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**Faculty of Engineering and  
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**جامعة النجاح الوطنية  
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المعلومات**

## **Graduation Project Report II**

**Design of Sahab building at Palestine -Nablu**

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## **DEDICATION:**

This report was written by students at the Civil Engineering Department in the faculty of Engineering at 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 student(s). AN-Najah National University accepts no responsibility or liability for the consequences of this report being used for a purpose other than the purpose for which it was commissioned.

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## **ABSTRACT:**

Reinforced concrete structures are one of the most popular structural systems used in Palestine. This project is about the structural design of a new residential building in the city of Nablus (Sahab building).

This building consists of nine floors with total area of 4065 m<sup>2</sup>. This project will be divided into two parts. The first part intends to prepare the basic information required to facilitate efficient modeling and structural design and analysis of the building under static loads. The design of concrete elements is controlled by ACI 318-14 code. The second phase will be redesign of building considering gravity as well as seismic loads. This design will be based on ASCE 7-10 / IBC codes. Structural elements such as footings, stairs, columns, beams, slabs and shear walls will be completed and finalized in part two.

To make a mathematical model that represents the reality, 3-D structural model will be made using ETABS 2019 program. The analysis is verified using conceptual hand calculation for sample output from the program. Detailed design of all structural elements for gravity and seismic loads will be done and drawn using AutoCAD program in Project 2.

# 1 Chapter1: INTRODUCTION

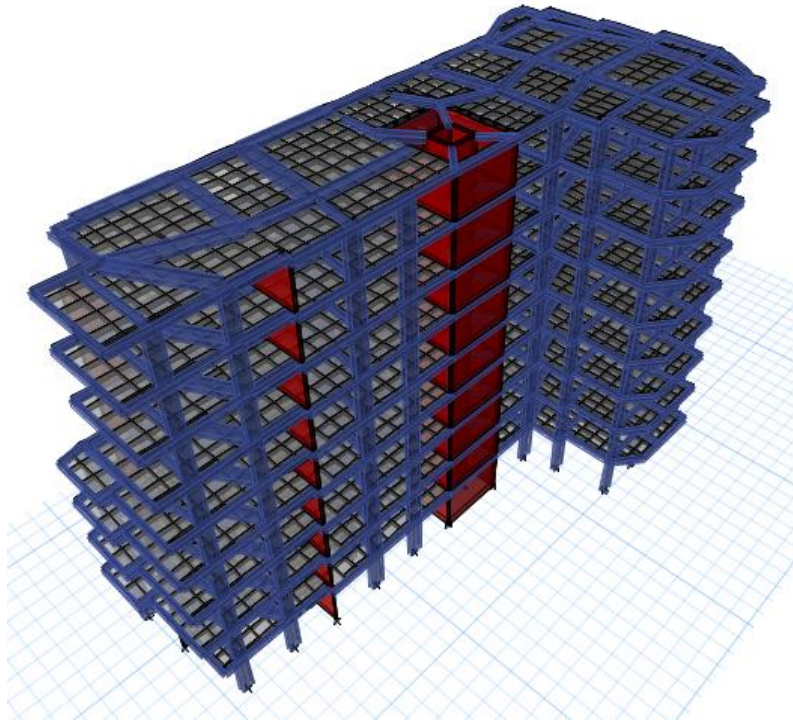
## 1.1 Purposes of the chapter

This chapter will discuss the project as a whole, first section will give a general description of the project, which includes the name, location, total area, and the number of floors. After that, it Indicates for each floor the contents, the approximate area, and the total height. also, it shows architectural plans and sections for the structure. The second section is the analysis and design principles, which is going to show designed methods such as the Working Stress Method, Ultimate Load Method, and Limit State Method. The software that will be used for design and analysis is ETABS. The third section will describe the soil type, the allowable bearing capacity, and site class. The fourth one will define codes and standards that will be utilized in this project. after that, a description will be given for materials that make up the building such as the structural elements mainly reinforcing bar, concrete. In addition to the non-structural components, such as plastering, tiles, and bricks. The most important properties that will be specified for concrete are the unit weight, modulus of elasticity, poissons ratio, and concrete compressive strength. For reinforcing bars, the properties that will be determined are yield strength. Finally, loads of both types: Dead loads, seismic loads, and live loads will be defined and set. besides, the load combinations from the code will be mentioned.

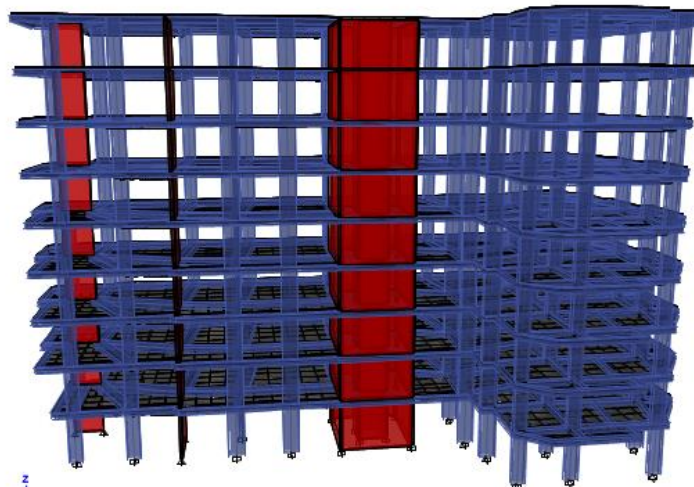
## 1.2 Project description

*Table 1-1 The height, finish ground level, and area for each floor of the building*

Level	Height (m)	Area ( $m^2$ )	Use
Ground floor	3.4	451	main entrance & Parking
(First – fourth) floor	3.	435	apartments
(Fifth- ninth) floor	3	422	apartments



*Figure 1.1 Sahab building*



*Figure 1.2 Sahab Building*

### **1.3 Analysis and design principle**

Many methods can be used for the design process such as Working Stress Method, which is called an Allowable stress method (ASM) is within its range of elastic limit. Limit State Method used for Limit Stress with two factors is used for Yield Stress and Partial Safety Factor. Ultimate Load Method This method is also called 'plastic design. And Ultimate Load Method will be used in this project, they will be used as a design method of the building structural elements. For the analysis and design process, ETABS software programs will be used.

### **1.4 Geotechnical investigation**

The soil information and the nature of the site are obtained after investigations of the soil, to determine if a structure can be built on it that is considered safe and economical. After conducting the tests, the nature of the area was determined as being semi-flat for this project, and it was recognized that the permissible bearing capacity of the soil is 166 KN/m<sup>2</sup>, which means that the soil is defined as Moist clay and Sand clay mixture.

### **1.5 Codes and standards:**

There are many codes in the world that can be used, but choosing the most appropriate one considering different variables and each has its own characteristics and standards, materials information will be obtained from Jordanian Code for loads and forces for structural and nonstructural components - used in the building in terms of the characteristics of concrete such as its unit weight .Also, the American Society of Civil Engineers (ASCE 7-16) has been relied on as a source of information and a reference in the same time associated with wind, and gravity loads, in addition to the American Concrete Institute (ACI 318-14) code will be utilized for design columns, beams, slabs, and walls, in addition, to obtain the yield, compressive strength and modulus of elasticity for concrete material, American Society for Testing Materials (ASTM) also will be used for materials sections and specifications, yield ultimate strength for reinforcing bars. Also, we use International Building Code (IBC- 2012) for seismic loads.

## 1.6 Materials:

Materials are the basis for setting up a structural element that is comprised together to accomplish such buildings, two types of materials will be explained down below.

### 1.6.1 Structural elements

Concrete and steel are playing a huge role as a fundamental material in building for a long time, concrete is a brittle material and has high resistance for compression forces on the other side it is a weak material in the tension zone, steel considers as ductile material and has an almost similar level of resistance in both tension and compression.

#### 1.6.1.1 Concrete

There are different properties for concrete to look for such as compressive strength, unit weight, modulus of elasticity, modulus of rupture, concrete tensile capacity, and poisons ratio. In this project, the ACI 318-14 code will be used to obtain information on these properties.

- **Concrete compressive strength ( $f_c'$ ): -**

The limitation values are determined according to ACI 318-14 code (Table 19.2.1.1). As the normal weight concrete will be used, there is no limitation on the maximum value, but the minimum value is equal to 21 MPa. So, the value of compressive strength that will be used is 28 MPa, which is equal to 350 kg/cm<sup>2</sup> as known in the domestic markets and already used in the field.

- **Modulus of elasticity ( $E_c$ ): -**

Which is the measurement of a material's elasticity and it quantifies a material's resistance to non-permanent, or elastic, deformation.

Calculated as the following equation:

$$E_c = 4700\sqrt{f_c'} \text{ (in MPa)}$$

$$E_c = 4700\sqrt{f_c'} = 4700 \times \sqrt{28} = 24870.06 \text{ MPa.}$$

- **Unit weight ( $\gamma$ ):**

It's primarily affected by the unit weight of the aggregate, which varies by geographical location and increases with concrete compressive strength, also according to the Jordanian code (Table 1-3), the unit weight for plain concrete that will be used is 23 KN/m<sup>3</sup> and 25 KN/m<sup>3</sup> for reinforced concrete.

- **Poisson's ratio:**

Measures the normal deformation to an applied force, and for the concrete, the Poisson's ratio is equal to 0.2.

### ***1.6.1.2 Reinforcing steel:***

- Yield strength ( $f_y$ )

18 Represent steel strength, the yield strength is equal to 420 MPa according to ASTM A615, and according to ACI 318-14 code (Table 20.2.2.4a).

- Modulus of elasticity ( $E_p$ )

The common ranges are between 197 and 200 GPa which are commonly used. So, the modulus of elasticity of 200 GPa will be chosen.

### **1.6.2 Non-structural elements:**

Non-structural elements are those elements within a building that are not considered to be part of either the primary or secondary structural systems. Examples of non-structural elements include components such as mechanical and electrical plants, windows, plastering, doors, tiles, masonry walls, and other non-load bearing elements.

- **Plastering**

The concrete mortar will be at 2cm thickness and unit weight equal to 23 kN/m<sup>3</sup>.

- **Tiles**

There are many types of tiles such as ceramic, stone, metal, marble and granite, the marble type will be used to cover floors with 3cm thickness and unit weight equal to 27 KN/m<sup>3</sup>.

- **Blocks**

Hollow blocks will be utilized in the project, 20 cm blocks will be used on the outer wall (perimeter), and Blocks 10 cm will be used in the internal partitions, And the unit weight for them shown in the (Table: 1-2) according to Jordanian code.

- **Fill material**

It consists mainly of sand and the different size of aggregate. And the thickness of the fill material is considered to be near 10 cm with a unit weight of almost 20 KN/m<sup>3</sup> .

- **Masonry Stone**

The main purpose of using the stones in building construction for beauty aspect and Insulation purposes, there is different types of stone in the domestic markets like mofajer (bomber), msemsem, tabzh and matabba, in this project mofajer and tabzh will be used, the unit weight depends on the classes of the stones, A = 2.56, class B = 2.45 -2.56, class C = 2.16 -2.45. in addition to the Absorption must not exceed 3% absorption for Class A and no more than 4.2% for Class B and not more than 7.5% of Class C, and all these properties depend on the location that the stone extracted from.

The specific gravity of used material shown in Table 1-2:

*Table 1-2 Material used in design and their specific gravity according to Jordanian code*

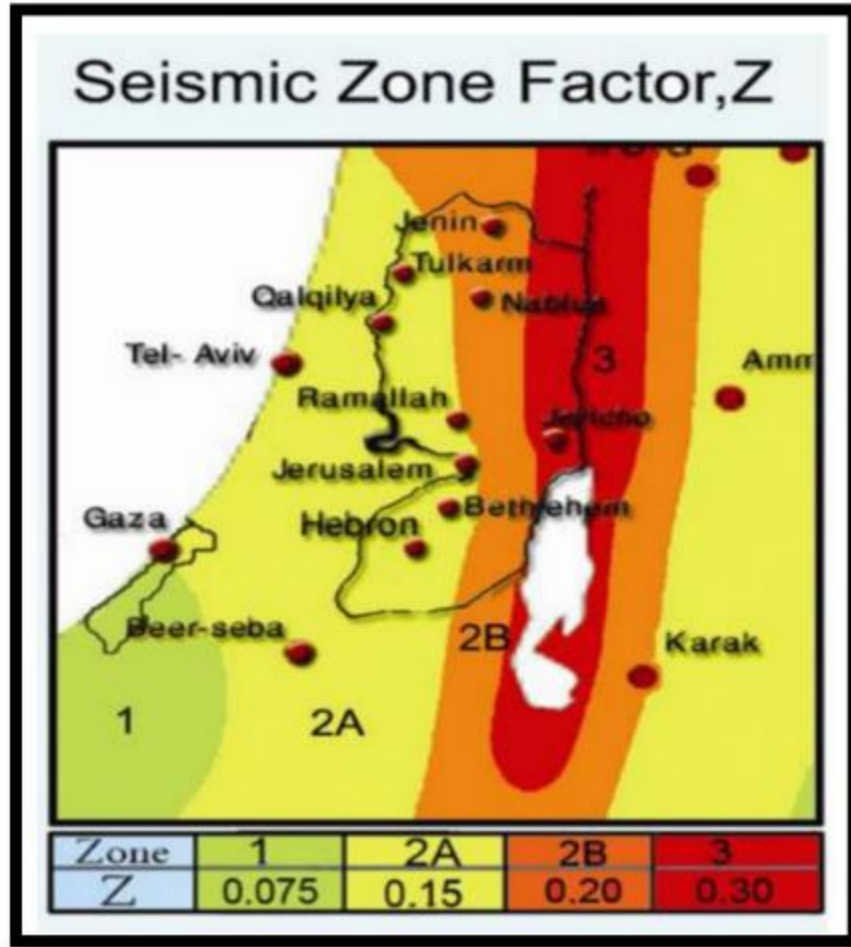
Material name	$\gamma$
Tiles	27kN/m <sup>3</sup>
Mortar	23kN/m <sup>3</sup>
filling material	20kN/m <sup>3</sup>
Plastering	23kN/m <sup>3</sup>
concrete block wall – 200 mm thick	13.5kN/m <sup>3</sup>
concrete block wall – 150 mm thick	14kN/m <sup>3</sup>
concrete block wall – 100 mm thick	14.5kN/m <sup>3</sup>
Stone	27kN/m <sup>3</sup>
false ceiling	0.25kN/m <sup>3</sup>
Glass	1.5kN/m <sup>3</sup>
Solar cells	0.16kN/m <sup>3</sup>

## **1.7 Seismicity of the Site and Structure:**

This section will discuss the main parameters that determine the effect of earthquake on the building and the proper structural system according to these parameters:

### **1.7.1 Seismicity of the site:**

Seismicity of the site can be determined using several parameters, the major one is peak ground acceleration which expressed by Z factor. The Z factor gives the hazard associated with earthquake in different regions. Figure 1- 3 show the Palestine seismic hazard map with 10% probability of exceedance in 50 years.



*Figure 1.3 Palestine seismic hazard map*

Nablus region, where our building will be built, is classified as zone 2B with  $Z=0.2$ .

ASCE 7-10 use the parameters  $S_s$  and  $S_1$  to establish the loading criteria for seismic design.

These parameters are risk targeted. To evaluate these parameters, there are two methods as the following:

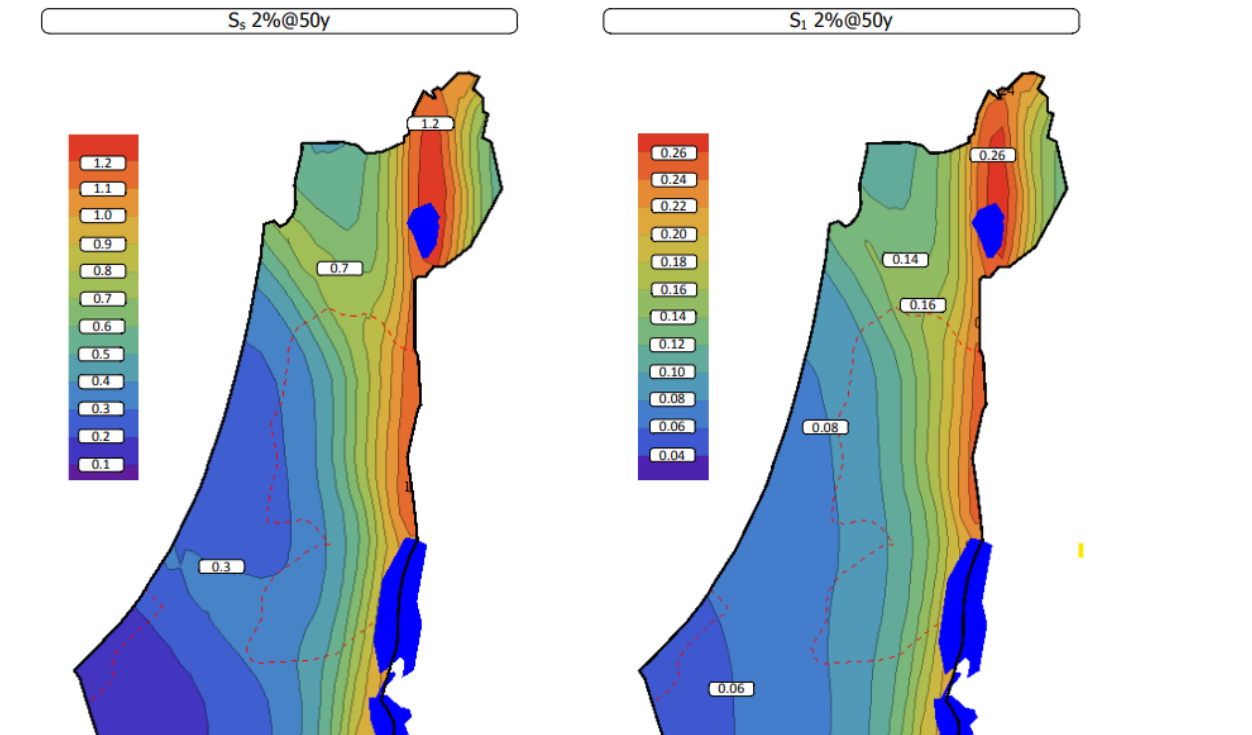
1. Empirical formulas have been developed to calculate  $S_s$  and  $S_1$  based on Z factor.

$$S_s = 2.5 \times Z \dots\dots\dots (1-1)$$

$$S_1 = 1.25 \times Z \dots\dots\dots (1-2)$$

## 2. Acceleration Parameters Maps:

These maps provide the value of  $S_s$  and  $S_1$ , there are many sources for these maps like USGS web site, SEI web site, and Israel maps, these maps are available with 2%,5% and 10% probability of exceedance within 50 years. Figure 1- 4 shows the Israel acceleration parameters  $S_s$  ,  $S_1$  maps for 2% probability of exceedance.



*Figure 1.4 acceleration parameters maps*

In this Project, the acceleration parameters are taken from the maps

we take  $S_s = 0.6$  and  $S_1 = 0.14$

Soil may have a disastrous effect on the building by amplifying the peak ground acceleration. Site-specific soil parameters are used to account for the soil. ASCE 7-10 gives the values of the amplifying factor  $F_a$  and  $F_v$  as function of site class and  $S_s$ ,  $S_1$  as illustrated in Table 1.3 and Table 1-4:

*Table 1-3 Site Coefficient,  $F_a$*

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

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

*Table 1-4 Site Coefficient,  $F_v$*

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

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

For site class D,  $F_a = 1.2$  and  $F_v = 1.65$

Then, the amplified accelerations,  $S_{ms}$  and  $S_{m1}$ , are:

$$S_{ms} = F_a \times S_s = 1.2 \times 0.6 = 0.72$$

$$S_{m1} = F_v \times S_1 = 1.65 \times 0.14 = 0.231$$

Design Earthquake spectral response acceleration parameter at short period 0.2s ( $S_{DS}$ ), and at 1s ( $S_{D1}$ ), shall be determined from the following equation:

$$S_{DS} = \frac{2}{3} \times S_{ms} = \frac{2}{3} \times 0.72 = 0.48$$

$$S_{D1} = \frac{2}{3} \times S_{m1} = \frac{2}{3} \times 0.231 = 0.154$$

## 1.7.2 Seismicity of the Structure:

The structure response to earthquake is governed by the ductility of the responding system. The design earthquake is established based on the following concept:

- Risk category
- Seismic design category

### 1.7.2.1 Risk category

A categorization of buildings and other structures for determination of flood, wind, snow, ice, and earthquake loads based on the risk associated with unacceptable performance. So, the risk category for this structure is II as shown in Table 1-5 (ASCE 7-16 Table 1.5-1).

**Table 1-5 Risk category of building and other structures (ASCE 7-16**

**Table 1.5-1)**

**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. <sup>a</sup>	
Buildings and other structures required to maintain the functionality of other Risk Category IV structures.	

<sup>a</sup>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.

ASCE 7-10 assign importance factor ( $I_e$ ) for seismic design of 1.00 for risk category II as shown in table 1-6:

**Table 1-6 Importance Factors of Buildings and Other Structures (ASCE 7-16 Table 1.5-2)**

**Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads<sup>a</sup>**

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

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

**1.7.2.2 Seismic design category:**

Seismic design category also associated with the risk of the structure and used in evaluating the analytical procedure of seismic analysis and the permitted seismic force-resisting system.

**Table 1-7 Seismic Design Category Based on Short Period Response Acceleration Parameter**

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

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

**Table 1-8 Seismic Design Category Based on 1-S Period Response Acceleration Parameter**

**Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter**

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

According to the previous tables the Seismic Design Category is C.

## 1.8 Load

In any building design, loads are a primary consideration since they determine the type and severity of hazards or external forces a building must resist to provide acceptable efficiency over the useful life of the structure. The predicted loads depend on the planned usage of a building, size, type, and shape.

In this section, a set of loads is identified and listed as gravity loads, lateral loads, in addition to temperature load. All loads are evaluated either using specifications of ASCE (7-16) Minimum Design Loads for Buildings and Other Structures or using Jordanian code.

## 1.8.1 Gravity load:

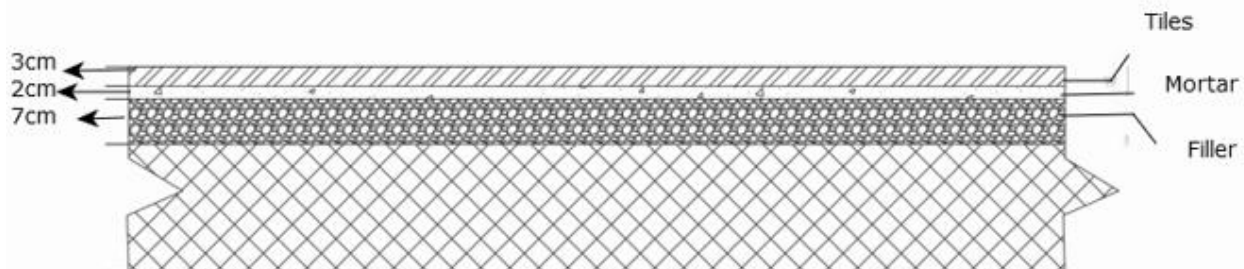
Gravity loads are all vertical forces that act on a structure. The weight of the structure, human occupancy, floor coverings, furniture, machine, and snow are all types of loads that need to have a complete load path to the ground. So, detailed information about each one will be discussed.

### 1.8.1.1 Dead load

They are the permanent forces resulting from the gravitational forces they are constant in magnitude, position, and do not change with the increase in the structures' age. And it is due to the self-weight of structural members. The analysis program will consider the weight of the slabs, beams, columns and other structural members after specifying the cross-sections of members and identifying materials properties that are used in their construction.

### 1.8.1.2 Superimposed dead

load Superimposed dead load is the weight of the non-structural and semi-permanent members. And it includes tiles, fill materials, mortar, bricks, claddings, finishes, plastering, and other material. The weight of these materials is calculated according to the information from the Jordanian code.



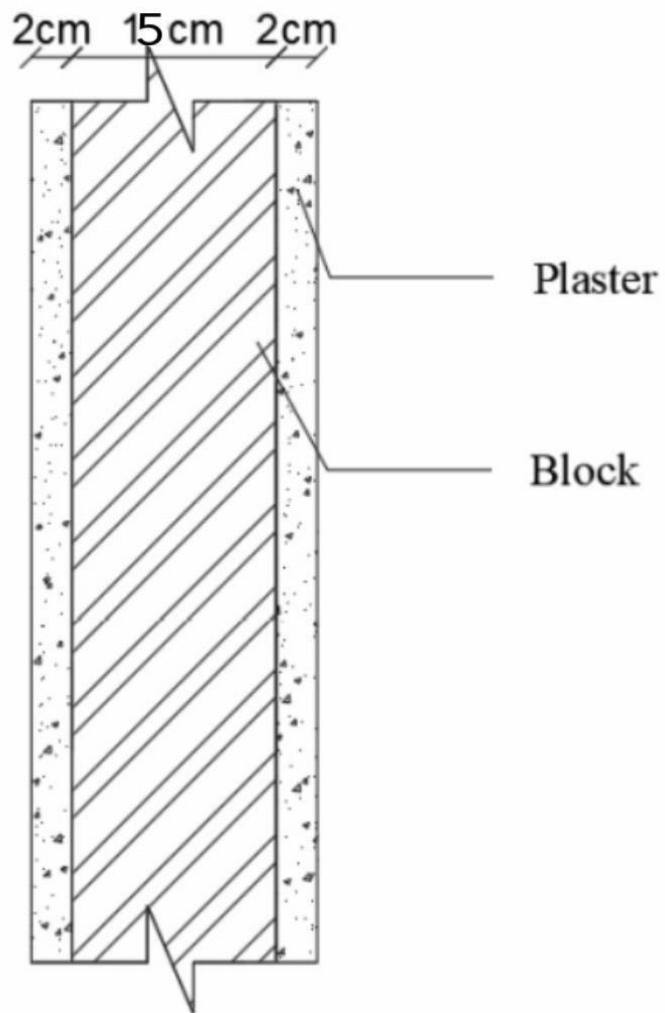
*Figure 1.5 Slab cross-section*

$$WSD = W_{\text{tile}} + W_{\text{mortar}} + W_{\text{filler}}$$

$$= 27 \times 0.03 + 23 \times 0.02 + 20 \times 0.07 = 2.67 \text{ kN/m}^2$$

**- Partition walls dead load: -**

Partition walls are the walls, which use to divide the apartment into a certain number of rooms. They consist of blocks and plaster. A Partition's cross-section shown in Figure 1-5.



