Residential-Commercial Building

Graduation project report submitted to the faculty of
Civil Engineering Department
The Hashemite University

in partial fulfillment
of the requirements for the degree of
Bachelor's in civil engineering

By

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ABSTRACT

Structural Design is the second process after Architectural, it is the process that transfers the project from papers to reality and produce a structure capable of resisting all applied loads without failure during its intended life.

The Structural Design process is divided to 3 main phases, planning phase, Design and Construction phases and are defined as the following:

1- Planning phase: during this phase, we study the building layout, from its dimensions to the shape of it as all, prepare the basic structural plans and drawings and plan the schedule of the construction.

2- Design phase: Study the stability, strength and rigidity of the structure and ensure that the planned structure will be sufficiently strong to carry its intended load.

3- Construction phase: it's the stage that the planned project start becoming real.

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LIST OF SYMBOLS AND ABBREVIATIONS

- Ag= gross area of section, mm^2.
- As= area of non-prestressed tension reinforcement, mm^2.
- As'= area of compression reinforcement, mm^2.
- Av= area of shear reinforcement within a distance s, mm^2.
- b= width of compression face of member, effective compressive flange width of a T beam, mm.
- bw = web width, or diameter of circular section, in.
- bo = perimeter of critical section for two-way shear in slabs and footings, mm.
- d= effective depth, mm.
- d'= distance from extreme-compression fiber to centroid of compression reinforcement, mm.
- db= nominal diameter of a bar, mm.
- DL= dead loads.
- f'c= compressive strength of a concrete, MPa.
- fy= yield strength of a non prestressed reinforcement, MPa.
- h= thickness of a member, overall height, mm.
- jd= distance between the resultant of the internal compressive and tensile forces on a cross section, mm.
- Ln= clear span measured face to face of supports, mm.
- LL= live loads.
- S= spacing of shear reinforcement, mm.
- n= number of bars in a layer being spliced or developed at a critical section.
- Pu= axial force due to factored loads.
- qu= factored load per unit area.
- R= flexural resistance factor.
- t= thickness of wall in hollow section, mm.
- Vs= nominal shear strength provided by shear reinforcement, KN.
- Vu= factored shear force at section, KN.
- Wu= total factored load per unit length of beam or one-way slab.
- αs= coefficient used to compute Vc in slabs and footings.
- β= ratio of clear spans in long to short direction of two-way slabs.
- β= ratio of long side to short side of footing.
- β= ratio of long side to short side of column or concentrated load.
- ɤ= ratio of the distance between the outer layers of reinforcement in a column to the overall depth of the column.

- ε= strain.
- ρ= ratio of non-prestressed tension reinforcement.
- Φ= strength-reduction factor.
- Mn= total factored static moment, kN.m.
- Mu= moment due to factored loads.
- c= distance from extreme-compression fiber to neutral axis, mm.
- Cc= clear cover from the nearest surface in tension to the surface of the flexural tension reinforcement, mm.
- c1= size of rectangular or equivalent rectangular column, capital, or bracket, measured parallel to the span L1, mm.
- c2 = size of rectangular or equivalent rectangular column, capital, or bracket, measured parallel to L2, mm.
- Ld = development length, mm.
- Ldh= development length of standard hook in tension, measured from critical section to outside end of hook, mm.

Chapter 1 : Introduction

1.1 General Information

Steel, Aggregate and Cement paste are the main ingredients of Concrete, the main building material.

Concrete is strong in both compression and tension (Thanks to the steel bars inside), durable and cost effective, that’s why it is considered number 1 when it comes to building materials.

Concrete strength varies because of the different mixes and ratios between the materials used, and can be measured by using either cylindrical or cubic specimen (Depends upon the code adapted by the country). The strength of concrete should be much higher than the service loads applied to assure safety of the building.

- The Resident-Comm Building consists of 5 Floors divided as the following:

- The Ground floor, has a resident-commercial area of (520m2), includes 7 shops, a studio, electricity controlling devices, pump room and security housing areas.
- First to third floor are symmetric, each has a residential area of $558.45m^2$, each consists of 6 small studios and 2 master rooms
- Roof, includes 2 stores with total area of $275m^2$

- The building was built in UAE
- Flooring is designed on flat slab system
- As overall, the design included the following structural components:

  (Flat slabs, beams, stairs, shear walls and footings)
- All structural elements will be designed based on the maximum values of the ultimate loads resulted from load combinations specified by the ACI code.

1.2 Objectives

The main idea of this project is to design all the structural elements of a Residential-Commercial building located in the UAE.

Table 1.1: Different areas of the building

Ground Floor	$452.58m^2$
First Floor	$522m^2$
Second Floor	$522m^2$
Third Floor	$522m^2$
Roof	$275m^2$
Staircases	$25m^2$

1.3 Motivations and Importance

Flat Slab system is a unique designing system is an R.C slab built monolithically with the supporting columns and reinforced in two or more directions. the following are the main advantages of flat slab construction:

1.the floor system requires lesser depth and thus there will be a reduction in storey height

2.there is a reduction in the dead load and foundation loads since the overall weight and storey height are decreased.

3.flat slab construction has improved fire resistance as compared to other types of floor systems

4. it is better able to withstand concentrated loads.

We think it will teach us useful experience and techniques.

1.4 Methodology and Implementation

- Analysis by the ETABS, SAFE and Prokon programs to obtain design moments and shear and torsion on each element of the project
- Implement the engineering techniques and approaches to determine the required dimensions of the elements, as well as the reinforcement needed.
- Design was done considering the maximum moment along the element.

1.5 Codes, books, and programs

1. Building Code Requirements for Structural Concrete and Commentary ((ACI 318M05)).
2. The Jordanian's Forces and Loads Code.
3. Reinforced concrete Mechanics and Design-by JAMES MACGREGOR.
4. AUTOCAD.
5. SAFE (design of slab and foundation).
6. ETABS (Modeling & design of column and shear wall).
7. Prokon (design of beams).

Chapter 2 : Structure Description and Analysis

In this chapter we will discuss some elements of our project and how it behaves when it loaded and in the end of chapter a modeling of our project by ETABS program and will be added here.

2.1 Structural System Components

Each chapter will usually be broken down into major sections. Each section should have its own header as shown here.

2.2 Structural Elements

All structural systems are composed of elements.

The following are considered to be the primary elements in a structure:

1- **truss**:
is a structure that consists of two-force members only, where the members are organized so that the assemblage as a whole behaves as a single object. A "two-force member" is a structural component where force is applied to only two points. Although this rigorous definition allows to have any shape connected in any stable configuration, trusses typically comprise five or more triangular units constructed with straight members whose ends are connected at joints referred to as nodes.

Fig.2.1: Truss

2- **Beam**:
those members that are primarily subjected to flexural forces. They usually are thought of as being horizontal members that are primarily subjected to gravity forces.

3- **Column**:
those members that are primarily subjected to axial compression forces. A column may be subjected to flexural forces also. Columns usually are thought of as being vertical members, but they may also be inclined.

4- **foundation**:
is the element of a structure which connects it to the ground, and transfers loads from the structure to the ground .Foundations provide the structure's stability from the ground:
To distribute the weight of the structure over a large area in order to avoid overloading the underlying soil, to anchor the structure against natural forces including earthquakes, floods, frost heaves, tornadoes and wind. To provide a level surface for construction. To anchor the structure deeply into the ground, increasing its stability and preventing overloading.

5- **concrete slab**:

It is an essential structural element in concrete buildings, consisting of a flat, horizontal surface made of cast concrete. Steel-reinforced slabs, typically between 100 and 500 mm thick, are most often used to construct floors and ceilings.

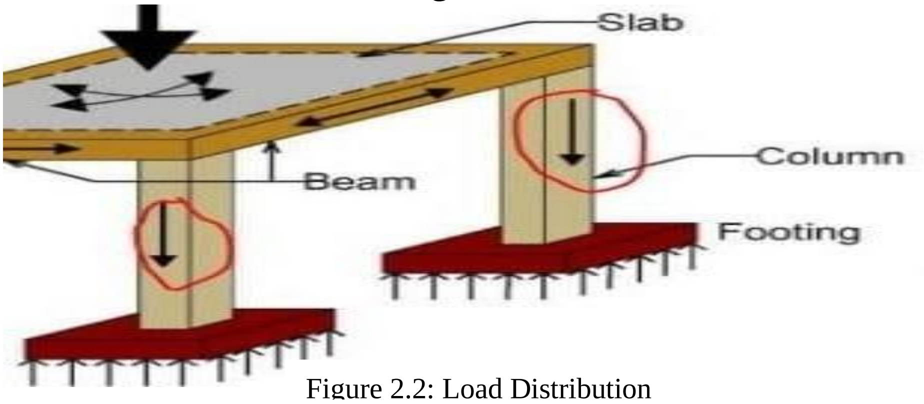


Figure 2.2: Load Distribution

6- **shear wall**:

is a vertical element of a seismic force resisting system that is designed to resist in-plane lateral forces, typically wind and seismic loads. in many countries, the Building Code and Residential Code govern the design of shear walls.

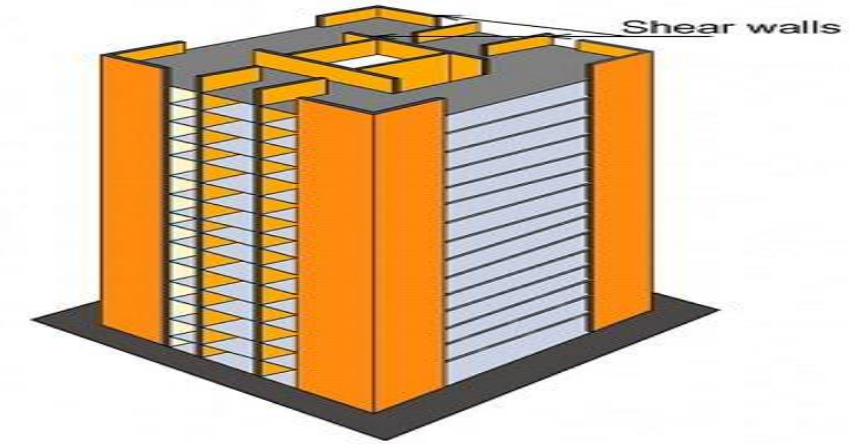


Figure 2.3: Shear Wall

7- **stair:**

is a construction designed to bridge a large vertical distance by dividing it into smaller vertical distances, called steps (used for leading from one floor to another). Stairs may be straight, round, or may consist of two or more straight pieces connected at angles.

Figure 2.4: Stair

2.3 Modeling

We Make modeling using ETABS program.

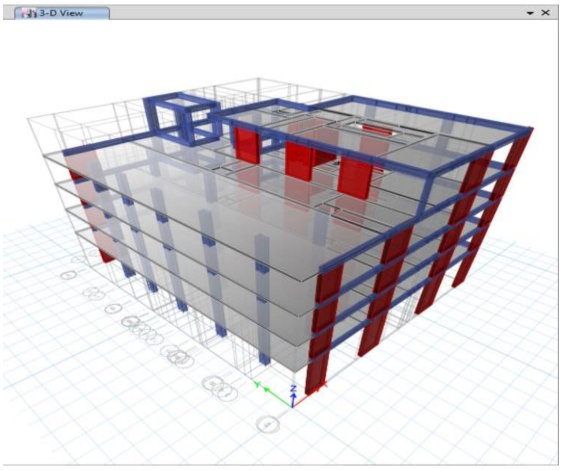


Figure 2.5: ETABS 3D Modeling 1

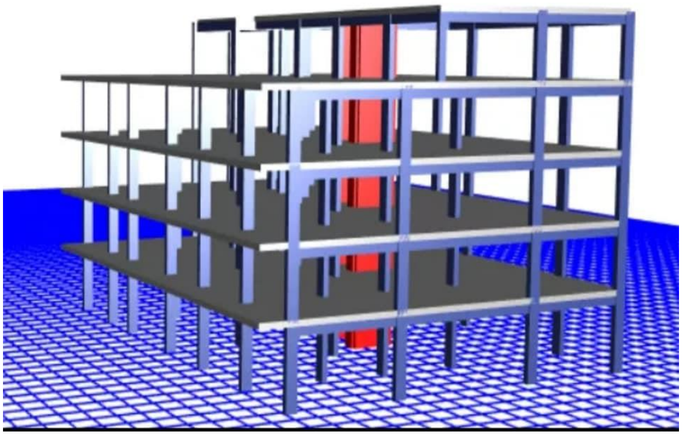


Figure 2.6: ETABS 3D Modeling 2

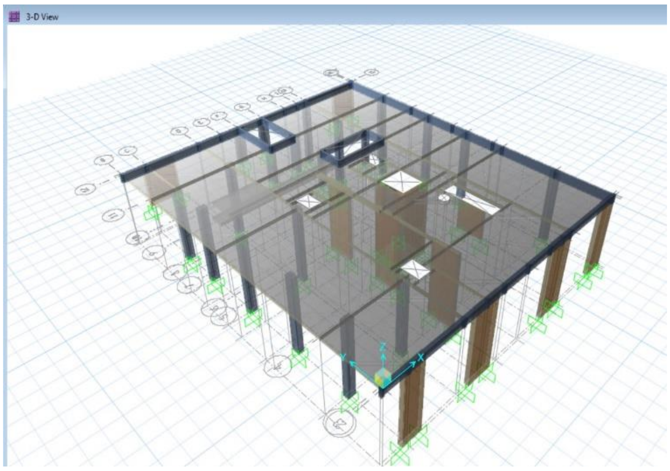


Figure 2.7: SAFE Typical Slab Modeling

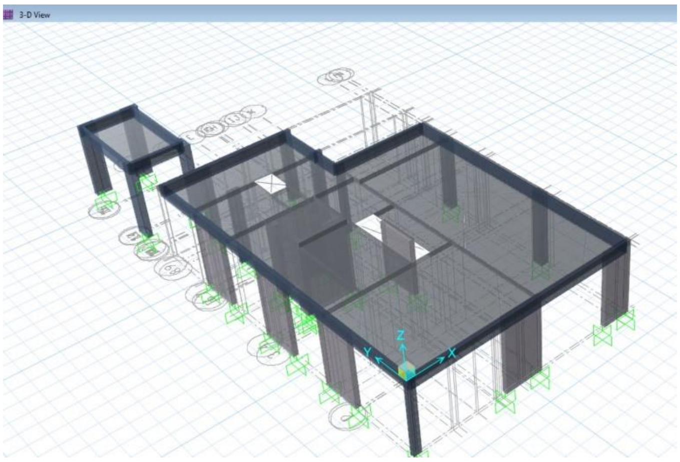


Figure 2.8: SAFE Roof Slab Modeling

Chapter 3 : Slabs

In this chapter we will analyze and design the slab, finding needed area of steel to resist the moment and load and needed steel to resist shear and we will check the deflection and punching shear and solve any problem.

3.1 Slab type

Flat Slab: -

It is a type of slabs that is spread in building systems around the world, usually in halls and large rooms. The most important advantage is that it provides large distances between the columns to the next room and the adjustment inside the building because there are no beams.

The flat slabs are slabs that rest directly on columns.

Architecturally, it is better because there are no beams, and it is preferable to use it in the case of a large area, and it is also used in the case of large loads. The wooden tension is easier to implement because it is horizontal and there are no beams. The thickness of the tile is relatively large, due to its resistance the(-ve) moment and punching shear. The amount of reinforcement steel is relatively large in comparison with Solid slab.

Figure 3.1: Real flat slab with drop panels

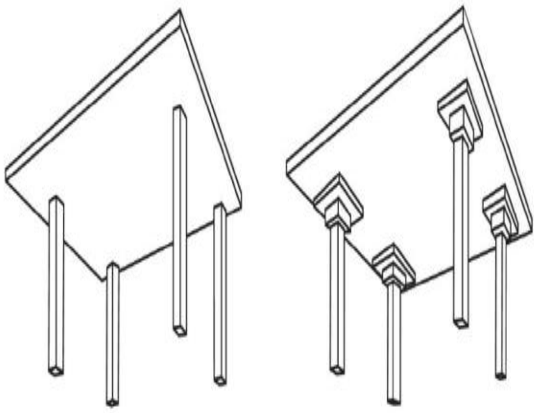


Figure 3.2: Flat Slab

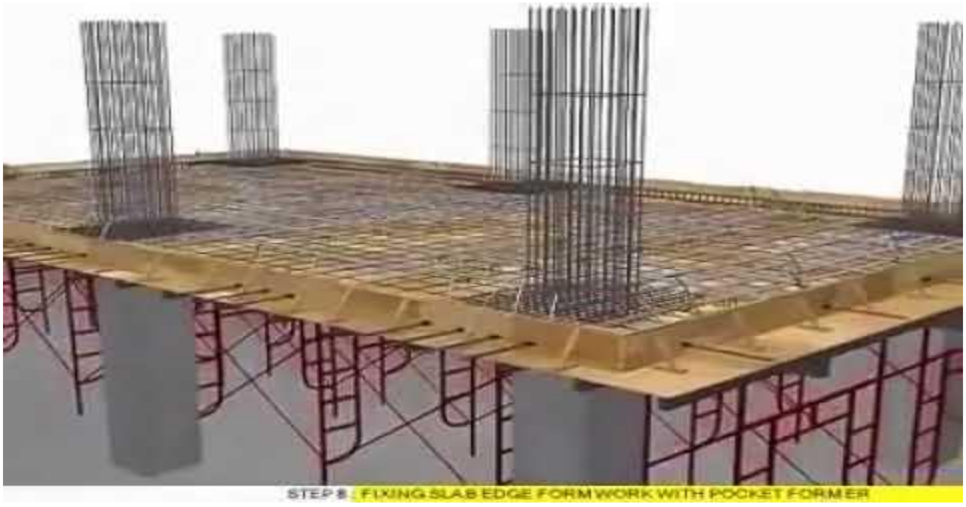


Figure 3.3: construction of Flat Slab

In our project we used this type of slab for several things; -

1) The building contains civil defense services, and therefore it is forbidden to have a drop beam in the center because it will affect the services on the roof.

2) Having the beam will reduce the net floor height

3) There is central air conditioning inside the building which will also reduce the floor height.
For example, if we assume that the height of the floor is 4 meters and the beam is 60 cm, the central air conditioning system will be done under the beam, and thus there will be only about 3 meters of net floor height remaining.

4) The distribution of the columns is irregular and thus it is difficult to use the beam between rooms.

3.2 Loading

Type of load in our project:

In the typical floor we have five type of load:

1 - Dead load 2- live load

3- super imposed dead load 4- wall load (partition allowance)

5- snow load

In the roof floor we have four loads

1 - Dead load 2- live load

3- super imposed dead 4- isolation system load

1. Dead load:

The dead load calculated by safe program internally.

2. Live load:

Suggestion live load where taken from the Jordanian code.

الأحمال الحية للأرضيات والعقدات

نوع المبنى		الاستعمال	الحمل الموزع	الحمل المركز البديل
عام	خاص	الاشغال	كن/م²	كن
المباني الخاصة والسكنية.	النوع الأول : مباني الشقق السكنية التي لا يزيد ارتفاعها عن ثلاثة طوابق ولا يزيد عدد الشقق التي يمكن الوصول إليها من خلال درج مشترك عن أربع شقق للطابق الواحد.	جميع الغرف بما في ذلك غرف النوم والمطابخ وغرف الغسيل وما شابه ذلك (All Usages).	2.0	1.4
	النوع الثاني : المباني التي لا ينطبق عليها ما ورد في النوع الأول و البيوتات والمباني المخصصة لاقامة الضيوف.	غرف النوم.	2.0	1.8
		الحمامات.	2.0	–
		الطعام وردهات الاستراحة والبلياردو.	2.0	2.7
		الممرات والمداخل والأدراج و . . . سطحات الأدراج والممرات المرتفعة الموصلة بين المباني.	3.0	4.5
		المطابخ وغرف الغسيل.	3.0	4.5

figure 3.4: Live Load from Jordanian code

3. Wall load (partition allowance):

- **Partitions allowance Calculations**:

There are 200m of block (25cm) (IN THE TYPICAL FLOOR)

There are 205m of block (25cm) (Ground Floor)

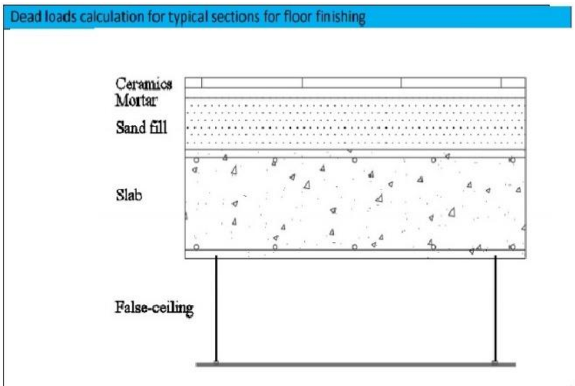
Height of walls=3.5m

Total area of floor= 520m^2

Load (m^2) = $\frac{(3.5 * 14 * 0.25) * 200}{520}$ = 4.7 KN/m² so use 5 kN/m²

Wall load = 5 kN/m²

As we see we have different value of loads, thus we divided the slab according to the loads.



SL NO	LEGAND	DESCRCRIPTION	SDL (KN/M²) DESCRCRIPTION	VALUE	TOTAL	LIVE LOAD (KN/M²)
1		Bed Room , pantry,lobby	FINISH THICK 100 mm	2.5	8.0	2.0
			SERVICES & CEILING	0.5		
			PARTITION ALLOWANCE	5.0		
2		corridor	FINISH THICK 100 mm	2.5	3.0	3.0
			SERVICES & CEILING	0.5		
			PARTITION ALLOWANCE	0.0		
3		Bath Room	FINISH THICK 220 mm	4.5	10.0	2.0
			SERVICES & CEILING	0.5		
			PARTITION ALLOWANCE	5.0		
4		Stair Case	FINISH THICK 100 mm	3.0	3.0	3.0
			SERVICES & CEILING	0.0		
			PARTITION ALLOWANCE	0.0		
5		Roof Area	FINISH THICK 100 mm	2.5	5.5	2.0
			SERVICES & CEILING	0.5		
			ISOLATION SYSTEM	2.5		

Figure 3.5: Load distribution table

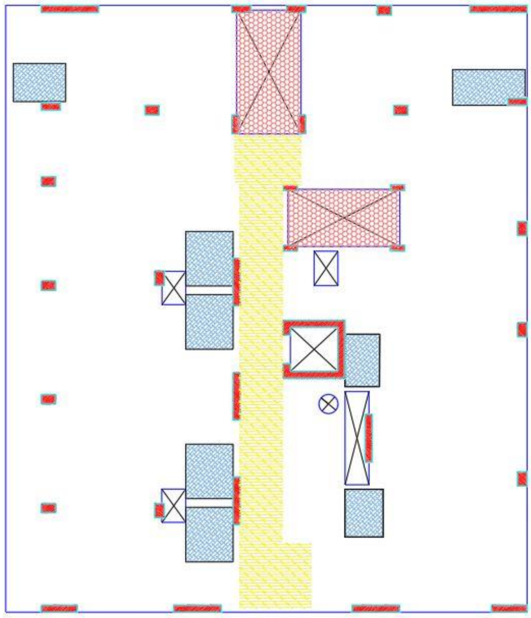


Figure 3.6: Load distribution on slab

Putting loads and dividing them on the slab using "Safe" program:

1.All slab:

The wall load is similar for all slab is:

Wall load =5 kN/m^2

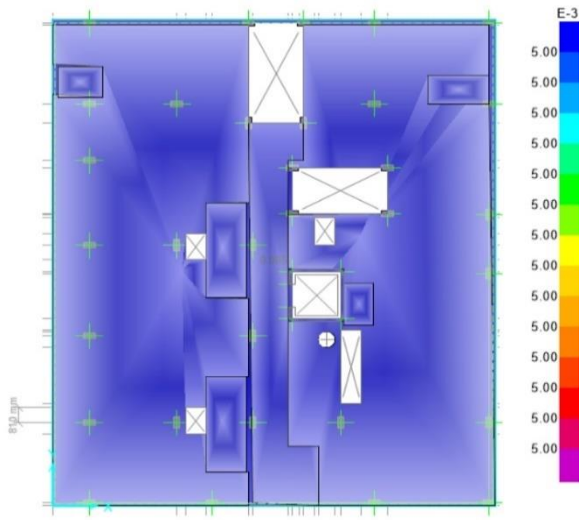


Figure 3.7: Safe Load distribution in all slab

2. Corridor:

In the below we see the photo assigned load on corridor.

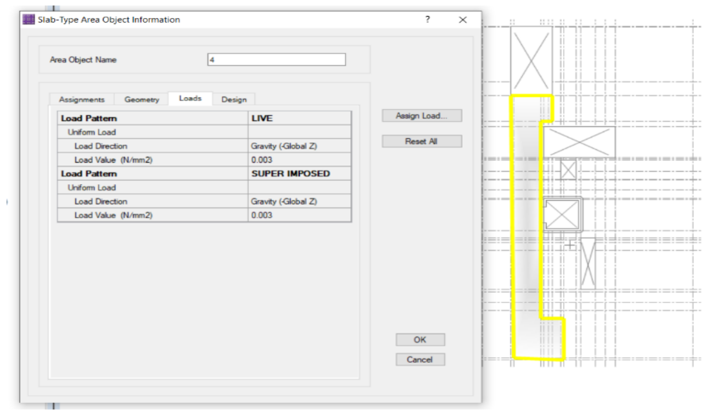


Figure 3.8: Safe Load distribution in corridor

Live load =3 kN /m^2

Super imposed =3 kN /m^2

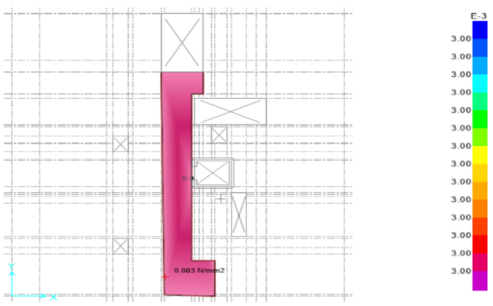


Figure 3.9: Safe live Load in corridor

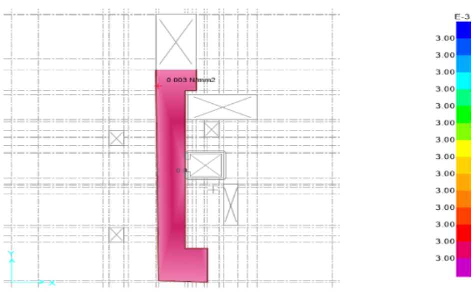


Figure 3.10: Safe super imposed Dead Load in corridor

3. Bedroom, pantry and lobby:

All the load on lobby, pantry and bedroom shown in the photo.

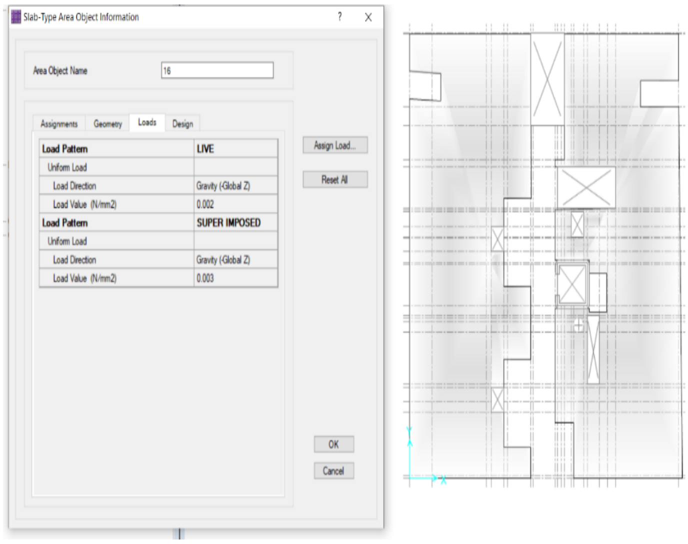


Figure 3.11: Safe Bed room load distribution

Live load =2 kN $/m^2$

Super imposed =3 kN $/m^2$

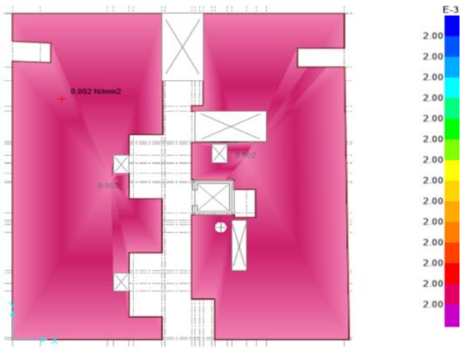


Figure 3.12: Safe Live load in bed room

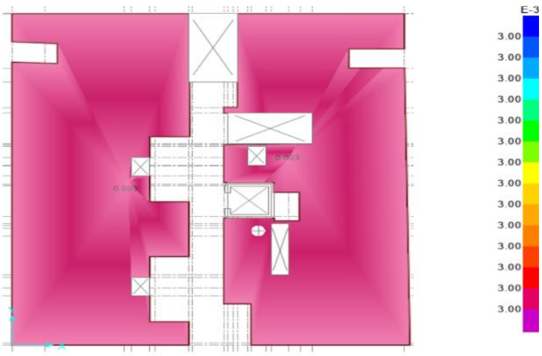


Figure 3.13: Safe super imposed Dead Load in bed room

4. Bath room:

All load on bathroom shown in this photo.

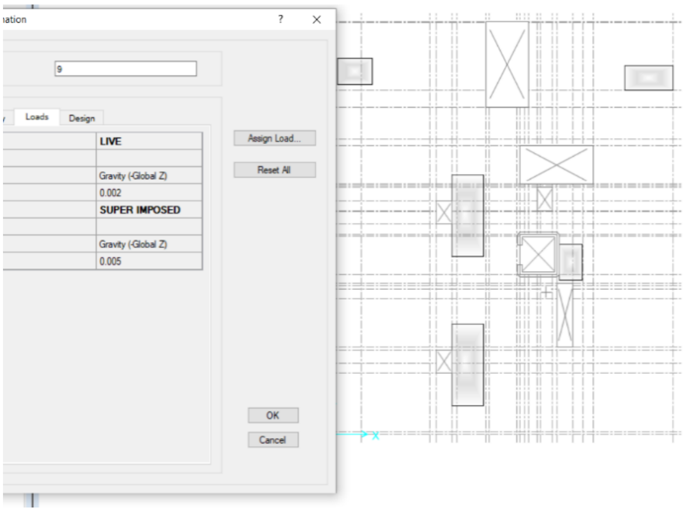


Figure 3.14: Safe Bath room load distribution

Live load =2 kN $/m^2$ Super imposed =5 kN $/m^2$

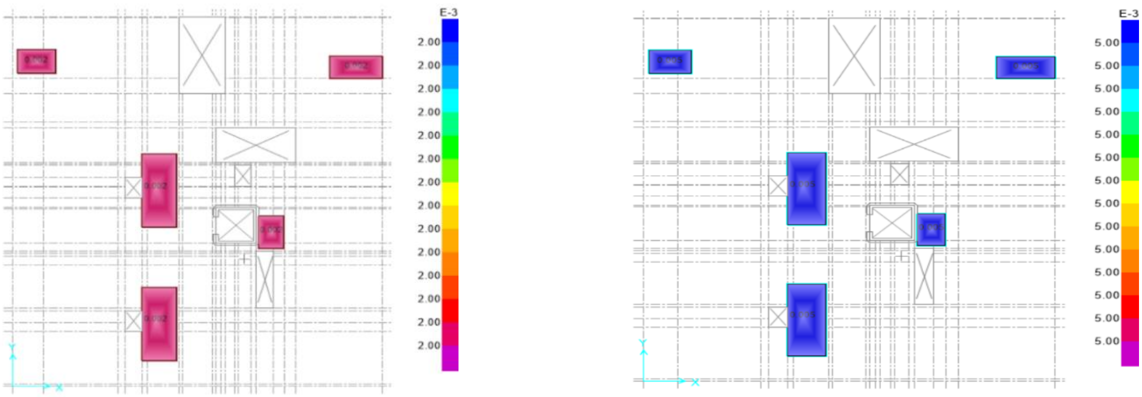


Figure 3.15: Safe Live load and super imposed Dead Load in bath room

SL NO	LEGAND	DESCRCRIPTION	SDL (KN/M²) DESCRCRIPTION	VALUE	TOTAL	LIVE LOAD (KN/M²)
1		Bed Room , pantry,lobby	FINISH THICK 100 mm	2.5	8.0	2.0
			SERVICES & CEILING	0.5		
			PARTITION ALLOWANCE	5.0		
2		corridor	FINISH THICK 100 mm	2.5	3.0	3.0
			SERVICES & CEILING	0.5		
			PARTITION ALLOWANCE	0.0		
3		Bath Room	FINISH THICK 220 mm	4.5	10.0	2.0
			SERVICES & CEILING	0.5		
			PARTITION ALLOWANCE	5.0		
4		Stair Case	FINISH THICK 100 mm	3.0	3.0	3.0
			SERVICES & CEILING	0.0		
			PARTITION ALLOWANCE	0.0		
5		Roof Area	FINISH THICK 100 mm	2.5	5.5	2.0
			SERVICES & CEILING	0.5		
			ISOLATION SYSTEM	2.5		

Figure 3.16: Summary of all load on whole slab

3.3 Calculations and Part Selections:

Importance Note: **we use "direct design method" but actually this method is invalid because its didn't fulfill the requirements.(more than in Appendix B)**

For manual calculation, we choose **second floor** slab to design it

The slab has exterior panel (edge beam).

We want to design column strip and middle strip between column K4 and I4 in W – E direction.

Some information about slab:

Thickness 250mm = 25 cm F′c=28 MPa

Fy=460 MPa Edge beam 20 X 60 cm

All column 60 X 30 cm

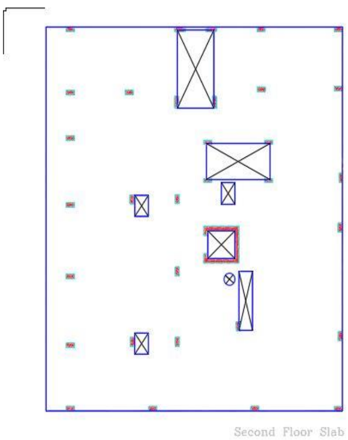


Figure 3.17: Slab section

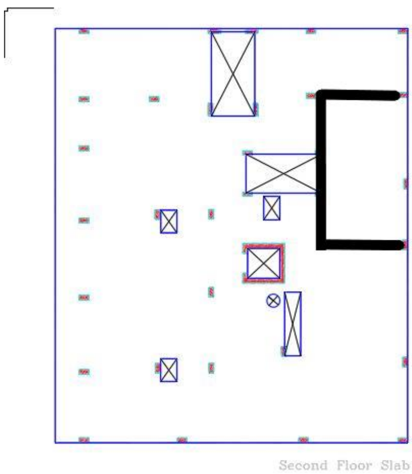


Figure 3.18: slab section with black corner

According to table 9.5 c in ACI code

we assume slabs do **not** have drop panels and only have edge beam.

Minimum thickness of two way slabs to control deflection

17

TABLE 9.5(c)—MINIMUM THICKNESS OF SLABS WITHOUT INTERIOR BEAMS*

f_y, MPa†	Without drop panels‡			With drop panels‡		
	Exterior panels		Interior panels	Exterior panels		Interior panels
	Without edge beams	With edge beams§		Without edge beams	With edge beams§	
280	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$	$\ell_n/36$	$\ell_n/40$	$\ell_n/40$
420	$\ell_n/30$	$\ell_n/33$	$\ell_n/33$	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$
520	$\ell_n/28$	$\ell_n/31$	$\ell_n/31$	$\ell_n/31$	$\ell_n/34$	$\ell_n/34$

*For two-way construction, ℓ_n is the length of clear span in the long direction, measured face-to-face of supports in slabs without beams and face-to-face of beams or other supports in other cases.
†For f_y between the values given in the table, minimum thickness shall be determined by linear interpolation.
‡Drop panels as defined in 13.2.5.
§Slabs with beams between columns along exterior edges. The value of α_f for the edge beam shall not be less than 0.8.

Figure 3.19: Table 9.5 (c)

$$h_{min} = \frac{483}{33} = 14.69 \text{ cm}$$

$$h_{min} = \frac{335}{33} = 10.152 \text{ cm}$$

$$h_{min} = \frac{504}{36} = 14 \text{ cm}$$

$$h_{min} = \frac{400}{36} = 11.11 \text{ cm}$$

$$h_{min} = \frac{776}{36} = 21.56 \text{ cm}$$

$$h_{min} = \frac{180}{36} = 5 \text{ cm}$$

$\boldsymbol{h_{min}}$ **= 21.56 then we take it = 25 cm**

S.D.L = 8 KN/m^2 , L.L = 2 KN/m^2, $\gamma_{concrete}$ = 24 KN/m^2

Wu = 1.2(0.25 x 24 + 8) + 1.6 x 2 = 20 KN/m^2

d (effective depth) = 250-20-12 = 218mm

Check #1: one-way shear

we make all shear test in column K4

$$V_{u1} = \left(\frac{4.83}{2} + \frac{1.15*3.35}{2}\right) \text{ x } \left(\frac{1.15*5.04}{20} - \frac{0.3}{2} - 0.218\right) \text{ x } 20$$

V_{u1} = 219.667 KN

$\Phi Vc = 0.75\ (\frac{1}{6} \text{ x } \sqrt{28} \text{ x } (4.83 \text{ x } \frac{1000}{2} + 1.15 \text{ x } 3.35 \text{ x} \frac{1000}{2}) \text{ x } 0.218$

ΦVc = 663.17 KN

ΦVc > Vu1 ⇒ OK

<u>Check #2: two-way shear</u>

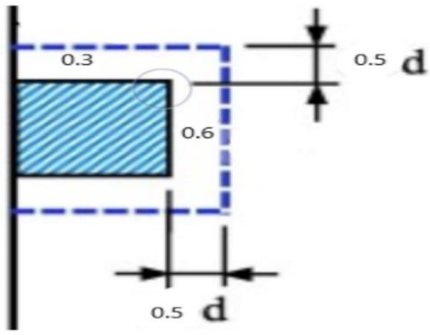


Figure 3.20: column two way shear section

0.6+0.218 = 0.818

0.3+0.218 = 0.518

Vu = 20*((4.83+1.15*3.35*5.04/4)-(0.818*0.518))

. = 210.32 KN

$\beta = \frac{0.6}{0.3} = 2$, b.=2(818+518) = 2672 mm

V_c is taken as the smallest of (a), (b) and (c) where

(a) $V_c = \left(2 + \frac{4}{\beta_c}\right)\frac{\sqrt{f_c'}\, b_o d}{12}$

(b) $V_c = \left(\frac{\alpha_s d}{b_o} + 2\right)\frac{\sqrt{f_c'}\, b_o d}{12}$

(c) $V_c = \frac{1}{3}\sqrt{f_c'}\, b_o d$

a) Vc= 1148.7Kn
b) Vc = 1511.5
c) **Vc= 1148.7Kn <u>min</u> (we use it)**

ΦVc= 0.75*1148.7 = 861.5 KN

ΦVc > Vu SAFE punching shear

****Static Moment**

M. = $\frac{W_u L_2 \, Ln^2}{8}$ = 20*(4.83/2 + 3.352) * 5.04²/8

M. = 259.73 KN

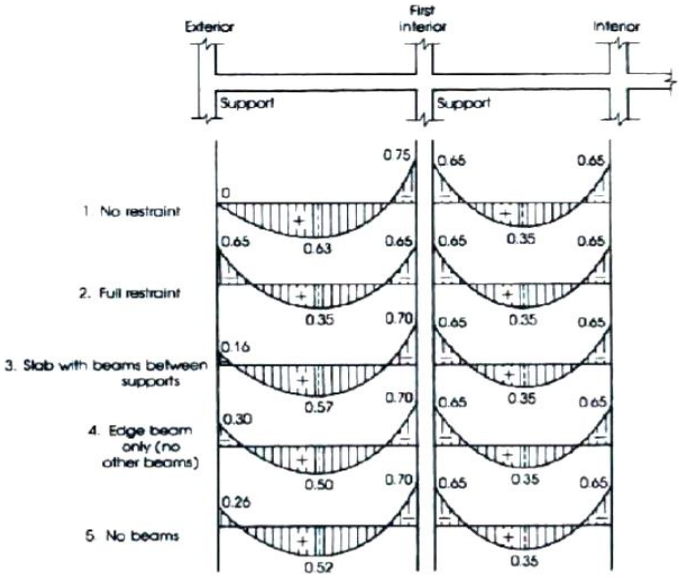


Figure 3.21: moment distribution for beams

Num 4 edge beam only

Interior negative = 0.65 x 259.73 = 168.82 KN.m

Interior positive = 0.35 x 259.73 = 90.91 KN.m

Negative moment for column strip = 0.75 x 168.82 = 126.62 KN.m

Negative moment for middle strip = (1-0.75) x 168.82 = 42.205 KN.m

Positive moment for column strip = 0.6 x 90.91 = 54.55 KN.m positive moment for middle strip = (1-0.6) x 90.91 = 36.36 KN.m

Percentage of Longitudinal Moment in Column Strips, Exterior Panels (ACI Code, Section 13.6.4)

	$\alpha_{f_1} l_2/l_1$	β_t	Aspect Ratio l_2/l_1 0.5	1.0	2.0
Negative moment at exterior support	0	0	100	100	100
		≥2.5	75	75	75
	≥1.0	0	100	100	100
		≥2.5	90	75	45
Positive moment near midspan	0		60	60	60
	≥1.0		90	75	45
Negative moment at interior support	0		75	75	75
	≥1.0		90	75	45

Figure 3.22: longitudinal moment in column strip ACI section 13.6.4

**Design for column strip

* positive moment 54.55 KN

As = $\frac{Mu*10^6}{\Phi\ fy*j*d} = \frac{54.55*10^6}{0.9 \text{ x } 0.95 \text{ x } 460 \text{ x } 218}$ = 636.23 mm^2

a = 636.23 x 460 / 0.85 x 28 x 840 = 14.64 mm

As = 54.55 x 10^6 / 0.9 x 460 x (218-14.64/2) = 625.42 mm^2

As_{min} = 0.0018bh = 0.0018 x 840 x 250= 378 mm^2

As > As_{min} OK

From table steel we will choose 12 mm bar diameter

Number of bars 6 and$As_{provided}$ = 679 mm^2

ΦMn = ΦAsFy (d-a/2) = 0.9*679*460*(218-11.71/2)

= 59.64 KN.m > 54.55 **OK**

* negative moment 126.62 KN.m

As= 1476.8 mm^2 , a=27.2 , As= 1496.31 , As > As min

From table steel we will choose 12 mm bar diameter

Number of bars 14 and$As_{provided}$ = 1583.363 mm^2

**Design for middle strip

<u>*positive moment 36.36 KN.m</u>

As=424.1 mm^2 ,a=7.01mm , As=409.5 mm2

As$_{min}$=420.75mm^2

As$_{min}$ > As not ok

We take As min = As = 420.75 mm2

From table steel we will choose 12 mm bar diameter,

<u>Number of bars 4 and$As_{provided}$ = 452 mm^2</u>

<u>*Negative moment</u>

As = 114.77 mm^2 , a=1.9 mm , As=469.68 mm^2

As > As$_{min}$

From table steel we will choose 12 mm bar diameter,

<u>Number of bars 4 and $As_{provided}$= 452 mm^2</u>

3.4 Slab analysis and Final Design

Design of slab:

Related to choose the flat slab system, we need to check the deflection, Punching Shear then we will find the suitable design for the Slab.

