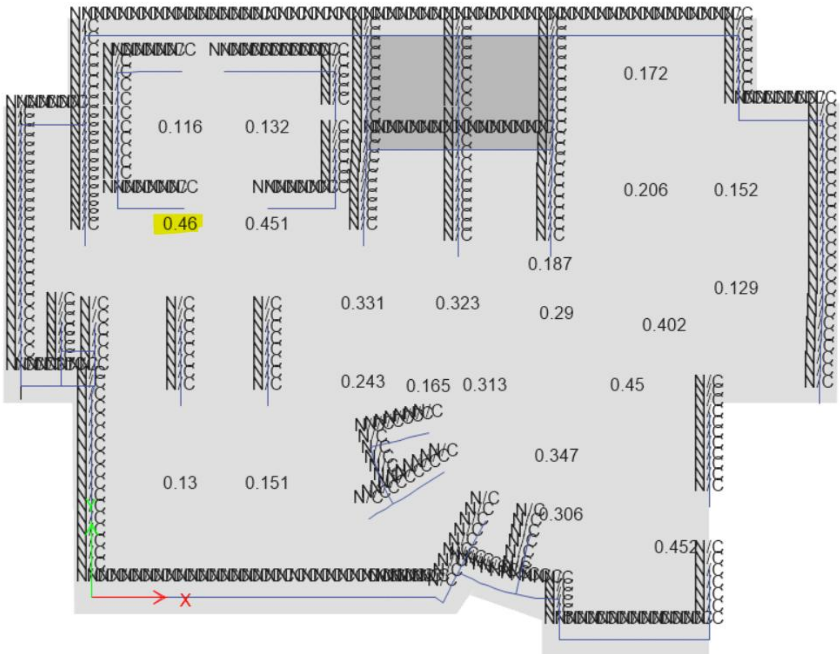


Figure3. 120 :punching shear in footing.

Max.Punching shear demand /capacity ration equal to 0.46 < 1 , OK

$$\phi V_c \leq \phi 0.33 \lambda_s \lambda \sqrt{f'_c}\, b_o d$$

$$\phi V_c \leq \phi 0.17 \lambda_s \lambda (1 + \frac{2}{\beta}) \sqrt{f'_c}\, b_o d$$

$$\phi V_c \leq \phi 0.083 \lambda_s \lambda (2 + \frac{\alpha_s d}{b_o}) \sqrt{f'_c}\, b_o d$$

Where:

$V_{u,p}$: ultimate punching shear (KN);

$\emptyset V_{c,p}$: punching shear capacity for footing (KN);

β = long side / short side of column = 0.9/0.5=1.8;

αs = factor to consider column location = 40;

b_o=perimeter length of the critical section (at d/2 from face of column) = 6400 mm; and

d = effective depth of footing =900 mm

$$\gamma_s = \sqrt{\frac{2}{(1+0.004d)}} \leq 1.0$$

$\gamma_s = 0.66$

$\rightarrow \emptyset V_{c,p}$= 9016.3 KN

From SAFE $V_{u,p}$ =3966.75 KN < $\emptyset V_{c,p}$, OK

$V_{u,p}/\emptyset V_{c,p}$= 0.44

Different percentage = (0.44-0.46)/0.44

= 4.55% OK

3.11.3 Check stresses under footings

This check will be made on mat foundation, to ensure that stresses under footings are less than bearing capacity and there is no tension under footings, so that no need for non-linear uplift analysis.

Allowable bearing capacity for the soil = 350 KN/m^2

Figure3.121 shows tension in the footing equal to 285 KN/m, then non-linear analysis is needed.

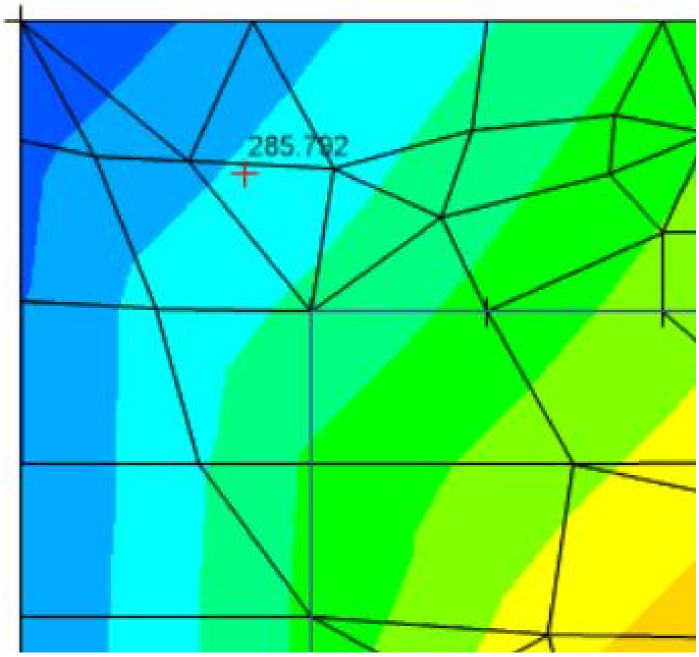


Figure3. 121 :stress in the footing .

*** Non-linear analysis result:**

Max. Stress equal to 530 KN/m^2 which is larger than bearing capacity (300KN/m^2), so piles will be used in this area .

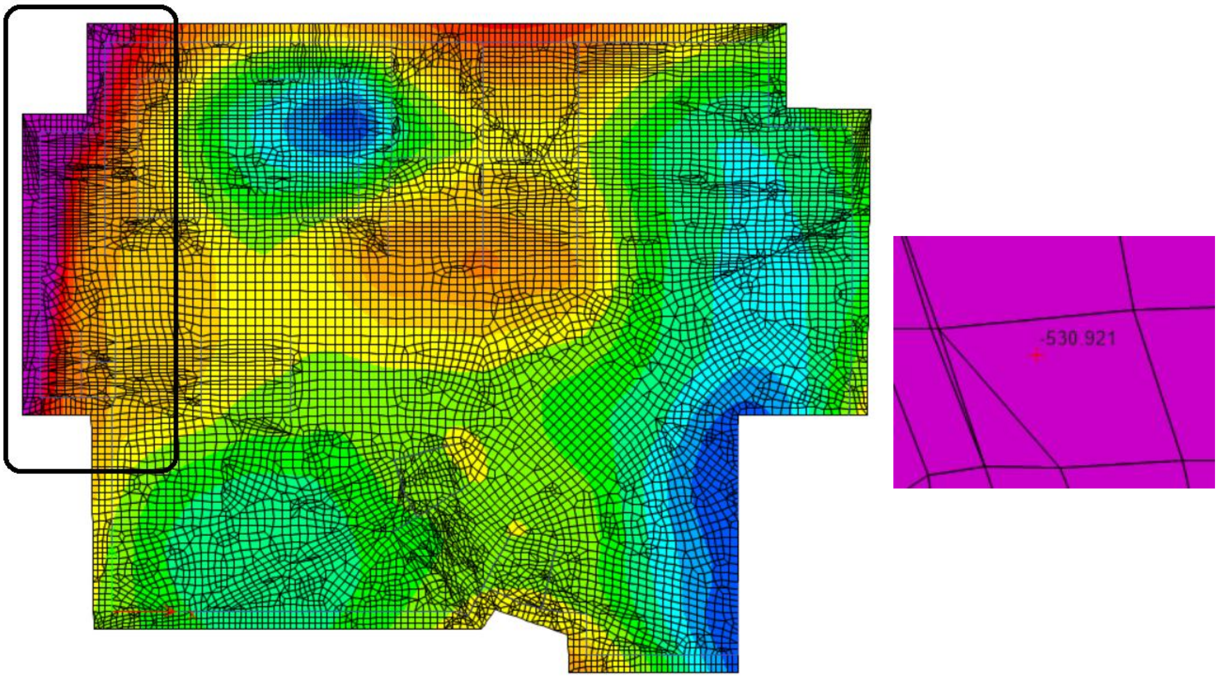


Figure3. 122 Maximum stress under footing.

Piles definition in SAFE:

A 0.8 m diameter pile with two different depths was used, Piles 8 m deep and 12 m deep. Deeper piles have been used in the central area where loading is higher, so larger capacities are required.

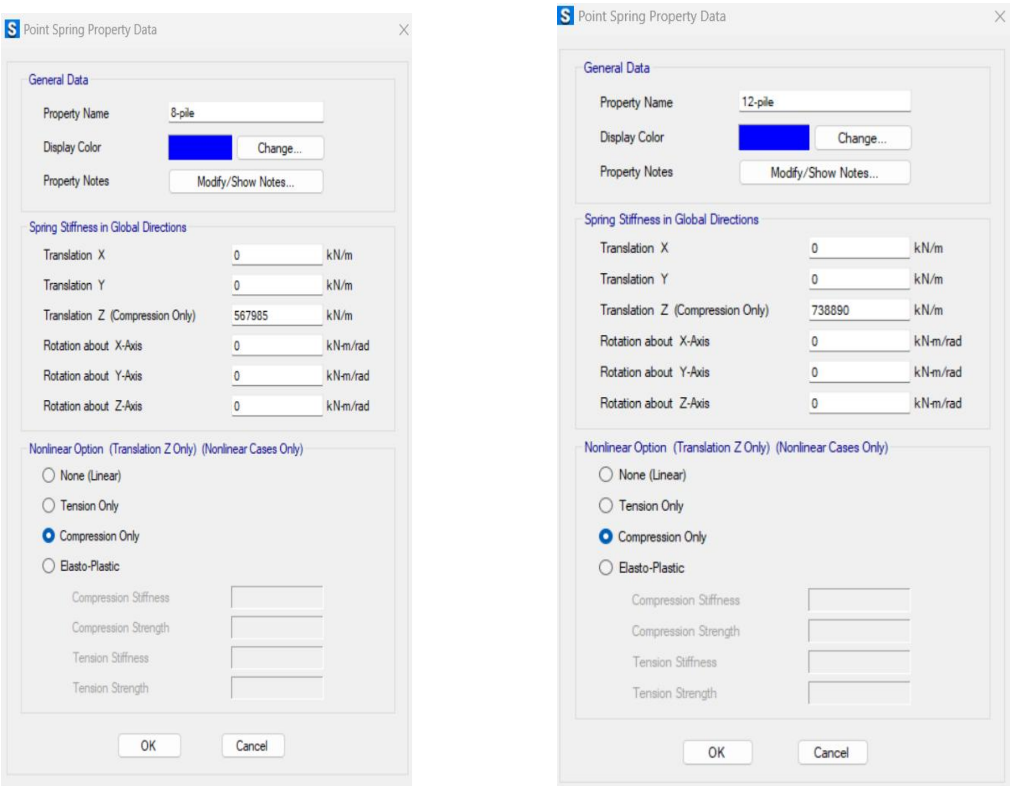


Figure3. 123 :Piles definition in SAFE.

Values for the spring stiffness in the Z-direction were calculated by dividing a pile's capacity over an assumed acceptable value of settlement of 20 millimeters. 12-metre pile capacity: A pile capacity is expressed by the combination of end bearing capacity (Mayerhof's method) and friction capacity (α method). A sample calculation for a 12-metre pile is as follows:

Meyerhof's method of determining end-bearing capacity:

$$Q_p = A_p C_u N_{q'} = 9 A_p C_u \qquad \text{Eq (9.18) in principles of foundation engineering book, Eng Yasser Almadhoun}$$

Where:

Q_p: pile end bearing capacity, kN/m^2.

C_u: undrained cohesion of the soil below the tip of the pile.

Ap: pile cross sectional area, mm^2.

$N_{q'}$, N_{q*}: bearing capacity factors.

Pa: atmospheric pressure = 100 kN/m^2

➔ $N_{q*} = 56.7$ (from **Table 3.40**)

➔ Pa = 100 kN/m^2

$$\rightarrow A_p = \frac{\pi D^2}{4} = 0.5026 \text{ m}^2$$

➔ $Q_p = 9 \times 3000 \times 0.5026 = 13571$KN.

Table3. 40:interpolated values of Nq*

Table 9.5 Interpolated Values of N_q^* Based on Meyerhof's Theory

Soil friction angle, ϕ (deg)	N_q^*
20	12.4
21	13.8
22	15.5
23	17.9
24	21.4
25	26.0
26	29.5
27	34.0
28	39.7
29	46.5
30	56.7
31	68.2
32	81.0
33	96.0
34	115.0
35	143.0
36	168.0
37	194.0
38	231.0
39	276.0
40	346.0
41	420.0
42	525.0
43	650.0
44	780.0
45	930.0

Alpha- method of determining friction capacity:

According to the Alpha- method, the unit skin resistance in clayey soils can be represented by the equation:

$$Q_s = f\,P\,L$$

$$f = \alpha \times C_u$$

where:

f: unit skin resistance in clayey soil.

α: empirical adhesion factor and it is equal to 0.34 (from **Table 3.41**).

C_u: undrained cohesion of the soil below the tip of the pile =3000KN/m^2.

L= length of pail section.

P = perimeter of pail section.

$$\frac{C_u}{P_a} = \frac{3000}{100} = 30$$

Table3. 41 Variation of α.

Table 9.10 Variation of α (Interpolated Values Based on Terzaghi, Peck and Mesri, 1996)

$\frac{c_u}{p_a}$	α
≤ 0.1	1.00
0.2	0.92
0.3	0.82
0.4	0.74
0.6	0.62
0.8	0.54
1.0	0.48
1.2	0.42
1.4	0.40
1.6	0.38
1.8	0.36
2.0	0.35
2.4	0.34
2.8	0.34

Note: p_a = atmospheric pressure ≈ 100 kN/m^2 or 2000 lb/ft^2

By using Table3.41, α = 0.34

L= 12 m ,

P = π d = π × 0.8 = 2.51m,

$$f = \alpha \times C_u = 0.34 \times 3000 = 1020$$

$Q_s = f\ P\ L$

$Q_s = 1020 \times 2.51 \times 12 = 30762$ KN

$Q_{total} = Q_p + Q_s = 30762 + 13571 = 44333$ KN.

Take F.S. = 3

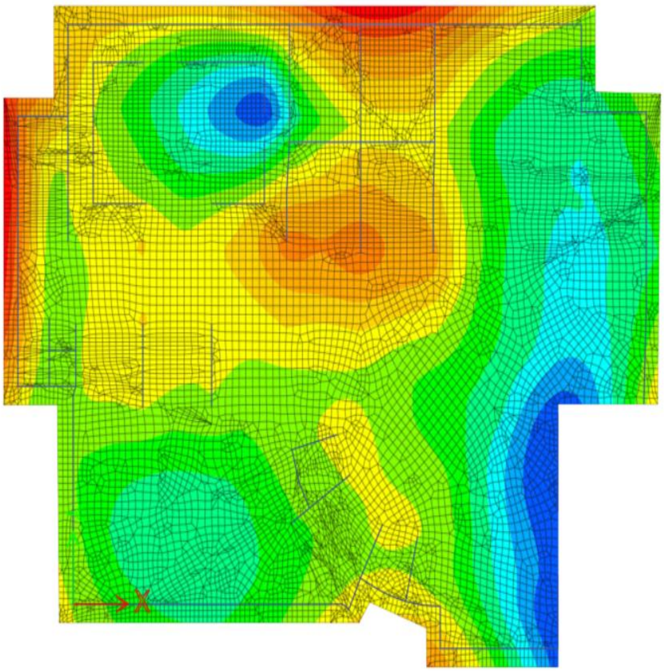
$$\text{Pile capacity} = \frac{Q}{F.S} = \frac{44333}{3} = 14777.6 \text{ KN}$$

$$\text{Stiffness} = \frac{\text{Capacity}}{\text{Settlement}} = \frac{14777.6}{20} = 783.88 \text{ KN/mm}$$

Table 3.40 and 3.42 from principles of foundation engineering book.

SAFE ruselt :

After put the piles, Max. Stress under the mat foundation is equal to 290KN/m^2, which is smaller than 350 KN/m^2 as shown in **Figure 3.124**.



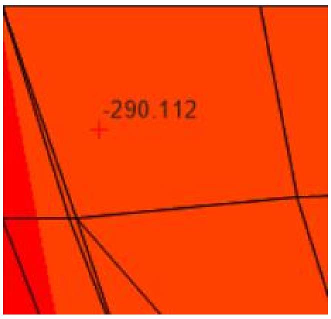


Figure3. 124: Maximum stress under footing after piles.

3.11.4 Footing layout :

Offset is 1 m from face of all support, excepted in the boundary between the two parts.

Figure 3.125 shows the left part footing layout.

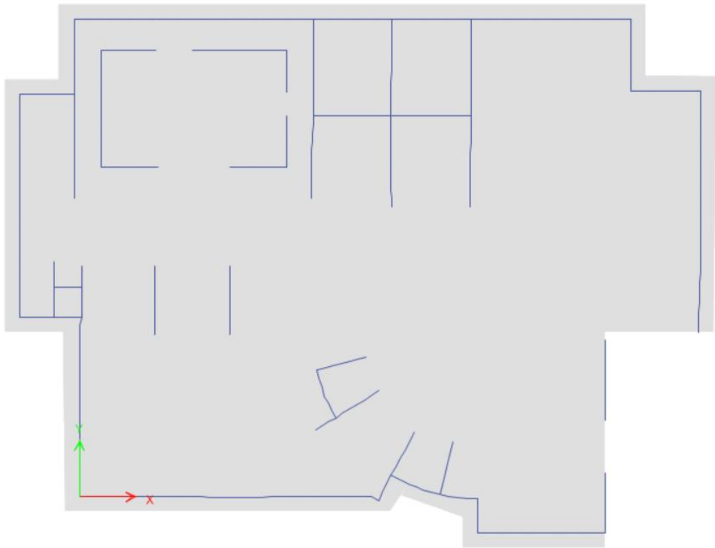


Figure3. 125 :Footing layout

3.11.5 Design verifications:

Design verifications made to ensure that all program's design process are correct and accurate and according to code requirements.

Designing of a footing in this project needs to be computerized in a three-dimensional (SAFE) model and gaining the results of the design from the computer program after making verifications and checks.

Footing design verification

Moment in direction 1 = 1700 kN.m, as shown in **Figure 3.126**.

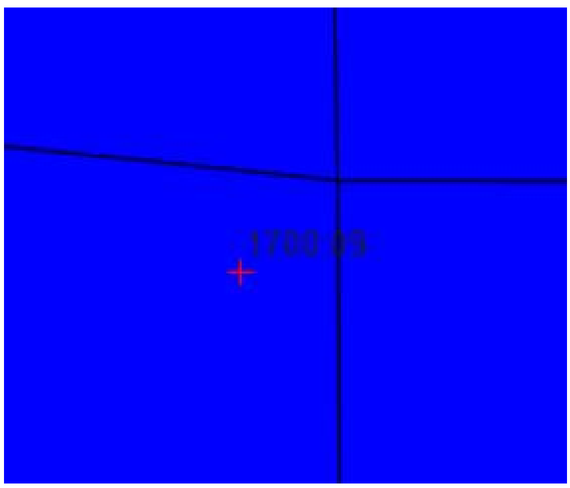


Figure3. 126 :Moment in footing.

Figure 3.127 shows area of steel reinforcement in the same point of previous figure

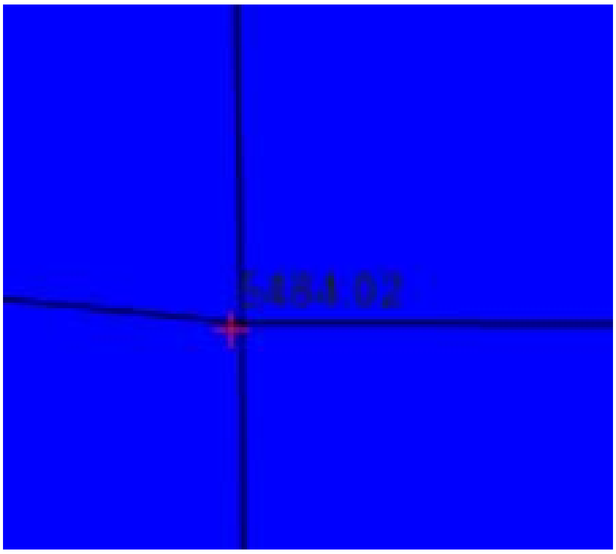


Figure3. 127 Area of steel in footing.

$$\rho = \frac{0.85f'_c}{f_y}(1-\sqrt{\frac{1-2.61M_u}{f'_c\,b\,d^2}})$$

$$= \frac{0.85\times 40}{420}(1-\sqrt{1-\frac{2.61\times 1700\times 10^6}{40\times 1000\times 900^2}})$$

$$= 0.00575$$

$$As = \rho\, b\, d$$

$$= 0.00575\times 1000\times 900$$

$$= 5175\ mm^2$$

$$A_{s,min} = 0.0018\, b\, d$$

$$= 0.0018\times 1000\times 900$$

$$= 1620\ mm^2$$

$$\rho_{max} = 0.375\beta_1\frac{0.85f'_c}{f_y}$$

$$\beta_1 = 0.85-0.05(\frac{f'_c-28}{7})$$

$$\rightarrow \rho_{max} = 0.0232$$

$$A_{s,max} = \rho_{max}\ b\,h$$

$$= 0.0232\times 1000\times 1000$$

$$= 23200\ mm^2$$

$$A_{s,min} < A_s < A_{s,max}, OK$$

$$\text{Different percentage} = \frac{5175\text{ - }5485}{5175}\times 100\%$$

$$= 6\% \qquad \text{Ok}$$

3.11.6 Flexural design:

Minimum area of steel = 0.0018b d

$= 0.0018 \times 1000 \times 900$

$= 1620\ mm^2$

***X-direction:**

Top reinforcement =Ø18 /150mm

Bottom reinforcement = Ø18 /100mm

***Y-direction:**

Top reinforcement =Ø18 /150mm

Bottom reinforcement = Ø18 /100mm

Additional reinforcement sample calculation:

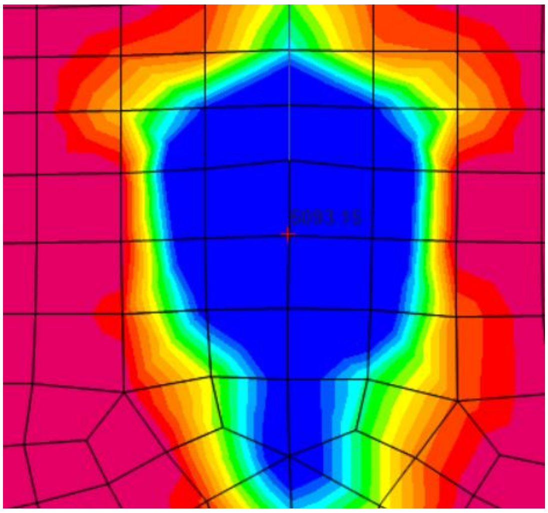


Figure3. 128 :Additional reinforcement required.

Additional reinforcement required is equal to 5100 mm^2, use Ø 32/150mm.

3.11.6 Piles design:

Pile design was constituted as a spiral circular column with a minimum reinforcement ratio of 0.01. The capacity was calculated as follows:

$$\phi P_n = \phi\lambda\left[0.85f_c'(A_g - A_s) + A_s f_y\right]$$

$$= 0.7 \times 0.85 \times \left[0.85 \times 28 \times (\frac{\pi}{4} \times 800^2 - 0.01 \times \frac{\pi}{4} \times 800^2) + 0.01 \times \frac{\pi}{4} \times 800^2\right] / 1000$$

$= 7050$ kN which is larger than the maximum axial force reaction.

Since piles are designed as spiral circular columns, a stirrup spacing of 150mm would be sufficient. Use 1Ø10/150mm.

3.12 Design of stairs :

Stairs are the conventional means of access between floors in buildings. A stair is described as a set of steps leading from one floor to another, and a staircase includes the part of the building surrounding the stair. Staircase is an important component of a building providing access to different floors including the roof of the building. It consists of a set of steps and one or more intermediate landing slabs between the floor levels.

In this section, stairs will be analyzed and designed using ETABS 20.3 software.

The plan of stairs was taken from the architectural plans, which is shown in **Figure3.129**.

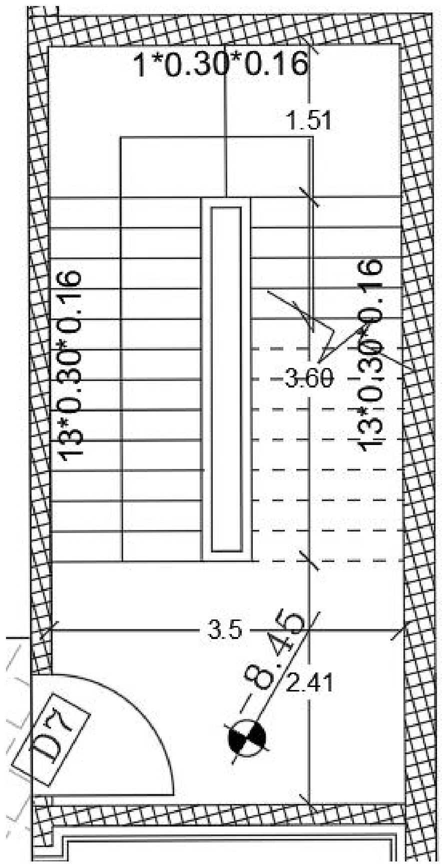


Figure3. 129 Plan of stairs.

3.12.1 Modelling:

For the modelling of stairs, initial dimensions were calculated in accordance with spans. For the landing slab, the support is assumed to be pin connected from both sides and thus the adequate depth would be expressed as the length of the slab divided by 20.

H = 3.6 \ 20 = 180 mm, use 180 mm .

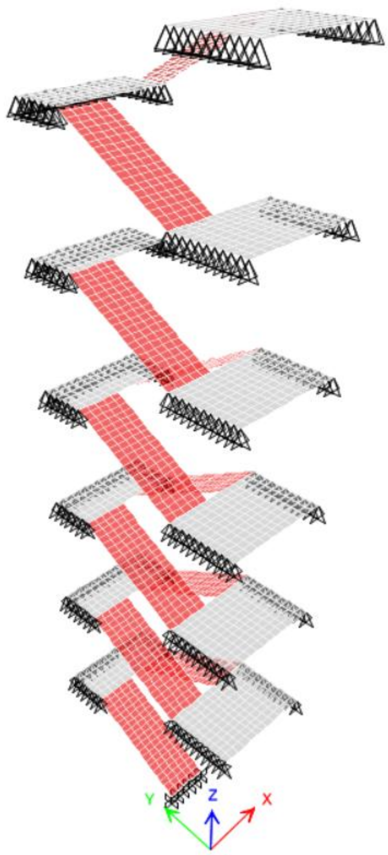


Figure3. 130: Structural model of the stair.

3.12.2 Design:

For the design of the landing slab and flight slab, ETABS results were used.

3.12.2.1 Design of flight slab:

Load calculation of flight slab:

The weight of the stairs which consists of mortar with thickness of 2cm, filling material with thickness of 5cm, and tiles (marble) with thickness of 3 cm, was computed as follows:

Assume simply supported slab:

SDL = 0.03×27 + 0.02×23 + 0.05×17 = 2.12 KN/m^2

Step width = 30cm and height = 17cm, so:

$$\text{Wight of concrete triangle of the stair} = 0.5\times0.3\times0.17\times25 = 0.6375 \text{ KN/m}$$
$$= 0.6375 / \sqrt{0.3^2+0.17^2} = 1.84 \text{ KN/m}^2$$

Total Super dead load = 2.12 + 1.84 = 3.96 KN /m^2

Dead load of stair slab= 0.25 × 0.18 = 4.5 KN/m^2

Wu = 1.2 × D + 1.6 × L

Wu = 1.2 × (4.5 +3.96) + 1.6 ×5 = 18.15 KN/m^2

Flexural reinforcement:

From ETABS :

Mu = 16 KN.m

$$\rho = \frac{0.85f'_c}{f_y}\left(1-\sqrt{1-\frac{2.61M_u}{b\,d^2f'_c}}\right)$$
$$= \frac{0.85\times28}{420}\left(1-\sqrt{1-\frac{2.61\times16\times10^6}{1000\times140^2\times28}}\right)$$
$$= 0.002198$$

$A_s = \rho$ b d = 0.002198× 1000 ×140 =307.8 mm^2 /m

$A_{s,min}$ = 0.0018 b h = 0.0018 × 1000 ×180 = 324 mm^2 /m. →use $A_{s,min}$

Use 1 Ø 12/250mm

For Top and transverse steel use shrinkage steel:

$A_{s,\ min}$ = 324 mm^2

Use 1 Ø 12 /250mm

Shear check:

From ETABS :

V_u= 22.5 KN

Assume that $A_v < A_{v,min}$

Table3. 42:Vc for nonprestressed members - (ACI 318-19 Table 22.5.5.1).

Table 22.5.5.1—V_c for nonprestressed members

Criteria		V_c	
$A_v \geq A_{v,min}$	Either of:	$\left[0.17\lambda\sqrt{f'_c}+\frac{N_u}{6A_g}\right]b_w d$	(a)
		$\left[0.66\lambda(\rho_w)^{1/3}\sqrt{f'_c}+\frac{N_u}{6A_g}\right]b_w d$	(b)
$A_v < A_{v,min}$		$\left[0.66\lambda_s\lambda(\rho_w)^{1/3}\sqrt{f'_c}+\frac{N_u}{6A_g}\right]b_w d$	(c)

Notes:
1. Axial load, N_u, is positive for compression and negative for tension.
2. V_c shall not be taken less than zero.

Where:

$\varnothing V_c$: shear strength capacity of the slab;

Ø: shear reduction factor = 0.75;

λ: 1 for normal weight concrete;

λs: factor used to modify shear strength based on the effects of member depth, commonly referred to as the size effect factor;

f'_c: concrete compressive strength = 28 MPa;

b_w: web width of the slab section = 1 m;

A_g: area gross; and

N_u: factored axial force normal to cross section occurring simultaneously with V_u or

T_u; to be taken as positive for compression and negative for tension, N.

$$\phi V_c = \phi \times \left[0.66\lambda(\rho_w)^{1/3}\sqrt{f_c'} + \frac{N_u}{6A_g} \right] b_w d$$

$$\rho_w = 0.0018 \times \frac{h}{d} = 0.0018 \times \frac{180}{140} = 0.002314$$

$$\phi V_c = 0.75 \times \left[0.66\lambda(\rho_w)^{1/3}\sqrt{f_c'} + \frac{N_u}{6A_g} \right] b_w d$$

$$= 0.75 \times \left[0.66 \times 1 \times (0.002314)^{1/3}\sqrt{28} + 0 \right] \times 1000 \times 140$$

$$= 48.5\ \text{KN} > V_u,\ \text{OK.}$$

No need for shear reinforcement.

3.12.2.1 Design of landing slab:

Load calculation of landing :

SDL = 0.03×27 + 0.02×23 + 0.05×17 = 2.12 KN/m^2

Dead load of stair slab= 25 × 0.18= 4.5 KN/m^2

Assume simply supported slab:

Wu= 1.2 D + 1.6 L+ reacion from flight slab

Wu = 1.2 × (4.5 +2.12) + 1.6 ×5 +35.4

= 51.4 KN/m

Flexural reinforcement:

Mu=25.5 KN.m

$$\rho = \frac{0.85 f_c'}{f_y}\left(1 - \sqrt{1 - \frac{2.61 M_u}{b\, d^2 f_c'}}\right)$$

$$= \frac{0.85 \times 28}{420}\left(1 - \sqrt{1 - \frac{2.61 \times 25.5 \times 10^6}{1000 \times 140^2 \times 28}}\right)$$

$$= 0.00355$$

As = ρ b d = .00355× 1000 ×140 = 497 mm^2 /m

$A_{s,min}$ = 0.0018 b h = 0.0018 × 1000 ×180 = 324 mm^2 /m. < A_s, OK.

Use 1Ø12/200 mm

For top and transverse steel use shrinkage steel:

$A_{s,\,min}$ = 324 mm^2 /m.

Use 1 Ø 12 @ 250mm

Shear check:

From ETABS :

Vu=55.34 KN

Assume that $A_v < A_{v,min}$

Table3. 43:Vc for nonprestressed members - (ACI 318-19 Table 22.5.5.1)

Table 22.5.5.1—V_c for nonprestressed members

Criteria	V_c		
$A_v \geq A_{v,min}$	Either of:	$\left[0.17\lambda\sqrt{f'_c} + \frac{N_u}{6A_g}\right]b_w d$	(a)
		$\left[0.66\lambda(\rho_w)^{1/3}\sqrt{f'_c} + \frac{N_u}{6A_g}\right]b_w d$	(b)
$A_v < A_{v,min}$	$\left[0.66\lambda_s\lambda(\rho_w)^{1/3}\sqrt{f'_c} + \frac{N_u}{6A_g}\right]b_w d$		(c)

Notes:
1. Axial load, N_u, is positive for compression and negative for tension.
2. V_c shall not be taken less than zero.

Where:

$ØV_c$: shear strength capacity of the slab;

Ø: shear reduction factor = 0.75;

λ: 1 for normal weight concrete;

λs: factor used to modify shear strength based on the effects of member depth, commonly referred to as the size effect factor;

f'_c: concrete compressive strength = 28 MPa;

b_w: web width of the slab section = 1 m;

A_g: area gross; and

N_u: factored axial force normal to cross section occurring simultaneously with V_u or T_u; to be taken as positive for compression and negative for tension, N.

$$\phi V_c = \phi \times \left[0.66\lambda(\rho_w)^{1/3}\sqrt{f_c'} + \frac{N_u}{6A_g} \right] b_w d$$

$$\rho_w = 0.0018 \times \frac{h}{d} = 0.0018 \times \frac{180}{140} = 0.002314$$

$$\phi V_c = 0.75 \times \left[0.66\lambda(\rho_w)^{1/3}\sqrt{f_c'} + \frac{N_u}{6A_g} \right] b_w d$$

$$= 0.75 \times \left[0.66 \times 1 \times (0.002314)^{1/3}\sqrt{28} + 0\right] \times 1000 \times 140$$

$$= 48.5 \text{ KN} < V_u \text{, use } \rho_w = \rho_{moment} = 0.00355.$$

$$\phi V_c = 0.75 \times \left[0.66 \times 1 \times (0.00355)^{1/3}\sqrt{28} + 0\right] \times 1000 \times 140$$

$$= 55.9 \text{ KN} > V_u \text{, OK.}$$

$\therefore$ No need for shear reinforcement.

3.13 Design of diaphragms and collectors:

Floor and roof diaphragms shall be designed to resist in-plane seismic design forces from the structural analysis but shall not be less than that determined in accordance with Equation (12.10-1) as follows:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} W_i} W_{px} \qquad (\text{ASCE - 7 - 22 -12.10 -1})$$

$$F_{px,min} = 0.2 S_{DS} I_e W_{px} = 0.2 \times 0.784 \times 1.5 \times W_{px} = 0.2325 W_{px}$$

$$F_{px,min} = 0.4 S_{DS} I_e W_{px} = 0.4 \times 0.784 \times 1.5 \times W_{px} = 0.4704 W_{px}$$

Where:

F_{px} = Diaphragm design force at level x;

F_i = Design force applied to level i;

W_i = Weight tributary to level i; and

W_{px} = Weight tributary to the diaphragm at level x

Story weight is calculated by ETABS from load combination: 1 D+1 SD + 0.25 L.

Diaphragms shall be designed for the inertial forces determined from Equations (12.10-1) through (12.10-3) and for applicable transfer forces resisted by the diaphragm between vertical seismic force-resisting elements.

For structures assigned to seismic design category D, E, or F and having a horizontal structural irregularity of Type 1, 2, 3, or 4 as specified in Table 12.3-1 or a vertical structural irregularity of Type 3, the design forces determined shall be increased by 25% at each diaphragm level where the irregularity occurs for the following elements of the seismic force-resisting system:

1. Connections of diaphragms to vertical elements and to collectors, and

2. Collectors and their connections, including connections to vertical elements of the seismic force-resisting system.

For structures that have a horizontal structural irregularity of Type 4, the transfer forces between horizontally offset vertical seismic force resisting elements shall be increased by the overstrength factor before being added to the diaphragm inertial forces.

A collector is a diaphragm boundary element parallel to the applied load that collect transverse diaphragm shear forces to the vertical elements of the seismic force resisting system or distributes forces within the diaphragm as shown in **Figure 3.131.**

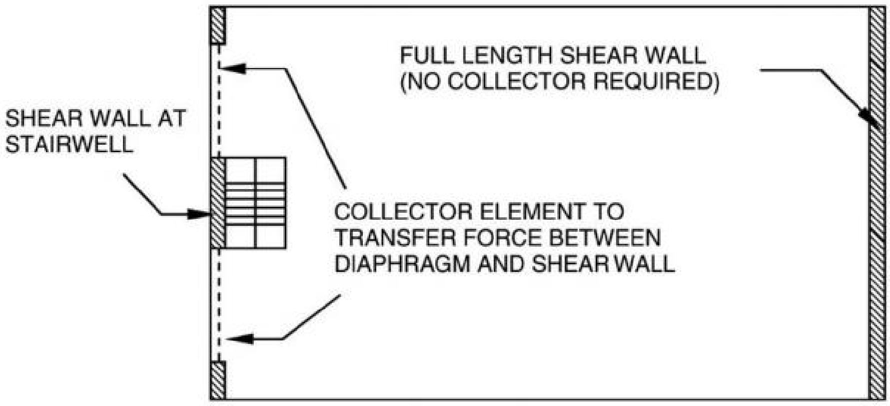


Figure3. 131 Collector (ASCE-7-22-Figure12.10-1)

In structures assigned to Seismic Design Category C, D, E, or F, collector elements and their connections, including connections to vertical elements, shall be designed to resist the maximum of the following:

(a) Forces calculated using the seismic load effects including overstrength with seismic forces determined by the equivalent lateral force or the modal response spectrum analysis procedure; forces from structural analysis using overstrength factor.