***Figure 1.6 Partition cross-section***

**For block 20: -**

$$\mathbf{WSD = W Plaster + W Block = 2 \times 23 \times 0.02 + 0.2 \times 13.5 = 3.62 \text{ kN/m}^2}$$

**For block 10: -**

$$\mathbf{WSD = W Plaster + W Block = 2 \times 23 \times 0.02 + 0.1 \times 14.5 = 2.37 \text{ kN/m}^2}$$

**For block 15: -**

$$\mathbf{WSD = W Plaster + W Block = 2 \times 23 \times 0.02 + 0.15 \times 14 = 3.02 \text{ kN/m}^2}$$

**Example:**

$$\mathbf{\text{Total force for ground floor} = WSD \times \text{Lenght} \times \text{Height}}$$

$$= (3.02 \times 20 \times 3.35) + (2.37 \times 30 \times 3.35) = 440.5 \text{ kN}$$

$$\mathbf{\text{Weight} = \text{Total force} / \text{Area}}$$

$$= 440.5 / 451 = 0.98 \text{ kN/m}^2$$

$$\mathbf{\text{Total force from first floor to sixth floor} = WSD \times \text{Lenght} \times \text{Height}}$$

$$= (3.02 \times 30 \times 3.05) + (2.37 \times 75 \times 3.05) = 818.5 \text{ kN}$$

$$\mathbf{\text{Weight} = \text{Total force} / \text{Area}}$$

$$= 818.5 / 554 = 1.48 \text{ kN/m}^2$$

$$\mathbf{\text{Total force for Roof} = WSD \times \text{Lenght} \times \text{Height}}$$

$$= (3.02 \times 25 \times 2.5) + (2.37 \times 65 \times 2.5) = 573.9$$

$$\mathbf{\text{Weight} = \text{Total force} / \text{Area}}$$

$$= 573.9 / 369.5 = 1.55 \text{ kN/m}^2$$

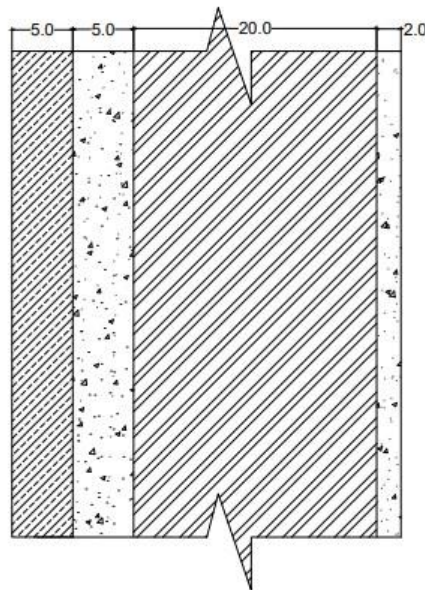
*Table 1-9 Slab superimposed dead load*

Floor Slab	Materials	Weight ( $\text{kN/m}^2$ )	Summation ( $\text{kN/m}^2$ )
Ground floor	Tiles, mortar and fill	<b>2.67</b>	<b>3.65</b>
	Internal partitions	<b>0.98</b>	
First floor	Tiles, mortar and fill	<b>2.67</b>	<b>4.15</b>
	Internal partitions	<b>1.48</b>	
Second floor	Tiles, mortar and fill	<b>2.67</b>	<b>4.15</b>
	Internal partitions	<b>1.48</b>	
Third floor	Tiles, mortar and fill	<b>2.67</b>	<b>4.15</b>
	Internal partitions	<b>1.48</b>	
Fourth floor	Tiles, mortar and fill	<b>2.67</b>	<b>4.15</b>
	Internal partitions	<b>1.48</b>	
Fifth floor	Tiles, mortar and fill	<b>2.67</b>	<b>4.15</b>
	Internal partitions	<b>1.48</b>	
Sixth floor	Tiles, mortar and fill	<b>2.67</b>	<b>4.15</b>
	Internal partitions	<b>1.48</b>	

Seventh	Tiles, mortar and fill	<b>2.67</b>	<b>4.15</b>
	Internal partitions	<b>1.48</b>	
Eighth	Tiles, mortar and fill	<b>2.67</b>	<b>4.15</b>
	Internal partitions	<b>1.48</b>	
Nineth	Tiles, mortar and fill	<b>2.67</b>	<b>4.15</b>
	Internal partitions	<b>1.48</b>	

### - Stone walls

They are the building's exterior walls where, in addition to providing additional protection for the building from weather conditions, they are used to provide an aesthetic view. A Perimeter wall shown in Figure 1-6.



*Figure 1.7 Perimeter wall*

$$\begin{aligned} \text{WSD} &= \text{Wstone} + \text{Wconcrete} + \text{Wblock} + \text{Wplaster} \\ &= 0.05 \times 27 + 0.05 \times 23 + 0.2 \times 13.5 + 0.02 \times 23 = 5.66 \text{ KN/m}^2 \end{aligned}$$

The value of superimposed dead load that will be taken is equal to 5.66 kN/m<sup>2</sup>.

**-Staircase:**

- The superimposed dead load for the staircase will be determined, as the riser is 0.17 m and the going is 0.27 m. Then:

$$\begin{aligned} \text{SD} &= \text{weight of reinforced concrete, tiles, plastering, concrete and fill material} \\ &= ((0.5 \times 0.3 \times 0.176 \times 25) / \sqrt{(0.3^2 + 0.176^2)}) + (0.03 \times 27) + (0.02 \times 23) + \\ & (0.05 \times 20) = 4.17 \text{ kN/m}^2 \end{aligned}$$

The value of superimposed dead load for the stair that will be taken is 4.17 kN/m<sup>2</sup>.

- The superimposed dead load for the stand slab will be determined, as the riser is 0.45 m and the going is 0.9m. Then:

$$\begin{aligned} \text{SD} &= \text{weight of reinforced concrete (steps)} \\ &= (0.5 \times 0.3 \times 1.5 \times 25) / \sqrt{(0.3^2 + 1.5^2)} = 4.9 \text{ kN/m}^2 \end{aligned}$$

The value of superimposed dead load for the stair that will be taken is 4.9 kN/m<sup>2</sup>.

### **1.8.1.3 Live load**

Loads that do, or can, change over time, such as people walking around a building (occupancy) or movable objects such as furniture. They vary according to usage and capacity. However, design codes can provide equivalent loads for various structures. These loads will be taken according to the ASCE 7-16 Table 1-4, which are empirical and conservative based on experience and accepted practice.

Occupancy or Use	Uniform load
Residential	1.92 KN/m <sup>2</sup>
Parking	6 KN/m <sup>2</sup>

### **1.8.2 Lateral load**

Seismic forces are distributed based on the stiffness of the structural elements. The total base shear is calculated based on the peak ground acceleration (section 1-7) and the effective mass of the structure. For comparison and convenience reason, the dynamic analysis is discussed in earthquake section

## **1.9 Load combinations:**

Load combinations are an important aspect of design. These load combinations are produced when more than one load type is operating on the chassis.

In this section, there are two basic types of load combinations to consider during the design process which are service load and ultimate load. Some combinations must also be considered according to special conditions and specifications related to earthquake loads and directions. Therefore, ASCE 7-16 code will be used to obtain the appropriate loading combinations and when they can be used.

## 1.9.1 Service load combinations:

### Without Earthquake:

The design process will be done according to the following combinations:

- $D+SD + L$
- $D+ SD$

### With Earthquake:

1.  $1D + 1SD$
2.  $1D + 1SD + 1L$
3.  $1.109 D + 0.91EQ_x+0.273EQ_y$
4.  $1.109D + 0.91EQ_y+0.273EQ_x$
5.  $1.081D+ 0.75L+ 0.683EQ_x +0.205 EQ_y$
6.  $1.081D+ 0.75L+ 0.683EQ_y +0.205 EQ_x$
7.  $0.492D + 0.91EQ_x+0.273EQ_y$
8.  $0.492D + 0.91EQ_y+0.273EQ_x$

## 1.9.2 Ultimate load combinations:

### Without Earthquake:

Structures and components shall be designed so that their design strength equals or exceeds the effects of the factored loads in the following combinations:

- $1.4D + 1.4SD$
- $1.2D + 1.2SD + 1.6L$

### With Earthquake:

1.  $1.4 D + 1.4 SD$
2.  $1.2D + 1.2SD + 1.6L$
3.  $1.355D + 1.0L + 1.3EQ_x + 0.39EQ_y$
4.  $1.355D + 1.0L + 1.3EQ_y + 0.39EQ_x$
5.  $0.745D + 1.3 EQ_x + 0.39 EQ_y$
6.  $0.745D + 1.3 EQ_y + 0.39 EQ_x$

## 2 CHAPTER 2: PRELIMINARY DESIGN:

### 2.1 General:

In this section the preliminary dimensions of the main elements of the structure (column, slab, wall and beam) will be determined, the dimensions will be determined so that these elements will fulfill the serviceability requirement.

### 2.2 Preliminary thickness of slab:

Our project consists of a residential building that contains long span sections and short span sections, so we used one-way ribbed slabs and two-way ribbed slabs.

*Table 2-1 Minimum depth of non-prestressed beams (ACI 318-14 Table 9.3.1.1)*

<b>Support condition</b>	<b>Minimum <math>h^{(1)}</math></b>
Simply supported	$\ell/20$
One end continuous	$\ell/24$
Both ends continuous	$\ell/28$
Cantilever	$\ell/10$

### 2.2.1 Preliminary slab thickness for the Ground floor:

*Table 2-2 Preliminary slab thickness for the Ground floor:*

Panel number	Length(m)	Factor	Thickness(mm)
1	2.83	21	134.761905
2	3.32	21	158.095238
3	3.28	21	156.190476
4	2.26	18.5	122.162162
5	2.89	18.5	156.216216
6	3.04	18.5	164.324324
7	3.87	21	184.285714
8	4.04	18.5	218.378378
9	5.3	18.5	286.486486
10	4.08	18.5	220.540541
11	3.36	21	160
12	2.99	21	142.380952
13	3.21	21	152.857143
14	1.2	8	150
15	1.47	8	183.75
16	1.23	8	153.75

## 2.2.2 Preliminary slab thickness of first to fourth floor:

*Table 2-3 Preliminary slab thickness of first to fourth floor:*

Panel number	Length(m)	Factor	Thickness(mm)
1	2.83	21	134.7619
2	3.32	21	158.0952
3	3.28	21	156.1905
4	2.26	18.5	122.1622
5	3.87	21	184.2857
6	4.04	18.5	218.3784
7	5.3	18.5	286.4
8	4.08	18.5	220.5405
9	3.36	21	160
10	2.99	21	142.381
11	3.21	21	152.8571
12	1.2	8	150
13	1.47	8	183.75
14	1.23	8	153.75

### 2.2.3 Preliminary slab thickness from fifth to ninth floors:

Table 2-4 Preliminary slab thickness from fifth to ninth floors:

Panel number	Length(m)	Factor	Thickness(mm)
1	2.83	21	134.7619
2	3.32	21	158.0952
3	3.28	21	156.1905
4	2.26	18.5	122.1622
5	3.87	21	184.2857
6	4.04	18.5	218.3784
7	5.3	18.5	286.4
8	4.08	18.5	220.5405
9	3.36	21	160
10	2.99	21	142.381
11	3.21	21	152.8571
12	1.2	8	150
13	1.47	8	183.75
14	1.23	8	153.75

→ the minimum thickness slab of the = 286.4 mm. take thickness 320mm.

The final used thickness for slabs will be 320 mm.

Table 2-5 Ultimate loads for each slab

Floor slab	Own weight concrete (KN/m <sup>2</sup> )	Rib weight (KN/m <sup>2</sup> )	Superimposed dead (KN/m <sup>2</sup> )	Live load (KN/m <sup>2</sup> )	Ultimate load (W <sub>u</sub> ) KN/m <sup>2</sup>
1-9floor One way	3.6	2.15	4.15	1.92	14.9
1-9 floor two way	4.83	1.52	4.15	1.92	15.7

## 2.3 Preliminary slab thickness and widths For Beams:

### Thickness:

We use a hidden beams so the thickness = 320mm

### Widths:

We use this equation to find width of beams:

$$bd^2 = \frac{M_u}{\phi \rho f_y \left(1 - \frac{\rho f_y}{1.7 f'_c}\right)}$$

Table 2-6 widths For Beams

Length	Mu	Area	b(mm)	b(cm)
1.65	5.07	0.9	14	1.4
1.72	5.51	0.91	16	1.6
1.59	4.71	1.81	27	2.7
1.57	4.59	2.15	31	3.1
1.61	4.83	2.21	33	3.3
1.61	4.83	2.22	33	3.3
2.88	15.45	0.73	35	3.5
2.39	10.64	1.08	36	3.6
2.63	12.88	0.9	36	3.6

1.6	4.77	2.7	40	4.0
3.52	23.08	0.65	47	4.7
3.47	22.43	0.72	50	5.0
2.53	11.92	1.53	57	5.7
2.6	12.59	1.74	68	6.8
3.53	23.21	0.95	69	6.9
2.1	8.21	2.87	74	7.4
3.6	24.14	0.99	75	7.5
5	46.56	0.58	84	8.4
3.09	17.78	2.7	150	15.0

b1=30  
cm

5.02	46.94	2.07	303	30.3
3.6	24.14	4.08	307	30.7
4.21	33.01	3.11	320	32.0
4.5	37.72	2.8	329	32.9
3.64	24.68	4.38	337	33.7
3.72	25.77	4.22	339	33.9
4.65	40.27	2.8	352	35.2
3.5	22.82	5.3	377	37.7
3.55	23.47	5.35	392	39.2
4.47	37.21	3.42	397	39.7
4.13	31.77	4.35	431	43.1
5.01	46.75	3.11	454	45.4
4.31	34.60	4.54	490	49.0
5.15	49.40	3.75	578	57.8

b2=60  
cm

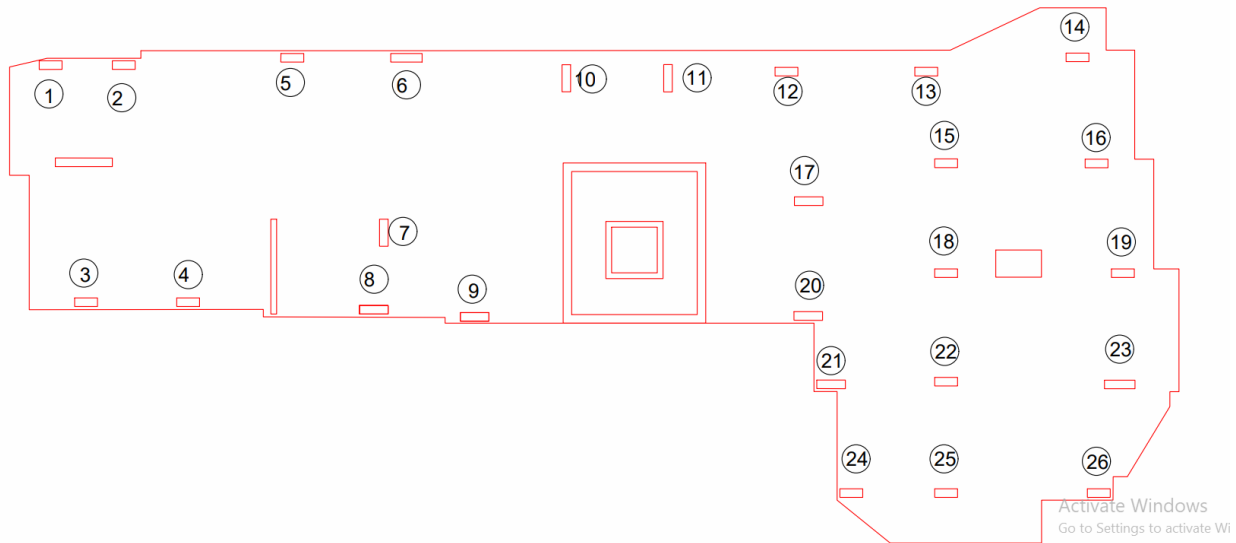
5.42	54.71	3.78	645	64.5
5.4	54.31	4.24	718	71.8
5.4	54.31	4.24	718	71.8
4.96	45.82	5.06	723	72.3

b3=80  
cm

6.13	69.99	3.71	810	81.0
5.54	57.16	4.78	853	85.3
5.91	65.05	4.27	867	86.7
5.56	57.58	5.07	911	91.1

b4=100cm

## 2.4 Preliminary dimensions of Column:



$$P_u = \#floors \times ultimate\ load \times A = (1 \times 19.98 \times 22.9) + (5 \times 14.9 \times 22.9) + (1 \times 15 \times 22.9) = 2507.1\ KN$$

$$\text{For beams } P_u = \#floors \times ultimate\ load \times A = 1 \times (2.98 \times 0.5 \times 0.25 \times 25) + (1.66 \times 0.27 \times 0.135 \times 25) + (2.76 \times 0.4 \times 0.2 \times 25) + (1.21 \times 0.2 \times 0.1 \times 25) = 16.98\ KN$$

$$5 \times 8.61 \times 0.32 \times 0.65 \times 25 = 223.86\ KN$$

$$\sum P_u = 2507.1 + 16.98 + 223.86 + 1.2(0.85 \times 0.3 \times 25 \times 21.4) = 2911.65\ KN$$

Find the dimensions:

Assume  $\rho = 0.01$

so  $A_s = 0.01 A_g$

$$A_c = (A_g - 0.01 A_g) = 0.99 A_g$$

$$\phi P_n = \phi \lambda (0.85 f_c' A_c + f_y A_s)$$

$$2911.65 \times 10^3 = 0.6 \times 0.8 \times ((0.85 \times 28 \times 0.99 \times A_g) + (0.01 \times 420 A_g))$$

$$\text{So, } A_g = 219 \times 10^3 \text{ mm}^2$$

We assume the dimension of the column equal to  $0.85 \times 0.3$

**Take  $0.95 \times 0.3$**

$$P_u = \# \text{floors} \times \text{ultimate load} \times A = (1 \times 19.98 \times 22.9) + (5 \times 14.9 \times 22.9) + (1 \times 15 \times 22.9) = 2507.1 \text{ KN}$$

$$\text{For beams } P_u = \# \text{floors} \times \text{ultimate load} \times A = 1 \times (2.75 \times 0.4 \times 0.2 \times 25) + (1.5 \times 0.27 \times 0.135 \times 25) + (1.86 \times 0.27 \times 0.135 \times 25) + (1.6 \times 0.27 \times 0.135 \times 25) = 10.02 \text{ KN}$$

$$5 \times 7.71 \times 0.32 \times 0.65 \times 25 = 200.46 \text{ KN}$$

$$\sum P_u = 2507.1 + 10.02 + 200.46 + 1.2(0.7 \times 0.3 \times 25 \times 21.4) = 2852.4 \text{ KN}$$

Find the dimensions:

$$\text{Assume } \rho = 0.01$$

$$\text{so } A_s = 0.01 A_g$$

$$A_c = (A_g - 0.01 A_g) = 0.99 A_g$$

$$\phi P_n = \phi \lambda (0.85 f_c' A_c + f_y A_s)$$

$$2852.4 \times 10^3 = 0.6 \times 0.8 \times ((0.85 \times 28 \times 0.99 \times A_g) + (0.01 \times 420 A_g))$$

$$\text{So, } A_g = 214 \times 10^3 \text{ mm}^2$$

We assume the dimension of the column equal to  $0.75 \times 0.3$

**Take  $0.8 \times 0.3$ .**

*Table 2-7 Preliminary dimensions of Column*

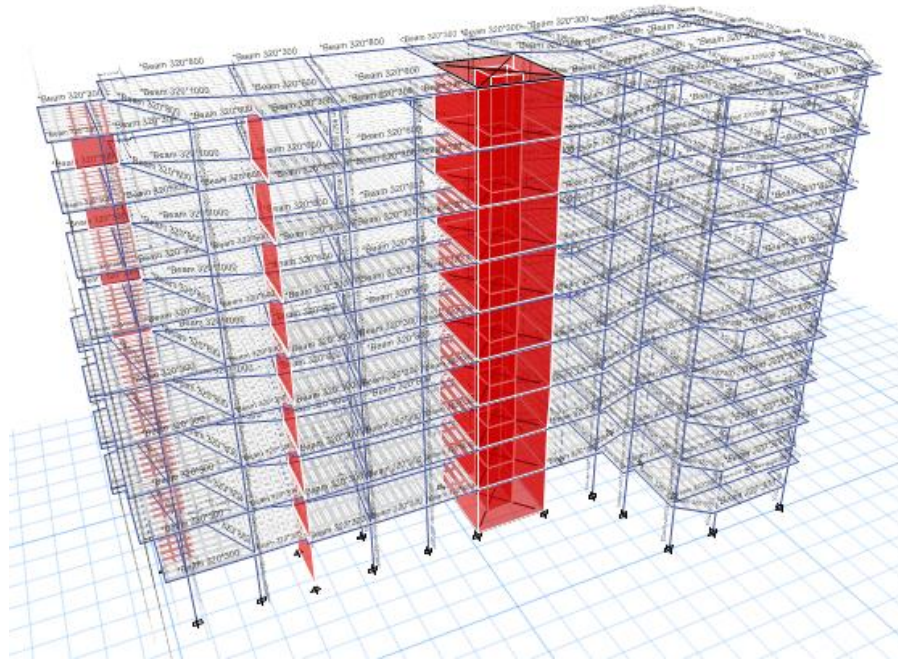
Group	Number	Dimension (m)
1	1,2,3,4,5,12,13,14,15,16,18,19,22,24,25,26	0.8*0.3
2	7,10,11	0.95*0.3
3	6,8,9,17,20,21	1*0.3

# 3 Chapter 3: THREE-DIMENSIONAL ANALYSES

## 3.1 General:

This chapter will demonstrate the definition of materials and sections in the first part, and then defining load patterns and combination, which will be used in Etabs. and counted in the analysis process. In the second part verification checks will take place starting with the main check (compatibility check) to make sure every element connected and the building is a rigid body. Then stress-strain check to ensure loads and dimensions as decided and verifying the inputs in the software. Then internal check comparing manual calculations to sap results to make certain We are on the right track, ending these checks with the deflection one considering structural and non-structural elements.

## 3.2 Structural modeling of the building:



*Figure 3.1 model of the structure in ETABS.*

- **Units:**

The main units are Celsius degree ( $^{\circ}\text{C}$ ) for temperature load, (kilo newton (KN) for internal forces, millimeter (mm) for displacements, (KN.m) for internal moments, meter (m) for lengths, ( $\text{mm}^2$ ) for area of steel, (KN/m) for line uniform loads and (KN/m<sup>2</sup>) for area uniform load.

- **Gridlines:**

Gridlines did not used in the ETABS software model. The structural elements in ETABS software were imported from AutoCAD.

- **Materials:**

Concrete B350 (28MPa) was used for all slabs, beams, columns and walls with properties as shown in Figure 3-2.


Name	Type	E(Mpa)	v	$\gamma$ (KN/m <sup>3</sup> )	Design Strengths
Concrete 28MPa	Concrete	24870.06	0.2	25	$f'_c = 28\text{Mpa}$

**General Data**

Material Name: Concrete

Material Type: Concrete

Directional Symmetry Type: Isotropic

Material Display Color:  Change...

Material Notes: Modify/Show Notes...

---

**Material Weight and Mass**

Specify Weight Density       Specify Mass Density

Weight per Unit Volume: 25 kN/m<sup>3</sup>

Mass per Unit Volume: 2549.29 kg/m<sup>3</sup>

---

**Mechanical Property Data**

Modulus of Elasticity, E: 24855.58 MPa

Poisson's Ratio, U: 0.2

Coefficient of Thermal Expansion, A: 0.000099 1/C

Shear Modulus, G: 10356.49 MPa

---

**Design Property Data**

Modify/Show Material Property Design Data...

---

**Advanced Material Property Data**

Nonlinear Material Data...      Material Damping Properties...

Time Dependent Properties...