⇒ **Typical slab (From First Slab –Fourth Slab)**

- Deflection
  - Deflection checked on long term deflection
  - From ACI CODE table 9.5 (b)

Allowable Deflections

- ACI Table 9.5(b) = max. permissible computed deflection

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)†	$\ell/480$‡
Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections		$\ell/240$§

*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 3.23: ACI Table 9.5 (b)

- The longest Span From Project L=15.2 m
  So to the case 3 from above table,
  permissible deflection= $\frac{L}{480}$= $\frac{15.2}{480}$ =0.032m

- From “SAFE” program the value of long-term deflection shown in the picture.

long term deflection = 6 cm

6 cm > 3.2 cm ⇒ so, it’s not safe

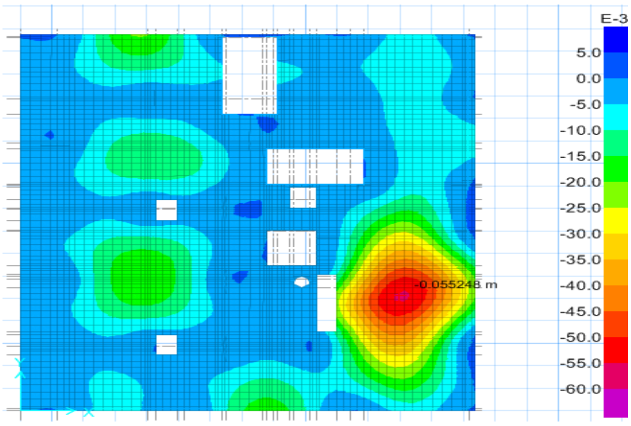


Figure 3.24: SAFE longterm deflection before add column

- So we can solve this problem by many ways but, finally we find the best way by adding new Shear Wall (we checked to ensure that’s identical with architectural drawing)

- (before)

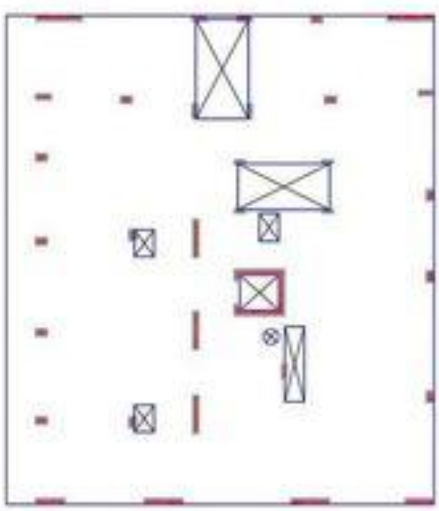


Figure 3.25: section of slab before add column

(after)

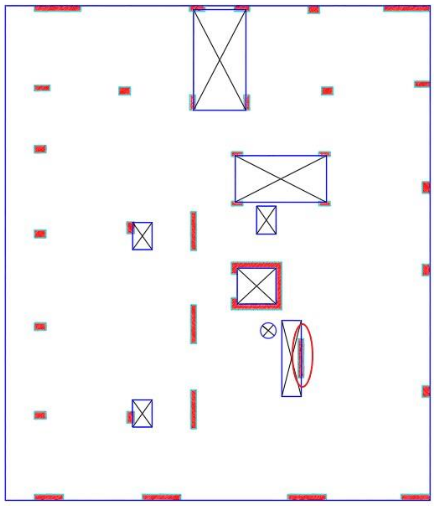


Figure 3.26: section of slab after add shear wall

- After that we checked the deflection again and we found that: deflection = 1.74 cm = permissible deflection

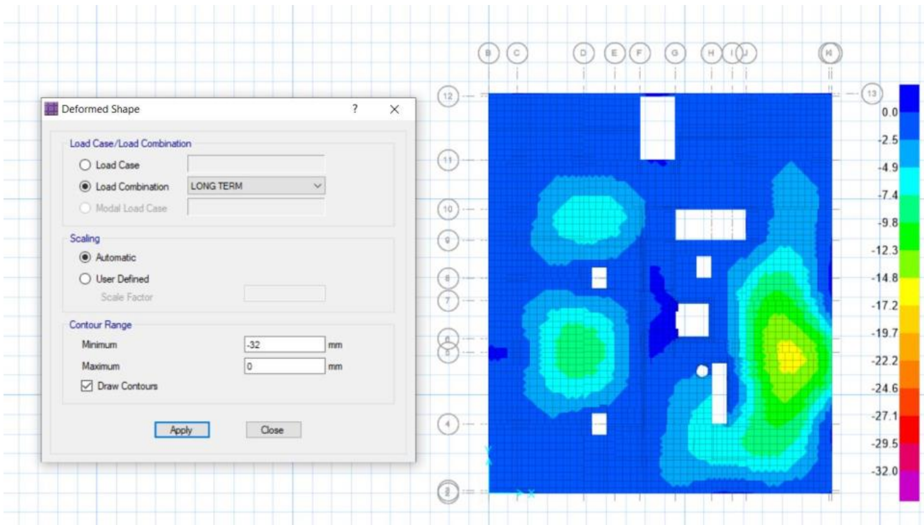


Figure 3.27: SAFE longterm deflection after add shear wall

-Punching shear

- The results of Punching Shear are shown in the figure below, and we notice that the columns which circled have the biggest value of punching shear.

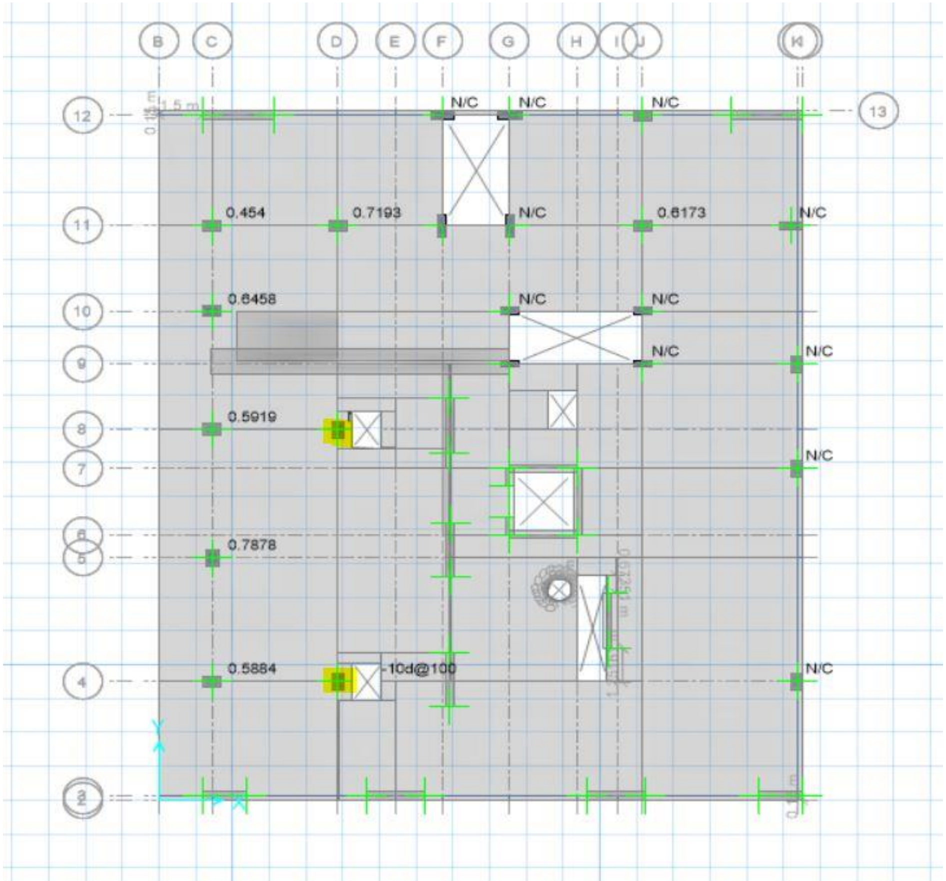


Figure 3.28: punching shear column

- To overcome of checking the punching shear is a failure problem, there are many ways to increase the capacity of column to resist the punching shear of concrete slabs:
  <u>Solution 1</u> increasing slab thickness: we refuge to this solution as a last one because we have five slabs with large area, and increasing the thick 1 cm will cost a lot.
  <u>Solution 2</u> Providing drop panel and/or column capital:
  may it will be economical, but we can't provide it because it would be against the Architectural restrictions.
  <u>Solution 3</u> Using shear studs:
  This system consists of double-headed studs welded or spliced to flat rails, positioned around the head of column or the base. The studs are welded to the rail at centers. its designed and determined by using Safe program, its suitable for all column shapes and its

installed by specialists sub-contractor, but this solution the most suitable alternative with our situation, Due to that, shear stud will be used.

- ☒ From 8 columns that have punching shear we take the most critical column, and we design shear stud related to this column, the remaining columns will be reinforced like selected column to ensure more safety.

- ☒ From the Fig, we have 7 legs every leg has 6 studs with 10 cm as spacing
(stud diameter = 10 mm).

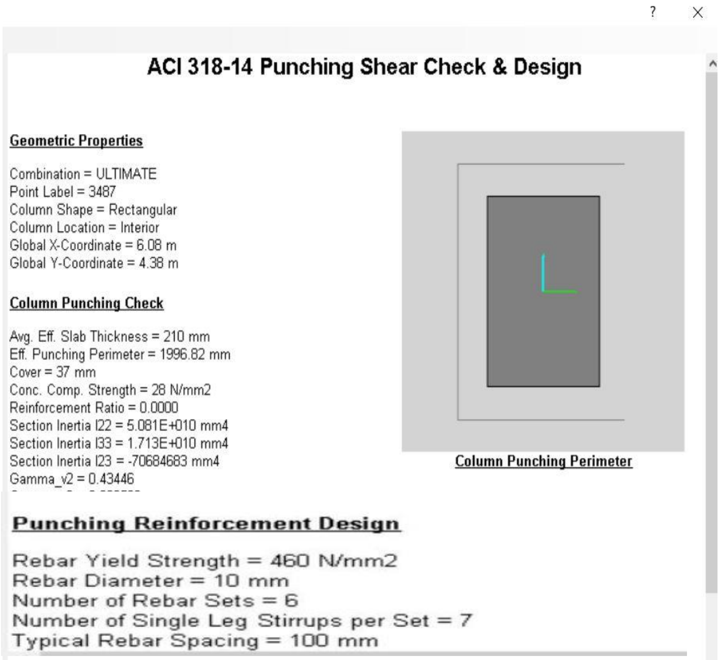


Figure 3.29: SAFE shear stud

- ☒ But we will use 10 leg to get most uniform distribution on every edge.

- ☒ The design of shear stud
Is shown in the Fig.

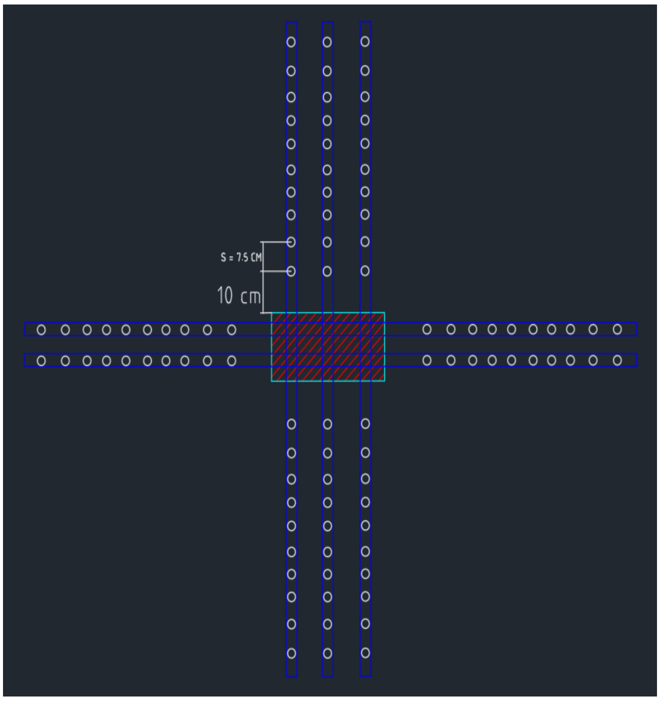


Figure 3.30: Design of Shear stud

- <u>Steel Reinforcement</u>

The value of moment at:

a) X- direction on all slab and every strip

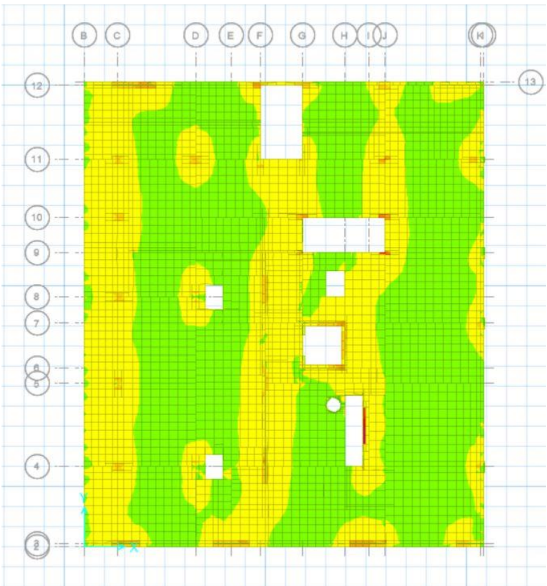


Figure 3.31: Value of moment in X axis

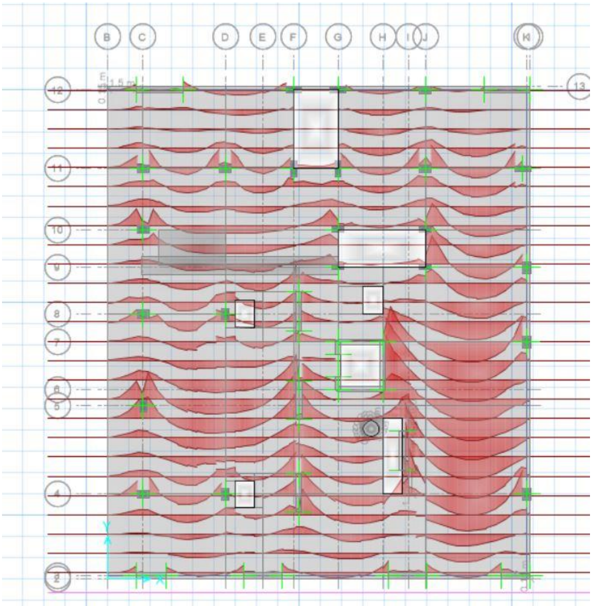


Figure 3.32: moment diagram in X axis

b) Y – direction on all slab and every strip

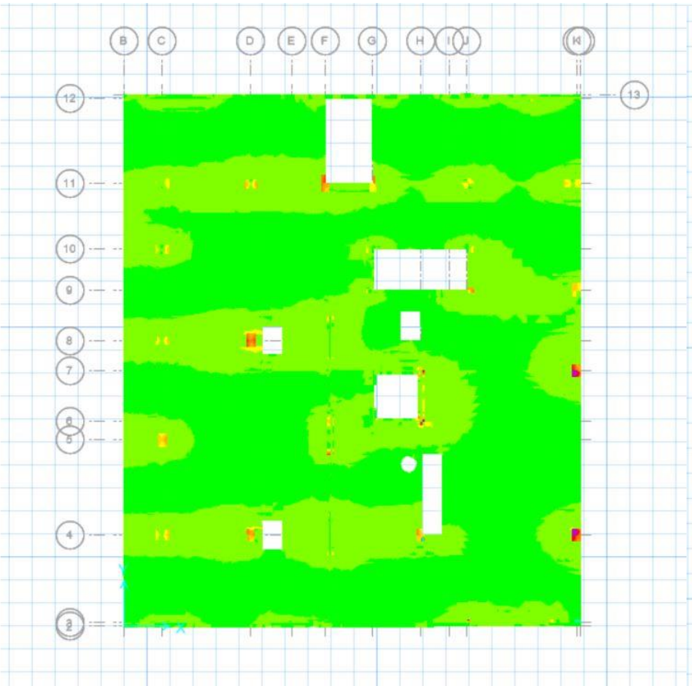


Figure 3.33: Value of moment in Y axis

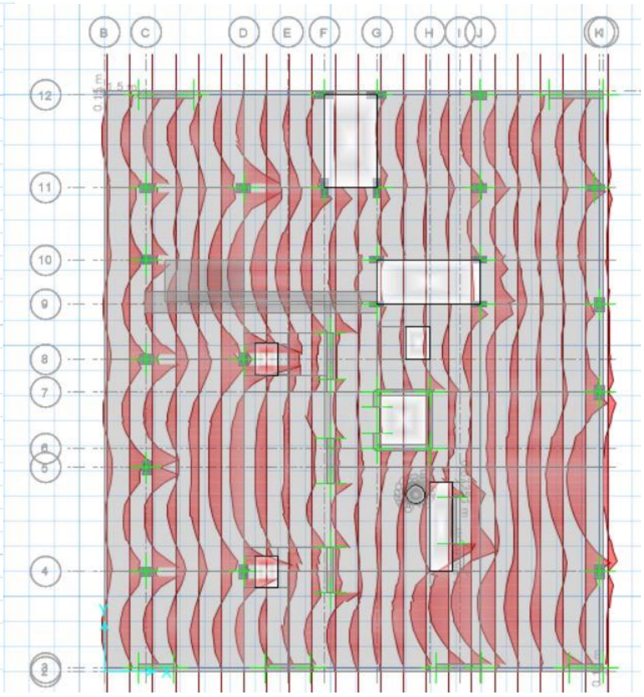


Figure 3.34: moment diagram in Y axis

⇒Needed area of steel to resist the moment

Positive moment and Negative moment (at mid span)

Use Φ12 , A = 113 mm^2

-Min area of steel (As_{min})

As_{min} = 0.0018bh = 0.0018*250*1000

As_{min} = 450 mm^2

- Number of steel bar = $\frac{450}{113}$ = 3.9

- So we try 4 Φ12

$$As = \frac{M_u}{0.9*Fy*J_d} \Rightarrow 0.45 = \frac{M_u}{0.9*460*0.95*0.25}$$

$$M_u = 40\ KN.m$$

we use the safe program to introducing the Moment as shown:

X – direction

as we see in picture 4 Φ 12 Not enough

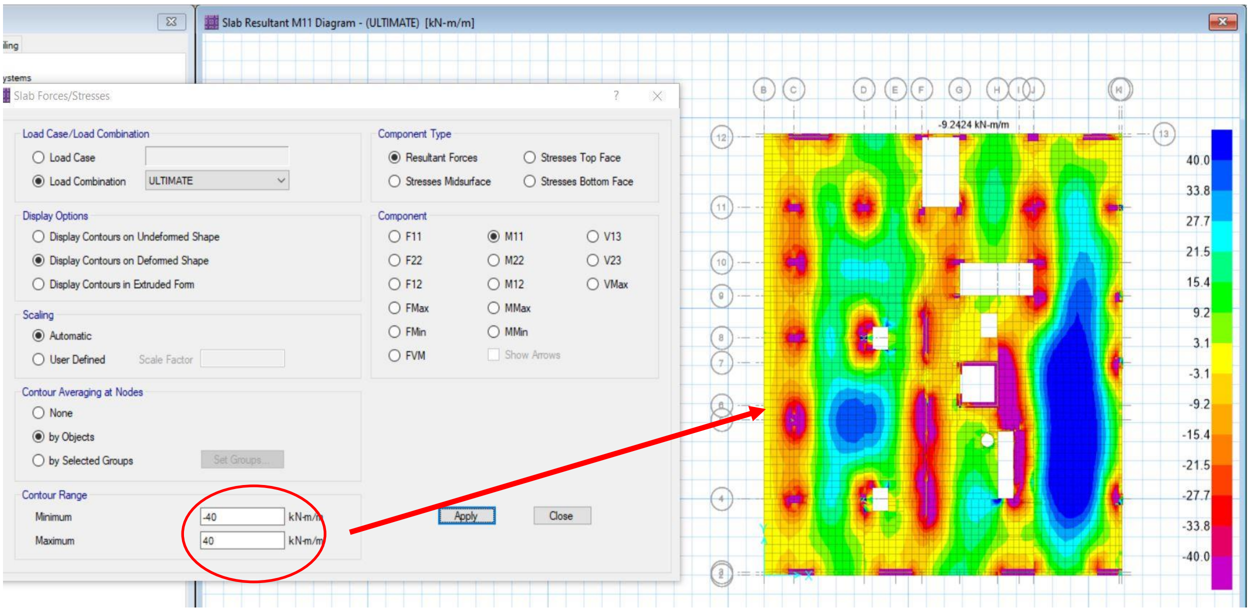


Figure 3.35: reinforcement Steel in X axis not enough

Y – direction

from picture 4Φ12 is not enough

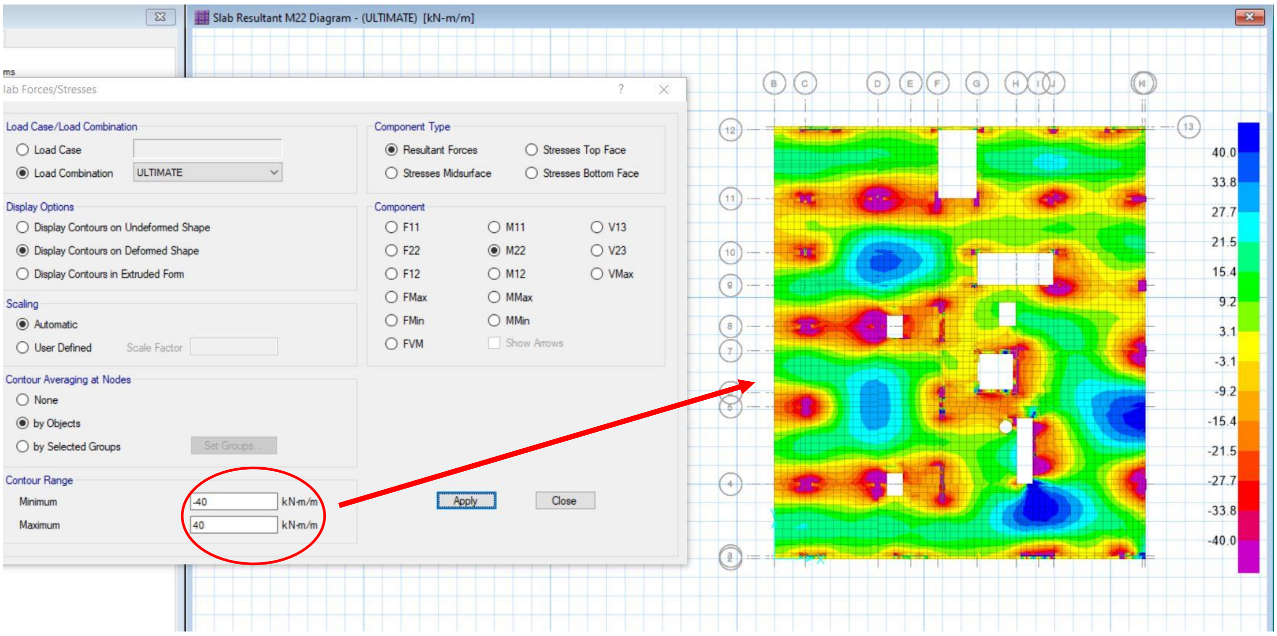


Figure 3.36: reainforcement Steel in Y axis not enough

The iterations for suitable area of steel:

2.try 5 Φ 12

A=5413 , As= 565 mm^2

$$0.565 = \frac{M_u}{0.9*460*0.95*0.25}$$

$$M_u = 50\ KN.m.$$

<u>X – direction</u>

From picture 5 Φ 12 not enough

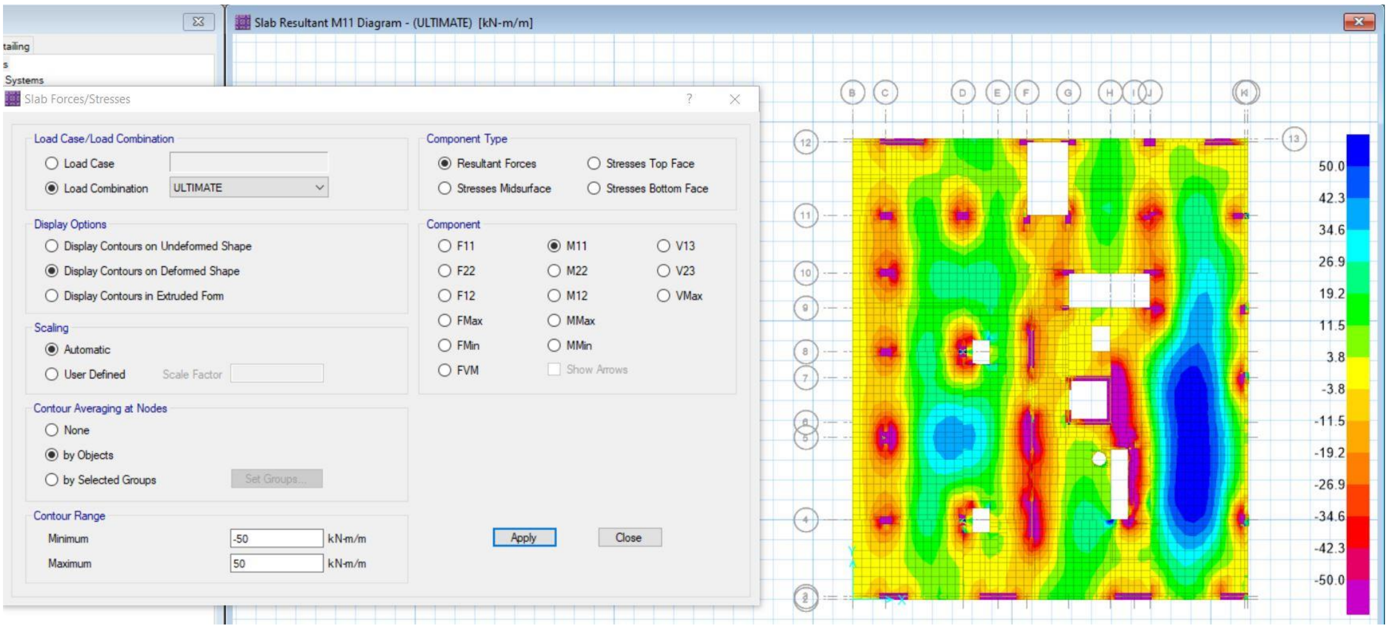


Figure 3.37: reainforcement Steel in X axis not enough around coloumn

<u>Y – direction</u>

From picture 5Φ12 not enough.

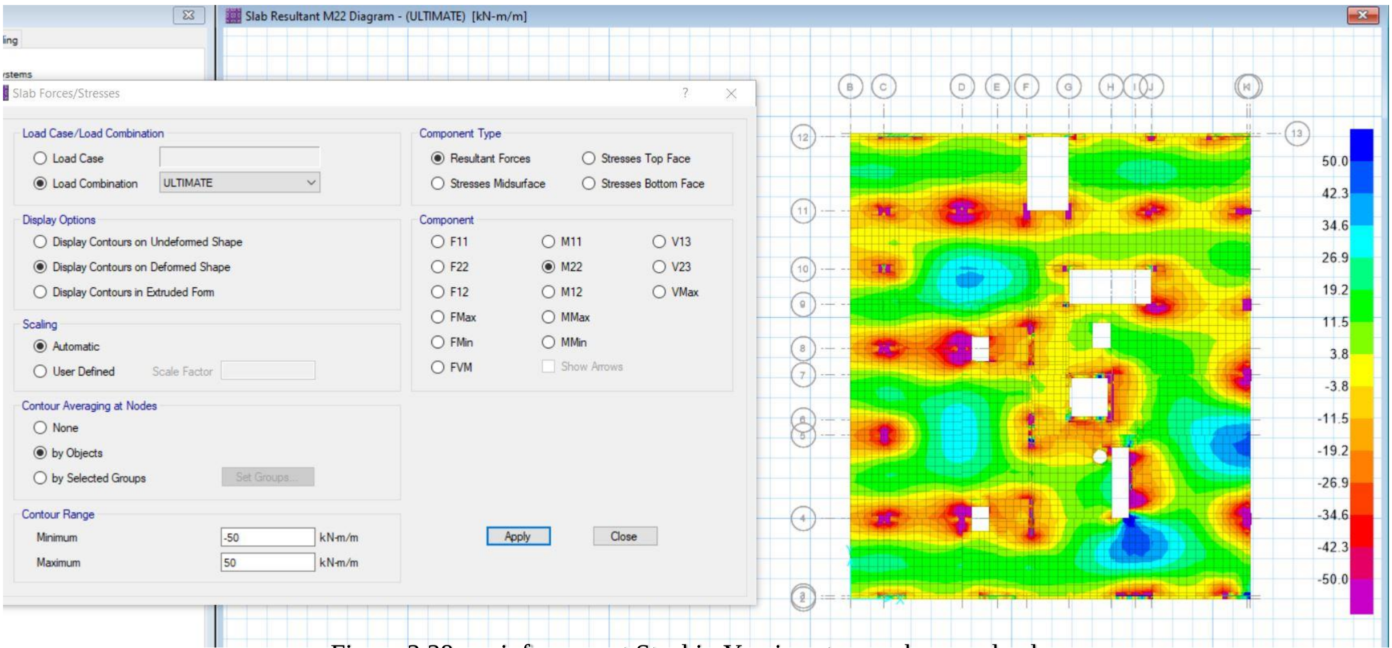


Figure 3.38: reainforcement Steel in Y axis not enough around coloumn

3. try 6 Φ12

A= 6*113 ⇒ A = 678 mm^2

$$0.678 = \frac{M_u}{0.9*460*0.95*0.25}$$

$$M_u = 60\ KN.m$$

X–direction

6Φ12⇒enough

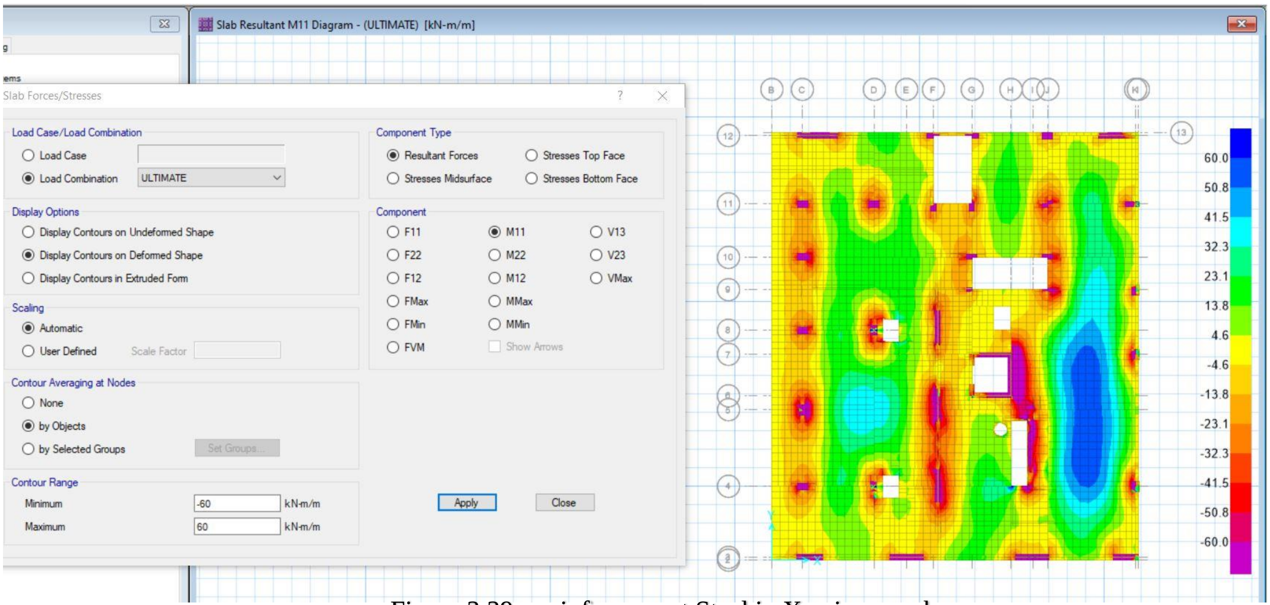


Figure 3.39: reainforcement Steel in X axis enough

Y–direction

6Φ12 ⇒ enough in the all slab

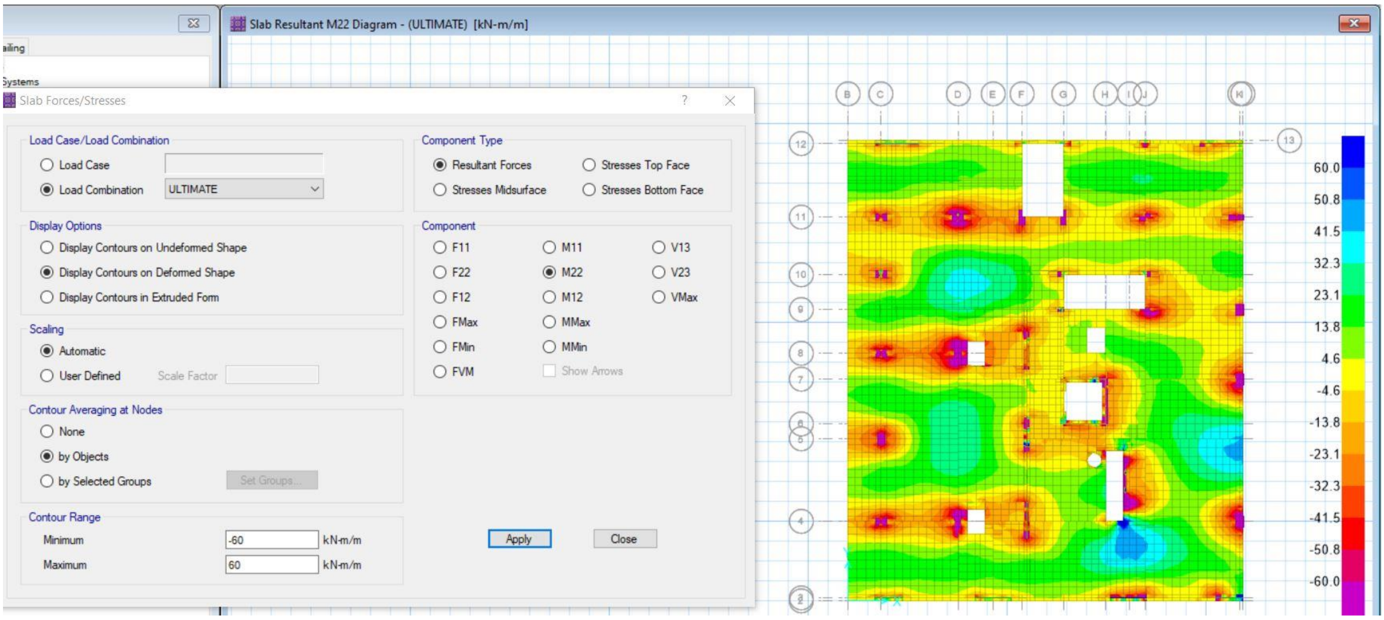


Figure 3.40: reainforcement Steel in Y axis enough

Around Columns (negative Moment):

Around the column we use Φ14 to decrease the number of steel bar in 1 m (strip).