(b) Forces calculated using the seismic load effects including overstrength with seismic forces determined by ASCE 7-22 (12.10-1); force F_{px} using overstrength factor.

(c) Forces calculated using the load combinations with seismic forces determined by ASCE 7-22 (12.10-2); forces from structural analysis plus $F_{px,min}$.

Transfer forces from above floor elements (out of plane) shall be considered.

Diaphragm in-plan shear strength:

For a diaphragm that entirely cast-in place, V_n shall be calculated by ACI 318-19 Eq.(12.5.3.3).

$$Vn = A_{cv}(0.17\lambda\sqrt{f'_c} + \rho_t f_y) \quad \text{ACI 318-19 equation 12.5.3.3}$$

Where:

Φ: strenght reduction factor and equal to 0.6 and ,Acv: is the gross area of concrete bounded by diaphragm web thickness and depth, reduced by void areas if present the value of f_c' used to calculate V_n shall not exceed 8.3 MPa; and ρ_t refers to the distributed reinforcement oriented parallel to the in-plane shear. Cross-sectional dimensions shall be selected to satisfy Eq. (12.5.3.4).

$$Vu \le 0.66\phi Acv\sqrt{f'_c} \quad \text{ACI 318-19 equation 12.5.3.4}$$

Collecter elements with compressive stresses exceeding 0.2 $f`_c$ at any section shall have transverse reinforcement satisfying 18.7.5.2 (a) through (e) (hoops in special columns) and 18.7.5.3 (spacing of hoops), expect the spacing limit of 18.7.5.3 (a) shall be one-third of the least dimension of the collecter. Tha amount of thr transverse reinforcement shall be:

$$A_{sh} = 0.09\, b_c \frac{f'_c}{f_{yt}}$$

The specified transverse reinforcement is permitted to be discontinued at a section where the calculated compressive stress is less than 0.15 $f`_c$.

If design forces have been amplified to account the overstrength of the vertical elements of the seismic force resisting system, the limit of 0.2 $f`_c$ shall be increased to 0.5 $f`_c$, and the limit of 0.15 $f`_c$ shall be increase to 0.4 $f`_c$.

In collector, center to center spacing of at least three longitudinal diameters, but not less than 40 mm, and concrete clear cover of at least two and one-half longitudinal bar diameters, but not less than 50 mm. And the area of transverse reinforcement, providing Av at least greater of:

$$\frac{0.062\sqrt{f'_c}b_{ws}s}{f_{yt}}, \frac{0.35b_{ws}s}{f_{yt}}$$

The width of the collector or the chord can be obtained by limiting the compression stress concrete to 0.2 $f`_c$.

Y- direction:

Table 3.44 and **Table 3.45** shows diaphragm design force calculation in Y-direction from load case Qy .

Table3. 44: diaphragm design force calculation in Y-direction .

Story	$\sum(W_{px})$ (KN)	(W_{px}) (KN)	$\sum(F_{px})$ (KN)	F_{px} ratio	$F_{px, min}$ Ratio	$F_{px,max}$ Ratio	Final F_{px} ratio	F_{px} (KN)
Stair	2110.75	2110.75	564.57	0.27	0.24	0.47	0.27	564.57
Roof	13600.97	11490.22	4039.83	0.30	0.24	0.47	0.30	3412.89
F3	30805.24	17204.27	9652.25	0.31	0.24	0.47	0.31	5390.64
F2	49172.14	18366.90	14411.89	0.29	0.24	0.47	0.29	5383.16
F1	69238.55	20066.41	18342.19	0.26	0.24	0.47	0.26	5315.85
Gf	100536.89	31298.35	22124.68	0.22	0.24	0.47	0.24	7361.37
B1	129572.63	29035.73	24087.02	0.19	0.24	0.47	0.24	6829.20

Because of having horizontal irregularity in this structure, the design forces determined shall be increased 25% .

Table3. 45: diaphragm design force calculation in Y-direction .

Story	Fpx (KN)	Story Area (m^2)	Fpx/Area (KN/m^2)	1.25×(Fpx/Area) for connections
Stair	564.5722	77.91	7.25	9.06
Roof	3412.890075	642.72	5.31	6.64
F3	5390.63706	700.99	7.69	9.61
F2	5383.164777	742.36	7.25	9.06
F1	5315.852123	791.8	6.71	8.39
Gf	7361.371308	1250.45	5.89	7.36
B1	6829.204237	1196.02	5.71	7.14

Sample of calculation for Basement 1 floor:

By export Basement 1 story to new ETABS model, and assign area load equal to determined design forces in positive and negative Y-direction to the slab area as shown below:

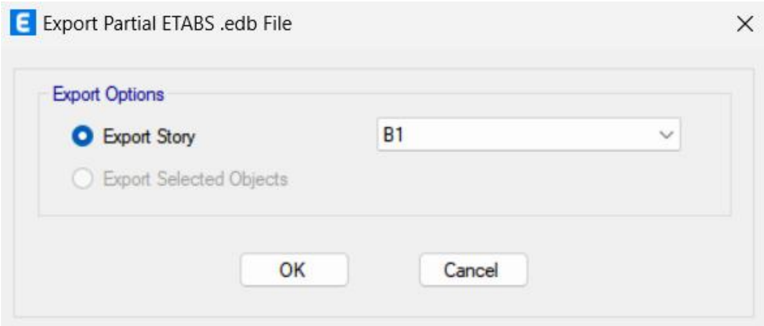


Figure3. 132 Exporting Basement 1 floor to new ETABS model.

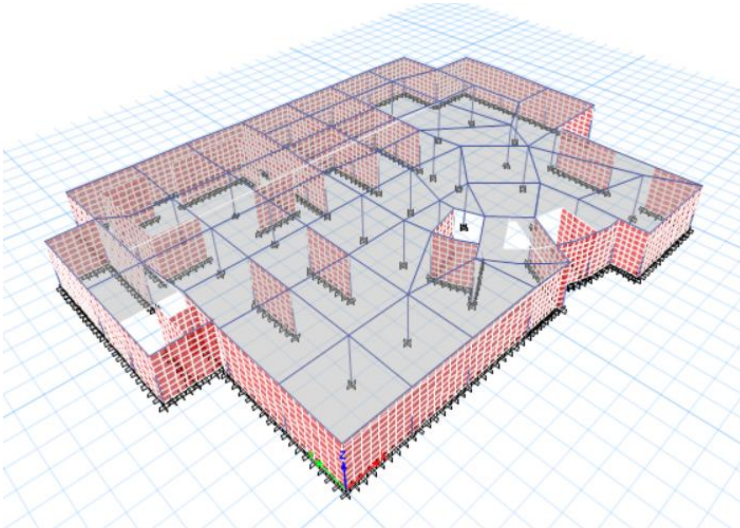


Figure3. 133 New ETABS model for Basement 1 floor.

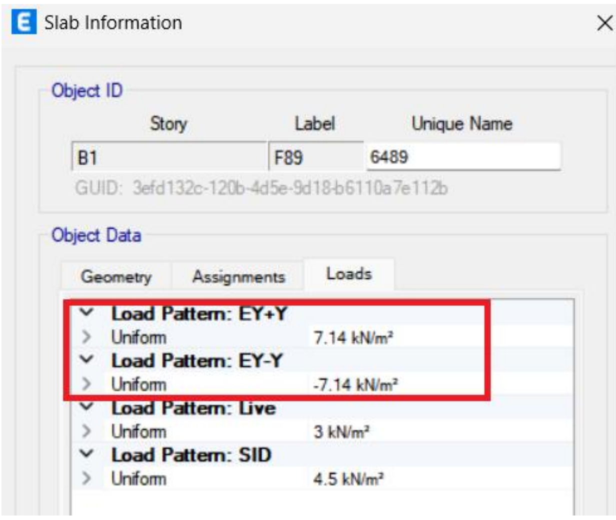


Figure3. 134 Assign design forces in positive and negative Y-direction.

By display F22 by design force load cases EY+Y and EY-Y, the moment and shear forces in the diaphragm can be obtained by doing section cut in Y-direction along the slab and taking into account the maximum values.

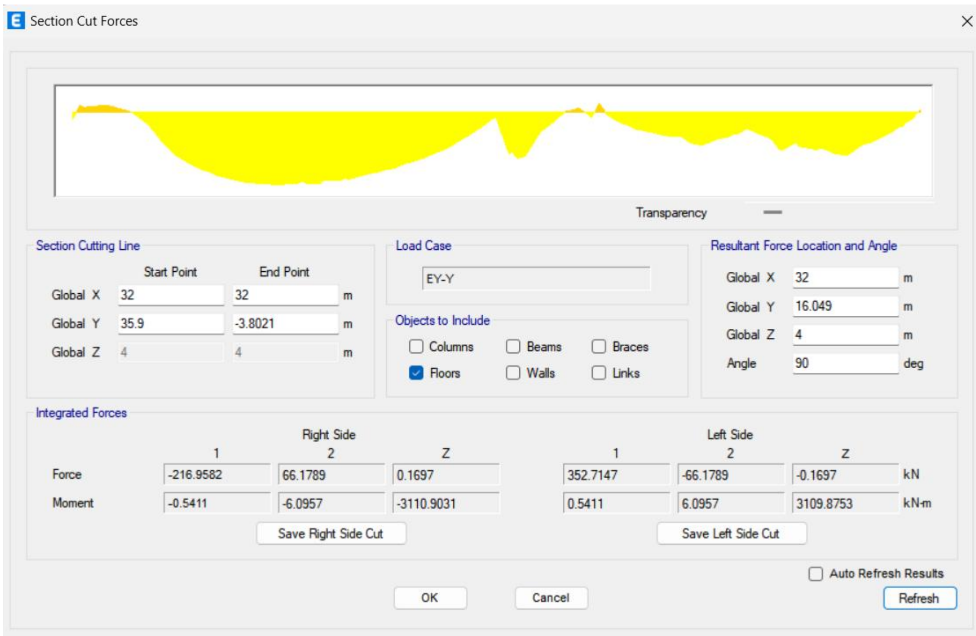


Figure3. 135 Maximum bending moment in Y- direction in the diaphragm in Basement 1 floor.

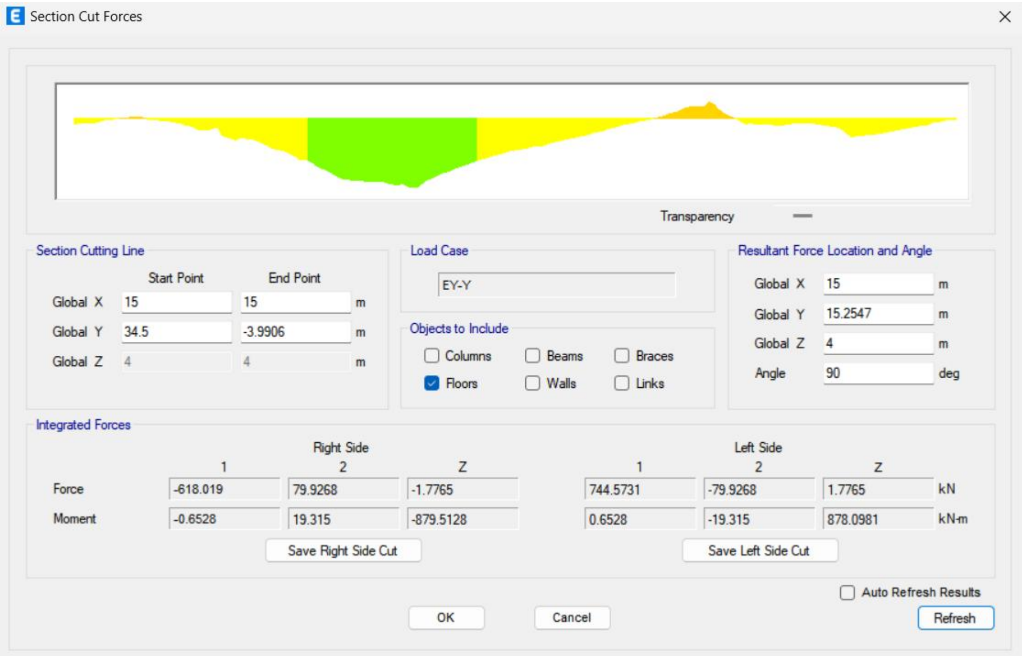


Figure3. 136 Maximum shear force in Y- direction in the diaphragm in Basement 1 floor.

M_u =3111 KN.m

$F_{chord} = M_u/d$

= 3111/(32-1)

= 100 KN

As = 100×1000/0.9×420

= 265 mm^2

b_{chord} = 100 ×1000 /0.2×28×250

= 71.4 mm, very small , so used width equal to beam width 400 mm.

The shear strength of the diaphragms is given by assuming there is no shear reinforcement:

$$\phi Vn = \phi A_{cv}(0.17\lambda\sqrt{f'_c} + \rho_t f_y)$$
$$= \frac{0.6 \times 32000 \times 250(0.17 \times 1 \times \sqrt{28} + 0)}{1000}$$
$$= 4317.8 \text{ KN} > Vu = 618 \text{ KN, there is no need for shear reinforcement.}$$

Collectors:

By using the same model, which used to design diaphragm in Y- direction, assign area load in positive and negative Y-direction equal to F_{px} without multiply with 1.25. **Figure 3.137** shows the assigned area load on slab area in basement 1 floor.

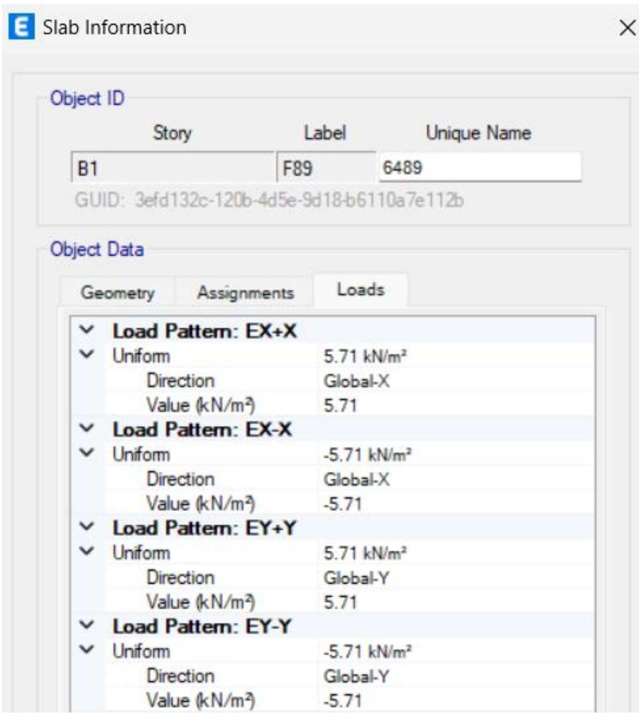


Figure3. 137 assigned area load on slab area in basement 1 floor

As shown in **Figure 3.138** the force at the edge of wall will be 201.11 KN , this force requires area of reinforcement equals :

$$A_s = \frac{F \times \Omega_\circ}{\varphi fy} = \frac{201.11 \times 1000 \times 2.5}{0.9 \times 420} = 1330 \ \ mm^2$$

Width of the collector :

Area $_{collector =}$ Width of collector × depth of slab = b ×250

Area $_{collector}$×0.2fc = force

$$b = \frac{201.11 \times 1000 \times 2.5}{250 \times 0.2 \times 28} = 359.125mm$$

There is a beam in the floor system of width 400 mm , the reinforcement can be added in the slab mid-depth along the beam line , 3Ø25 .

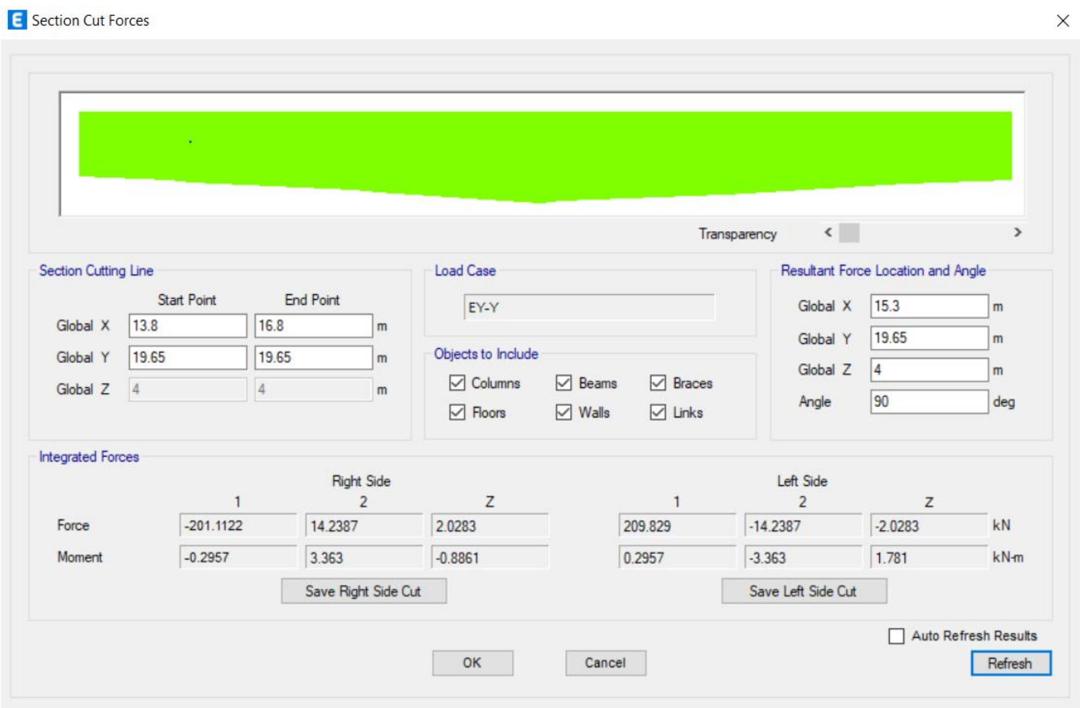


Figure3. 138 :section cut obtain the axial force in the collector(Y direction) .

X-Direction:

Table3. 46: diaphragm design force calculation in X-direction .

Story	$\sum(W_{px})$ (KN)	(W_{px}) (KN)	$\sum(F_{px})$ (KN)	Fpx ratio	Fpx, min Ratio	Fpx,max Ratio	Final Fpx ratio	Fpx
								(KN)
Stair	2110.75	2110.75	202.18	0.10	0.24	0.47	0.24	496.45
Roof	13600.97	11490.22	2133.48	0.16	0.24	0.47	0.24	2702.50
F3	30805.24	17204.27	5075.75	0.16	0.24	0.47	0.24	4046.44
F2	49172.14	18366.90	7550.72	0.15	0.24	0.47	0.24	4319.89
F1	69238.55	20066.41	9573.10	0.14	0.24	0.47	0.24	4719.62
Gf	100536.89	31298.35	11490.82	0.11	0.24	0.47	0.24	7361.37
B1	129572.63	29035.73	12460.87	0.10	0.24	0.47	0.24	6829.20

Table3. 47: diaphragm design force calculation in X-direction .

Story	Fpx (KN)	Story Area (m^2)	Fpx/Area (KN/m^2)	1.25×)Fpx/Area(
Stair	496.45	77.91	6.37	7.97
Roof	2702.50	642.72	4.20	5.26
F3	4046.44	700.99	5.77	7.22
F2	4319.89	742.36	5.82	7.27
F1	4719.62	791.8	5.96	7.45
Gf	7361.37	1250.45	5.89	7.36
B1	6829.20	1196.02	5.71	7.14

Sample of calculation for Basement 1 floor:

By using the same model, which used to design diaphragm in Y- direction, assign area load in positive and negative x-direction equal to F_{px} which calculated in Table **3.47.**

By display F11 by design force load cases EX+X and EX-X the moment and shear forces in the diaphragm can be obtained by doing section cut in X-direction along the slab and taking into account the maximum values.

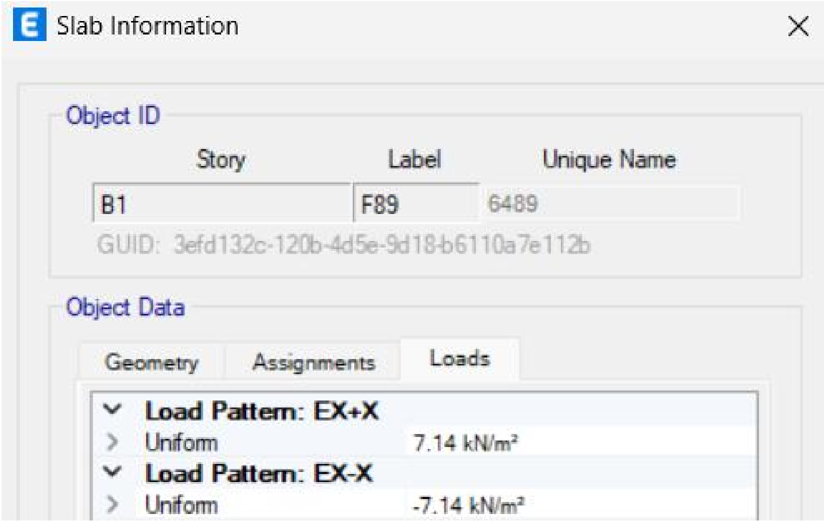


Figure3. 139 Assign design forces in positive and negative Y-direction.

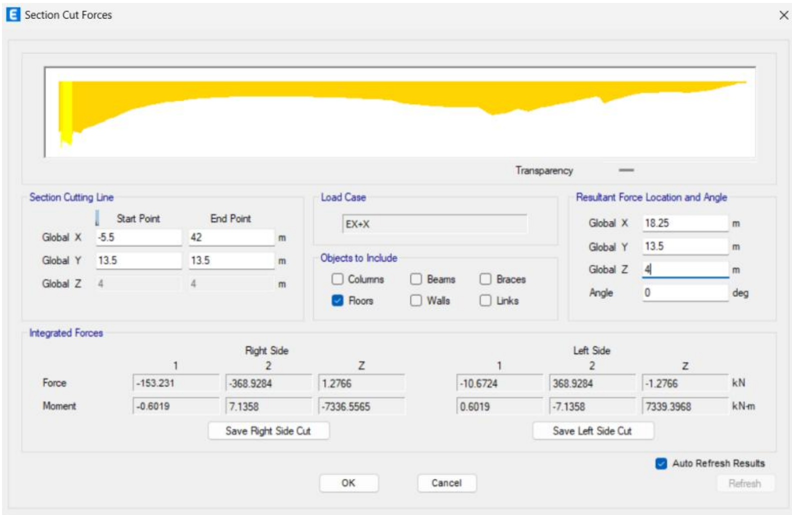


Figure3. 140 Maximum bending moment in X- direction in the diaphragm in Basement 1 floor.

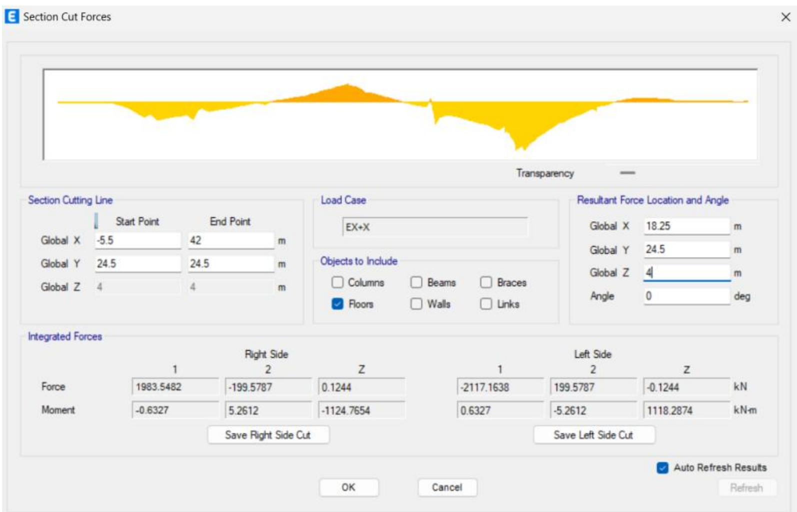


Figure3. 141 Maximum shear force in X- direction in the diaphragm in Basement 1 floor.

M_u =7336.5 KN.m

$F_{chord} = M_u/d$

$= 3111/(34.5-1)$

$= 219$ KN

As = 219×1000/0.9×420

$= 579.4\ mm^2$

$b_{chord} = 219 \times 1000 / 0.2 \times 28 \times 250$

= 156 mm, very small , so used width equal to beam width 400 mm.

The shear strength of the diaphragms is given by assuming there is no shear reinforcement:

$$\phi Vn = \phi A_{cv}(0.17\lambda\sqrt{f_c'} + \rho_t f_y)$$
$$= \frac{0.6 \times 34500 \times 250(0.17 \times 1 \times \sqrt{28} + 0)}{1000}$$
$$= 4655.2 \text{ KN} > Vu = 1938.5 \text{ KN, there is no need for shear reinforcement.}$$

Collecters:

By using the same model, which used to design diaphragm in X- direction, assign area load in positive and negative X-direction equal to F_{px} without multiply with 1.25.

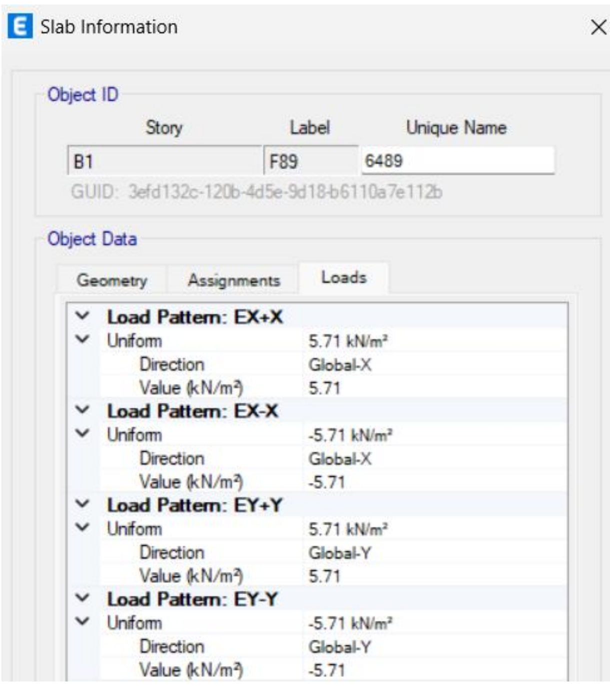


Figure3. 142 assigned area load on slab area in basement 1 floor

As shown in **Figure 3.143** the force at the edge of wall will be 109.7 KN , this force requires area of reinforcement equals :

$$A_s = \frac{F \times \Omega_o}{\varphi fy} = \frac{109.7 \times 1000 \times 2.5}{0.9 \times 420} = 725.5\text{mm}$$

Width of the collector :

Area $_{collector}$ = Width of collector × depth of slab = b ×250

Area $_{collector}$×0.2fc = force

$$b = \frac{109.7 \times 1000 \times 2.5}{250 \times 0.2 \times 28} = 195.9\text{mm}$$

the reinforcement can be added in the slab mid-depth along line with 200 mm width , 4Ø16 .

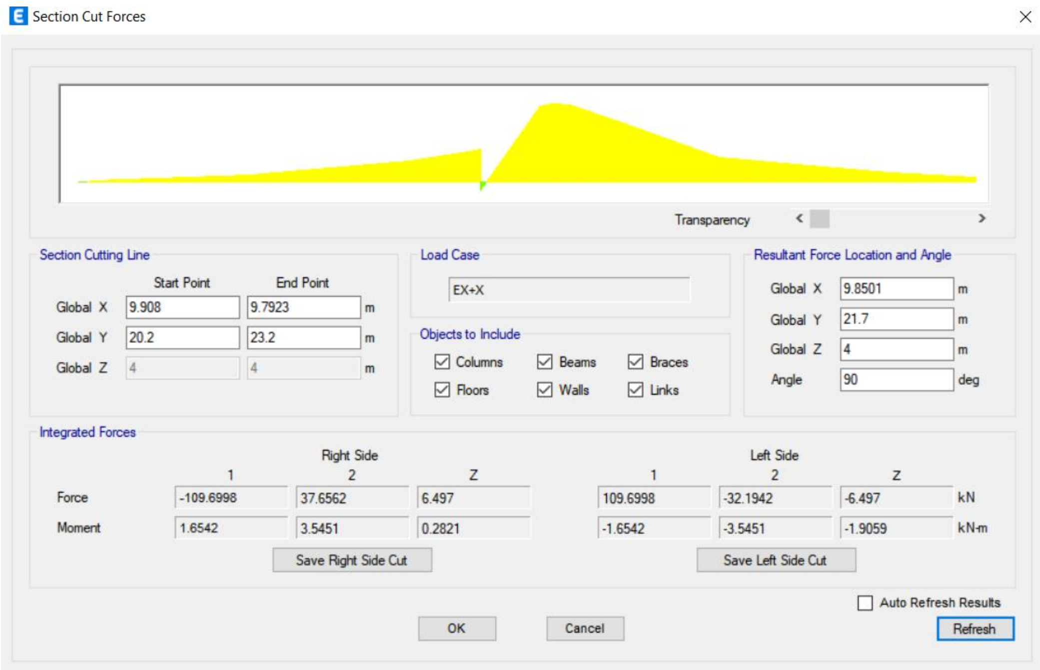


Figure3. 143 :section cut obtain the axial force in the collector(X direction) .

3.14 Analysis of non-structural walls (Design forces):

This section established the analysis of the forces on the connections of non-structural components, such as exterior walls and partitions.

Exterior walls:

The exterior walls contain 3cm of stone and 7cm of unreinforced concrete. And it must be ensured that the stones do not fall when the building is exposed to an earthquake. Therefore, the stones must be attached and fixed in the horizontal and vertical directions.

It was decided to stack four rows of stone units vertically together. Each stone is 25 cm high, so every four rows of stone units would be one meter high. The calculations will be performed for every 1 horizontal meter.

Area load of stone equal to 2.5 KN/m^2, refer to section **1.6.3**.

Weight force = Stone Load × Area

= 2.5(KN/m^2) × 1 (m^2)

= 2.5 KN

Vertical earthquake force component, $E_v = 0.2\ S_{DS}\ D$

$E_v = 0.2 \times 0.784 \times 2.5 = 0.392$ kN.

Vertical force on steel angle = E_v + Weight of 100mm wall

= 0.392+ 2.5

= 2.90 kN.

Then it requires selecting a steel angle to withstand this force.

For Horizontal force F_p:

$$F_p = 0.4 S_{DS} I_P W_P \left[\frac{H_f}{R_\mu}\right]\left[\frac{C_{AR}}{R_{po}}\right]$$

$$0.3 S_{DS} I_P W_P \le Fp \le 1.6 S_{DS} I_P W_P$$

$$H_f = 1 + 2.5\left(\frac{z}{h}\right)$$

$$R_\mu = \left[1.1R/I_e\Omega_o\right]^{1/2} \ge 1.3$$

Where :

F_p: Seismic design force;

S_{DS} :Spectral acceleration, short period;

I_p: Component Importance Factor;

W_p: Component operating weight;

H_f : Factor for force amplification as a function of height in the structure;

R_μ : Structure ductility reduction factor;

C_{AR}: Component resonance ductility factor that converts the peak floor or ground acceleration into the peak component acceleration;

R_{po}: Component strength factor;

z: Height above the base of the structure to the point of attachment of the component;

h : Average roof height of structure with respect to the base,Take z/h=1 to include the design of the non-structural walls for all the stories of the structure;

R: Response modification factor for the building or non building structure supporting the component; and

Ω_0: Overstrength factor for the building or non building structure supporting the component.

Table3. 48:architectural components coefficients.

Table 13.5-1. Coefficients for Architectural Components.

Architectural Component	C_{AR} Supported at or below grade plane	C_{AR} Supported above grade plane by a structure	R_{po}	Ω_{op}[a]
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

For an 800 x 250 x 30 mm masonry

Unit weight of masonry = 26.5 KN/m^3.

W_p= weight of one piece of stone unit = 0.8 × 0.25 × 0.03× 26.5 = 0.159 kN

H_f =1+2.5×1

= 3.5

C_{AR} =1

R_{po} =1.5

$$F_p = 0.4S_{DS}I_PW_P\left[\frac{H_f}{R_\mu}\right]\left[\frac{C_{AR}}{R_{po}}\right]$$
$$= 0.4\times0.784\times1.5\times0.159\times\left[\frac{3.5}{1.3}\right]\left[\frac{1}{1.5}\right]$$
$$= 0.1342\ \text{KN}$$

$$F_{p,min} = 0.3\times0.784\times1.5\times0.159 = 0.0561\ \text{KN}$$
$$F_{p,max} = 1.6\times0.784\times1.5\times0.159 = 0.3\ \text{KN}$$
$$\rightarrow F_p = 0.1342\ \text{KN}$$

Partitions:

This section establishes the design criteria for non-structural components (block partitions) that are attached to structures for their supports. The design of every structure subjected to seismic movements should consider that the non-structural elements in the building, such as partition walls so that it can withstand the “movements” of the structure.

The significance of non – structural component design is to ensure the safety of the occupants in the building that are endangered by the collapse of such elements due to poor response of the non – structural elements.

According to ASCE 7 – 22 the structural elements should withstand the horizontal force .

$$F_p = 0.4S_{DS}I_PW_P\left[\frac{H_f}{R_\mu}\right]\left[\frac{C_{AR}}{R_{po}}\right]$$
$$0.3S_{DS}I_PW_P \le Fp \le 1.6S_{DS}I_PW_P$$
$$H_f = 1 + 2.5\left(\frac{z}{h}\right)$$

$$R_\mu = [1.1R/I_e\Omega_o]^{1/2} \ge 1.3$$

Table3. 49:architectural components coefficients.

Table 13.5-1. Coefficients for Architectural Components.

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

The design shall be taken of a one-meter strip width of block partition wall.

Block thickness = 0.1m

Unit weight of block = 15 KN/m^3.

W_P= 0.1 × 15 = 1.5 KN/m^2.

H_f =1+2.5×1

= 3.5

C_{AR} = 2.2

R_{po} =1.5

$$F_p = 0.4 S_{DS} I_P W_P \left[\frac{H_f}{R_\mu}\right]\left[\frac{C_{AR}}{R_{po}}\right]$$

$$= 0.4 \times 0.784 \times 1.5 \times 1.5 \times \left[\frac{3.5}{1.3}\right]\left[\frac{2.2}{1.5}\right]$$

$$= 2.794 \text{ KN}$$

$$F_{p,min} = 0.3 \times 0.784 \times 1.5 \times 1.5 = 0.53 \text{ KN/m}^2$$

$$F_{p,max} = 1.6 \times 0.784 \times 1.5 \times 1.5 = 2.82 \text{ KN/m}^2$$

$$\rightarrow F_p = 1.27 \text{ KN/m}^2.$$

CHAPTER 4: THREE-DIMENSIONAL ANALYSIS AND DESGIN FOR THE RIGHT PART OF THE HOSPITAL BULDING

4.1 General:

This chapter includes a three-dimensional analysis of the project, it contains structural modeling of the building using ETABS software, analysis procedure was done, to check and evaluate the suggested preliminary dimensions of the structural elements and achieve all assumptions mentioned in the previous chapters. As a result, elements can be added, and preliminary dimensions are subjected to change, in the analysis process structural mesh is automatic in slabs and manual in walls. Analysis results of the program are checked to verify and validate that the program has correct outcome results, such as, compatibility check, equilibrium check, stress-strain check…etc. Moreover, design checks were preformed to ensure the stability of the structure, structural irregularities check and P – delta effect.

4.2 Structural Modeling of the Building:

In this section the phase of modeling the structure on the ETABS 20 software, starts with defining the materials, section properties, load pattern load cases…. etc. Also assigning gravity loads and lateral load… etc. Running the analysis and verifying the outcomes (results) of the program through multiple checks to ensure that the results are valid and can be dependable for design, moreover structural checks are done to make sure that the structure is stable and safe, Figures 4.1 shows the 3D – structure.

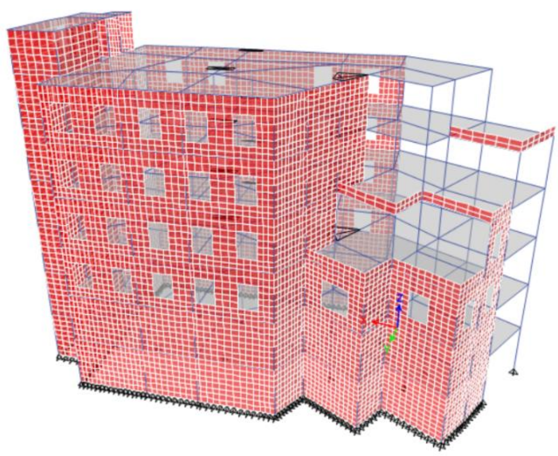


Figure 4.1 3D – Structural model for right side

4.2.1 Units:

Main three units used are meter (m) for length, kilo newton (kN) for force and Celsius (C) for temperature.