---

**Modulus of Rupture for Cracked Deflections**

Program Default (Based on Concrete Slab Design Code)

User Specified

---

**Material Property Design Data**

**Material Name and Type**

Material Name: Concrete

Material Type: Concrete, Isotropic

Grade:

---

**Design Properties for Concrete Materials**

Specified Concrete Compressive Strength, f<sub>c</sub>: 28 MPa

Lightweight Concrete

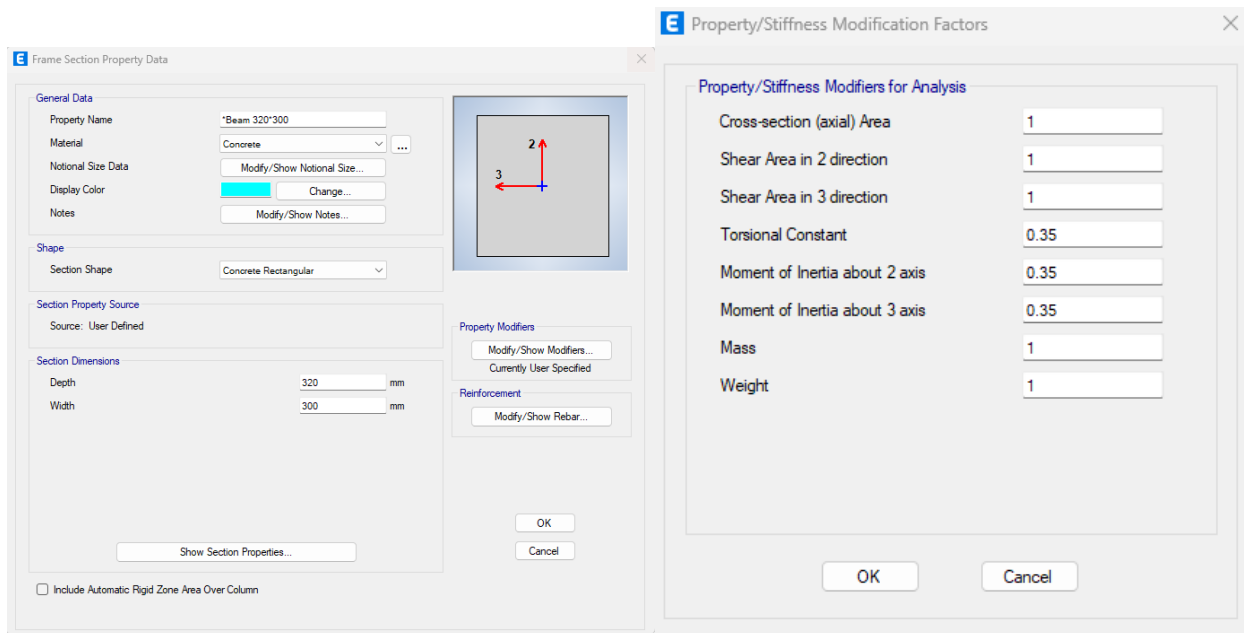
Shear Strength Reduction Factor:

OK      Cancel

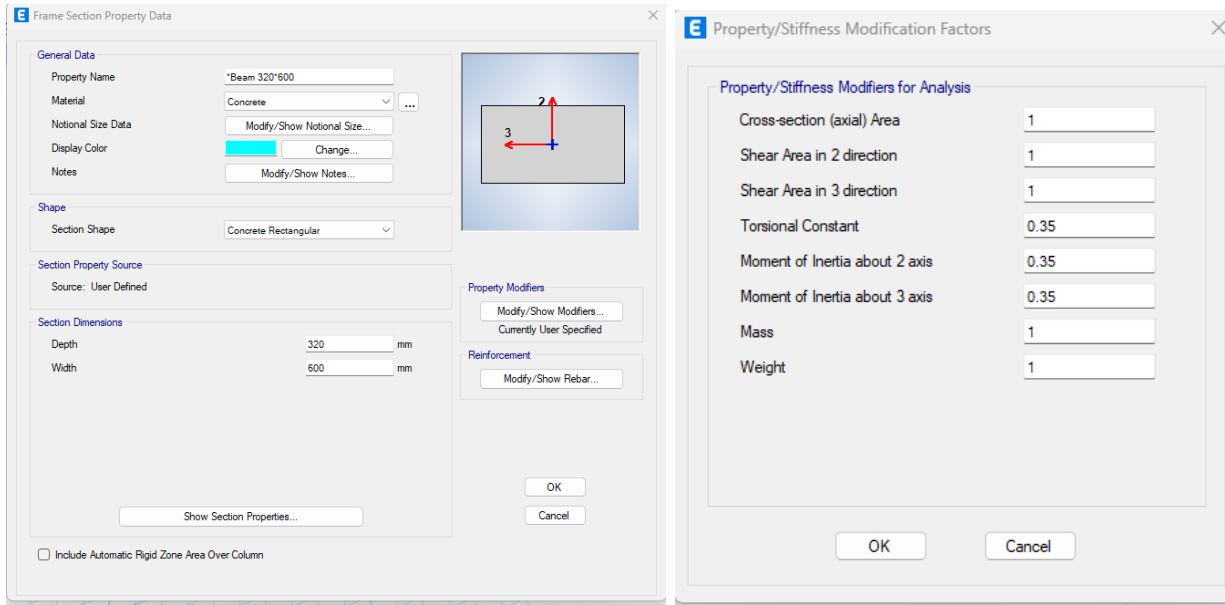
**Figure 3.2 Material properties for all Slabs, beam, columns and walls concrete**

- **Beam's properties: -**

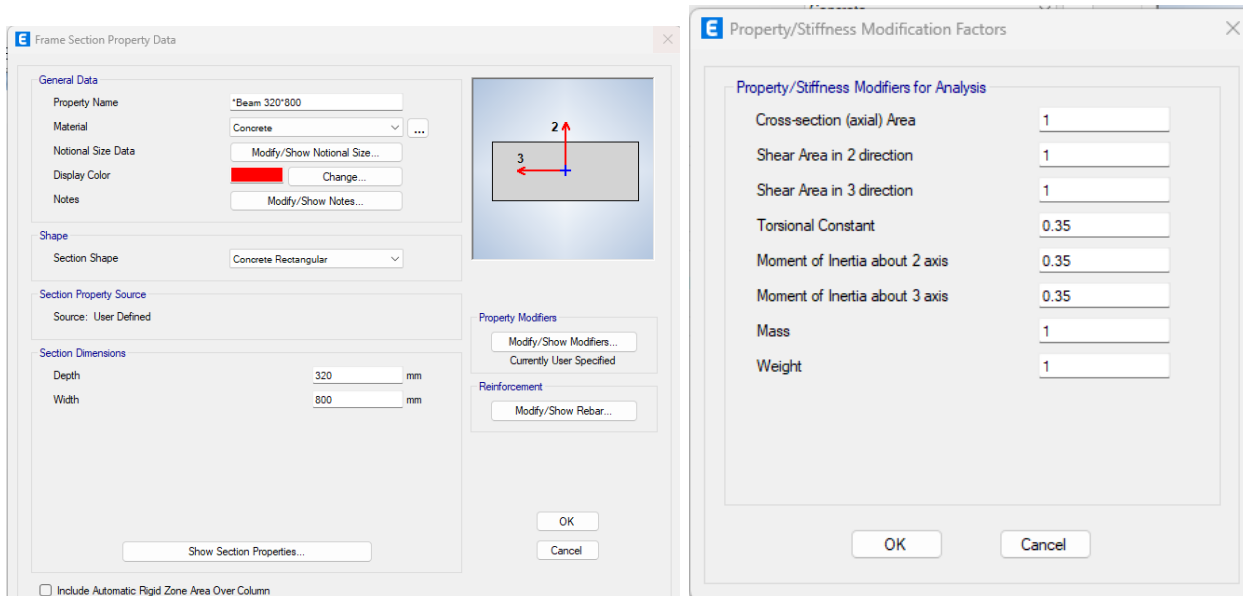
Different sections and groups of beams were defined in ETABS software as mentioned in section 2.3, with bending modifiers equal to 0.35 according to ACI318-14 code.



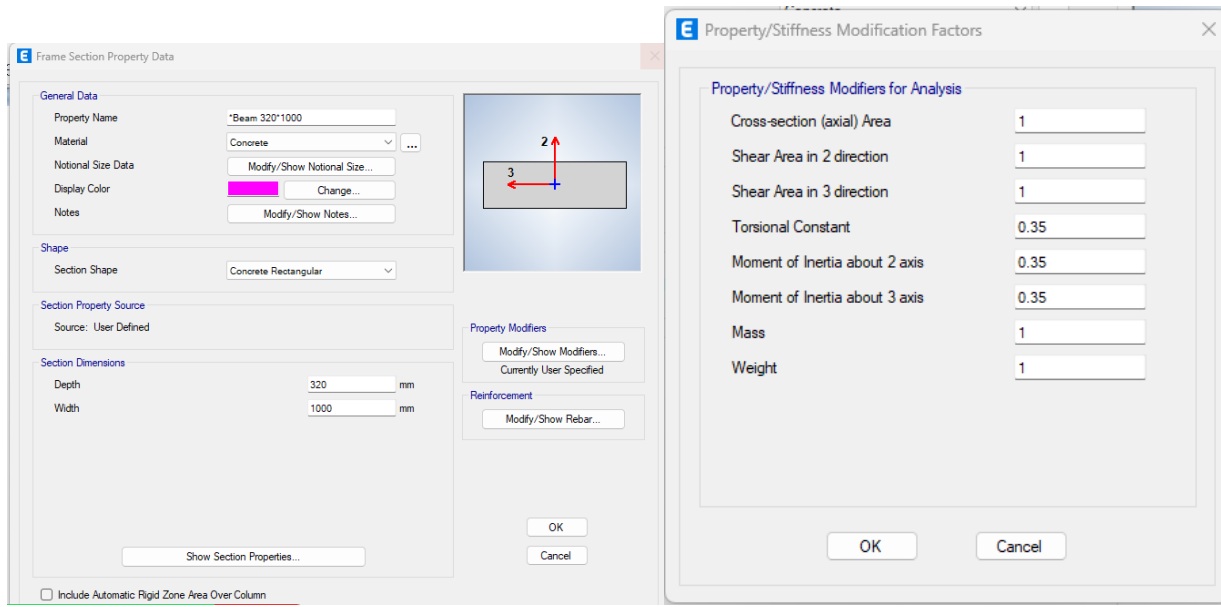
**Figure 3.3** *Figure: Section properties for Beam 320\*300mm*



**Figure 3.4 Section properties for Beam 320\*600mm**



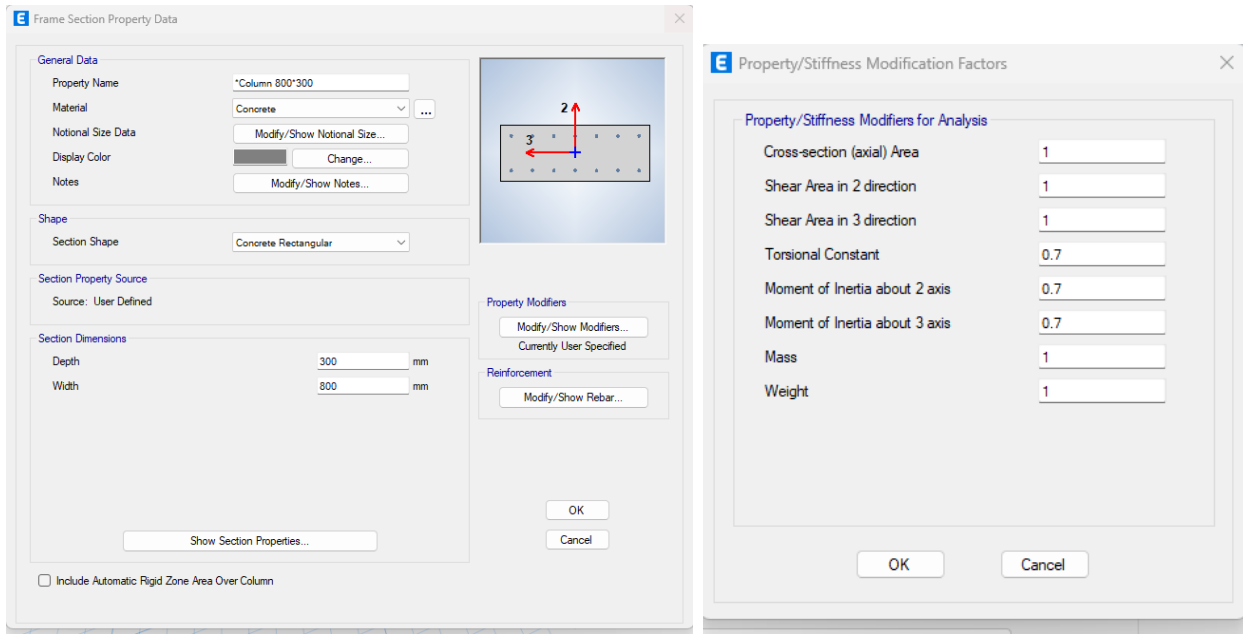
**Figure 3.5 Section properties for Beam 320\*800mm**



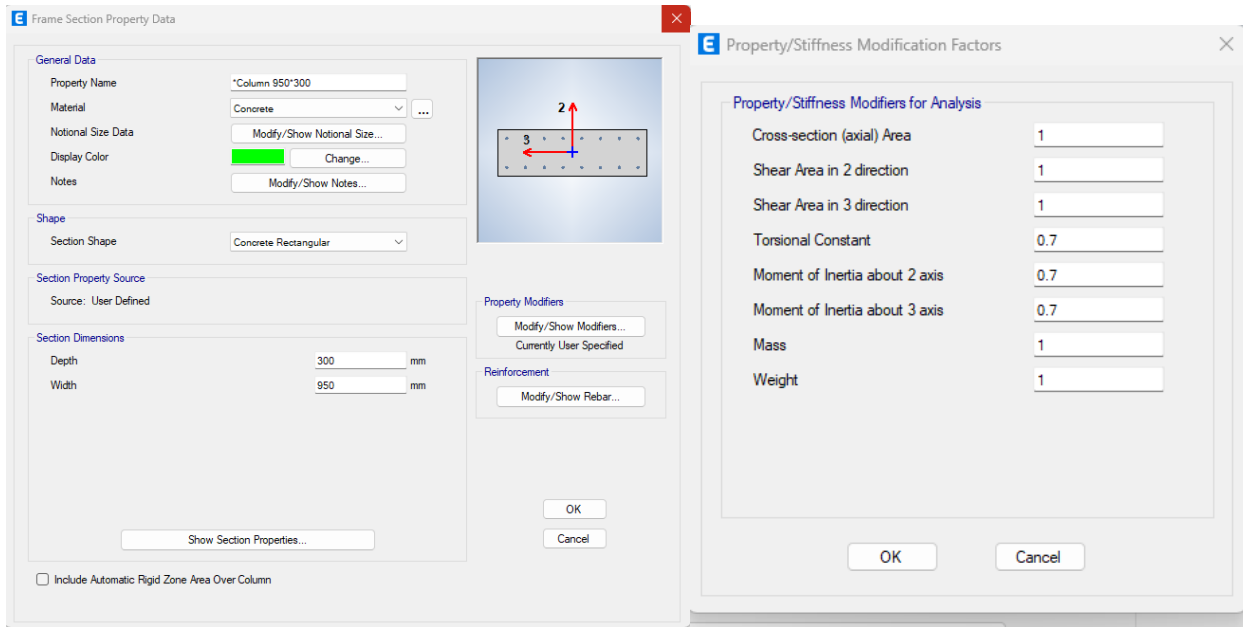
***Figure 3.6 Section properties for Beam 320\*1000mm***

- **Column's properties: -**

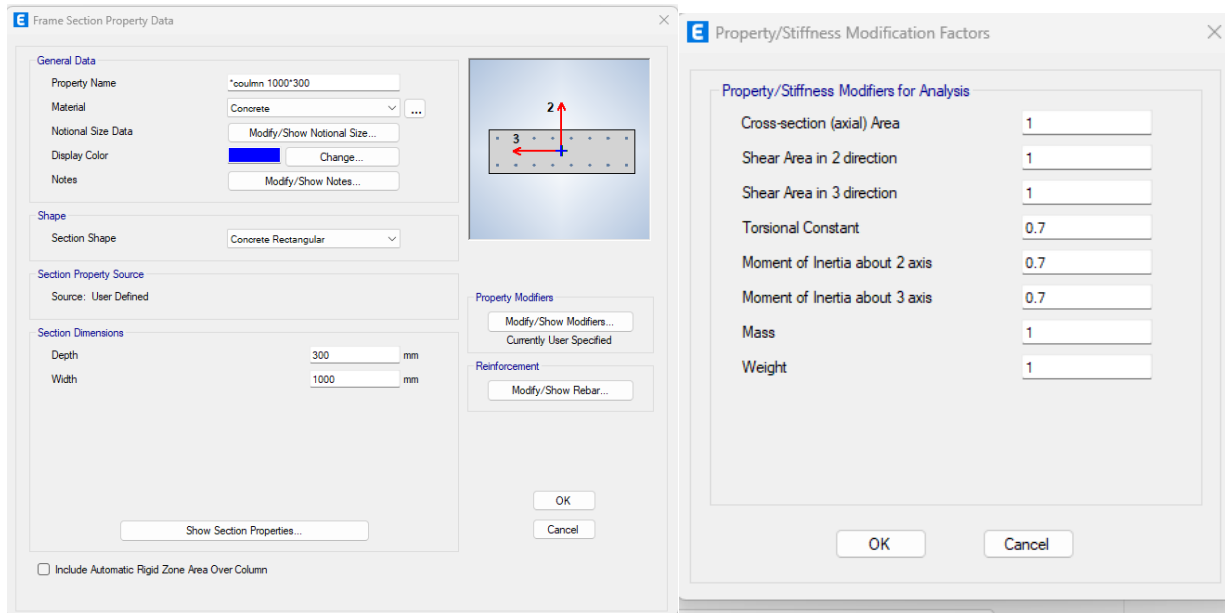
Section of columns were defined in ETABS software as mentioned in section 2.5, with bending modifiers equal to 0.7 according to ACI 318-14 code as shown in Figures: -



**Figure 3.7 Section and Modifiers for columns 800\*300**



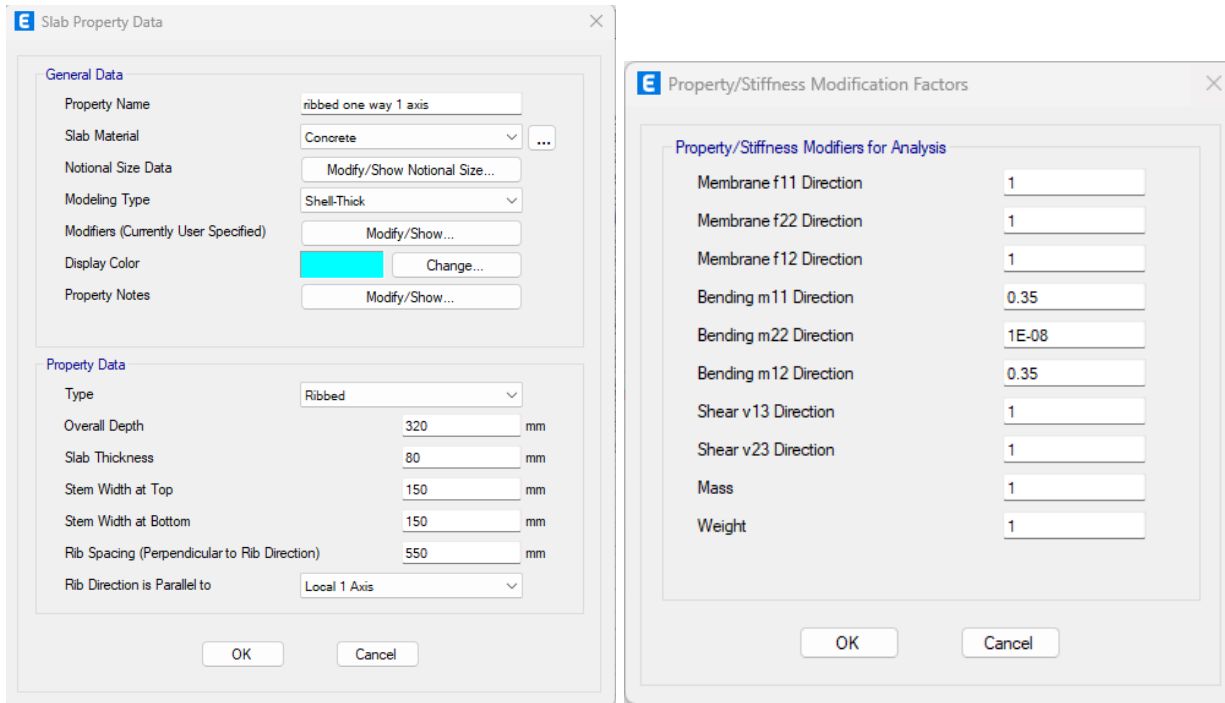
**Figure 3.8 Section and Modifiers for columns 950\*300**



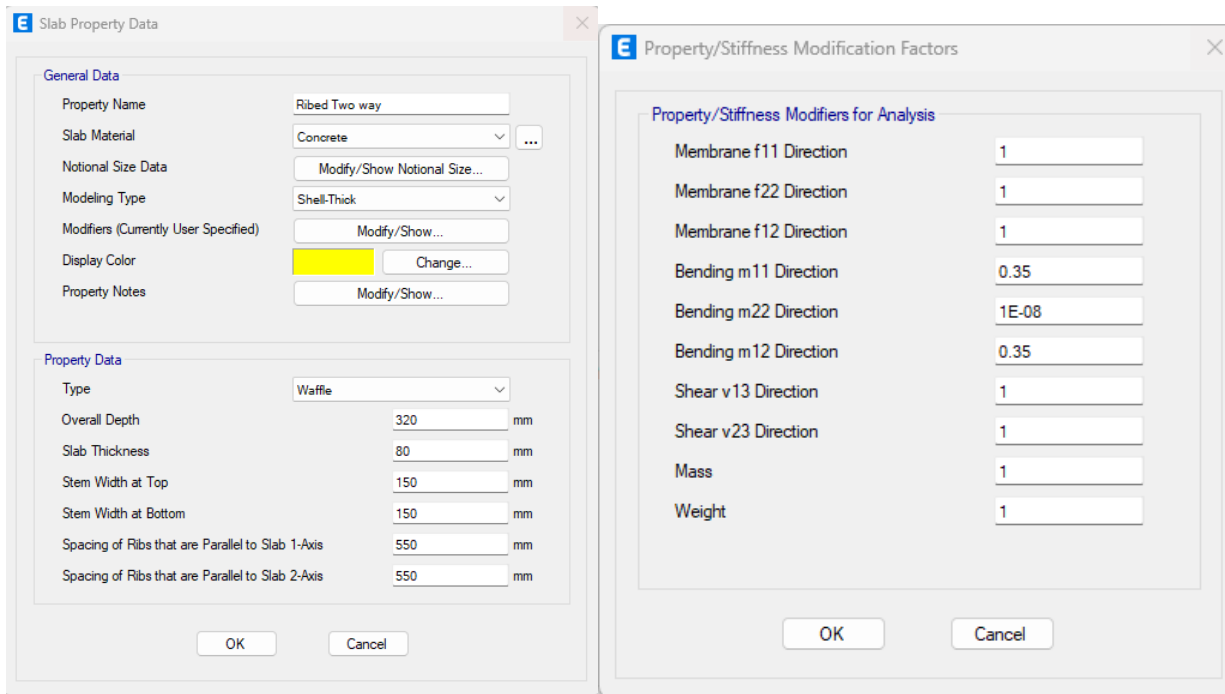
**Figure 3.9 Section and Modifiers for columns 1000\*300**

- **Slab's properties:**

One-way ribbed slab and two-way ribbed slab with thickness 320 mm were defined in ETABS software with bending modifiers equal to 0.35 according to ACI 318-14 code, as shown in Figure 3-10, 3-11.



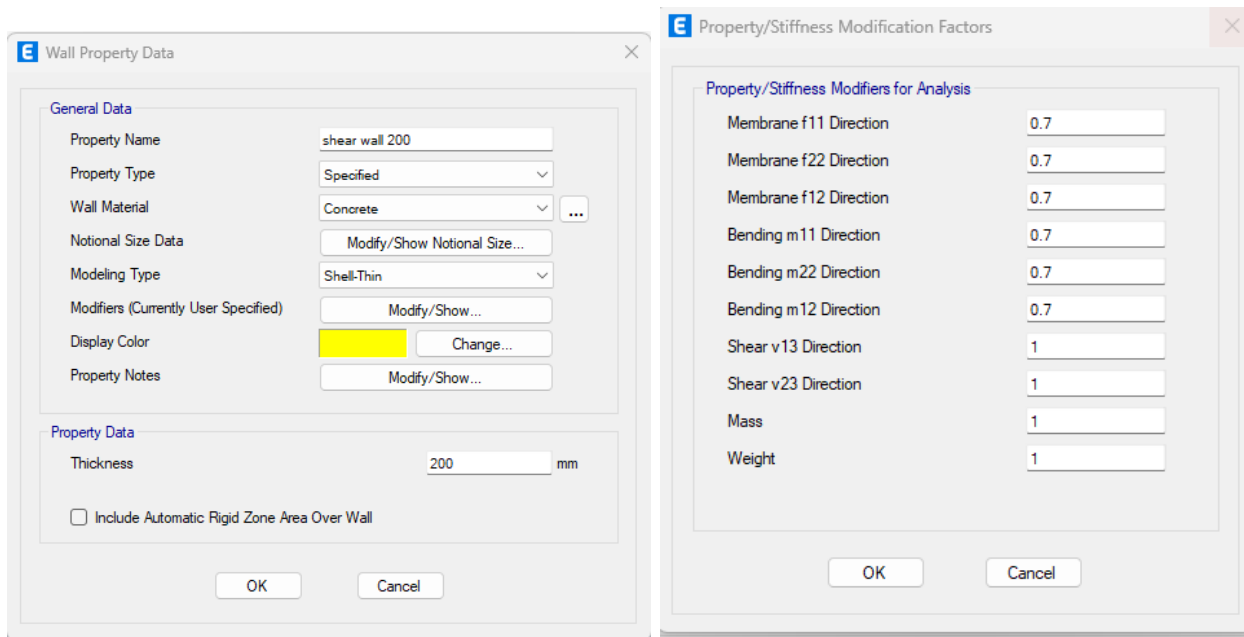
**Figure 3.10 Section properties for one-way ribbed slab 320mm**



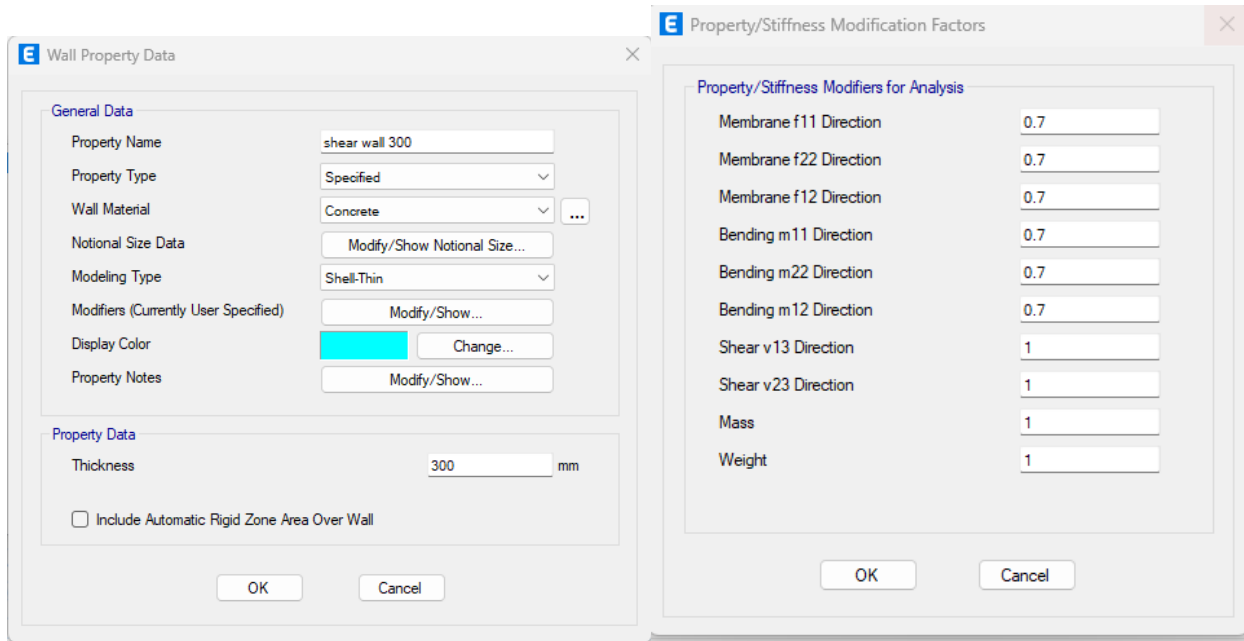
**Figure 3.11 Section properties for two-way ribbed slab 320mm**

- **Wall properties:**

Walls around squash hall with thickness 300mm and other walls with thickness 200mm in ETABS software with bending and membrane modifiers equal to 0.7 according to ACI 318- 14code, as shown in figures below.