Try 9Φ14

$$A = 9*154 = 1380\ mm^2$$

$$1.380 = \frac{M_u}{0.9*460*0.95*0.25}$$

$$M_u = 120\ KN.m$$

X – direction

9Φ14 ⇒ enough

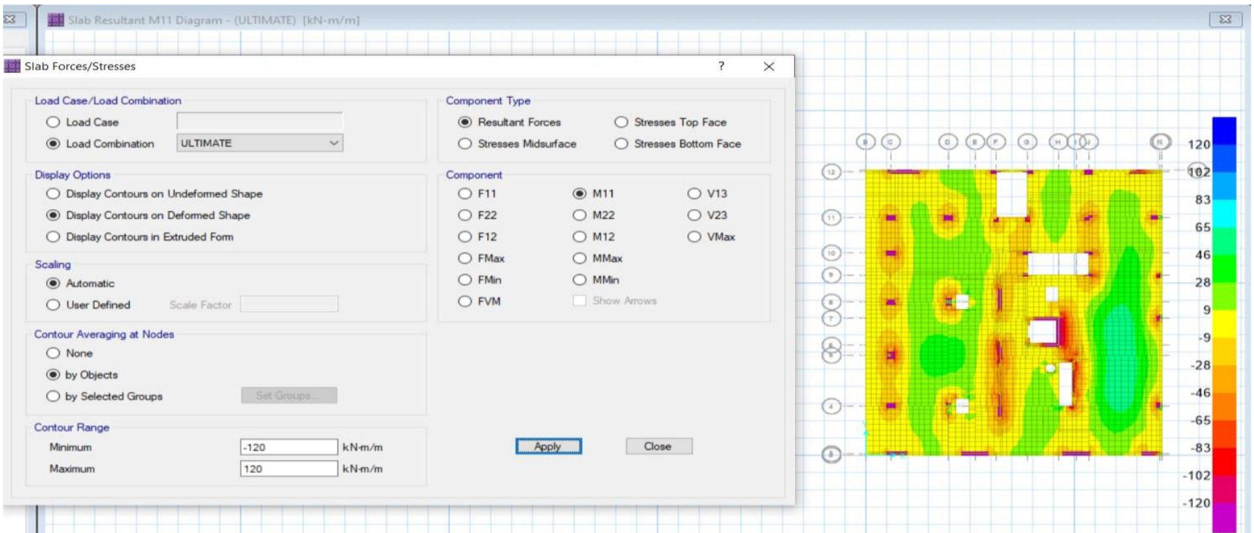


Figure 3.41: reinforcement Steel in X axis enough around coloumn

Y – direction

9Φ14 ⇒ enough

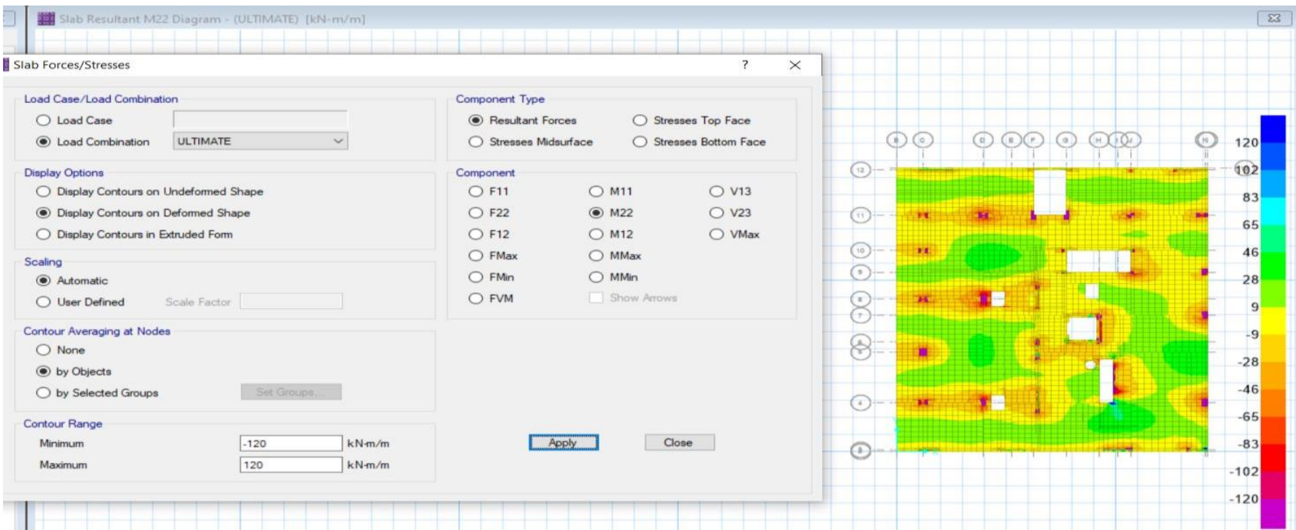


Figure 3.42: reinforcement Steel in Y axis enough around coloumn

Check S_{max}

S_{max} = smaller of $\begin{cases} 2\,h = 2 * 250 = 500\,mm \\ \quad 500\,mm \end{cases}$

actual spacing for 7 bar = 14 cm < S_{max} (50 cm) OK
actual spacing for 6 bar = 17 cm < S_{max} (50 cm) OK
actual spacing for 9 bar = 11 cm < S_{max} (50 cm) OK

Check S_{min}

$$S_{min} = \max of \begin{cases} Bar\ diameter & = 14\,mm \\ 25\,mm & = 25mm \\ diameter\ of\ vibrat\ or & (unknown) \\ 1.33 \max C.A\ size & (unknown) \end{cases}$$

Actual spacing for 6 bars = 160mm > 25mm (OK)
Actual spacing for 7 bars = 140mm > 25mm (OK)
Actual spacing for 9 bars = 110mm > 25mm (OK)

Check As_{max}

$$Asmax = \ 0.319\beta\ \frac{f'c}{Fy}\ bd = 5317\ mm$$

For 6 Φ 12

As = 678 mm <5317 (OK)

For 7 Φ 12

As =791 mm <5317 (OK)

For 9 Φ 14

As =1385 mm <5317 (OK)

⇒**Roof**

1. Deflection

Permissible deflection = 3.2 cm

From “SAFE” program

actual deflection = 1.74 cm

1.74 < 3.2 ⇒ its OK

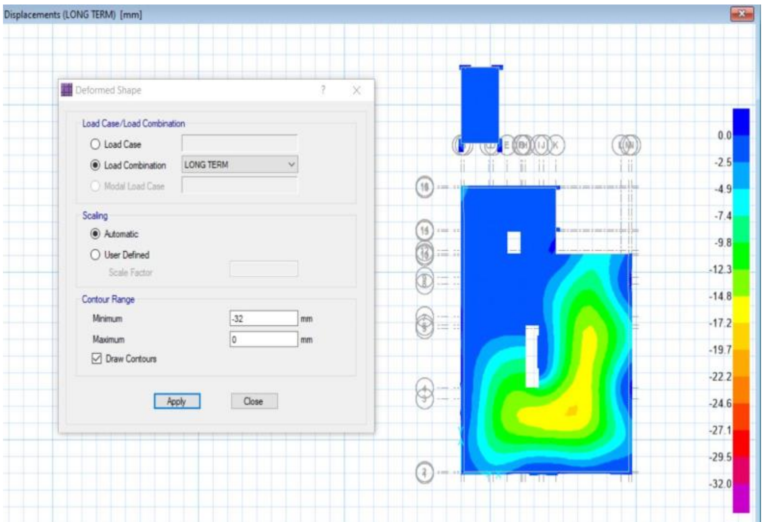


Figure 3.43: deflection roof

2. Punching Shear

all columns good & don’t have punching shear.

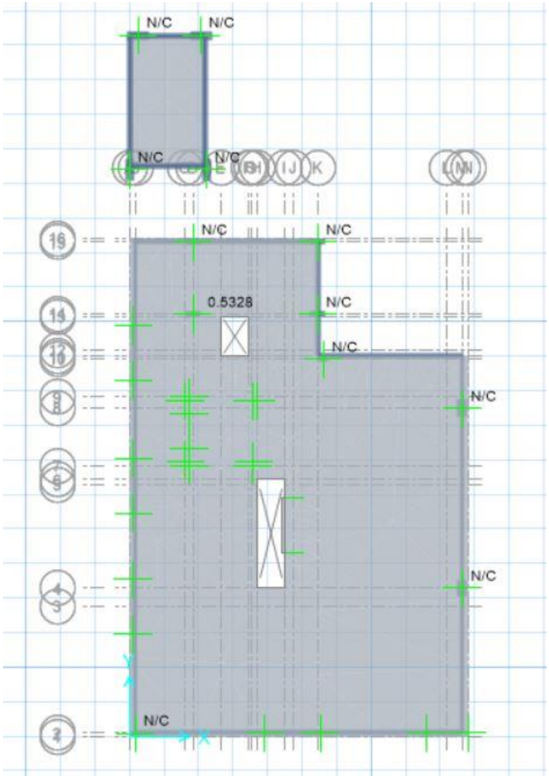


Figure 3.44: punching shaer roof

3. Steel reinforcement

The value of moment:

X – direction: on all slab and on every strip

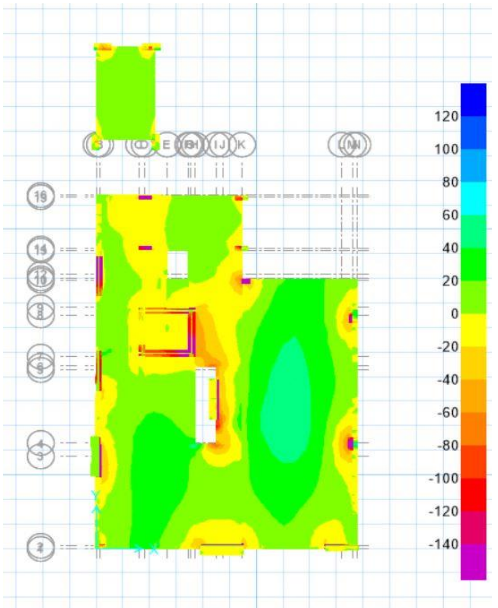


Figure 3.45: moment value in X axis roof

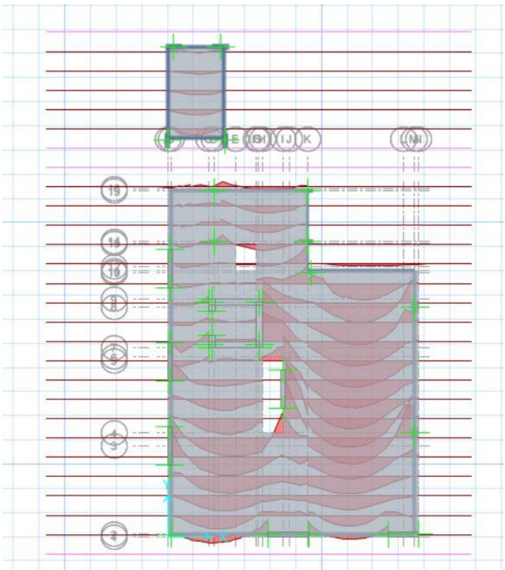


Figure 3.46: moment diagram in X axis roof

Y – direction: on all slab and on every strip

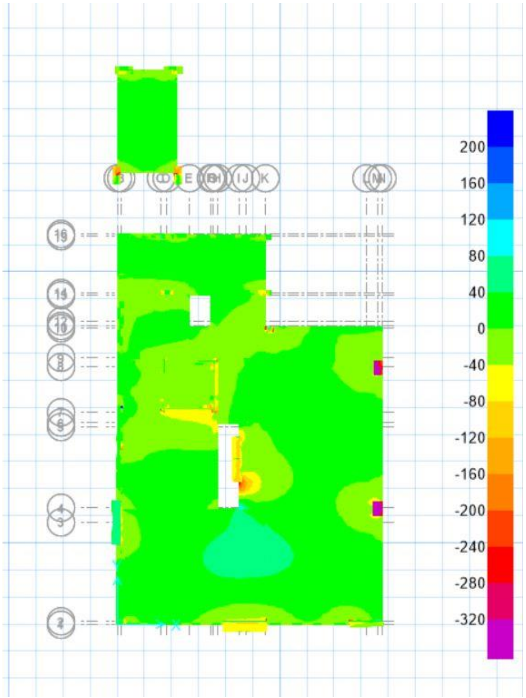


Figure 3.47: moment value in Y axis roof

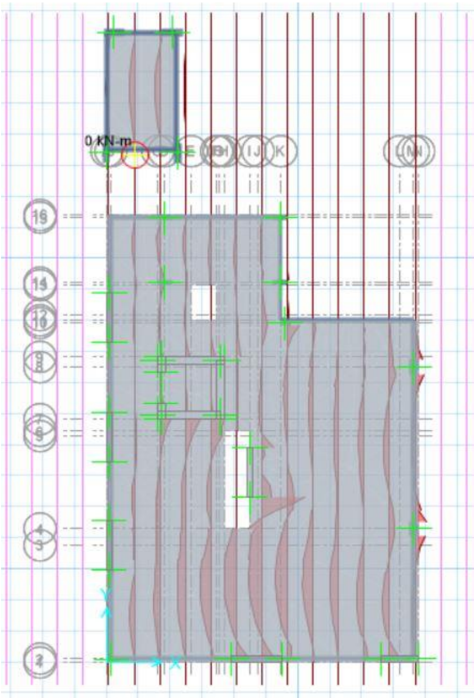


Figure 3.48: moment diagram Y axis roof

⇒Needed area of steel to resist the moment

- Positive moment + Negative moment (at mid span)

Try 4Φ12

$$M_u = 40KN.m$$

- X – direction

  4 Φ 12 not enough

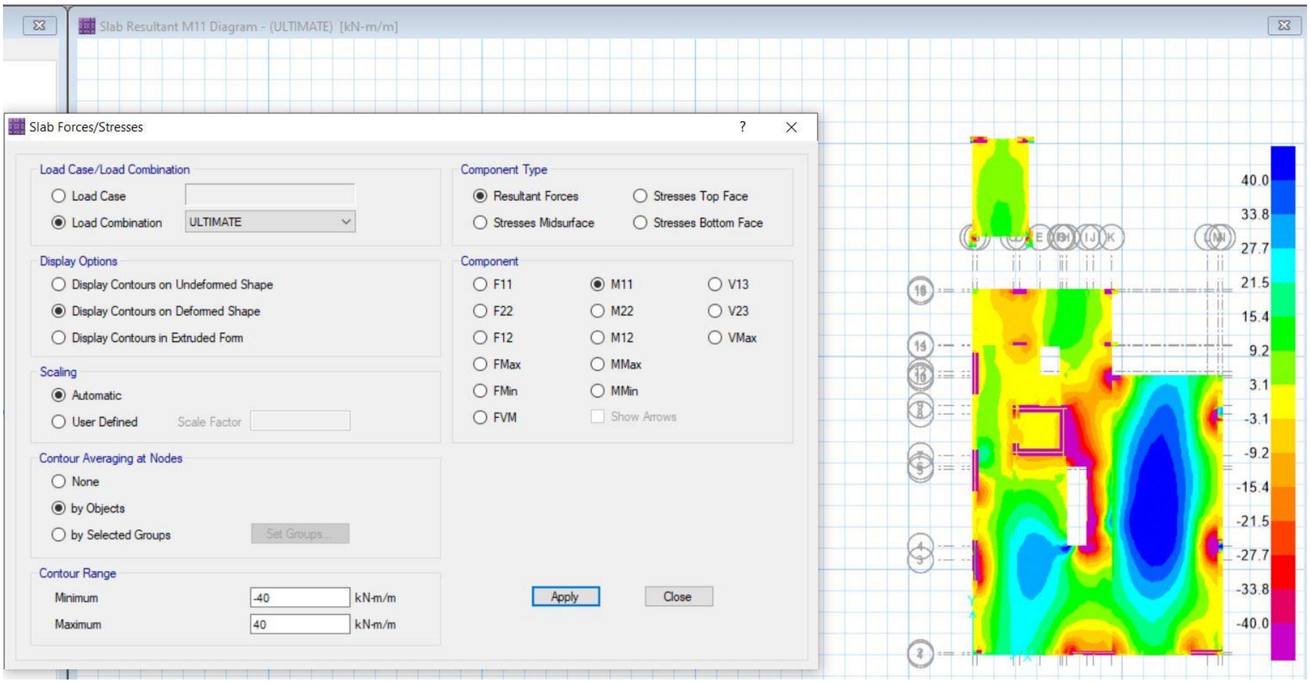


Figure 3.49: reainforcement Steel in X axis1

- Y – direction

  4 Φ 12 not enough

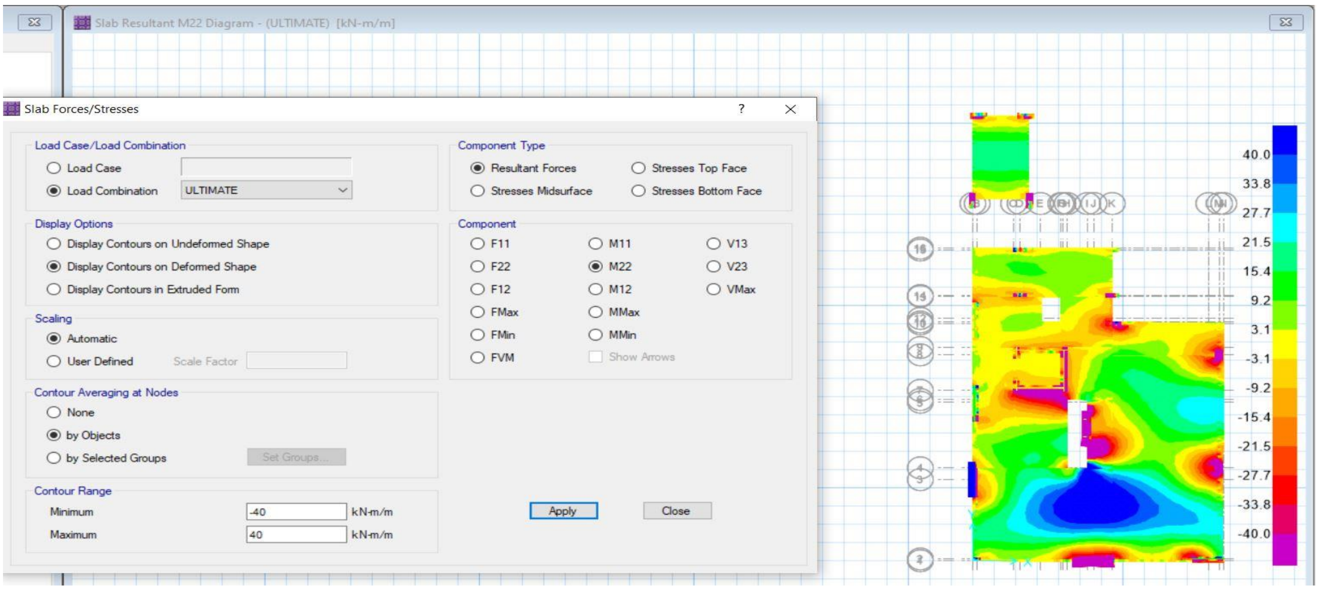


Figure 3.50: reainforcement Steel in Y axis1

Try 5 Φ 12

$$M_u = 50KN.m$$

X – direction

5 Φ 12 not enough

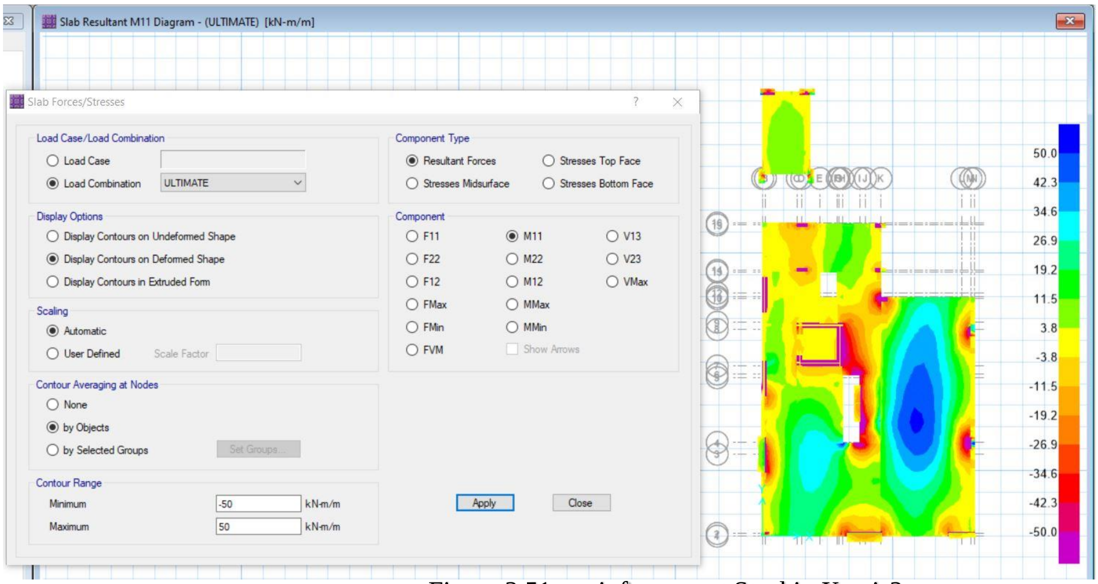


Figure 3.51: reainforcement Steel in X axis2

Y – direction

5 Φ 12 not enough

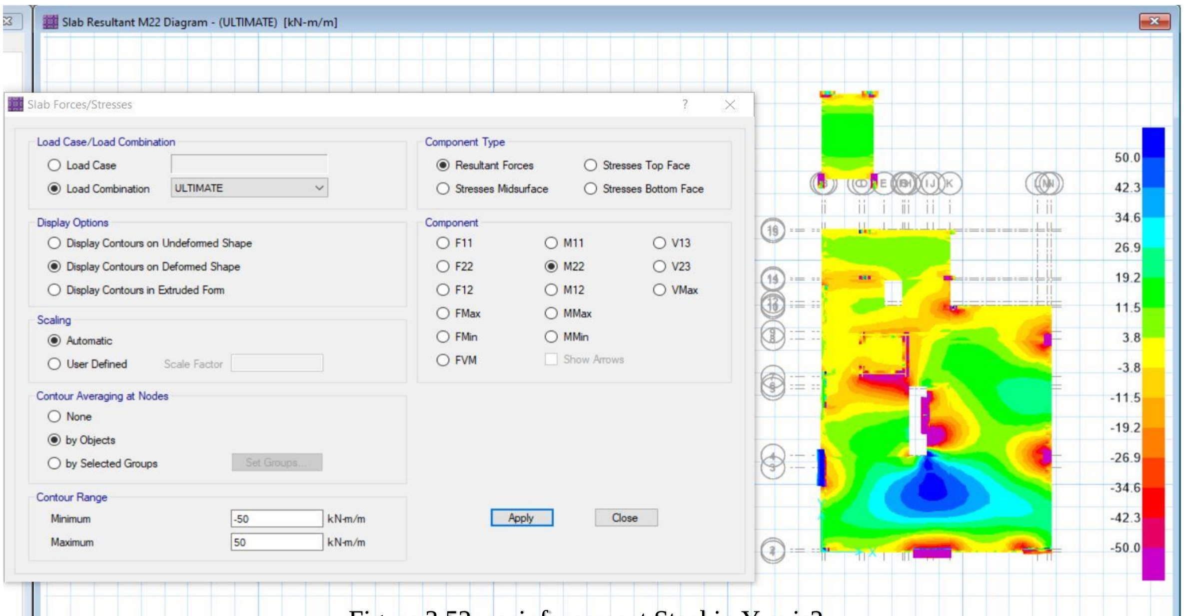


Figure 3.52: reainforcement Steel in Y axis2

Try 6 Φ 12

$M_u = 60KN.m$

X – direction

6 Φ 12 enough

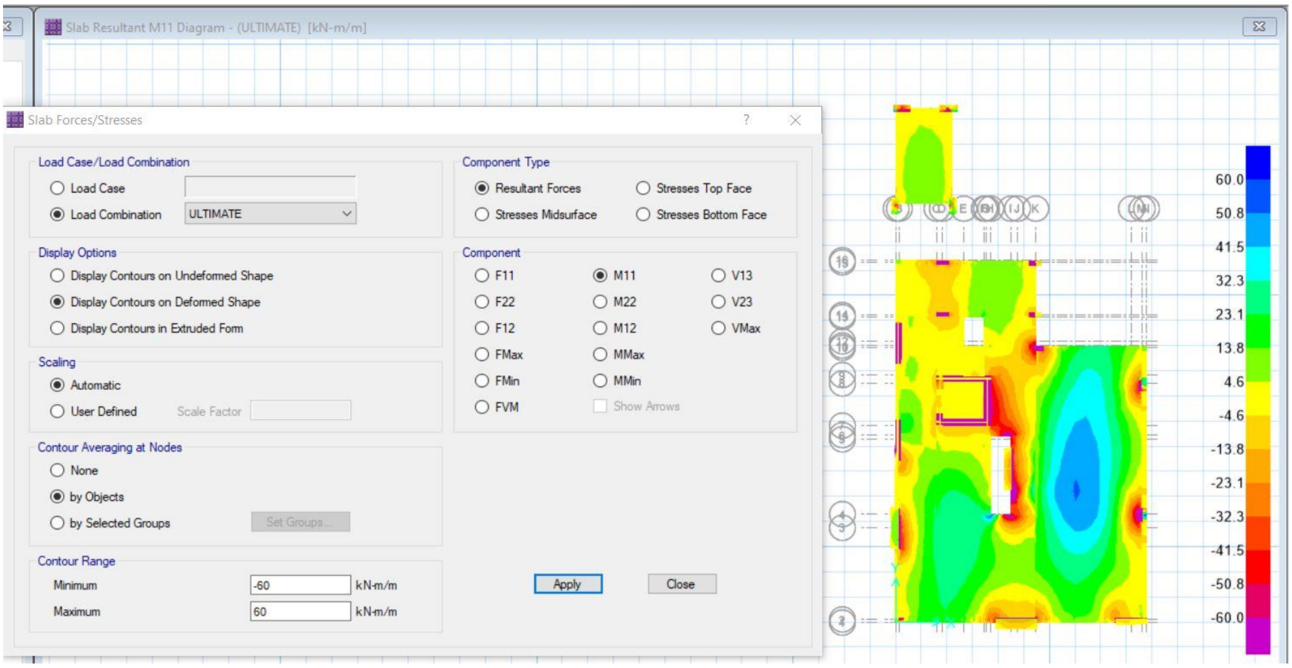


Figure 3.53: reainforcement Steel in X axis3

Y – direction

6 Φ 12 enough

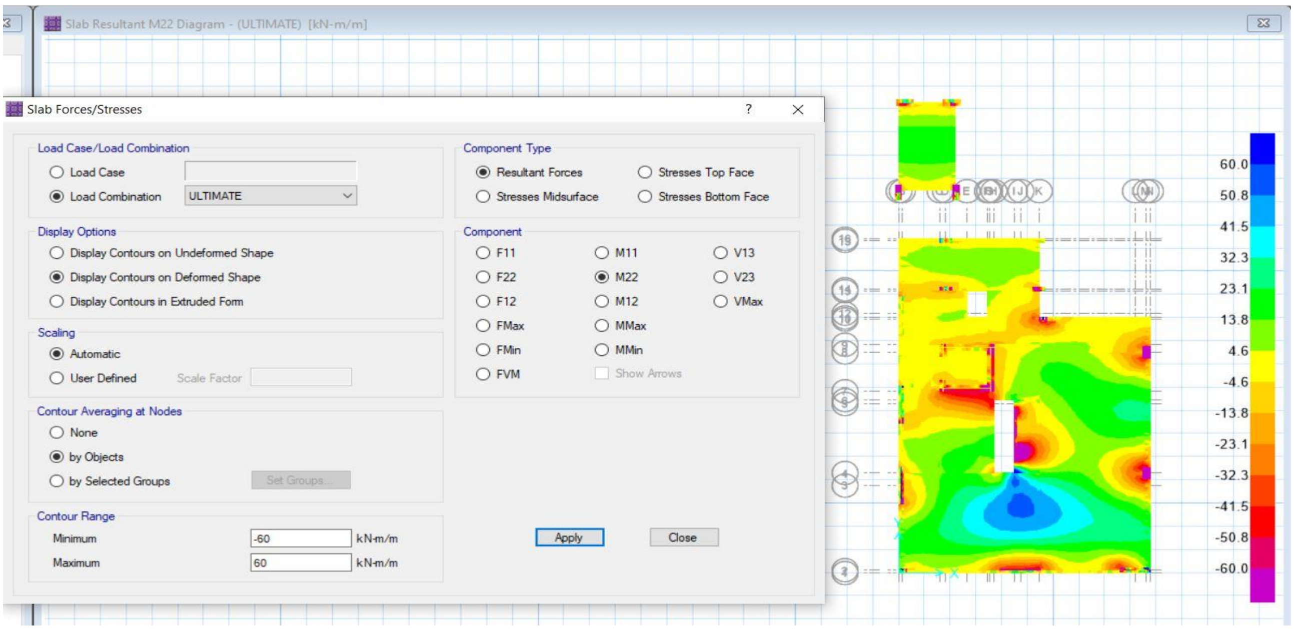


Figure 3.54: reainforcement Steel in Y axis3

Under column (negative Moment)

as typical slabs we use 9Φ14 , it's enough

A = 9*154 = 1380 mm^2

$$1.380 = \frac{M_u}{0.9*460*0.95*0.25}$$

$$M_u = 120\ KN.m$$

X – direction

9Φ14 ⇒ enough

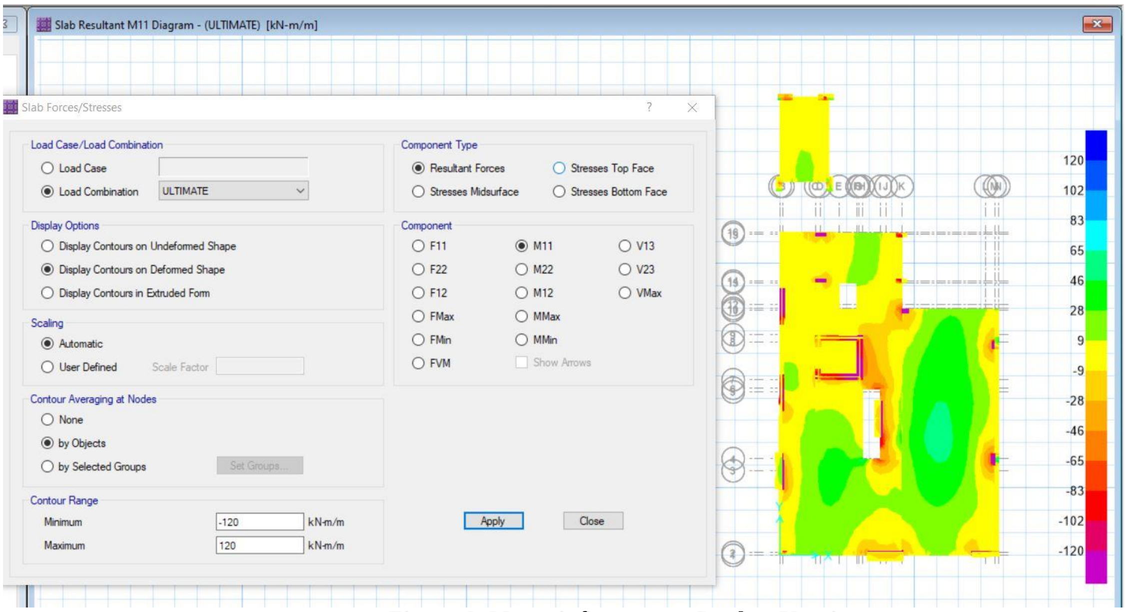


Figure 3.55: reainforcement Steel at X axis

Y – direction

9Φ14 ⇒ enough

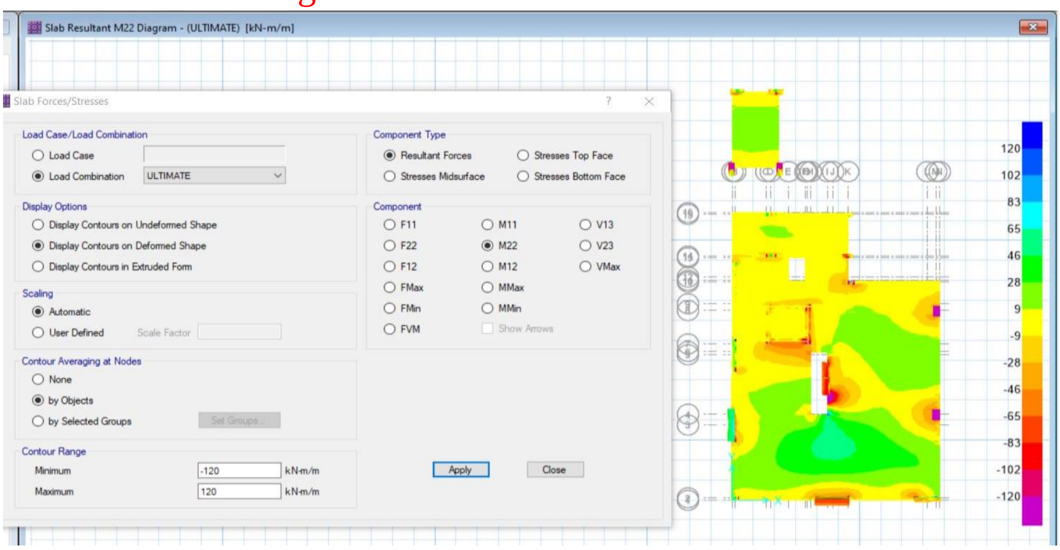


Figure 3.56: reainforcement Steel at Y axis

Check S_{max}

$$S_{max} = \text{smaller of} \begin{cases} 2\,h = 2 * 250 = 500\ mm \\ \qquad 500\ mm \end{cases}$$

actual spacing for 5 bar = 20 cm < S_{max} (50 cm) OK
actual spacing for 6 bar = 16 cm < S_{max} (50 cm) OK
actual spacing for 9 bar = 11 cm < S_{max} (50 cm) OK

Check S_{min}

$$S_{min} = \max of \begin{cases} Bar\ diameter & = 14\ mm \\ 25\ mm & = 25mm \\ diameter\ of\ vibrat\ or & (unknown) \\ 1.33 \max C.A\ size & (unknown) \end{cases}$$

Actual spacing for 5 bars = 200mm > 25mm (OK)
Actual spacing for 6bars = 160mm > 25mm (OK)
Actual spacing for 9 bars = 110mm > 25mm (OK)

Check As_{max}

$$Asmax = \ 0.319\beta\ \frac{f'c}{Fy}\ bd = 5317\ mm$$

For 5 Φ 12

As = 565 mm <5317 (OK)

For 6 Φ 12

As =678 mm <5317 (OK)

For 9 Φ 14

As =1385 mm <5317 (OK)

☒ Summary of reinforcement:

Table 3-1: Summary of rainforcment

		X-direction		Y-direction	
		Positive steel	Negative steel	Positive steel	Negative steel
Typical	slab	6 Φ 12	6 Φ 12	6 Φ 12	6 Φ 12
	Columns	6 Φ 12	9 Φ 14	6 Φ 12	9 Φ 14
Roof	slab	5 Φ 12	5 Φ 12	5 Φ 12	5 Φ 12
	Columns	5 Φ 12	9 Φ 14	5 Φ 12	9 Φ 14

☒ Around opening we add (5 Φ 20) for every edge, it will be shown in structural plans on "AutoCAD".
(Detailed Drawing in appendix A)

Try 5 Φ 20

$$1.575 = \frac{M_u}{0.9*460*0.95*0.25}$$

$$M_u = 140 \ KN.m$$

TYPICAL X – AXIS 5 Φ 20 enough

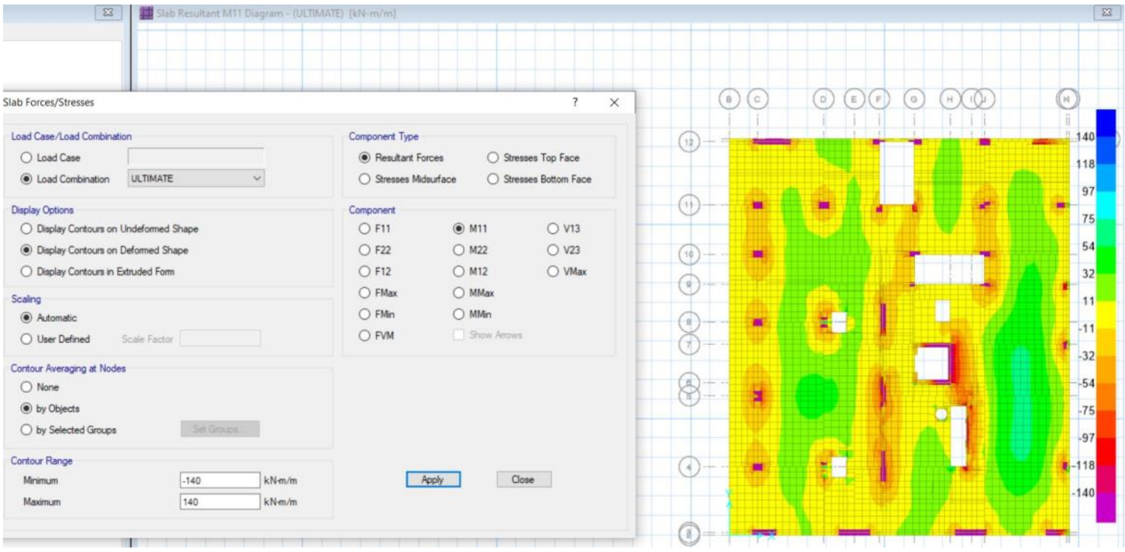


Figure 3.57: reinforcement Steel in typical x – axis around oppening

TYPICAL Y – AXIS 5 Φ 20 enough

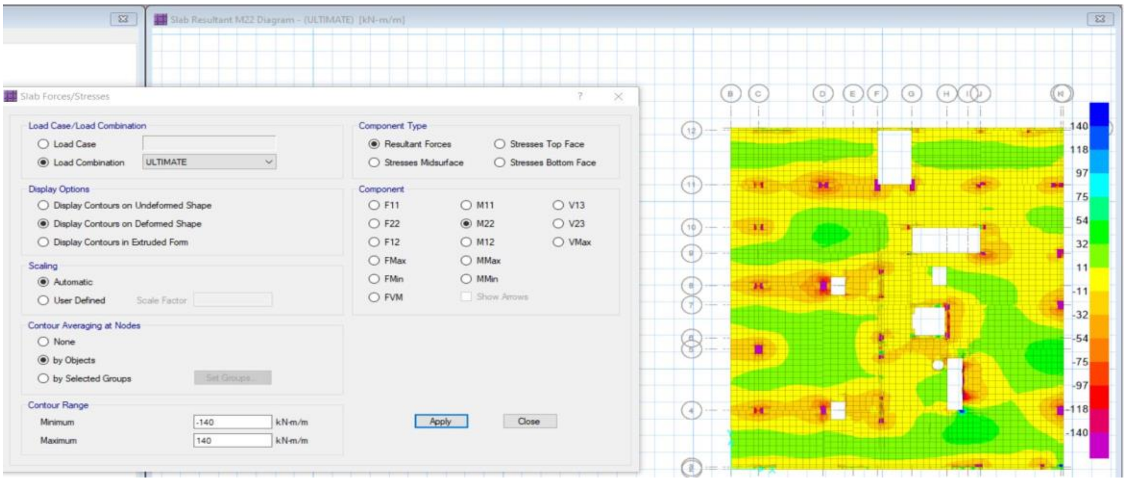


Figure 3.58: reinforcement Steel in typical y – axis around oppening

ROOF X – AXIS 5 Φ 20 enough

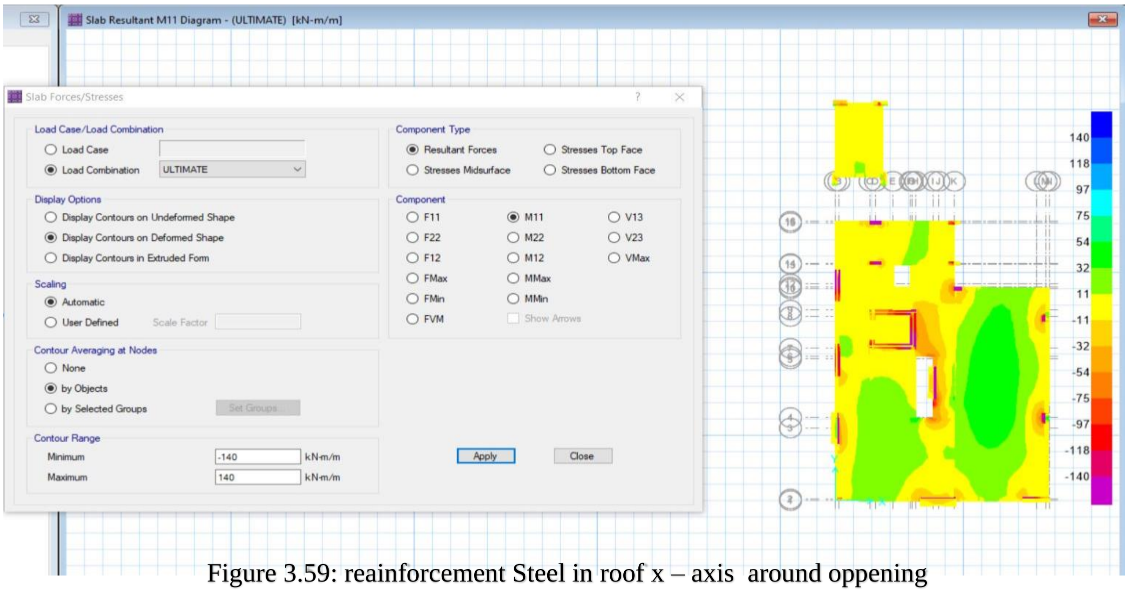


Figure 3.59: reainforcement Steel in roof x – axis around oppening

ROOF Y – AXIS   5 Φ 20 enough

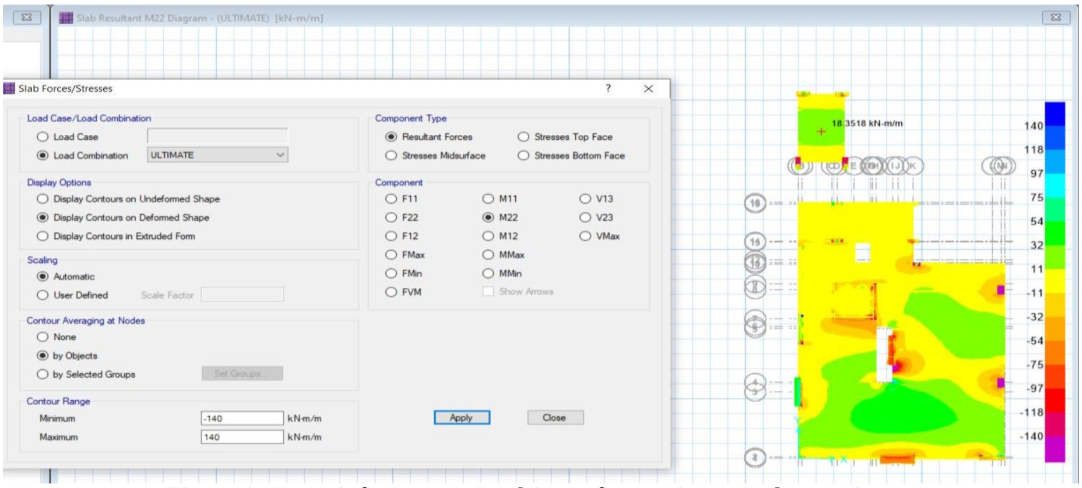


Figure 3.60: reainforcement Steel in roof y – axis around oppening

Chapter 4 : Beams

4.1 Beams Distribution

4.1.1 Load calculations

f′c=28MPa , fy =420MPa , β=0.85 , j_d=0.9,   cover=40mm,   stirrups Φ10

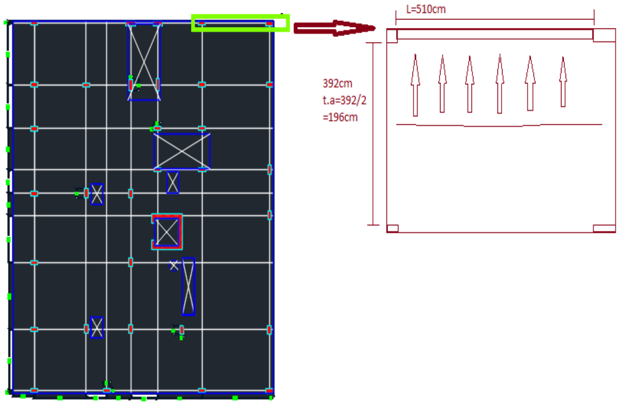


Figure 4.1: beam selection in manual calculation

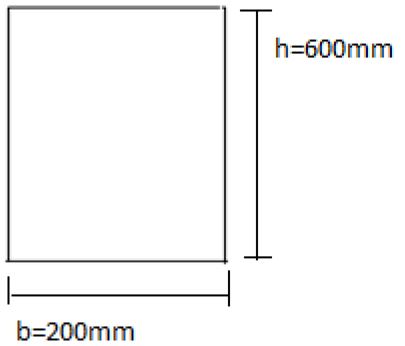


Figure 4.2: beam section

L.L from slab = 2×1.96 = 3.92 KN/m

Load from S.D.L = 8×1.96 =15.68 KN/m

Load from S.W slab =γ×Thickness×T.A=24×0.25×1.96 =11.76 KN/m

Load from S.W beam = γ×h×b = 24×0.6×0.2 = 2.88 KN/m

wall Load= γ_{wall}×Thic.wall×hight.wall =14×0.25×3.5 =12.25 KN/m

Total D.L = 15.68+11.76+2.88+12.25 = 42.57 KN/m

Total L.L= 3.92 KN/m

Wu=1.2D.L+1.6 L.L=1.2×42.57+1.6×3.92 = 57.4 KN/m

4.1.2 Calculations and Part selection

Mu= $\frac{w_u L^2}{8}$

Mu=186.6 KN.m

h_{min} from table 9.5(a)

TABLE 9.5(a)—MINIMUM THICKNESS OF NONPRESTRESSED BEAMS OR ONE-WAY SLABS UNLESS DEFLECTIONS ARE COMPUTED

	Minimum thickness, *h*			
	Simply supported	One end continuous	Both ends continuous	Cantilever
Member	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections.			
Solid one-way slabs	$\ell/20$	$\ell/24$	$\ell/28$	$\ell/10$
Beams or ribbed one-way slabs	$\ell/16$	$\ell/18.5$	$\ell/21$	$\ell/8$

Notes:
Values given shall be used directly for members with normalweight concrete and Grade 60 reinforcement. For other conditions, the values shall be modified as follows:
a) For lightweight concrete having equilibrium density, w_c, in the range of 90 to 115 lb/ft³, the values shall be multiplied by $(1.65 - 0.005w_c)$ but not less than 1.09.
b) For f_y other than 60,000 psi, the values shall be multiplied by $(0.4 + f_y/100{,}000)$.