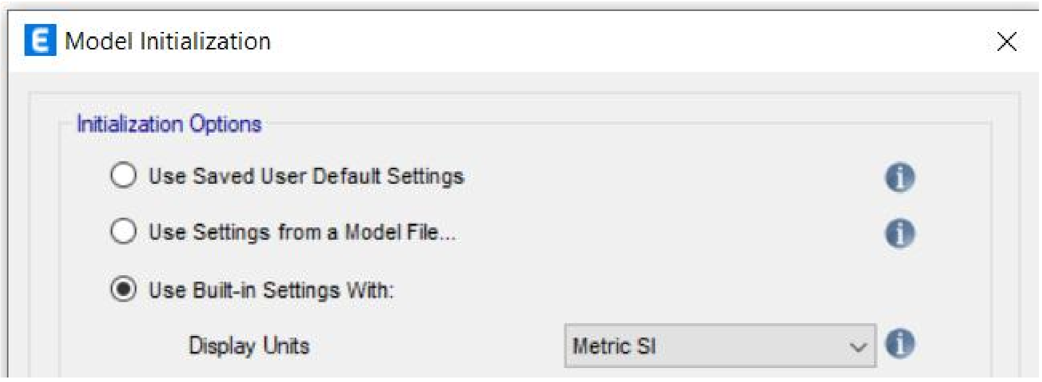


Figure 4.2 Display Units in ETABS

4.2.2 Material:

Material used for slabs, beams, column and wall have compressive strength of 28 Mpa, properties are shown in Figure 4.2.

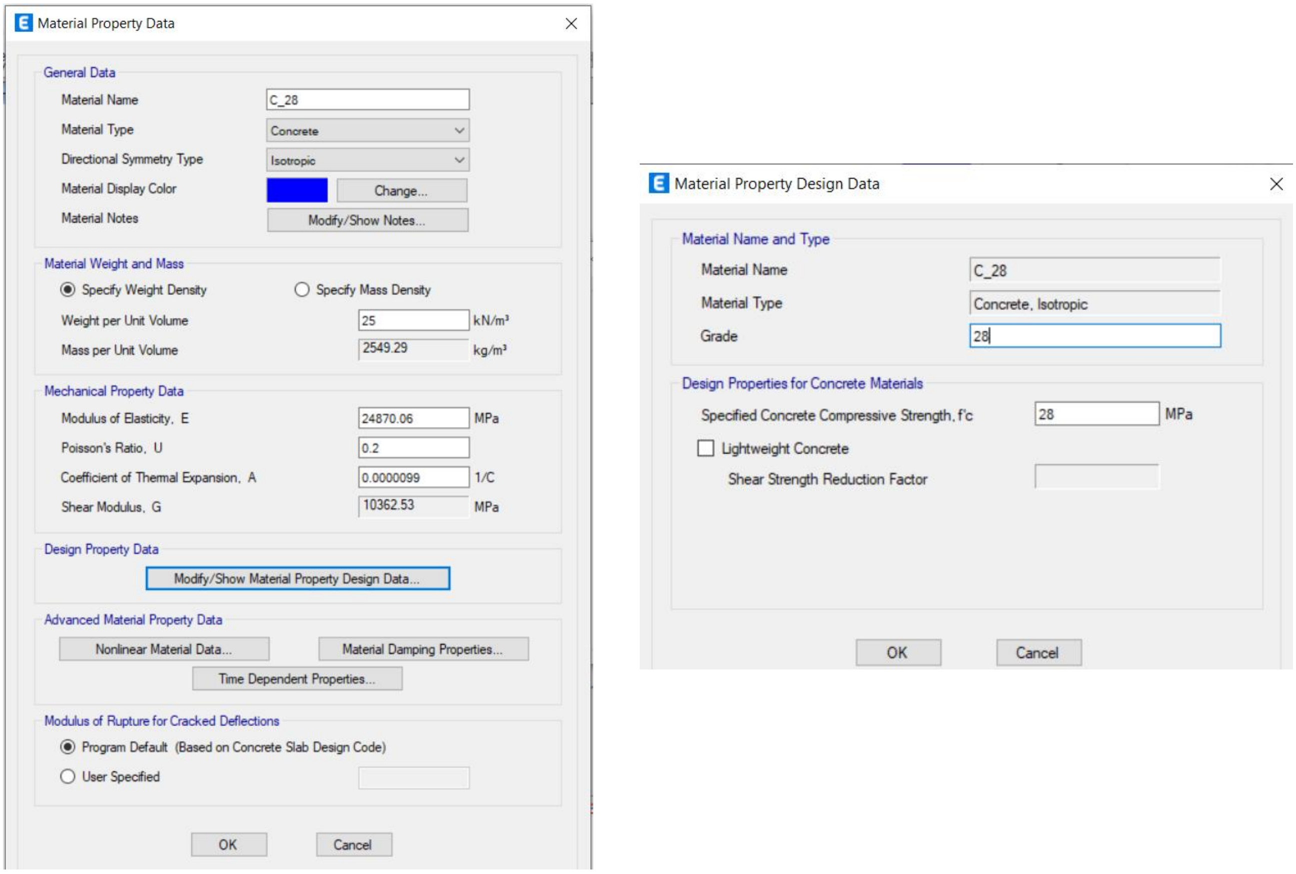


Figure 4.3 Merial Properties used for Slab, beams and walls

4.2.3 Frame properties:

Frame:

Columns and beams are defined with preliminary dimensions mentioned in sections 2.5 and 2.6 respectively.

1. **Columns:**

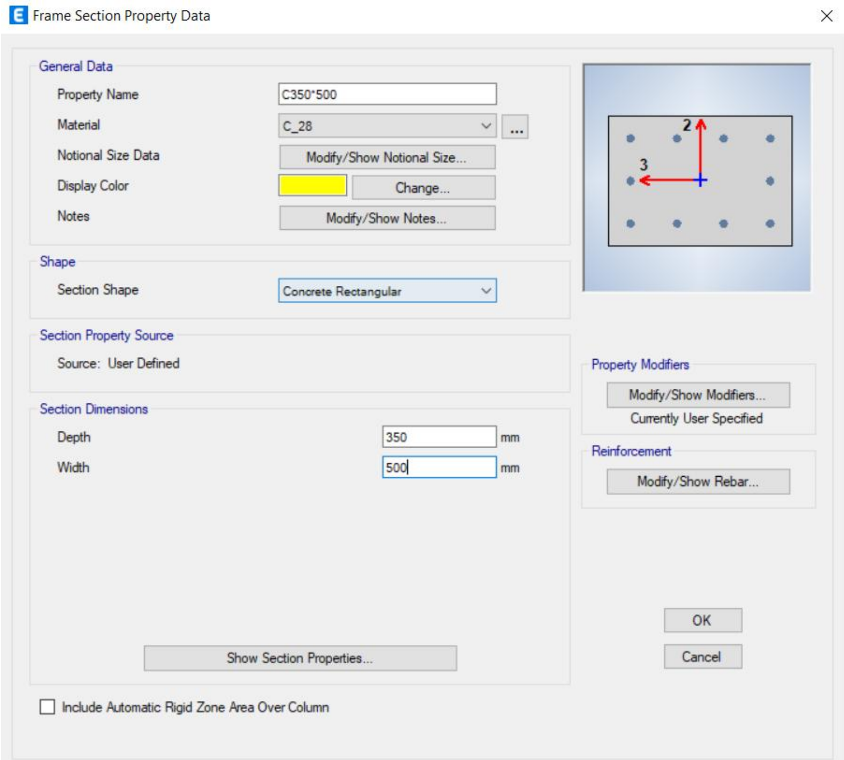


Figure 4.6 Rectangular 350*500 column properties.

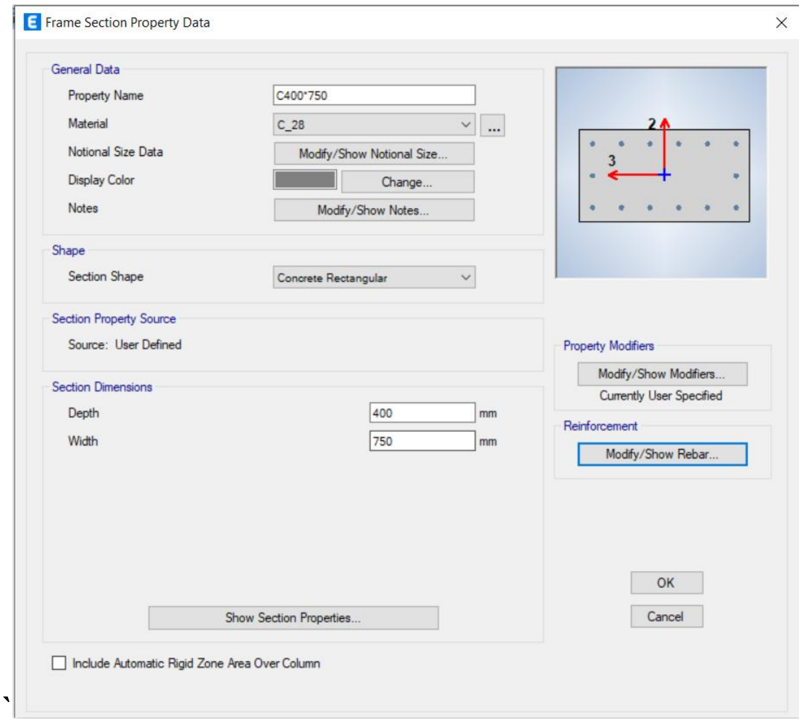


Figure 4.7 Rectangular 400*750 column properties

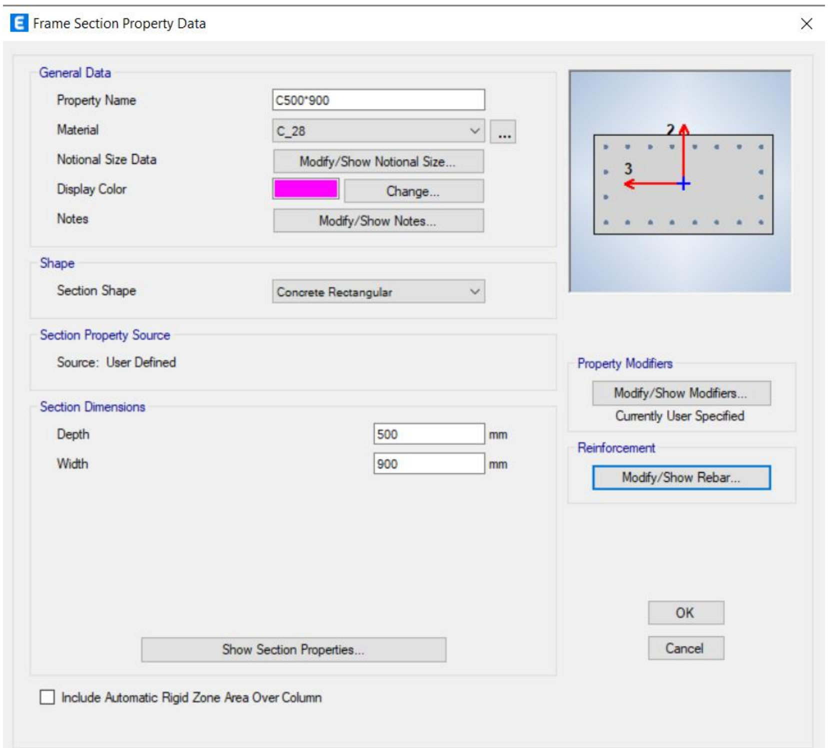


Figure 4.8 Rectangular 500*900 column properties.

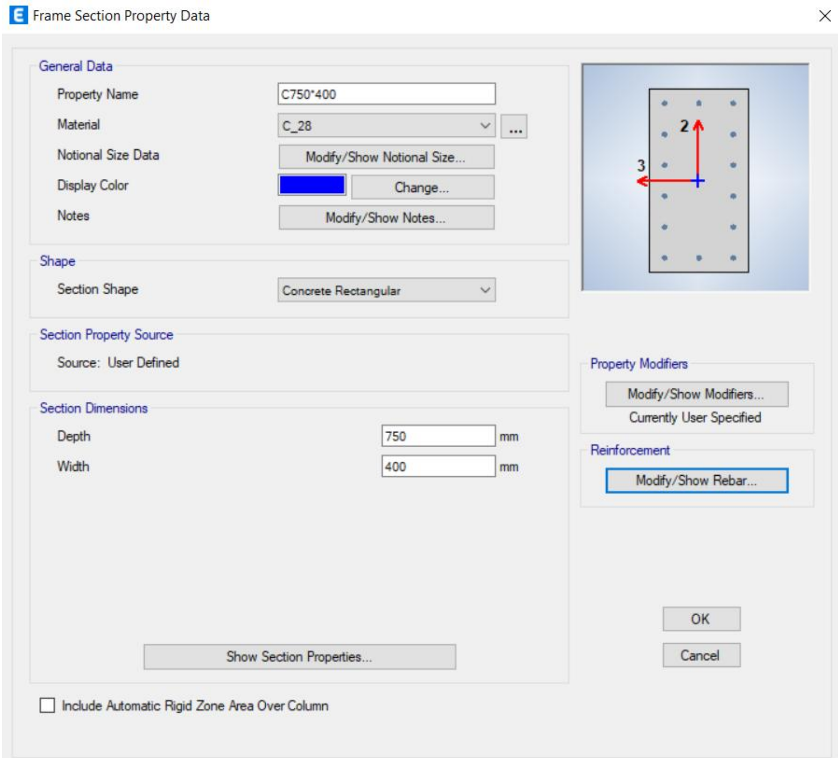


Figure 4.9 Rectangular 750*400 column properties.

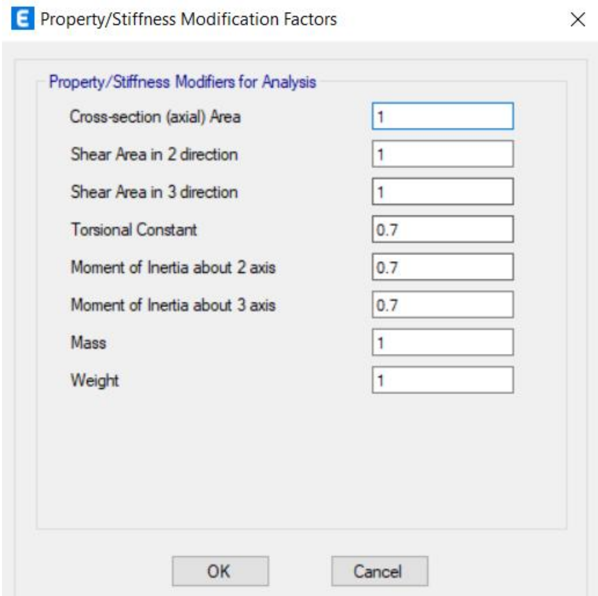


Figure 4.10 Modifiers for columns.

2. Beams:

Beam definitions and properties are shown in Figure 4.9. Where beam modifiers are shown in Figure 4.10

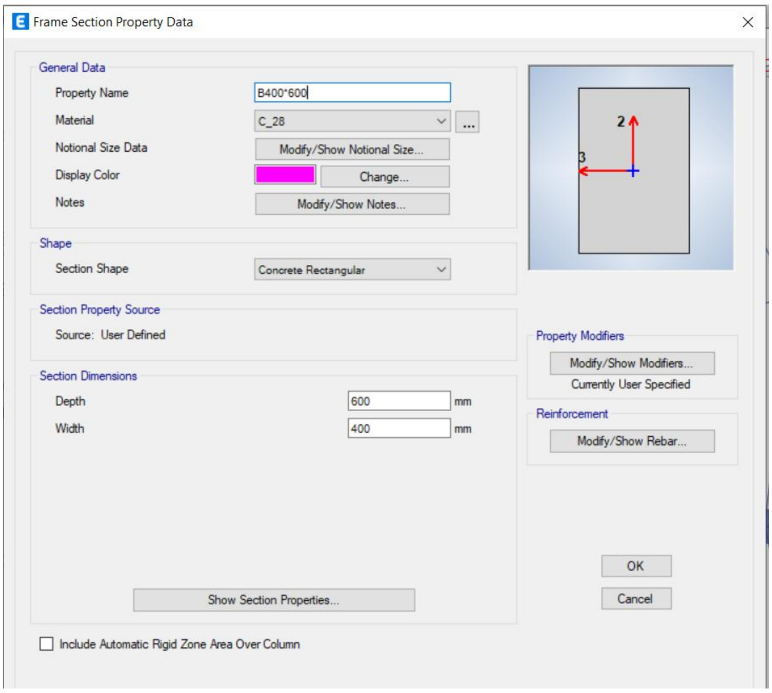


Figure 4.11 Drop beam 400*600 properties.

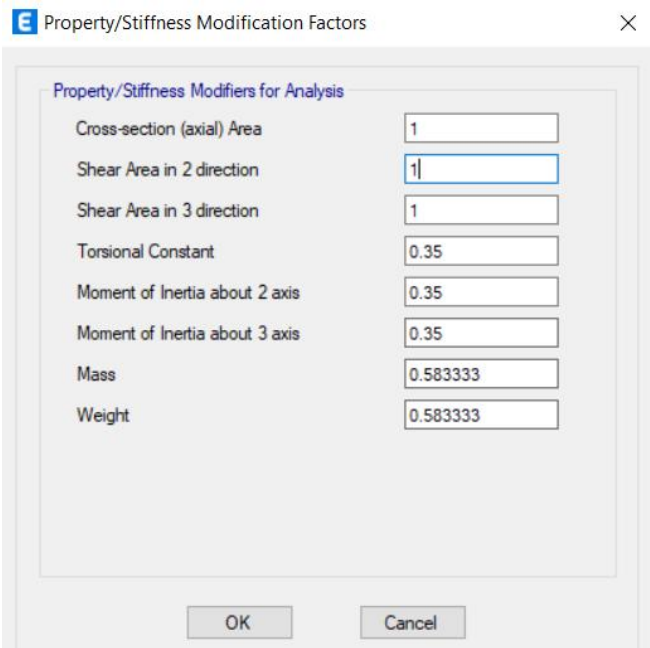


Figure 4.12 Modifiers for beams.

Shells:

III. Slabs are defined with preliminary dimensions that are mentioned in section 2.4, horizontal slab, stairs properties are shown in Figures 4.1, 4.12 respectively.

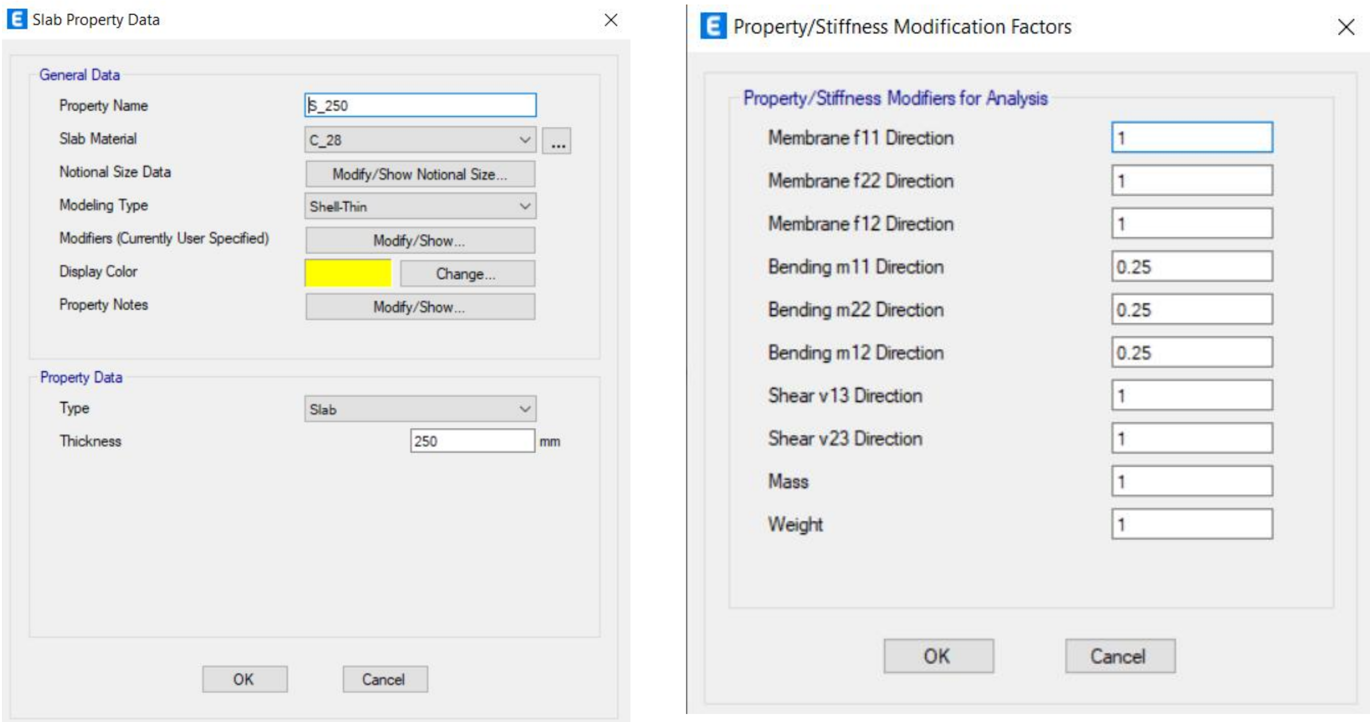


Figure 13 horizontal slab properties.

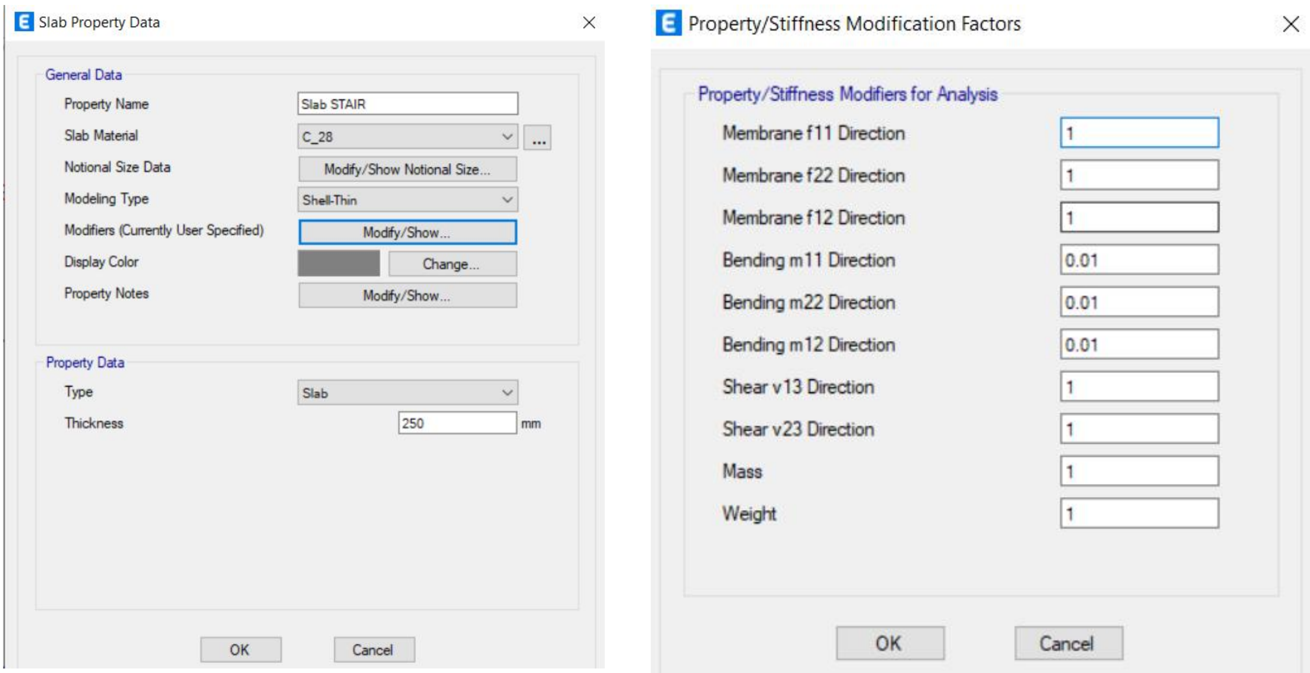


Figure 4.14 stairs properties.

IV. Shear walls:

Shear walls properties shown in figure 4.13. Where Shear modifiers are shown in figure 4.14.

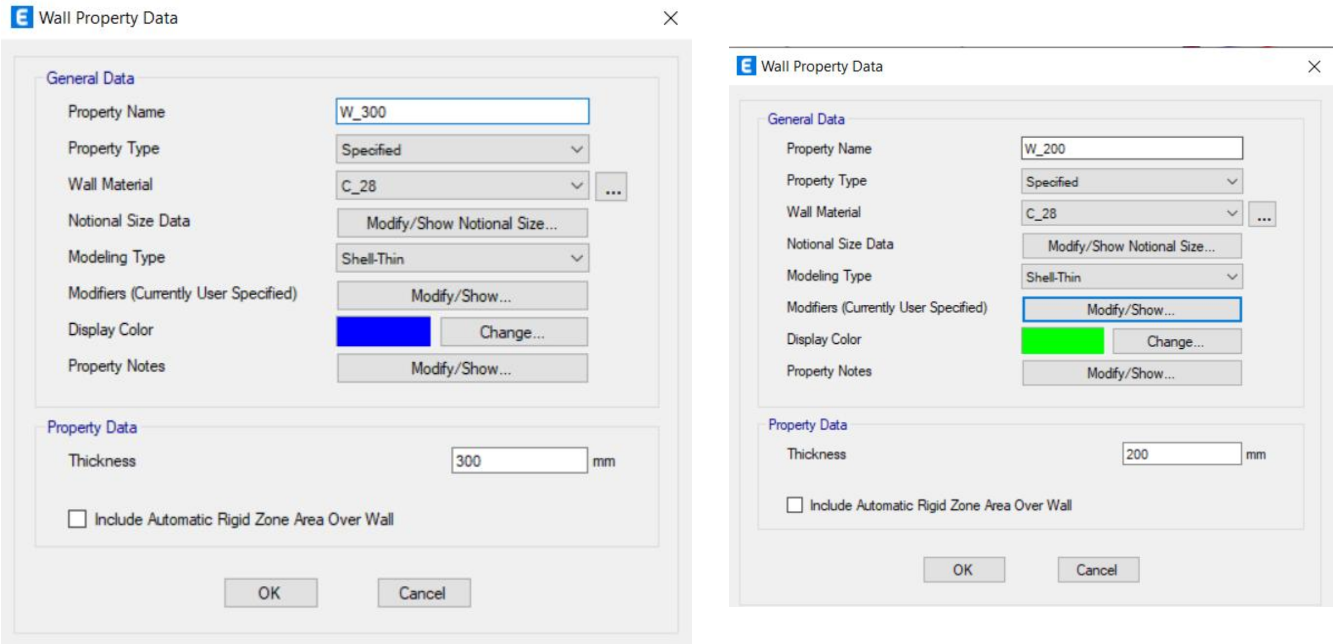


Figure 4.15 shear wall properties.

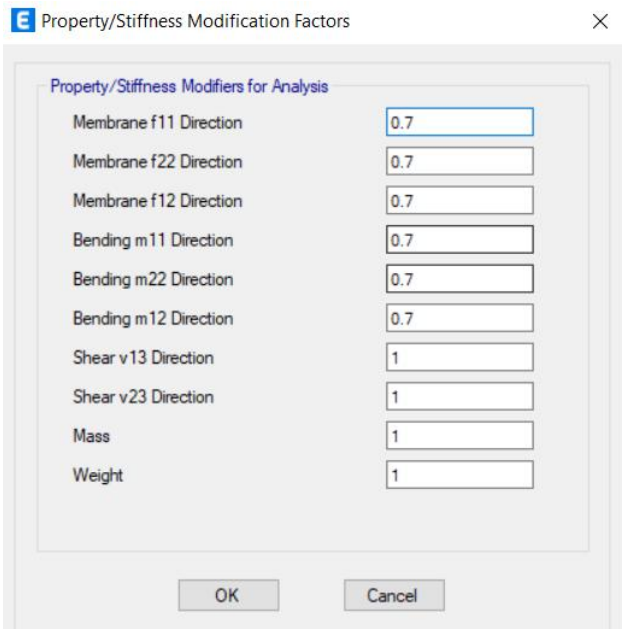


Figure 4.16 Shear wall modifiers.

4.2.4 Section Modifiers:

Taking the cracks into consideration may occurs in structure which make reduction in the structural element capacity, these modifiers represent the effective properties of section after specific period, and each element has a value according to importance of this element.

- Slab modifier: have bending modifiers changed to 0.25, ramps and stairs have their bending and membrane modifiers changed to 0.001, both cases.

Figures 4.15 and 4.16 show the used modifiers for slab and stair slab.

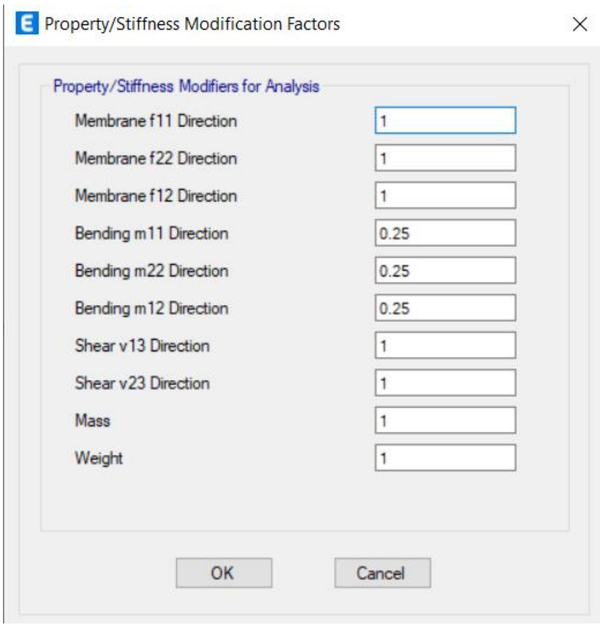


Figure 4.17 Modification factor for slab section.

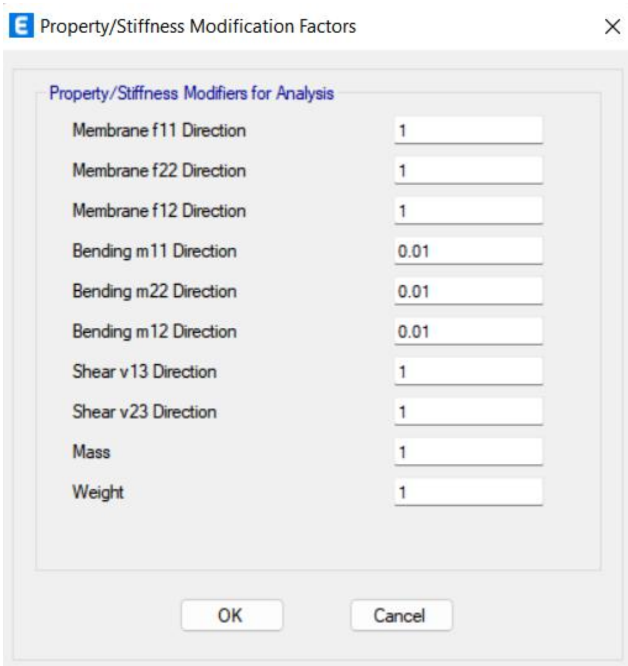


Figure 4.18 Modification factor for Stair slab section.

-Beam modifier:

Mass and weight modifiers = thickness of drop / thickness of beam

= (600-250)/600

= 0.5833

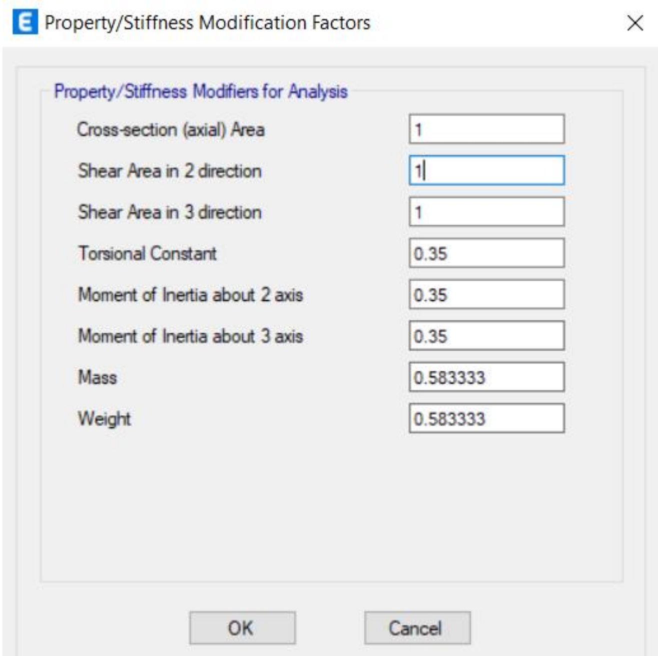


Figure 4.19 Modification factor for Beam section

- Column modifier:

Figure 4.18 bellow shows the used modifiers for any column section.

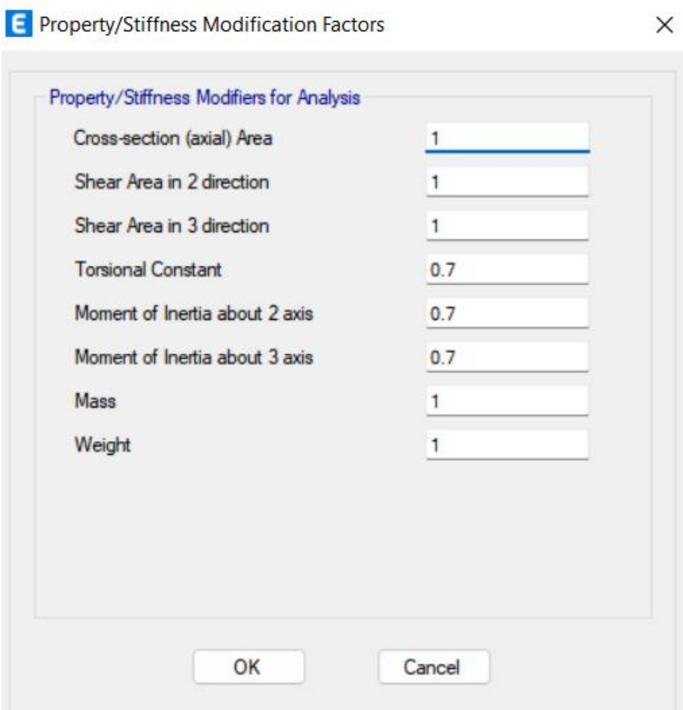


Figure 4.20 Modification factor for column section.

- Wall modifiers:

Shear walls have membrane and bending modifiers equal to 0.7 shown in Figure 4.19.

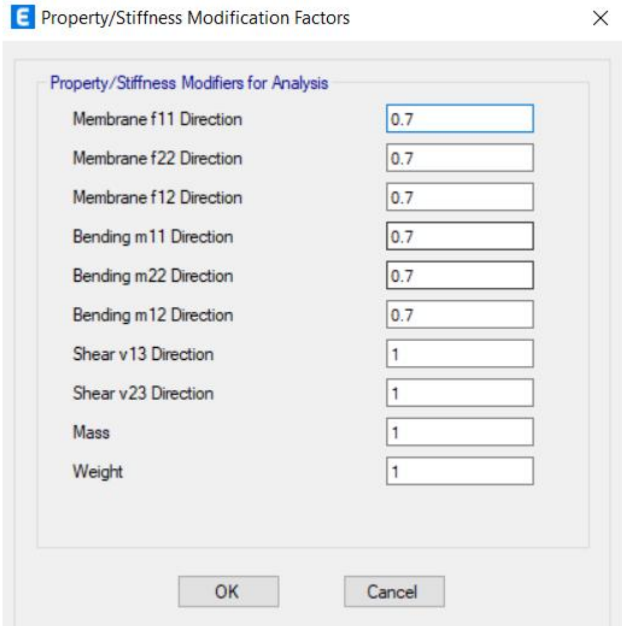


Figure 4.21 Modification factor for walls section.