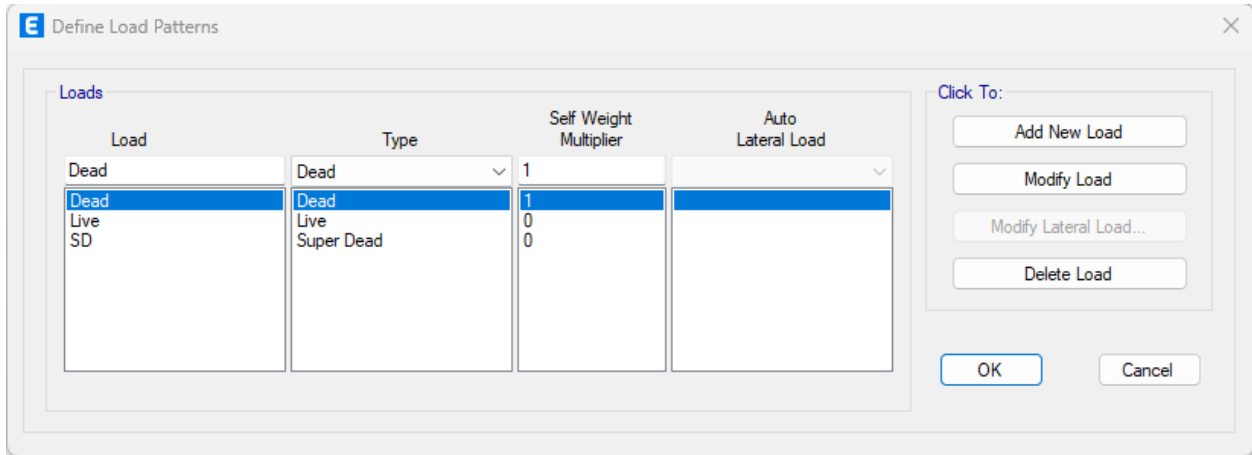
***Figure 3.12 Section properties for wall 200mm***



***Figure 3.13 Section properties for wall 300mm***

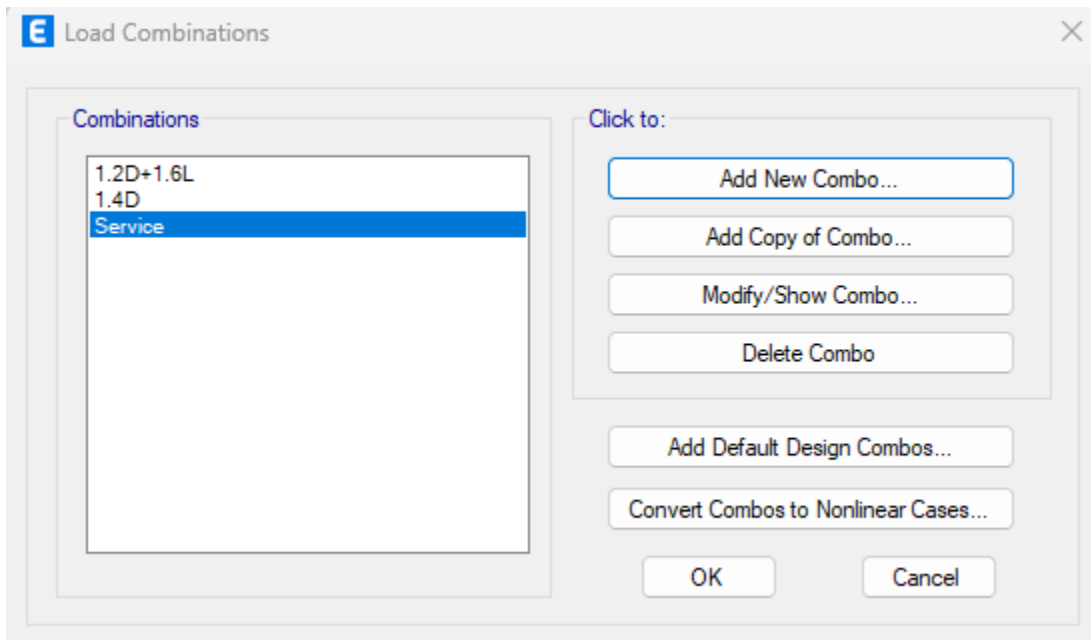
- **Load definition: -**

Load Patterns were identified for all types of loads affecting on the structure which is dead load (D), Superimposed dead load (SD), Live load (L), as shown in Figure 3-14:



*Figure 3.14 Load patterns*

- **Load combination:**



*Figure 3.15 Load combination*

- **Design specifications: -**

The design code selected in ETABS software is ACI 318-14code, and other specifications for

design as shown in Figure 3-16. Also, the framing type for all structural elements was

defined as sway ordinary, as shown in Figure 3-17.

	Item	Value
01	Design Code	ACI 318-14
02	Multi-Response Case Design	Step-by-Step - All
03	Number of Interaction Curves	24
04	Number of Interaction Points	11
05	Consider Minimum Eccentricity?	Yes
06	Design for B/C Capacity Ratio?	Yes
07	Ignore Beneficial $P_u$ for Beam Design?	No
08	Seismic Design Category	D
09	Design System $\Omega_0$	2
10	Design System $\rho$	1
11	Design System $S_{ds}$	0.5
12	Consider ICC-ES ESR-2107	No
13	$\Phi$ (Tension Controlled)	0.9
14	$\Phi$ (Compression Controlled Tied)	0.65
15	$\Phi$ (Compression Controlled Spiral)	0.75
16	$\Phi$ (Shear and/or Torsion)	0.75
17	$\Phi$ (Shear Seismic)	0.6
18	$\Phi$ (Joint Shear)	0.85

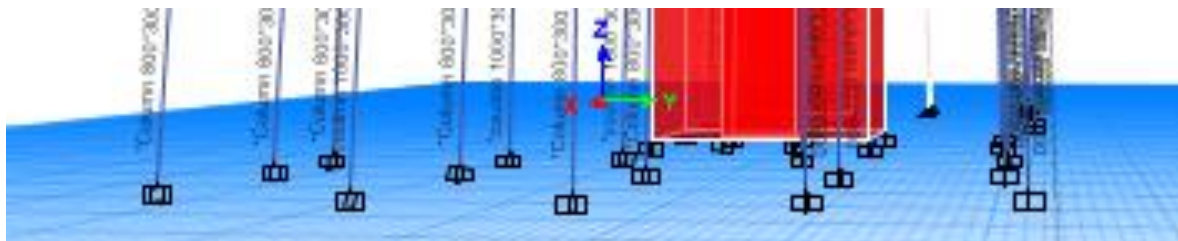
***Figure 3.16 Concrete frame design preferences for ACI 318-14***

	Item	Value
01	Current Design Section	Varies
02	Framing Type	Varies
03	Live Load Reduction Factor	Varies
04	Unbraced Length Ratio (Major)	Varies
05	Unbraced Length Ratio (Minor)	Varies
06	Effective Length Factor (K Major)	1
07	Effective Length Factor (K Minor)	1
08	Moment Coefficient (Cm Major)	1
09	Moment Coefficient (Cm Minor)	1
10	NonSway Moment Factor (Dns Major)	1
11	NonSway Moment Factor (Dns Minor)	1
12	Sway Moment Factor (Ds Major)	1
13	Sway Moment Factor (Ds Minor)	1
14	Consider Minimum Eccentricity?	Yes
15	Seismic Detailing in CSI/CAD?	Varies

*Figure 3.17 Concrete frame design overwrites for ACI 318-14*

- **Supports: -**

The supports restraints were assumed to be fixed supports as shown in Figure 3-18:



*Figure 3.18 Supports of the structure*

### 3.3 Evaluation of preliminary design:

In the 3D models, the preliminary dimensions were generally suitable. The final sections for all beams were shown in Table 3-1:

*Table 3-1 Final beams dimension*

<b>Ribbed Slab</b>	<b>Final section</b>
	Beam 320×300 mm
	Beam 320×600 mm
	Beam 320×800 mm
	Beam 320×1000 mm

Also, the new dimensions for columns are shown in **Table 3-2**.

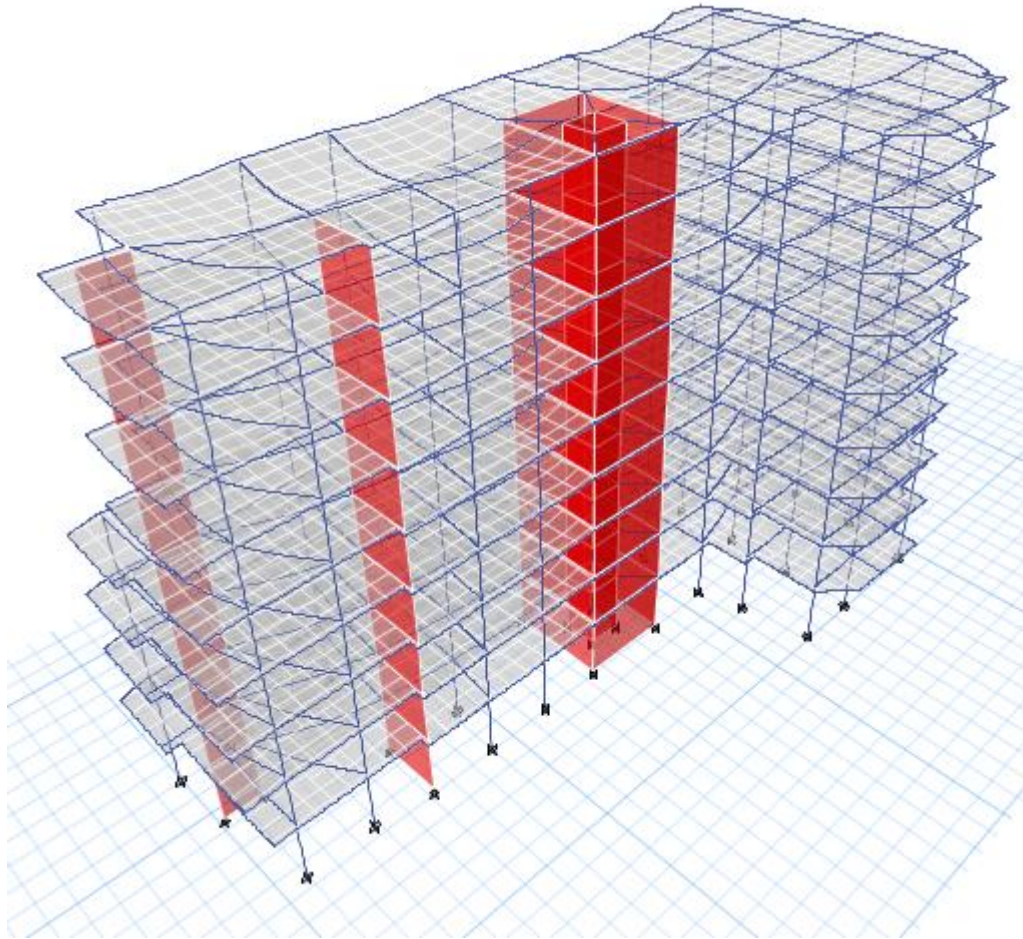
*Table 3-2 Final columns dimension*

<b>All Floors</b>	
	Column 800*300
	Column 950*300
Column 1000*300	

### 3.4 Verification of structural analysis:

#### 3.4.1 Compatibility of the structure:

To achieve a compatible structure, all structural elements were drawn center to center. The meshing for all slabs was performed automatically by ETABS software, as a result, the structure was moving as one unit and all structural elements were attached together very well.



*Figure 3.19 Compatibility of the structure*

### 3.4.2 Equilibrium of the structure:

- **Slabs:**

Loads on slabs were computed by multiplying the area of each slab by the load per meter square on each one. The area and loads for each slab were shown in Table 3-3.

*Table 3-3 Slabs areas and loads*

<b>Slabs</b>	<b>Area (m<sup>2</sup>) One- way</b>	<b>Area (m<sup>2</sup>) two- way</b>	<b>Own weight (KN/m<sup>2</sup>) One- way</b>	<b>Own weight (KN/m<sup>2</sup>) two-way</b>	<b>Live load (KN/m<sup>2</sup>)</b>
GF	277.11	176.2	3.6	4.83	1.92
F1	265.39	169.91	3.6	4.83	1.92
F2	265.39	169.91	3.6	4.83	1.92
F3	265.39	169.91	3.6	4.83	1.92
F4	265.39	169.91	3.6	4.83	1.92
F5	251.3	170.65	3.6	4.83	1.92
F6	251.3	170.65	3.6	4.83	1.92
F7	251.3	170.65	3.6	4.83	1.92
F8	251.3	170.65	3.6	4.83	1.92

*Table 3-4 Total load on different slabs*

<b>Slabs</b>	<b>Own weight Total (KN)</b>	<b>Live load (KN/m<sup>2</sup>)</b>
GF	1848.642	870.35
F1	1776.07	835.776
F2	1776.07	835.776
F3	1776.07	835.776
F4	1776.07	835.776
F5	1728.92	810.14
F6	1728.92	810.14
F7	1728.92	810.14
F8	1728.92	810.14

**The total weight of slabs = 15868.6 KN**

**The total live load = 7454 KN**

- **Columns:**

**Weight of columns =**

$$(3 \times 0.95 \times 0.3 \times 27.4 \times 25) = 585.68 \text{ KN}$$

$$(7 \times 1 \times 0.3 \times 27.4 \times 25) = 1438.5 \text{ KN}$$

$$(15 \times 0.8 \times 0.3 \times 27.4 \times 25) = 2466 \text{ KN}$$

$$(0.8 \times 0.3 \times 15.4 \times 25) = 92.4$$

**The Total weight of columns = 4582.58 KN**

- **Shear wall:**

Two type of walls was used in the structure and the own weight for each one of them was computed by multiplying the weight per one meter square of wall by the total area of the wall.

*Table 3-5 Table: Total weight of walls*

<b>Wall thickness</b>	<b>Wall area (m2)</b>	<b>Wall weight (kN)</b>
<b>300mm thickness</b>	<b>636.228</b>	<b>4771.71</b>
<b>200mm thickness</b>	<b>310.168</b>	<b>1550.84</b>
<b>Total</b>		<b>6322.55</b>

- **Beams:**

Different sections of beams were used in the structure, and the own weight for each of them was computed by multiplying the weight per one meter of beam by the total length of beams considering the weight modifiers for each beam as follows:

*Table 3-6 Total weight of beams*

	<b>Total length (m)</b>	<b>Total own weight (kN)</b>
Beam 320×300 mm	<b>1546.86</b>	<b>3712.5</b>
Beam 320×600 mm	<b>646.96</b>	<b>3105.4</b>
Beam 320×800 mm	<b>306.12</b>	<b>1959.17</b>
Beam 320×1000 mm	<b>61.29</b>	<b>498.32</b>
<b>Total dead load</b>		<b>9275</b>

$$\begin{aligned} \text{Total Dead Load} &= 15868.6 + 4582.58 + 6322.5 + 9275 \\ &= 36048.68 \text{ KN} \end{aligned}$$

$$\text{Live Load} = 7454 \text{ KN}$$

The difference percentage in the dead loads and live loads were acceptable as shown in Table 3-7. However, the difference percentage in the dead loads equal to 0.75%, because ETABS software calculates the own weight for all beams as clear span while the manual calculations for all beams were computed as center to center.

*Table 3-7 Equilibrium check for gravity loads*

<b>Load</b>	<b>Manual calculations</b>	<b>ETABS calculations</b>	<b>Difference percentage</b>
Dead load	36048.68	35575.083	1.33%
Live load	7454	7405.6915	0.64%

### **3.4.3 Verification of internal forces:**

The stress strain check was made for one span in frame within the slab, one beam and one column. The values of moment for the selected span in the frame were obtained using ETABS software.

The first value of negative moment in the beam using ETABS software was shown in Figure 3-20.

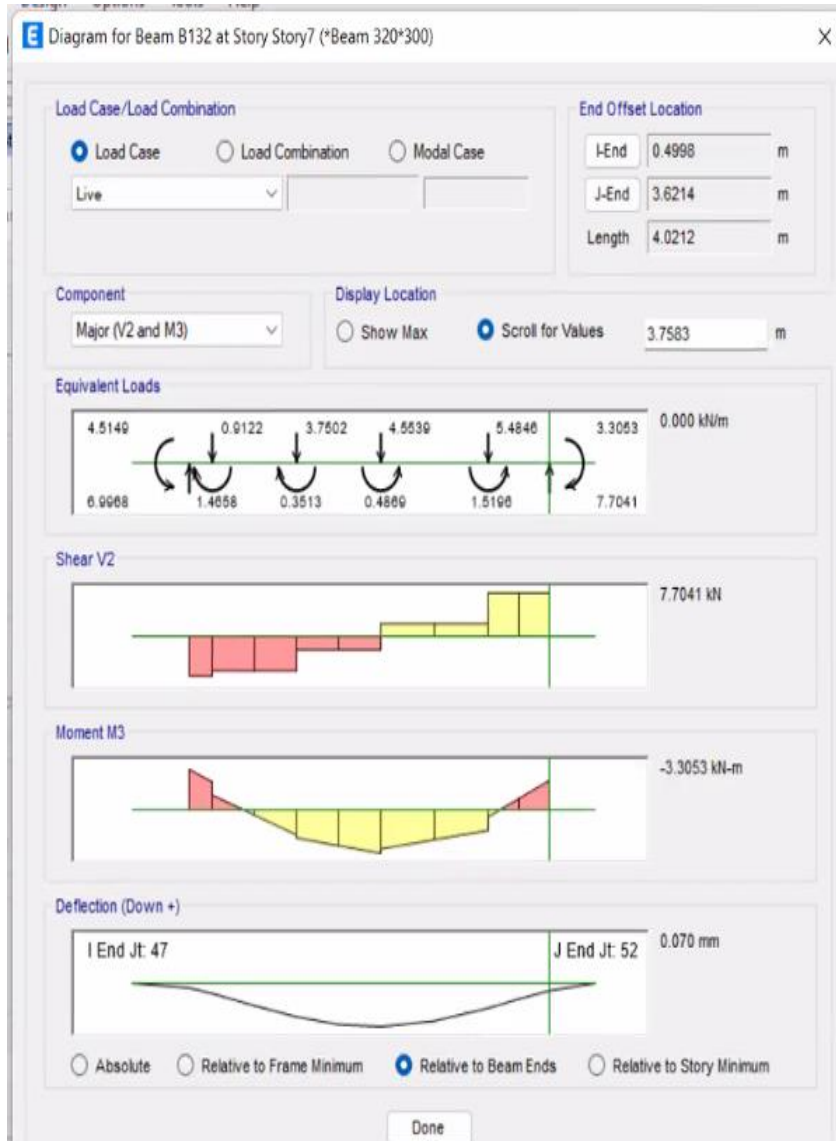


Figure 3.20 The first value of negative moment using ETABS software

The second value of negative moment in the beam using ETABS software was shown in Figure 3-21

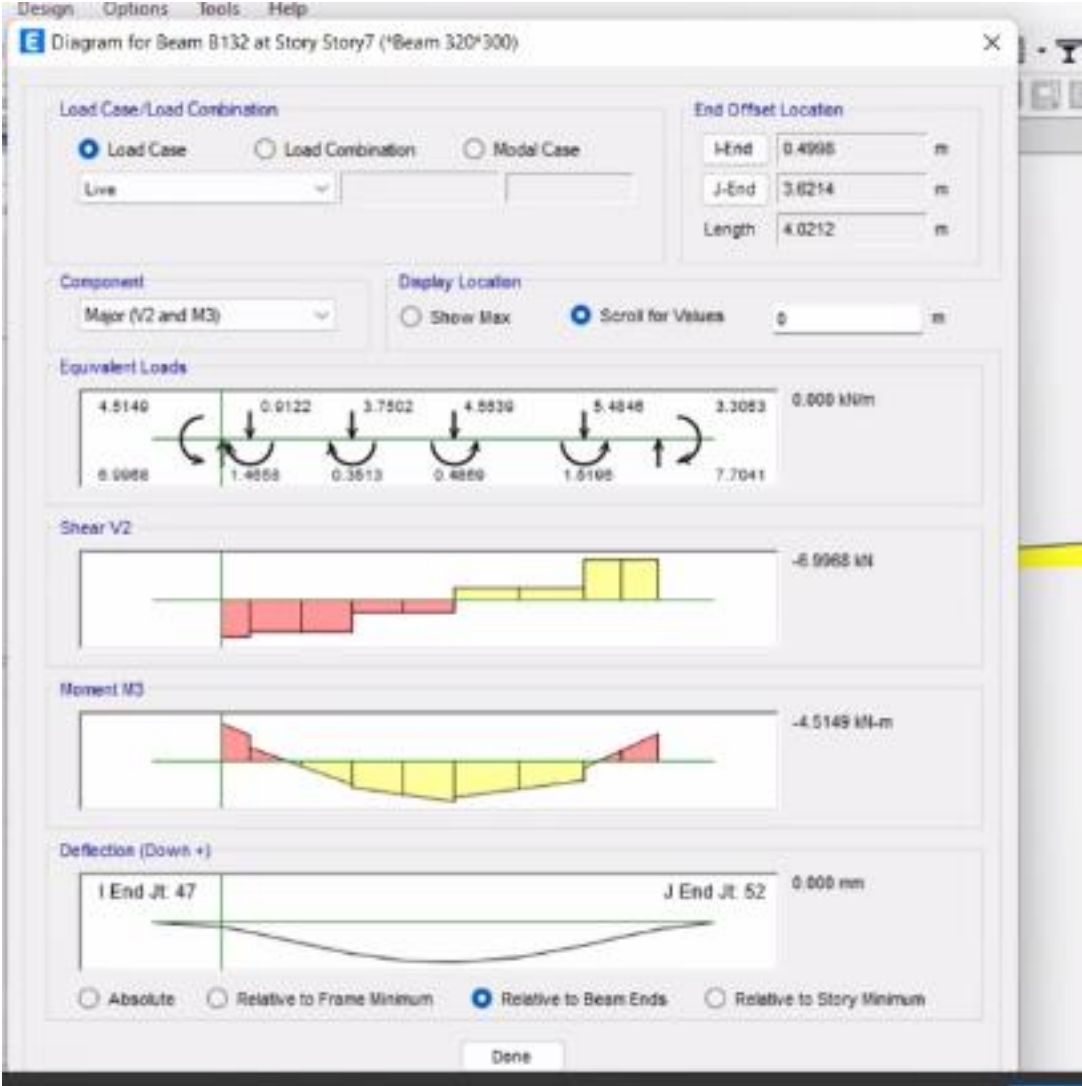


Figure 3.21 The second value of negative moment using ETABS software

The value of positive moment in the beam using ETABS software was shown in Figure 3-22.