Figure 4.3: Table 9.5 (a)

$\boldsymbol{h_{min}}=\frac{L}{16}=\frac{5100}{16}=$ **318.75 mm**

h_{min} = 318.75mm < h=600mm OK

<u>assume two Layer</u>:

d= h-90 = 600-90 $\Rightarrow$ d=510mm

As=$\frac{Mu}{\Phi\, fy\, j_d}$,Φ= 0.9

As=1075.5mm^2

<u>-iteration:</u>

$a=\frac{A_S fy}{0.85\, f_C'\, b}$ $A_s=\frac{M_u}{\emptyset\, fy\, (d-0.5a)}$

a=94.9mm As=1067.7 mm^2

a=94.2mm As=1066 mm^2

then $As_{required}$ =1066 mm^2

-From steel table

<u>USE 6 bars Φ16 mm</u>

$As_{provided}$=1206 mm^2

check As_{min}

$$As_{min} = \xi^{0\cdot25\sqrt{f'}c\ bd\ =321}_{\frac{1.4bd}{fy}=340}$$

$$As_{min} = 340\text{mm}^2 < \text{As}=1206\ \text{mm}^2 \ \text{OK}$$

$$S_{\min}(H) = \max of \begin{cases} Bar\ diameter \\ 25\ mm \\ diameter\ of\ vibrator\ (no\ data) \\ 1.33 \max C.A\ size\ \ (no\ data) \end{cases}$$

$$S_{\min}(H) = \max of \begin{cases} 16mm \\ 25mm \\ - \\ - \end{cases}$$

$\boldsymbol{S_{\min}(H) = 25\text{mm}}$

b_{min}= 2×cover+2×sirrups diameter + n (N0. Of bars) ×diameter of bar + (n-1) ×SH(min.)+2(2ds-0.5db)

b_{min}=2×40+2×10+4×16+3×25+2(2×10-0.5×16)
b_{min}= 272mm
$b_{min} > \text{b}$ then assumption OK (two layer)

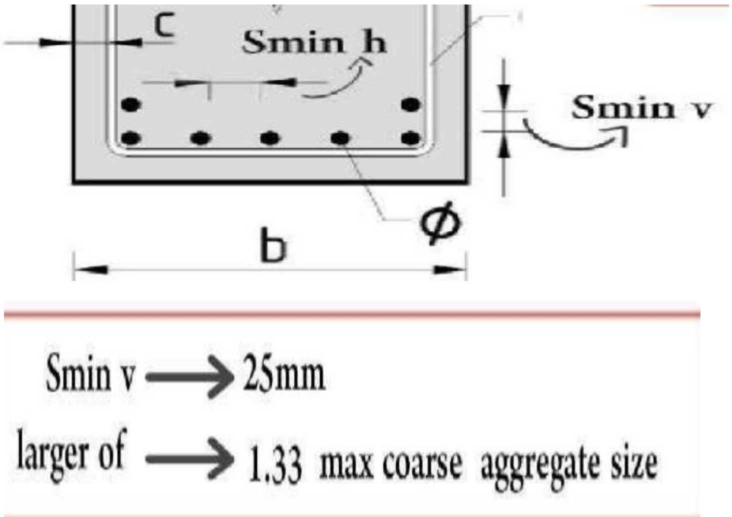


Figure 4.4: Smin

use $S_{\min}(V)$ =25mm

$S_{\min}(H)$ =200-2×40-2×19-2×10-2(2×10-0.5×19)
$S_{\min}(H)$ = 41mm

- check As_{max}:

$As_{max} = 0.319\beta \frac{f_c'}{fy} bd$
As_{max} = 1843mm^2 > As=1206 mm^2

analysis:
T=Cc
As ×fy=0.85f_c' a b
1140×420=0.85×28×a×200
a=100.6mm
c=$\frac{a}{\beta}$=118.3mm

Σs=0.003($\frac{d-c}{c}$)=0.0099
Σs=0.0099>0.005 ok tension control
in tension control Φ=0.9
ΦMn=Φ×T×(d-$\frac{a}{2}$) = 0.9×1140×420(510-$\frac{100.6}{2}$)
ΦMn= 198 MPa
ΦMn>Mu OK

- shear in beam:

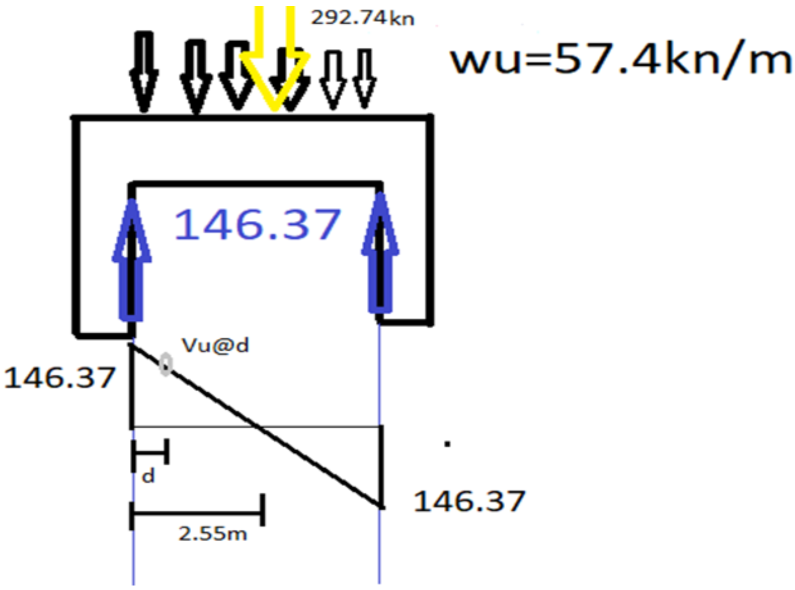


Figure 4.5: shear in beam

$\frac{146.37}{2.55} = \frac{v_{u@d}}{(2.55-0.51)}$ $\Rightarrow$ Vu@d=117.1 KN

$v_c = \frac{\sqrt{f_c'}}{6} bd \Rightarrow v_c = \frac{\sqrt{28}}{6} \times 200 \times 510 = 89.96$ KN

Vu_{max}= 5ΦVc = 5×0.75×89.96 = 337.3 KN
Vu_{max} >Vu@d OK

ΦVc=0.75×89.96 = 67.47 KN

$\Phi Vc2 = \frac{\Phi Vc}{2}$ = 33.735 KN

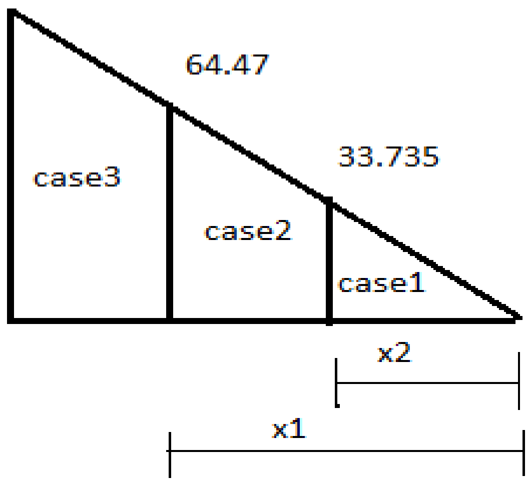


Figure 4.6: shear cases

- $\frac{146.37}{2.59} = \frac{67.47}{x_1} = \frac{33.735}{x_2}$

x_1=1.175m x_2 =0.5877m

$\Rightarrow$ case 1:

Vu$\leq \frac{\Phi Vc}{2}$

<u>no need to shear reinforcement</u>

$\Rightarrow$ case 2:

$\frac{\Phi Vc}{2}$ < Vu< ΦVc

<u>required stirrup spacing</u>

$$S_{max.} = \begin{cases} 600\ mm & \\ d/2 & 255\text{mm} \\ \dfrac{A_v f_y}{0.33 b_w} & 999.6\text{mm} \\ \dfrac{16 A_v f_y}{b_w \sqrt{f_c'}} & 997.43\text{ mm} \end{cases}$$

$A_{v1} = \frac{\pi}{4}(10)^2 = 78.5$

A_v=2A_{v1}=157.08

S_{max}= 255mm

⇒ case3: (Vu>ΦVc) and (Vs≤ 4Vc)

<u>required stirrup spacing</u>

$$S_{max.} = \begin{cases} 600\ mm \text{ or } \dfrac{d}{2}; if\ V_s \le 2V_c & 600\text{mm} \\ 300\ mm \text{ or } \dfrac{d}{4}; if\ V_s > 2V_c & 255\text{mm} \\ \dfrac{A_v f_y}{0.33 b_w} & 999.6\text{mm} \\ \dfrac{16 A_v f_y}{b_w \sqrt{f_c'}} & 997.4\text{mm} \\ \dfrac{A_v f_y d}{V_s} & 508486\text{mm} \end{cases}$$

Vs=$\frac{v_u @d}{\emptyset}$-Vc

Vs=$\frac{117.1}{0.75}$ – 89.96 =66.17 KN

Vs<2Vc

S_{max} = 255mm

4.2 Design

☒ ***Design of beams:***

✓ We use "Prokon" program in the analysis and design.

Typical Floor:

- 3 strip (side view) will be analyzed and designed.

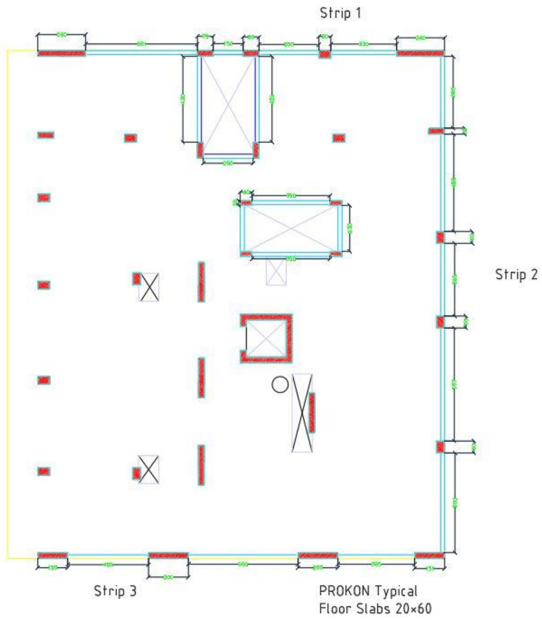


Figure 4.7: Design of beam strip

✓ Strip 1:

✓ Deflection:

- Using ACI-Code, Table 9.5 ➡ (b) Case 4.
- Impressible deflection = 8.1/240

  = 3.375 cm

  Actual deflection = 2.652 cm < 3.375 (Okay)

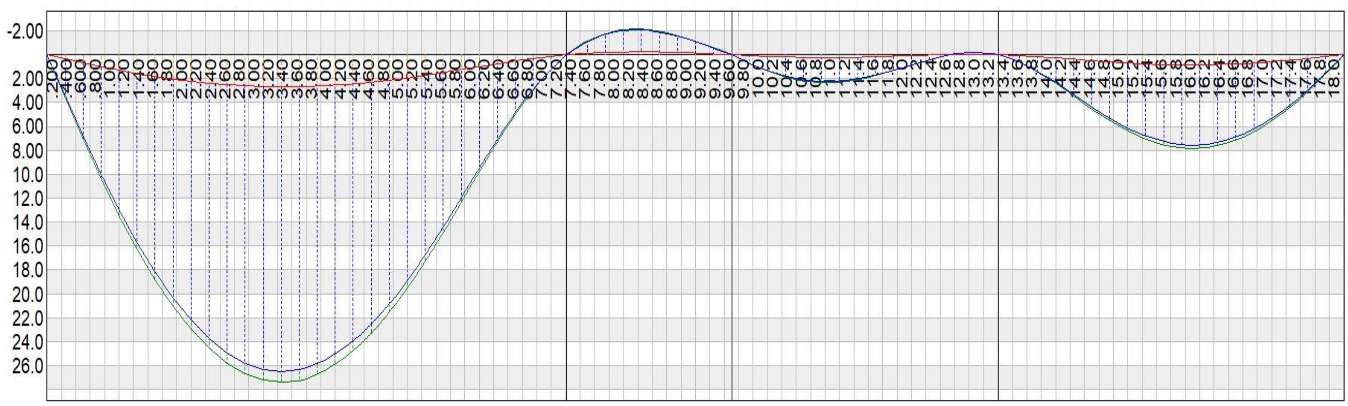
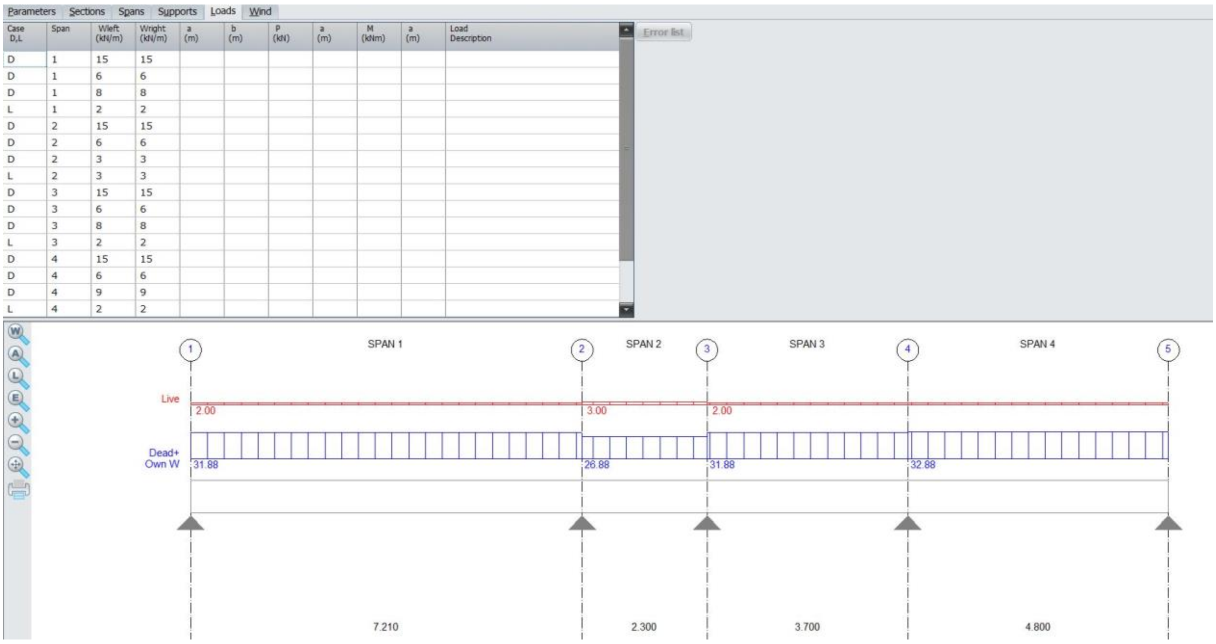


Figure 4.8: Prokon deflection typical slab

✓ Load value:



Case D,L	Span	Wleft (kN/m)	Wright (kN/m)	a (m)	b (m)	P (kN)	a (m)	M (kNm)	a (m)	Load Description
D	1	15	15							
D	1	6	6							
D	1	8	8							
L	1	2	2							
D	2	15	15							
D	2	6	6							
D	2	3	3							
L	2	3	3							
D	3	15	15							
D	3	6	6							
D	3	8	8							
L	3	2	2							
D	4	15	15							
D	4	6	6							
D	4	9	9							
L	4	2	2							

Figure 4.9: Prokon load value1

✓ Moment +Shear value:

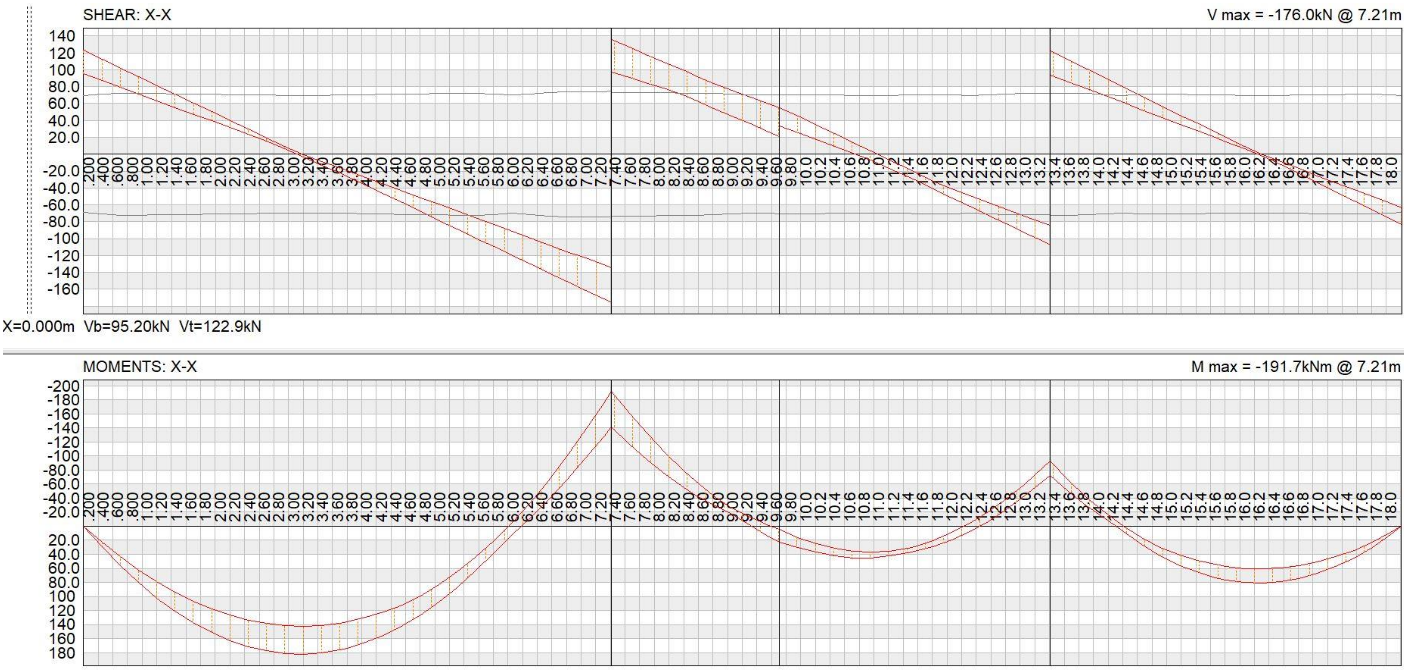


Figure 4.10: Prokon moment shear 1

✓ Section of Strip:

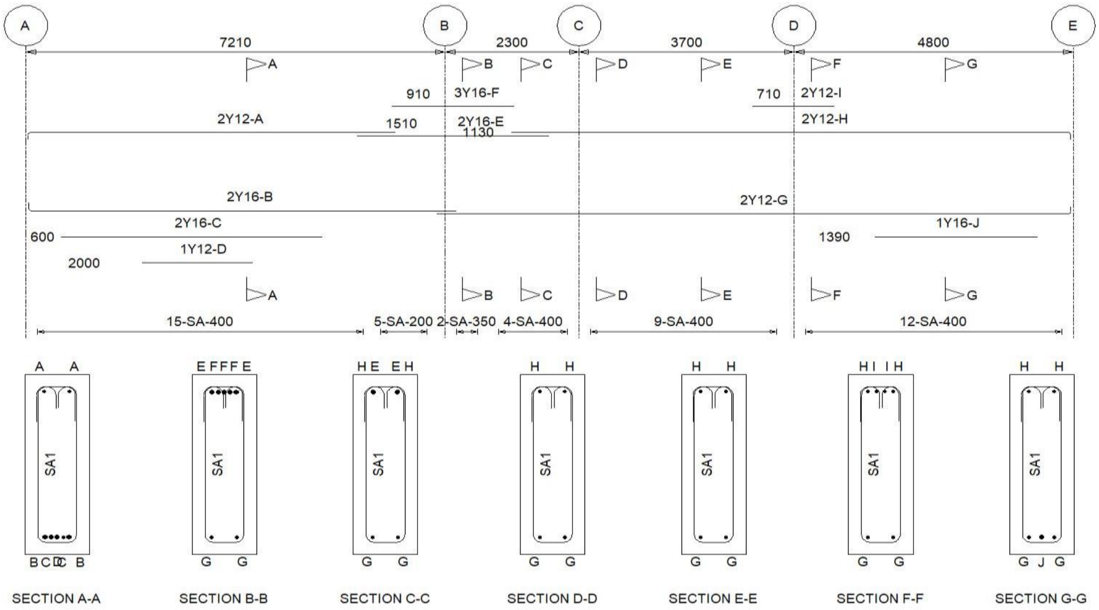


Figure 4.13: Section of Strip1

✓ Strip 2:

✓ Deflection:

Using ACI-Code, Table 9.5 (b) ➡ Case 4.

Permissible deflection = 7.85/240

= 3.2 cm

Actual deflection =1.085 cm < 3.2 (okay)

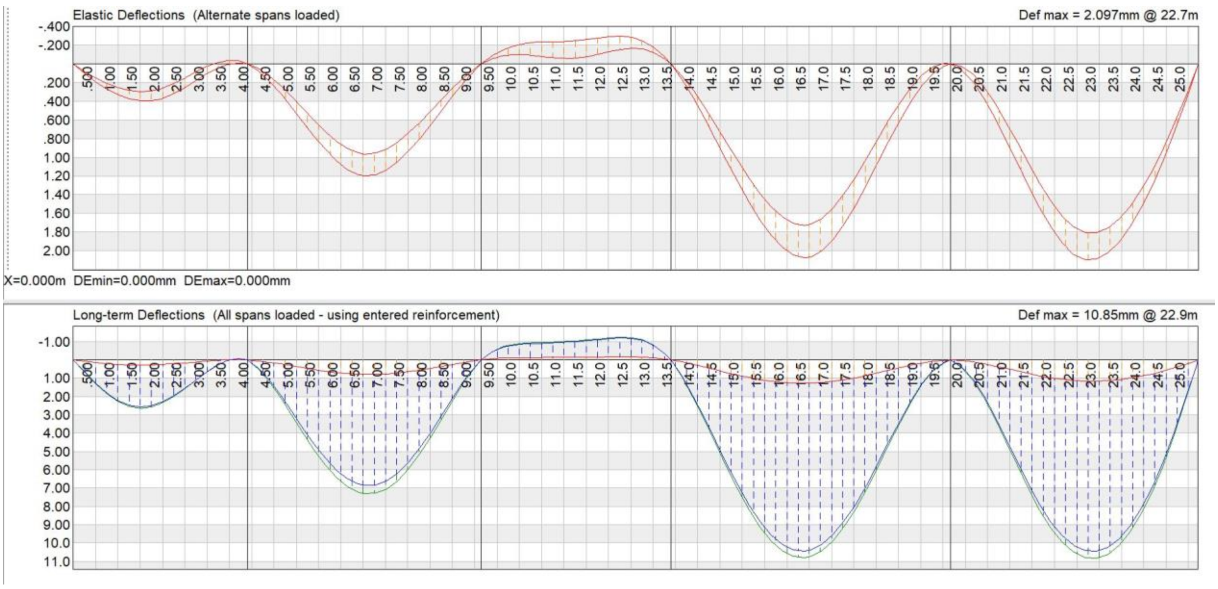


Figure 4.14: Prokon deflection2

✓ Load value:

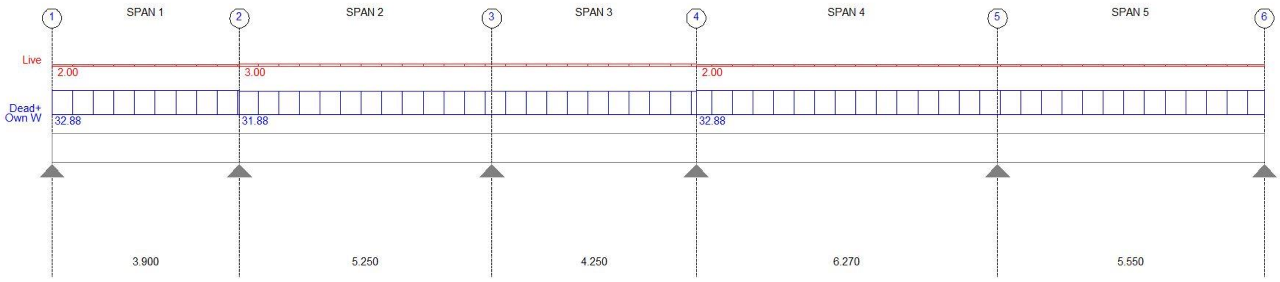


Figure 4.15: Prokon load value2

✓ Moment & Shear value:

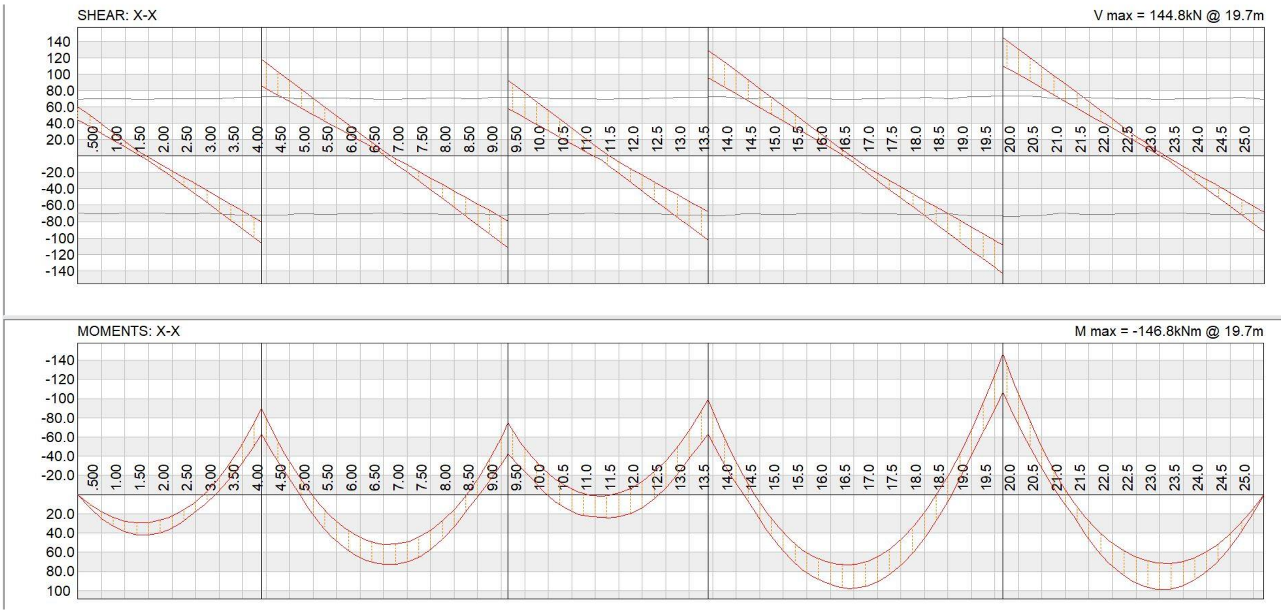


Figure 4.16: Prokon moment shear 2

✓ Section of Strip:

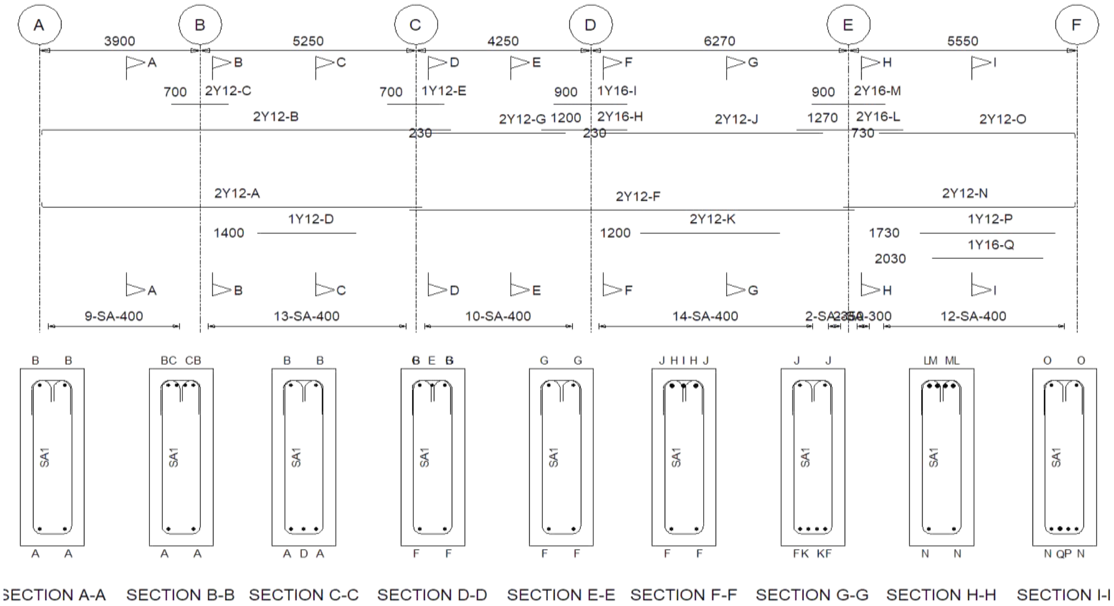


Fig.4.19: Section of Strip2

✓ Strip 3:

✓ Deflection:

Using ACI-Code, Table 9.5 ➡ (b) Case 4.

Permissible deflection = 8/240

= 3.333 cm

Actual deflection = 2.1 cm < 3.333 (okay)

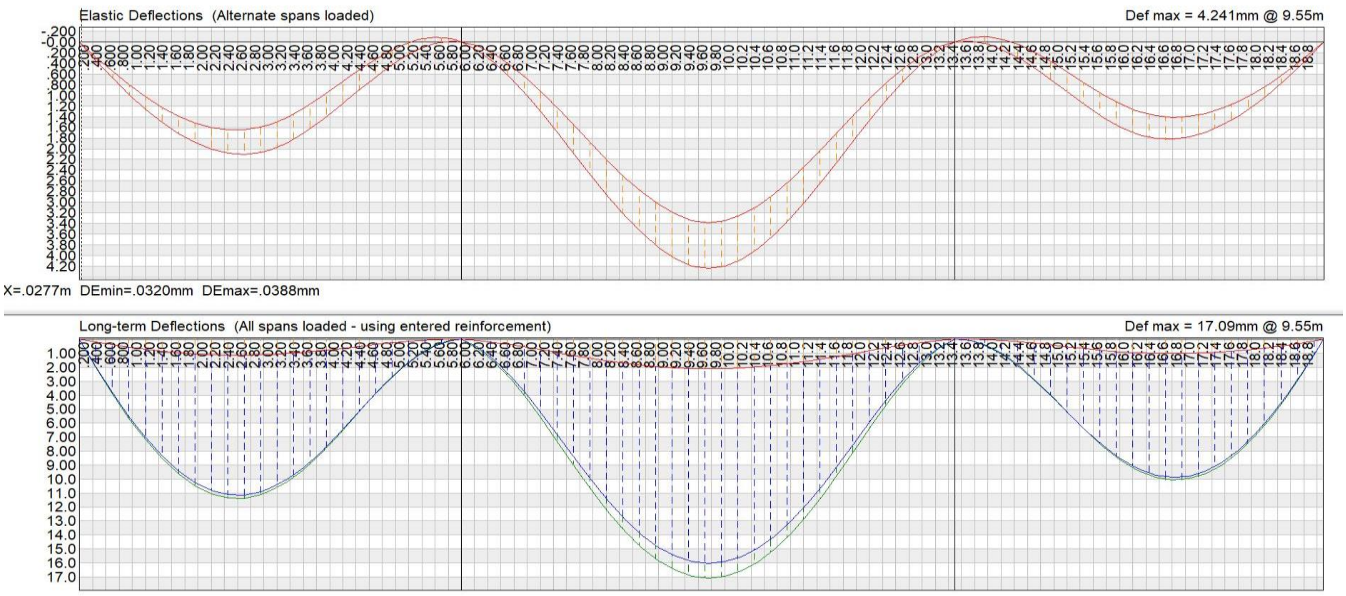


Figure 4.20: Prokon deflection3

✓Load value:

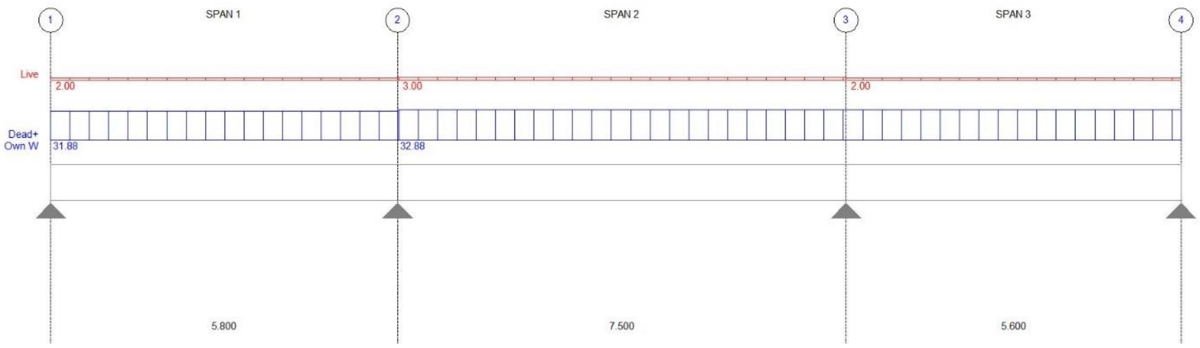


Figure 4.21: Prokon load value3

✓Moment & shear value:

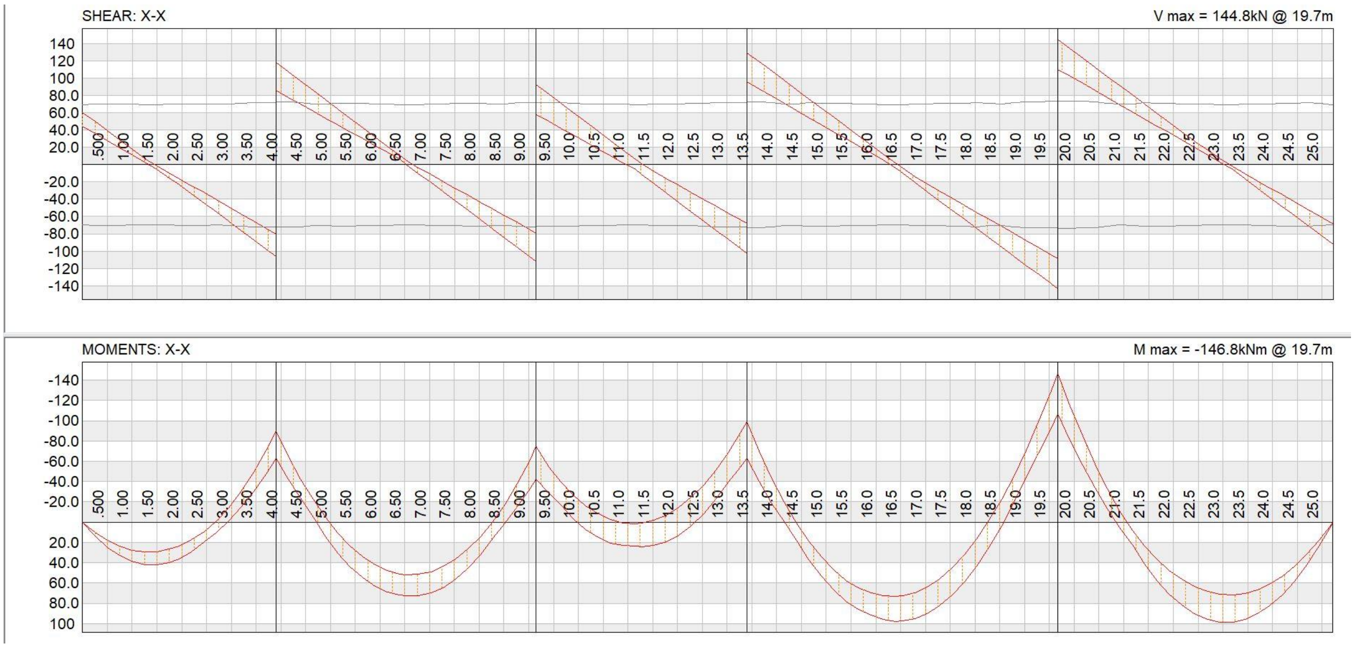


Figure 4.22: Prokon moment shear 3

✓ Section of Strip:

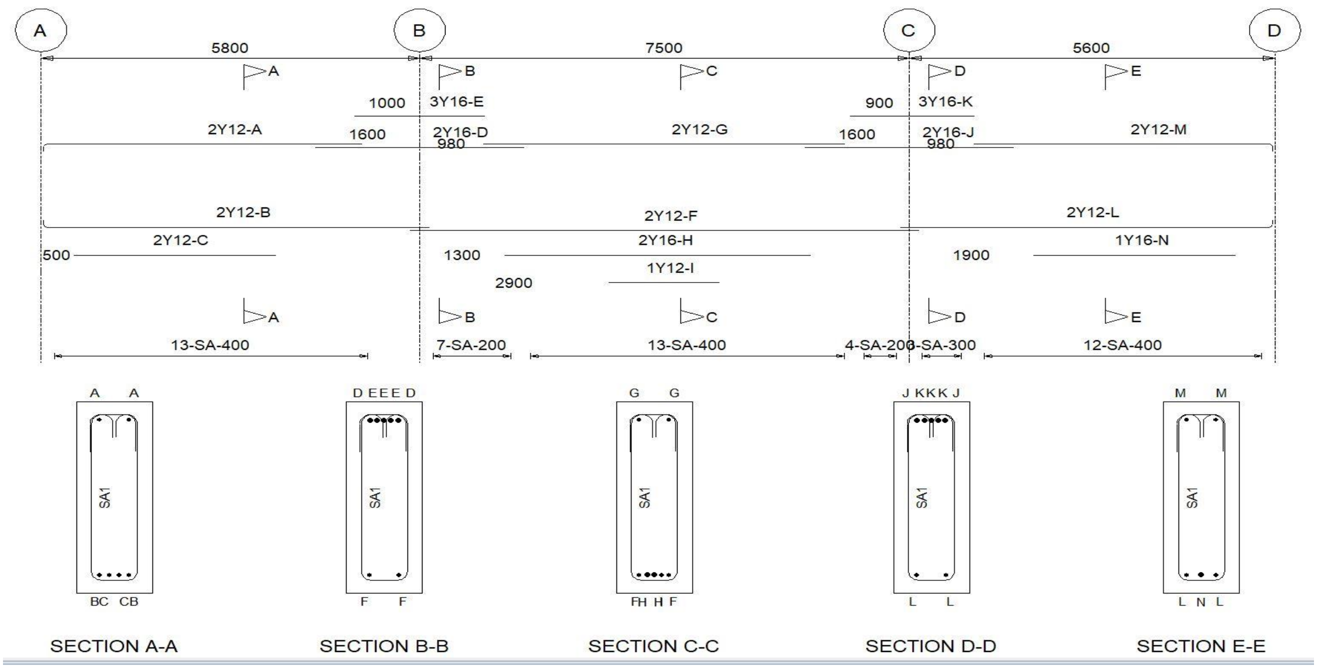


Fig.4.25: Section of Strip3

❖Strip 1 (first span) will be checked as sample of calculation:

✓ Check As_{min} :

$$As_{min} = \begin{cases} 0.25\sqrt{f'c}\ bd & = 325 \\ \dfrac{1.4\ bd}{Fy} & = 340\ \ control \end{cases}$$

As=1260 mm > 340 (Ok)

✓ Check As_{max}

$$As_{max} = \ 0.319\beta\ \frac{f'c}{Fy}\ bd \quad = \ 2169.2\ mm$$

As=1260 mm < 2169.2 (OK)

✓ Check S_{max}:

$$S_{max} = \begin{cases} 600\ mm & = 600mm \\ 300\ mm & = 300mm \\ \dfrac{Av\ Fy}{0.33bw} & = 1000mm \\ \dfrac{16\ Av\ Fy}{bw\sqrt{f'c}} & = 1000mm \\ \dfrac{Av\ Fy\ d}{Vs} & = 650mm \end{cases}$$

Actual Spacing = 40 mm < 300 (OK)

✓ Check S_{min}

$$S_{min} = \begin{cases} Bar\ dia & = 16\ mm \\ 25mm & = 25mm \\ dia.\ of\ vibrator & (\ unkown) \\ 1.33\ c.a\ size & (unkown) \end{cases}$$

Actual Spacing = 40 mm > 25 (OK)

"***All spans in every strip checked and its OK*** "

Roof Floor:

- 6 strip (side view) will be analyzed and designed.