4.2.5 Load definitions:

Expected loads on the structure will be defined in the software, such as dead, live, snow ...etc loads. As shown in the Figure 4.20

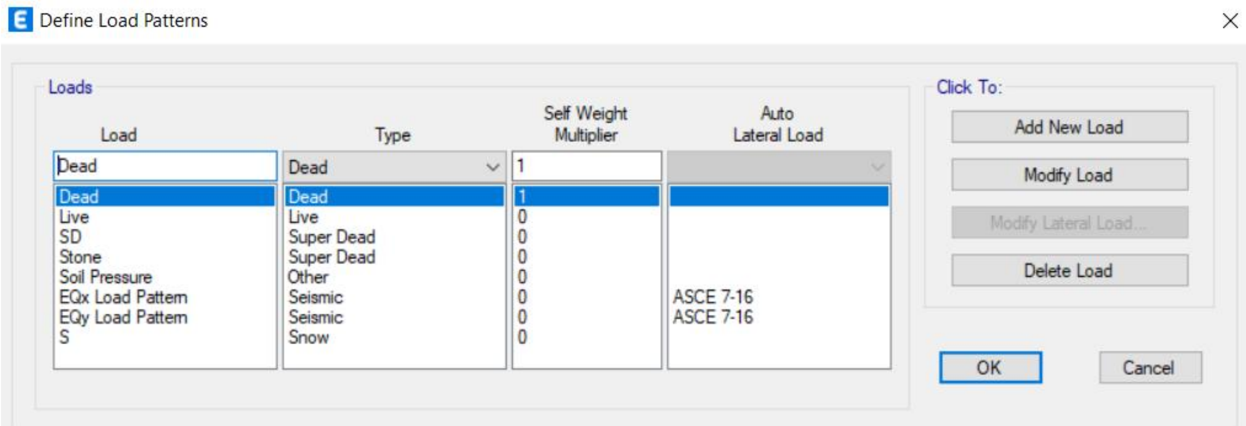
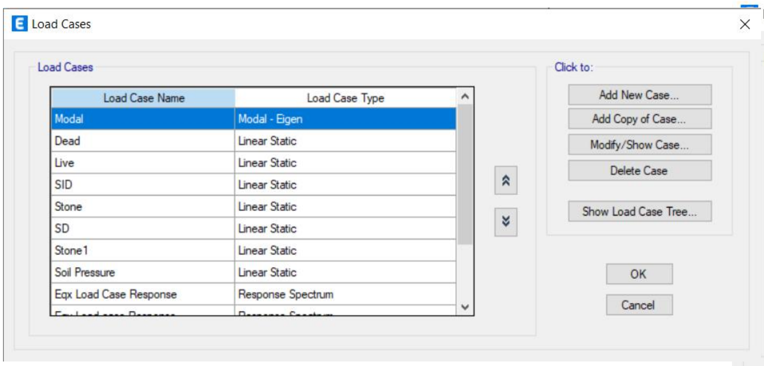
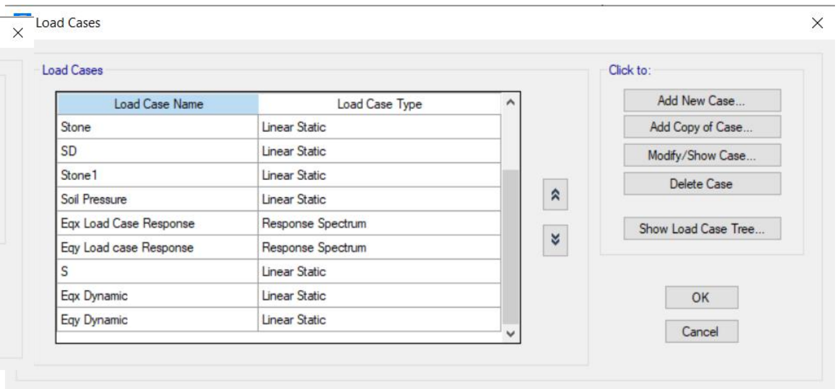


Figure 4.22 Definition of load patterns.





4.2.6 Load cases:

Figure 4.23 Definition of load cases.

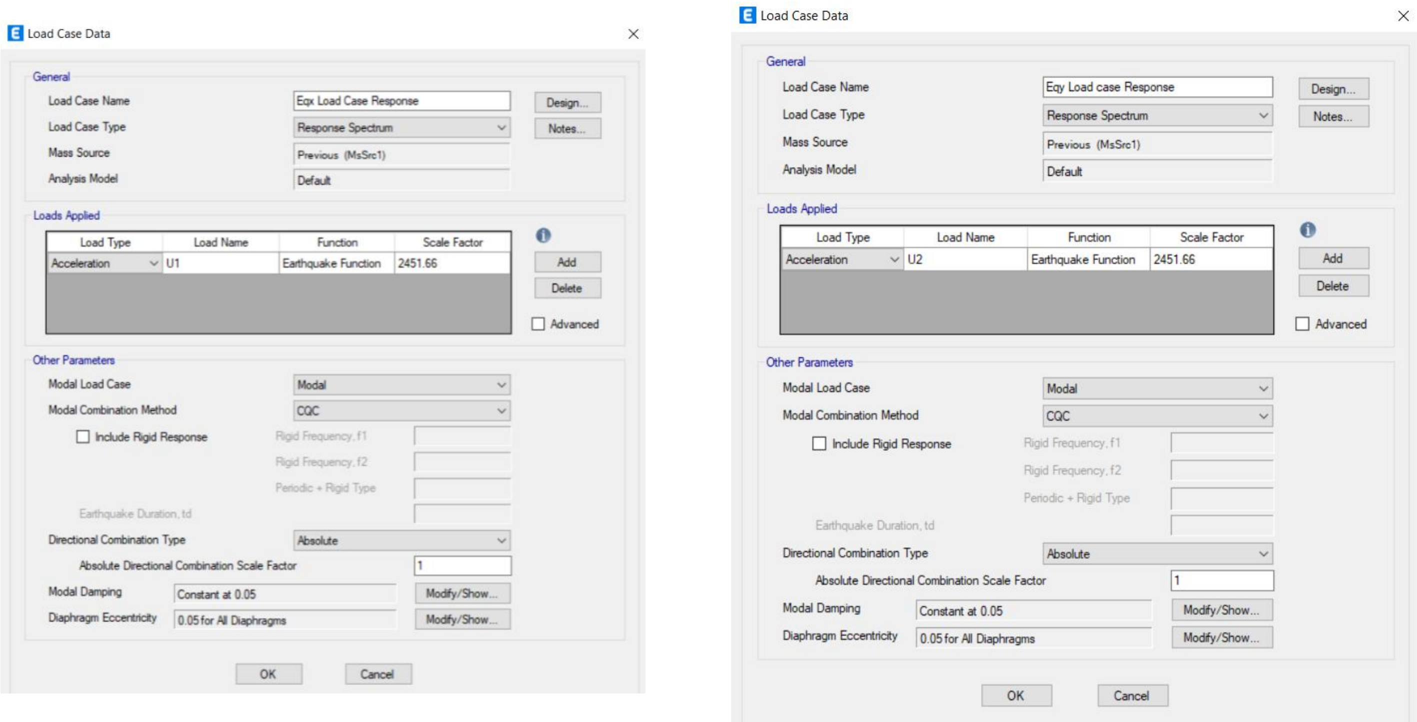


Figure 4.24 Definition of Response spectrum earthquake Eq_x and Eq_y load cases.

4.2.7 Load combination:

According to section 1.7 that describes load combinations defined in both directions X and Y (positive and negative for static combinations). Figure 4.23 shows these combinations. Where:

S: Service combinations.

U: Ultimate combination.

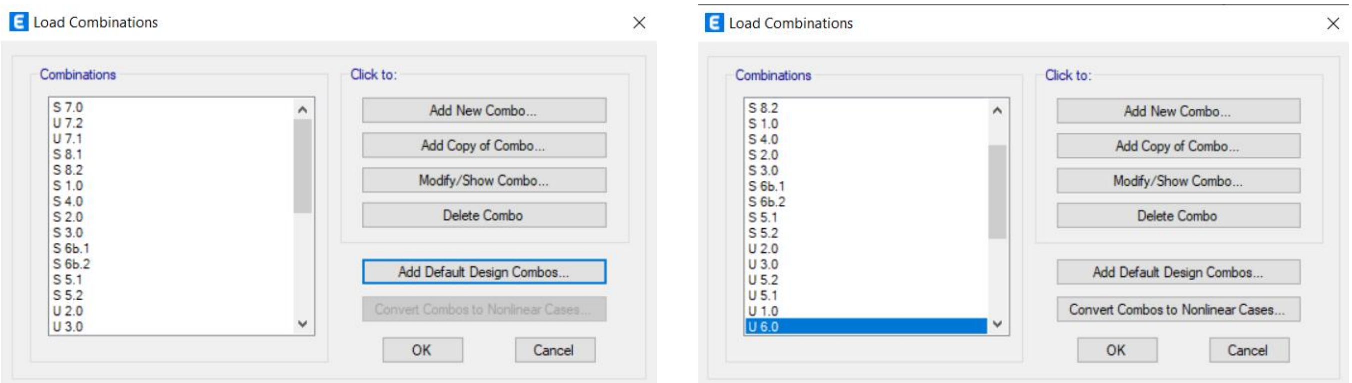


Figure 4.25 Definition of load combination.

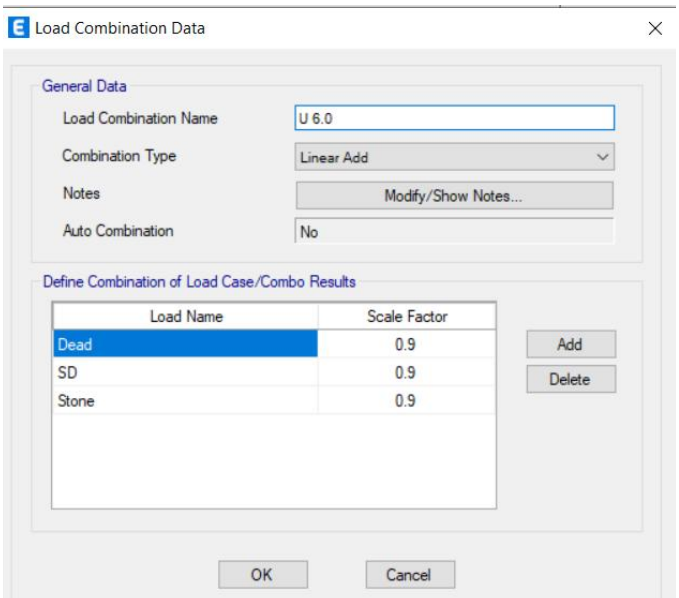


Figure 4.26 Sample of load combination.

4.2.8 Loads assignments:

Table 4.6 Load assignment on Slabs

Floor	SID kN/m^2	LL kN/m^2
B2 Ceiling	4.5	6
B1 Ceiling	7	5
GF Ceiling	7.5	6
F1 Ceiling	7.5	5
F2 Ceiling	7	5
Stair slab	4	5

Floor	Snow kN/m^2
Roof slab	1.92
Staircase slab	1.92

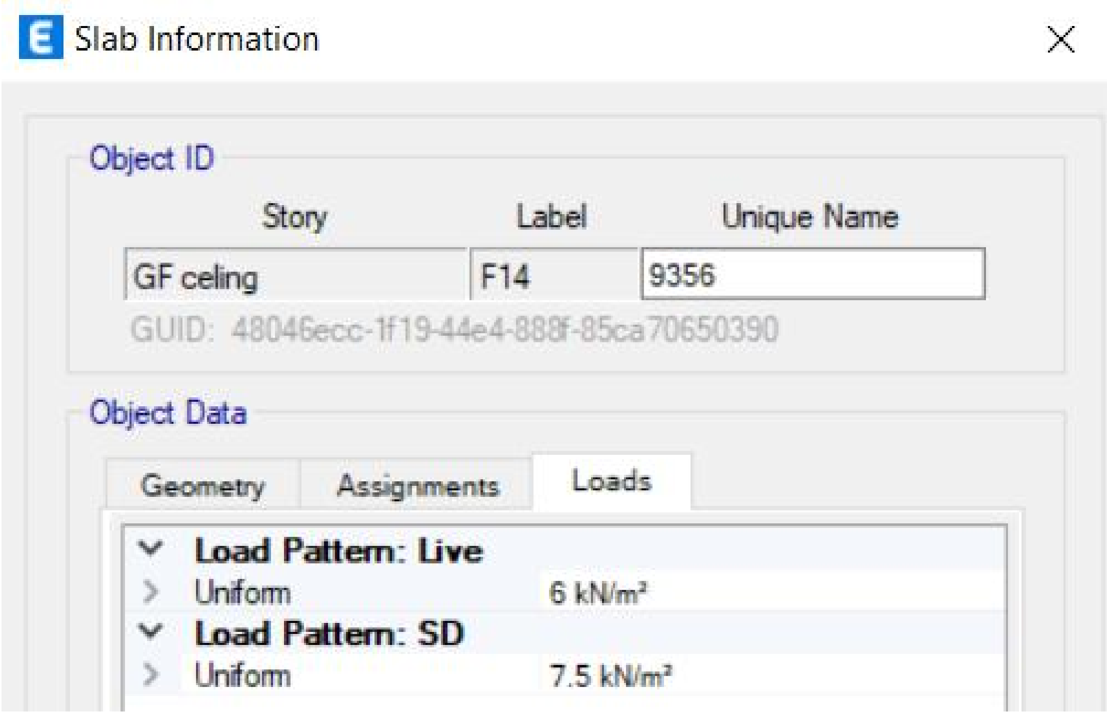


Figure 4.27 sample for load assignment for GF ceiling.

4.2.9 Masses:

Base shear calculations are related to mass source definition, which consider the superimposed dead load and a percentage of live load.

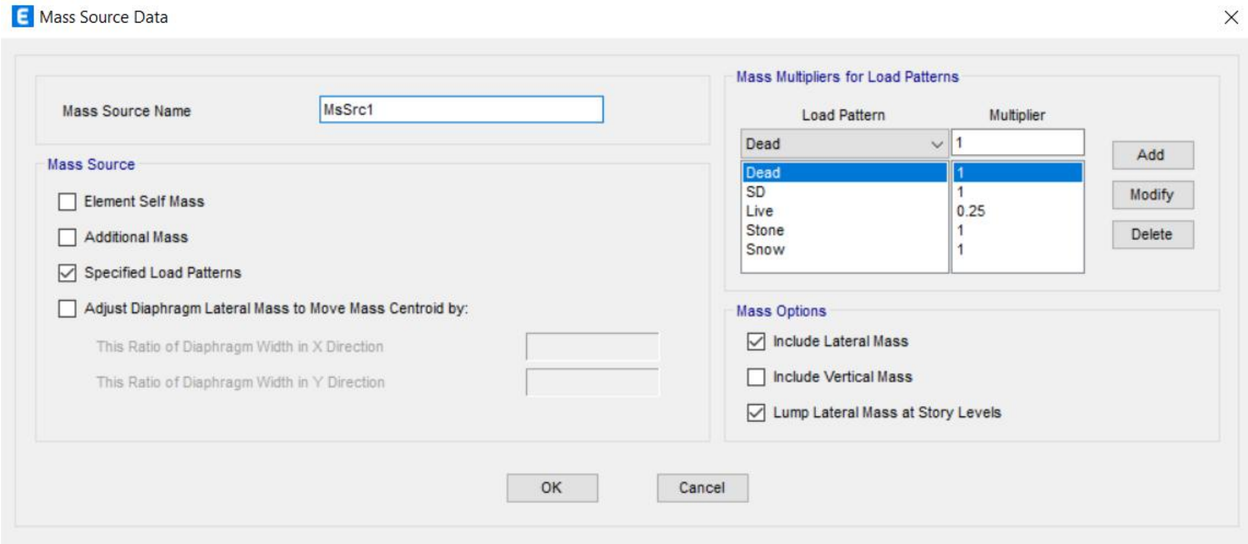


Figure 4.28 Definition of mass source data.

4.2.10 Functions:

A response spectrum function is used to define the response spectrum curve.

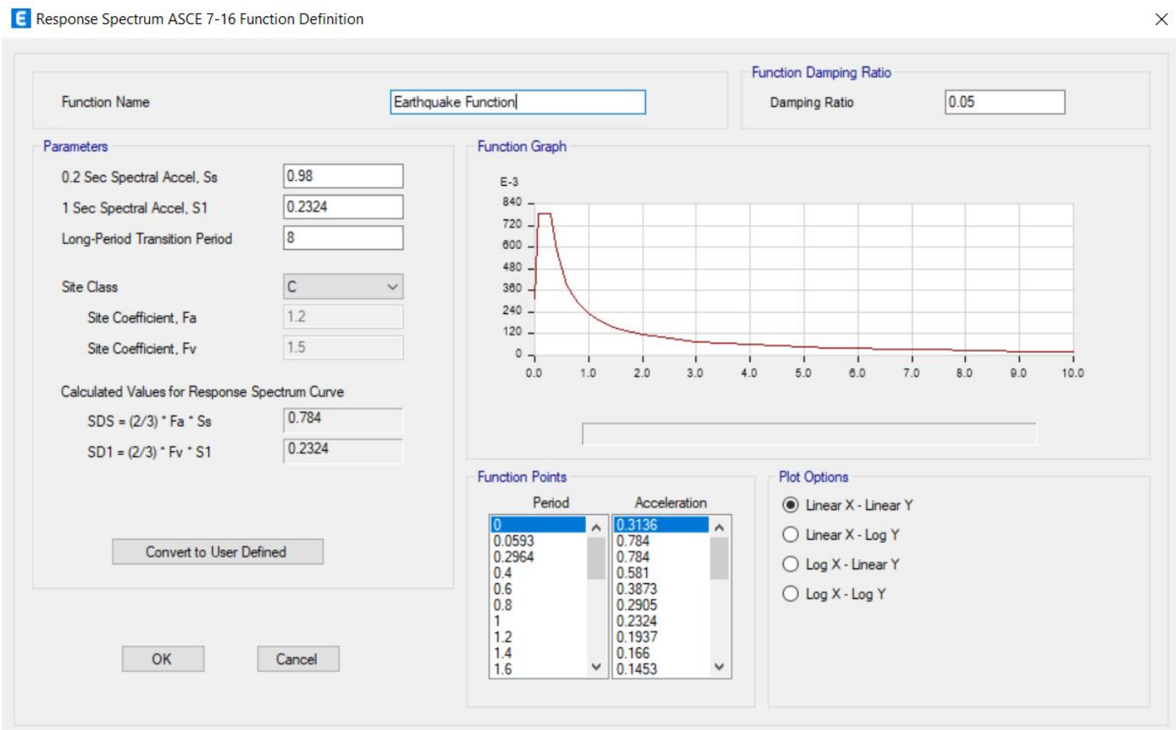


Figure 4.29 Definition of response spectrum function.

4.2.11 Design Specification:

The code used for the design is defined and seismic factors such as: Ω, ρ and SD_s

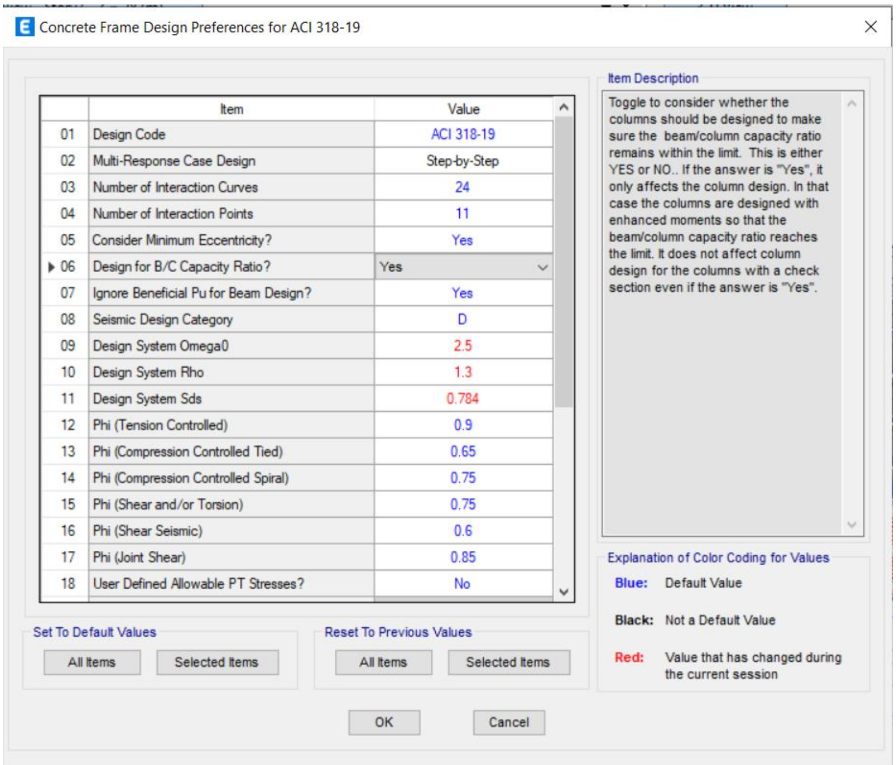


Figure 4.30 Concrete frame design preferences for ACI 318-19.

4.2.12 Supports:

Fixed support was used for each joint in base story.

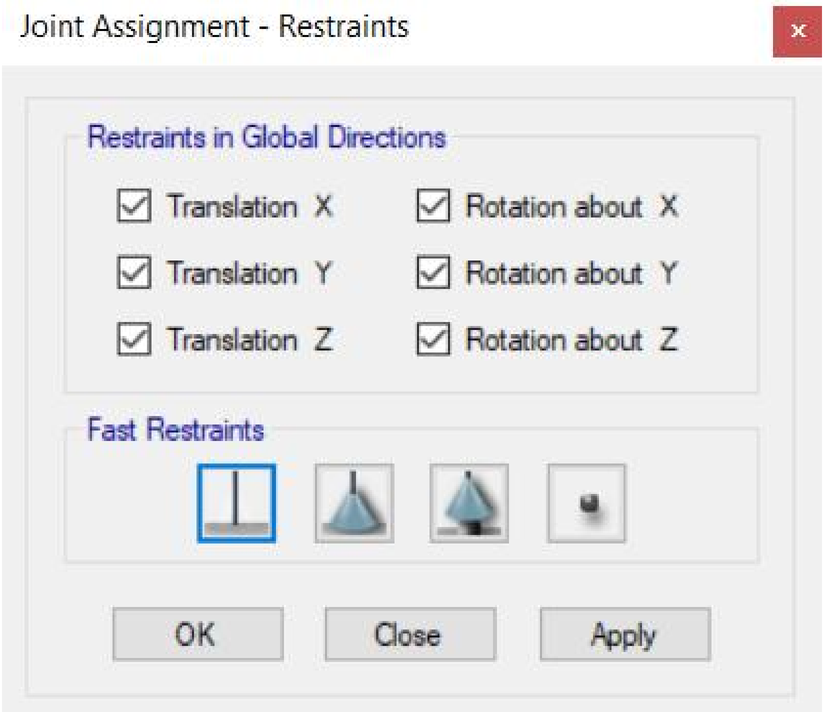


Figure 4.31 Restrain type in ETABS.

4.2.13 Codes:

The used codes are discussed in section 1.4, but in program the selected code is ACI 318-19 for design and ASCE 7-16 for loads.

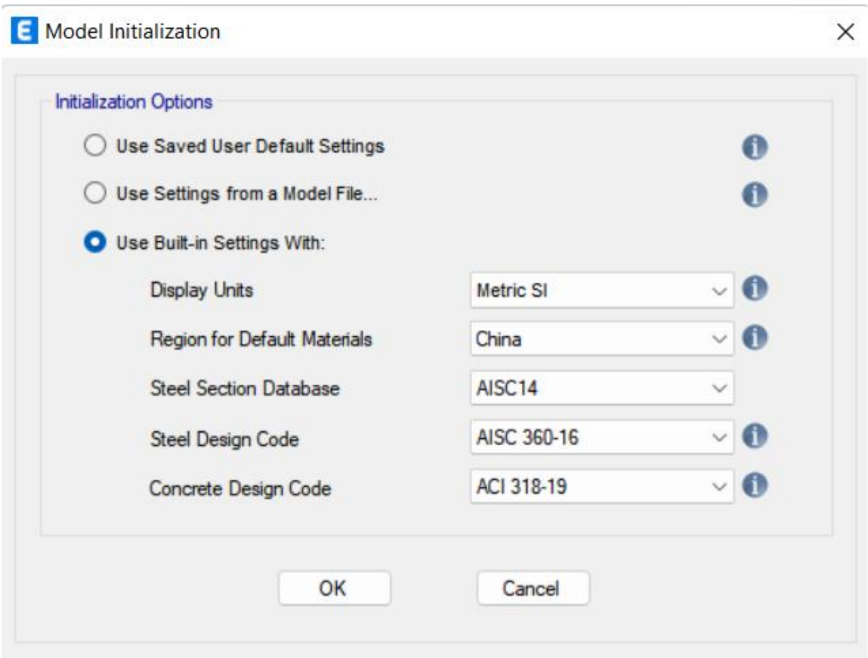


Figure 4.32 Codes used in ETABS

4.3 Evaluation of the preliminary design:

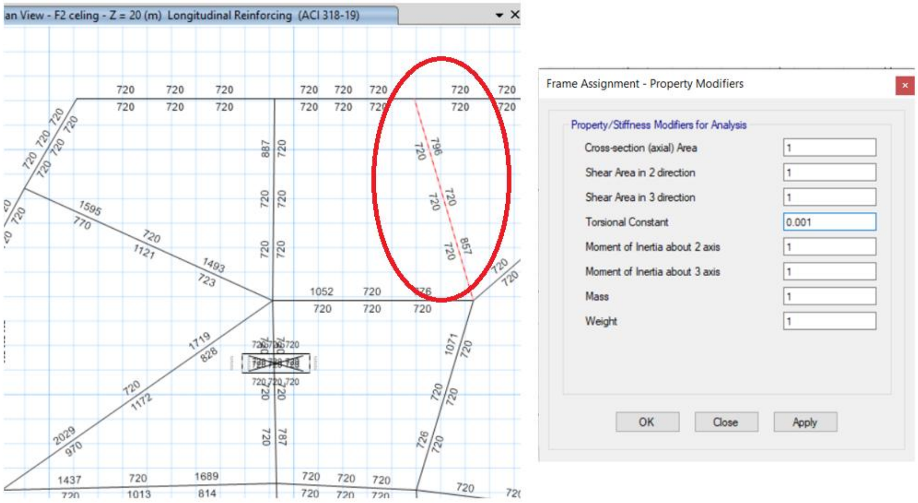
4.3.1 Check Dimensions of Beams:

This check is done by designing frames in the structure and then checking that there is no failure due to shear and torsion

If the beams light up with red color, so one of the two warning messages will appear:

1. Shear stress due to shear force and torsion together exceeds maximum allowed, therefore may torsion be the reason or shear.

2. Reinforcing required exceeds maximum allowed.



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
69.5934	9.4943	37.9771	1351.5	1644.4	829.82	1365

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

Figure 4.33 A red colored beam with new modifiers for this specific beam.

Thid Beam was Red and the problem was Shear Stress due to shear force and torsion together exceeds maximum allowed
and we changed torsion modifier from 0.35 to 0.001 and the torsion was compatibility torsion

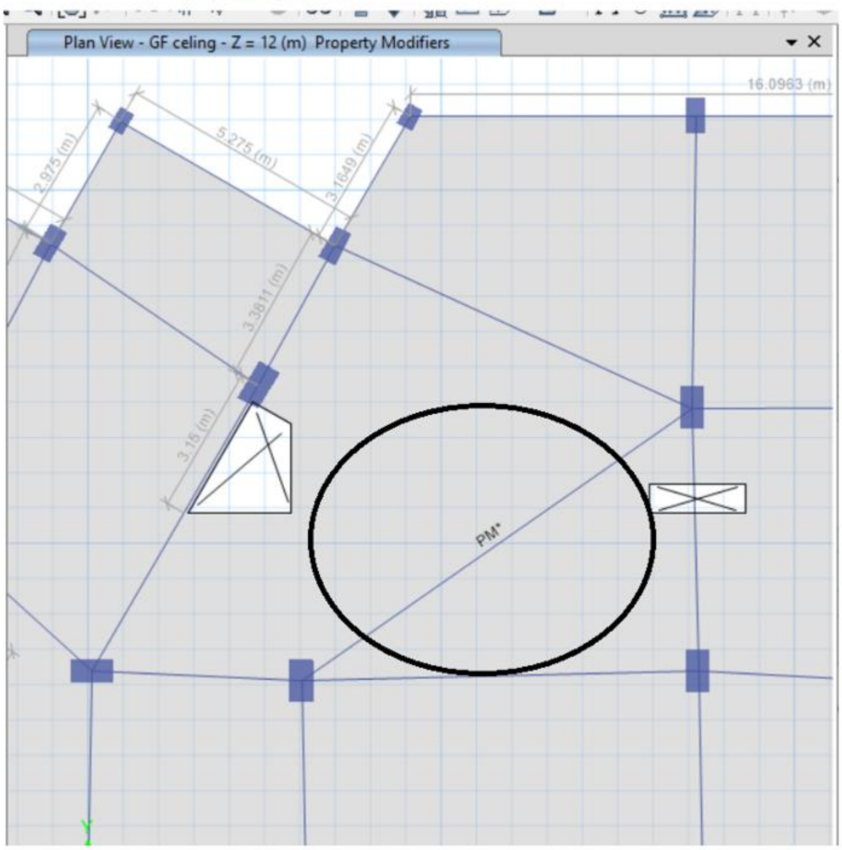


Figure 4.34 A red colored beam with compatibility torsion problem.

4.3.2 Check Dimensions of Columns:

This check is done by designing frames in the structure and then checking the Rebar percentage within the limitation of maximum and minimum.

If the columns light up red so one of the three warning messages will appear:

- Reinforcing required exceeds maximum allowed → enlarge column section.
- Column factored axial load exceeds Euler force → enlarge column section.
- Strong column weak beam is not satisfied → the used framing type is special.

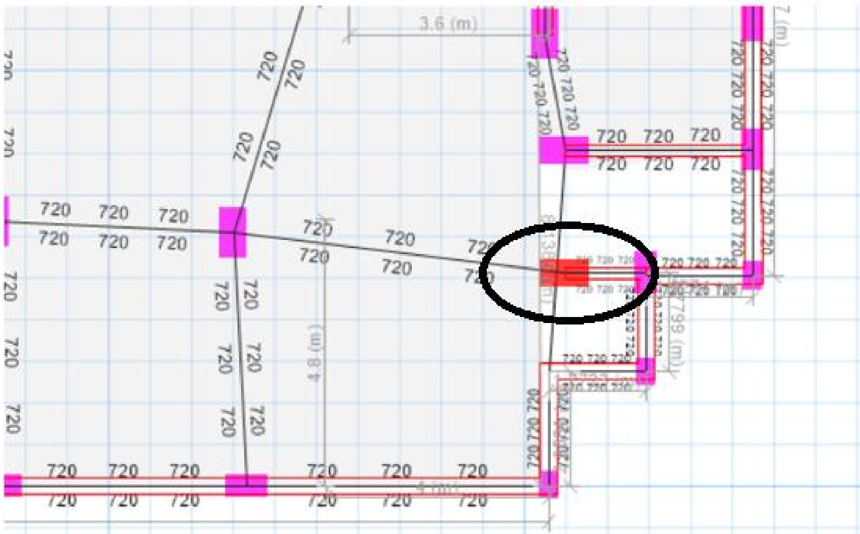


Figure 4.35 A red colored 900*500 Column.

According to the message appeared, it indicates to enlarge column to overcome the error.

As a result, this column was enlarged to 1100*700 section instead of 900*500.

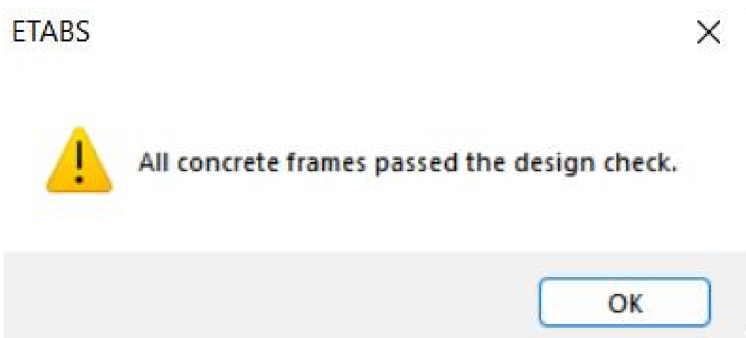


Figure 4.36 verification that all concrete frames have passed the design check

4.4 Verification of structural analysis:

4.4.1 Compatibility of the structure:

Using start animation button ensures that all the elements in the model moves as a one group; which means that all the elements are connected to each other and there are no problems with drawing the model

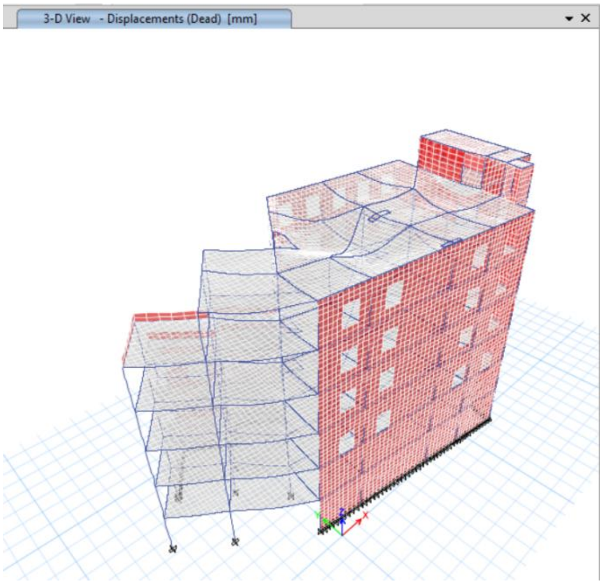


Figure 4.37 compatibility check

4.4.2 Equilibrium Check:

4.4.2.1 Check equilibrium for dead loads:

Calculations of own weight for structural elements (slab, beam, column, and wall) by hand

Table 4.7 Slabs Dead load.

Slab			
Floor	**Area m2**	**Depth m**	**Total Weight kN =Area* Depth * 25**
B2 Ceiling	509.52	0.25	3184.50
B1 Ceiling	510.30	0.25	3189.38
GF Ceiling	510.30	0.25	3189.38
F1 Ceiling	475.97	0.25	2974.81
F2 Ceiling	442.62	0.25	2766.38
Roof	414.44	0.25	2590.25
Stair case	40.38	0.25	10.10
Sum	2903.53		17904.78

Table 4.8 Columns Dead Load.

Column					
Column	**Width m**	**Length m**	**Area m^2**	**Total Length m**	**Total Weight kN**
C 400*750	0.4	0.75	0.3	372.12	2790.9
C 350*500	0.35	0.5	0.175	177.09	774.76875
C 500*900	0.5	0.9	0.45	297.09	3342.2625
Sum					6907.93125

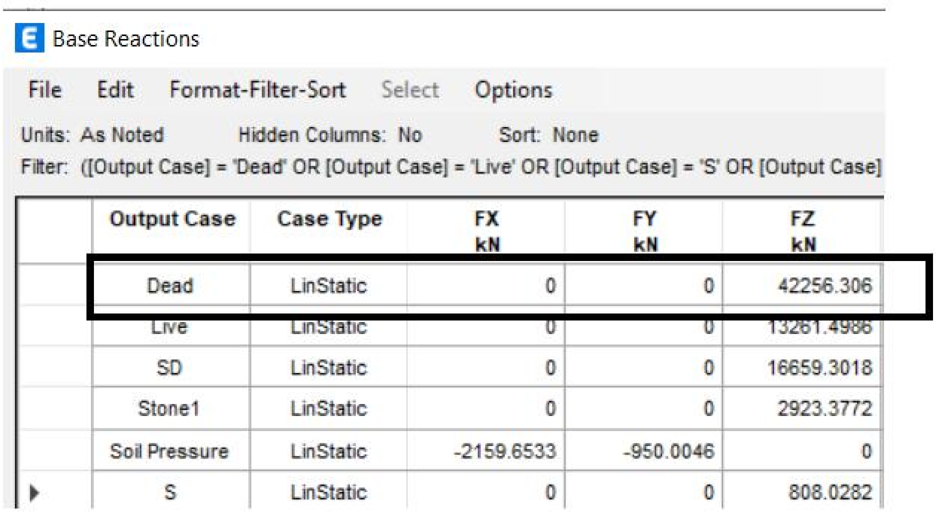
Table 4.9 Beams Dead Load.

Beams					
Beam	Depth (m)	Width (m)	Area (m2)	Total Length (m)	Total Weight
400*600	0.6	0.4	0.24	1629.1	5701.85

Table 4.10 shear wall Dead Load.

Wall			
Wall Section	**Area m2**	**Depth m**	**Total Wight kN**
W 200 mm	1455.4011	0.2	7277.0055
W 300 mm	691.92	0.3	5189.4
Sum			12466.4055

By ETABS:



Base Reactions

File Edit Format-Filter-Sort Select Options

Units: As Noted Hidden Columns: No Sort: None

Filter: ([Output Case] = 'Dead' OR [Output Case] = 'Live' OR [Output Case] = 'S' OR [Output Case]

Output Case	Case Type	FX kN	FY kN	FZ kN
Dead	LinStatic	0	0	42256.306
Live	LinStatic	0	0	13261.4986
SD	LinStatic	0	0	16659.3018
Stone1	LinStatic	0	0	2923.3772
Soil Pressure	LinStatic	-2159.6533	-950.0046	0
S	LinStatic	0	0	808.0282

Figure 4.38 Total dead load from ETABS.

Table 4.11 Dead Load Summation manually and ETABS with %error.

DL Manually	42980.96925
DL ETABS	42256.306
% Error	1.686009559

$$\%error = \frac{\text{DL Manually} - \text{DL ETABS}}{DL\ Manually} \times 100\%$$

$$\%error = \frac{42980.96925 - 42256.306}{42980.96925} \times 100\% = 1.686\% < 5\%, which\ is\ acceptable.$$

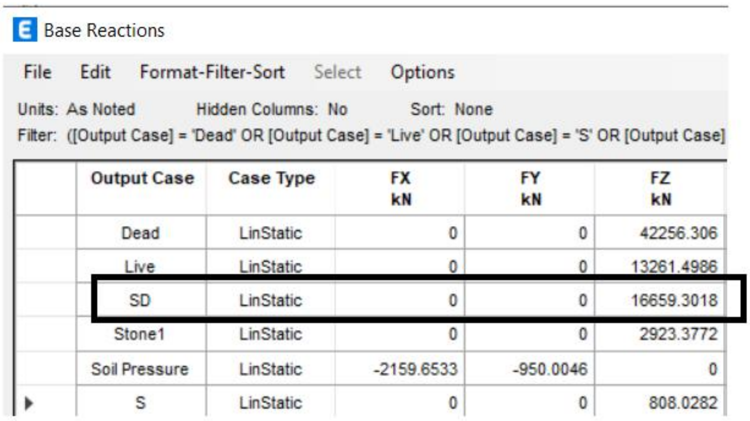
4.4.2.2 Check equilibrium for superimposed dead loads:

Hand calculations:

Table 4.12 Slab SID hand calculations.