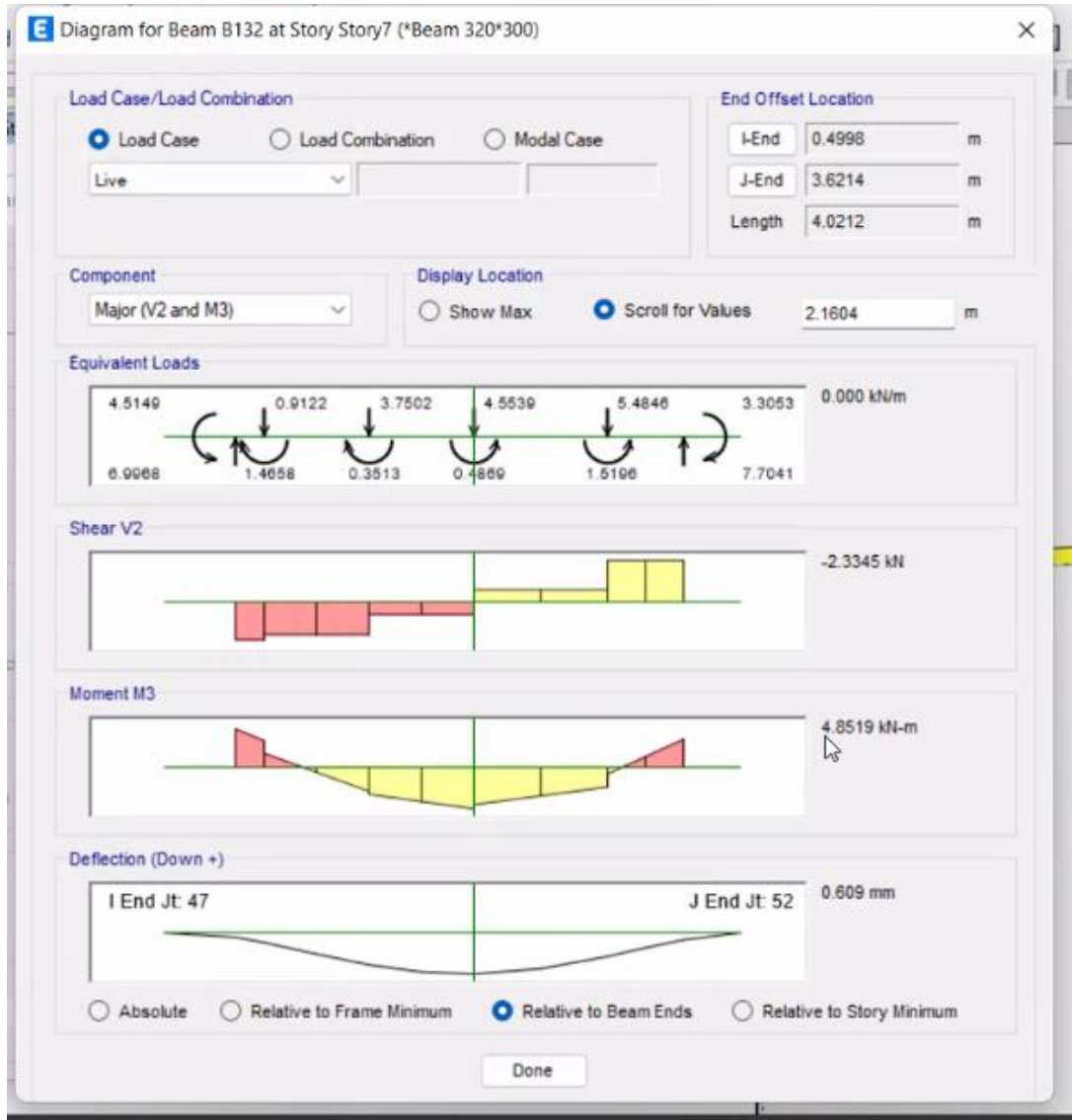


Figure 3.22 The value of positive moment using ETABS software

Moment from Etabs software=  $(3.305+4.515)/2 +4.852=8.76$  KN.m

Moment from manual calculation=

$$(3.8/2 + 3.9/2) * 1.92 = 7.392$$

$$M= WL^2/8 = (7.392*3.12^2)/8= 8.99$$
 KN.m

$$\text{Error}= 8.99-8.76/8.99 =2.6 \%$$

From Load combination

The value of negative moment in the span using ETABS software was shown in Figure 3-23.

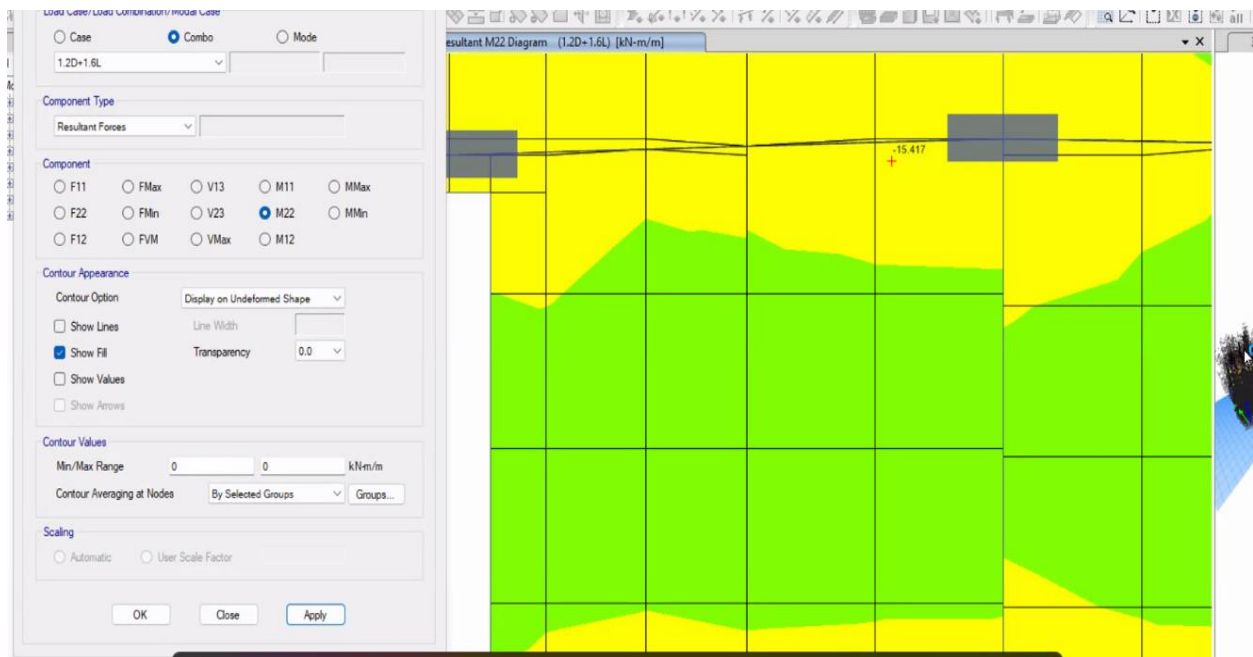
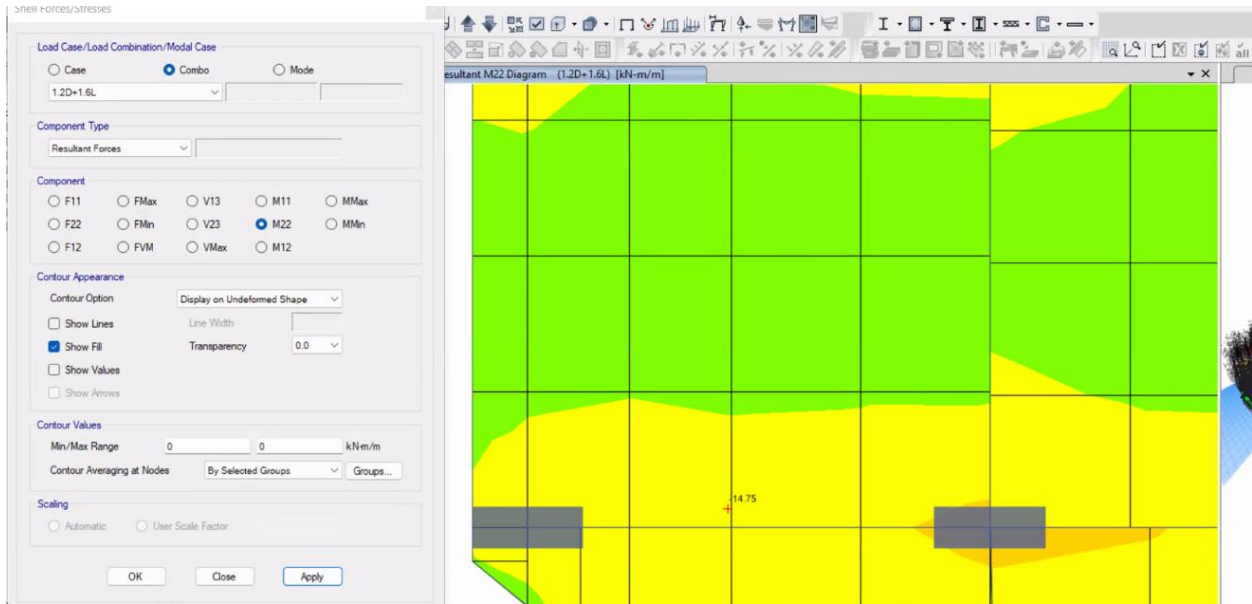


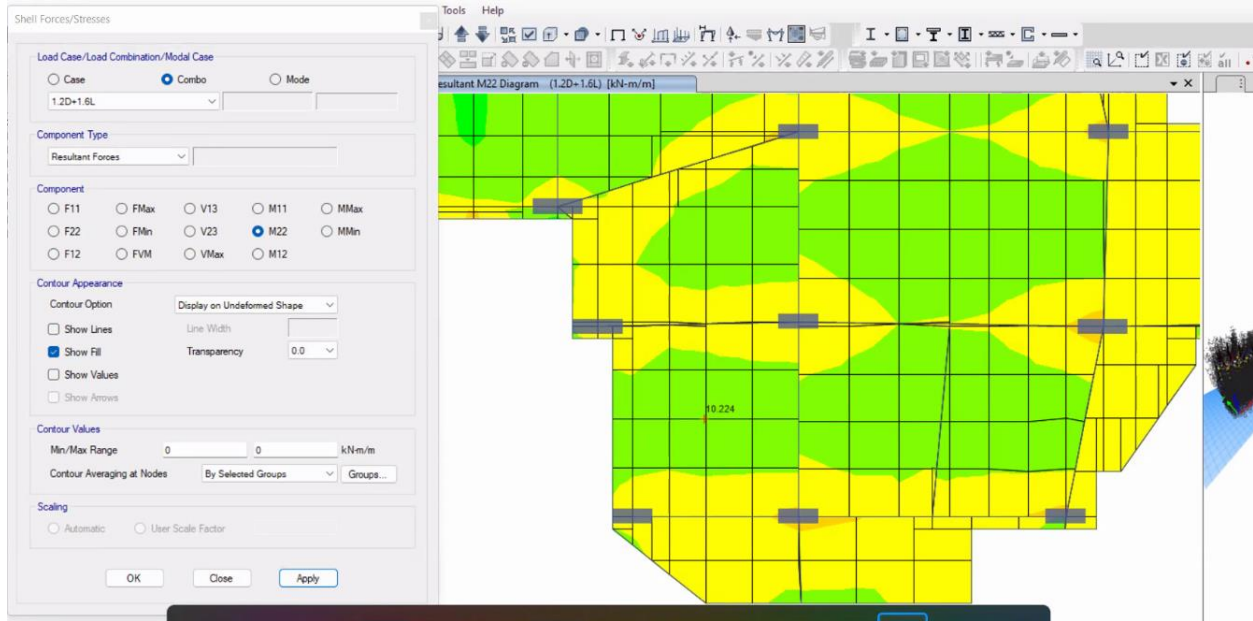
Figure 3.23 negative moment in the span using ETABS software

The value of negative moment in the span using ETABS software was shown in Figure 3-24.



*Figure 3.24 negative moment in the span using ETABS software*

The value of positive moment in the span using ETABS software was shown in Figure 3-25.



*Figure 3.25 of positive moment in the span using ETABS software*

Form manual:

$$14.9 * 3.6^2 / 8 = 24.138 \text{ KN.m}$$

From ETABS:

$$(14.75 + 15.4) / 2 + 10.2 = 25.27 \text{ KN.m}$$

$$\text{Error} = 24.138 - 25.27 / 24.138 = 4.7\%$$

## 4 Chapter 4

### 4.1 General

This chapter is devoted for seismic analysis of the structure. The requirements of strength, stability and serviceability are checked along with ASCE 7-10 code. The seismicity of the site and structure and the seismic force-resisting system previously discussed in section 1.7.

### 4.2 Modeling:

Design factors for Special reinforced concrete system of special shear walls of resisting 100% of the seismic force were discussed in section 1-7 and they are repeated here in Table 4-1 for convenience.

*Table 4-1 Response Spectrum Modeling parameters*

Response Modification Factor $R$	4
Overstrength Factor $\Omega_0$	2
Deflection Amplification Factor $C_d$	5
Importance Factor $I_e$	1

Non-symmetrical structure (where center of rigidity does not coincide with center of mass) requires dynamic analysis procedure based on modal analysis, effective masses and response spectrum accelerations. Table 4-2 below shows the modal participating mass ratios for the first 3 modes. Note that, the first mode is a twisting mode.

Table 4-2 Modal Participating Mass Ratios as obtained from ETABS

Case	Mode	Period sec	UX	UY	UZ	SumUX	SumUY	SumUZ	RX
Modal	1	0.907	0.4051	0.0006	0	0.4051	0.0006	0	0.0006
Modal	2	0.82	0.0488	0.5562	0	0.4539	0.5568	0	0.2984
Modal	3	0.794	0.208	0.1084	0	0.6619	0.6652	0	0.0533
Modal	4	0.277	0.0127	0.0062	0	0.6746	0.6714	0	0.0084
Modal	5	0.205	0.0083	0.1788	0	0.6828	0.8502	0	0.2926
Modal	6	0.184	0.1904	0.005	0	0.8733	0.8552	0	0.0079
Modal	7	0.165	0.0022	0.0074	0	0.8754	0.8626	0	0.0162
Modal	8	0.125	0	0.0013	0	0.8754	0.864	0	0.0007
Modal	9	0.119	0.0004	0.0047	0	0.8758	0.8687	0	0.0124
Modal	10	0.112	0.0055	0.0469	0	0.8813	0.9156	0	0.1016
Modal	11	0.096	0.0018	0.0142	0	0.8831	0.9298	0	0.0233
Modal	12	0.093	0.0004	0.0043	0	0.8835	0.9341	0	0.0106
Modal	13	0.091	0.0641	0.0013	0	0.9478	0.9354	0	0.0034
Modal	14	0.08	0.0027	0.0195	0	0.9503	0.9549	0	0.0523
Modal	15	0.077	2.742E-06	0.002	0	0.9533	0.9569	0	0.0054

### 4.2.1 Response Spectrum Definition:

Response spectrum are established based on the parameters disused in section 1.7 as shown:

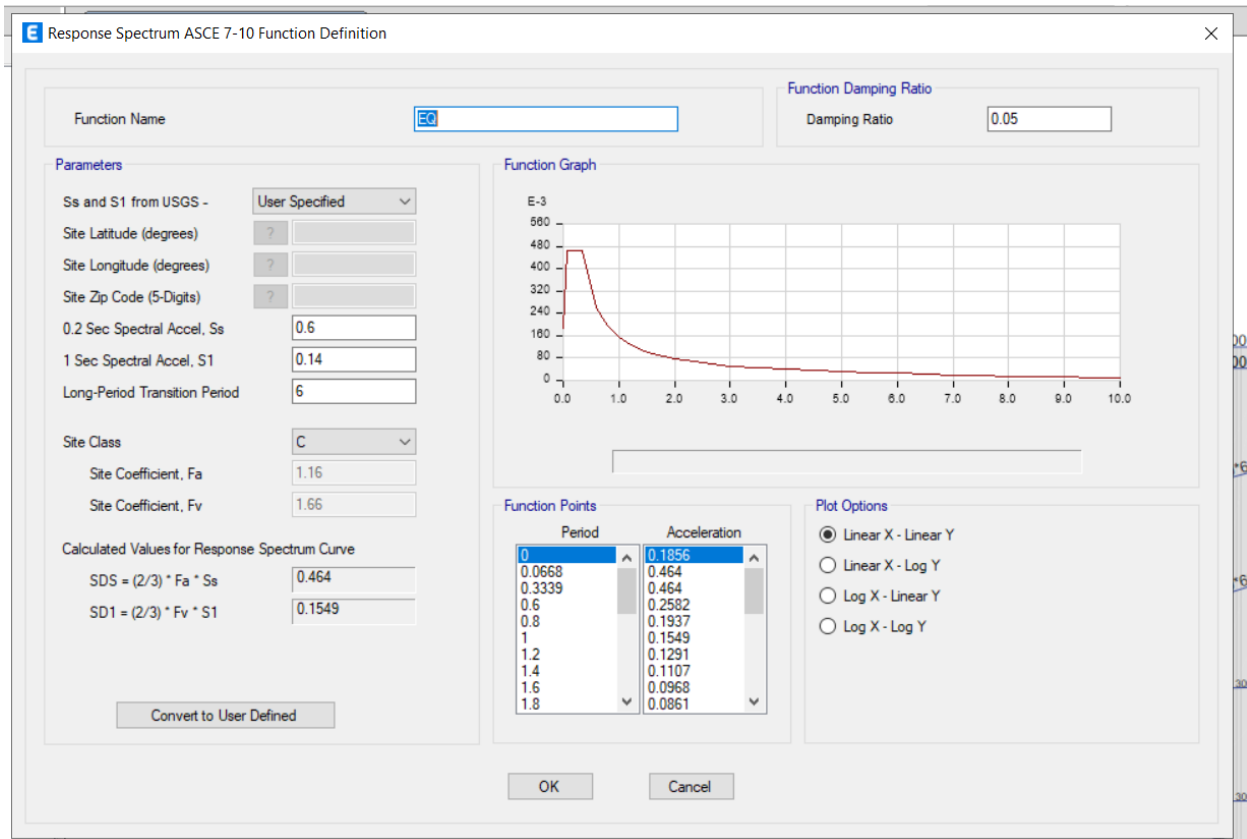


Figure 4.1 Response Spectrum Definition from ETABS

Two load cases are defined as shown in Figure 4-2 and Figure 4-3. Case one includes the response spectrum function applied in X direction and case two includes response spectrum function applied in Y direction.

CQC method is implemented in combining the resulting modes. Note that, scale factor is used to account for (R) and (I) and it is equal to:

$$\text{Scale factor} = (g * 1)/R \dots\dots\dots (4-1)$$

Where g is the gravity acceleration (mm / (s ^ 2));

$$\text{Scale factor} = (9810 * 1) / 4 = 2451.75$$

5 % of damping is generally assumed for vibration of concrete structures, also, 5% of diaphragm eccentricity is initially assumed in both directions. This eccentricity shall be amplified in some cases of rotational irregularity.

**E** Load Case Data ×

**General**

Load Case Name: EDx Design...

Load Case Type: Response Spectrum Notes...

Mass Source: Previous (MASS source)

Analysis Model: Default

**Loads Applied**

Load Type	Load Name	Function	Scale Factor
Acceleration	U1	EQ	2451.75
Acceleration	U2	EQ	817.25

Add  
Delete  
 Advanced

**Other Parameters**

Modal Load Case: Modal

Modal Combination Method: CQC

Include Rigid Response

Rigid Frequency, f1:

Rigid Frequency, f2:

Periodic + Rigid Type:

Earthquake Duration, td:

Directional Combination Type: Absolute

Absolute Directional Combination Scale Factor: 1

Modal Damping: Constant at 0.05 Modify/Show...

Diaphragm Eccentricity: 0.05 for All Diaphragms Modify/Show...

OK
Cancel

*Figure 4.2 Load case in X direction*

**E** Load Case Data ✕

**General**

Load Case Name: EDy Design...

Load Case Type: Response Spectrum Notes...

Mass Source: Previous (MASS source)

Analysis Model: Default

**Loads Applied**

Load Type	Load Name	Function	Scale Factor
Acceleration	U2	EQ	2451.75
Acceleration	U1	EQ	817.25

Add  
Delete  
 Advanced

**Other Parameters**

Modal Load Case: Modal

Modal Combination Method: CQC

Include Rigid Response

Rigid Frequency, f1:

Rigid Frequency, f2:

Periodic + Rigid Type:

Earthquake Duration, td:

Directional Combination Type: Absolute

Absolute Directional Combination Scale Factor: 1

Modal Damping: Constant at 0.05 Modify/Show...

Diaphragm Eccentricity: 0.05 for All Diaphragms Modify/Show...

OK Cancel

*Figure 4.3 Load Case in Y direction*

Mass source include the masses used in model analysis and they include Dead load, Superimposed dead load and 25% of the Live load.

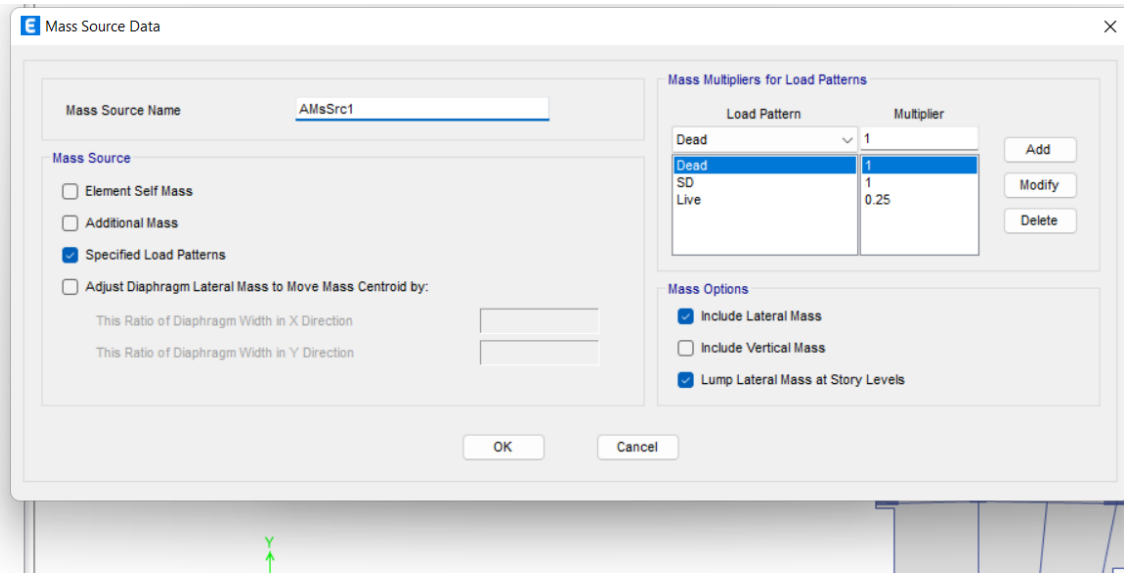


Figure 4.4 Mass Source definition

Concrete slabs are often semi-rigid and are defined as semi-rigid diaphragm

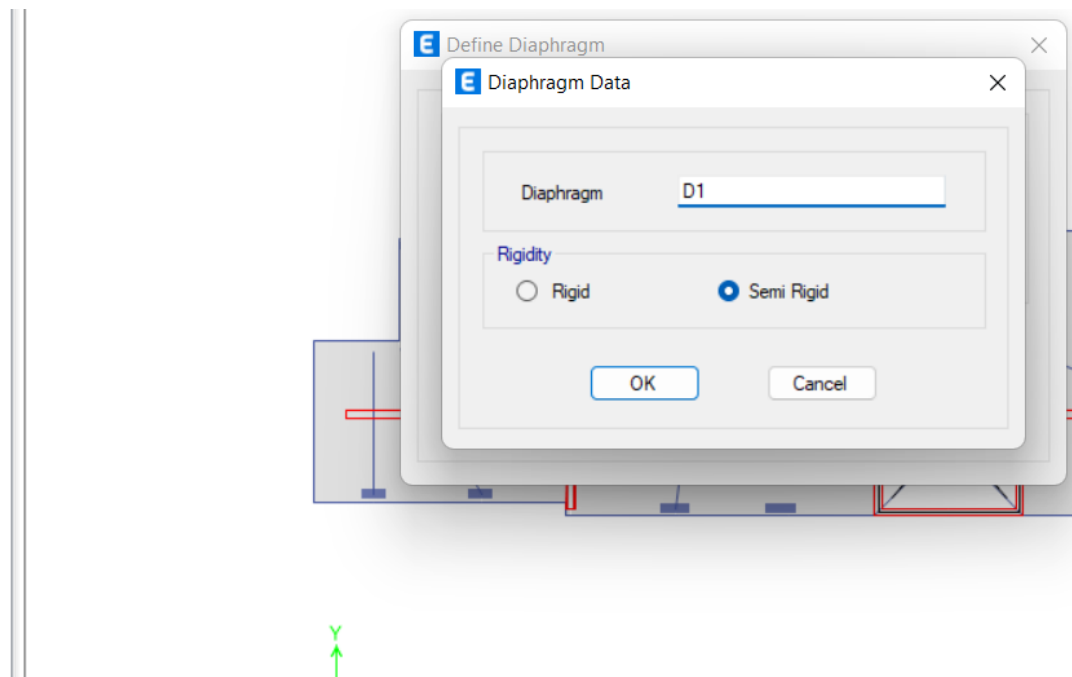


Figure 4.5 Diaphragms definitions

## 4.3 Design checks:

### 4.3.1 General:

Several design checks are required to insure strength, stability and serviceability of the structure. These checks are base shear check, P-delta check, and drift check. These checks are evaluated in the following sections.

### 4.3.2 Base shear check:

Table 4-3 shows the base shear computed using response spectrum.

*Table 4-3 Base shear results for response spectrum from as obtained from ETABS:*

Response spectrum	$F_x$	$F_y$
$ED_x$	2837.899	3409.58
$EQ_x$	2847.54	3382.65

Computation of base shear manually:

$$V = C_s \times W$$

Where  $C_s$  = Seismic Response Coefficient.

$W$  = Effective seismic weight of the structure to include in base shear calculation

$C_s$  is computed according to the following equations:

$$C_s = \frac{S_{Ds}}{R \cdot I}$$

$$C_{s \max} = \frac{S_{D1}}{T \left( \frac{R}{I} \right)}$$

$$C_{s \min} = 0.044 S_{Ds} I$$

Where:

$S_{Ds}$ ,  $S_{D1}$  = The design Spectral response acceleration parameters in the short period and 1sec. period and they were calculated in section 1.7.

$R$  = The Response Modification Factor.

$I$  = The Importance Factor.

$T$  = The Fundamental Period of the structure.

ASCE 7-10 requires that the calculated fundamental period of the structure not to be greater than the product of approximate period ( $T_a$ ) and upper limit coefficient  $C_u$ .

The approximate period ( $T_a$ ) is found using the following equation:

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

Where:

$h_n$  = Structural height (m)

$C_t, x$  = Parameters specified in Table 4-4

Table 4-4 The value of  $C_t$  and  $x$  according to ASCE 7-10

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

<sup>a</sup>Metric equivalents are shown in parentheses.