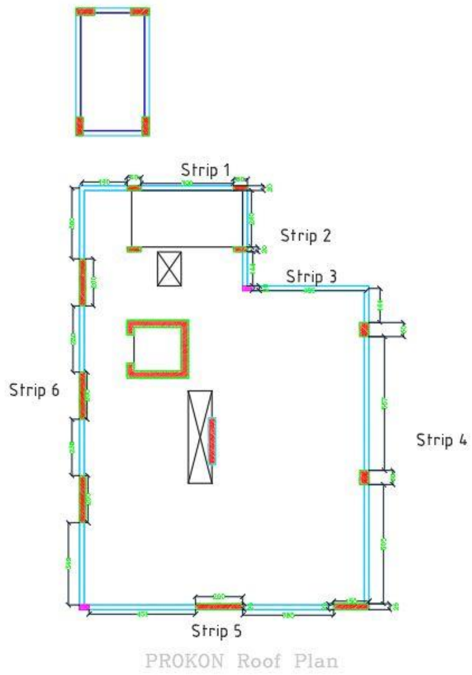


Figure 4.26: Design of roof srip

1- Strip 1:

✓ Deflection:

- Using ACI-Code, Table 9.5 (b) ➡ Case 4.
- Permissible deflection = 4.8/240

= 2 cm

Actual deflection = 0.6945 cm < 2 (okay

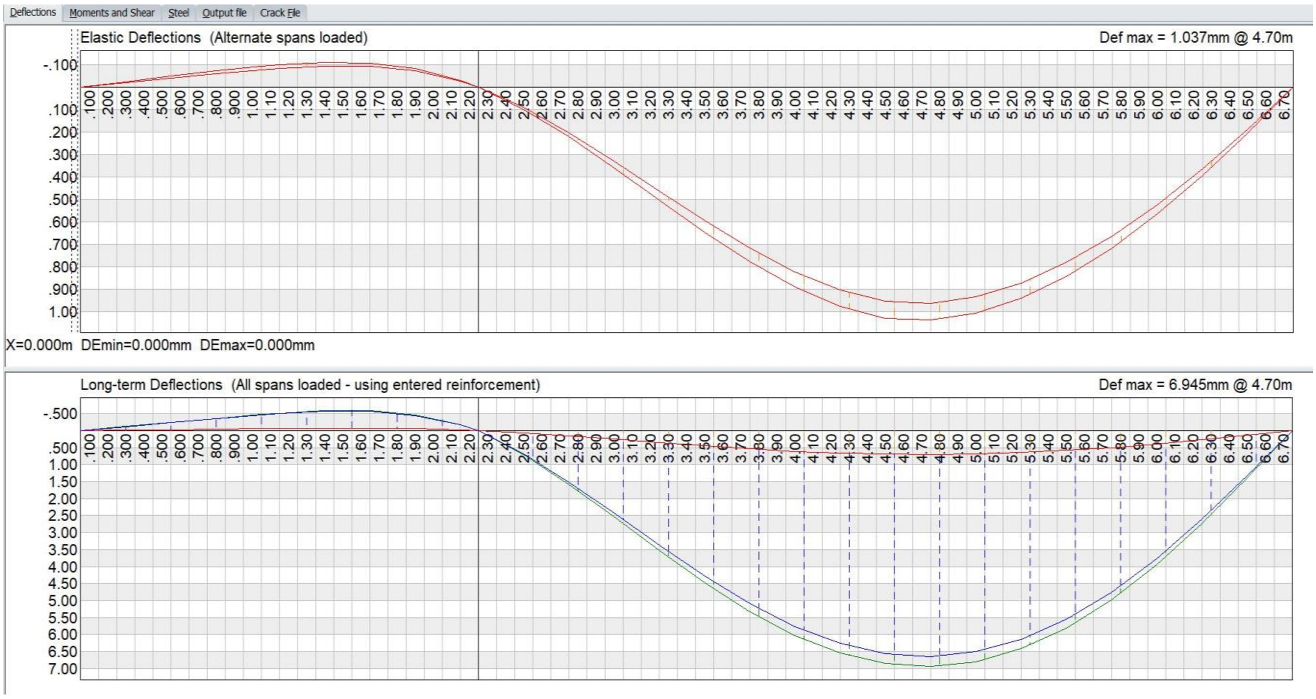


Figure 4.27: Prokon Defletion roof value

✓ Load value:

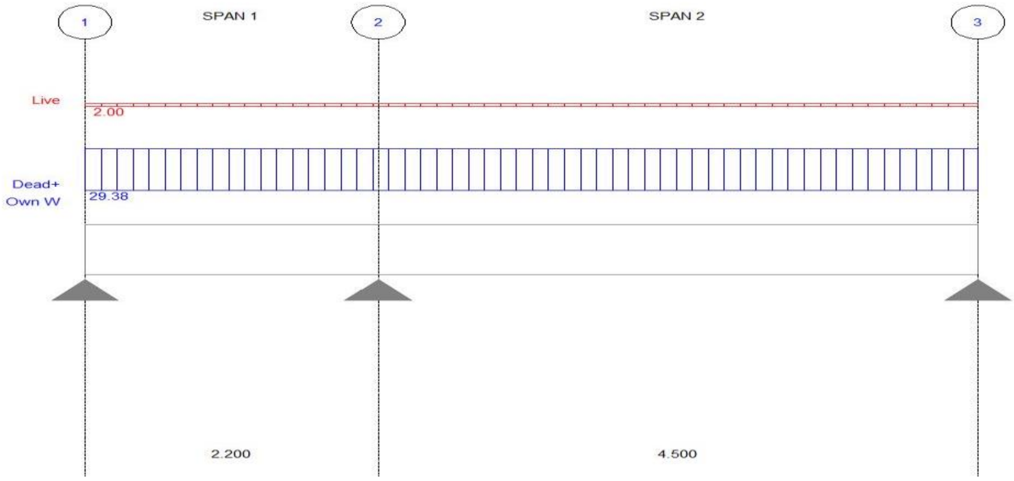


Figure 4.28: Prokon load value 1

✓ Moment & Shear value:

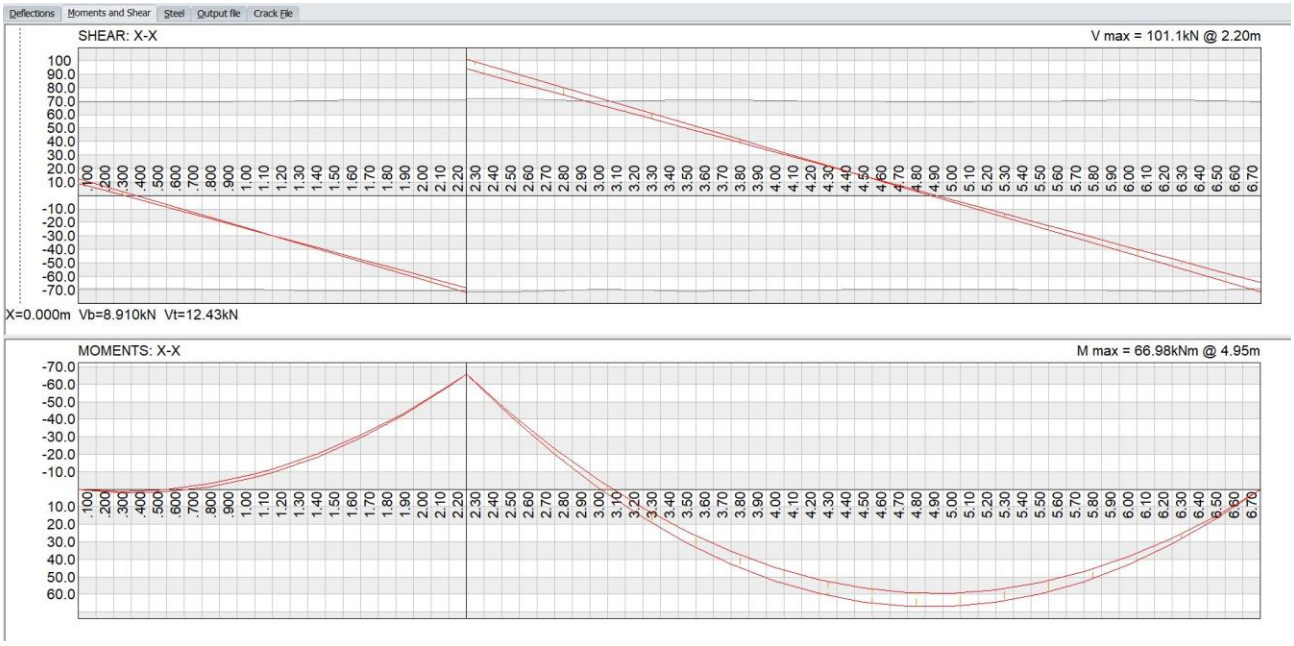


Figure 4.29: Prokon roof moment and shear 1

✓ Section of Strip:

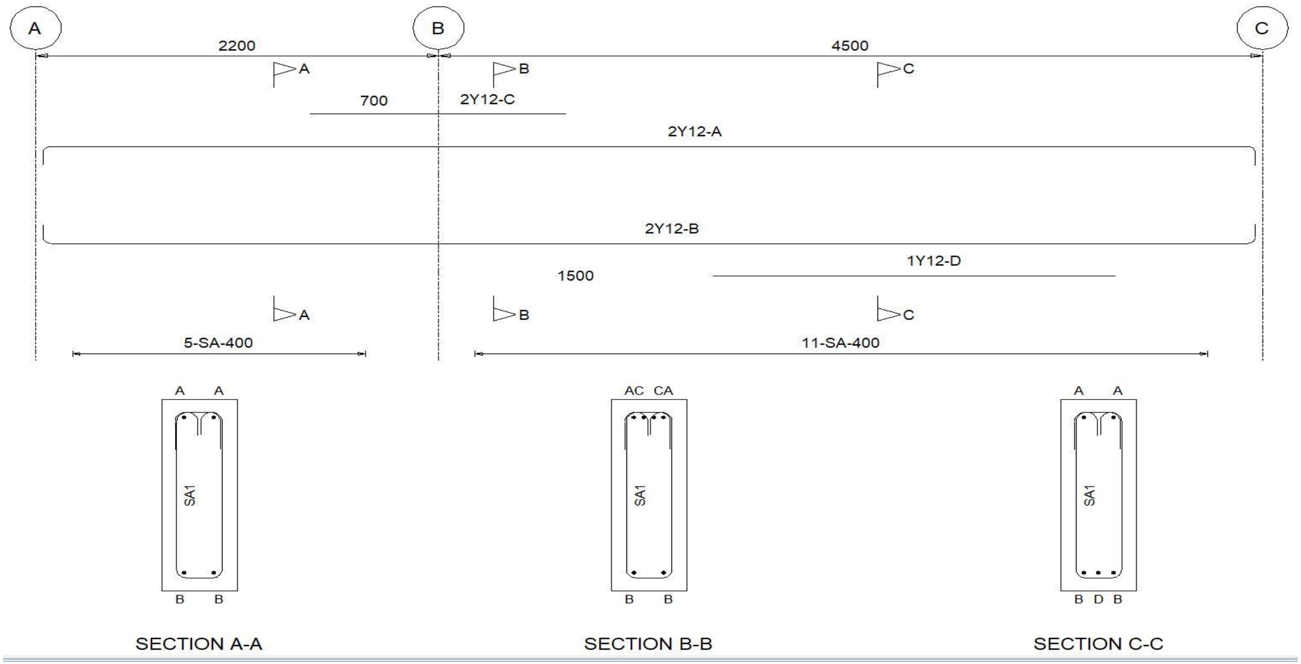


Fig.4.32: Section of Strip1

2- Strip 2:

✓ Deflection:

Using ACI-Code, Table 9.5 (b) Case 4.

Permissible deflection = 4.5/240

= 1.875 cm

Actual deflection = 0.03956 cm < 1.875 (okay)

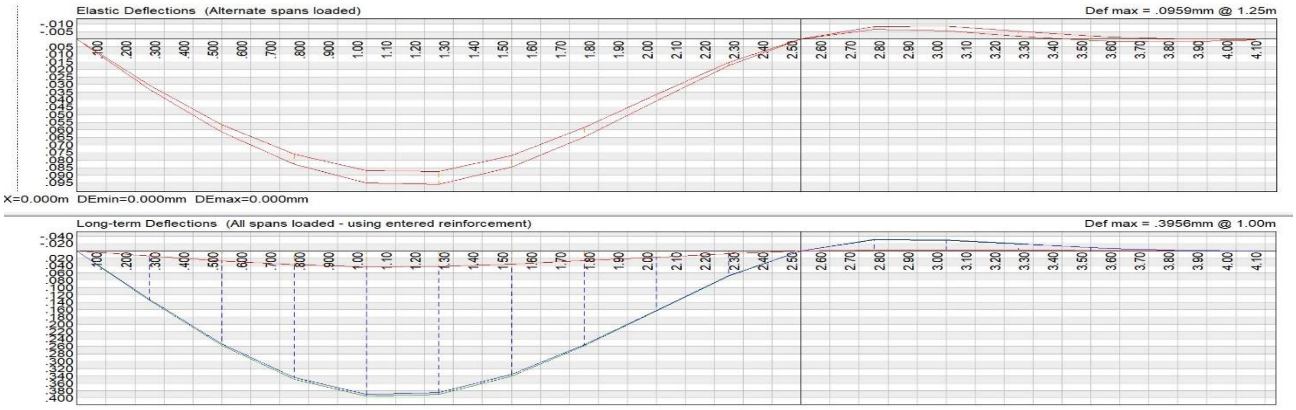


Figure 4.33: Prokon Defletion2

✓ Load value:

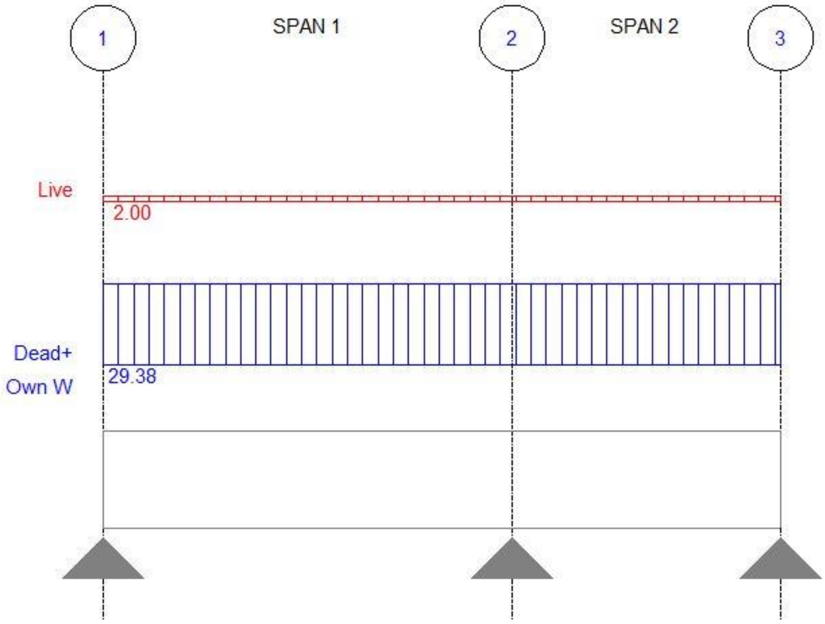


Figure 4.34: Prokon load value 2

✓ Moment & Shear value:

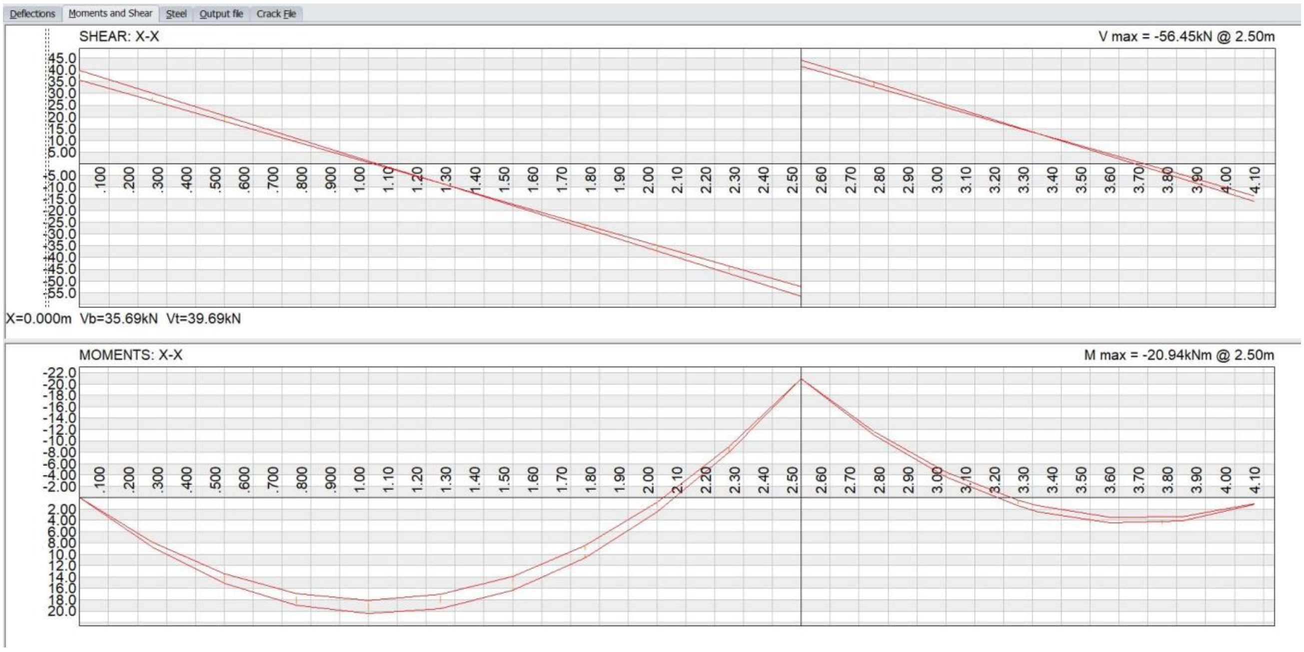


Figure 4.35: Prokon roof moment and shear 2

✓ Section of Strip:

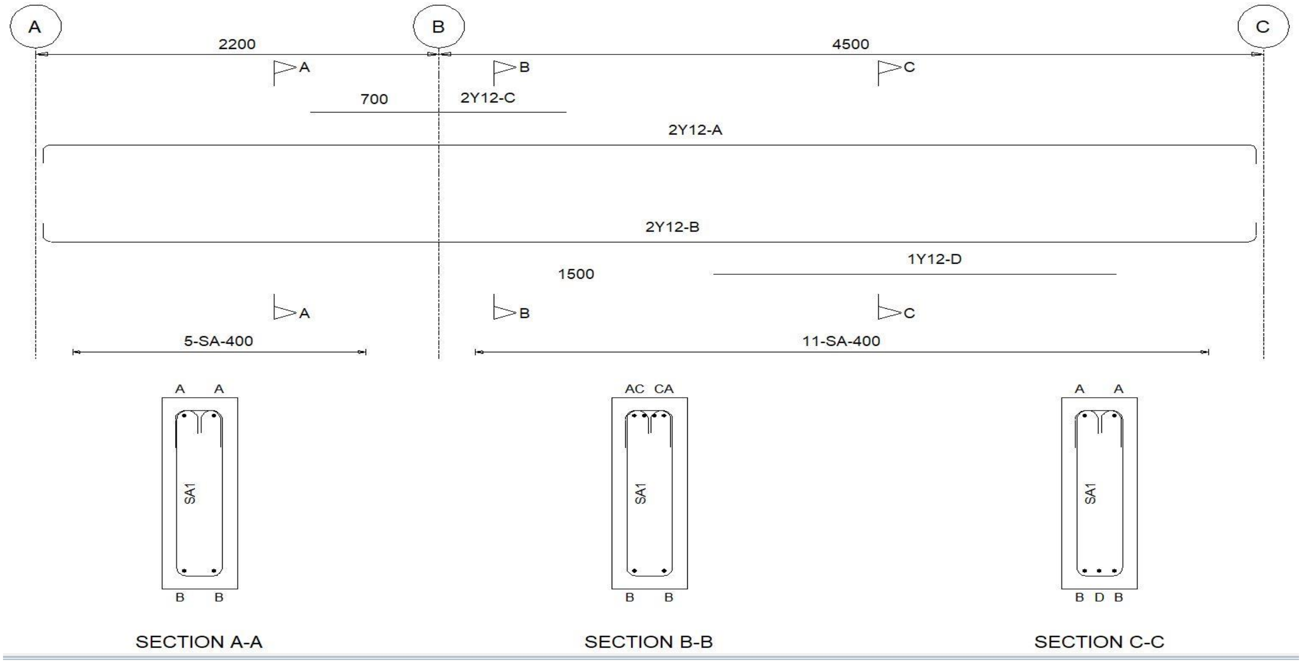


Fig.4.38: Section of Strip2

3- Strip 3:

✓ Deflection:

Using ACI-Code, Table 9.5 (b) ➡ Case 4.

Permissible deflection = 6.2/240

= 2.6 cm

Actual deflection = 1.431 cm < 2.6 (okay)

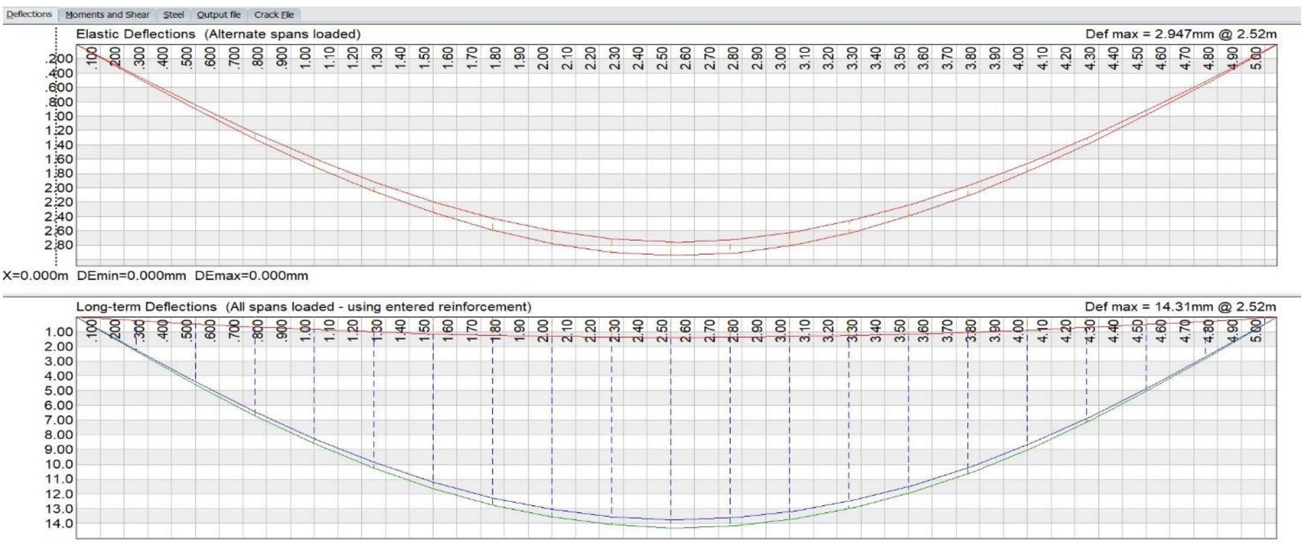


Figure 4.39: Prokon Defletion3

✓ Load value:

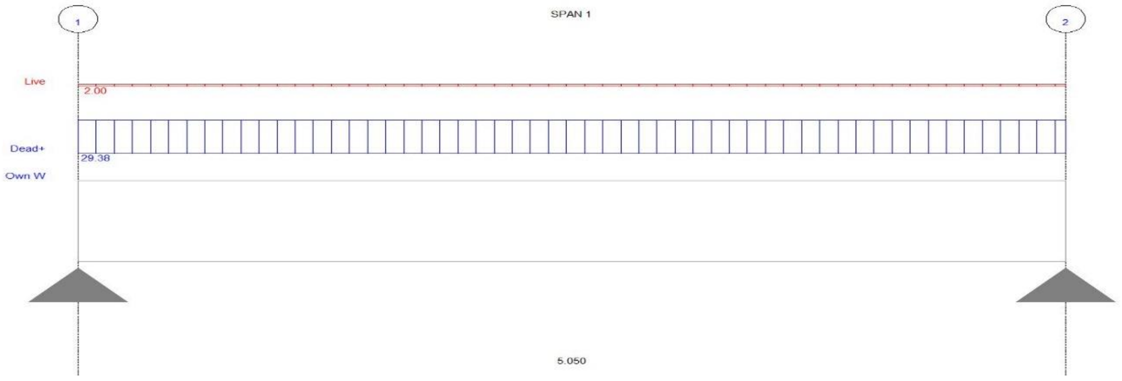


Figure 4.40: Prokon load value 3

Moment & Shear value

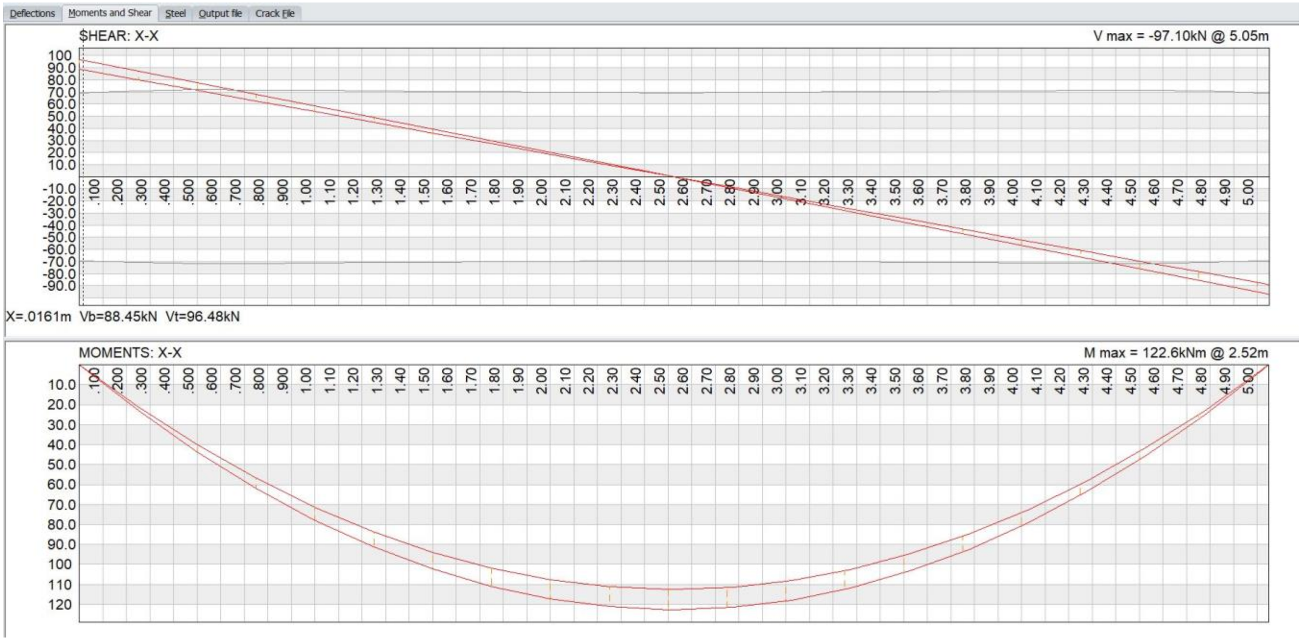


Figure 4.41: Prokon roof moment and shear 3

✓ Section of Strip:

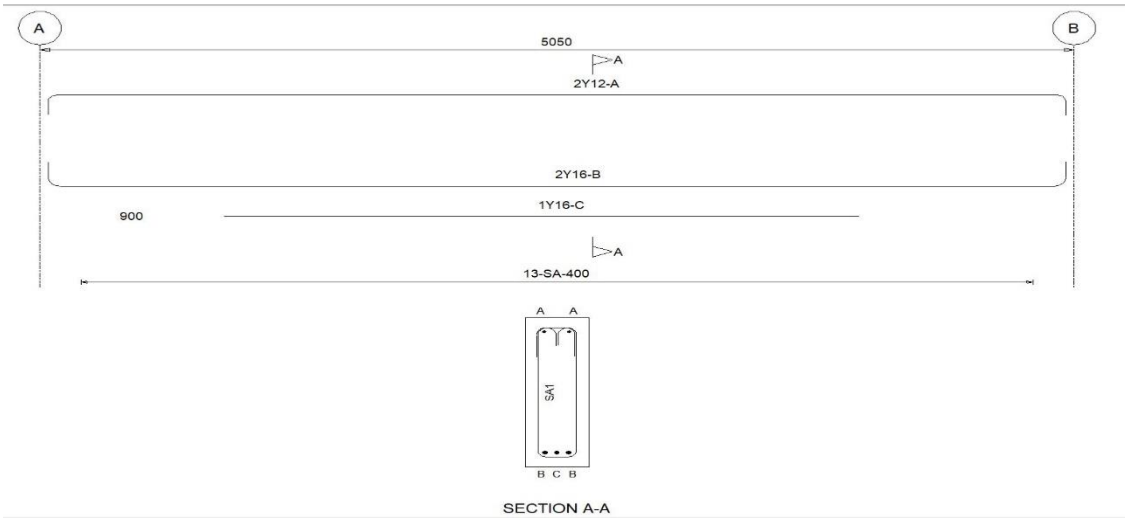


Figure 4.44: Section of Strip3

4- Strip 4:

✓ Load value:

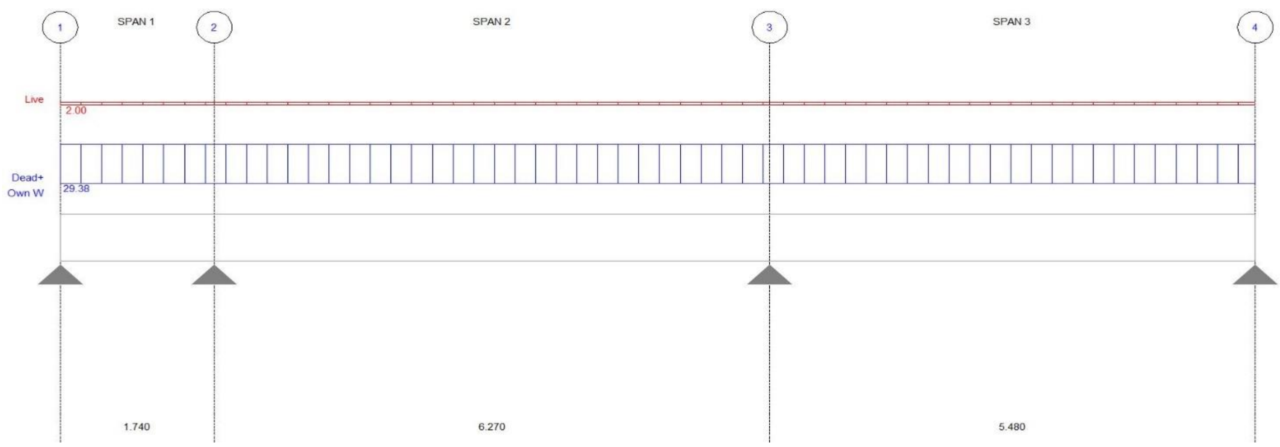


Figure 4.45: Prokon load value 4

✓ Deflection:

Using ACI-Code, Table 9.5 (b) Case 4.