Slab SID						
Floor	**Area 1 m2**	**SID Value For Area 1**	**Area 1 * SID**	**Area 2 m2**	**SID Value For Area 2**	**Area 2 *SID**
B2 Ceiling	479.87	4.5	2159.415	29.65	3	88.95
B1 Ceiling	481.76	7	3372.32	28.54	3	85.62
GF Ceiling	481.76	7.5	3613.2	28.54	3	85.62
F1 Ceiling	447.43	7.5	3355.725	28.54	3	85.62
F2 Ceiling	414.08	7	2898.56	28.54	3	85.62
Roof	385.9	0	0	28.54	3	85.62
Stair Case	40.38	0	0	0	0	0
Sum			15399.22			517.05
Slab SID Total Sum manually	15916.27					

By ETABS:



Base Reactions

File Edit Format-Filter-Sort Select Options

Units: As Noted Hidden Columns: No Sort: None

Filter: ([Output Case] = 'Dead' OR [Output Case] = 'Live' OR [Output Case] = 'S' OR [Output Case]

Output Case	Case Type	FX kN	FY kN	FZ kN
Dead	LinStatic	0	0	42256.306
Live	LinStatic	0	0	13261.4986
SD	LinStatic	0	0	16659.3018
Stone1	LinStatic	0	0	2923.3772
Soil Pressure	LinStatic	-2159.6533	-950.0046	0
S	LinStatic	0	0	808.0282

Figure 4.39 Total Superimposed dead load from ETABS.

Table 4.13 Superimposed dead Load Summation manually and ETABS with %error.

SID ETABs	16659.1517
SID Manually	15916.27
%Error	4.45930089

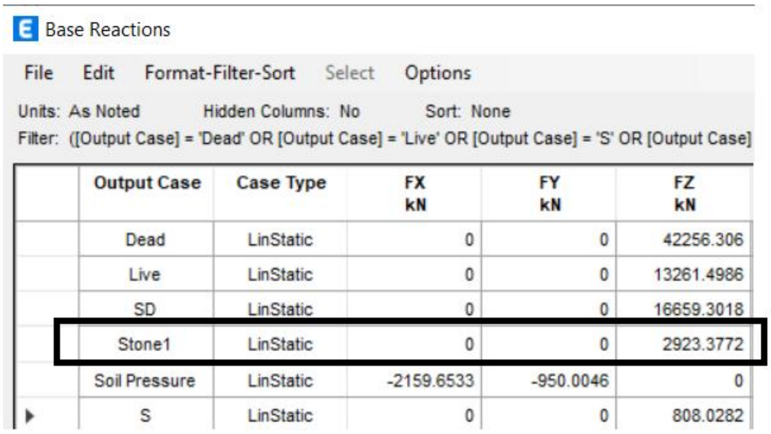
$$\%\text{Error} = \frac{16659.1517-15916.27}{16659.1517} \times 100\% = 4.46\% < 5\%, \textit{which is acceptable.}$$

4.4.2.3 Check equilibrium for stone loads:

Hand calculations:

Table 4.14 Stone walls loads hand calculations

Stone Wall							
Floor	**Wall Length m**	**Wall Height m**	**Wall Area m^2**	**SID kN/m2**	**Sum Opening Area m^2**	**Final Area**	**Final Area * SID kN/ m^2**
Base to B2 Ceiling							
B2 Ceiling to B1 Ceiling							
B1 Ceiling to GF Ceiling	90.0025	4	360.01	2.5	40.95	319.06	797.65
GF Ceiling to F1 Ceiling	72.7353	4	303.3162	2.5	32.175	271.1412	677.853
F1 Ceiling to F2 Ceiling	72.7302	4	303.73005	2.5	29.25	274.48005	686.200125
F2 Ceiling to Roof	72.7302	4	298.834125	2.5	29.25	269.584125	673.960313
Roof to StairCase	19.1844	3.03	58.128732	2.5	2.925	55.203732	138.00933
Sum							2973.67277



Base Reactions

File Edit Format-Filter-Sort Select Options

Units: As Noted Hidden Columns: No Sort: None

Filter: ([Output Case] = 'Dead' OR [Output Case] = 'Live' OR [Output Case] = 'S' OR [Output Case]

Output Case	Case Type	FX kN	FY kN	FZ kN
Dead	LinStatic	0	0	42256.306
Live	LinStatic	0	0	13261.4986
SD	LinStatic	0	0	16659.3018
Stone1	LinStatic	0	0	2923.3772
Soil Pressure	LinStatic	-2159.6533	-950.0046	0
S	LinStatic	0	0	808.0282

Figure 4.40 Total stone loads from ETABS

Table 4.15 Superimposed dead Load Summation manually and ETABS with %error.

Stone ETABS	2923.3772
Stone Manually	2973.672768
%Error	1.691361876

$$\%\text{error} = \frac{2973.672768 - 2923.3772}{2973.672768} \times 100\% = 1.69\% < 5\%, \textit{which is acceptable.}$$

4.4.2.4 Check equilibrium for Live loads:

Hand calculations:

Table 4.16 Live load hand calculations

	Floor	Area m^2	LL kN	Area m^2	LL kN	Total Area m^2	Total LL kN manually
LL Manual	**B2 Ceiling**	479.87	6	29.65	5	509.52	3027.47
	B1 Ceiling	481.76	5	28.54	5	510.3	2551.5
	GF Ceiling	481.76	6	28.54	5	510.3	3033.26
	F1 Ceiling	447.43	5	28.54	5	475.97	2379.85
	F2 Ceiling	414.08	5	28.54	5	442.62	2213.1
	Roof	385.9	0	28.54	5	414.44	142.7
	Staircase	40.38	0	0	0	40.38	0
	Sum					2903.53	13347.88

By ETABS:

Base Reactions

File Edit Format-Filter-Sort Select Options

Units: As Noted Hidden Columns: No Sort: None

Filter: ([Output Case] = 'Dead' OR [Output Case] = 'Live' OR [Output Case] = 'S' OR [Output Case]

Output Case	Case Type	FX kN	FY kN	FZ kN
Dead	LinStatic	0	0	42256.306
Live	LinStatic	0	0	13261.4986
SD	LinStatic	0	0	16659.3018
Stone1	LinStatic	0	0	2923.3772
Soil Pressure	LinStatic	-2159.6533	-950.0046	0
S	LinStatic	0	0	808.0282

Figure 4.41 Total live loads from ETABS

Table 4.17 Live Loads Summation manually and ETABS with %error.

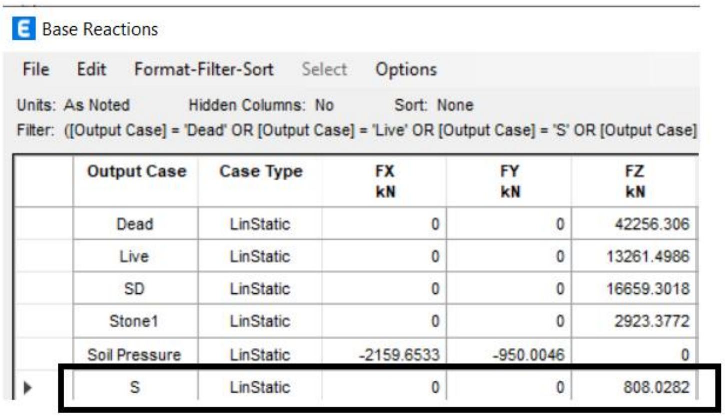
LL ETABS	13261.3233
LL Manually	13347.88
%Error	0.648467772

4.4.2.5 Check equilibrium for snow loads:

Hand calculations:

Table 4.18 Snow load hand calculations

Snow			
Floor	**Area m^2**	**Snow Load kN/m^2**	**Total Snow load Manually kN**
B2 Ceiling	0	0	-
B1 Ceiling	0	0	-
GF Ceiling	0	0	-
F1 Ceiling	0	0	-
F2 Ceiling	0	0	-
Roof	385.9	1.92	740.928
Staircase	40.38	1.92	77.5296
Sum			818.4576



Base Reactions

File Edit Format-Filter-Sort Select Options

Units: As Noted Hidden Columns: No Sort: None

Filter: ([Output Case] = 'Dead' OR [Output Case] = 'Live' OR [Output Case] = 'S' OR [Output Case]

Output Case	Case Type	FX kN	FY kN	FZ kN
Dead	LinStatic	0	0	42256.306
Live	LinStatic	0	0	13261.4986
SD	LinStatic	0	0	16659.3018
Stone1	LinStatic	0	0	2923.3772
Soil Pressure	LinStatic	-2159.6533	-950.0046	0
S	LinStatic	0	0	808.0282

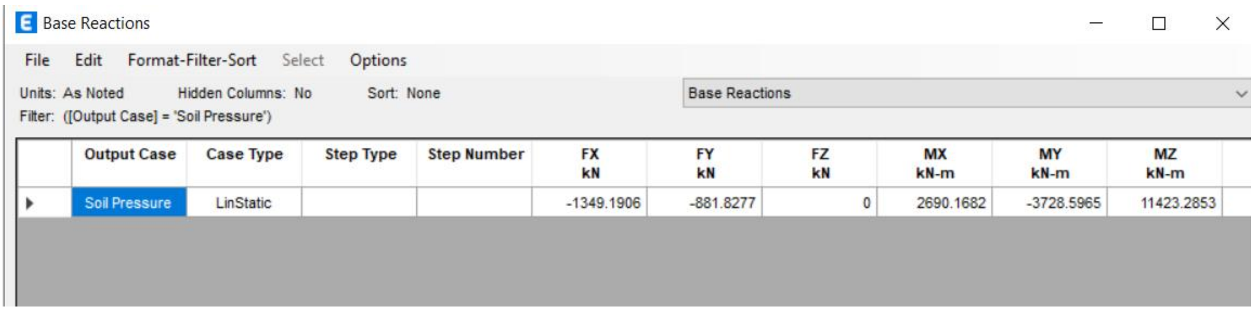
Figure 4.42 **Total Snow loads from ETABS**

Table 4.19 Live Loads Summation manually and ETABS with %error.

Snow ETABS	808.0462
Snow Manually	818.4576
%Error	1.27207567

$$\%\text{error} = \frac{818.4576 - 808.0462}{818.4576} \times 100\% = 1.27\% < 5\%, \textit{which is acceptable}.$$

4.4.2.6 Check equilibrium for Soil loads:



Base Reactions

File Edit Format-Filter-Sort Select Options

Units: As Noted Hidden Columns: No Sort: None

Filter: ([Output Case] = 'Soil Pressure')

Base Reactions

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
Soil Pressure	LinStatic			-1349.1906	-881.8277	0	2690.1682	-3728.5965	11423.2853

Figure 4.43 Total soil loads from ETABS

Table 4.20 Soil error percentage

Direction	**X-Dir.**	**Y-Dir.**	**Z-Dir.**
Sum. Of load (KN)	1289.7	838.24	0
%error	4.409	4.942	0

Soil percentage of error is < 5% → ok.

4.4.3 Seismic Forces:

The structural analysis required consist of one of the types permitted in Table 12.6-1, based on the structures seismic design category, structural system, dynamic properties, and regularity.

Table 12.6-1 Permitted Analytical Procedures

Seismic Design Category	Structural Characteristics	Equivalent Lateral Force Procedure, Section 12.8[a]	Modal Response Spectrum Analysis, Section 12.9.1, or Linear Response History Analysis, Section 12.9.2[a]	Nonlinear Response History Procedures, Chapter 16[a]
B, C	All structures	P	P	P
D, E, F	Risk Category I or II buildings not exceeding two stories above the base	P	P	P
	Structures of light-frame construction	P	P	P
	Structures with no structural irregularities and not exceeding 160 ft (48.8 m) in structural height	P	P	P
	Structures exceeding 160 ft (48.8 m) in structural height with no structural irregularities and with $T < 3.5T_s$	P	P	P
	Structures not exceeding 160 ft (48.8 m) in structural height and having only horizontal irregularities of Type 2, 3, 4, or	P	P	P

Figure 4.42 Permitted analytical procedures

In this project, the seismic design category is D and the structure characteristics is all other structures.

The Permitted analytical procedures is Response spectrum and seismic response history.

The analysis method used in this project is response spectrum analysis using equivalent lateral force method to base shear check.

4.4.3.1 Check equilibrium for base shear:

We have to check that Base shear value in x manually approximately equals to base shear value in x from ETABS Eqx static load case, and so on for y direction.

$$V_{Manually} = \text{W.t Building} \times \text{Cs final}$$

Weight Combination= 1.0 DL + 1.0 SID + 1.0 Stone + 0.25 LL

Base Reactions

File Edit Format-Filter-Sort Select Options

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

Filter: ([Output Case] = 'Wright Building')

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
Wright Building	Combination			0	0	62403.3527	532354.2798	-687879.8382	-2.331E-06

Figure 4.44 Building weight from ETABS

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

1

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} \quad \text{for} \quad T \le T_L \qquad (12.8\text{-}3)$$

2

$$C_s = \frac{S_{D1}T_L}{T^2\left(\frac{R}{I_e}\right)} \quad \text{for} \quad T > T_L \qquad (12.8\text{-}4)$$

2

$$C_s = 0.044 S_{DS} I_e \ge 0.01 \qquad (12.8\text{-}5)$$

3

Figure 4.45 Cs rules for base shear requirements

After this check, the values of dynamic earthquakes must be equal to the value of earthquakes in the static method, but the values were not equal on ETAPS, which led to change the scale factor for each of the dynamic earthquake forces in X and Y.

Modification factor in X direction = 1.414

Modification factor in Y direction = 1.378

The figures below show the scale factor for earthquake loads (equivalent and dynamic) before and after changes:

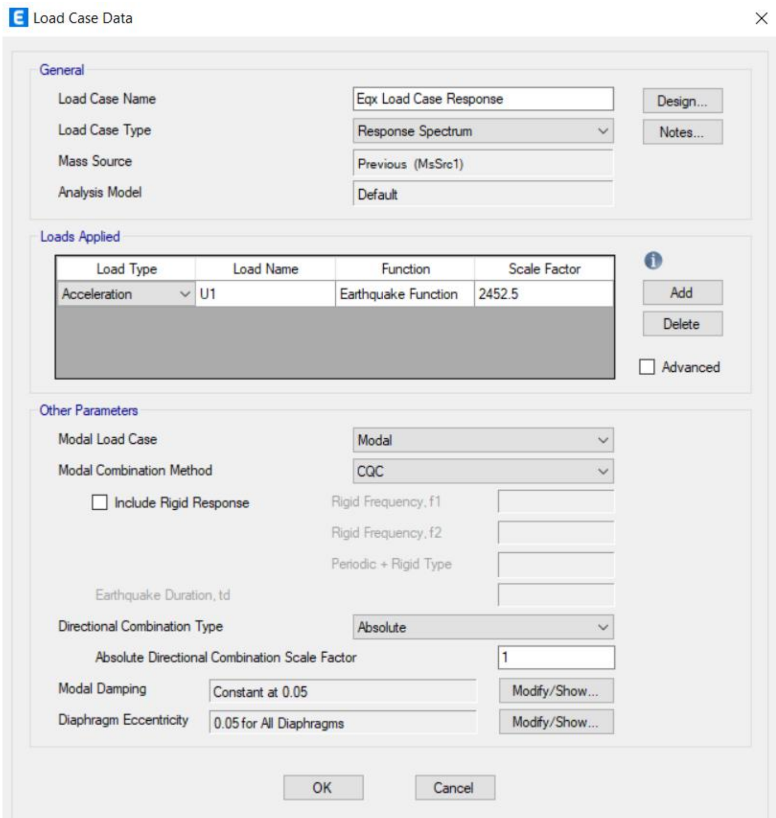


Figure 4.46 Definition of EDx before modification.

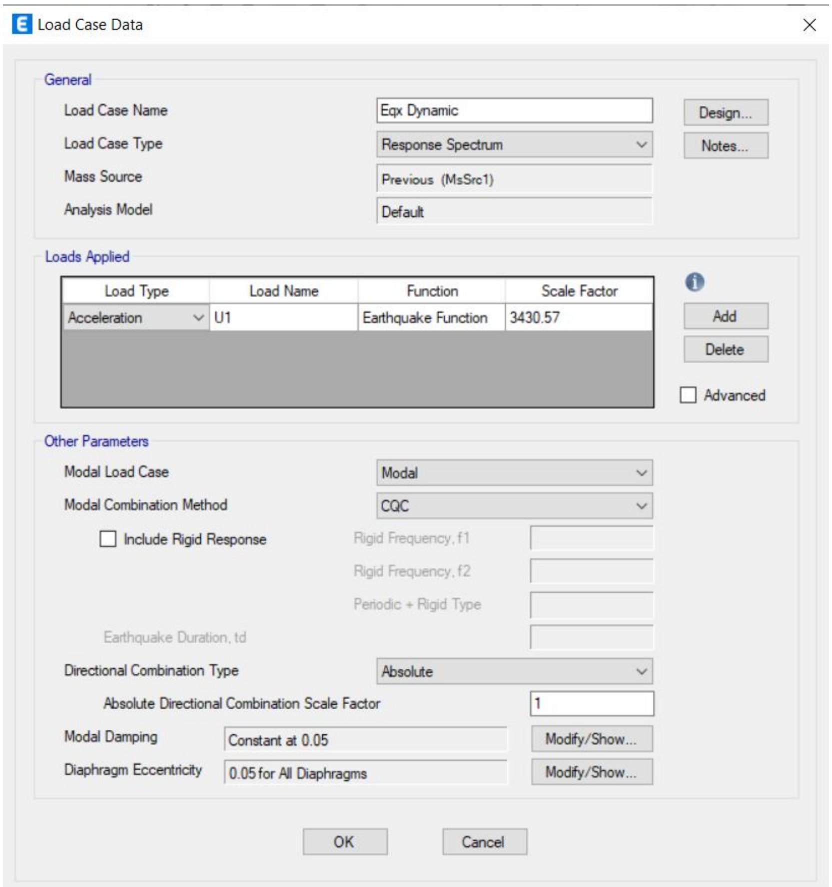


Figure 4.47 Definition of EDx after modification.

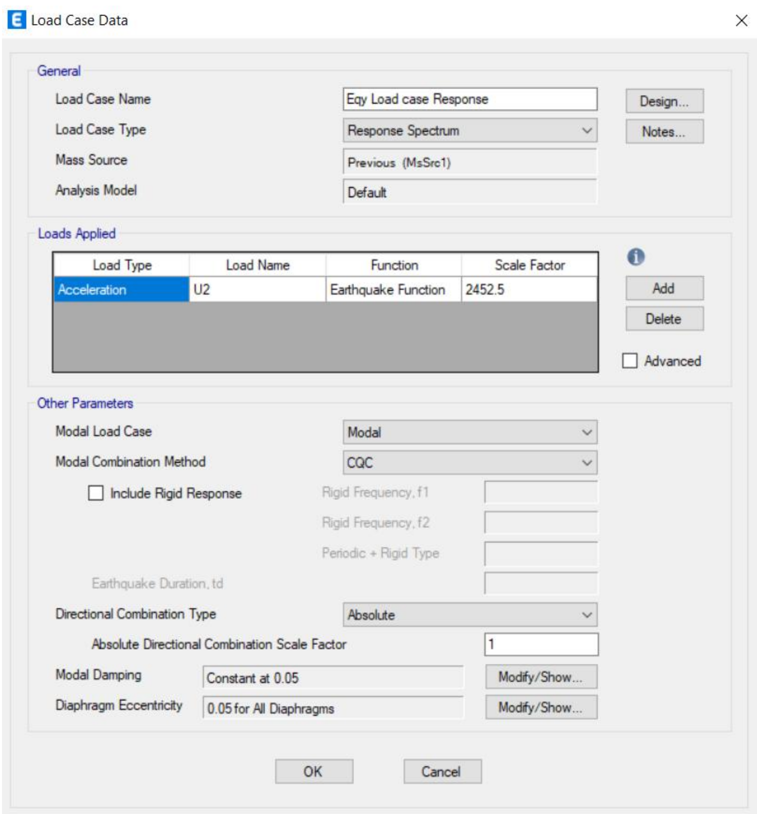


Figure 4.48 Definition of EDy before modification.

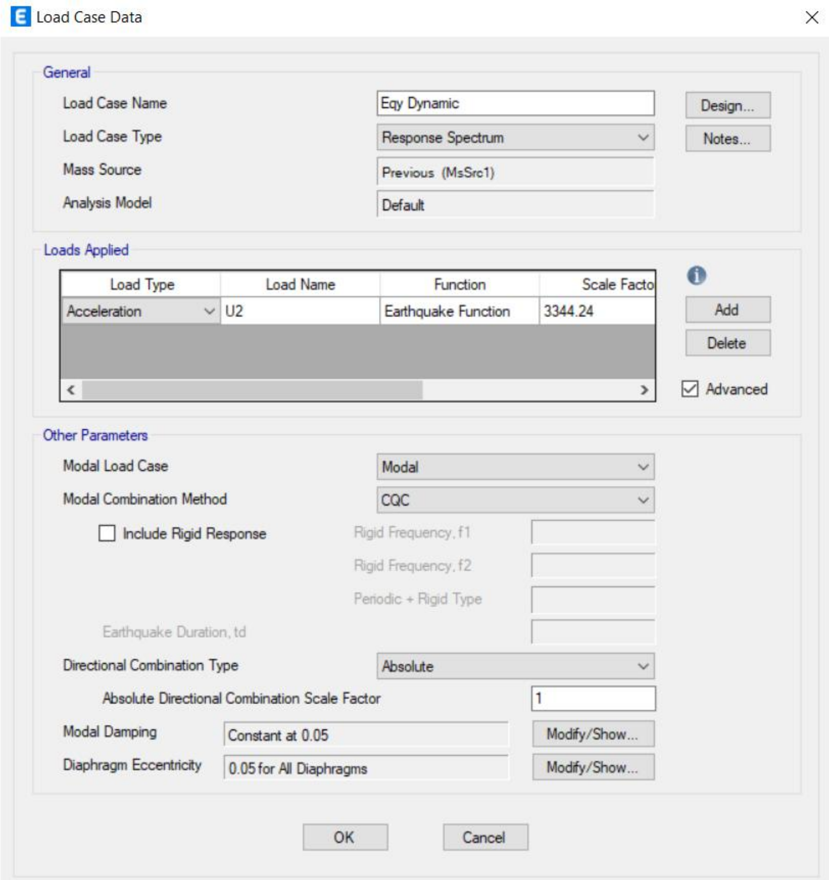


Figure 4.49 Definition of EDy after modification.

Axis	T From Mode	Ta limit	Tfinal	Cs1	Cs2(min)	Cs3(max)
X	0.233	0.849009	0.233	0.196	0.24935622	0.051744
Y	0.426	0.849009	0.426	0.196	0.13638498	0.051744

Table 4.21 Modification of Scale Factor

Min(Cs1,Cs2)	Cs Final	V manual	V Etabs static	Scale Factor Old	Scale Factor New
0.196	0.196	12231.06	-12411.592	2425.5	3430.57
0.136384977	0.136384977	8510.88	-8663.384	2425.5	3344.24

Base Reactions

File Edit Format-Filter-Sort Select Options

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

Filter: ([Output Case] = 'Eqx Dynamic' OR [Output Case] = 'Eqx Load Case Response' OR [Output Case] = 'Eqy Dynamic' OR [Output Case] = 'Eqy Load case Response')

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
Eqx Load Cas...	LinRespSpec	Max		12570.8605	3198.7981	4.906E-05	50432.4488	211182.3946	148811.0589
Eqy Load cas...	LinRespSpec	Max		4149.2631	8817.9532	4.49E-05	138687.8837	67205.2334	131663.1338
Eqx	LinStatic	Step By Step	1	-12411.5922	0	-3.264E-05	-0.0001	-202705.12	102610.4216
Eqx	LinStatic	Step By Step	2	-12411.5922	0	-3.207E-05	-0.0001	-202705.12	113685.8181
Eqx	LinStatic	Step By Step	3	-12411.5922	0	-3.322E-05	-0.0001	-202705.12	91535.0252
Eqy	LinStatic	Step By Step	1	7.551E-07	-8663.384	3.342E-05	141489.6877	-0.0008	-102109.5785
Eqy	LinStatic	Step By Step	2	7.654E-07	-8663.384	3.275E-05	141489.6877	-0.0008	-117292.7301
Eqy	LinStatic	Step By Step	3	7.449E-07	-8663.384	3.409E-05	141489.6877	-0.0009	-86926.4268

Figure 4.49 base shear values for Eqx response spectrum

Figure 4.50 base shear values for Eqy response spectrum

Base Reactions

File Edit Format-Filter-Sort Select Options

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

Filter: ([Output Case] = 'Eqx Dynamic' OR [Output Case] = 'Eqx Load Case Response' OR [Output Case] = 'Eqy Dynamic' OR [Output Case] = 'Eqy Load case Response')

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
Eqx Load Cas...	LinRespSpec	Max		12570.8605	3198.7981	4.906E-05	50432.4488	211182.3946	148811.0589
Eqy Load cas...	LinRespSpec	Max		4149.2631	8817.9532	4.49E-05	138687.8837	67205.2334	131663.1338
Eqx	LinStatic	Step By Step	1	-12411.5922	0	-3.264E-05	-0.0001	-202705.12	102610.4216
Eqx	LinStatic	Step By Step	2	-12411.5922	0	-3.207E-05	-0.0001	-202705.12	113685.8181
Eqx	LinStatic	Step By Step	3	-12411.5922	0	-3.322E-05	-0.0001	-202705.12	91535.0252
Eqy	LinStatic	Step By Step	1	7.551E-07	-8663.384	3.342E-05	141489.6877	-0.0008	-102109.5785
Eqy	LinStatic	Step By Step	2	7.654E-07	-8663.384	3.275E-05	141489.6877	-0.0008	-117292.7301
Eqy	LinStatic	Step By Step	3	7.449E-07	-8663.384	3.409E-05	141489.6877	-0.0009	-86926.4268

Figure 4.51 base shear values for Eq$_{x,y}$ static

Table 4.22 Error percentage for base shear

Axis	Base Shear Manually	Base Shear Static
X	12231.06	12411.592
Y	8510.88	8663.384
%error X	1.454543462	<5% ok
%error Y	1.760328297	<5% ok

4.4.3.2 Story Forces:

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

$$C_{vx} = \frac{w_x h_x^k}{\sum_{n}^{i=1} w_i h_i^k}$$ (ASCE 7-22 equation (12.8-12))

$$F_x = C_{vx} V$$ (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).

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

hi and hx: the height (m) from the base to Level i or x.

k: an exponent related to the structure period.

As follows:

For structures that have a period of 0.5 s or less, k = 1.

for structures that have a period of 2.5 s or more, k = 2.

and for structures that have a period between 0.5 and 2.5 s, k shall be 2 or shall be determined by linear interpolation between 1 and 2.

K_x= 1.1745

K_y= 1.1745

Table 4.23 Calculation of k; an exponent related to the structure period

K=1	**Ta = 0.5**	**--> K = 1.1745**
K=?	**Ta = 0.849**	
K = 2	**Ta = 2.5**	

Table 4.24 Manually force X distribution

Story	K	H(m) =hx	Weight Accumulative	Weight for Individual Floor(KN) Wx	Wx*(hx)^k	Cvx	Csx final	Vx = Csx * Total W kN	F(X) Force(KN)
Stair Case	1.1745	27.03	1066.394	1066.39	51241.4	0.03859	0.196	12231.06	471.975
Roof	1.1745	24	7252.902	6186.51	258526	0.19469	0.196	12231.06	2381.23
F2 C	1.1745	20	17122.16	9869.25	332923	0.25071	0.196	12231.06	3066.49
F1 C	1.1745	16	27775.7	10653.5	276524	0.20824	0.196	12231.06	2547.01
GF C	1.1745	12	39318.93	11543.2	213710	0.16094	0.196	12231.06	1968.45
B1 C	1.1745	8	51398.77	12079.8	138912	0.10461	0.196	12231.06	1279.49
b2 C	1.1745	4	62403.35	11004.6	56065.2	0.04222	0.196	12231.06	516.406
Sum				62403.4	1327902	1			12231.1

Table 4.25 Etabs force X distribution

story	output case	Vx(KN) acummulative	Force(KN)
Stair Case	Eqx D Static	-313.9961	-313.9961
Roof	Eqx D Static	-2327.7166	-2013.7205
F2 C	Eqx D Static	-5454.7112	-3126.9946
F1 C	Eqx D Static	-8151.7722	-2697.061
GF C	Eqx D Static	-10336.5546	-2184.7824
B1 C	Eqx D Static	-11756.0812	-1419.5266
b2 C	Eqx D Static	-12411.5922	-655.511
sum			12411.592

Table 4.26 Manually force Y distribution

Story	K	H(m) =hy	Weight Accumulative	Weight for Individual Floor(KN) Wx	Wy*(hy)^k	Cvy	Csy	Vy kN	F(y) Force(kN)
Stair Case	1.1745	27.03	1066.394	1066.39	51241.4	0.03859	0.136385	8510.88	328.42
Roof	1.1745	24	7252.902	6186.51	258526	0.19469	0.136385	8510.88	1656.96
F2 C	1.1745	20	17122.16	9869.25	332923	0.25071	0.136385	8510.88	2133.79
F1 C	1.1745	16	27775.7	10653.5	276524	0.20824	0.136385	8510.88	1772.32
GF C	1.1745	12	39318.93	11543.2	213710	0.16094	0.136385	8510.88	1369.73
B1 C	1.1745	8	51398.77	12079.8	138912	0.10461	0.136385	8510.88	890.325
B2 C	1.1745	4	62403.35	11004.6	56065.2	0.04222	0.136385	8510.88	359.337
sum				62403.4	1327902	1			8510.88

Table 4.27 ETABS force Y distribution

story	output case	Vy(KN) acummulative	Force(KN)
Stair Case	Eqy D Static	-219.1716	-219.1716
Roof	Eqy D Static	-1624.7636	-1405.592
F2 C	Eqy D Static	-3807.4292	-2182.6656
F1 C	Eqy D Static	-5689.9979	-1882.5687
GF C	Eqy D Static	-7214.9923	-1524.9944
B1 C	Eqy D Static	-8205.8325	-990.8402
B2 C	Eqy D Static	-8663.384	-457.5515
sum			8663.384

Table 4.28 Error Percentage of story Forces

Force	Sum Force X	Sum Force Y
Manually	12231.05713	8510.879793
ETABS	12411.5922	8663.384
%Error	1.45456818	1.760330685

4.4.3.3 Period:

Refer to section 1.6.7

T manually = T limit = Cu*Ta,

Ta = Ct*(hn)^x

Table 4.29 requirements for $T_{manually}$ calculation

Ct	0.0488
x	0.75
hn	27.03
Ta	0.578501
Cu	1.4676
T limit = T	0.849009

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

Figure 4.52 Values of Cu

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

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

[a]Metric equivalents are shown in parentheses.

Figure 4.53 Values of Approximate Period Parameters Ct and X

Tx ETABS = 0.426 Sec.

Ty ETABS = 0.233 Sec.

Table 4.30 ETABS Period

Mode	Period Sec	UX	UY	Sum UX	Sum UY
1	0.426	0.0068	0.659	0.0068	0.6593
2	0.233	0.6861	0.007	0.6929	0.6663
3	0.183	0.0009	0.037	0.6938	0.7033
4	0.148	0.0018	0.176	0.6956	0.8791
5	0.089	0.155	3E-04	0.8506	0.8793
6	0.084	0.0053	0.046	0.8559	0.9248
7	0.076	0.011	0.022	0.8669	0.9469
8	0.073	0.0109	#####	0.8778	0.9469
9	0.066	0.0011	0.016	0.8789	0.9633
10	0.06	0.0298	0.004	0.9087	0.9667
11	0.057	0.0112	0.001	0.9199	0.9678
12	0.055	0.0198	9E-04	0.9397	0.9687

Table 4.31 Final X and y Period

Tx ETABS	0.233
Ty ETABS	0.426
T Manually	0.84901
T Final X	0.84901
T Final Y	0.84901

4.4.3.4 Type of lateral force structural system:

To determine the system of load resistance, a horizontal load must be inserted in a certain amount on all floors, after which the reactions to the walls will be read and the percentage of it will compute and compare with the following:

Table 4.32 Type of lateral force structural system

Percentage of Loads Resisting by Walls	Type
Less than 25%	Moment Resisting Frame System
25% to 75%	Dual System
More than 75%	Shear wall Resisting System

\

For X direction

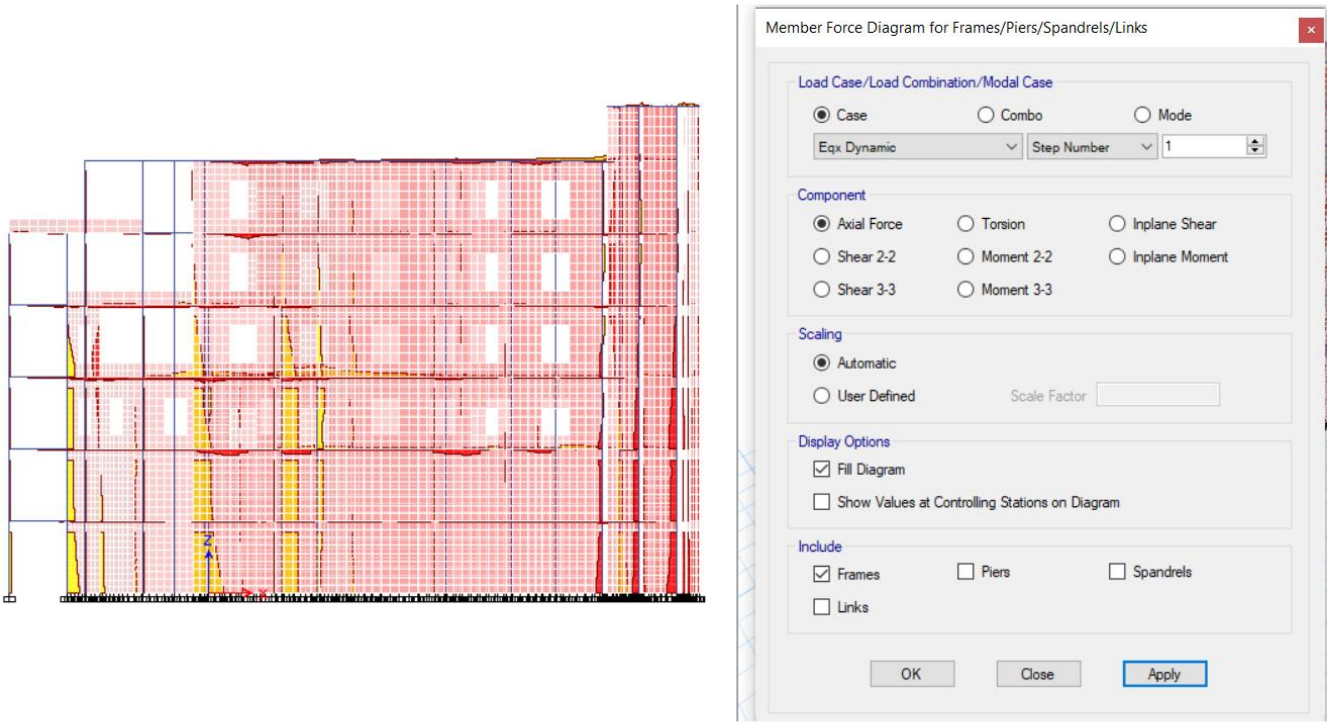


Figure 4.54 Display Axial Force due to Eqx Response Spectrum Load Case

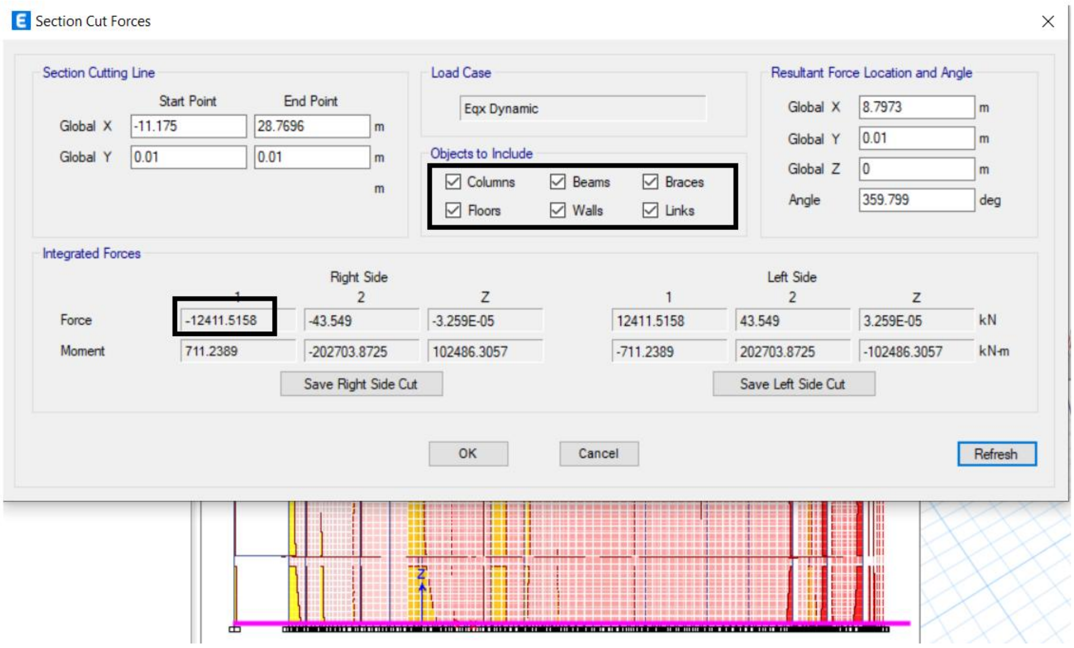


Figure 4.54 Total vertical reactions in X direction

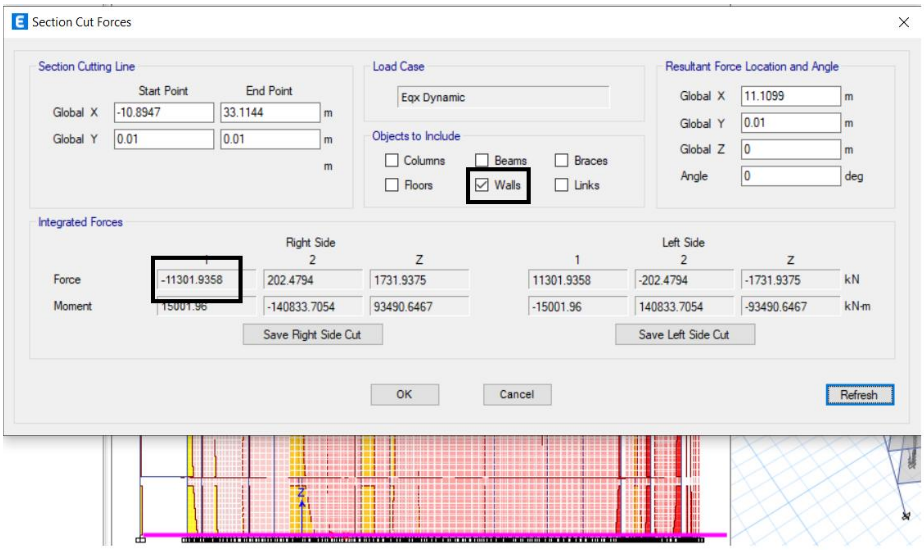


Figure 4.55 Walls vertical reactions in X direction

Table 4.33 Type of lateral force structural system in X

Total vertical reactions in X direction	12411.5158
Wall vertical reactions in X direction	11301.9358
% =	91%
Result > 75%	Shear wall Resisting System