Therefore

$$T_a = 0.0488 \times 27.4^{.75}$$

$$= 0.5844 \text{ sec.}$$

Table 4-5 shows the values of the period upper limit coefficient  $C_u$

$$C_u T_a = 1.6 \times 0.5844 = 0.935 \text{ sec.}$$

Table 4-5 the value of  $C_u$  according to ASCE 7-10

**Table 12.8-1 Coefficient for Upper Limit on Calculated Period**

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

$C_s$  now could be found as follows:

$$C_s = \frac{0.6293}{\frac{4}{1}} = 0.157$$

And

$$C_{s\max} = \frac{0.2669}{0.847(4/1)} = 0.0787$$

$$C_{s\min} = 0.044 \times 0.464 \times 1 = 0.0204$$

$C_s > C_{s\max}$  and  $C_{s\min}$

The effective weight W is calculated as follows:

$$W = D_l + SD_l + 0.25 L_l$$

The dead load for effective weight includes the total Dead load plus the Superimposed dead load plus 25% of the live load.

$$W = 37697.3 + 21062.412 + 0.25 (7381.05) = 60604.97 \text{ KN}$$

$$V_x = 0.0482 * 60604.5 = 2921.13 \text{ KN}$$

$$V_y = 0.0579 * 60604.5 = 3509 \text{ KN}$$

From ETABS Fx due to EQX = 2847.54

**Difference percentage =  $\frac{2847.54 - 2921.13}{2921.13} = 2.5$  then <5% ok**

From ETABS Fy due to EQy = 3409.58 KN

**Difference percentage =  $\frac{3382.65 - 3509}{3509} = 3.6$  then <5% ok**

Check base shear from response spectrum method

From ETABS Fx due to EDX = 2837.899 KN

**Difference percentage =  $\frac{2837.899 - 2921.13}{2921.13} = 2.85$  then <5% ok**

From ETABS FY due to EDY = 3409.58 KN

**Difference percentage =  $\frac{3409.58 - 3509}{3509} = 2.83$  then <5% ok**

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:

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-direction:

**Total Reaction=2238.1488**

The screenshot shows the 'Section Cut Forces' dialog box with the following settings and results:

- Section Cutting Line:**
  - Global X: Start Point 0.297, End Point 0.885 m
  - Global Y: 0.319 m
- Load Case:** EQx
- Objects to Include:**
  - Columns,  Beams,  Braces
  - Floors,  Walls,  Links
- Resultant Force Location and Angle:**
  - Global X: 0.591 m
  - Global Y: 0.319 m
  - Global Z: 0 m
  - Angle: 358.636 deg
- Integrated Forces:**

	Right Side			Left Side		
	1	2	Z	1	2	Z
Force	2238.1488	58.8928	0	-2238.1488	-58.8928	0
Moment	-1011.7188	51934.1105	-23538.8356	1011.7188	-51934.1105	23538.8356

For wall Reaction= 1817.76

**Section Cutting Line**

	Start Point	End Point	
Global X	0.297	0.885	m
Global Y	0.319	0.319	m

**Load Case**: EQx

**Objects to Include**

- Columns
- Beams
- Braces
- Floors
- Walls
- Links

**Resultant Force Location and Angle**

Global X	0.591	m
Global Y	0.319	m
Global Z	0	m
Angle	358.636	deg

**Integrated Forces**

	Right Side			Left Side			
	1	2	Z	1	2	Z	
Force	1817.7684	48.2984	322.9968	-1817.7684	-48.2984	-322.9968	kN
Moment	1151.2207	37098.5164	-19538.6209	-1151.2207	-37098.5164	19538.6209	kN-m

Buttons: Save Right Side Cut, Save Left Side Cut, OK, Cancel, Refresh

Percentage of load resisting by walls =  $(1817.76/2238) \times 100\% = 81.5\%$ , so the system is Shear Wall Resisting System in the X direction

And we find the same calculators for Y-direction.

### 4.3.3 drift check:

Story drift is concerned with the structure serviceability. The design story drift ( $\Delta$ ) shall be computed as the difference of the deflections at the center of mass at the top and bottom of the story under consideration Table 4-6 shows the allowable story drift according to ASCE 7-10. The allowable limit of drift is equal to  $0.02h_{sx}$  where  $h_{sx}$  is the story height below level x.

*Table 4-6 Allowable Story Drift adopted from ASCE7-10*

Structure	Risk Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{xx}^c$	$0.020h_{xx}$	$0.015h_{xx}$
Masonry cantilever shear wall structures <sup>d</sup>	$0.010h_{xx}$	$0.010h_{xx}$	$0.010h_{xx}$
Other masonry shear wall structures	$0.007h_{xx}$	$0.007h_{xx}$	$0.007h_{xx}$
All other structures	$0.020h_{xx}$	$0.015h_{xx}$	$0.010h_{xx}$

The maximum elastic drift for each story in each direction are obtained from ETABS farther deformations expected due to inelastic response of the structure. So, the elastic values are multiplied by factor equal to  $C_d / 1$  were term previously explained.

Table 4-7 shows the calculated drift along with the allowable drift All drifts are within the allowable limit.

*Table 4-7 Drift calculation*

Story	Output Case	Case Type	Step Type	Direction	Drift	Label	X		Y		Z	DELTA
							m	2.58	m	15.22	m	
Story9	ENV/S	Combination	Max	X	0.0019	4	2.58	15.22	27.4	0.00949		
Story9	ENV/S	Combination	Max	Y	0.00275	3	-0.73	12.05	27.4	0.01376		
Story9	ENV/S	Combination	Min	X	0.00178	16	38.02	15.74	27.4	0.00892		
Story9	ENV/S	Combination	Min	Y	0.00196	22	39.57	3.8	27.4	0.0098		
Story8	ENV/S	Combination	Max	X	0.00201	16	38.02	15.74	24.4	0.01003		
Story8	ENV/S	Combination	Max	Y	0.00283	3	-0.73	12.05	24.4	0.01417		
Story8	ENV/S	Combination	Min	X	0.00189	16	38.02	15.74	24.4	0.00945		
Story8	ENV/S	Combination	Min	Y	0.00211	22	39.57	3.8	24.4	0.01057		
Story7	ENV/S	Combination	Max	X	0.00202	16	38.02	15.74	21.4	0.01012		
Story7	ENV/S	Combination	Max	Y	0.00292	3	-0.73	12.05	21.4	0.01461		
Story7	ENV/S	Combination	Min	X	0.00192	16	38.02	15.74	21.4	0.00959		
Story7	ENV/S	Combination	Min	Y	0.00226	3	-0.73	12.05	21.4	0.01131		
Story6	ENV/S	Combination	Max	X	0.00198	70	35.91	15.49	18.4	0.00992		
Story6	ENV/S	Combination	Max	Y	0.0029	138	-0.73	6.67	18.4	0.01449		
Story6	ENV/S	Combination	Min	X	0.00189	70	35.91	15.49	18.4	0.00947		
Story6	ENV/S	Combination	Min	Y	0.00234	138	-0.73	6.67	18.4	0.01169		
Story5	ENV/S	Combination	Max	X	0.00192	117	38.01	15.75	15.4	0.00958		
Story5	ENV/S	Combination	Max	Y	0.00278	109	-1.42	15.18	15.4	0.01392		
Story5	ENV/S	Combination	Min	X	0.00184	117	38.01	15.75	15.4	0.0092		
Story5	ENV/S	Combination	Min	Y	0.00235	109	-1.42	15.18	15.4	0.01174		
Story4	ENV/S	Combination	Max	X	0.00176	117	38.01	15.75	12.4	0.00882		
Story4	ENV/S	Combination	Max	Y	0.00274	109	-1.42	15.18	12.4	0.0137		

Story4	ENV/S	Combination	Max	Y	0.00274	109	-1.42	15.18	12.4	0.0137
Story4	ENV/S	Combination	Min	X	0.0017	117	38.01	15.75	12.4	0.00849
Story4	ENV/S	Combination	Min	Y	0.00233	109	-1.42	15.18	12.4	0.01163
Story3	ENV/S	Combination	Max	X	0.00152	11	8.49	15.49	9.4	0.00762
Story3	ENV/S	Combination	Max	Y	0.00255	109	-1.42	15.18	9.4	0.01277
Story3	ENV/S	Combination	Min	X	0.00147	117	38.01	15.75	9.4	0.00735
Story3	ENV/S	Combination	Min	Y	0.00222	109	-1.42	15.18	9.4	0.0111
Story2	ENV/S	Combination	Max	X	0.00119	11	8.49	15.49	6.4	0.00593
Story2	ENV/S	Combination	Max	Y	0.00202	58	0.18	11.8	6.4	0.01008
Story2	ENV/S	Combination	Min	X	0.00114	11	8.49	15.49	6.4	0.0057
Story2	ENV/S	Combination	Min	Y	0.00179	1	0	15.22	6.4	0.00895
Story1	ENV/S	Combination	Max	X	0.00057	34	12.48	15.49	3.4	0.00286
Story1	ENV/S	Combination	Max	Y	0.00111	1	0	15.22	3.4	0.00553
Story1	ENV/S	Combination	Min	X	0.00055	34	12.48	15.49	3.4	0.00273
Story1	ENV/S	Combination	Min	Y	0.00086	58	0.18	11.8	3.4	0.00432

All values are less than 0.02 so all values are accepted.

#### 4.3.4 P-delta check:

P-delta is concerned with stability of the structure, P-delta effect may be ignored if the stability index ( $\Theta$ ) is less than 0.1 as defined in ASCE7-10.

The stability index ( $\Theta$ ) is equal to:

$$\Theta = P \times \text{Stiffens}$$

Where:

$\Theta$  = The stability index.

P = Total Unfactored vertical design load.

$\Delta$  = Design Story Drift.

V = Seismic Shear Force.

$C_d$  = deflection amplification factor.

We take the value of P from Etabs as shown in table 4-8:

*Table 4-8 Value of Total Unfactored vertical design load.*

Story	P
	kN
Story9	9539.8477
Story8	19079.6951
Story7	28619.5432
Story6	38159.3734
Story5	47900.9801
Story4	57642.5865
Story3	67384.1925
Story2	77125.7986
Story1	89964.3586

**Stiff values are shown in table 4-9 for Stiff in x direction and table 4-10 for Stiff in y direction:**

*Table 4-9 stiff values in x direction*

Story	Output Case	Stiff X
		kN/m
Story9	ENV/U	260113.71
Story8	ENV/U	458260.547
Story7	ENV/U	610687.278
Story6	ENV/U	762853.212
Story5	ENV/U	888414.407
Story4	ENV/U	1071188.134
Story3	ENV/U	1349391.528
Story2	ENV/U	1930175.961
Story1	ENV/U	3766017.596

*Table 4-10 Stiff values in y direction*

Story	Output Case	Stiff Y
		kN/m
Story9	EQy	284673
Story8	EQy	504361
Story7	EQy	673990
Story6	EQy	829014
Story5	EQy	994096
Story4	EQy	1188444
Story3	EQy	1488868
Story2	EQy	2096335
Story1	EQy	3988593

Table of 4-11 shows the values of theta for all stories in x direction:

*Table 4-11 the values of theta for all stories in x direction*

Story	Output Case	P kN	H		Story	Output Case	Stiff X kN/m	THETA
Story9	ENV/U	9539.85	3		Story9	EQx	260114	0.01222523
Story8	ENV/U	19079.7	3		Story8	EQx	458261	0.01387835
Story7	ENV/U	28619.5	3		Story7	EQx	610687	0.01562149
Story6	ENV/U	38159.4	3		Story6	EQx	762853	0.01667397
Story5	ENV/U	47901	3		Story5	EQx	888414	0.01797246
Story4	ENV/U	57642.6	3		Story4	EQx	1071188	0.01793727
Story3	ENV/U	67384.2	3		Story3	EQx	1349392	0.01664557
Story2	ENV/U	77125.8	3		Story2	EQx	1930176	0.0133193
Story1	ENV/U	89964.4	3.4		Story1	EQx	3766018	0.00702602

Table of 4-12 shows the values of theta for all stories in y direction:

*Table 4-12 the values of theta for all stories in y direction*

Story	Output Case	Stiff Y kN/m	P kN	H	THETA
Story9	EQy	284673	9539.85	3	0.01117051
Story8	EQy	504361	19079.7	3	0.01260983
Story7	EQy	673990	28619.5	3	0.0141543
Story6	EQy	829014	38159.4	3	0.01534327
Story5	EQy	994096	47901	3	0.01606183
Story4	EQy	1188444	57642.6	3	0.01616752
Story3	EQy	1488868	67384.2	3	0.01508623
Story2	EQy	2096335	77125.8	3	0.01226359
Story1	EQy	3988593	89964.4	3.4	0.00663395

All theta values are less than 0.1

So, according to ASCE7-10, P-Delta effect could be ignored.

### 4.3.5 Structural irregularity

#### For Horizontal Structural Irregularities

Torsional Irregularity	Not Exist
Extreme Torsional Irregularity	Not Exist
Reentrant Corner Irregularity	Not Exist
Diaphragm Discontinuity Irregularity	Not Exist
Out-of-Plane Offset Irregularity	Not Exist
Nonparallel System Irregularity	Not Exist

#### For Vertical Structural Irregularities

Stiffness-Soft Story Irregularity	Not Exist
Stiffness-Extreme Soft Story Irregularity	Not Exist
Weight (Mass) Irregularity	Not Exist
Vertical Geometric Irregularity	Not Exist
In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity	Not Exist
Discontinuity in Lateral Strength-Weak Story Irregularity	Not Exist
Discontinuity in Lateral Strength-Extreme Weak Story Irregularity	Not Exist

### 4.3.6 check deflection

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 318-14:

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 $L$	$l/180^*$
Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load $L$	$l/360$
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) <sup>†</sup>	$l/480^{\ddagger}$
Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections		$l/240^{\S}$

\*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.

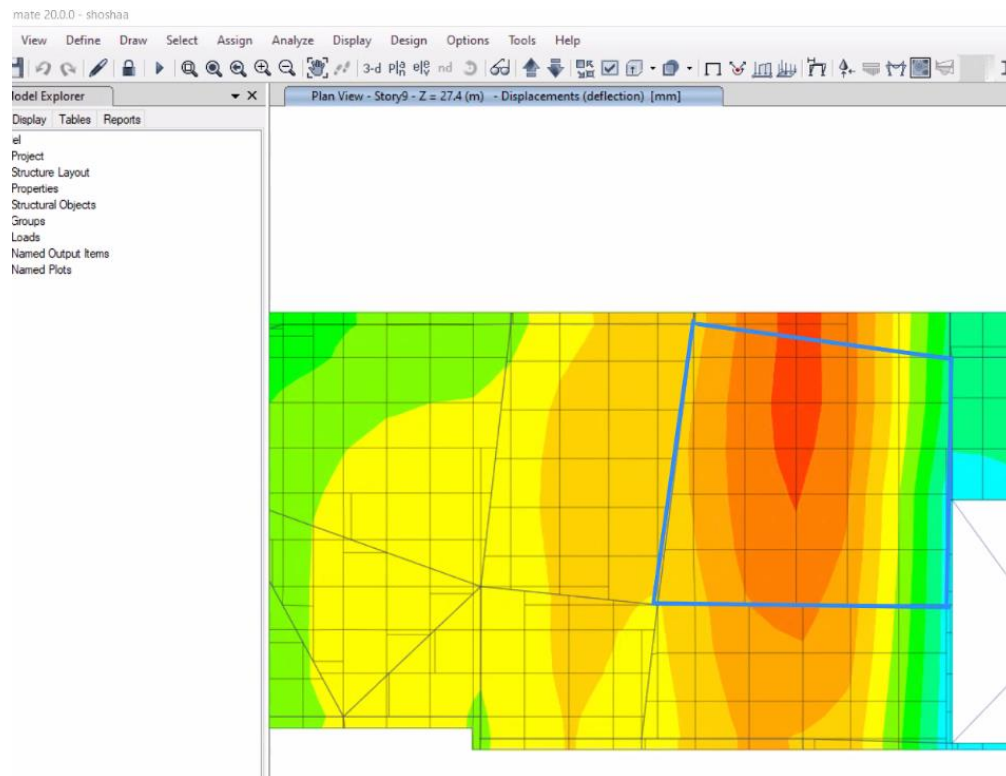
†Long-term deflection shall be determined in accordance with 9.5.2.5 or 9.5.4.3, but may be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be determined on basis of accepted engineering data relating to time-deflection characteristics 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.

Long term deflection combination: 2D+2SD+1.5 L

The figure 4-13 below shows the contour line for deflection in slab



**In X direction:**

long term deflection allowable deflection =  $L/240 = (6.305/240) * 1000 = 26.27$  mm .

long-term deflections for slab systems at the middle = 15.12 mm

long-term deflections for slab systems at the right side = 3.11 mm

long-term deflections for slab systems at the left side = 10.9 mm

Relative long-term deflection =  $15.12 - 3.11 + 10.9/2 = 8.17 < 26.27$  it is ok

**In Y direction:**

long term deflection allowable deflection =  $L/240 = (6.17/240) * 1000 = 25.7$  mm .

long-term deflections for slab systems at the middle = 15.12 mm

long-term deflections for slab systems at the top = 15.17 mm

long-term deflections for slab systems at the bottom = 13.5 mm

Relative long-term deflection =  $15.12 - 15.17 + 13.5/2 = 0.78 < 25.7$  it is ok

## 5 .Design verification

### 5.1 Beam design verification

Use the following beam for check, This beam has a length of 4.3m and the dimensions for the cross section are 800 mm in width and 300 mm in depth (d=240 mm).

**Design Moment and Flexural Reinforcement for Moment,  $M_{u3}$**

	Design Moment kN-m	Design $P_u$ kN	-Moment Rebar mm <sup>2</sup>	+Moment Rebar mm <sup>2</sup>	Minimum Rebar mm <sup>2</sup>	Required Rebar mm <sup>2</sup>
Top (+2 Axis)	-84.2839	0	1011	20	640	1011
Bottom (-2 Axis)	0	0	0	20	26	26

The area of steel for this beam due to the ultimate negative moment = 1011 mm<sup>2</sup> .

$$M_u = 84.28 \text{ KN.m}$$

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

$$\rho = 0.0051$$

$$A_s = \rho b d = 0.0051 \times 800 \times 240 = 979.2 \text{ mm}^2$$

$$. A_s \text{ min} = \rho \text{ min} \times b \times d = 0.00333 \times 800 \times 240 = 640 \text{ mm}^2 .$$

$A_s > A_s \text{ min}$  , its ok.

$$\rho \text{ max.singly} = 0.01806$$

$$A_s, \text{ max} = 0.01806 \times 800 \times 240 = 3476.5 \text{ mm}^2 \text{ Ok}$$

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

$$\text{Difference percentage} = \frac{A_s \text{ from manual} - A_s \text{ from ETABS}}{A_s \text{ from ETABS}} \times 100\% = 3.2\%$$

**Shear Design Verification:**

### Shear Force and Reinforcement for Shear, $V_{u2}$

Shear $V_{u2}$ kN	Shear $\phi V_c$ kN	Shear $\phi V_s$ kN	Shear $V_p$ kN	Rebar $A_v / s$ mm <sup>2</sup> /m
77.654	125.5861	49.6423	0	666.67

$V_u = 77.654 \text{ kN}$

$V_c = 1/6 * \sqrt{28} * b_w * d = 1/6 * \sqrt{28} * 800 * 240 / 1000 = 169.32 \text{ kN}$

$\phi V_c = 0.75 * 169.32 = 127 \text{ kN}$

$\phi V_c > V_u$  then no need for shear reinforcement, use  $A_v/s$  min

$A_v/S_{min} = \max \text{ of } (0.062 \sqrt{f_c} * b_w / f, 0.35 * b_w / f_y)$

$(0.062 \sqrt{f_c} * b_w / f = 625 \text{ mm}^2/\text{m}$

$, 0.35 * b_w / f_y = 666.67 \text{ mm}^2/\text{m}$

### 5.2 Column design verification

Use the following column for check, which is located in the third floor. This column has a length of 3 m and the dimensions for the cross section are 1050mm and 300 mm.

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

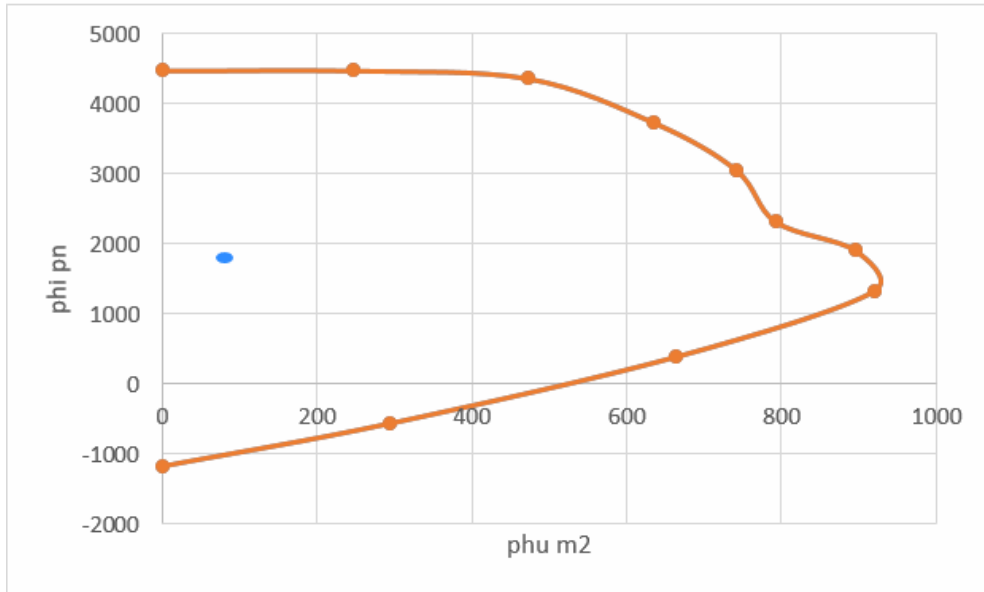
Design $P_u$ kN	Design $M_{u2}$ kN-m	Design $M_{u3}$ kN-m	Minimum M2 kN-m	Minimum M3 kN-m	Rebar Area mm <sup>2</sup>	Rebar % %
1957.6986	79.7838	-55.1985	91.5028	47.4546	3150	1

$P_u = 1957.7 \text{ kN}$

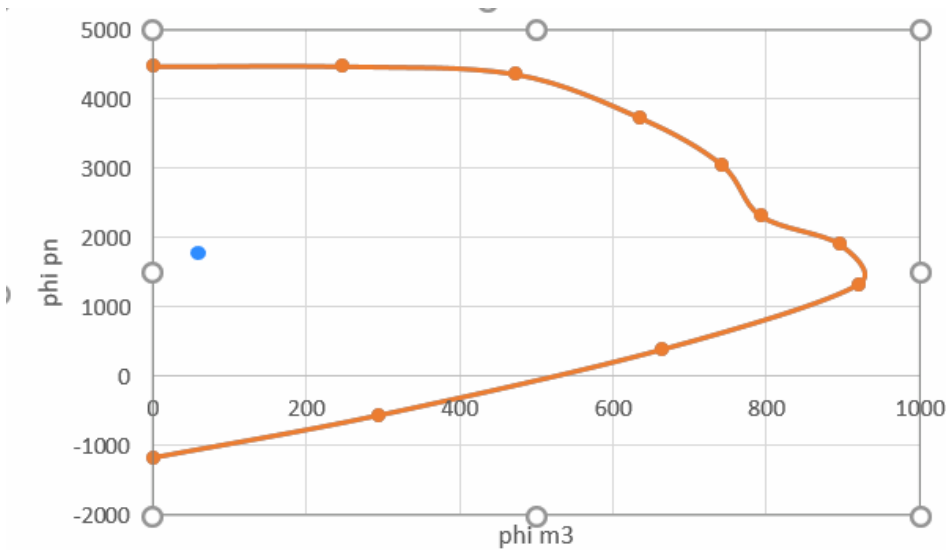
$M_{u,2} = 79.8 \text{ kN.m}$

$M_{u,3} = 55.2 \text{ kN.m}$

interaction diagram between  $\phi P_n$  and  $\phi M_{n,2}$ , to check the column capacity for them.



interaction diagram between  $\phi P_n$  and  $\phi M_{n,3}$ , to check the column capacity for them



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

$$\frac{1}{\phi P_n} = \frac{1}{\phi P_{n,3}} + \frac{1}{\phi P_{n,2}} - \frac{1}{\phi P_{n,0}}$$

Where:  $\phi P_n$ : Axial load capacity for  $M_{u,3}$  and  $M_{u,2}$ .

$\phi P_{n,3}$ : Axial load capacity for  $M_{u,3}$ .

$\phi P_{n,2}$ : Axial load capacity for  $M_{u,2}$ .

$\phi P_{n, o}$ : Axial load capacity without moment.

$$\phi P_{n,3} = 4500 \text{ kN}$$

$$\phi P_{n,2} = 4500 \text{ kN}$$

$$\begin{aligned} \phi P_{n, o} &= \phi \lambda (0.85 f'_c (A_g - A_s) + A_s f_y) \phi P_n, \\ &= 0.65 * 0.8 * (0.85 * 28 * (315000 - 3150) + 3150 * 420) \end{aligned}$$

$$\phi P_{n,o} = 4547.4 \text{ kN}$$

$\phi P_n = 4453.6 \text{ kN} > 1957.7 \text{ kN}$  then its ok

□

## 6 .Design of structural element

This section will include sections, reinforcement and detailing of structural elements.

### 6.1 Design of slab

Check for shear:  $\phi V_c = \phi 6 * \sqrt{f'_c} * b_w * d$

$$\phi V_c = 0.75/6 * \sqrt{28} * 1000 * 300 = 198.4 \text{ kN}$$

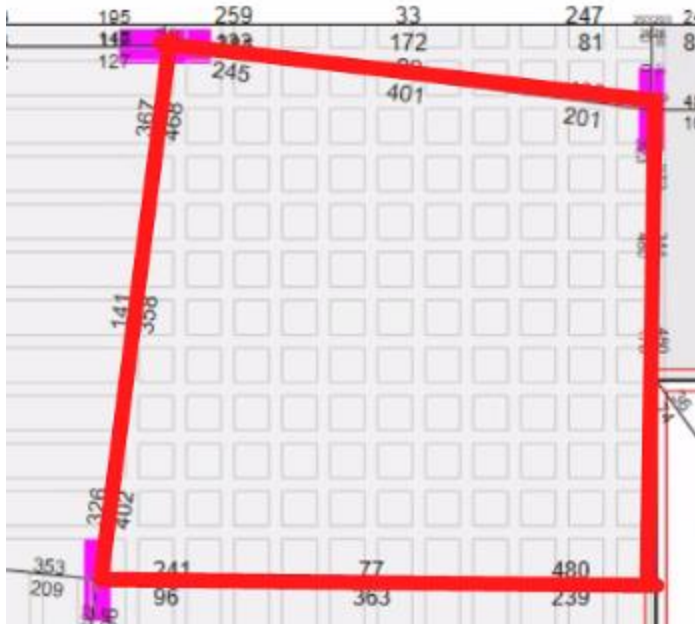
$V_u$  in first floor = 106 kN <  $\phi V_c$  , then it's OK

$V_u$  in (1st-5<sup>th</sup>) floor = 100 kN <  $\phi V_c$  , then it's OK

$V_u$  in sixth floor = 121 kN <  $\phi V_c$  , then it's OK

$V_u$  in (7<sup>th</sup>-9<sup>th</sup>) floor = 122 kN <  $\phi V_c$  , then it's OK

∴ No Need for shear reinforcement.