Permissible deflection = 4.25/240

= 1.77 cm

Actual deflection = 1.021 cm <1.77 (okay)

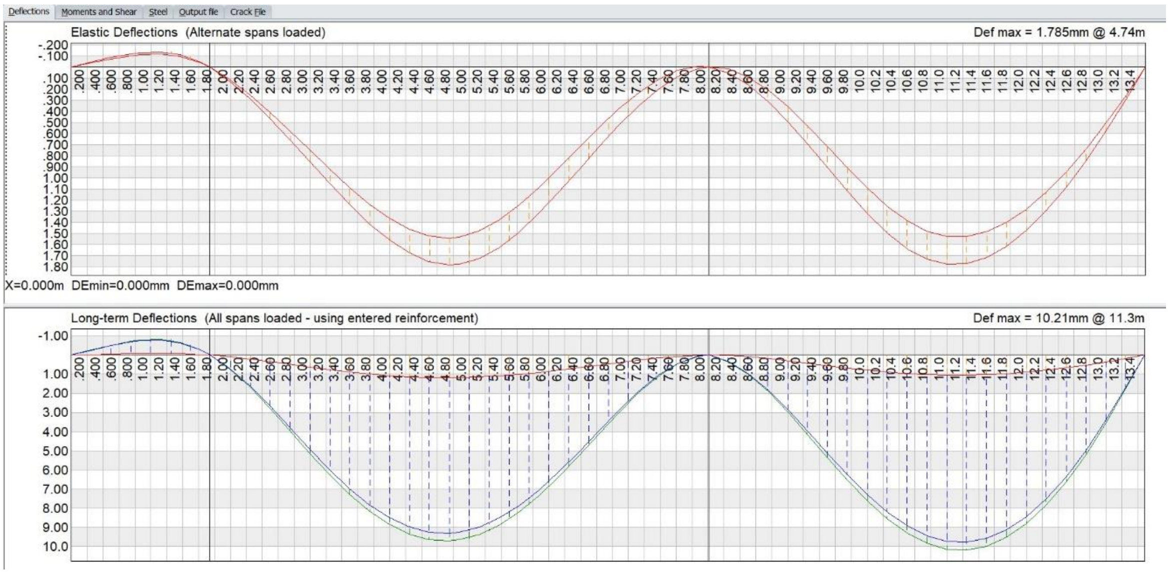


Figure 4.46: Prokon Defletion 4

✓ Moment & Shear value:

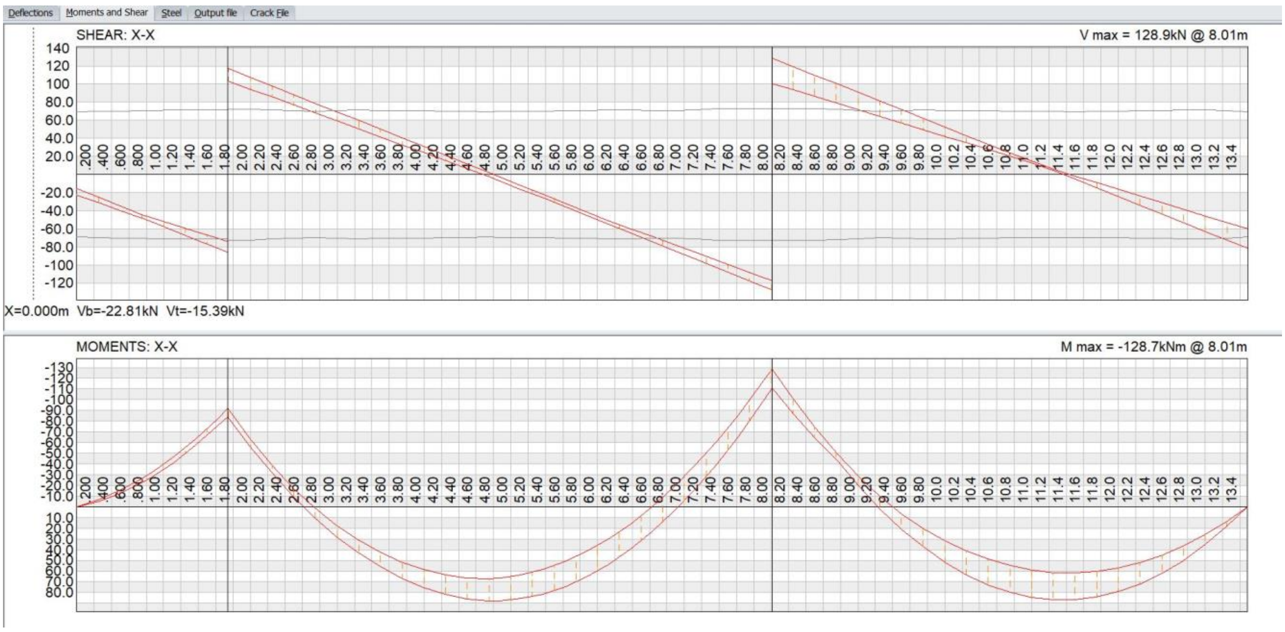


Figure 4.47: Prokon roof moment and shear 4

✓ Section of Strip:

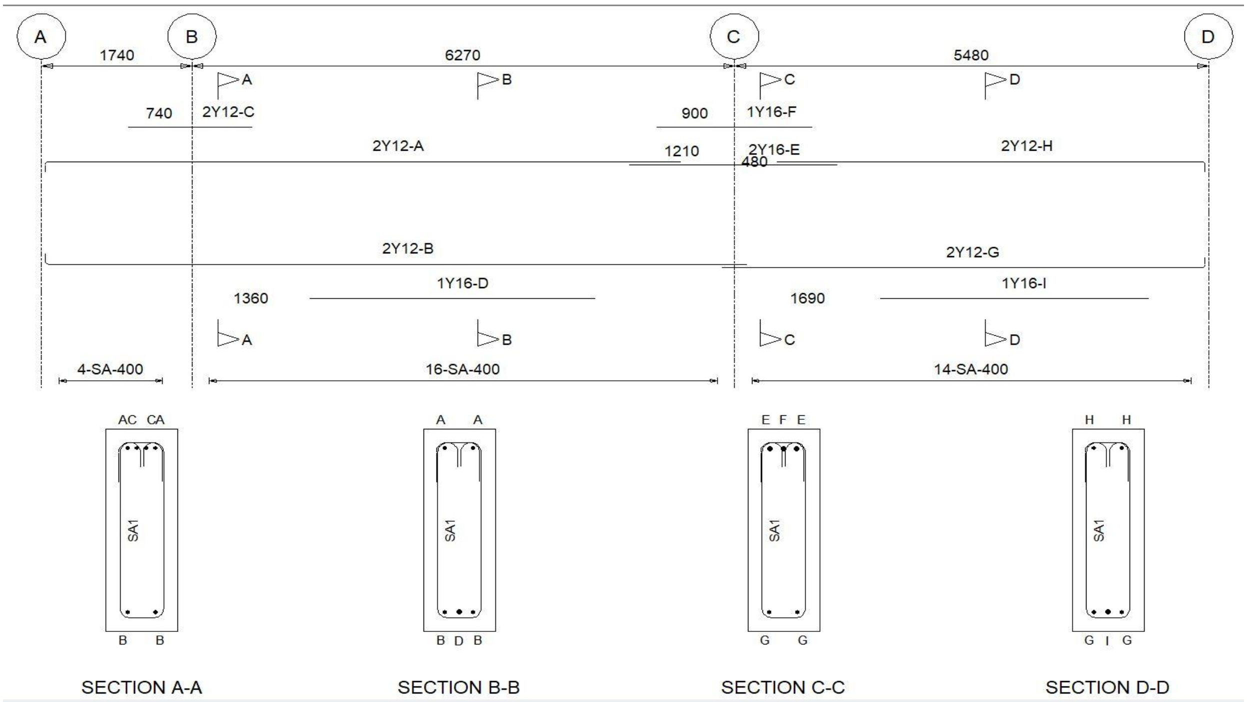


Figure 4.50: Section of Strip4

5- Strip 5:

✓ Load value:

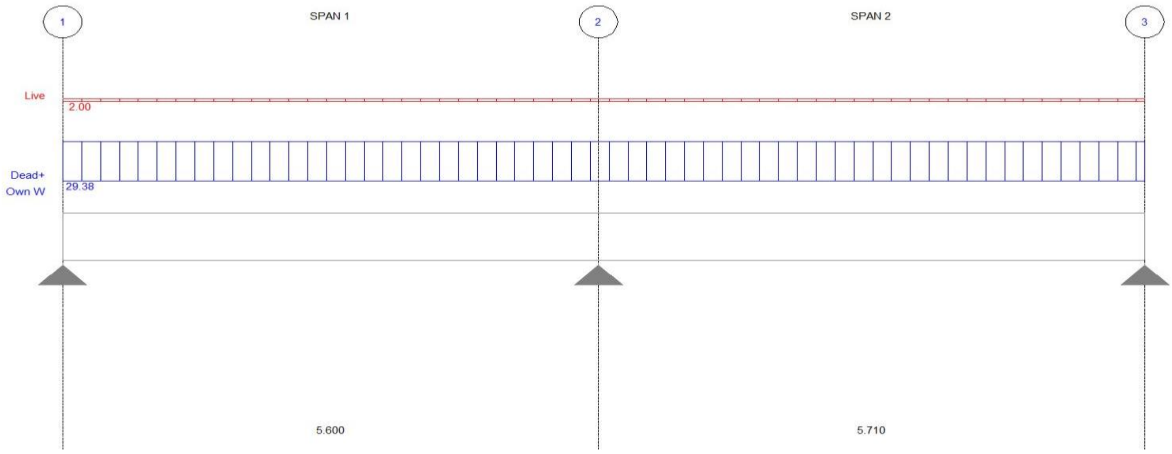


Figure 4.51: Prokon load value 5

✓ Deflection:

Using ACI-Code, Table 9.5 (b) ➡ Case 4.

Permissible deflection = 5.15/240

= 2.14 cm

Actual deflection = 1.164cm < 2.14 (okay)

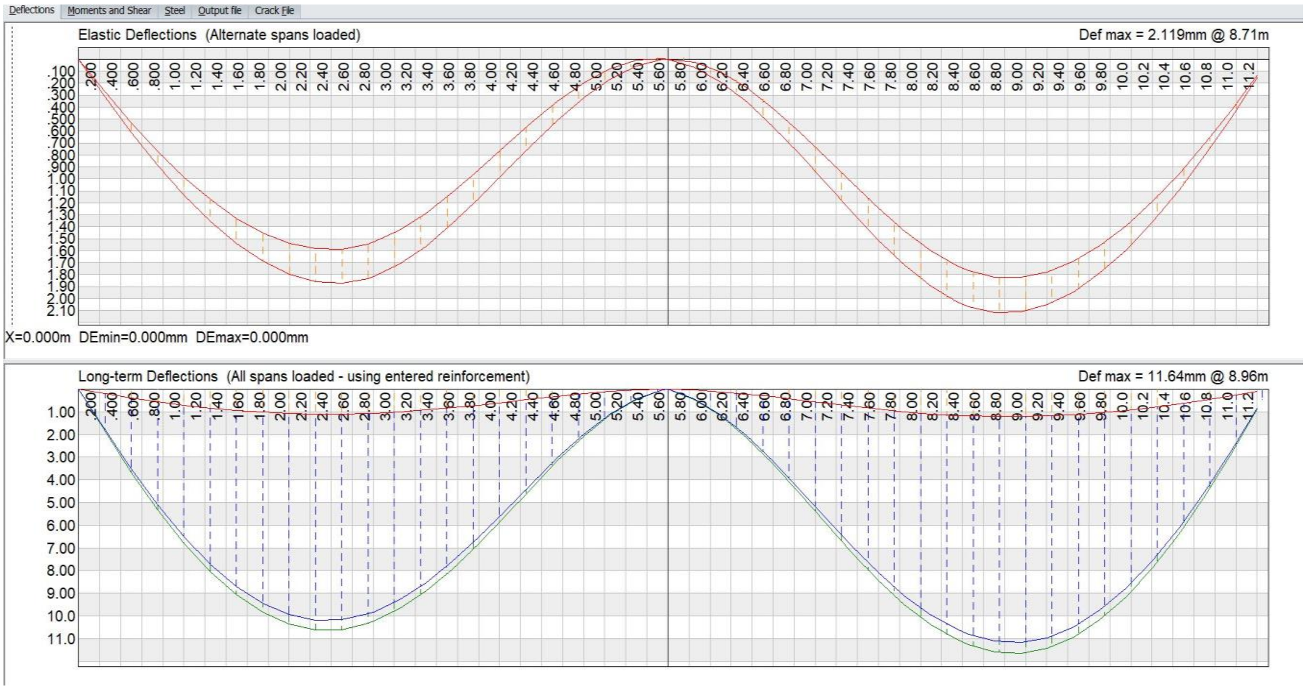


Figure 4.52: Prokon Defletion 5

✓ Moment & Shear value:

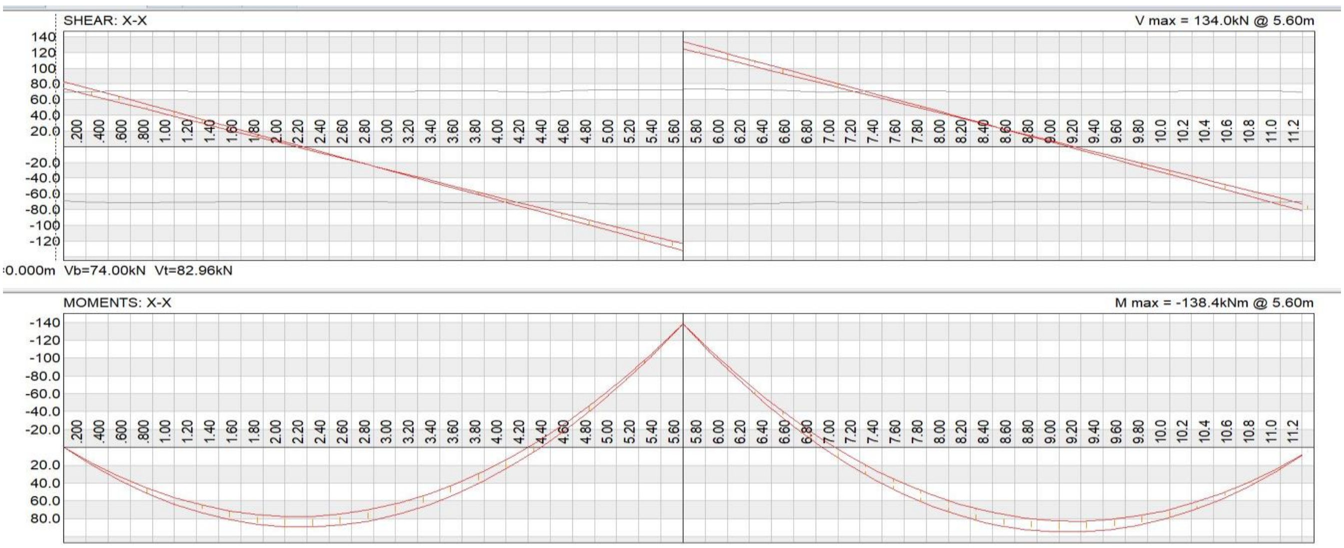


Figure 4.53: Prokon roof moment and shear 5

✓ Section of Strip:

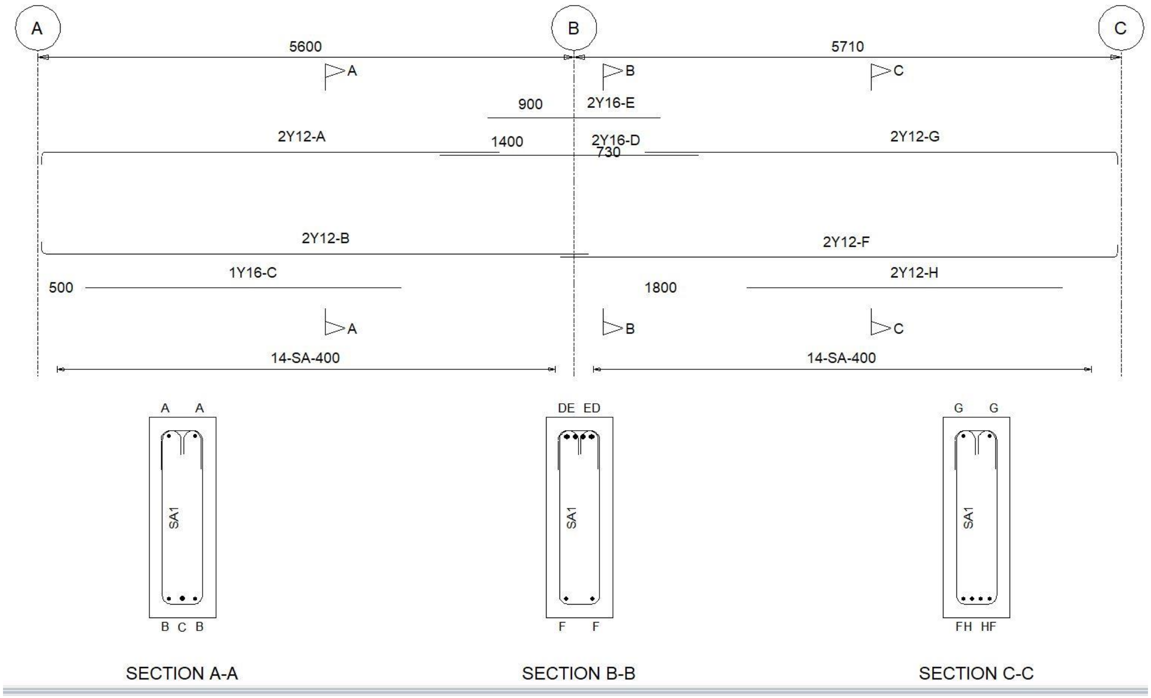


Figure 4.56: Section of Strip5

6- Strip 6:

✓ Load value:

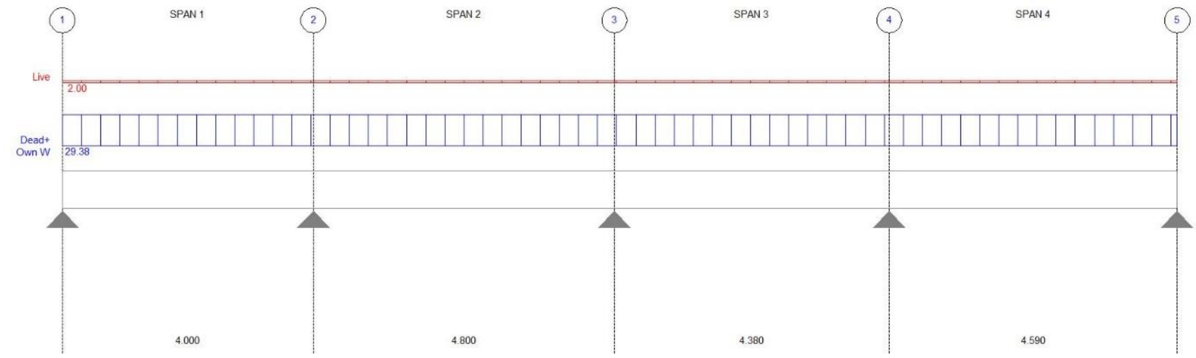


Figure 4.57: Prokon load value 6

✓ Deflection:

Using ACI-Code, Table 9.5 (b) ➡ Case 4.

Permissible deflection = 7.75/240

= 3.22 cm

Actual deflection = 0.716 cm < 3.22 (okay)

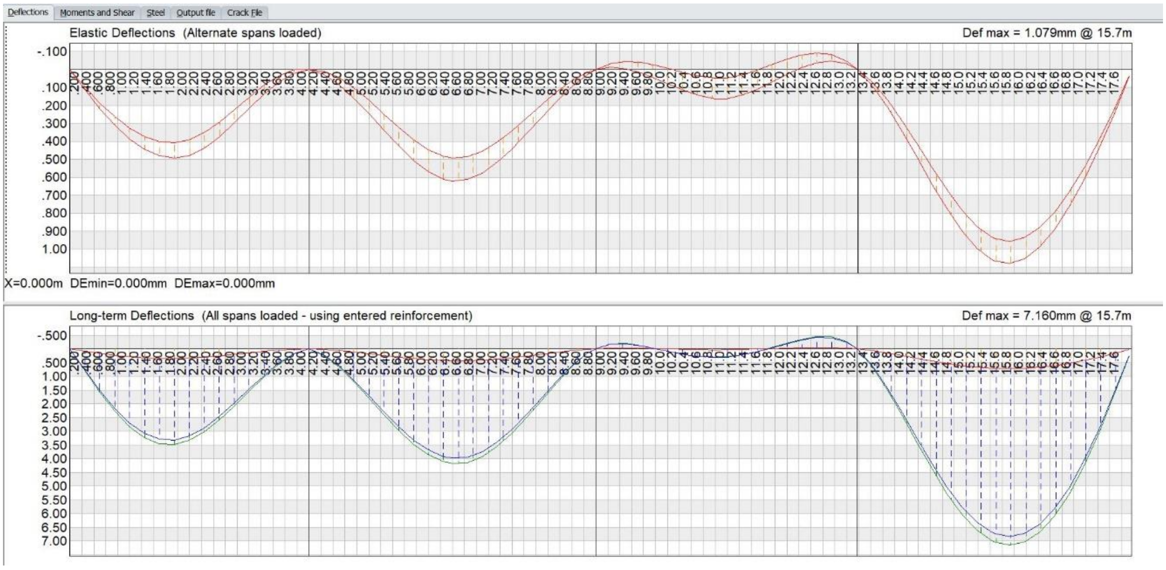


Figure 4.58: Prokon Defletion 6

✓ Moment & Shear value:

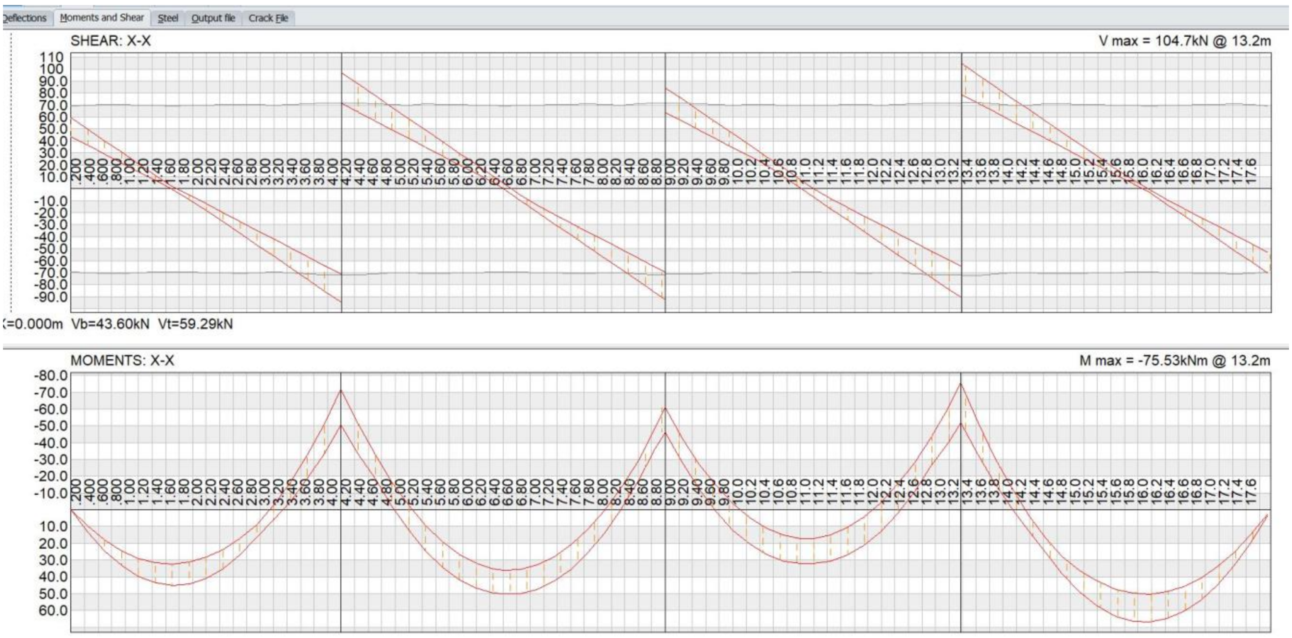


Figure 4.59: Prokon roof moment and shear 6

✓ Section of Strip:

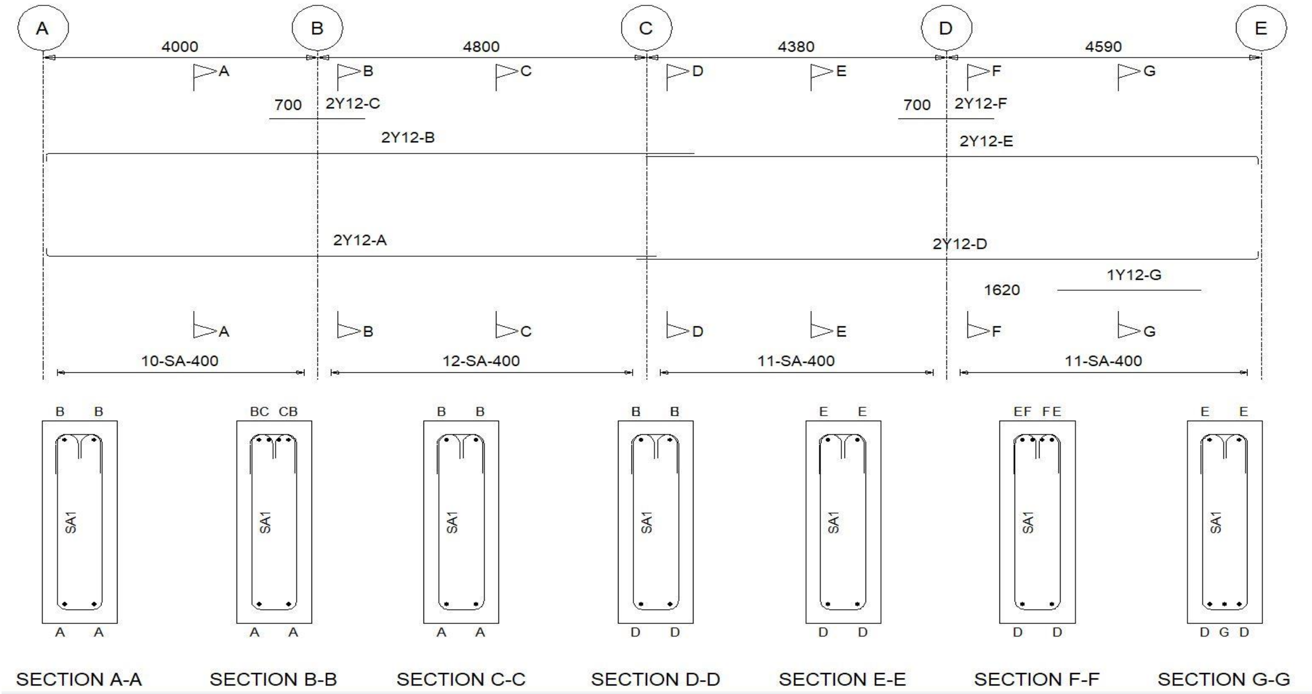


Figure 4.62: Section of Strip 6

❖ Summary of Reinforcement:

Table 4-1: Summary of reinforcement by prokon for typical slab

<table>
<tr><th>Floor</th><th>Strip 1</th></tr>
<tr><td>Typical</td><td>

✓ Main Reinforcement:

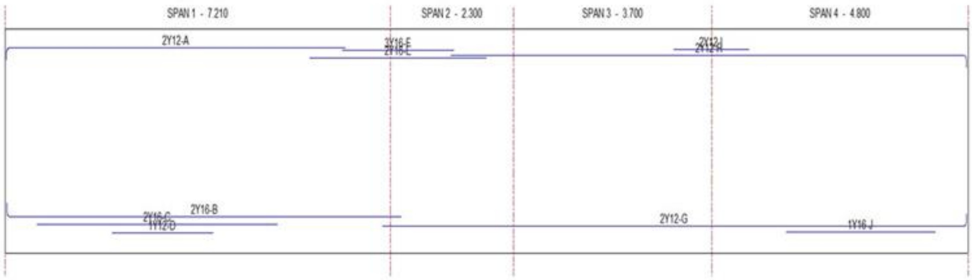


Figure 4.11: Prokon typical reinforcement moment 1

✓ Shear Reinforcement:

Figure 4.12: Prokon typical reinforcement shear 1

</td></tr>
</table>

Strip 2

✓ Reinforcement (Moment):

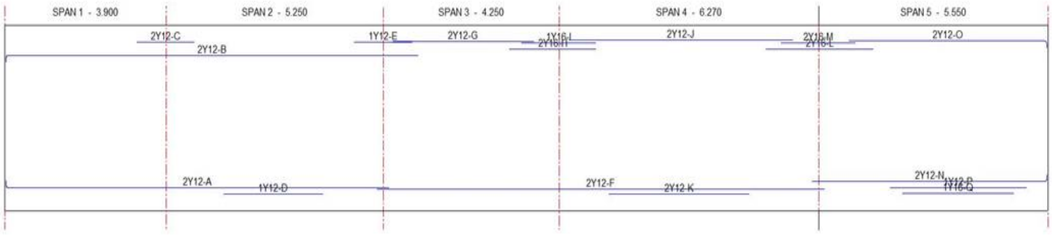


Figure 4.17: Prokon typical reainforcment moment 2

✓ Reinforcement (Shear):

Figure 4.18: Prokon typical reinforcement shear 2

Strip 3

✓ Reinforcement (Moment):

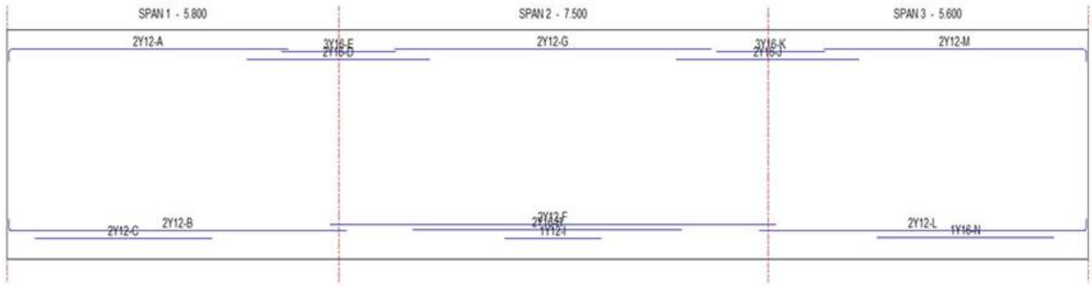


Figure 4.23: Prokon typical reainforcment moment 3

Reinforcement (shear):

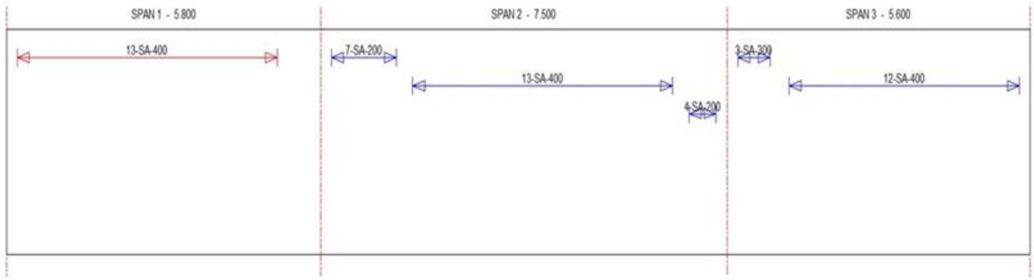


Figure 4.24: Prokon typical reainforcment shear 3

Table 4-2: Summary of rainforcment by prokon for roof slab

<table>
<tr><th>Floor</th><th>Strip 1</th></tr>
<tr><td rowspan="3">Roof</td><td>

✓ Reinforcement (Moment):

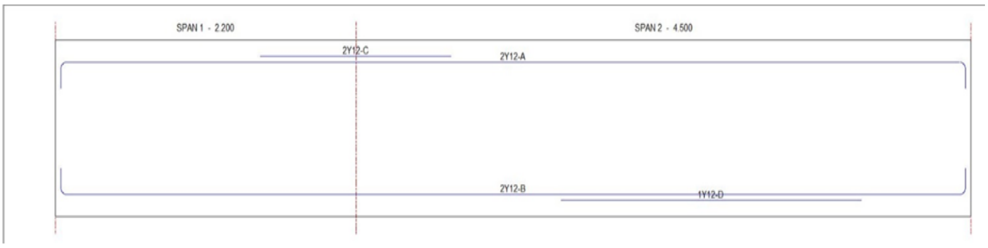


Figure 4.30: Prokon roof reainforcment moment1

Reinforcement (Shear):

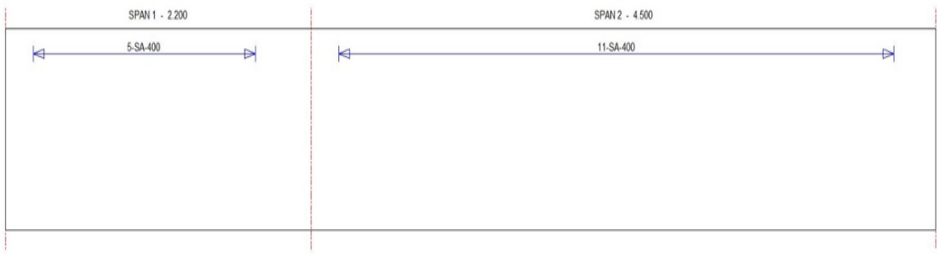


Figure 4.31: Prokon roof reainforcment shear1

</td></tr>
<tr><td>Strip 2</td></tr>
<tr><td>

✓ Reinforcement (Moment):

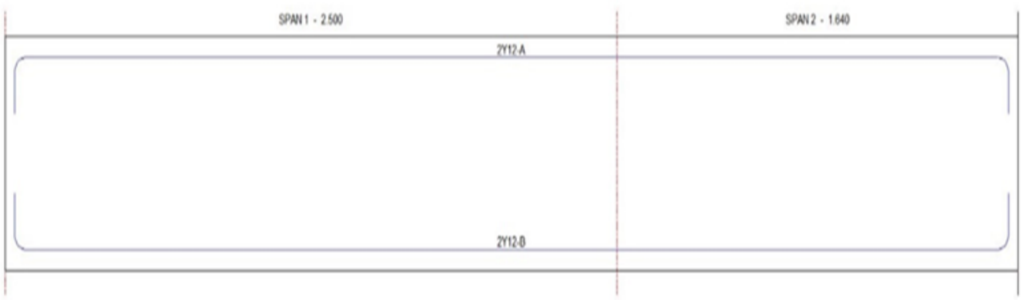


Figure 4.36: Prokon roof reinforcement moment 2

</td></tr>
</table>

Reinforcement (Shear):

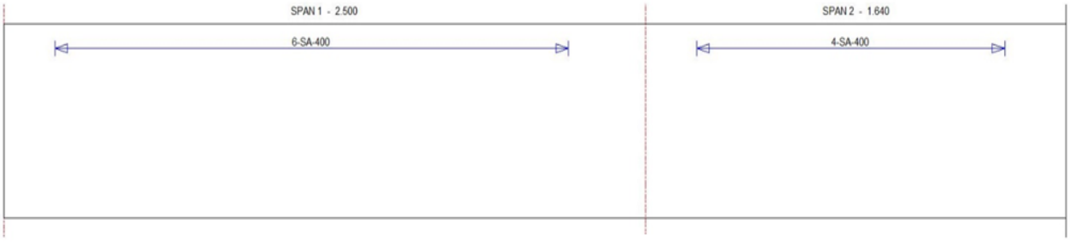


Figure 4.37: Prokon roof reinforcement shear 2

Strip 3

- ✓ Reinforcement (Moment):

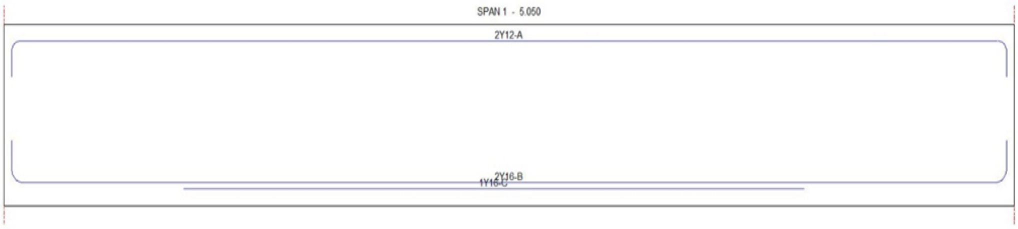


Figure 4.42: Prokon roof reinforcement moment 3

- ✓ Reinforcement (Shear):

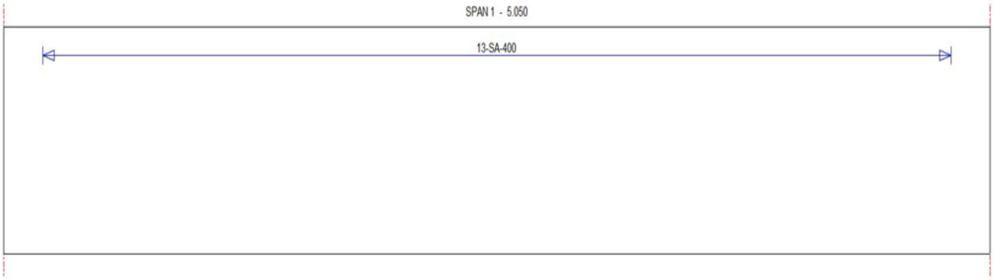


Figure 4.43: Prokon roof reinforcement shear 3

Strip 4

✓ Reinforcement (Moment):

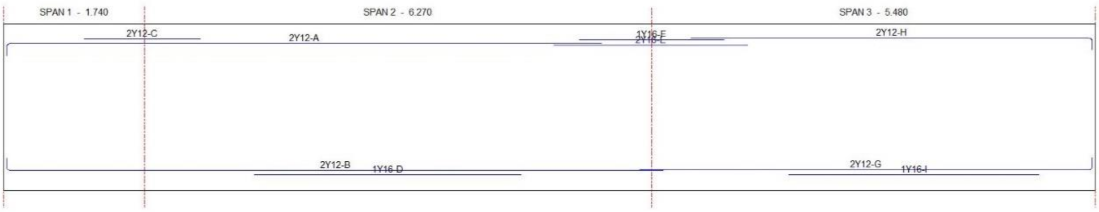


Figure 4.48: Prokon roof reainforcment moment 4

✓ Reinforcement (Shear):

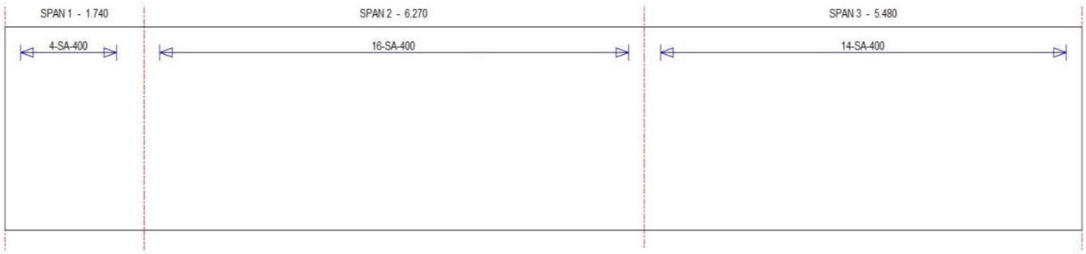


Figure 4.49: Prokon roof reinforcement shear 4

Strip 5

✓ Reinforcement (Moment):

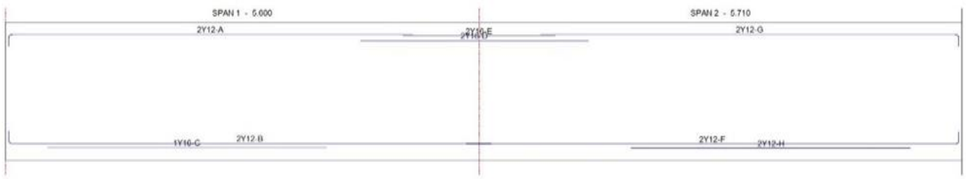


Figure 4.54: Prokon roof reinforcement moment 5

✓ Reinforcement (Shear):

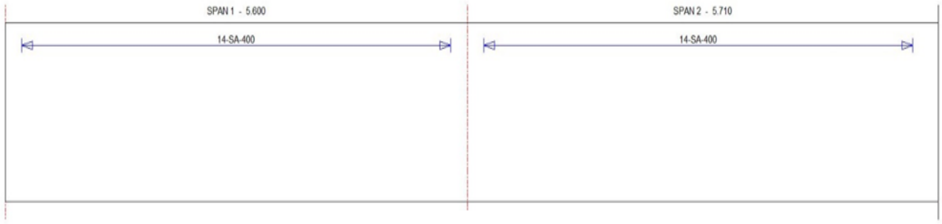


Figure 4.55: Prokon roof reinforcement shear 5

Strip 6

✓ Reinforcement (Moment):

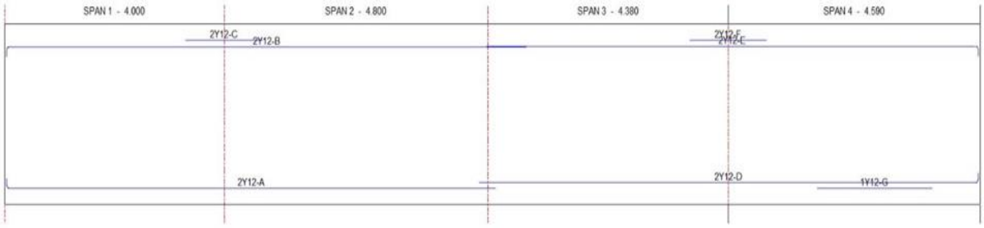


Figure 4.60: Prokon roof reainforcment moment 6

✓ Reinforcement (Shear):

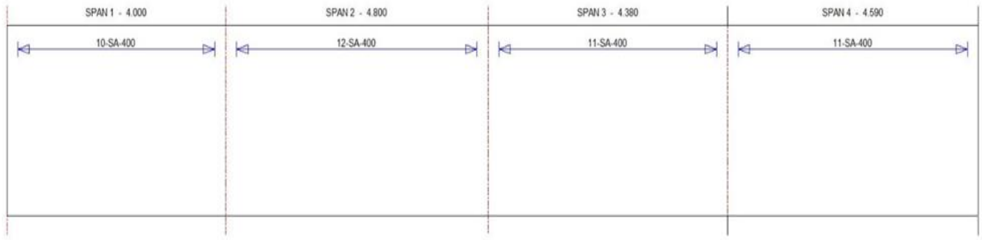


Figure 4.61: Prokon roof reainforcment shear 6

☒ Roof checks:

❖Strip 2 (first span) will be checked as sample of calculation:

✓ Check As_{min} :

$$As_{min} = \begin{cases} 0.25\sqrt{f'c}\ bd & = 325 \\ \frac{1.4\ bd}{Fy} & = 340\ \ control \end{cases}$$

As=2310mm > 340 (Ok)

✓ Check As_{max}

$$As_{max} = 0.319\beta\ \frac{f'c}{Fy}\ bd \quad = 2169.2\ mm$$

As=2310 mm < 2169.2 (OK)

✓ Check S_{max}:

$$S_{max} = \begin{cases} 600\ mm & = 600mm \\ 300\ mm & = 300mm \\ \frac{Av\ Fy}{0.33bw} & = 1000mm \\ \frac{16\ Av\ Fy}{bw\ \sqrt{f'c}} & = 1000mm \\ \frac{Av\ Fy\ d}{Vs} & = 650mm \end{cases}$$

Actual Spacing = 25 mm < 300 (OK)

✓ Check S_{min}

$$S_{min} = \begin{cases} Bar\ dia & = 16\ mm \\ 25mm & = 25mm \\ dia.\ of\ vibrator & (\ unkown) \\ 1.33\ c.a\ size & (unkown) \end{cases}$$

Actual Spacing = 25 mm = 25 (OK)

" ***<u>All spans in every strip checked and its OK</u>*** "

Tie beams:

- Tie beams establish in the ground floor under every wall.
- 6 strip (side view) will be analyzed and designed.
- As a sample of calculation Strip 1 will be shown.