For Y-Direction:

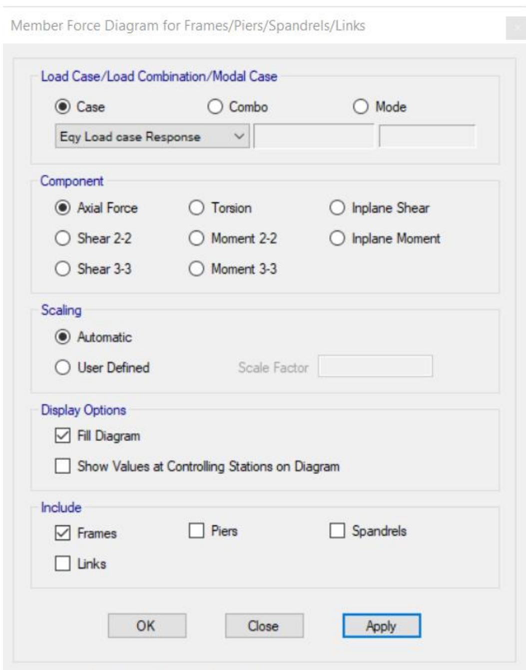


Figure 4.56 Display Axial Force due to Eqy Response Spectrum Load Case

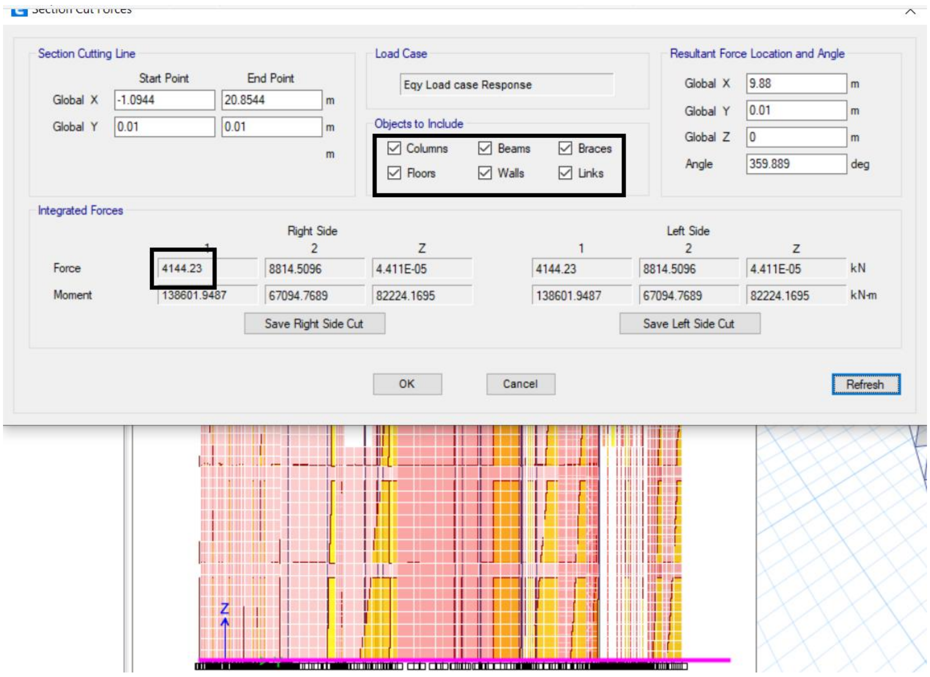


Figure 4.57 Total vertical reactions in Y direction

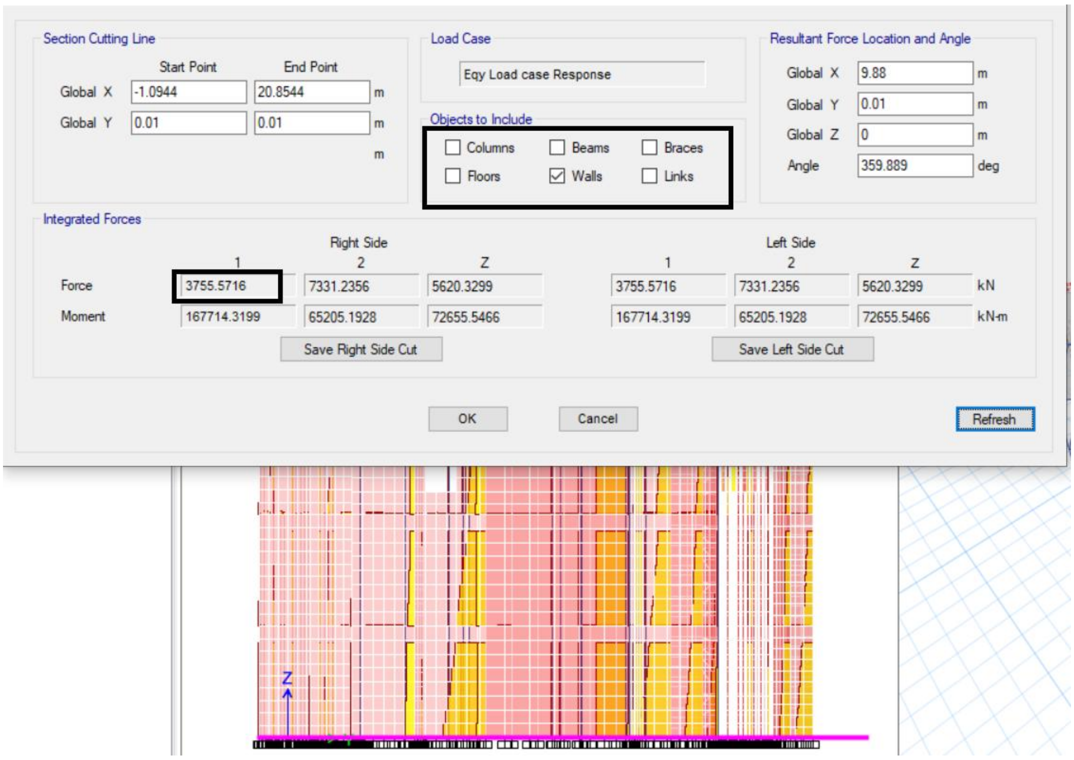


Figure 4.58 Walls vertical reactions in Y direction

Table 4.34 Type of lateral force structural system in y

Total vertical reactions in Y direction	4144.23
Wall vertical reactions in Y direction	3755.5716
%Error	91%
Result > 75%	Shear wall Resisting System

4.4.3.5 Mass participation ratio for modal analysis:

Table 4.35 Mass participation ratio

Case	Mode	Period, Sec	UX	UY	UZ	Sum UX	Sum UY
Modal	1	0.425	0.0003	0.6613	0	0.0003	0.6613
Modal	2	0.233	0.6905	0.0004	0	0.6907	0.6617
Modal	3	0.184	0.003	0.0358	0	0.6937	0.6974
Modal	4	0.147	0.0001	0.1746	0	0.6938	0.872
Modal	5	0.089	0.153	0.0013	0	0.8468	0.8733
Modal	6	0.083	0.002	0.0457	0	0.8488	0.919
Modal	7	0.076	0.0145	0.0192	0	0.8633	0.9382
Modal	8	0.073	0.0109	0.0002	0	0.8742	0.9384
Modal	9	0.065	0.0028	0.0169	0	0.877	0.9554
Modal	10	0.06	0.027	0.007	0	0.904	0.9624
Modal	11	0.057	0.0116	0.0005	0	0.9156	0.9628
Modal	12	0.055	0.0207	0.0002	0	0.9362	0.963

The Result:

In general, we need enough modes to achieve 90% of Sum UX and SUM UY both, and we accomplished that at mode 10. Therefore 10 modes are enough but we will take 12 modes.

4.4.4 Structural irregularity:

There are two types of structural irregularities, horizontal and vertical, the existence of irregulates can affect and change the forces upon which the structure is exposed to, therefore it is important to have these irregularities checked taking into consideration the seismic design category (project seismic design category is D) which determines what types will be applicable to be checked, the intent of this process is to see the effect on the structure in order to achieve and ensure safety and serviceability of the structure.

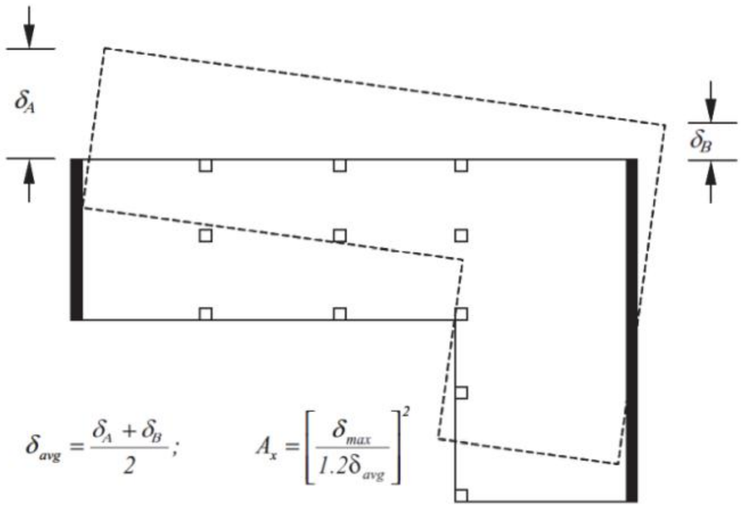
4.4.4.1 Horizontal irregularity:

Table 4.36 Horizontal irregularities

Type	Horizontal irregularities description	Commentary
1	Torsional Irregularity: Torsional irregularity, defined to exist where either: • More than 75% of any story's lateral strength below the diaphragm is provided at or on one side of the center of mass, or • The Torsional Irregularity Ratio (TIR) exceeds 1.2.	Exist, the ratio exceeded 1.2
2	Reentrant Corner Irregularity: Reentrant corner irregularity, defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 20% of the plan dimension of the structure in the given direction.	Dose not exists.
3	Diaphragm Discontinuity Irregularity: Diaphragm discontinuity irregularity, defined to exist where there is a diaphragm with an abrupt discontinuity or variation in stiffness, including one that has a cut-out or open area greater than 25% of the gross enclosed diaphragm area, or a change in effective diaphragm stiffness of more than 50% from one story to the next.	Exist, elements of such kind will be designed using over strength factor.
4	Out-of-Plane Offset Irregularity: Out-of-plane offset irregularity, 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.	Dose not exists.
5	Non-parallel System Irregularity: Non-parallel system irregularity, defined to exist where vertical lateral force-resisting elements are not parallel to the major orthogonal axes of the seismic force-resisting system	Dose not exists.

Torsional Irregularity:

Torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with Ax= 1.0, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure, as shown in Figure 4.60.



$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2$$

Figure 4.59 Torsional irregularity definition

Where:

δmax = maximum displacement at level x computed assuming. Ax = 1.

δavg = average of the displacements at the extreme points of the structure at level x computed.

The torsional amplification factor Ax shall not be less than 1 and is not required to exceed 3.0

Table4.37 Torsional Irregularity Limitations

If δ(Δ) max /Δavg	**Δ <= 1.2**	
	1.2 < Δ < 1.4	**Torsional Irregular**
	Δ >= 1.4	**Extremely Irregular**

Table 4.38 Torsion irregularity in X-direction.

Story	Max Drift	Avg Drift	Ratio	(Ratio/1.2)^2	Result
Stair case	0.000244	0.000226	1.082	0.81	Regular
Roof	0.000194	0.000164	1.18	0.97	Regular
F2 celing	0.000316	0.000263	1.2	1	Regular
F1 celing	0.000417	0.000342	1.219	1.03	Regular
GF celing	0.000314	0.000247	1.274	1.13	Regular
B1 celing	0.000163	0.000131	1.238	1.07	Regular
B2 ceiling	0.000116	0.000106	1.103	0.85	Regular

No torsion irregularity in X direction.

Table 4.39 Torsion irregularity in Y-direction.

Story	Combo	Max Drift	Avg Drift	Ratio	(Ratio/1.2)^2	Result
Stair case	Eqy	0.000281	0.000238	1.18	0.97	Regular
Roof	Eqy	0.000639	0.000437	1.462	1.49	Extremely Irregular
F2 celing	Eqy	0.00092	0.000598	1.538	1.64	Extremely Irregular
F1 celing	Eqy	0.000939	0.000621	1.513	1.59	Extremely Irregular
GF celing	Eqy	0.000846	0.000574	1.473	1.51	Extremely Irregular
B1 celing	Eqy	0.000534	0.000375	1.425	1.41	Extremely Irregular
B2 ceiling	Eqy	0.000312	0.000233	1.341	1.25	Torsion irregular

There is a torsion and extreme torsion irregularity in Y direction. Therefore, the eccentricity will be normalized to the largest value of A_y.

Eccentricity = $0.05 \times A_y$

$0.05 \times 1.59 = 0.0795$

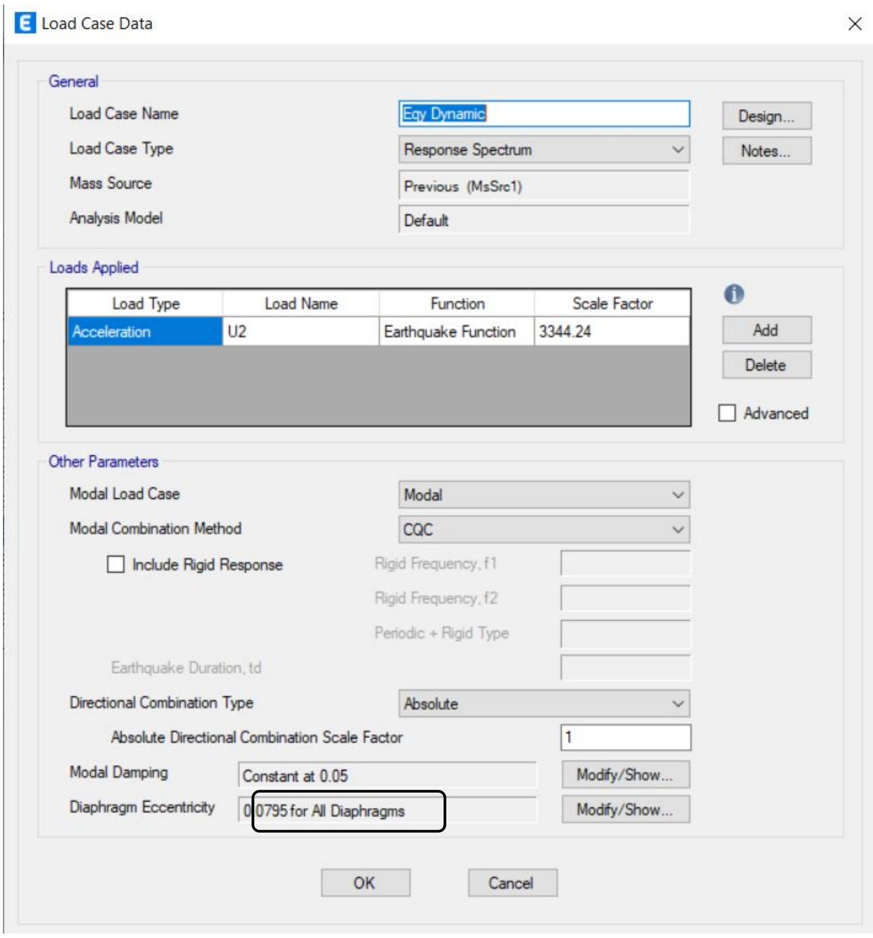


Figure 4.60 Eccentricities in Y direction for response spectrum analysis

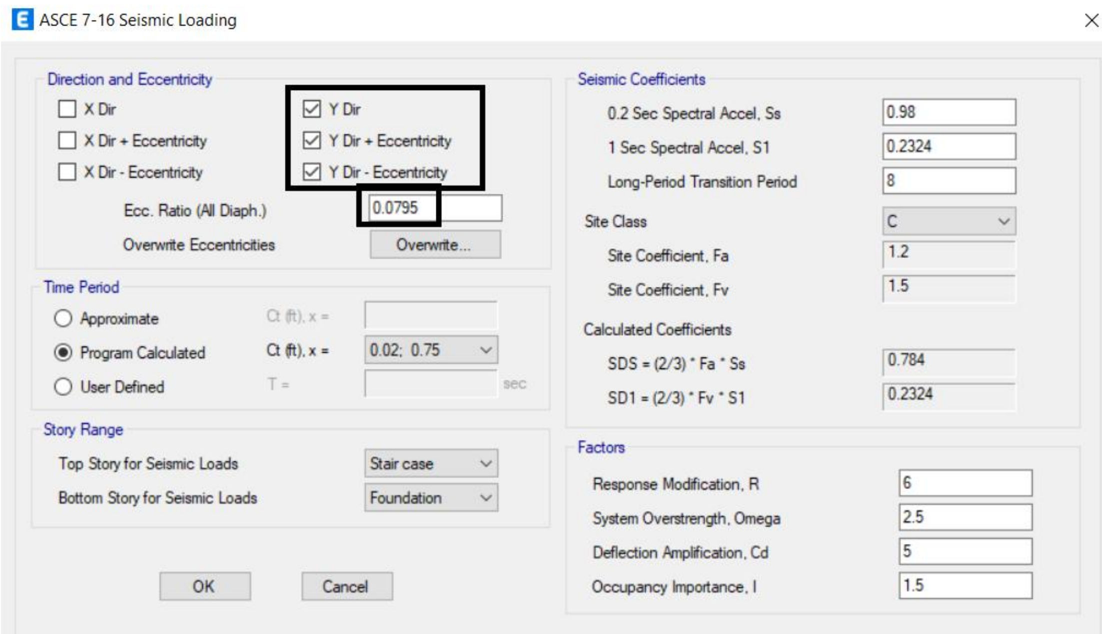


Figure 4.61 Eccentricities in Y direction for equivalent analysis

Diaphragm Discontinuity Irregularity :

Diaphragm discontinuity irregularity, defined to exist where there is a diaphragm with an abrupt discontinuity or variation in stiffness, including one that has a cut-out or open area greater than 25% of the gross enclosed diaphragm area, or a change in effective diaphragm stiffness of more than 50% from one story to the next.

In this building the open area is less than 25% of the gross diaphragm area but the chance in effective diaphragm stiffness more than 50% from story to the next.

To simplify the diaphragm stiffness calculation, it was assumed that there is an load effects on each diaphragm in the building in a global-X and Y directions, and the maximum resulting deformation values are read.

Assume load equals = 15 KN/m^2 in global-Y, and X direction.

Stiffness X = $\frac{Area \times \text{Load X}}{deformation\ Ux}$

Stiffness Y = $\frac{Area \times \text{Load Y}}{deformation\ Uy}$

Table 4.40 effective diaphragm stiffness Y effective diaphragm stiffness Y

Slab Area m2	Deflection Uy	load kN/m2	stiffness (Load/Uy)	Difference% in stiffness
509.52	0.0013	15	5879076.923	-
510.3	0.0013	15	5888076.923	0.152851264
510.3	0.0017	15	4502647.059	23.52941176
475.97	0.00195	15	3661307.692	18.68543893
442.62	0.001816	15	3656002.203	0.14490696
414.44	0.00155	15	4010709.677	8.844007752
40.38	0.00008	15	7571250	47.02711339

Table 4.41 effective diaphragm stiffness X

Slab Area m2	Deflection Uy	load kN/m2	stiffness (Load/Uy)	Difference% in stiffness
509.52	0.00042	15	18197142.86	-
510.3	0.00042	15	18225000	0.152851264
510.3	0.000707	15	10826732.67	40.59405941
475.97	0.0011	15	6490500	40.05116598
442.62	0.001423	15	4665706.254	28.11484085
414.44	0.001041	15	5971757.925	21.87047243
40.38	0.0002	15	3028500	49.28628993

Reentrant Corner Irregularity:

Reentrant corner irregularity, defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 20% of the plan dimension of the structure in the given direction.

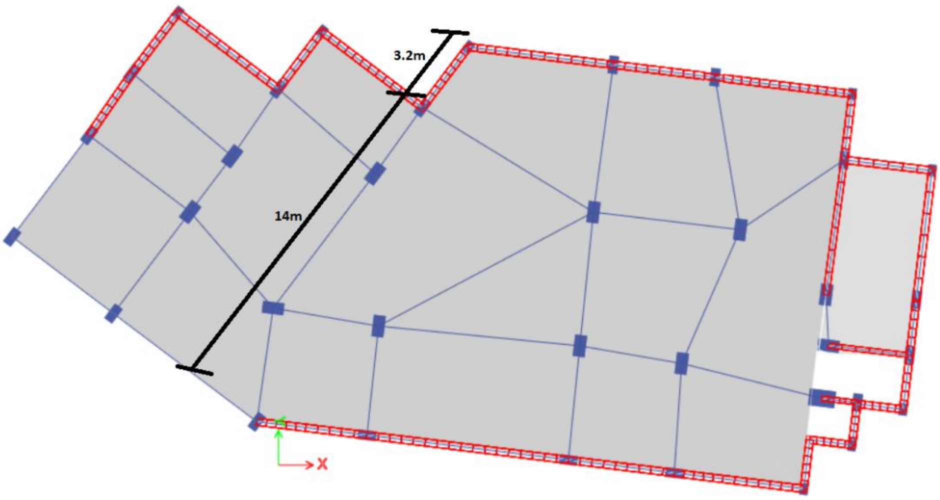


Figure 4.62 dimensions used in Reentrant Corner Irregularity

Sample Calculation:

Percent is = $\frac{3.2}{14+3.2} \times 100\% = 18.6\% < 20\%$ → No reentrant corner irregularity.

4.4.4.2 Vertical irregularities:

Table 4.42 Vertical irregularities description

Type	Description	Commentary
1a.	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.	_
1b.	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.	Only applicable for Seismic design categories E and F.
2.	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.	_
3.	In-Plane Discontinuity in Vertical Lateral Force Resisting Element Irregularity is defined to exist where there is an in-plane offset of a vertical seismic force-resisting element resulting in overturning demands on supporting structural elements.	Does not exist
4a.	Discontinuity in Lateral Strength–Weak Story Irregularity is defined to exist where the story lateral strength is less than that in the story above. The story lateral strength is the total lateral strength of all seismic force-resisting system elements resisting the story shear for the direction under consideration.	Only applicable for Seismic design categories E and F.
4b.	Discontinuity in Lateral Strength–Extreme Weak Story Irregularity is defined to exist where the story lateral strength is less than 65% of that in the story above. The story lateral strength is the total lateral strength of all seismic force-resisting system elements resisting the story shear for the direction under consideration.	Does not exist

4.4.5 Effect of P-delta:

P-delta affects 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 0.10:

$$\theta = \frac{P_x / h_{sx}}{V_x / \Delta_{xe}}$$

Where:

θ: Stability coefficient.

P_x: Total un-factored vertical design load at and above level x.

Δ_{xe}: Design story drift.

V_x: Seismic shear force.

h_x: story height below level x.

Stiffness: V/Δ.

Px represented this load combination: S Envelope.

In X direction:

Table 4.43 Effect of p-delta in X direction

Story	P(kN)	Stiffness X(kN/m)	H_{xs}(m)	Theta=P/(h×S)	Check
Story7	1066.3945	501810.793	3.03	0.000701351	ok
Story6	7359.9541	2803622.973	4	0.00065629	ok
Story5	18868.705	4566491.655	4	0.001032998	ok
Story4	31286.799	7541705.61	4	0.001037126	ok
Story3	45080.613	11211710.27	4	0.001005213	ok
Story2	59074.088	22320513.28	4	0.000661657	ok
Story1	72349.477	29177063.97	4	0.000619917	ok

The values of θ in X direction are less than 0.1. Therefore, neglect the effect of P-delta.

In Y direction:

Table 4.44 Effect of p-delta in Y direction

Story	P(kN)	Stiffness Y(kN/m)	H_{xs}(m)	Theta=P/(h×S)	Check
Story7	1066.3945	155618.697	3.03	0.002261588	ok
Story6	7359.9541	1820415.778	4	0.001010752	ok
Story5	18868.705	4077125.994	4	0.001156986	ok
Story4	31286.799	7095750.419	4	0.001102308	ok
Story3	45080.613	11235460.41	4	0.001003088	ok
Story2	59074.088	29256833.13	4	0.000504789	ok
Story1	72349.477	49406965.84	4	0.000366089	ok

The values of θ in Y direction are less than 0.1. Therefore, neglect the effect of P-delta.

4.4.6 Verification of internal forces:

4.4.6.1 Slabs:

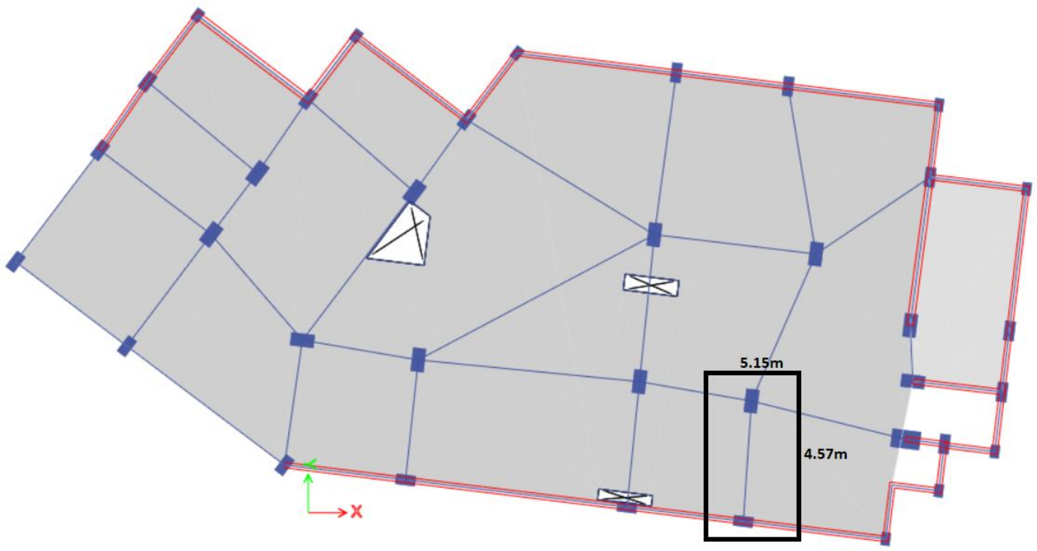


Figure 4.63 Sample Slab strip in Y-Dir used for Slab Internal Force Check

Table 45 Calculations for Y-Dir Slab Strip Internal Forces

γ c kN/m^3	25
Slab thickness m	0.25
DL own weight kN/m2	6.25
SID kN/m^2	7.5
LL kN/m^2	6
Ult W Load kN	26.1
L2 m	5.15
L1 m	4.57
qu kN	134.4
Mo kN.m	350.9
Mb kN.m	7.832
Total M kN.m	358.7
M ETABS kN.m	170
Error%	52.61

Where:

- Wu = 1.2 D+1.2 SID +1.6 L
- $q_u = W_u \, L_2$
- $M_o = \dfrac{q_u \, L_1^{\,2}}{8}$
- $M_b = \dfrac{W_u \, L_1^{\,2}}{8}$

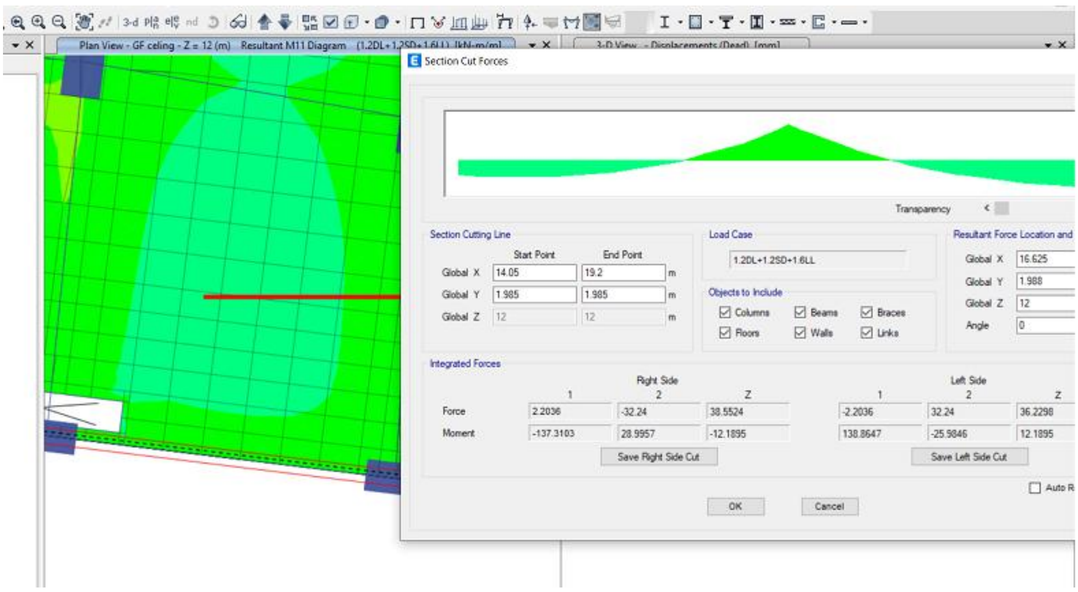


Figure 4.64 Slab Section Cut Results for Y-Dir Strip at the middle

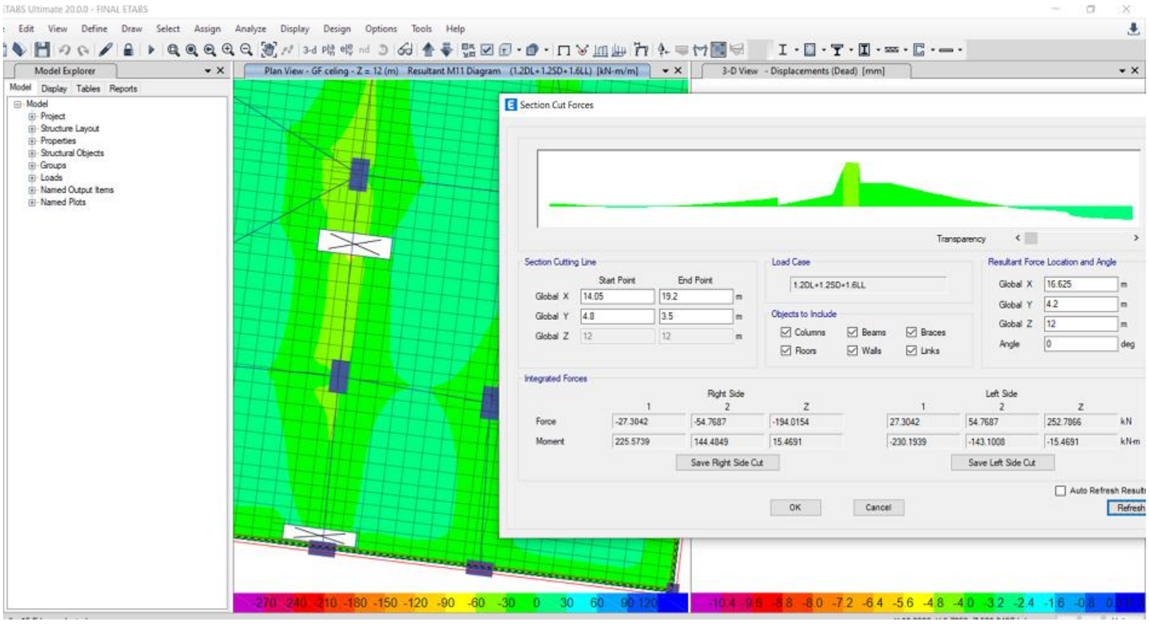


Figure 4.65 Slab Section Cut Results for Y-Dir Strip at the corner

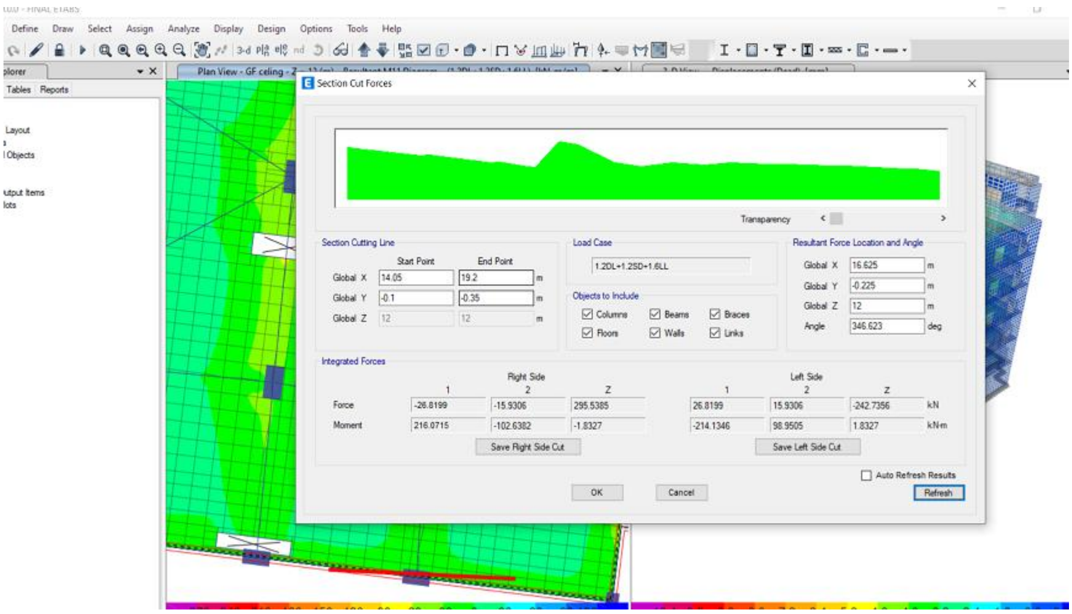


Figure 4.66 Slab Section Cut Results for Y-Dir Strip at the Corner

4.4.6.2 Beams:

In Y-Dir

Hand calculation:

Positive factored moment = $0.65 \times M_o$

$= 0.65 \times 351$

$= 228.15$ kN.m

Negative factored moment = $0.35 \times M_o$

$= 0.35 \times 351$

$= 122.85$ kN.m

Table 4.46 Calculations for Beam in Y-Dir for Internal Forces

I slab m^4	0.006706
I beam m^4	0.0072
Alpha α	1.073709
L2 m	5.15
L1 m	4.57
L2/L1	1.126915
α*(L2/L1)	1.209978

Calculation of the coefficients is done by using the following Tables.

Table 4.47 ACI 318-14 Table 8.10.5.5-Portion of positive M, in column strip

$\alpha_f L_2/L_1$	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

Table 4.48 ACI 318-14 Table 8.10.5.7.1-Portion of column strip Mu in beams

$\alpha_f L_2/L_1$	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

Table 4.49 Distribution Coefficient Table

$\alpha_f L_2/L_1$	**Distribution Coefficient**
0.0	0.0
≥ 1.0	0.85

Factor For Positive Moment = 0.711 → (M+) × Factor = 0.711 × 228.15 = 162.215 kN.m .
Factor For Negative Moment = 0.711 → (M-) × Factor = 0.711 × 122.85 = 87.35 kN.m.

Factor For Beam = 0.85 → (M+) × Factor = 0.85 × 162.215 = 137.9 kN.m.

Factor For Beam = 0.85 → (M-) × Factor = 0.85 × 87.35 = 74.25 kN.m.