This is the big panel, And we take two strip in (x,y) direction.

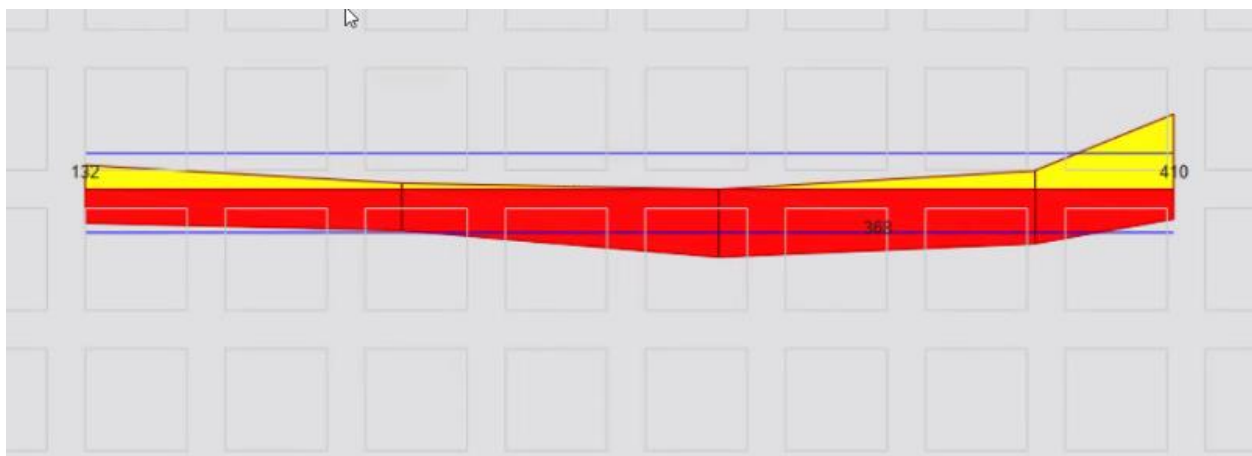
In x-direction(Top Steel)

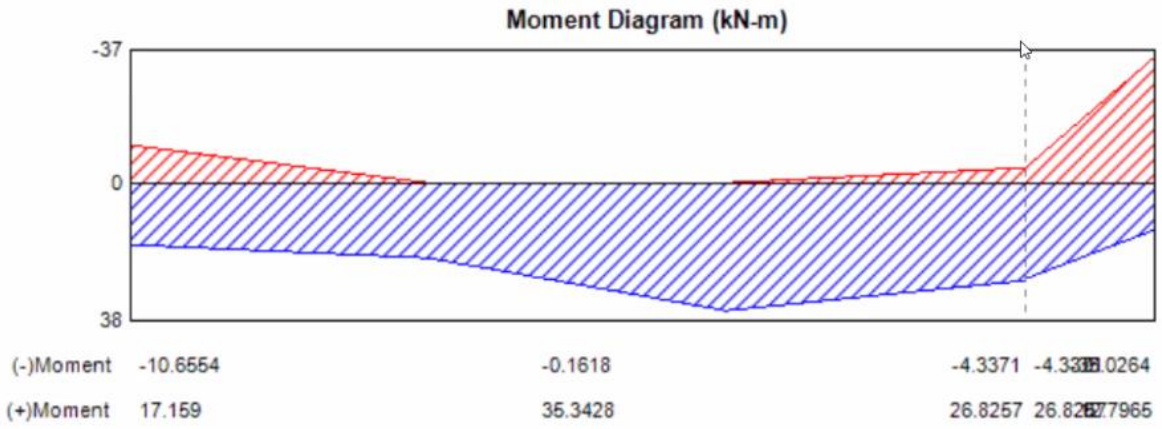
$$\rho = 0.00265$$

$$A_s = 0.00265 * 550 * 260 = 379 \text{ mm}^2$$

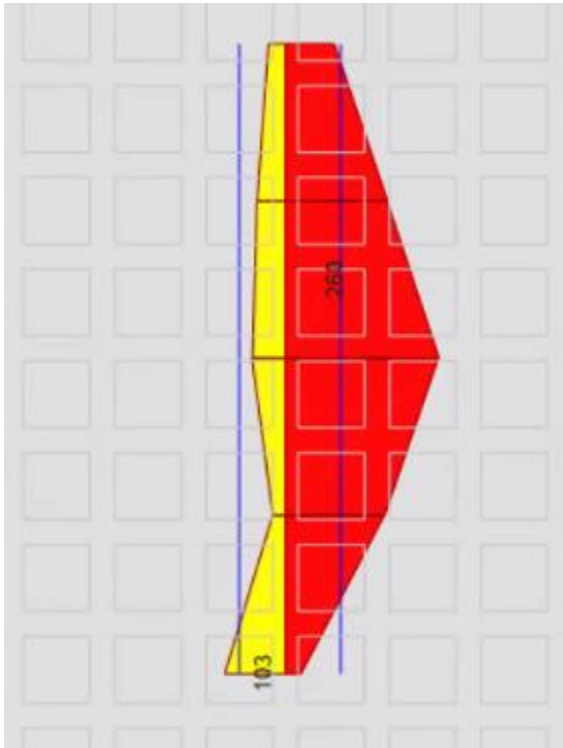
$$\text{Error} = \frac{379 - 361}{379} = 4.74\% < 25\% \text{ .ok}$$

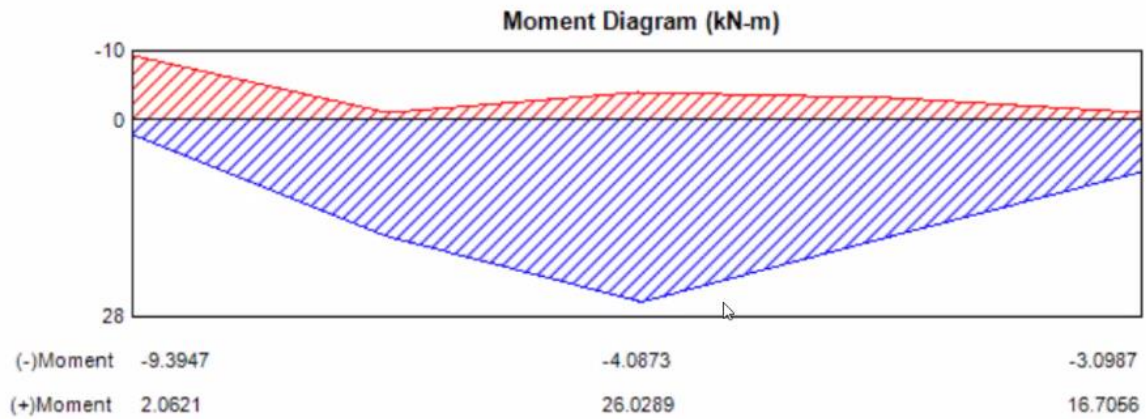
Use 2Ø16 And we use 2Ø14 in Top steel





**In Y-direction(Top Steel)**





$$\rho = 0.0019$$

$$A_s = 0.0019 \times 550 \times 260 = 271.7 \text{ mm}^2$$

$$\text{Error} = \frac{271.7 - 263}{271.7} = 3.2\% < 25\% \text{ .ok}$$

Use 2  $\phi$  14

And we use 2 $\phi$ 12 in Top steel

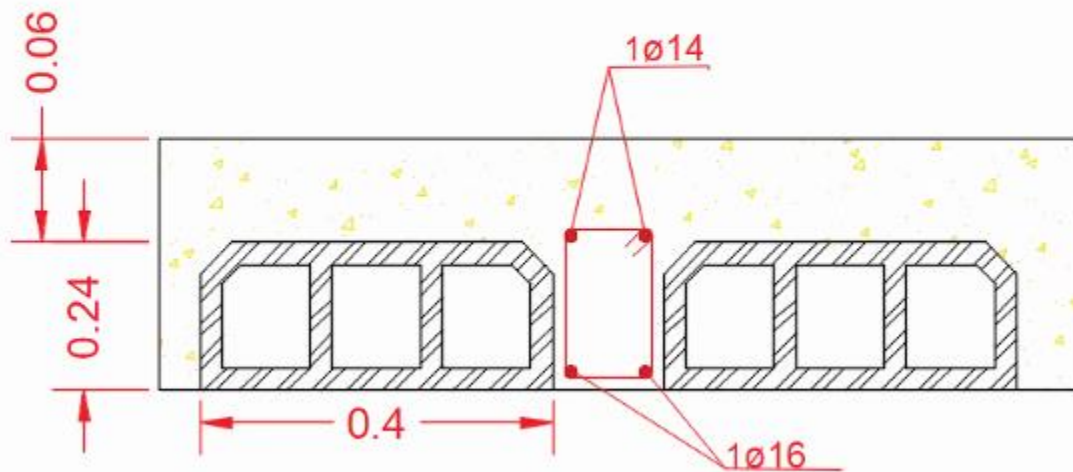


Fig4-6: Cross section for slab (X-direction)

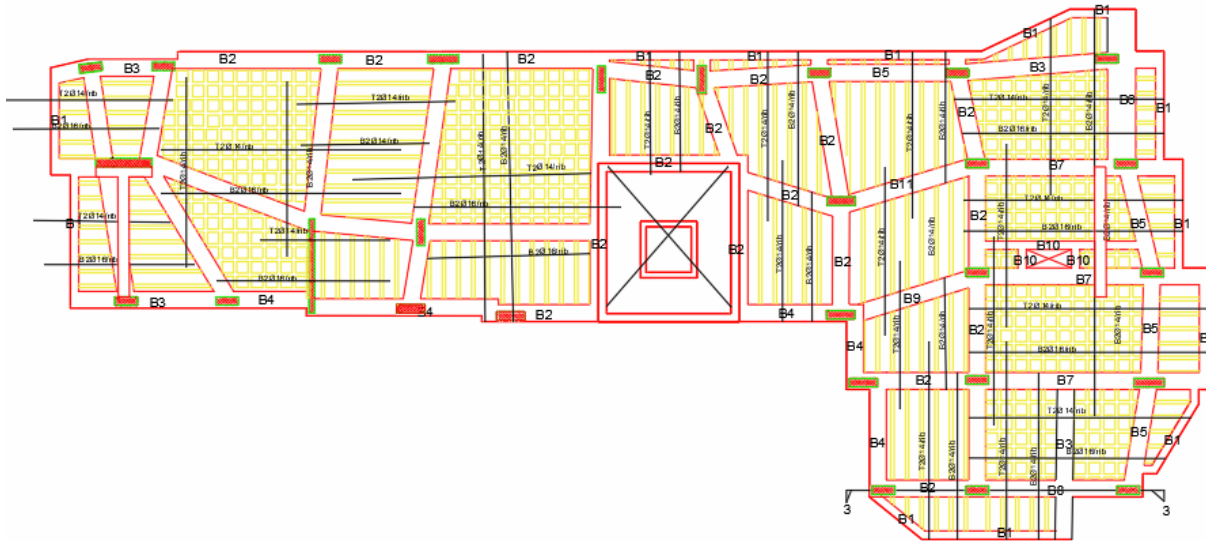


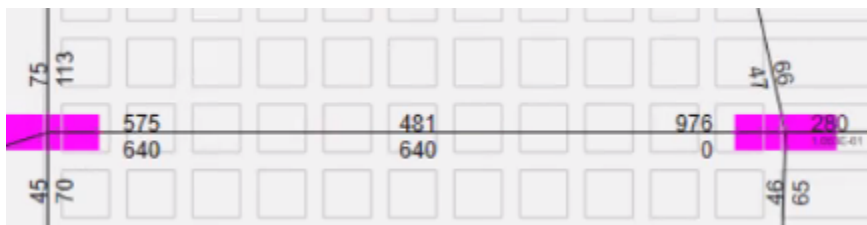
Fig4-7:Top view of slab Reinforcement

## 6.2 Design of beams

Beams with special moment frames :

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 .

### Shear Strength



H=300

D=240

B=800

As Bottom = 640mm<sup>2</sup> .

As Top1 = 640 mm<sup>2</sup> .

$A_s \text{ Top2} = 976 \text{ mm}^2$  .

$A_s, \text{ min} = 0.0033 * 800 * 240 = 633.6 \text{ mm}^2 \rightarrow \text{O.K}$

$\rho \text{ max.singly} = 0.01806$

$A_s, \text{ max} = 0.01806 * 800 * 240 = 3467.52 \text{ mm}^2 \rightarrow \text{O.K.}$

❖ **Longitudinal Reinforcement:**

Bottom Steel:

$A_s = 640 \text{ mm}^2 \rightarrow \text{use } 4\phi 14$

Top1 Steel:

$A_s = 640 \text{ mm}^2 \rightarrow \text{use } 4\phi 14$

Top2 Steel:

$A_s = 976 \text{ mm}^2 \rightarrow \text{use } 5\phi 16$

**Shear strength:**

**$A_v/s = 0.6667$**

**$A_v = 3 * 78.5 = 335 \text{ mm}^2$**

**$S = 224 \text{ mm}$**

**$S \text{ max} = \text{min of } (d/2, 600) = (240/2, 600)$ . Then  $S_{\text{max}} = 120 \text{ mm}$**

**Use 3  $\phi 10$  each 12**



Reinforcement For Beams				
#.Beam	Depth	Width	Top	Bottom
B 1	30	30	2Ø12	2Ø12
B 2	30	100	6Ø12	6Ø14
B 3	30	80	5Ø16	5Ø12
B 4	30	100	4Ø16	3Ø14
B 5	30	60	6Ø12	6Ø14
B 6	30	80	5Ø14	5Ø16
B 7	30	60	5Ø12	5Ø14
B 8	30	60	4Ø12	4Ø14
B 9	30	80	3Ø14	5Ø14
B 10	30	60	4Ø14	5Ø14

Fig4-8:Reinforcement for beams

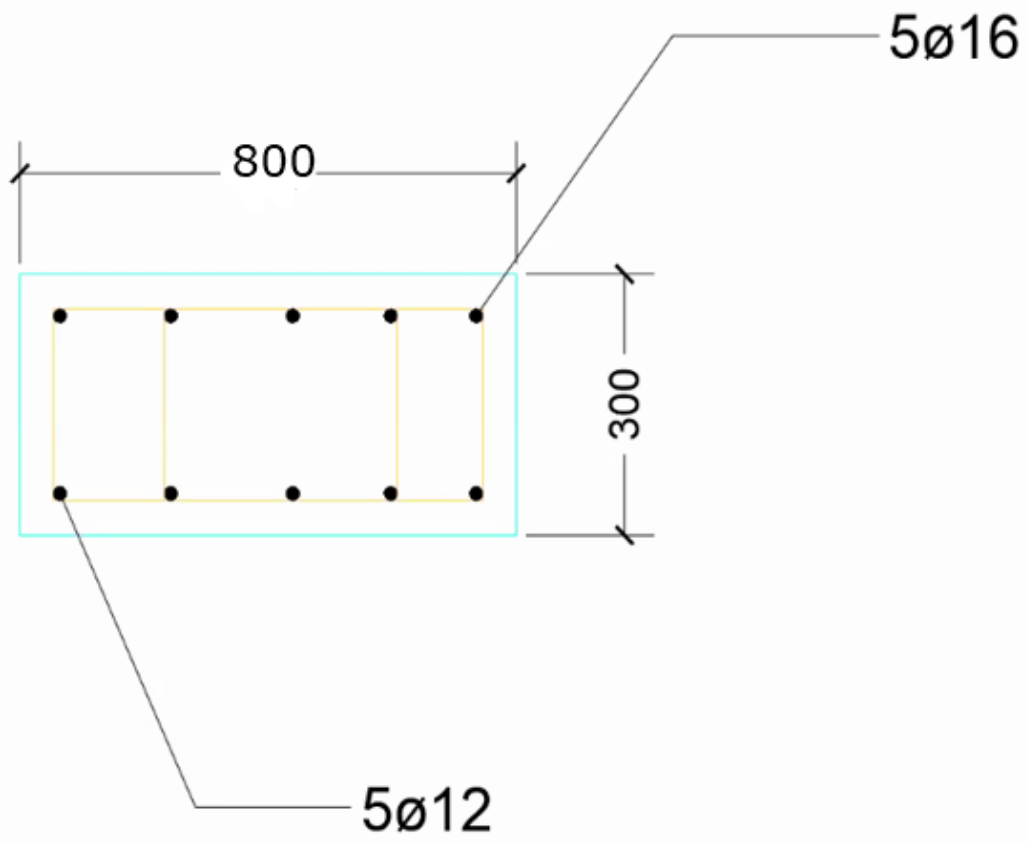


Fig4-9:Cross section For beam3

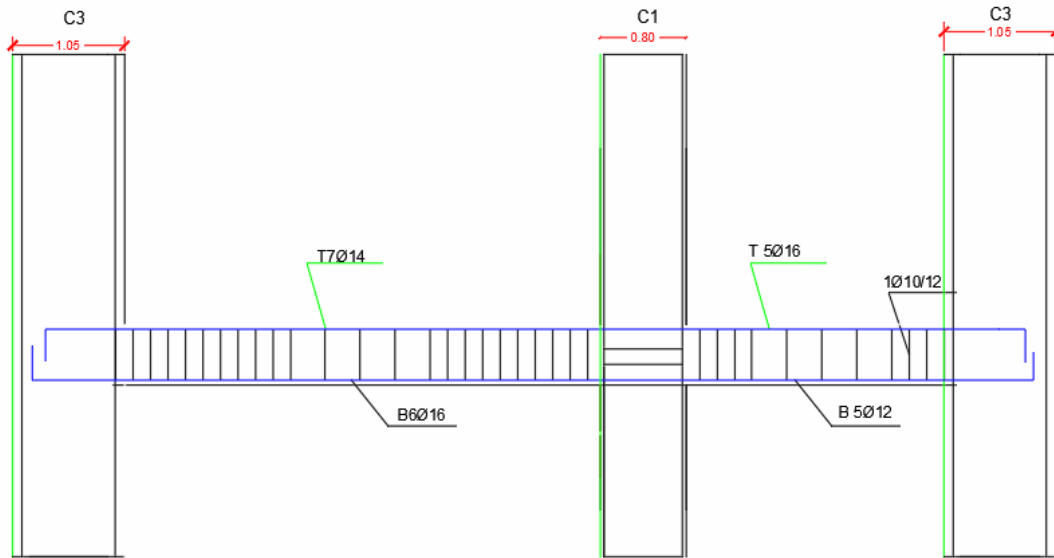


Fig 4-10:longitudinal section

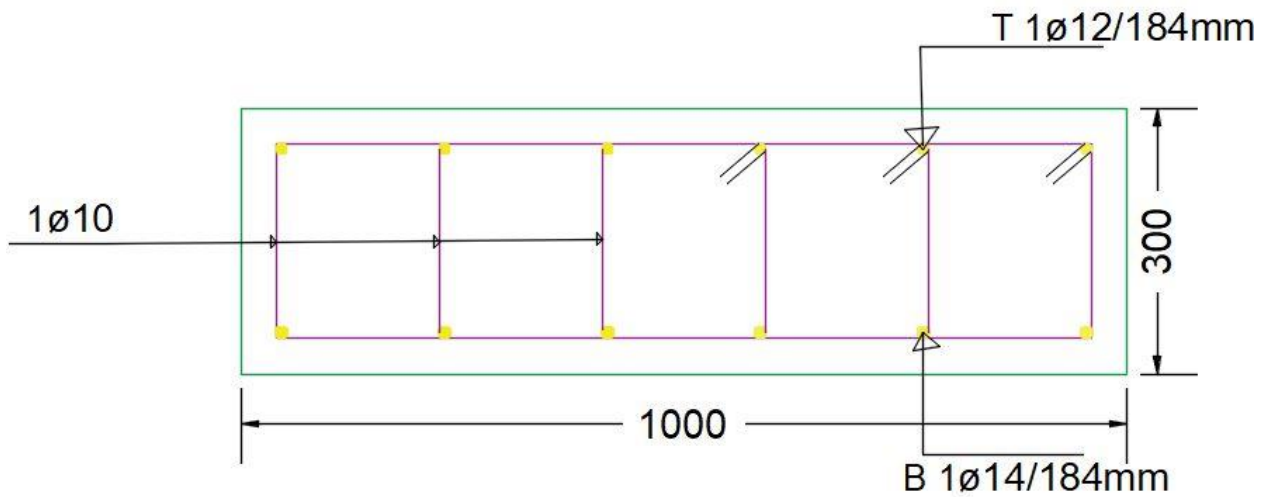


fig4-11:Cross section For Beam2

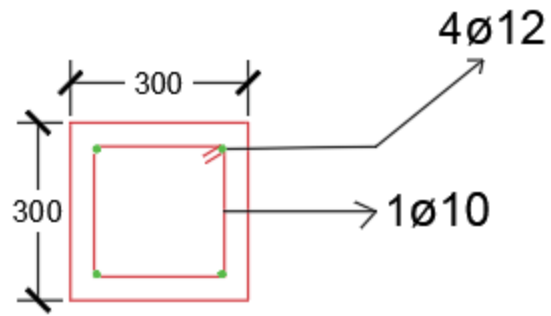


fig4-12:Cross section For Beam1

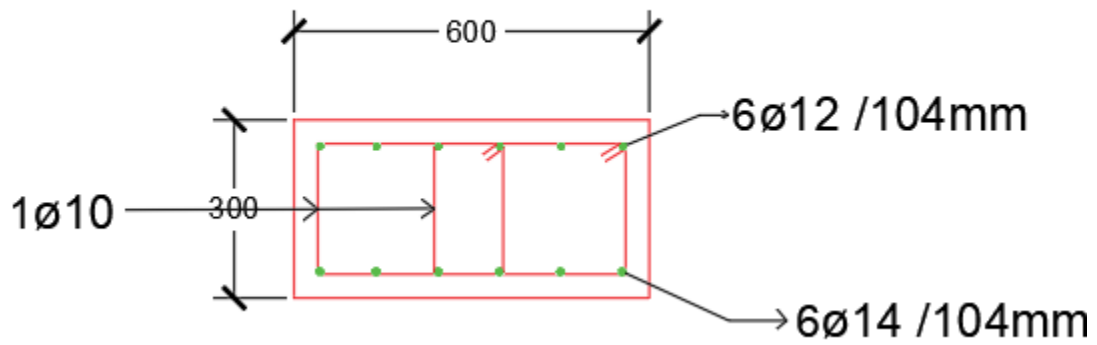


fig4-13:Cross section For Beam5

### 6.3 Design of columns

1-Design of column(400\*800)

2- Design of column(1050\*400)

3-Design of column(950\*300)



$$R=0.3*0.4=0.12$$

$$L_u=3.1\text{m}$$

$$K*L_u/r= 25.83$$

$34+12M1/M2= 39.7 < 40$  Then no slenderness, consider a short column.

$$P_u=2601\text{ KN}$$

$\rho$  From Etabs=0.01

$$A_s=0.01*400*850=3400\text{mm}$$

▲ Use 14 $\phi$ 18

Transverse reinforcement:

The spacing between stirrups is as the following

$$S < 48d_s = 48*10=480\text{ mm}$$

$$< 16d_p = 16*16=256\text{ mm}$$

< least lateral dimension of column=400 mm

Use 1 $\phi$ 10/250mm.