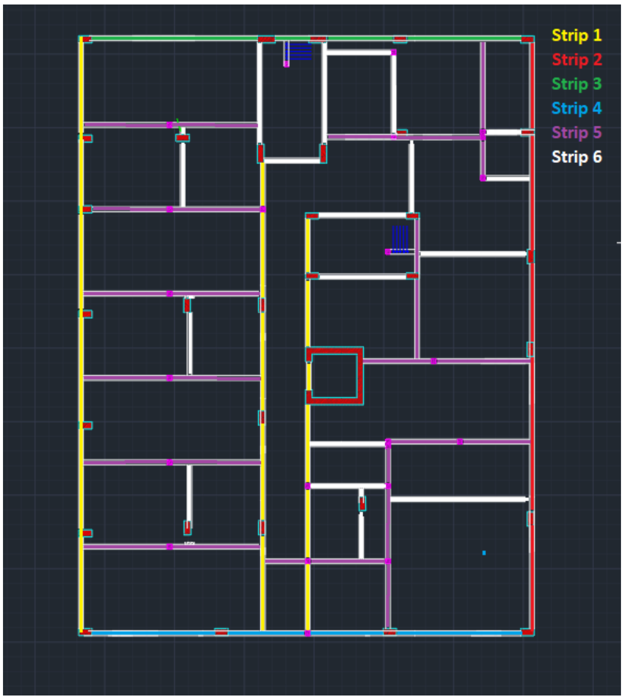
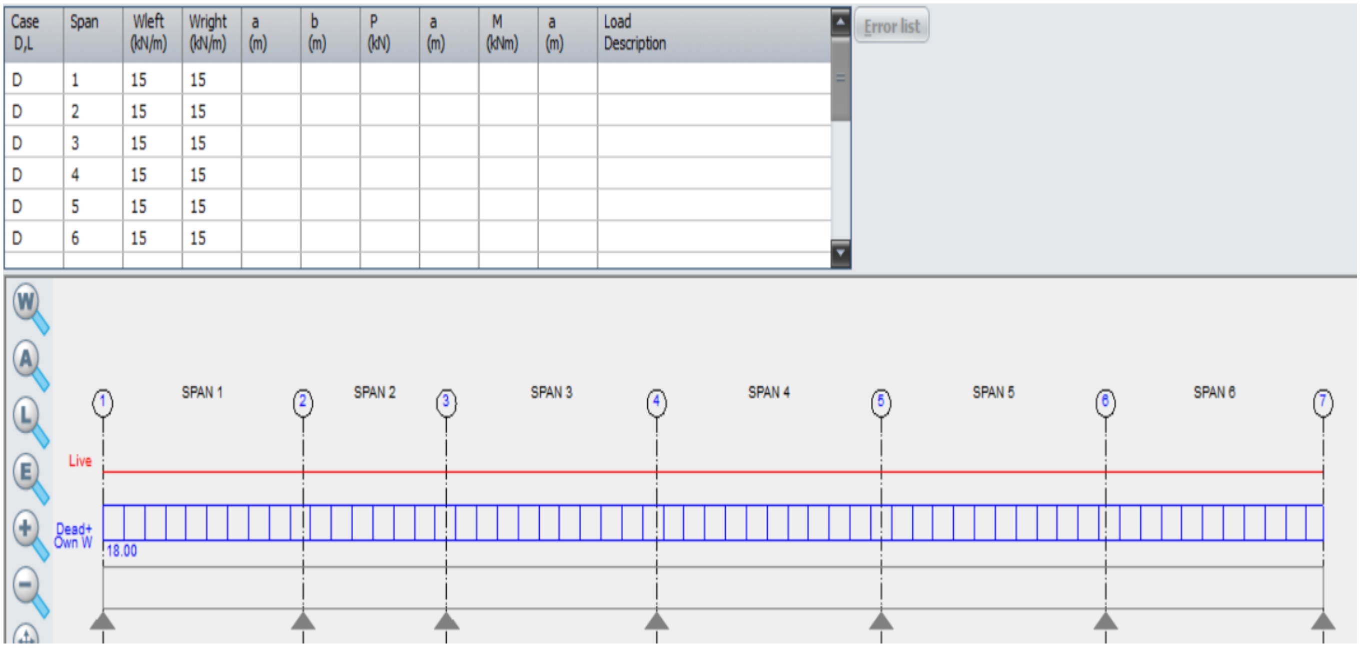


Figure 4.63: Tie beams strip

❖ Strip 1:

✓ Load value:



Case D,L	Span	Wleft (kN/m)	Wright (kN/m)	a (m)	b (m)	P (kN)	a (m)	M (kNm)	a (m)	Load Description
D	1	15	15							
D	2	15	15							
D	3	15	15							
D	4	15	15							
D	5	15	15							
D	6	15	15							

Figure 4.64: Prokon TB load 1

- ✓ Deflection:

Using ACI-Code, Table 9.5 (b) ➡ Case 4.

Permissible deflection = 4.7/240

= 1.95 cm

Actual deflection = 0.2886 cm < 1.95 (okay)

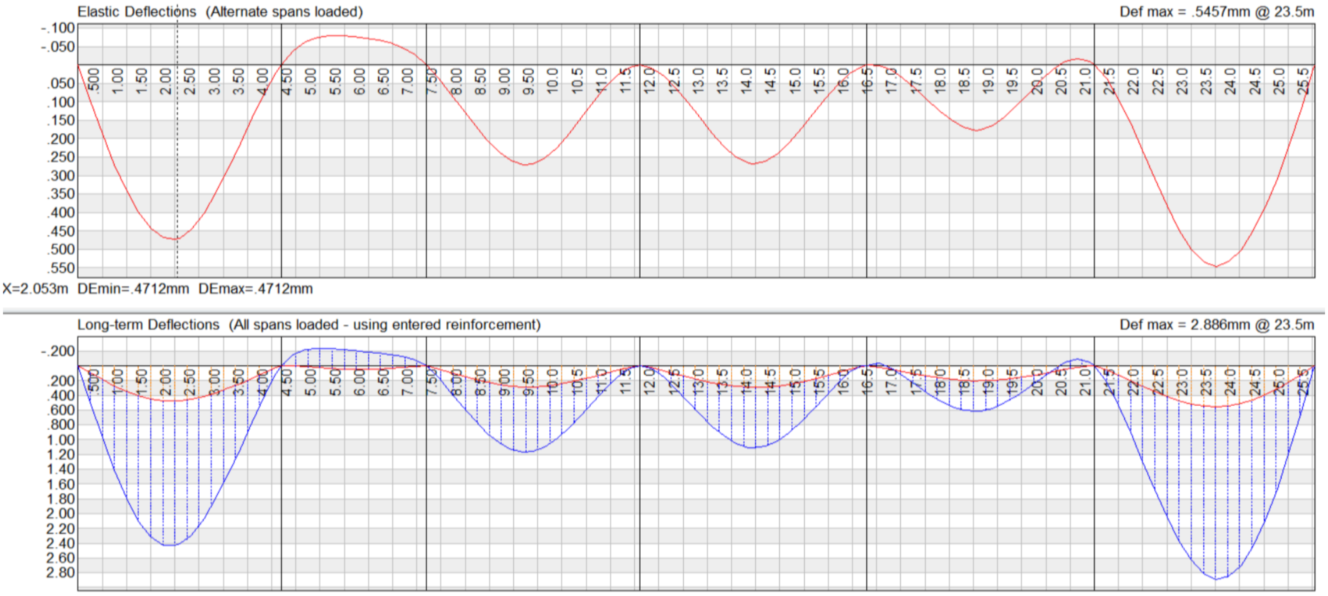


Figure 4.65: Prokon TB deflection 1

- ✓ Moment & Shear value:

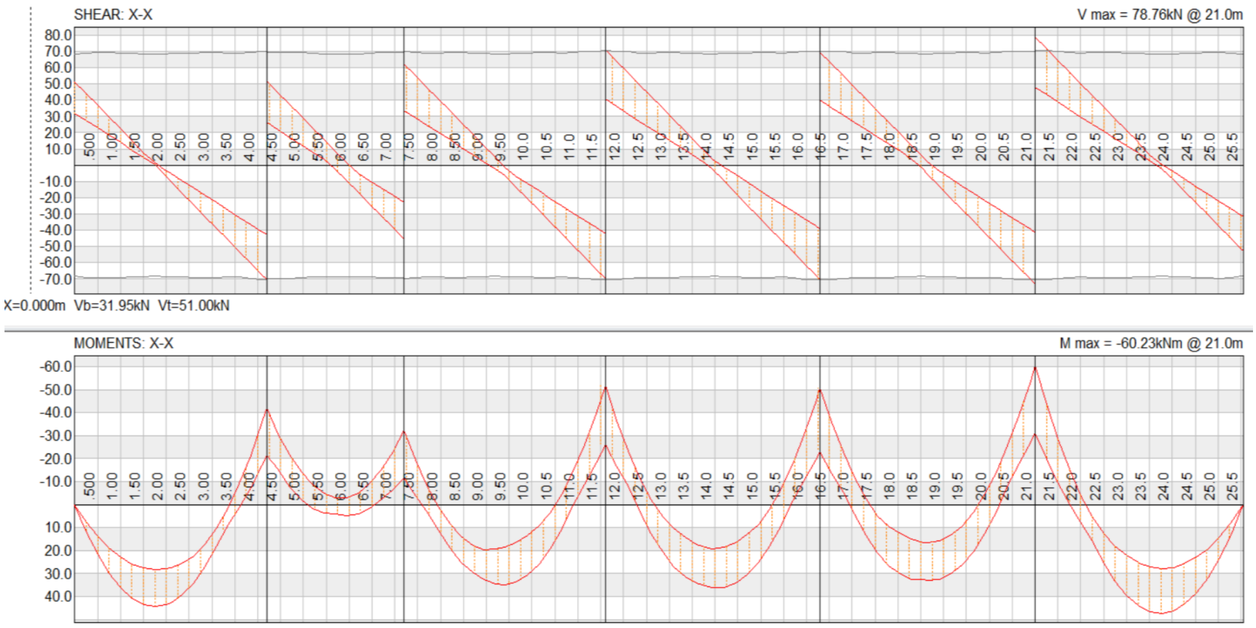


Figure 4.66: Prokon TB moment and shear 1

✓ Reinforcement (Moment):

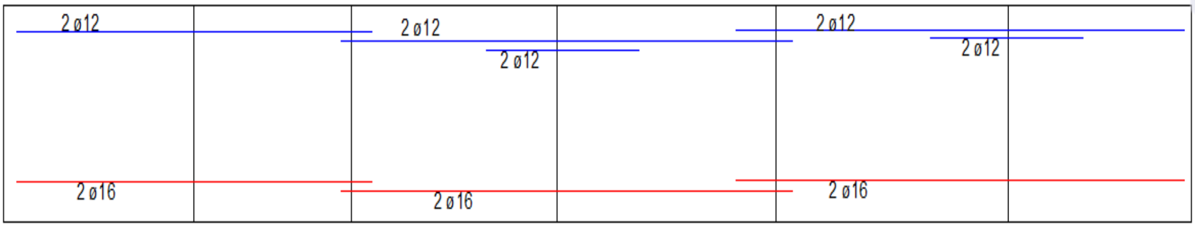
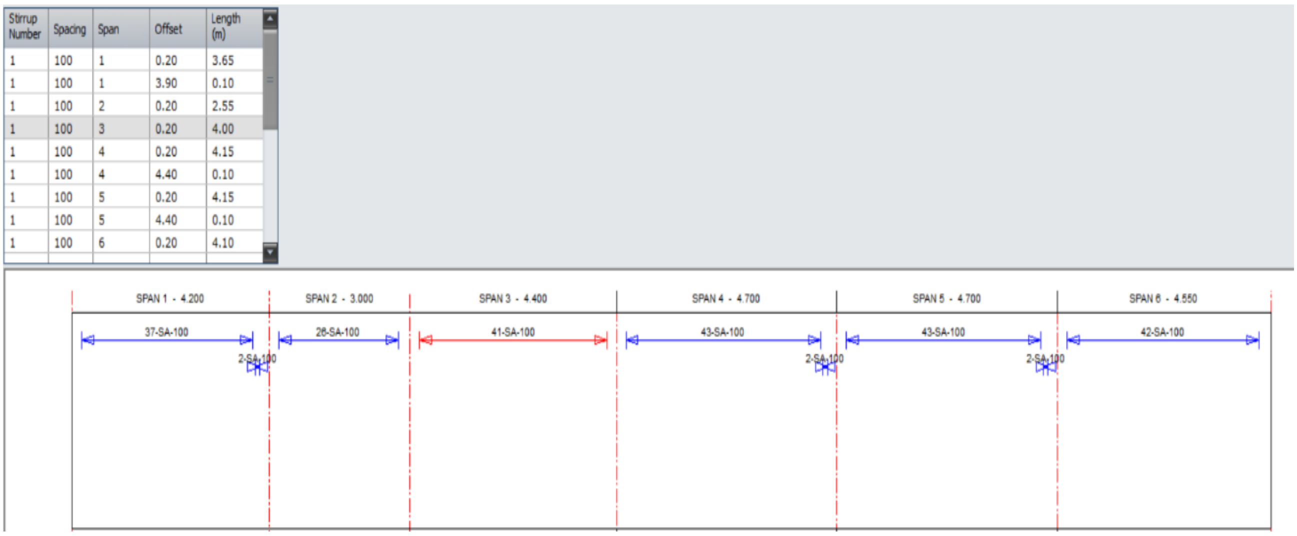


Figure 4.67: Prokon TB reainforcment moment 1

✓ Reinforcement (Shear):

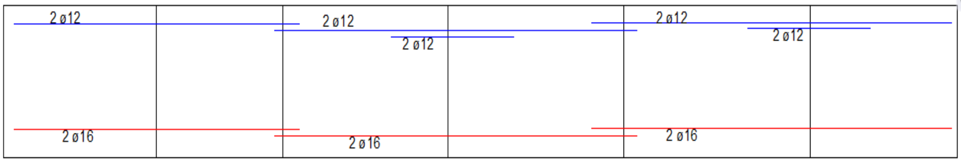
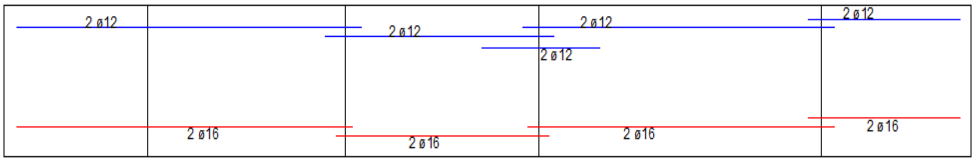
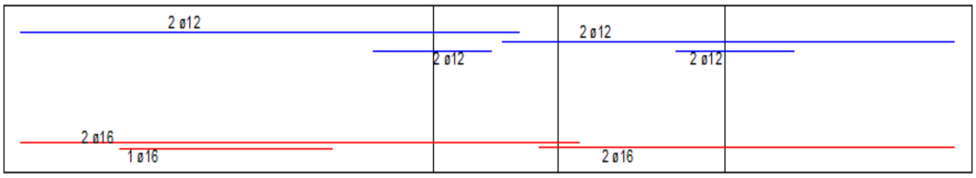
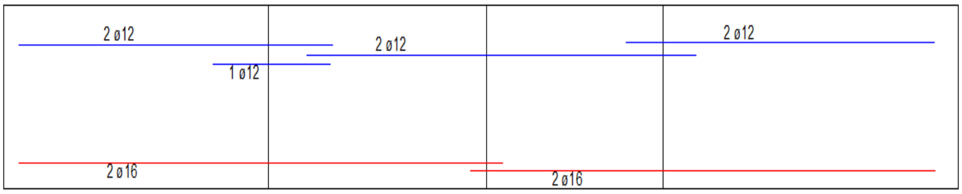
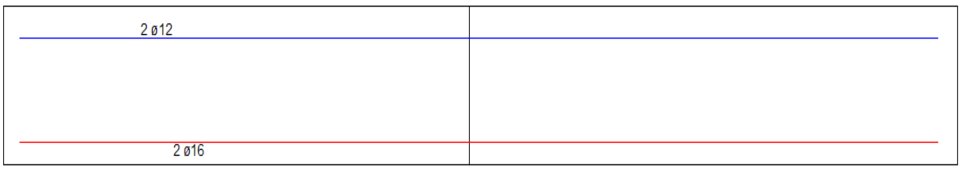
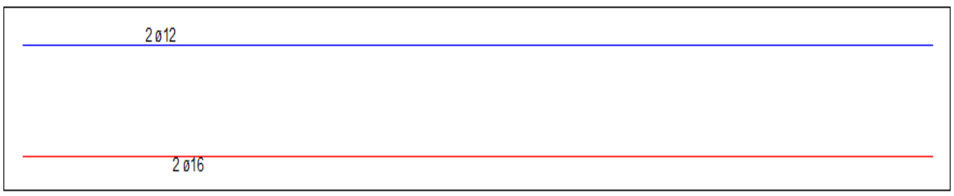


Stirrup Number	Spacing	Span	Offset	Length (m)
1	100	1	0.20	3.65
1	100	1	3.90	0.10
1	100	2	0.20	2.55
1	100	3	0.20	4.00
1	100	4	0.20	4.15
1	100	4	4.40	0.10
1	100	5	0.20	4.15
1	100	5	4.40	0.10
1	100	6	0.20	4.10

Figure 4.68: Prokon TB reainforcment shear 1

⇒As our studying for all strip we found these results:

Table 4-3: Summary of rainforcment by prokon for

<table>
<tr><th>Floor</th><th>Strip 1</th></tr>
<tr><td rowspan="11">(Ground) Tie Beams</td><td>
</td></tr>
<tr><td>Strip 2</td></tr>
<tr><td>
</td></tr>
<tr><td>Strip 3</td></tr>
<tr><td>
</td></tr>
<tr><td>Strip 4</td></tr>
<tr><td>
</td></tr>
<tr><td>Strip 5</td></tr>
<tr><td>
</td></tr>
<tr><td>Strip 6</td></tr>
<tr><td>
</td></tr>
</table>

☒ Ground floor (Tie beams) checks:

❖Strip 3 (first span) will be checked as sample of calculation:

✓ Check As_{min} :

$$As_{min} = \begin{cases} 0.25\sqrt{f'c}\ bd & = 325 \\ \frac{1.4\ bd}{Fy} & = 340\ control \end{cases}$$

As=630 mm > 340 (Ok)

✓ Check As_{max}

$$As_{max} = 0.319\beta \frac{f'c}{Fy} bd = 2169.2\ mm$$

As=630 mm < 2169.2 (OK)

✓ Check S_{max}:

$$S_{max} = \begin{cases} 600\ mm & = 600mm \\ 300\ mm & = 300mm \\ \frac{Av\ Fy}{0.33bw} & = 1000mm \\ \frac{16\ Av\ Fy}{bw\sqrt{f'c}} & = 1000mm \\ \frac{Av\ Fy\ d}{Vs} & = 650mm \end{cases}$$

Actual Spacing = 66 mm < 300 (OK)

✓ Check S_{min}

$$S_{min} = \begin{cases} Bar\ dia & = 16\ mm \\ 25mm & = 25mm \\ dia.\ of\ vibrator & (unkown) \\ 1.33\ c.a\ size & (unkown) \end{cases}$$

Actual Spacing = 66 mm > 25 (OK)

" ***All spans in every strip checked and its OK*** "

Chapter 5: columns

Columns are defined as vertical structural members subjected not only to compression force but also to bending moment about one or both axes of the section, and with a height at least three times its least lateral dimension. They collect loads from the different floors and then transfer it to the soil through the foundation.

5.1 Type of columns

Columns may be classified based on the following different categories:

1. Based on the position of the load on the cross section:

a. Axially loaded columns.

b. Eccentrically loaded columns.

c. Biaxial loaded columns.

2.Based on the length of the column in relation to its lateral dimensions:

a. Short columns.

b. Slender columns.

3. Based on shape and its arrangement of the reinforcement:

a. Tied columns.

b. Spiral column.

c. Composite column

5.2 Design

In this project, columns are subjected only to axial load (neglected effect of bending moment min eccentricity).

Design of Tied Reinforced Concrete Columns i.e. Axial force read from ETABS for col.(3).

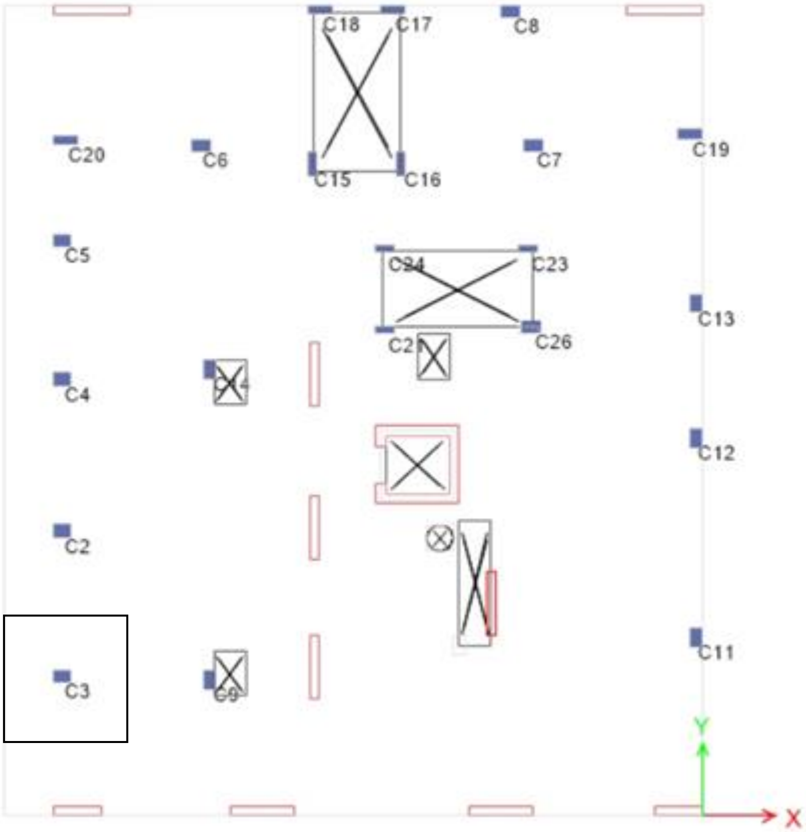


Figure 5.1: column selection in manual calculation

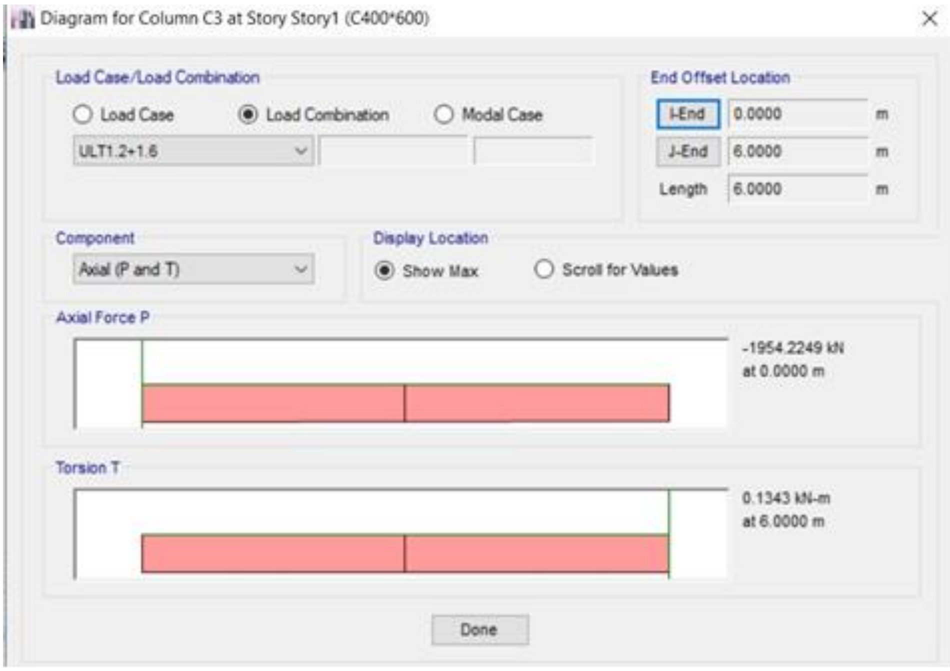


Figure 5.2: PU on the column (#3) from ETABS program

Using ETABS program, PU on the Column is 1954.22Kn

Rectangular column

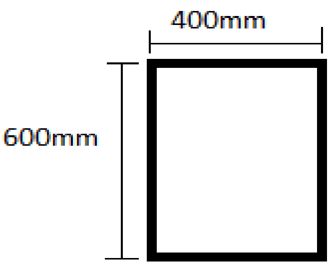


Figure 5.3: column section

Ag= 600×400

Ag =240000 mm^2

$$\gamma = \frac{h-2x}{h}$$

Where x = cover + ds + db/2

Assume Bar Diameter =16mm

X= 40 +10+ $\frac{16}{2}$ = 58mm

$\gamma = \frac{600-2*58}{600} = 0.806$

Also from ETABS

Mu=12Kn.m

- Reinforcement Ratio (ρ_g)
  - Minimum = 1% of the column gross area
  - Maximum = 8% of the column gross area
  - In practice, limit $\rho_{g,max}$ to 4% of gross area
  - For architectural reasons ρ_g = 0.5% is allowed
- Clear distance between bars > 1.5d_b or 40mm

$$S_{\max} = the\ smallest\ of \begin{cases} 16d_b \\ 48d_t \\ b \end{cases}$$

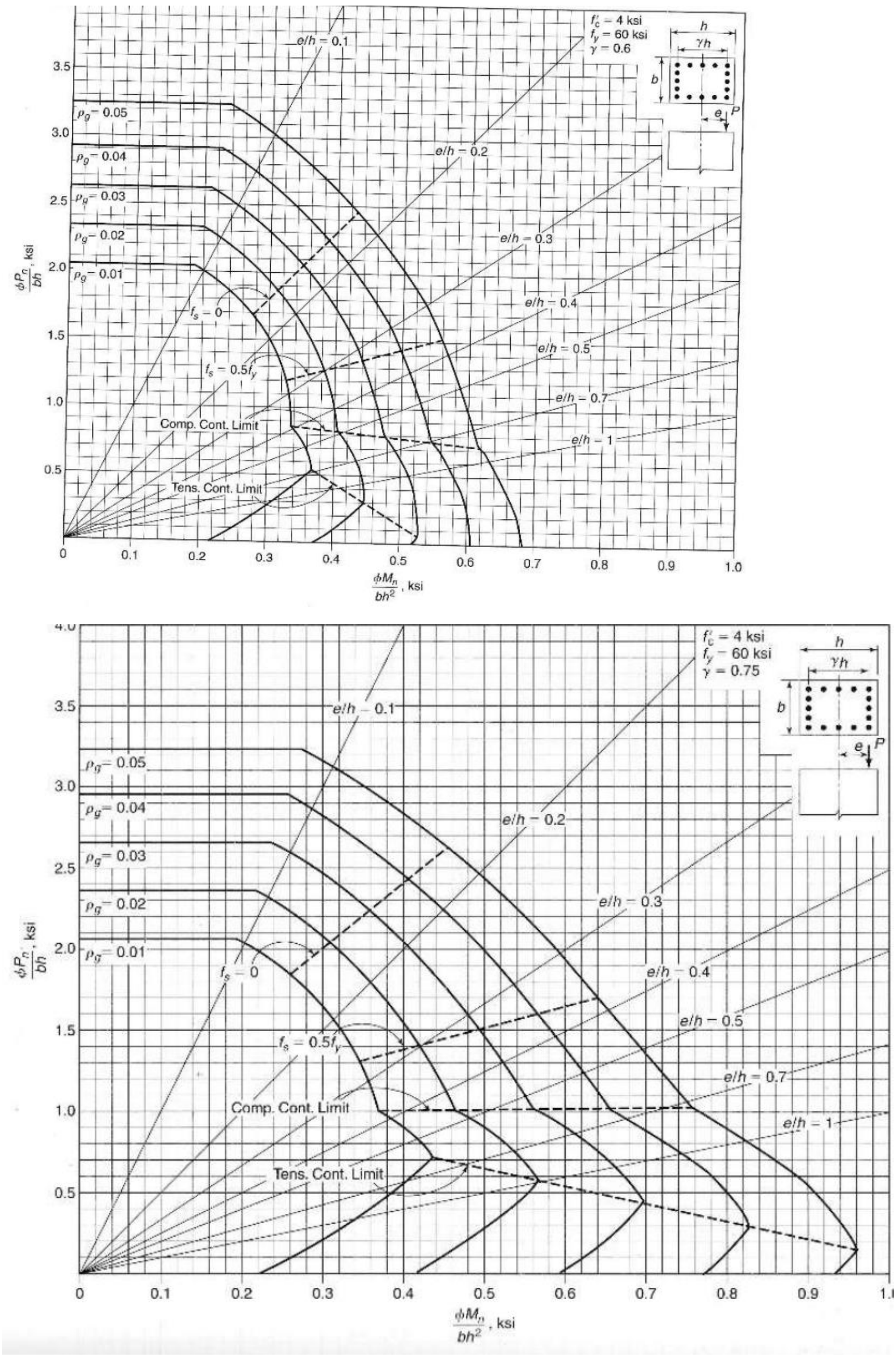


Figure 5.4: non dimensional interaction diagram for rectangular tied column

$\frac{\Phi\ Pn}{b\ h} = \frac{1954*10^3}{240000}$ = 8.15MPa = 1.18Ksi

$\frac{\Phi Mn}{bh^2} = \frac{12.16*10^6}{400*600^2}$ = 0.025Ksi

The value of ((ρ= 0.01))

Value of reinforcement required:

As=0.01*Ag =0.01*400*600=2400mm^2

Try 12ϕ16 = 2400mm^2

- Check on:

1. As(max)= 0.04*Ag= 0.04*400*600=9600mm^2 >As(use) ~Ok

2. Spacing between bar = largest of $\left\{\begin{matrix} 1.5db = 1.5*16 = 24 \\ 40mm \end{matrix}\right\}$

Then S=40mm (min)

3.Ties spacing = smallest of $\left\{\begin{matrix} 16db = 256mm \\ 48dt = 48 \times 10 = 480mm \\ b = 400mm \end{matrix}\right\}$

S of ties = 256=~250 mm ((min)).

4.ϕPn = 0.8(0.85*f′c*(Ac-As)+(fy*As))

ϕPn =0.8{0.85*28*(400*600-2400)+(460*2400)}

ϕPn =5627.91KN > Pu=1954.22KN ~ Ok

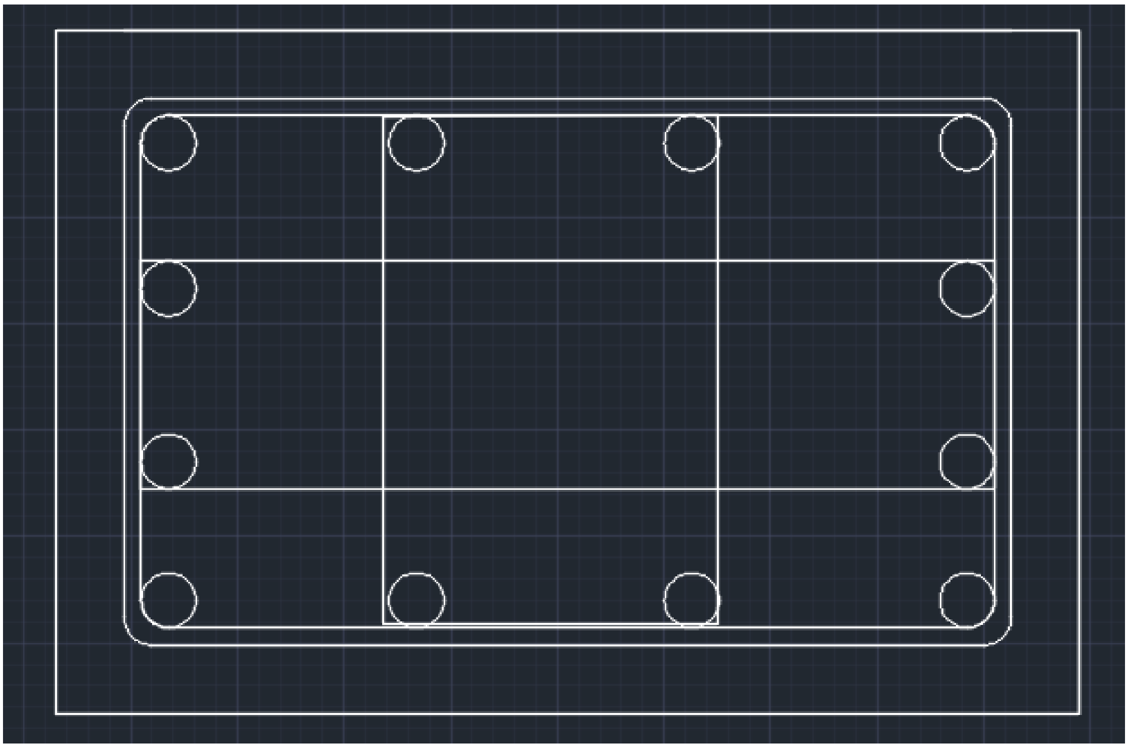


Figure (5.5): Cross section for column ((#3))

- Assumptions:

1. The section is symmetrically reinforced.

2. The specified design axial loads include the self-weight of the column.

3. The design axial loads are taken constant over the height of the column.

--- Design of Column-Footing Joint (Dowels)

12.3 — Development of deformed bars and deformed wire in compression

12.3.1 — Development length for deformed bars and deformed wire in compression, ℓ_{dc}, shall be determined from 12.3.2 and applicable modification factors of 12.3.3, but ℓ_{dc} shall not be less than 200 mm.

12.3.2 — For deformed bars and deformed wire, ℓ_{dc} shall be taken as the larger of $(0.24f_y/\lambda\sqrt{f_c'})d_b$ and $(0.043f_y)d_b$, with λ as given in 12.2.4(d) and the constant 0.0003 carries the unit of mm²/N.

12.3.3 — Length ℓ_{dc} in 12.3.2 shall be permitted to be multiplied by the applicable factors for:

$$l_{dh} = \frac{0.24 f_y \psi_e \lambda}{\sqrt{f_c}} d_b \geq \text{larger of} \left| \begin{array}{l} 8d_b \\ 150mm \end{array} \right.$$

12.16.1 — Compression lap splice length shall be $0.071f_y d_b$, for f_y of 420 MPa or less, or $(0.13f_y - 24)d_b$ for f_y greater than 420 MPa, but not less than 300 mm. For f_c' less than 21 MPa, length of lap shall be increased by one-third.

Figure (5.6): development length from ACI code

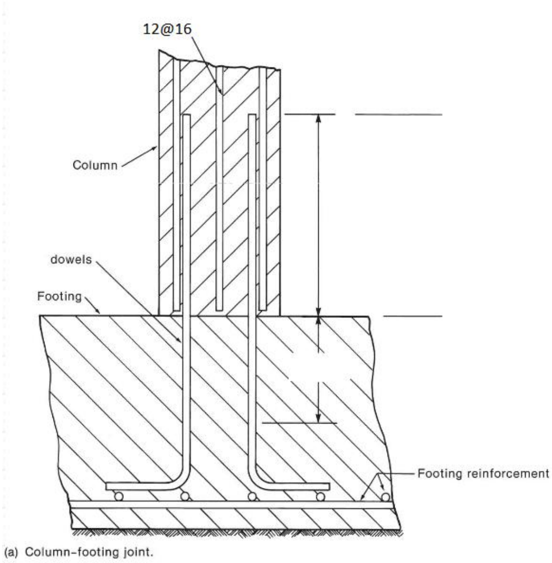


Figure (5.7):cross section of dowels details

As Dowels = 0.005A1 = 0.005*600*400

= 1200m2 ⇒ **6Φ16**

12.16.1 — Compression lap splice length shall be $0.071f_y d_b$, for f_y of 420 MPa or less, or $(0.13f_y - 24)d_b$ for f_y greater than 420 MPa, but not less than 300 mm. For f_c' less than 21 MPa, length of lap shall be increased by one-third.

-- Compression lap splice length = $0.071 f_y d_b$= 478mm

~ Extend dowels 500m in column

$$l_{dh} = \frac{0.24 f_y \psi_e \lambda}{\sqrt{f_c'}} d_b \geq \text{larger of} \left| \begin{array}{l} 8d_b \\ 150mm \end{array} \right.$$

Ldh=305 mm < 500mm (footing height) No need for hooks

5.3Final results

All Columns are Tied Rectangular Columns

Summary for all columns

Table 5.1: Summary for all columns

Section	Col. No	As	Bars
C400*600	C2	2400	12Φ16
C400*600	C3	2400	12Φ16
C400*600	C4	2400	12Φ16
C400*600	C5	2400	12Φ16
C400*600	C6	2400	12Φ16
C400*600	C7	2400	12Φ16
C400*600	C8	2400	12Φ16
C400*600	C9	2400	12Φ16
C400*600	C11	2400	12Φ16
C400*600	C12	2400	12Φ16
C400*600	C13	2400	12Φ16
C400*600	C14	2400	12Φ16
C300*800	C15	2400	12Φ16
C300*800	C16	2400	12Φ16
C300*800	C17	2400	12Φ16
C300*800	C18	2400	12Φ16
C300*800	C19	2400	12Φ16
C300*800	C20	2400	12Φ16
C250*600	C21	1600	8Φ16
C250*600	C23	1600	8Φ16
C250*600	C24	1600	8Φ16
C400*600	C26	2400	12Φ16

Chapter 6: Shear Walls

6.1 Introduction

Shear walls are vertical elements designed to resist axial load, lateral load, or both, with a horizontal length – to thickness ratio greater than 3 , used to enclose or separate spaces.-Two frequent characteristics of walls are their slenderness, height to thickness ratio, which is generally higher than for columns, and the reinforcement ratios, generally about a fifth to a tenth of those in columns.

-Major factors that affect the design of structural walls include the following:

(a) The structural function of the wall.

a. Bearing wall.

b. Non-Bearing wall.

(b) The types of loads the wall resists.

a. Ordinary wall for resist wind load.

b. Special wall for resist earthquakes load.

(c) The location and amount of reinforcement

6.2 Types of Walls

Structural walls can be classified as:

(a) Bearing walls—walls that are laterally supported and braced by the rest of the structure that resist primarily in-plane vertical loads acting downward on the top of the wall . The vertical load may act eccentrically with respect to the wall thickness, causing weak-axis bending

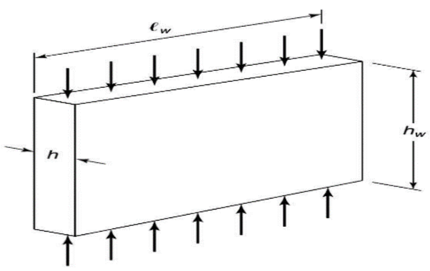


Figure 6.1: Bearing wall

b) Shear walls—walls that primarily resist lateral loads due to wind or earthquakes acting on the building are called shear walls or structural walls. These walls often provide lateral bracing for the rest of the structure. They resist gravity loads transferred to the wall by the parts of the structure tributary to the wall, plus, lateral-loads (lateral shears) and moments about the strong axis of the wall.

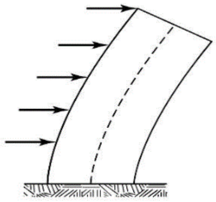


Figure 6.2: Shear wall

(c) Nonbearing walls—walls that do not support gravity in-plane loads other than their own weight. These walls may resist shears and moments due to pressures or loads acting on one or both sides of the wall. Examples are basement walls and retaining walls used to resist lateral soil pressures.

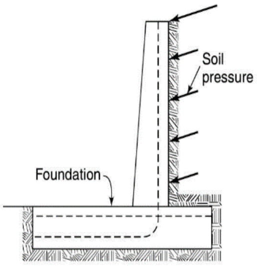


Figure 6.3: Cantilever retaining wall

(d) Tilt-up walls—are very slender walls that are cast in a horizontal position adjacent to the structure. They are then tilted into their intended vertical position and fastened to the foundation, to the roof or floor diaphragm, and to the adjacent panels .They are designed to resist vertical and lateral loads.

Figure 6.4: Compression panel in a bridge deck

<u>6.3</u> Design Response Spectra

for determining the design response spectra, we must define the soil type of site that our building will be constructed on.