ETABS Value → Mu+ = 66.7 kN.m

→ Mu- = 88.6 kN.m

% Error Mu+ → $\frac{137.9-66.8}{137.9} \times 100\% = 51.6\ \% > 10\% \rightarrow Not\ ok.$

% Error Mu- → $\frac{74.25-88.6}{74.25} \times 100\% =- 19.32\ \% > 10\% \rightarrow Not\ ok.$

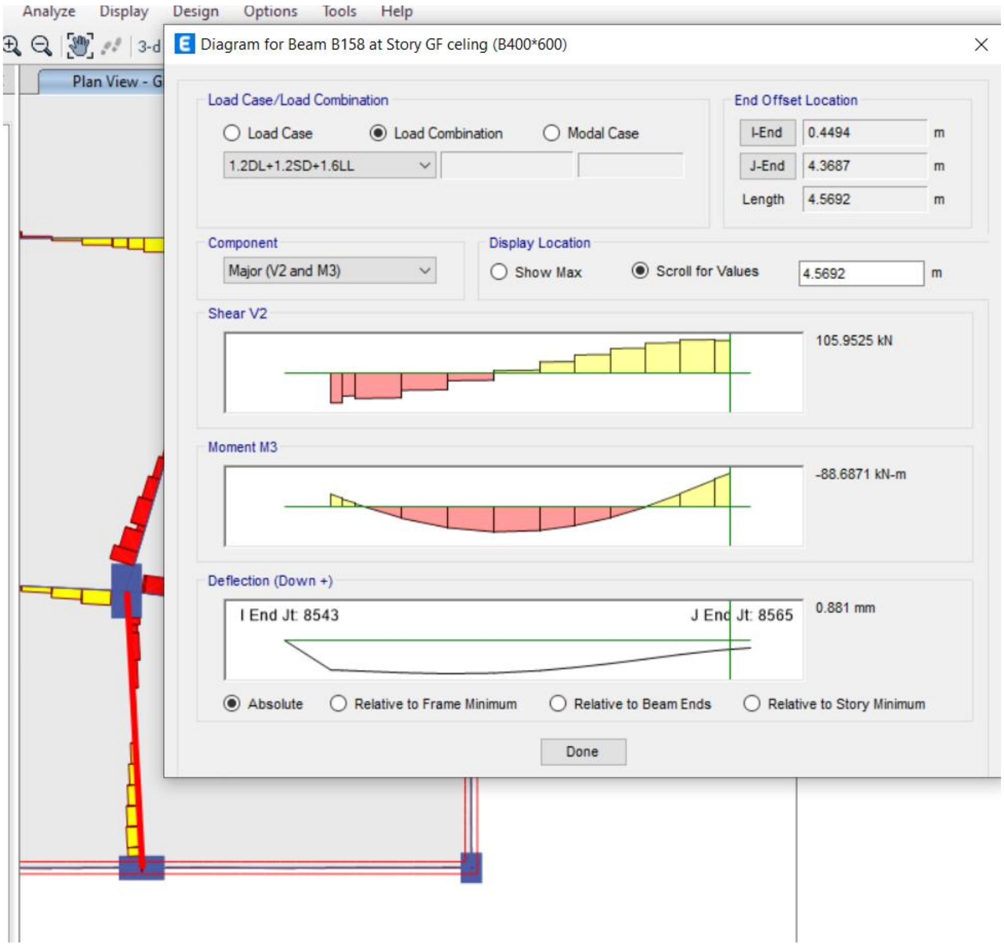


Figure 4.67 EATBS Internal Forces values in beam for Beam Internal forces Check in Y-Dir

In X-Dir

Hand calculation:

Positive factored moment = $0.65 \times M_o$

= 0.65×319.71

= 207.8 kN.m

Negative factored moment = $0.35 \times M_o$

= 0.35×319.71

= 111.9 kN.m

Table 4.50 Calculations for Beam in X-Dir for Internal Forces

I slab m^4	0.006751
I beam m^4	0.0072
Alpha α	1.066461
L2 m	5.185
L1 m	4.3
L2/L1	1.205814
α*(L2/L1)	1.285953

Factor For Positive Moment = 0.711 → (M+) × Factor = 0.711 × 207.8 = 147.75 kN.m .
Factor For Negative Moment = 0.711 → (M-) × Factor = 0.711 × 111.9 = 79.56 kN.m.

Factor For Beam = 0.85 → (M+) × Factor = 0.85 × 147.75 = 125.59 kN.m.

Factor For Beam = 0.85 → (M-) × Factor = 0.85 × 79.56 = 67.63 kN.m.

ETABS Value → Mu+ = 44 kN.m

→ Mu- = 98 kN.m

% Error Mu+ → $\frac{125.59-44}{125.59} \times 100\% = 64.96\ \%$

% Error Mu- → $\frac{67.63-98}{67.63} \times 100\% = -44.91\ \%$

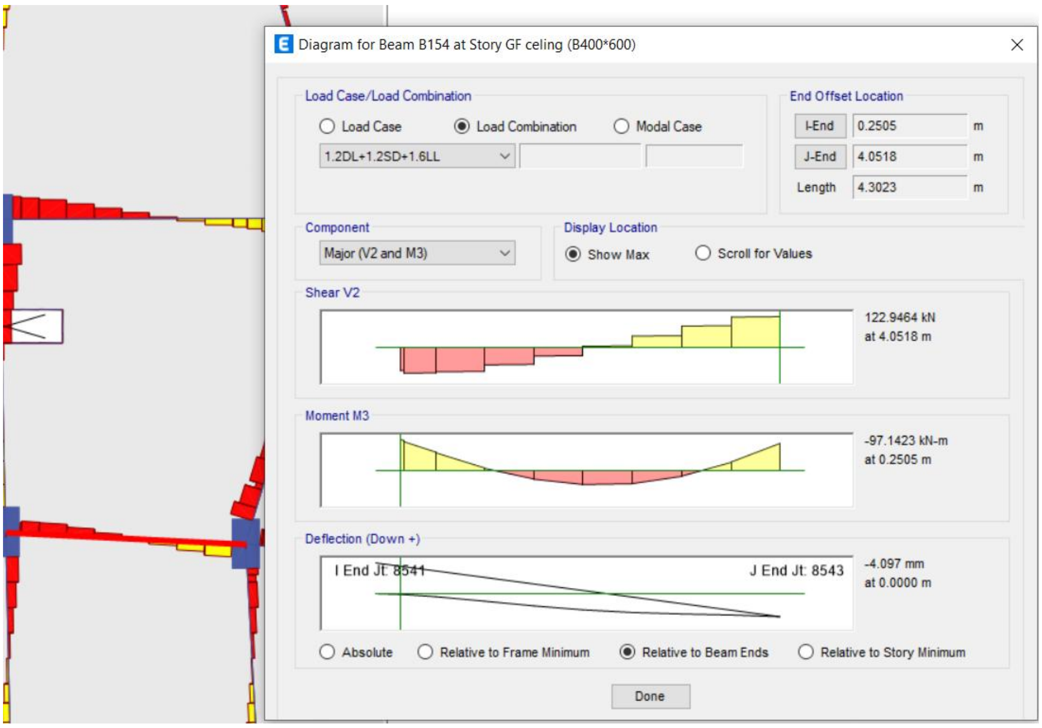


Figure 4.68 EATBS Internal Forces values in beam for Beam Internal forces Check in X-Dir

4.4.6.3 Columns:

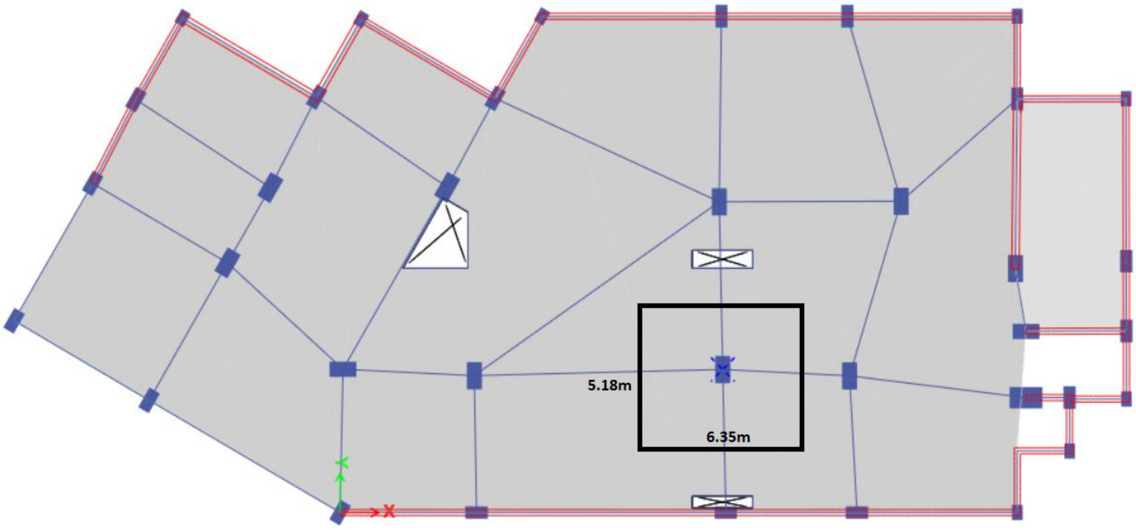


Figure 4.69 Colum used for Internal Forces Check

Table 4.51 Calculations for Column for Internal Forces

C Dimensions	900*500
C own weight	45
Tributary area	32.893
beam weight	69.18
slab weight	205.5813
dead load	319.7613
LIVE	197.358
SID	246.6975
PU	995.5233
ETABS	930
ERROR	6.581795

- ➔ Column self-weight = Area × Column length × Unit weight of reinforced concrete = 25 × 0.9 × 0.5 × 4 = 45 kN.
- ➔ Beam weight = Area × Beam length × Unit weight of reinforced concrete = (6.35+5.18) × 0.6 × 0.4 × 25 = 69.18 kN.
- ➔ Slab weight = Tributary area × Thickness of slab × Unit weight of reinforced concrete= 32.893 × 0.25 × 25 = 205.581 kN
- ➔ Total dead load = (45 + 69.18 + 205.581) = 319.761 kN
- ➔ Live load = Tributary area × Live load = 32.893 × 6kN/m^2 = 197.358 kN

➔ Super dead load = Tributary area × SID = 32.893 × 7.5kN/m^2 = 246.698 kN

➔ Pu manually = 1.2 DL + 1.2 SID + 1.6 LL = 1.2 (319.761) + 1.6 (197.385) = 995.5233 kN.

Results from ETABS are subject to ultimate load combination: 1.2 D+1.2SID + 1.6L.

= 3016.5568 – 2086.8996 = 930 KN

➔ Error% $= \frac{995.5233-930}{995.5233} \times 100\% = 6.58\ \% < 10\% \rightarrow ok.$

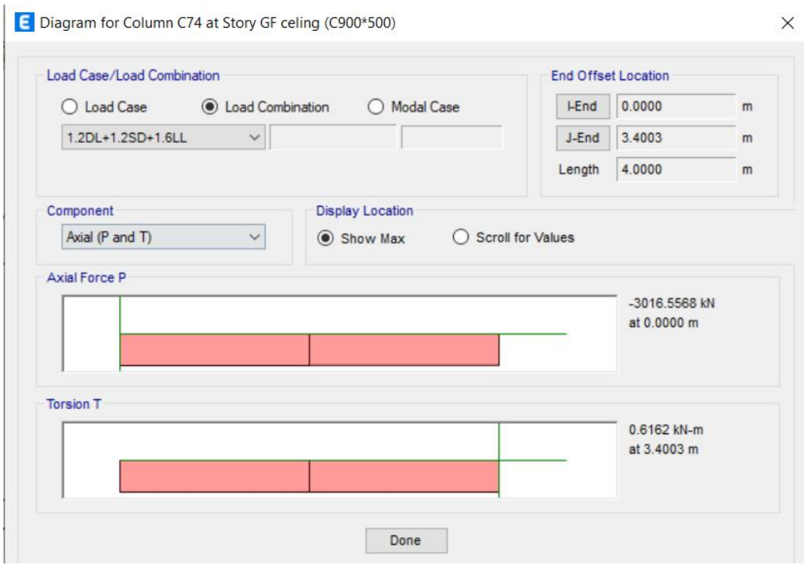


Figure 4.70 EATBS Internal Forces values in Column for Column Internal forces Check

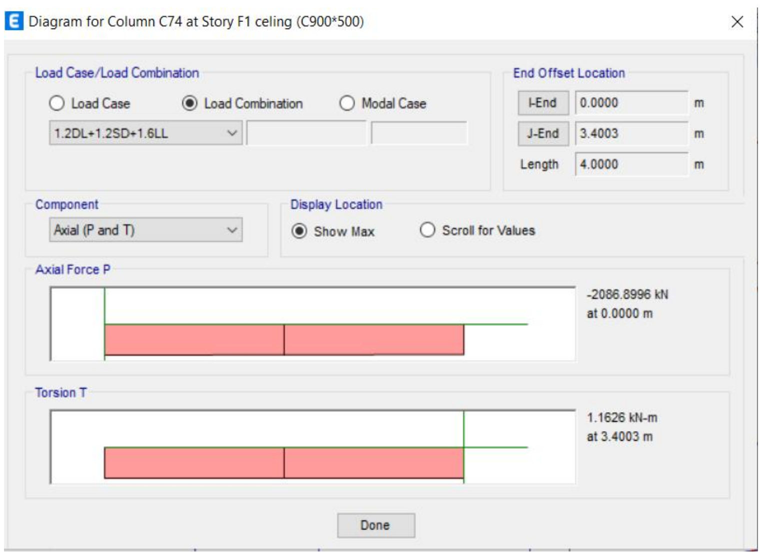


Figure 4.71 EATBS Internal Forces values in Column for Column Internal forces Check

4.5 Deflection computations:

4.5.1 Immediate and long-term deflections for slab systems

✓ long term deflections for slab systems

This check is done in order to ensure that the deflection of structural elements is not affecting the partitions, since excessive deflection will make cracks in partitions.
Allowable deflection according to ACI 318-19:

TABLE 9.5(b) — MAXIMUM PERMISSIBLE COMPUTED DEFLECTIONS

Type of member	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 live load ***L***	$\ell/180^{*}$
Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load ***L***	$\ell/360$
Roof or floor construction 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 (sum of the long-term deflection due to all sustained loads and the immediate deflection due to any additional live load)†	$\ell/480^{\ddagger}$
Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections		$\ell/240^{\S}$

*Limit not intended to safeguard against ponding. Ponding should be checked by suitable calculations of deflection, including added deflections due to ponded water, and considering long-term effects of all sustained loads, camber, construction tolerances, and reliability of provisions for drainage.
†Long-term deflection shall be determined in accordance with 9.5.2.5 or 9.5.4.3, but may be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be determined on basis of accepted engineering data relating to time-deflection characteristics of members similar to those being considered.
‡Limit may be exceeded if adequate measures are taken to prevent damage to supported or attached elements.
§Limit shall not be greater than tolerance provided for nonstructural elements. Limit may be exceeded if camber is provided so that total deflection minus camber does not exceed limit.

Figure 4.72 Max Permissible Computed Deflections

Long term deflection combination: 2D+2SD+1.5 L

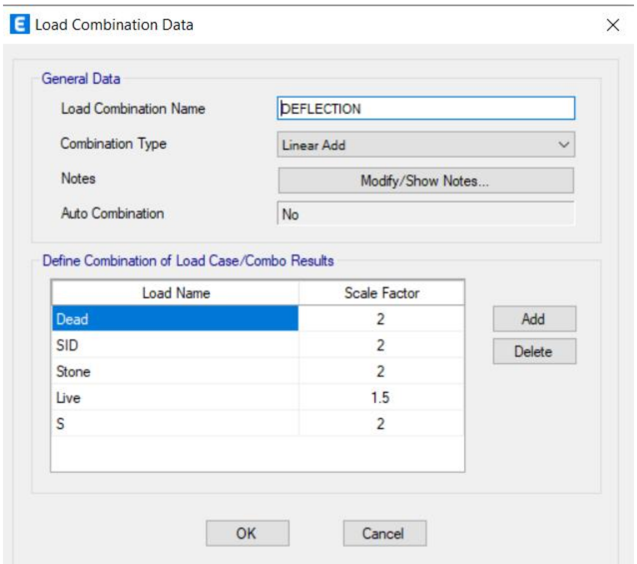


Figure 4.73 Long-Term Deflection Combination Definition

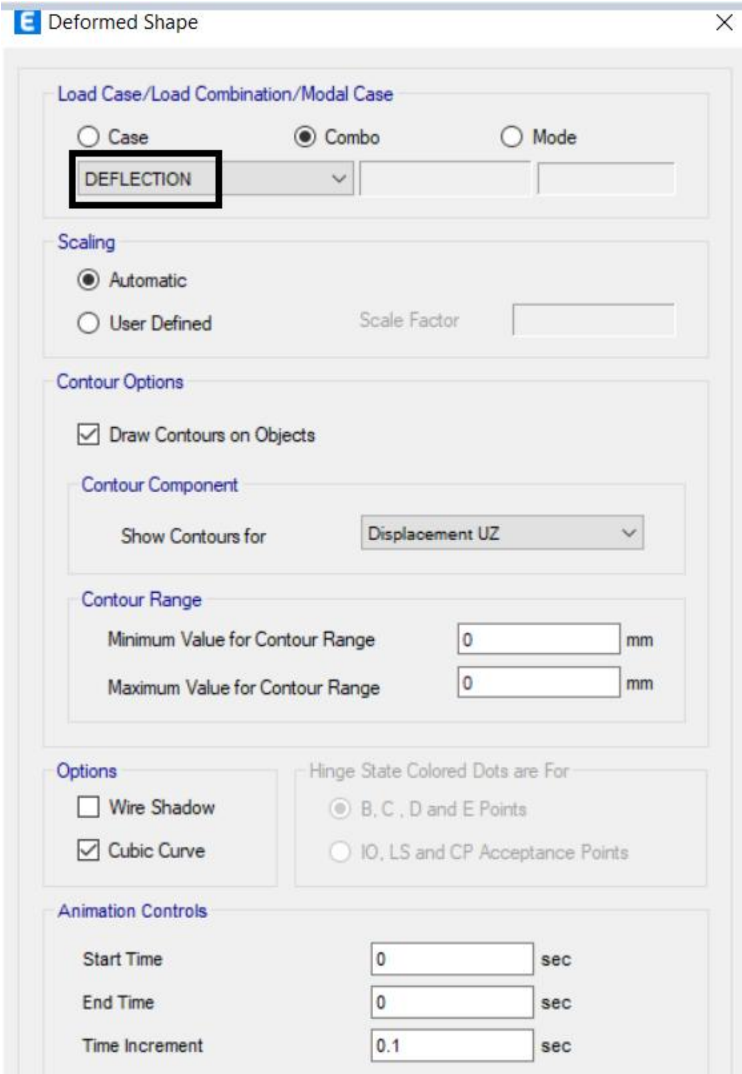


Figure 4.74 Display Deformed Shape from Long-Term Deflection Combination

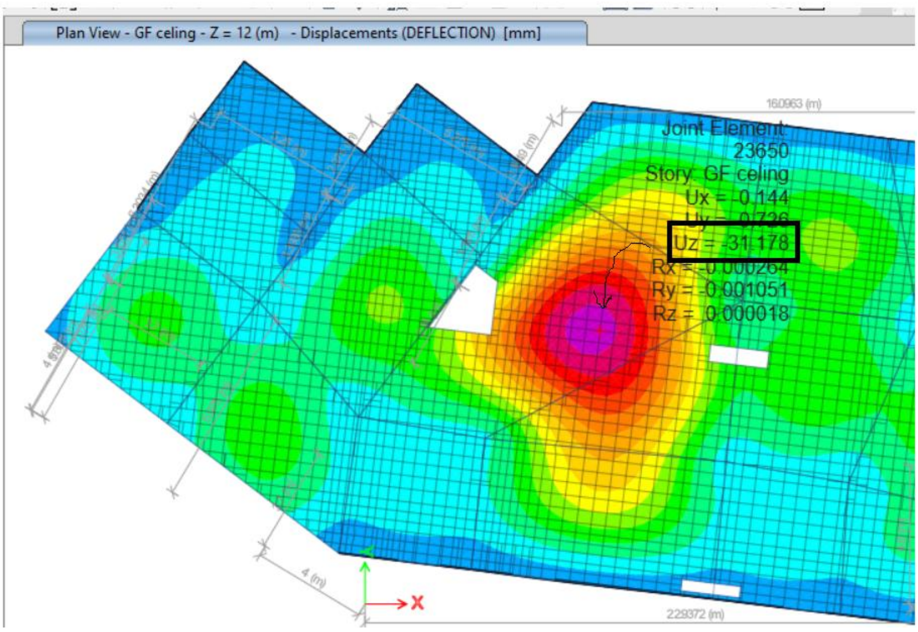


Figure 4.75 Deflection on slab.

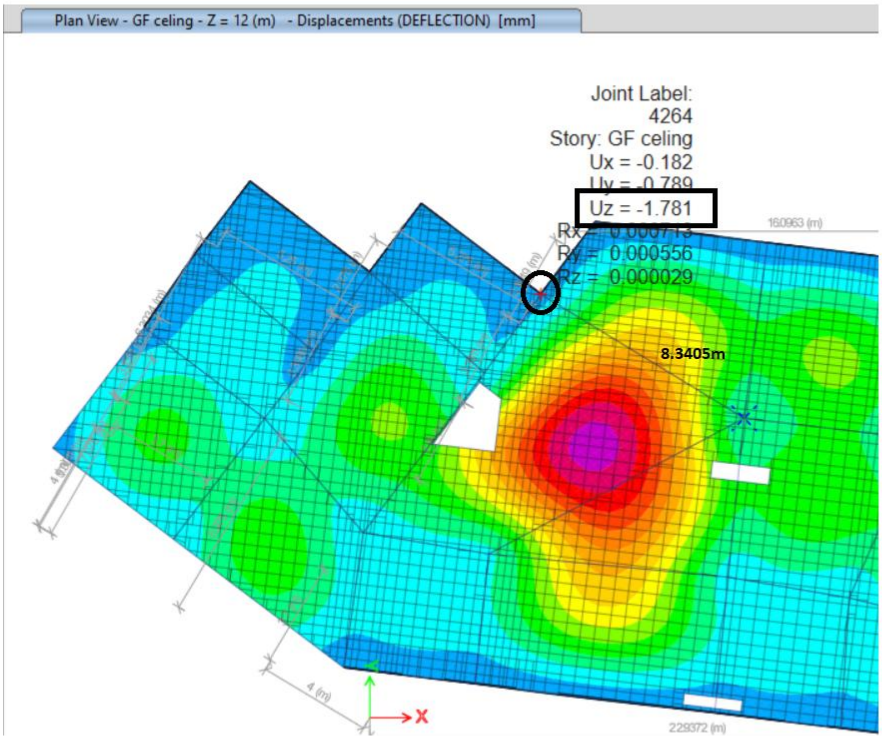


Figure 4.76 Deflection on Beam Corner near to the Slab.

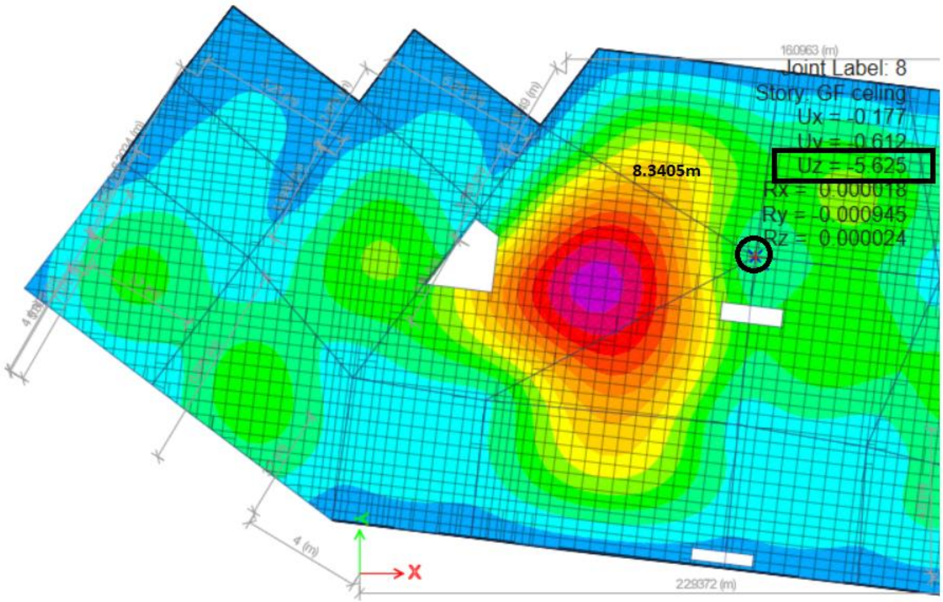


Figure 4.77 Deflection on Beam Corner near to the Slab.

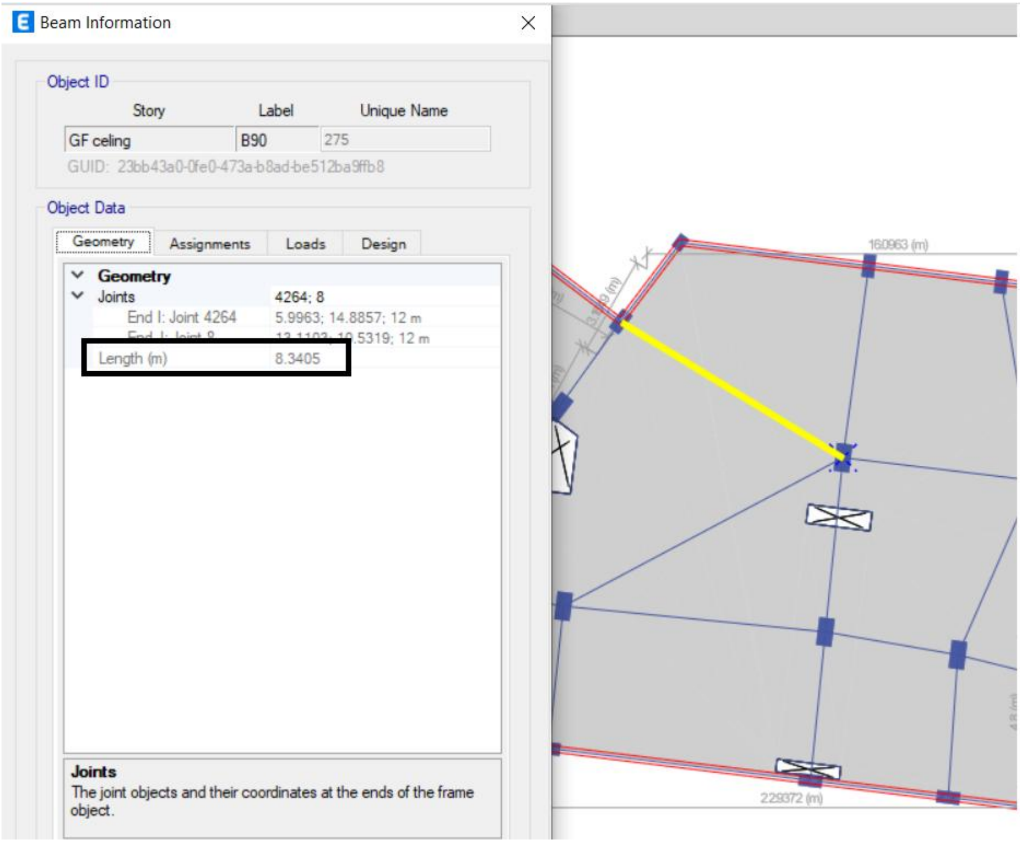


Figure 4.78 Adjacent beam Length

Max ETABS Slab Deflection= 31.178 mm

- ➔ $\delta\ ETABS = -\ 31.178 + \frac{-1.781 + -5.625}{2} = -\ 27.475\ mm$
- ➔ $\delta limit = \frac{L}{240} = \frac{8.3405 \times 10^3}{240} = 37.77\ mm$
- ➔ $\delta\ ETABS < \ \delta limit \rightarrow Ok.$

4.6 Verification of structural design:

4.6.1 Beam design verification :

Select the following beam (shown in **Figure 4.80**) for check, which is located in the second floor , This beam has a length of 7.5 m and dimensions for the cross section are 400 mm in width and 600 mm in depth (d=540 mm) .

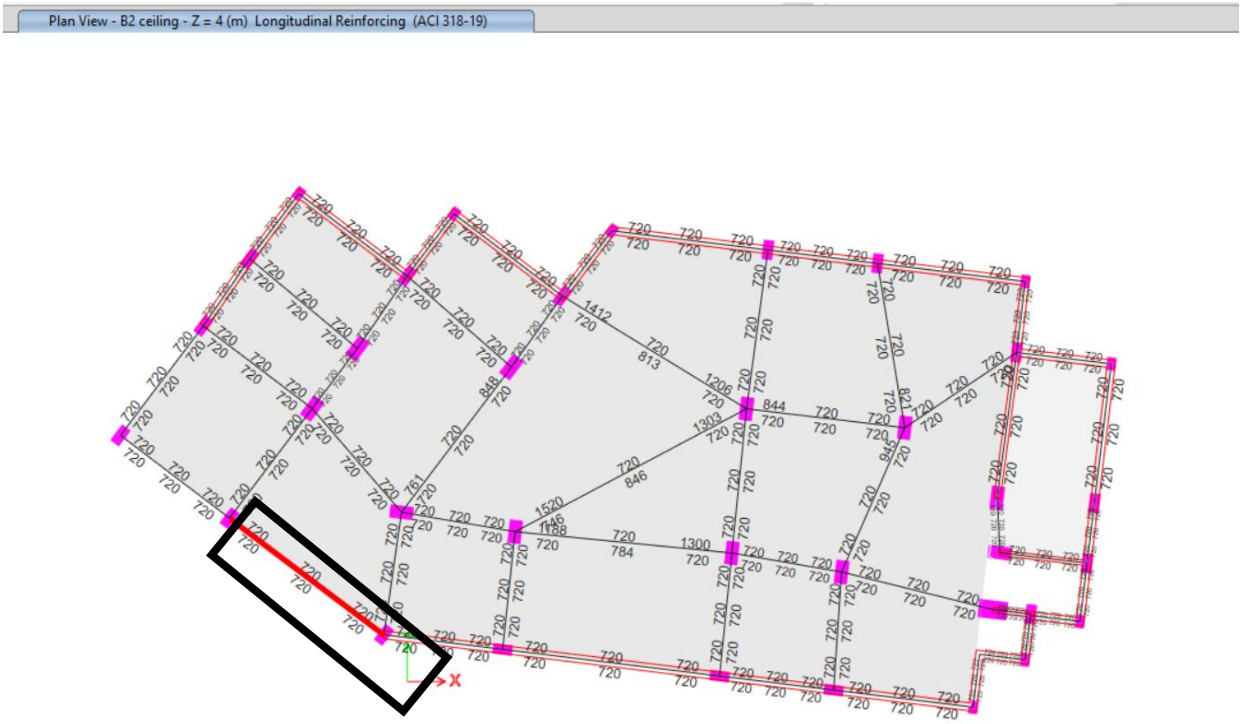


Figure 4.80: Beam selected for design verification.

Beam Element Details

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	LLRF	Type
B2 ceiling	B108	376	B400*600	U6	7276.8	7476.8	1	Sway Special

Figure 4.81: Beam Element Details

Section Properties

b (mm)	h (mm)	b_f (mm)	d_s (mm)	d_{ct} (mm)	d_{cb} (mm)
400	600	400	0	60	60

Figure 4.82: Beam section properties

4.6.1.1 Flexure Design Verification:

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)	-48.9378	0	246	0	720	720
Bottom (-2 Axis)	24.4689	0	0	122	720	720

Figure 4.83: Design moment and Flexural Reinforcement for Moment, M_{u3}

The area of steel for this beam due to the ultimate negative moment = 720 mm^2.

Mu = 48.9378 KN.m

Cover =60 mm

h= 600mm

d=600-60 =540 mm

b= 400mm

L = 7.476 m

$$\rho = \frac{0.85\,fc}{fy}\left(1-\sqrt{1-\frac{2.61Mu}{b\,d^2\,fc}}\right)$$
$$= 0.001136$$

$$\rho_{min} = 0.00333$$
$$\rho_{max} = 0.025$$

As = $\rho bd = 0.001136 \times 400 \times 540 = 432\ mm^2$

As min $= \rho \min \times bh = 0.003 \times 400 \times 600 = 720\ mm^2$
Use As_{min} > As → use As_{min}

Difference percentage between the longitudinal reinforcement from the manual and from ETABS:

Difference percentage= $\frac{720-720}{720} \times 100\%$ =0% < 5% →OK.

4.6.1.2 Shear Design Verification:

Ultimate shear value is shown in Figure **4.84**

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
35.1225	68.039	0	54.762	0

Figure 4.84: Shear Force and Reinforcement for Shear, V_{u2}

Vu = 35.1225 kN

$$V_c = (\frac{1}{6}\sqrt{f'_c} + \frac{N_u}{6A_g})b_w d$$

$$= (\frac{1}{6}\sqrt{28} + 0) \times 400 \times \frac{540}{1000}$$

$$= 190.5\ \text{KN}$$

$$\phi V_c = 0.75 \times 190.5$$

$$= 142.87\ \text{KN}$$

$$V_s = V_u / \phi - V_c$$

$$= 35.12/0.75 - 190.5$$

$$= 143.664\ \text{KN}$$

Vp ETABS = 54.762 kN

As Top right = 720 mm^2

As Bot left = 720 mm^2

As Top left = 720 mm^2

As Bot right = 720 mm^2

$$V_{p1} = \frac{\text{Mpr from As Top right} + \text{Mpr As Bot left}}{Beam\ L\ m}$$

$$V_{p2} = \frac{\text{Mpr from As Top left} + \text{Mpr As Bot right}}{Beam\ L\ m}$$

V_p manually final = Max. (V_p1 & V_p2)

$$M_{pr} = \phi A_s \times 1.25 \times f_y (d - \frac{a}{2})$$

$$a = \frac{A_s \times 1.25 \times f_y}{0.85 f_c b}$$

M_{pr} from A_s Top right $= 720(1.25)28 \times \frac{(540 - \frac{720(1.25)420}{1.7028 \times 400})}{10^6} = 196.62\ kN.$

M_{pr} from A_s Top left$=720(1.25)28 \times \frac{(540 - \frac{720(1.25)420}{1.7028 \times 400})}{10^6} = 196.62\ kN.$

M_{pr} from A_s Bot right$=720(1.25)28 \times \frac{(540 - \frac{720(1.25)420}{1.7028 \times 400})}{10^6} = 196.62\ kN.$

M_{pr} from A_s Bot left$=720(1.25)28 \times \frac{(540 - \frac{720(1.25)420}{1.7028 \times 400})}{10^6} = 196.62\ kN.$

$V_{p1} = \frac{196.62+196.62}{7.476} = 52.60\ kN$

$V_{p2} = \frac{196.62+196.62}{7.476} = 52.60\ kN$

$V_{p\ final}$ = 52.60 kN

V_p ETABS = 54.762 kN

% Difference $= \frac{\text{Vp Manually final} - \text{Vp ETABS}}{\text{Vp Manually final}} \times 100\% = 3.96\% < 5\% \rightarrow ok.$

4.6.1.3 Torsion Design Verification:

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
5.5643	9.5914	38.3655	1351.5	1644.4	0	0

Figure 4.85: Torsion Force and Torsion Reinforcement for Torsion, T_u

Tu= 5.5643 kN.m

$$\phi T_{th} = \phi 0.083 \lambda \sqrt{f'_c} \frac{A_{cp}}{P_{cp}}$$

Where:

A_{cp}: area enclosed by outermost perimeter of concrete cross section.

P_{cp} = outside perimeter of concrete cross section.

A_{cp}= 400*600 = 240000mm^2

P_{cp}= 2(400+600) = 2000 mm

Φ= 0.75

$$\phi T_{th} = \phi 0.083\lambda\sqrt{f'_c}\frac{A_{cp}}{P_{cp}}$$

$$= 0.75\times 0.083\times 1\times\sqrt{28}\times\frac{600\times 400}{2(600+400)}$$

$$= 9.5247 \text{ KN.m}$$

Ø T_{th} by ETABS = 9.5914 KN.m

Difference percentage between the torsion beam member capacity from manualluy and ETABS:

Difference percentage= $\frac{9.5914-9.5247}{9.5914}\times 100\% = 0.6954\%$ <5% →OK

Check Section dimension adequacy:

Section dimensions must be adequate to resist shear and torsion force, and must be checked using the following equation:

$\sqrt{\left(\frac{Vu}{bw\times d}\right)^2+\left(\frac{Tu\times Ph}{1.7Aoh}\right)^2} \le \emptyset\frac{5}{6}\sqrt{f'c}$ → section member dimensions are adequate to resist shear and torsion forces.

Where:

V_u = ultimate shear force, N.

T_u = ultimate torsion, N.mm.

P_h = peremiter of centerline of outermost closed transverse torsional reinforcement, mm.

A_{oh} = area enclosed by centerline of outermost closed transverse torsional reinforcement, mm^2.

b_w = width of web.

d = effective depth.

V_u= 35122.5 N

T_u= 5564300

b_w= 400mm

Cover= 60mm

d= 540mm

P_h= 2([400-2(60)] + [600-2(60)]) = 1520mm

A_{oh}= [400-2(60)] × [600-2(60)] = 134400 mm^2

$$\sqrt{\left(\frac{35122.5}{400 \times 540}\right)^2 + \left(\frac{5564300 \times 1520}{1.7 \times 134400}\right)^2} = 0.32$$

$$\emptyset \frac{5}{6}\sqrt{f'c} = 0.75 \times \frac{5}{6} \times \sqrt{28} = 3.31$$

Left side= 0.32 which is < right side= 3.31 → that points that section member dimensions are adequate to resist shear and torsion forces (no need to enlarge section).