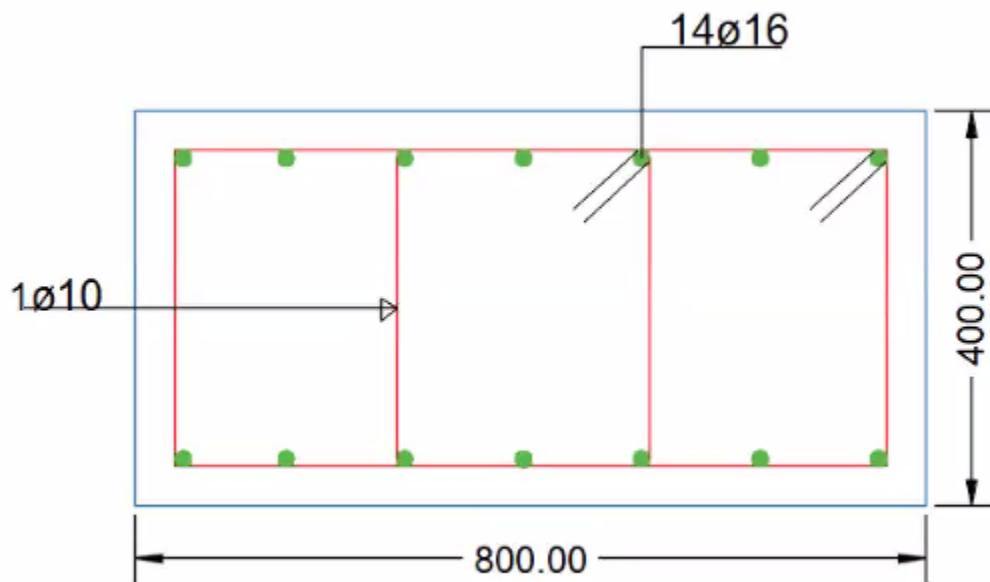


Fig 4.14: Cross section in column

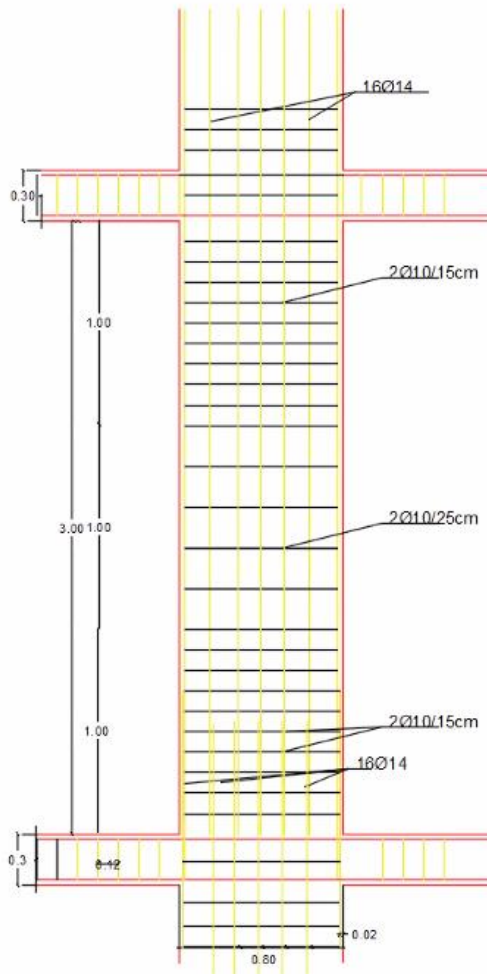


Fig4-15:General longitudinal section in column

## 2- Design of coulmn(1050\*400)



**As=4200mm**

**▲17φ18**

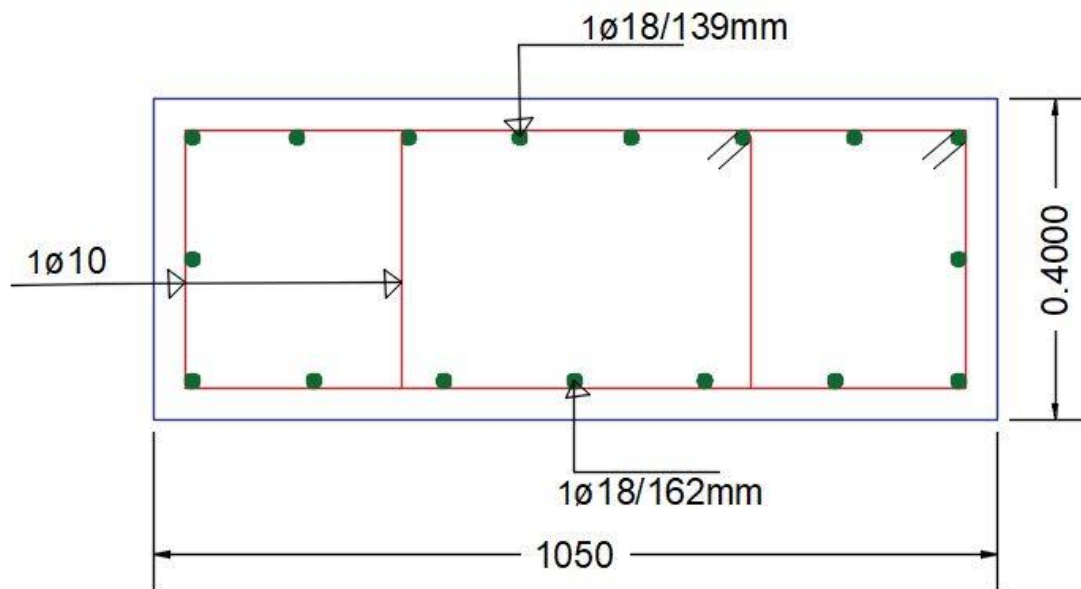


Fig 4.16: Cross section in column

### 3-Design of Column (950\*300) :

$A_s=3580\text{mm}^2$

▲18 $\phi$ 16

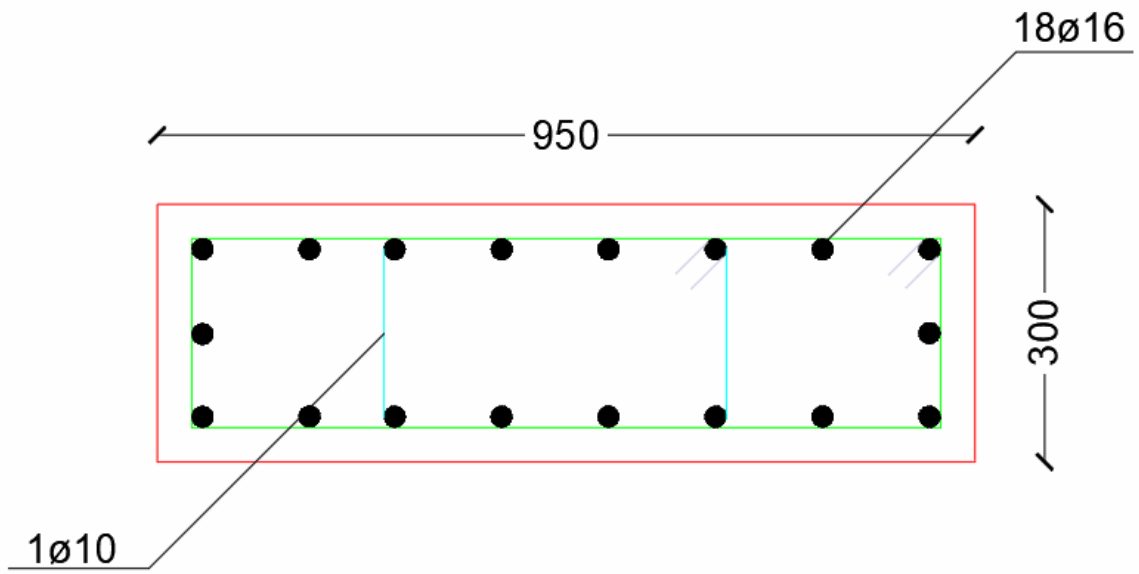
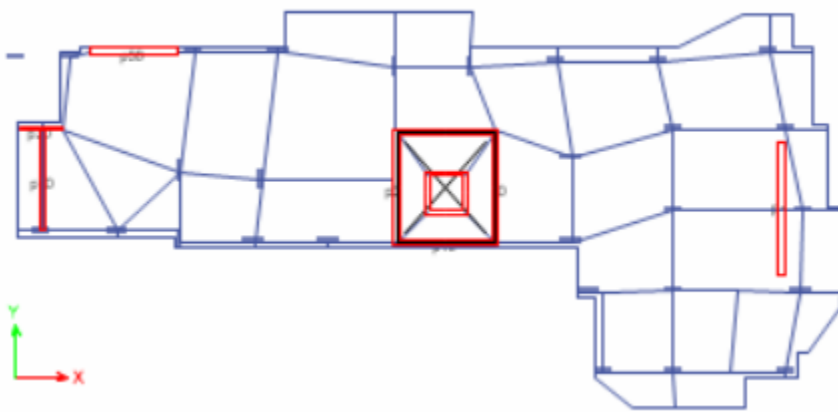
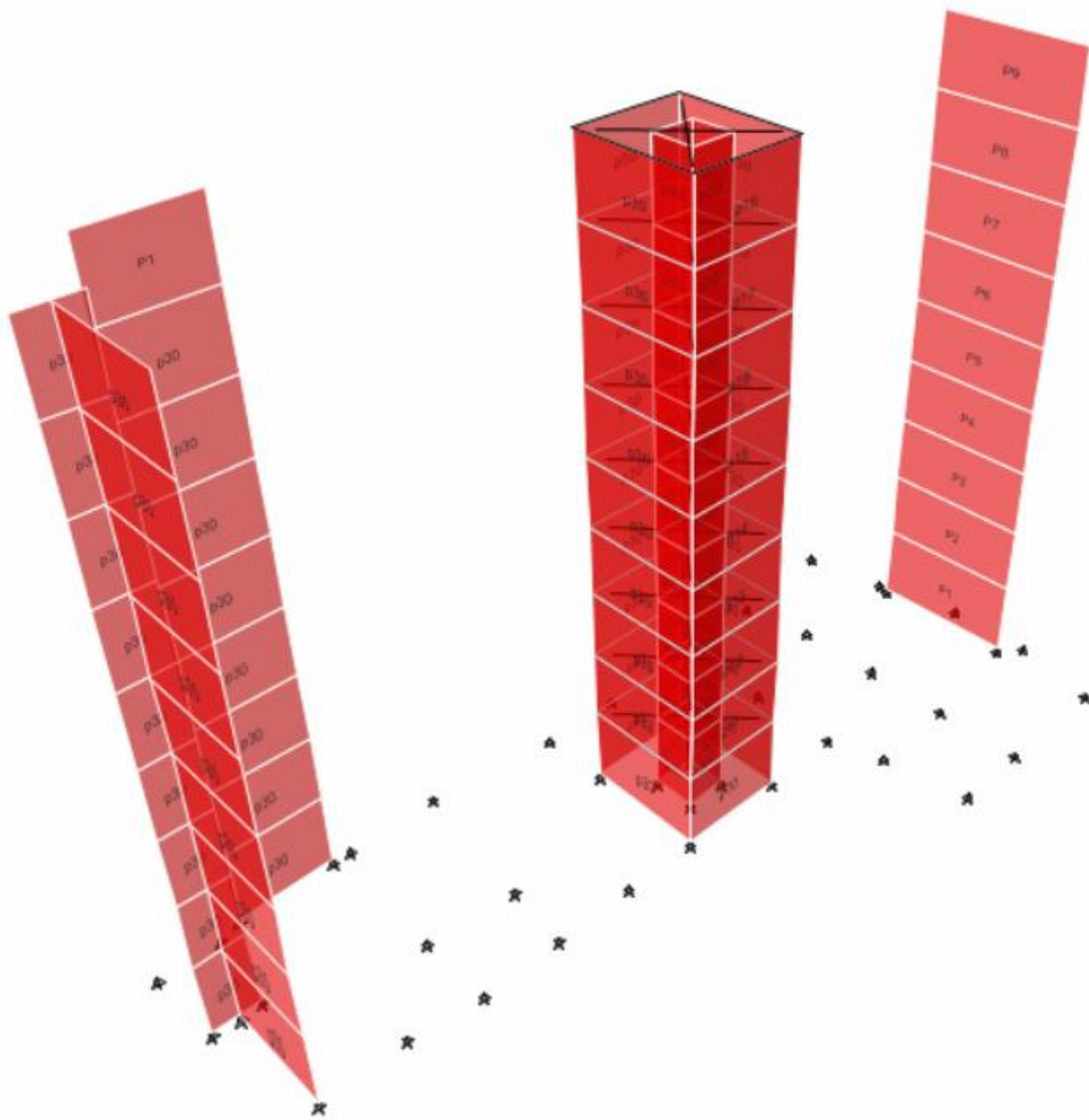
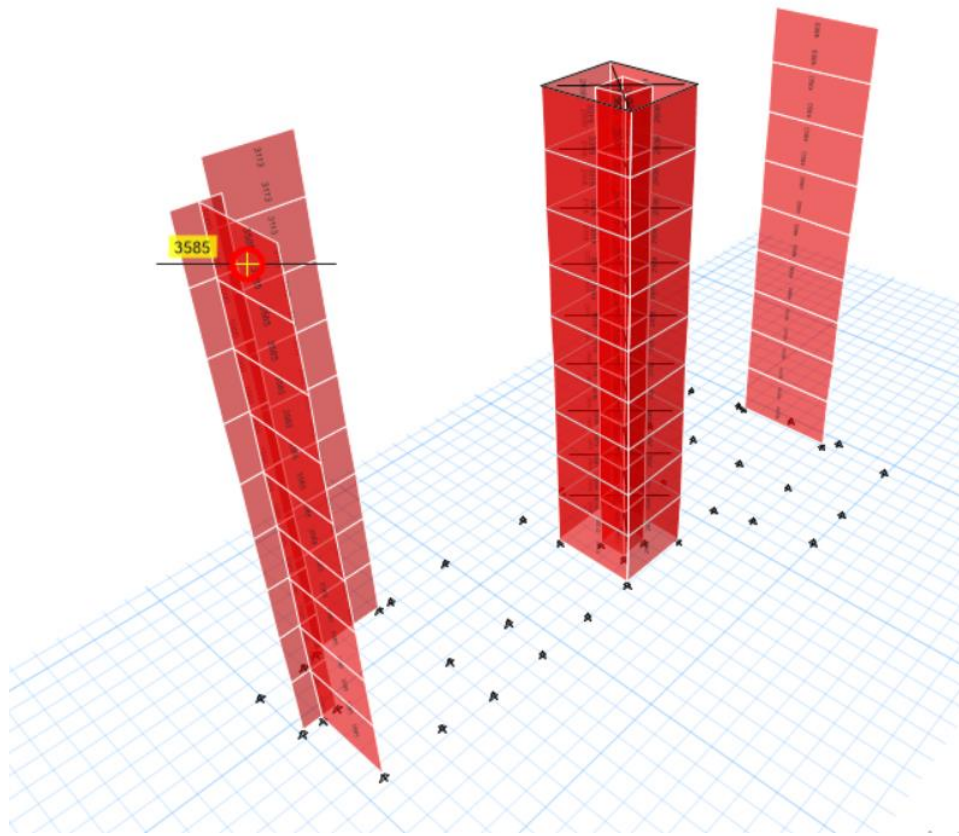


Fig 4.17: Cross section in column

#### 6.4 Design For shearwall





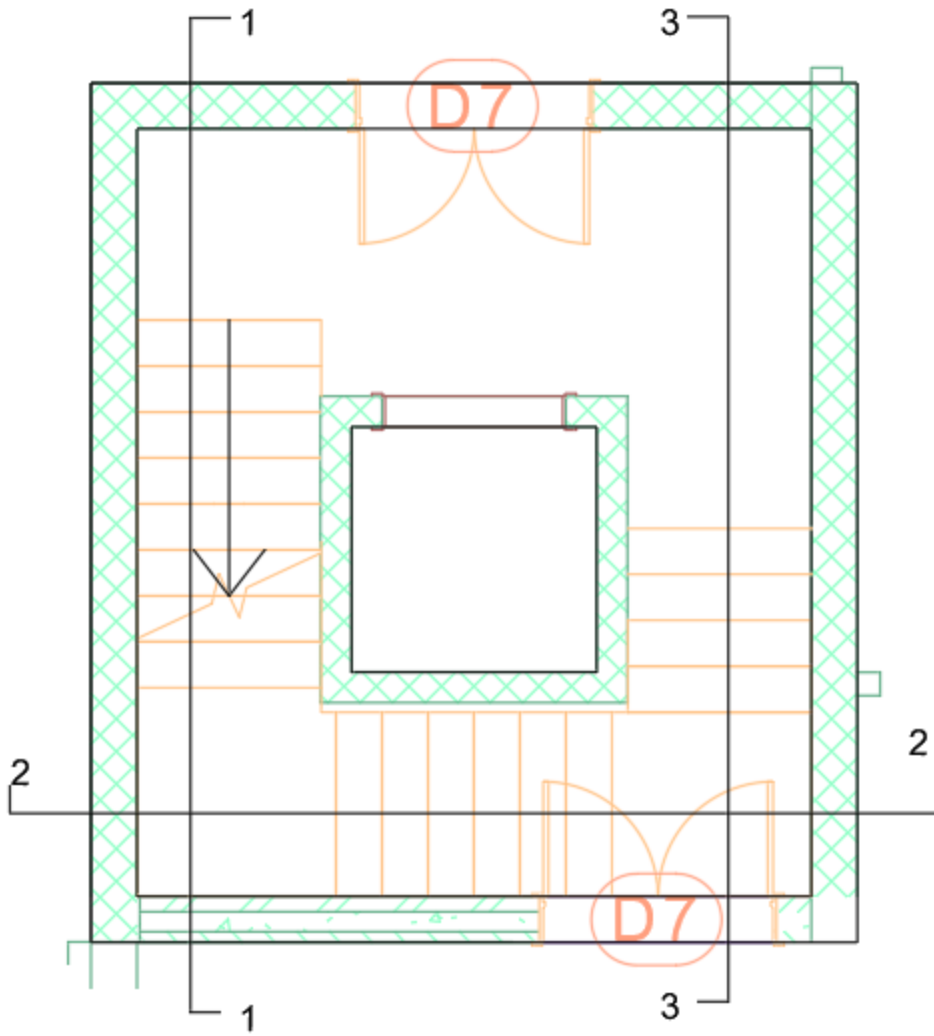


**Area of steel from Etabs =3585 mm<sup>2</sup> along the wall.**

$$= 3585/1=3585 \text{ mm}^2/\text{m} \rightarrow 8\phi 25/\text{m}$$

### 6.5 Design of staircase

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 stairs. The plan of stairs was taken from the Architectural plans, which was shown in Figure below.



*Stair Flight Reinforcement*

load calculation of flight (as 200mm 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 3cm, was computed as follows:

$$\text{Weight of staircase} = 0.02 * 23 + 0.05 * 17 + 0.03 * 27 = 2.12 \text{KN/m}^2 .$$

Stair width = 30cm and height = 15cm, so:

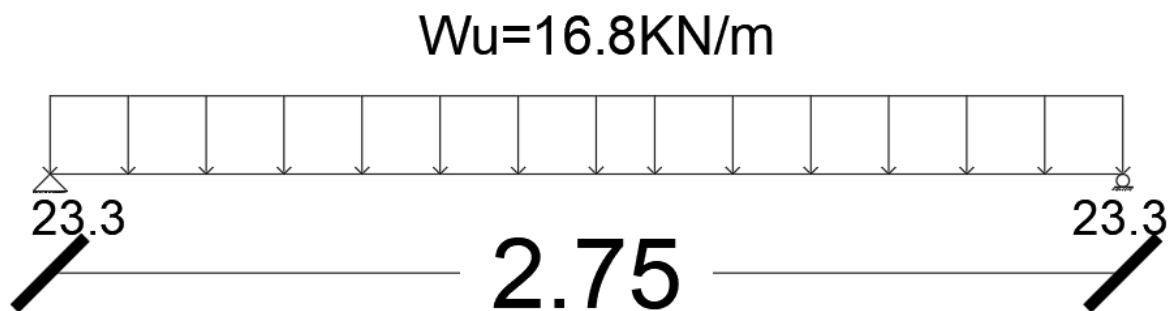
$$\text{Wight of concrete triangle of the stair} = 0.5 * 0.17 * 0.3 / 0.345^2 = 0.214 \text{KN/m}^2 .$$

Total S.D load on the stair=  $2.12 + 0.214 = 2.334 \text{ KN/m}^2$

Dead load of stair slab=  $25 * 0.2 = 5 \text{ KN/m}^2$

$W_u = 1.2D + 1.6L$

$W_u = 1.2 * (5 + 2.334) + 1.6 * 5 = 16.8 \text{ KN/mm}^2$ .



$f'_c = 28 \text{ MPa}$

$f_y = 420 \text{ MPa}$

$d = 160 \text{ mm}$

Assume simply supported slab:

✓ Shear check

$V_u = w_u * L / 2 = 16.8 * 2.75 / 2 = 23.3 \text{ K}$

$\phi V_c = \phi / 6 * \lambda * \sqrt{f'_c} * b_w * d = \phi / 6 * 1 * \sqrt{28} * 1000 * 160 = 105.8 \text{ kN} > V_u, \text{OK}$

✓ Flexural reinforcement

$M_u = w_u * L^2 / 8 = 16.8 * 2.75^2 / 8 = 15.88 \text{ kN} \cdot \text{M}$

$$\rho = \frac{0.85 * f'_c}{f_y} * \left( 1 - \sqrt{1 - \frac{2.61 * M_u}{b_w * d^2 * f'_c}} \right)$$

$P=0.0017$

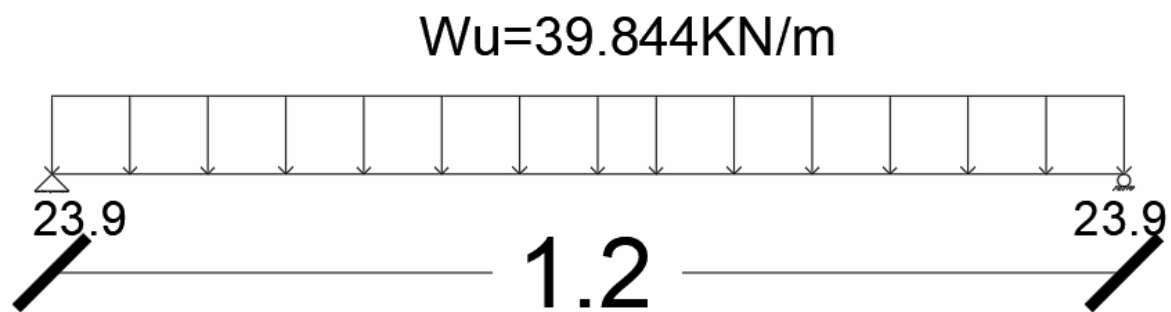
$A_s = \rho * b_w * d = 0.0017 * 1000 * 160 = 272 \text{ mm}^2 / \text{m}$

As shrinkage =  $.0018 * b_w * h = .0018 * 1000 * 200 = 360 \text{ mm}^2 / \text{m} > A_s, K$

So use  $A_s$  min, use  $\varnothing 12/250$  mm

For transverse steel use  $A_s$  min, use  $\varnothing 12/250$  mm

➤ Load calculation of landing (as a 200mm slab):



Assume that flight transfer the loads to landing.

$f'_c = 28 \text{ MPa}$

$f_y = 420 \text{ MPa}$

$d = 160 \text{ mm}$

$$S.D \text{ load} = 0.02 * 23 + 0.05 * 17 + 0.03 * 27 = 2.12 \text{KN/m}^2 .$$

$$\text{Dead load of stair slab} = 0.25 * 0.2 = 5 \text{ KN/m}^2 ..$$

$$W_u = 1.2D + 1.6L + \text{Reaction from flight slab.}$$

$$= 1.2 * (5 + 2.12) + 1.6 * 5 + 23.3 = 39.844 \text{KN/m}^2 .$$

➤ Design of landing (as a 200mm slab):

Assume simply supported slab:

✓ Shear check:

$$V_u = w_u * L / 2 = 39.844 * 1.2 / 2 = 23.9 \text{KN.}$$

$$\phi V_c = \phi / 6 * \lambda * \sqrt{f'_c} * b_w * d = \phi / 6 * 1 * \sqrt{28} * 1000 * 160 = 105.8 \text{KN} > V_u , \text{OK.}$$

✓ Flexural reinforcement

$$M_u = w_u * L^2 / 8 = 39.844 * 1.2^2 / 8 = 7.17 \text{KN. m.}$$

$$\rho = \frac{0.85 * f'_c}{f_y} * \left( 1 - \sqrt{1 - \frac{2.61 * M_u}{b_w * d^2 * f'_c}} \right)$$

$$\rho = 0.0008$$

$$A_s = \rho * b_w * d = 0.0008 * 1000 * 160 = 128 \text{mm}^2 / \text{m}$$

$$A_s \text{ shrinkage} = .0018 * b_w * h = .0018 * 1000 * 200 = 360 \text{mm}^2 / \text{m} > A_s, K$$

So, use 1Ø12/250 mm

For top and transverse steel use  $A_s \text{ min}$  , use Ø12/250 mm.

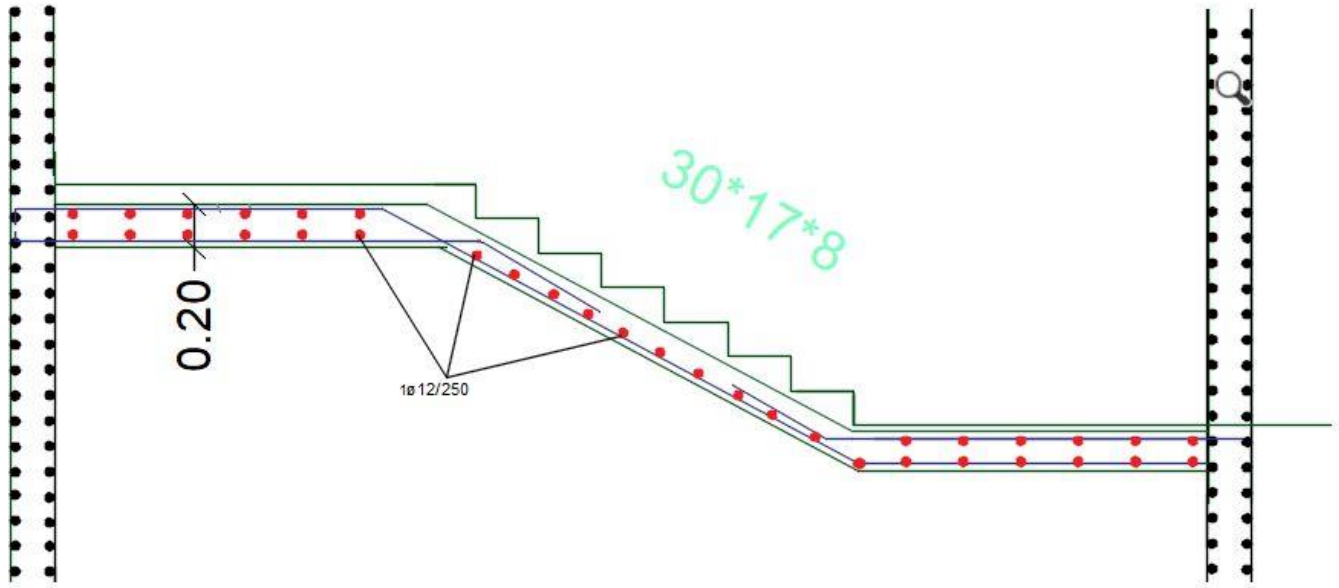


fig4-18: Reinforcement for staircase

## 6.REFERENCES

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- ASCE Committee, “American society of Civil Engineers, Minimum Design loads for Buildings and other Structures” American Society of Civil Engineers, Virginia 20191, 2010.

- ACI Committee 318 “Building Code Requirements for Structural Concrete (ACI 318M-11)” American Concrete Institute, Michigan, 2011.
- Reinforced concrete (I, II) notes.