4.6.2 Column design verification :

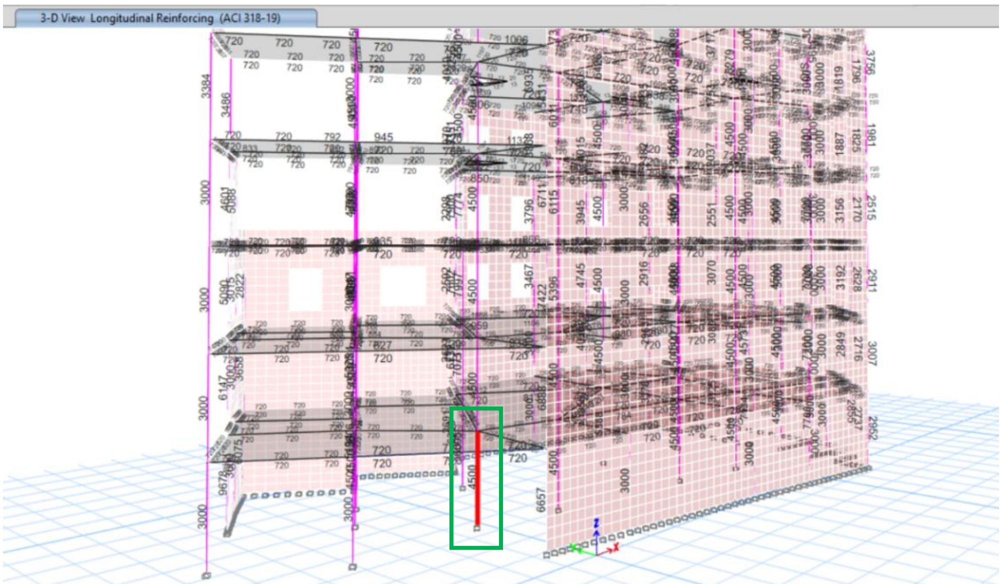


Figure 4.86: Column C45 selected for design verification

Use the following column for check, which is located on the (B2 ceiling) floor, as shown in both **Figures 4.86** and **4.87**.

Column Element Details

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	LLRF	Type
B2 ceiling	C45	18	C500*900	U5-Dyn	3400.3	4000	0.442	Sway Special

Figure 4.87: Column C45 Element Details

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
900	500	60	27.3

Figure 4.88: Column C45 section properties

This column has a length of 4 m and the dimensions for the cross section are 900 mm and 500 mm.

P_u = 3738.799 kN

$M_{u,2}$ = 19.4624 kN.m

$M_{u,3}$ = 221.9527 kN.m

The values of P_u, $M_{u,2}$ and Mu,3 is shown in **Figure 4.89**

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 % %
3738.799	19.4624	-221.9527	157.9269	113.0613	4500	1

Figure 4.89: Axial Force and Biaxial Moment Design for P_u, M_{u2}, M_{u3}

Figure **4.90** shows interaction diagram between ØPn and ØMn,3, to check the column capacity for them.

Figure **4.91** shows interaction diagram between ØPn and ØMn,2, to check the column capacity for them.

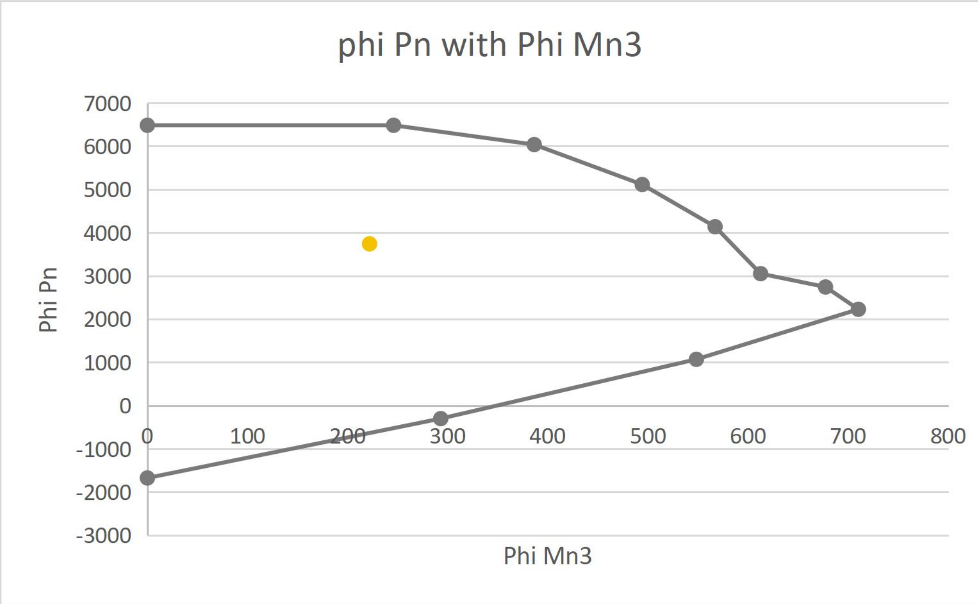


Figure 4.90: Interaction Diagram (ØPn and ØMn,3)

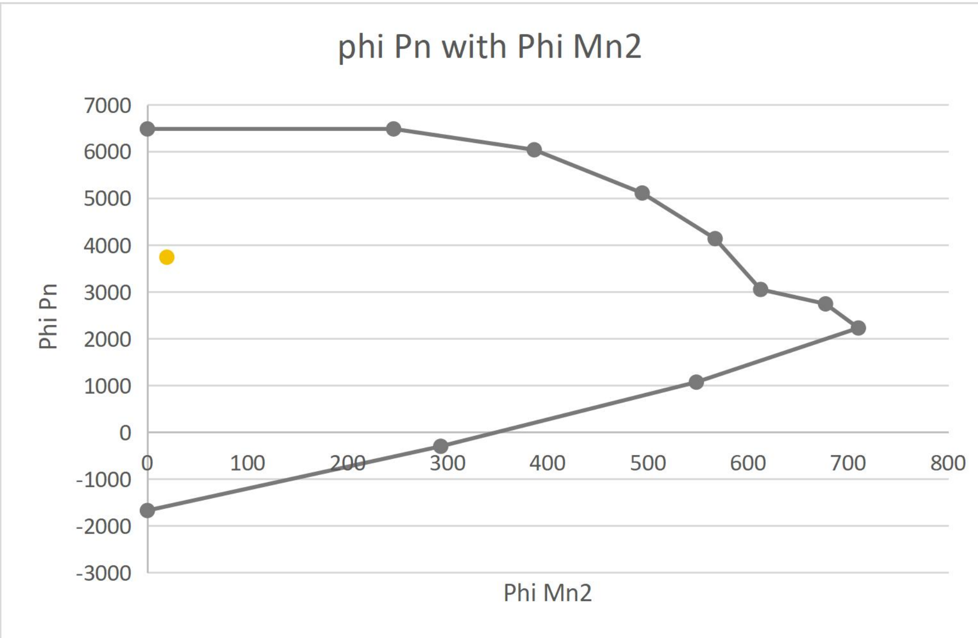


Figure 4.91: Interaction Diagram (ØPn and ØMn,2)

Bressler's Reciprocal load method shall be applied to check the column section for the biaxial moments.

$$\frac{1}{\phi Pn} = \frac{1}{\phi Pn_3} + \frac{1}{\phi Pn_2} - \frac{1}{\phi Pn_o}$$

Where:

ØPn: Axial load capacity for Mu,3 and Mu,2.

ØPn,3: Axial load capacity for Mu,3.

ØPn,2: Axial load capacity for Mu,2.

ØPn, o: Axial load capacity without moment.

ØPn,3 =6481.53 kN (from Figure 3.80)

ØPn,2 = 6481.53 kN (from Figure 3.81)

b= 500mm

h= 900mm

Ag= 500*900 = 450000mm2

As= 4500mm2

ØPn, o = Ø ƛ (0.85 f'c (Ag-As) + As fy)

ØPn, o = 0.65*0.8*(0.85*28* (450000 – 4500) + 4500*420)

ØPn, o = 6791.076 kN

Then, ØPn > Pu → 6199>3738.799 → Ok.

4.6.3 Wall design verification:

4.6.3.1 Spandrel design

Based on ETABS wall design manual, wall spandrels are designed as beam coupling for major direction flexure and shear only. Effects caused by any axial forces, minor direction bending, torsion or minor direction shear that may exist in the spandrels must be investigated by the user independent of the program. Spandrel flexural reinforcing is designed for each of design load combinations. The required are of reinforcing for flexure is calculated and reported only at the ends of the spandrel beam.

The program reports the ratio of top and bottom steel required in the web area. When compression steel is required, those ratios must be large because there is no limit on them. However, the program reports an overstress when the ratio exceeds 4%.

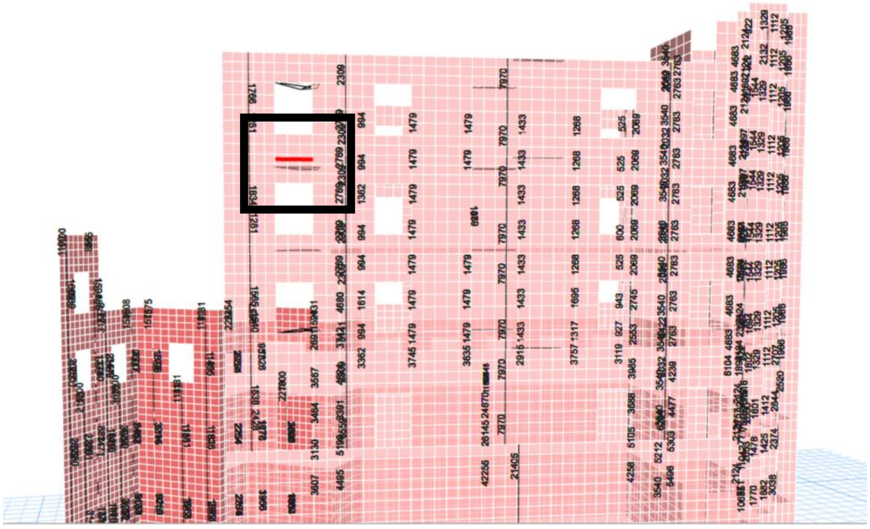


Figure 4.92: Selected spandrel shear wall for design verification.

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
F3	S23	1135.1	1555.9	1950	250	1

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
24870.06	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 mm^2	Reinf Percentage	Reinf Combo	Moment, M_u kN-m
Left	928	0.19	U7.2-Dyn	-348.1309
Right	1083	0.22	U5.2-Dyn	-411.8825

Spandrel Flexural Design—Bottom Reinforcement

Station Location	Reinf Area mm^2	Reinf Percentage	Reinf Combo	Moment, M_u kN-m
Left	993	0.2	U5.2-Dyn	391.861
Right	917	0.19	U7.2-Dyn	311.6161

Spandrel Shear Design

Station Location	A_{vert} mm^2/m	A_{horiz} mm^2/m	ShearCombo	V_u kN	ϕV_c kN	ϕV_s kN	ϕV_n kN
Left	643.34	625	U5.2-Dyn	506.6804	222.1566	284.5238	506.6804
Right	913.14	625	U5.2-Dyn	623.7272	219.8831	403.8441	623.7272

Spandrel Shear Design—Diagonal Reinforcement

Station Location	A_{diag} mm^2	Shear Combo	V_u kN	V_{uLimit} kN	L/H Ratio	Seismic Design	Diag Reinf Mandatory
Left	984	U5.2-Dyn	506.6804	771.1088	0.769	Yes	No
Right	1212	U5.2-Dyn	623.7272	771.1088	0.769	Yes	No

Figure 4.93: Data sheet for wall S39 in ETABS.

Beam section width, b= 250mm.

Beam section thickness, h= 1950mm

Beam effective depth, d= 0.9h = 1755mm.

Moment, Mu for bottom left station location= 391.861 kN.m

$$\rho = \frac{0.85f'_c}{f_y}\left(1-\sqrt{1-\frac{2.61M_u}{b\,d^2f'_c}}\right)$$
$$= \frac{0.85\times 28}{420}\left(1-\sqrt{1-\frac{2.61\times 391.861\times 10^6}{250\times 1950^2\times 28}}\right)$$
$$= 0.00136$$

A_s= 0.00136 × 250 × 1755 = 596.87 mm^2

$A_{s,min}$= 0.0033 × 250 × 1755 = 1608.75 mm^2

A_s by ETABS= 993 mm^2

To determine the shear design, the ration L_n /h shall be computed:

Ln / h = $\frac{1488.8}{1950} = 0.763$, must be $< 4 \rightarrow$ ok.

α_c= 0.25 for h_w/l_w = 0.763 < 1.5 → ok.

$$V_c = A_{cv}(\alpha_c\sqrt{f'_c} + \rho_t f_{yt})$$
$$= 250\times 1950\times(0.25\times\sqrt{28} + 0\times 420)/1000$$
$$= 644.9\text{ KN}$$
$$\phi V_c = 0.6\times 644.9 = 386.94\text{ KN}$$

ØV_c by ETABS = 222.16 kN → α_c by ETABS = 0.086

Using ØV_c,ETABS = 222.16 kN, the shear reinforcement is computed as follows:

$$V_s = V_u/\phi - V_c$$
$$= 506.68/0.75 - 222.16/0.6$$
$$= 474.21\text{ KN}$$

$$\frac{A_v}{S} = \frac{V_s}{f_{yt}d}$$
$$= \frac{474.21\times 1000}{420\times 1750}\times 10^6$$
$$= 0.643\text{ mm}^2/\text{mm}$$
$$= 643\text{ mm}^2/\text{m}$$

$\frac{A_v}{S}$ by ETABS = 643 mm^2

Different percentage = 0%

Diagonal shear reinforcement:

$$V_n = 2A_{vd} f_y \sin\alpha \le \sqrt{f'_c} A_{cw}, \text{or:}$$

$$A_{vd} = \frac{V_u}{2\phi f_y \sin\alpha}$$

$$\sin\alpha = \frac{0.8hs}{\sqrt{L_s^{\,2} + (0.8hs)^2}}$$

Where:

A_{vd} = area of diagonal reinforcement.

Øs = strength reduction factor = 0.85 (ACI 21.2-1, Table 21.2.1).

Ls = length of the spandrel; coupling beam.

hs = thickness of the spandrel; coupling beam.

α = is the angle between the diagonal bars and the longitudinal axis of the coupling beam.

$$\sin\alpha = \frac{0.8 \times 1950}{\sqrt{1488.8^2 + (0.8 \times 1950)^2}} = 0.723$$

$$A_{vd} = \frac{506.68 \times 1000}{2 \times 0.85 \times 420 \times 0.723} = 981.5 \text{ mm}^2$$

A_{vd} by ETABS = 984 mm^2

Different percentage = 0.312%

For design of slabs, beams, columns, walls, stair, diaphragms and collectors and non-structural elements refer to **chapter 3**.

4.7 Design of Footing:

This section includes the analysis and the design of the foundation system as a whole including the piles and matt foundation section using CSI SAFE 20.3 software.

4.7.1 Modelling:

Units: main three units used are meter (m) for length, kilo newton (KN) for force and Celsius (C) for temperature.

Materials: it was discussed in section **1.5**, but the definition of these materials on software program is shown in **Figure4.94.**

- Concrete:

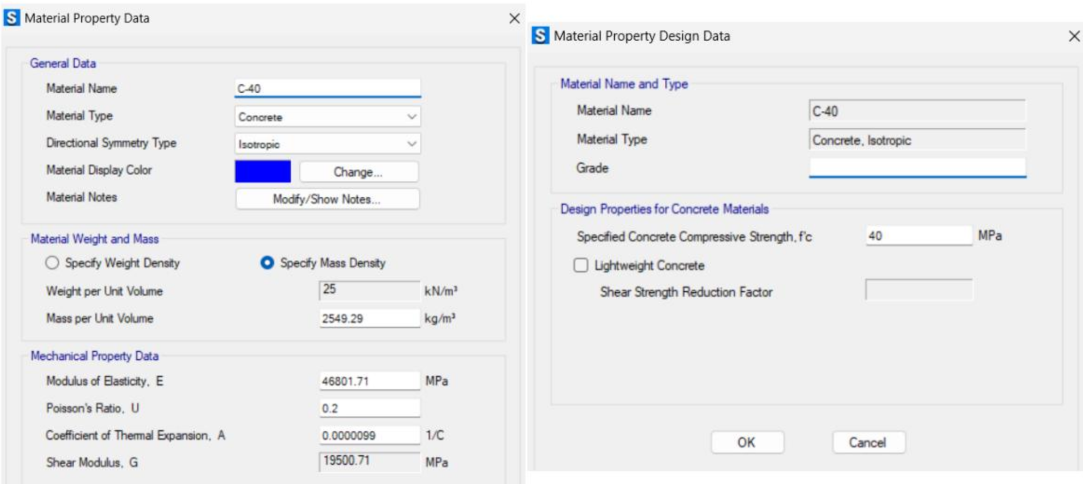


Figure4. 144: Concrete40 MPa definition in SAFE

Steel:

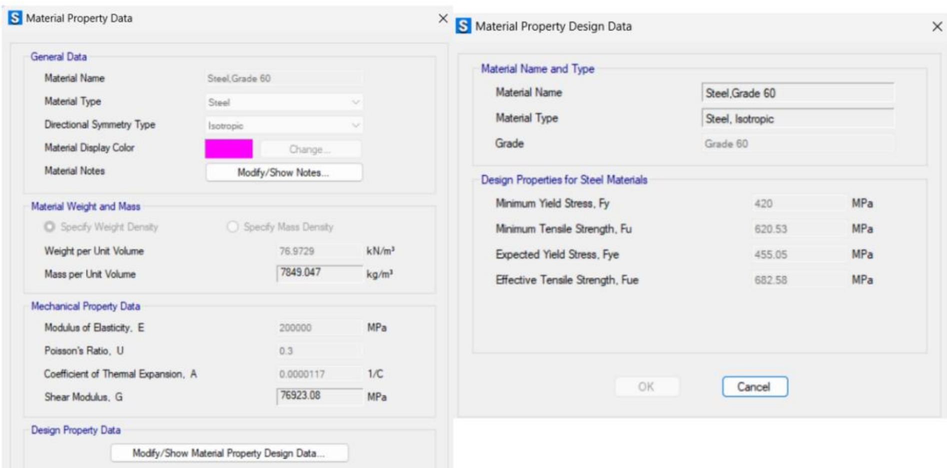


Figure4.95: Steel definition in SAFE.

Footing properties: the definition on software program will be as shown in **Figure 4.96.**

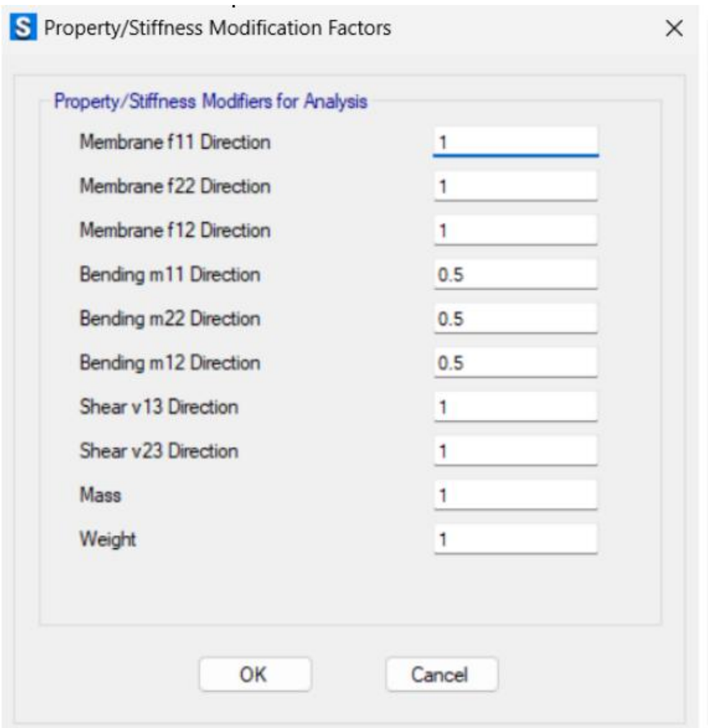


Figure4.96 :footing modifiers in safe.

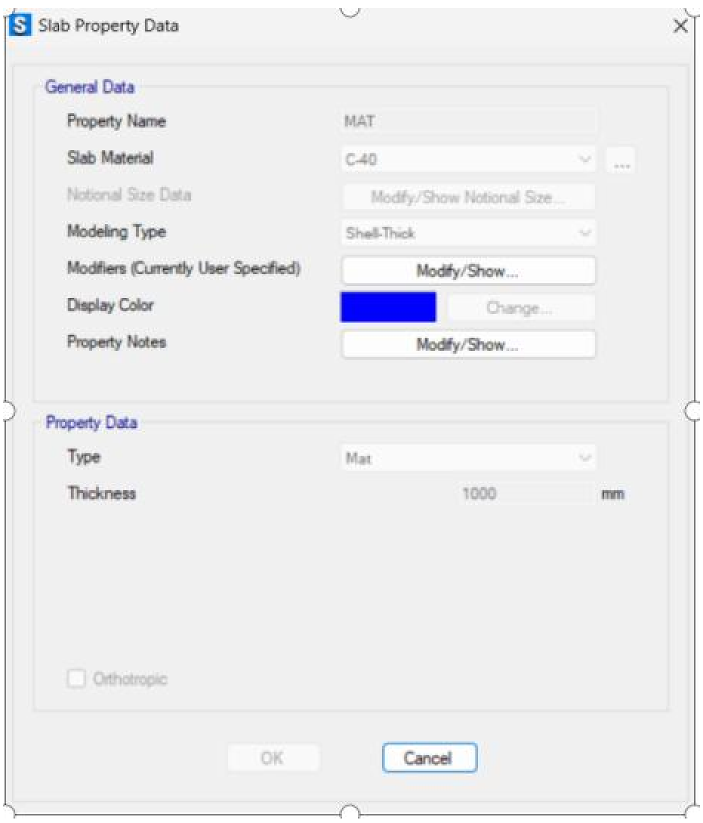


Figure4.97 :footing definition in SAFE.

Soil properties:

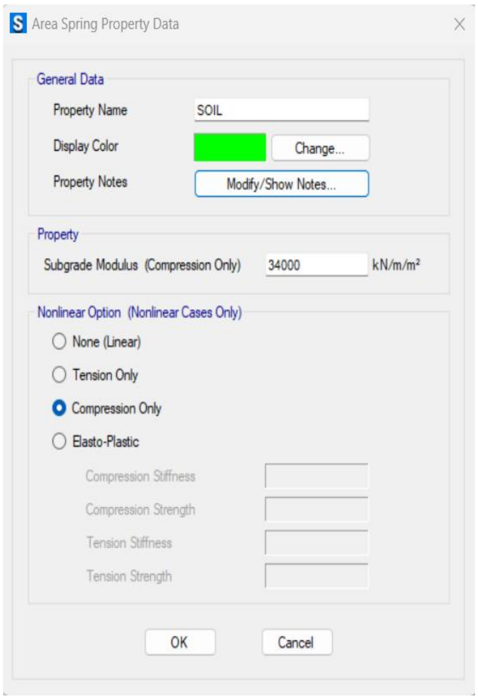


Figure4. 98 :Soil definition in SAFE.

Try mat thickness = 1100 mm

4.7.1 Check wide beam shear (One-way shear):

Maximum shear (at distance d from face of wall) 2174.6= KN/m, as shown in Figure

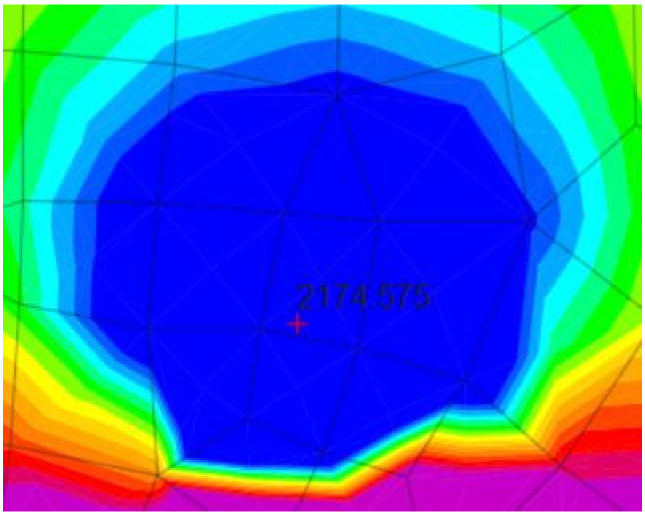


Figure4. 145 :One-way shear in footing

$$\phi V_c = \phi(0.17\lambda\sqrt{f'_c} + \frac{Nu}{6A_g})b_w d$$

Where:

$\varnothing = 0.75$

$\lambda = 1$

f'c: specified compressive strength of concrete, MPa (its equal to 40 MPa)

b_W: web width = 1000 mm

d: effective depth = 1000 mm

Nu: factored axial force normal to cross section occurring simultaneously with *Vu* or *Tu*; to be taken as positive for compression and negative for tension, N

$\varnothing V_C$ =790 KN/m

$V_u > \varnothing V_C$ (there is a need for shear reinforcement)

Shear reinforcement is needed in blue zones, where V_u larger than $\varnothing V_c$, **Figure4.100 shows shear forces in x direction from SAFE**…, expected zones around columns.

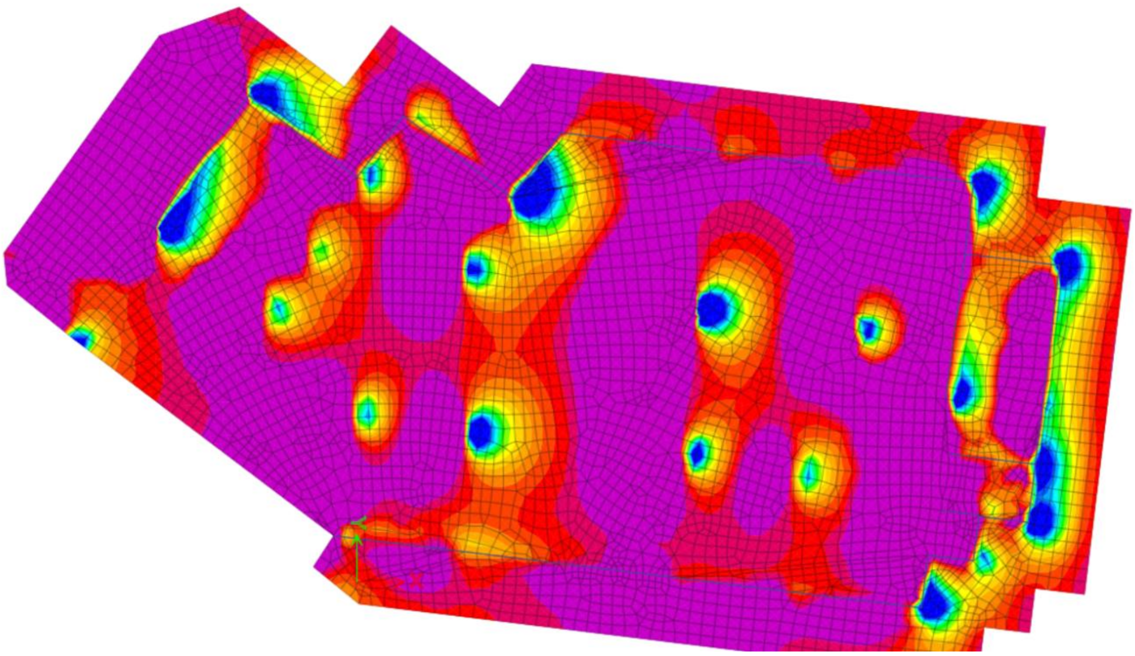


Figure4. 146 :shear forces in x direction.

Sample of calculations:

$V_n = V_c + V_s$

2175= 790/0.75+Vs

2175= 1053.33 +Vs

Vs = 1120 KN

$$0.33\sqrt{f_c'}b_w d = 0.33\times\sqrt{40}\times1000\times1000/1000 = 2090\text{KN}$$

$$\frac{A_v}{S} = \frac{V_s}{f_{yt}d} = \frac{1120\times1000}{420\times1000} = 2.67\ \text{mm}^2/\text{mm}$$

$$S = \frac{491\ \text{mm}^2}{2.67\ \text{mm}^2/\text{mm}} = 180\ \text{mm}$$

Use Ø20/150mm for shear reinforcement.

4.7.2 Check punching shear (Two-way shear):

Maximum punching shear = 0.574, as shown in **Figure 4.101**.

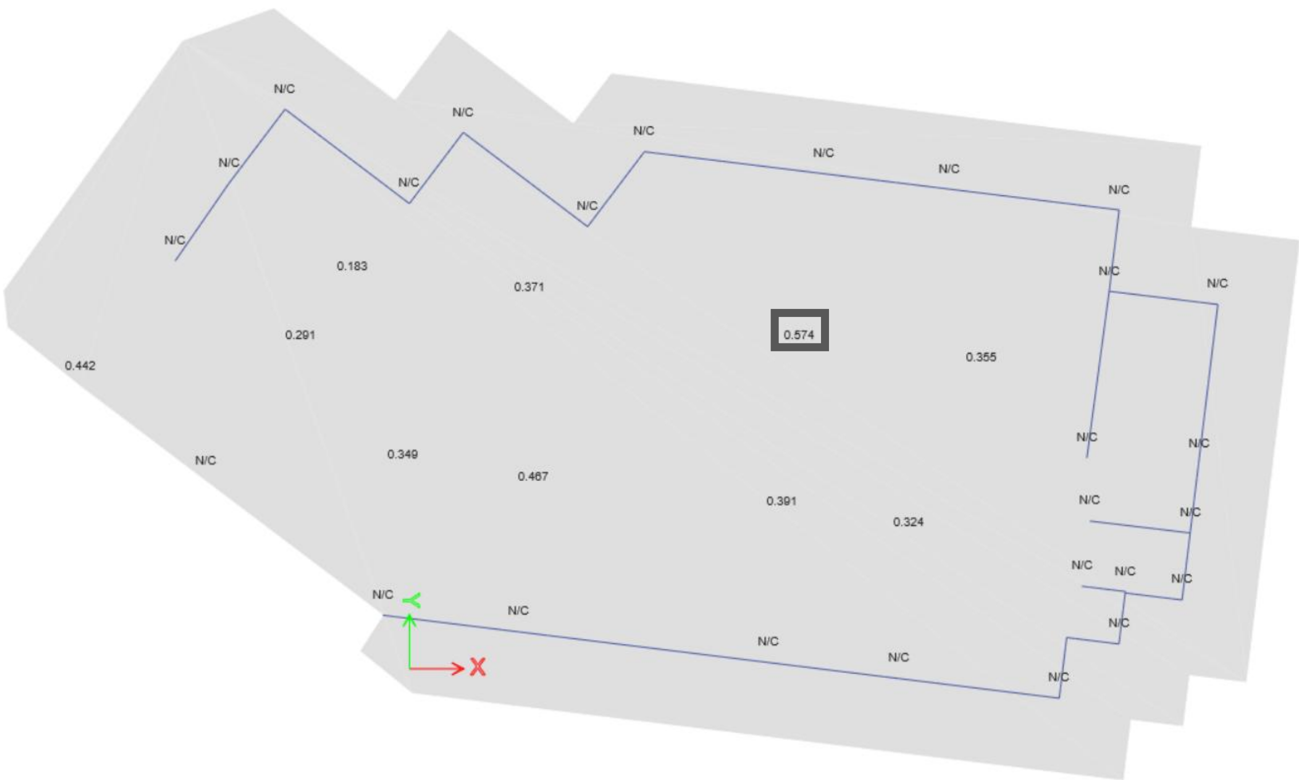


Figure4. 147 :punching shear in footing.

Max punching shear demand /capacity ration equal to 0.574 < 1 , OK

$$\phi V_c \le \phi 0.33\lambda_s\lambda\sqrt{f'_c}\, b_o d$$

$$\phi V_c \le \phi 0.17\lambda_s\lambda(1+\frac{2}{\beta})\sqrt{f'_c}\, b_o d$$

$$\phi V_c \le \phi 0.083\lambda_s\lambda(2+\frac{\alpha_s d}{b_o})\sqrt{f'_c}\, b_o d$$

Where:

$V_{u,p}$: ultimate punching shear (KN)

$\emptyset V_{c,p}$: punching shear capacity for footing (KN)

β = long side / short side of column = 0.9/0.5=1.8

αs = factor to consider column location = 40

b_o=perimeter length of the critical section (at d/2 from face of column) = 6400 mm

d = effective depth of footing =1000 mm

ɣs = √2/(1+.004d) ≤ 1.0

ɣs = 0.66

→$\emptyset V_{c,p}$= 10,018.09 KN

From SAFE $V_{u,p}$ =6011.854 KN < $\emptyset V_{c,p}$, OK

$V_{u,p}/\emptyset V_{c,p}$= 0.6

Diffirent percentage = (0.60-0.574)/0.6

= 4.33% →OK.

4.7.3 Check stresses under footings

This check will be made on mat foundation, to ensure that stresses under footings are less than bearing capacity and there is no tension under footings, so that no need for non-linear uplift analysis.

Allowable bearing capacity for the soil = 350 KN/m^2

Figure 4.102 shows soil pressure.

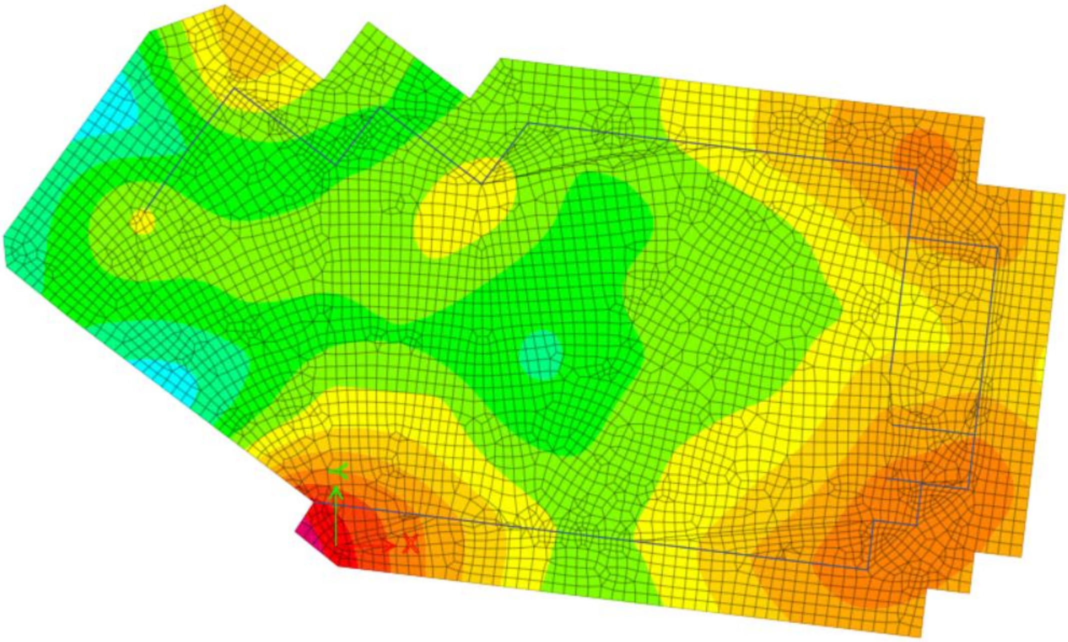


Figure4. 148 :stress in the footing.

4.7.4 Footing layout:

Figure 4.103 shows the right part footing layout.

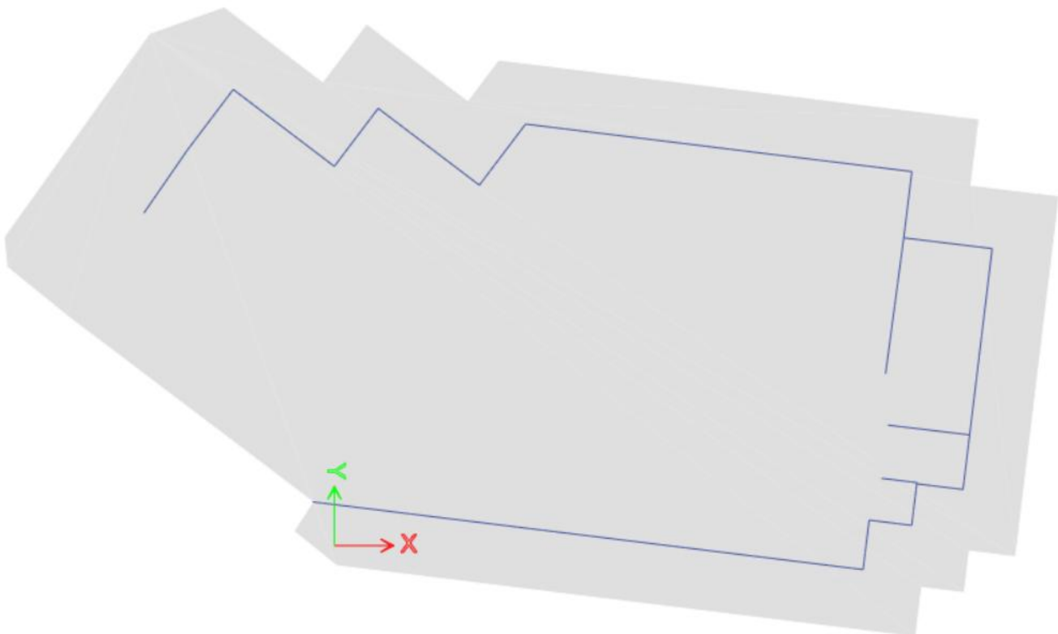


Figure4. 149 :Footing layout.

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