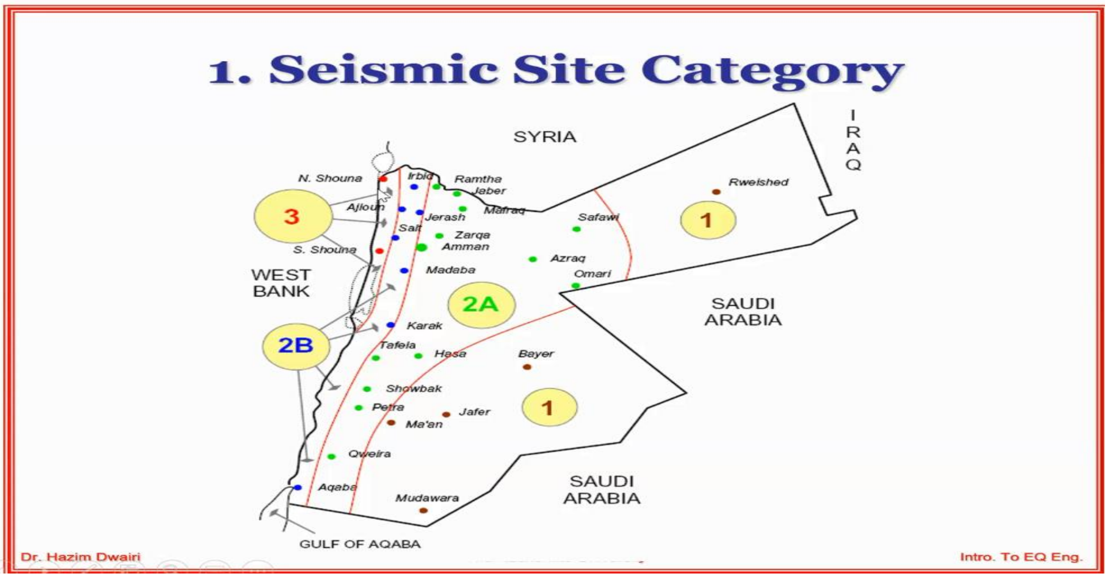
Table 6.1: Site Class Definitions from ASCE 7-02 and ASCE 7-05

Table 1 — Site Class Definitions from ASCE 7-02 and ASCE 7-05

Site Class	Site Profile Name	Soil Shear Wave Velocity, v_s(ft/sec)	Standard Penetration Resistance, $\bar{N}$ or Nch	Undrained Shear Strength, $\bar{S}_u$ (psf)
A	Hard rock	$\bar{v}_s > 5{,}000$	NA	NA
B	Rock	$2{,}500 < \bar{v}_s \leq 5{,}000$	NA	NA
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_s \leq 2{,}500$	> 50	> 2,000 psf
D	Stiff soil	$600 < \bar{v}_s \leq 1{,}200$	15 to 20	1,000 to 2,000 psf
E	Soft clay soil	$\bar{v}_s \leq 600$	<15	<1,000psf
		Any profile with more than 10 ft of soil having the following characteristics: • Plasticity index PI > 20 • Moisture content $w \geq 40\%$, and • Undrained shear strength S_u < 500 psf		
F	Soil requires site response analysis	Liquefiable soils, peat, high plasticity clay		

We will consider site **<u>class D (stiff soil).</u>**

Now we should know zone factor from map zone:



The most Dangerous Category in Jordan is zone 3

Zone	1	2A	2B	3	4
Z	0.075	0.15	0.2	0.3	0.4

So Z=0.3

Now we will determine Ca and Cv

Table 6.2: Seismic coefficient Ca

SOIL PROFILE TYPE	SEISMIC ZONE FACTOR, Z				
	$Z = 0.075$	$Z = 0.15$	$Z = 0.2$	$Z = 0.3$	$Z = 0.4$
S_A	0.06	0.12	0.16	0.24	$0.32N_a$
S_B	0.08	0.15	0.20	0.30	$0.40N_a$
S_C	0.09	0.18	0.24	0.33	$0.40N_a$
S_D	0.12	0.22	0.28	0.36	$0.44N_a$
S_E	0.19	0.30	0.34	0.36	$0.36N_a$
S_F	See Footnote 1				

Table 6.3: Seismic coefficient Cv

SOIL PROFILE TYPE	SEISMIC ZONE FACTOR, Z				
	$Z = 0.075$	$Z = 0.15$	$Z = 0.2$	$Z = 0.3$	$Z = 0.4$
S_A	0.06	0.12	0.16	0.24	$0.32N_v$
S_B	0.08	0.15	0.20	0.30	$0.40N_v$
S_C	0.13	0.25	0.32	0.45	$0.56N_v$
S_D	0.18	0.32	0.40	0.54	$0.64N_v$
S_E	0.26	0.50	0.64	0.84	$0.96N_v$
S_F	See Footnote 1				

From tables we found

Ca=0.36

Cv=0.54

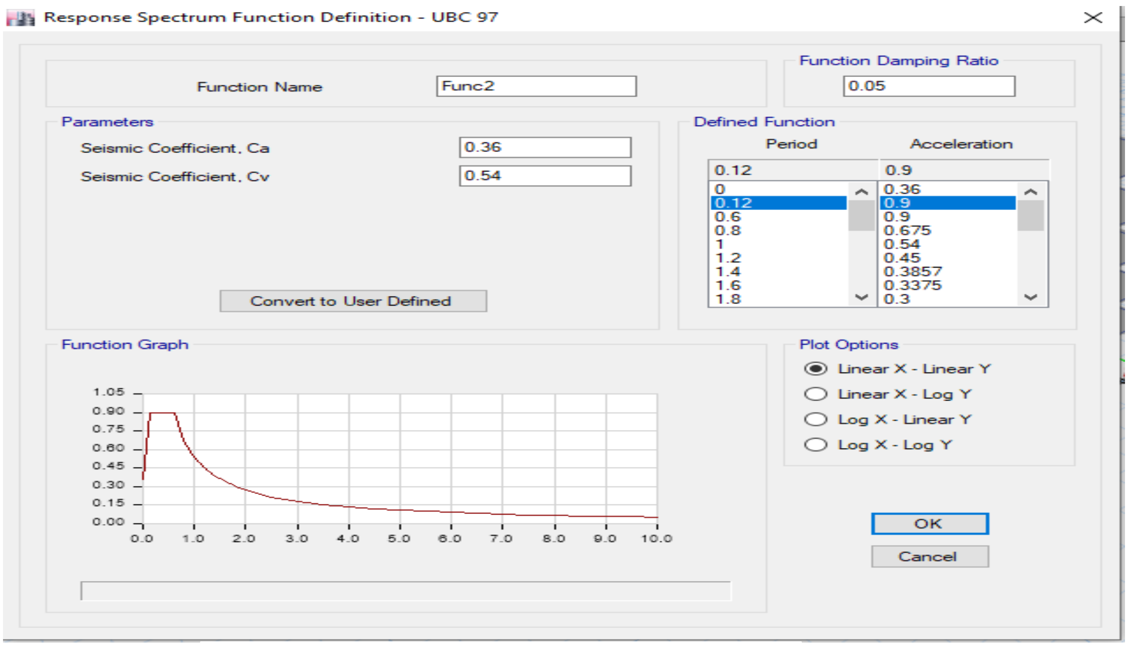


Figure 6.5: Response spectrum for our building

Ts =Cv/2.5Ca

=0.54/2.5*0.36 =0.6 sec

T0=0.2 Ts

=0.12 sec

Peak ground acceleration =Ca=0.36

6.4 Design

Shear wall Design in ETABS
First: Define

1) the diaphragms for each story

Diaphragms are a roof, floor or other system transferring lateral forces applied to a building to the vertical elements, such as the shear walls.

*There is two type of Diaphragms Rigid and Flexible,

We used rigid type.

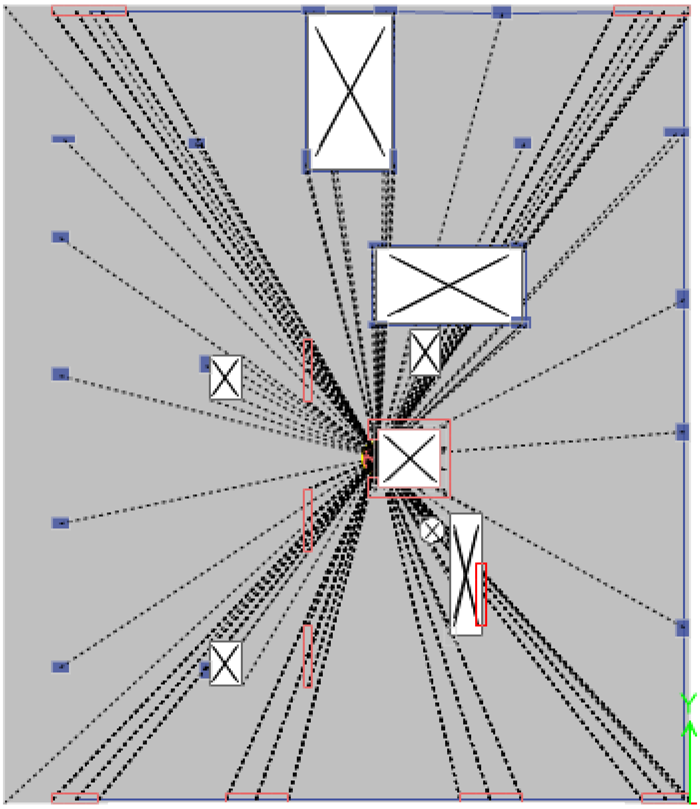


Figure 6.6: Rigid diaphragms

2) Mass Source: specified load pattern
for our building we multiply dead load by 1 because it is a residential building.

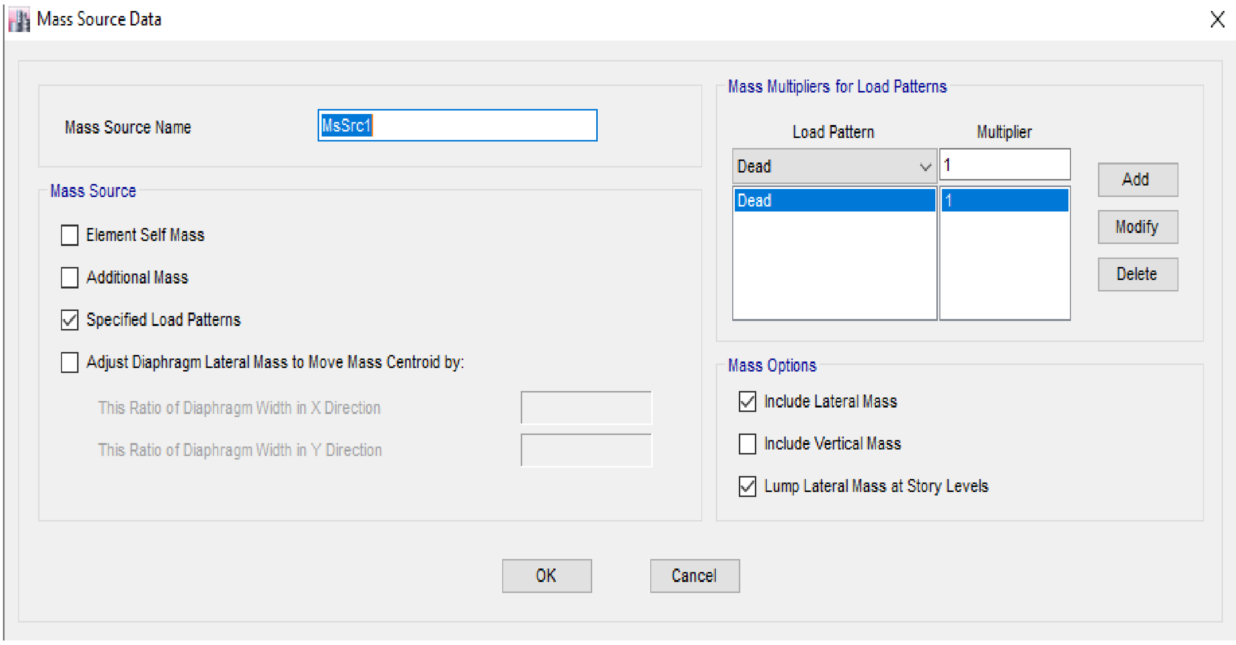


Figure 6.7: mass source

3) lateral Load Patterns

Seismic loads: EX1, EX2, EY1, EY2

EX1:

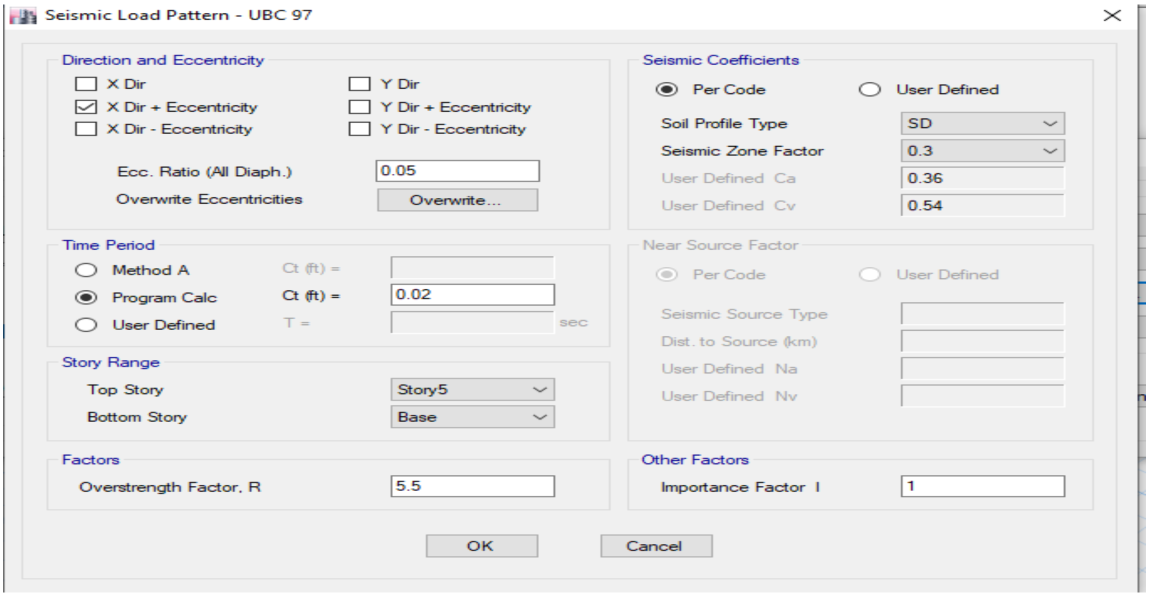


Figure 6.8: Seismic load For EX1

Wind Load:

Design wind speed in Jordan=120Km/h

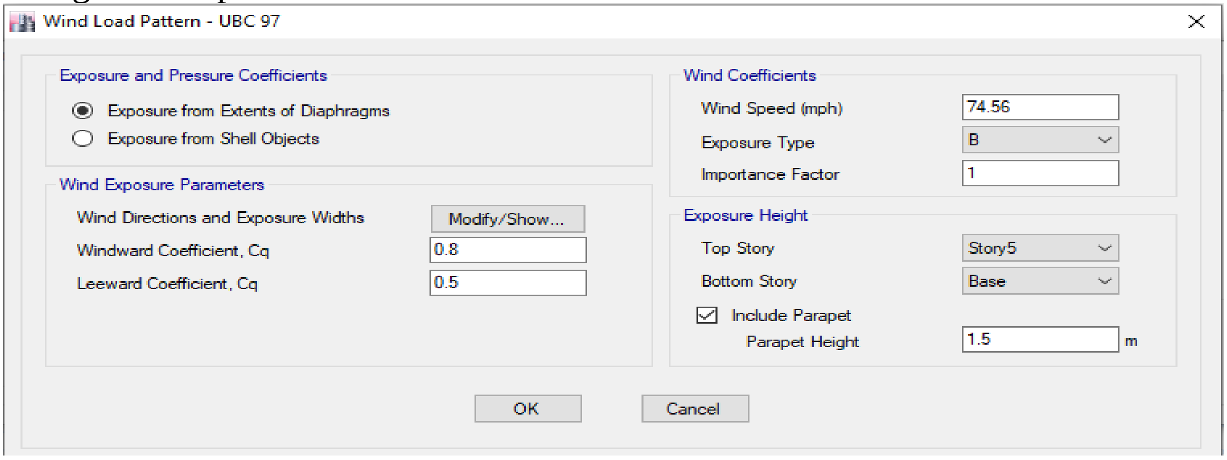


Figure 6.9: wind load

4) lateral Load Combinations

Load Combinations:

Based on section 1612 of UBC, structures are to resist the most critical effects from the following combinations of factored loads:

$1.4D + 1.7L$ (A-9)

$0.75(1.4D + 1.7L + 1.7W)$ (A-10)

$0.9D + 1.3W$ (A-11)

$1.32D + 1.1f_1 L + 1.1E$ (A-12)

$0.99D + 1.1E$ (A-13)

Where

f_1 = 1.0 for floors in public assembly, live loads in excess of 500 kg/m^2 and for garage live loads

f_1 = 0.5 for other live loads

$E = \rho E_h + E_v$ (A-14)

E_h = the earthquake load due to the base shear, V

E_v = the load effects resulting from the vertical component of the earthquake ground motion and is equal to the addition of $0.50 C_a I D$ to the dead load effects D

ρ = redundancy factor, to increase the effects of earthquake loads on structures with few lateral force resisting elements, given by

$$\rho = 2 - \frac{6.10}{r_{max}\sqrt{A_g}} \qquad \text{(A-16)}$$

Major seismic combinations:

U=1.518+0.55+-1.1Eh

1. 1.518DL+0.55LL+1.1EX1

2. 1.518DL+0.55LL+1.1EX2
3. 1.518DL+0.55LL+1.1EY1
4. 1.518DL+0.55LL+1.1EY2
5. 1.518DL+0.55LL-1.1EX1
6. 1.518DL+0.55LL-1.1EX2
7. 1.518DL+0.55LL-1.1EY1
8. 1.518DL+0.55LL-1.1EY2

Secondary seismic combinations:

U=0.792+-1.1Eh

9. 0.792DL+1.1EX1
10. 0.792DL+1.1EX2
11. 0.792DL+1.1EY1
12. 0.792DL+1.1EY2
13. 0.792DL-1.1EX1
14. 0.792DL-1.1EX2
15. 0.792DL-1.1EY1
16. 0.792DL-1.1EY2

Wind combinations:

17. 1.2DL+0.5LL+1.3Wind
18. 1.2DL+0.5LL-1.3Wind
19. 0.9DL+1.3Wind
20. 0.9DL-1.3Wind

Second: Assign Stiffness Modifiers

• Stiffness modifiers are the factors to increase or decrease some properties of the cross section like area, inertia, torsional constant etc. Generally, they are used to reduce stiffness of concrete sections to model cracked behavior of concrete. They are only applied to concrete members because it cracks under loading.

• Cracking reduces area of the cross section and reduces stiffness of the structure which effects the deflection and forces. So we need to assign stiffness modifier to use reduced stiffness which defines structural members as cracked sections. By considering cracked sections in analysis we can ensure safer/larger (than regular sections having 100% stiffness) structural sections to keep the structure safe in terms of serviceability under cracking condition.

Table 6.4: Stiffness factor

Member and condition		Moment of Inertia	Cross-sectional area
Columns		$0.70I_g$	$1.0A_g$
Walls	Uncracked	$0.70I_g$	
	Cracked	$0.35I_g$	
Beams		$0.35I_g$	
Flat plates and flat slabs		$0.25I_g$	

For shear Wall:

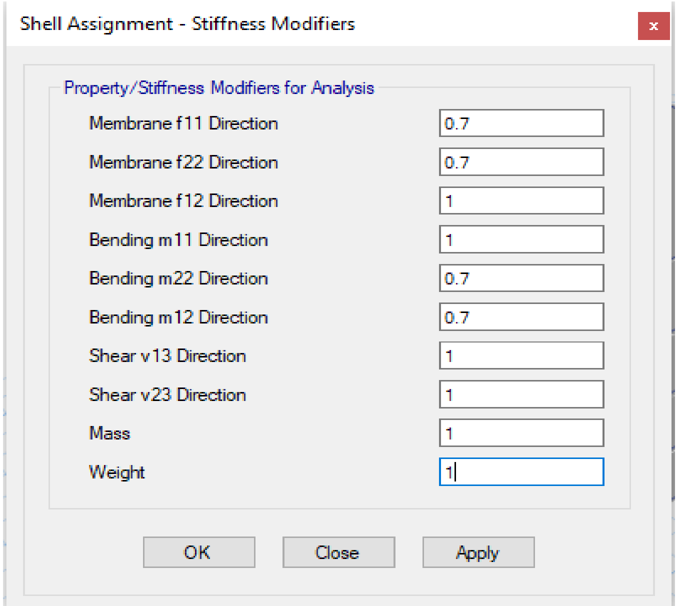
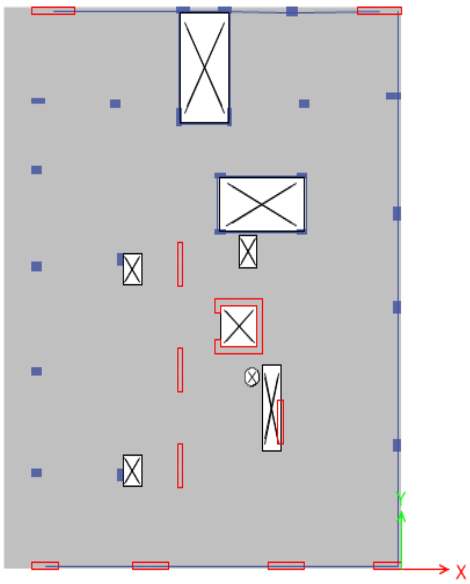
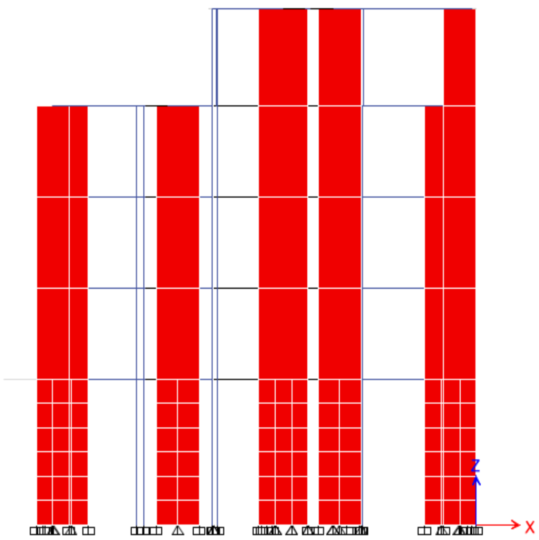


Figure 6.10: Stiffness modifiers for shear wall

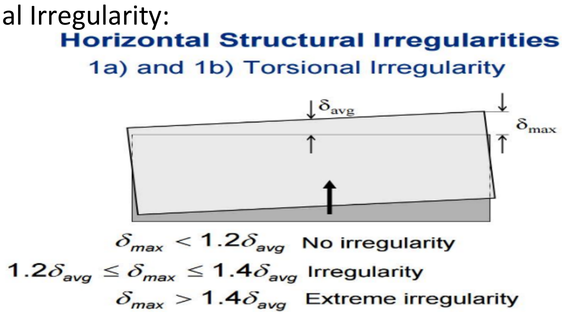
Third: Check for irregularities





Horizontal Irregularity:

Torsional Irregularity:

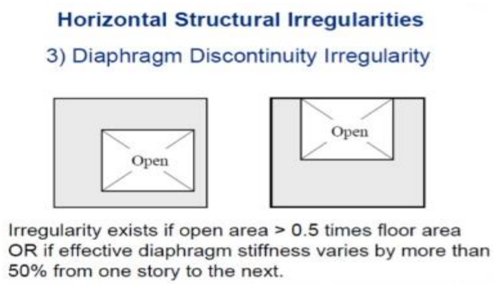


- After we checked displacement in ETABS for all stories we found the ratio was less than 1.2 Then there is Torsional Irregularity doesn't exist.

Corner irregularity:

- The building is a rectangular plane shaped building so the corner irregularity doesn't exist.

Diaphragm Irregularity Discontinuity:

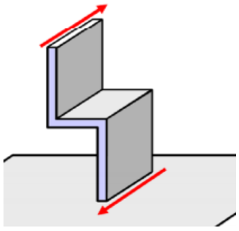


- Total Area=520m^2

Opening Area= 45m^2<0.5 Total Area

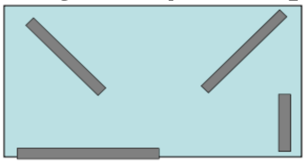
so, Diaphragm Irregularity Discontinuity doesn't exist.

Out of plane offsets



Out of plane offsets doesn't exist.

Non-parallel system irregularity



- Non-parallel system irregularity doesn't exist.

Vertical Irregularity:

- Shear walls and columns are continuous from bottom of the building to the top, so stiffness irregularity doesn't exist.

- The masses are equally distributed along the stories so the vertical mass irregularity of the stories doesn't exist.

Additional check

Overturning:
Overturning moment should be less than resisting moment.

Overturning moment from Etabs:

Base Reactions

1 of 4 Reload Apply — Base Reactions

Load Case/Combo	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
EX1	-3621.7867	0	0	59.8422	-9563.2415	43873.8029
EX2	-3621.7867	0	0	77.0073	-9548.5884	41982.609
EY1	0	-3595.8414	0	12124.2079	-58.6181	34502.8466
EY2	0	-3595.8414	0	12110.6008	-68.5944	35979.9099

Figure 6.11: Base reaction

Resisting Moment=258460 KN-m > all overturning moment OK

Finally: Design shear Wall

Wall#4 in story 2

	Item	Value
01	Design this Pier?	Yes
02	LL Reduction Factor	0.744081
03	Design is Seismic?	Yes
04	Pier Section Type	Uniform Reinforcing
05	End/Corner Bar Name	16
06	Edge Bar Name	16
07	Edge Bar Spacing	150
08	Clear Cover	40
09	Material	Concrete 28MPA
10	Check/Design Reinforcing	Check
11	Check Compression Block Depth for BZ?	No

Figure 6.12: Detail (1) of wall#4 in story2

ETABS Shear Wall Design

ACI 318-14 Pier Design

Pier Details

Story ID	Pier ID	Centroid X (mm)	Centroid Y (mm)	Length (mm)	Thickness (mm)	LLRF
Story2	P4	-13846	150	2000	300	0.744

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
30161.56	28	1	460	460

Design Code Parameters

Φ_T	Φ_C	Φ_V	Φ_V (Seismic)	IP_{MAX}	IP_{MIN}	P_{MAX}
0.9	0.65	0.75	0.6	0.04	0.0025	0.8

Pier Leg Location, Length and Thickness

Station Location	ID	Left X_1 mm	Left Y_1 mm	Right X_2 mm	Right Y_2 mm	Length mm	Thickness mm
Top	Leg 1	-14846	150	-12846	150	2000	300
Bottom	Leg 1	-14846	150	-12846	150	2000	300

Flexural Design for P, M_3 and M_2

Station	D/C	Flexural	P_u kN	M_{u2} kN-m	M_{u3} kN-m
Top	0.255	ULT1.2+1.6	2061.8657	58.4473	132.2961
Bottom	0.934	SX2N	434.578	-8.4667	-2109.3119

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	EX2N	838.6321	-525.6593	418.9003	355.8964	728.2133
Bottom	Leg 1	750	EX2N	934.2661	-2108.5485	425.3073	251.4773	623.7942

Boundary Element Check (ACI 21.9.6.3, 21.9.6.4)

Station Location	ID	Edge Length (mm)	Governing Combo	P_u kN	M_u kN-m	Stress Comp MPa	Stress Limit MPa	C Depth mm	C Limit mm
Top–Left	Leg 1	Not Required	EX1N	843.0901	-498.6124	3.9	5.6	389.8	395.6
Top–Right	Leg 1	199.4	EX1N	909.9719	645.6532	4.74	5.6	398.9	376.3
Bottom–Left	Leg 1	202.8	EX1N	938.7241	-2005.3024	11.59	5.6	402.8	395.6
Botttom–Right	Leg 1	211.8	EX1N	1005.6059	2114.8608	12.25	5.6	411.8	376.3

Figure 6.12: Detail (2) of wall#4 in story2

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	1500	0.0025	0.0101	SWIND-N	492.9745	16.5987	-13.3184	600000
Bottom	6115	0.0102	0.0101	SX2N	434.578	-8.4667	-2109.3119	600000

Figure 6.13: Detail (3) of wall#4 in story2

(D/C) =0.934 <1 OK

Reinforcement Ratio= 0.0101>0.0025 OK

So,

VERTICAL STEEL For Wall# 4 in story 2 is Ø16@15cm c/c.

HORIZONTAL STEEL is Ø12@15cm c/c.

6.5 Final results

Table 6.5: Final results

SCHEDULE OF SHEAR WALL					
TYPE	LOCATION	UP TO 1ST FLOOR	2ND & 3RD FLOOR	4th & TOP ROOF FLOOR	REMARK
SW1	THICKNESS	30CM	30CM	30CM	
	HORIZONTAL STEEL	Ø10@15cm c/c	Ø10@15cm c/c	Ø10@20cm c/c	
	VERTICAL STEEL	Ø20@15cm c/c	Ø16@15cm c/c	Ø14@15cm c/c	
SW2	THICKNESS	30CM	30CM	30CM	
	HORIZONTAL STEEL	Ø12@15cm c/c	Ø12@15cm c/c	Ø12@20cm c/c	
	VERTICAL STEEL	Ø20@15cm c/c	Ø16@15cm c/c	Ø14@15cm c/c	
SW3	THICKNESS	30CM	30CM	30CM	
	HORIZONTAL STEEL	Ø12@15cm c/c	Ø12@15cm c/c	Ø12@20cm c/c	
	VERTICAL STEEL	Ø20@15cm c/c	Ø16@15cm c/c	Ø16@15cm c/c	
SW4	THICKNESS	30CM	30CM	30CM	
	HORIZONTAL STEEL	Ø12@15cm c/c	Ø12@15cm c/c	Ø12@20cm c/c	
	VERTICAL STEEL	Ø20@15cm c/c	Ø16@15cm c/c	Ø16@15cm c/c	
SW5	THICKNESS	30CM	30CM	30CM	
	HORIZONTAL STEEL	Ø12@15cm c/c	Ø12@15cm c/c	Ø12@20cm c/c	
	VERTICAL STEEL	Ø20@15cm c/c	Ø18@15cm c/c	Ø14@15cm c/c	
SW6	THICKNESS	30CM	30CM	30CM	
	HORIZONTAL STEEL	Ø12@15cm c/c	Ø12@15cm c/c	Ø12@20cm c/c	
	VERTICAL STEEL	Ø16@15cm c/c	Ø16@15cm c/c	Ø14@15cm c/c	
SW7	THICKNESS	30CM	30CM	30CM	
	HORIZONTAL STEEL	Ø12@15cm c/c	Ø12@15cm c/c	Ø12@20cm c/c	
	VERTICAL STEEL	Ø20@15cm c/c	Ø18@15cm c/c	Ø14@15cm c/c	

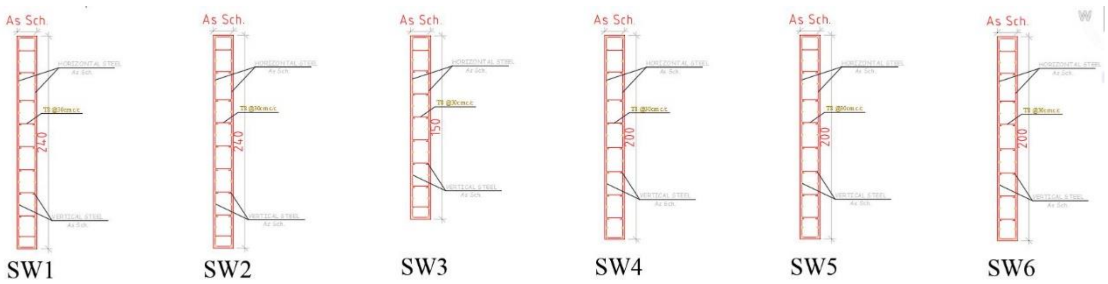


Figure 6.14: cross section of shear wall reinforcement

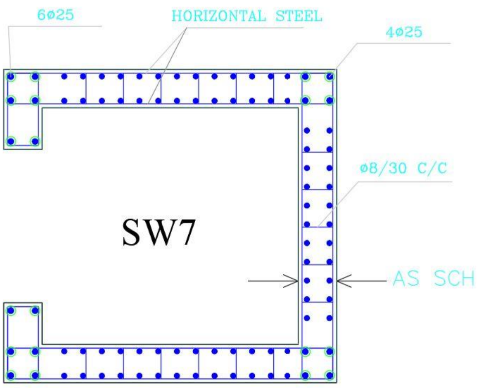


Figure 6.15: cross section of lift reinforcement

Chapter 7: Foundation

7.1Foundation Types

- In this project we use Raft foundation for many reasons; is often used when the soil is weak (Bearing capacity= 1700 Kn/m^2) and when we have a high building.

NOW: What is Raft foundation?

- Raft foundations (sometimes referred to as raft footing or mat foundations) are formed by reinforced concrete slab of uniform thickness that cover a wide area, often the entire footprint of a building. They spread the load imposed by several columns or walls over the area of foundation and can be considered to ‘float’ on the ground as a raft floats on water, The Fig show simple vision about raft foundation.

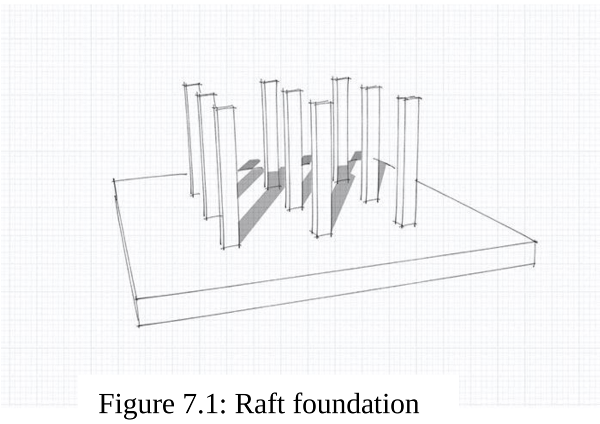


Figure 7.1: Raft foundation

- The reinforcement of raft footing is two mesh; top and bottom mesh (positive and negative moment), the picture shows the cross section of raft foundation.

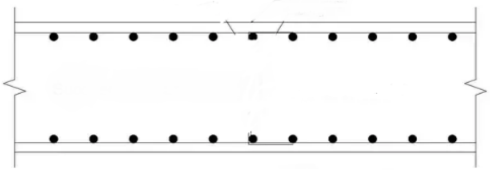


Figure 7.2: Typical section for raft foundation

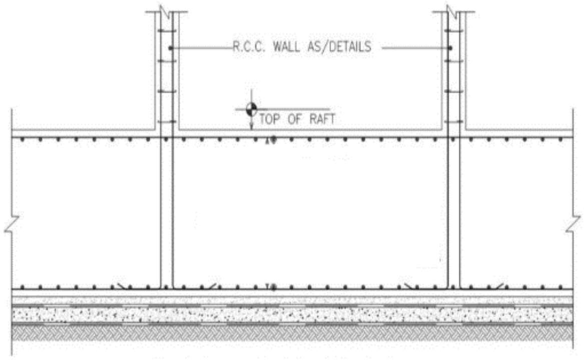


Figure 7.3: Typical Raft Section

- Under columns there is extra reinforcement to meet the high value of positive moment.
  (Red color in the shown Fig).

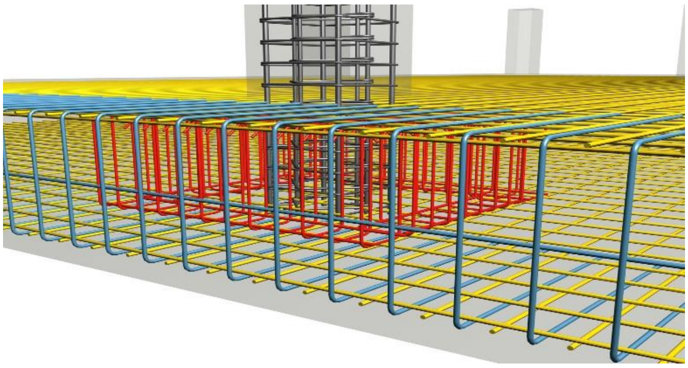


Figure 7.4: reinforcement in raft foundation under columns

- At the end of foundation (edge) the steel should be bent as the Fig shown.

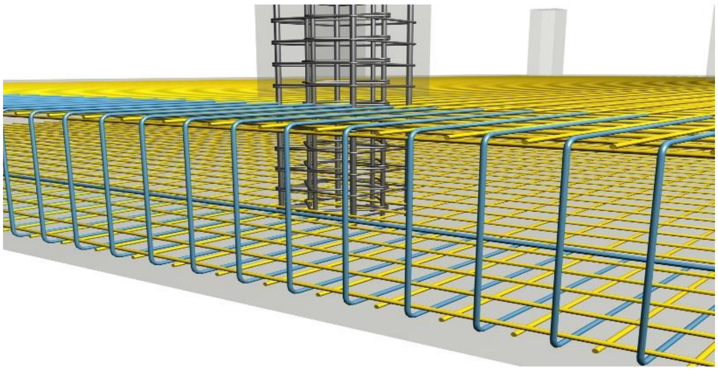


Figure 7.5: bent the steel at the end of foundation

- Always the main Reinforcement in raft foundation on the top face of footing depending on the profile of moment.

7.2 Loading and Size Selection

- The Safe program will be used in the design of Raft foundation.
- The base floor of whole building model exported from Etabs to Safe with the reaction centers and all loads (load cases).
- The combinations load of lateral loads (Earth quick and wind load) entered manually in safe.
- The f_y = 420 Mpa, $f_c' = 30\ Mpa$
- The thickness of footing is assumed depending on the this formula:

  Thick = # of store ∗ 10 cm ± 10 cm

  = 5 *10 ± 10 ⇒ (40 cm – 50 cm)

  Foundation thick = 50 cm ⇒ more conservative

- On Safe program the spring support type is Elastic support.

- Depending in our searching on suitable value of bearing capacity, we found a site investigation paper with value of bearing capacity = 1.7 Kg/cm^2

Site Investigation
For
حامد حسن محمد ابو ركبه
مادبا / الاردن

تقرير رقم S2017/C20/219-01
عمان 2017/12/29

الاستنتاجات والتوصيات:
1. نوع الأساسات وعمق التأسيس:
تم تحميل المنشأ القائم في الموقع على اساسات سطحية على شكل قواعد للأعمدة بالأبعاد التالية (2م * 2.5م * 0.5م عمق)، وضعت الأساسات ضمن حفريات مفتوحة داخل التربة الطينية السلتية على عمق حوالي (2) متر من منسوب التسوية.

يجب أن لا يكون هذا العمق أقل من (2) متر تحت المنسوب النهائي للأرض المحيطة بالمبنى.

2. قوة التحمل المسموح بها:
قوة تحمل التربة للأساسات الموضوعة على الأعماق المبينة أعلاه، مساوية (1.7) كغم/سم^2.

- ... كافية عن الفوالق الصخرية ولا يوجد لها اي تأثير على الموقع

Figure 7.6: bearing capacity of soil

- Soil subgrade property data:

Value of Subgrade modulus (K)

$$K = \frac{Bearing\ capacity\ (\frac{Kn}{m^2})}{Impressible\ deflction\ (m)}$$

B.C = 1700 KN/m^2
The permissible Def = 2.5 cm

$$\Rightarrow K = \frac{1700}{0.025} = 68000\ Kn/m^3$$

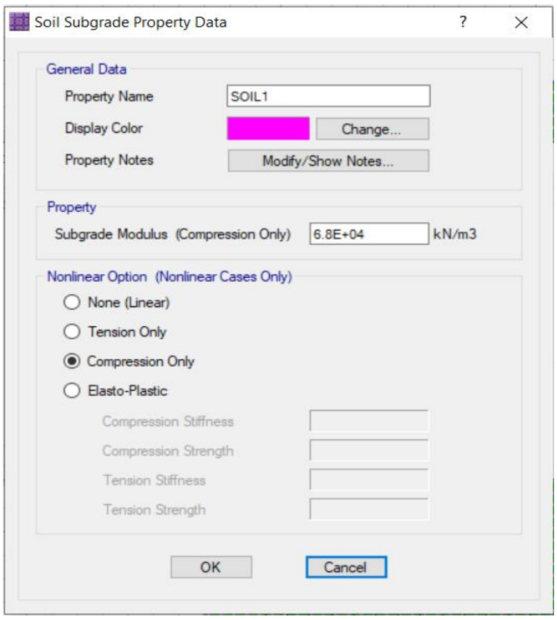


Figure 7.7: soil subgrade property date

- The combination load here added manually; and we add many types of combination load:

1. Working combination

   Working = 1.2 DL + 1.6 LL

2. Earth quick Combination load

☒ Major seismic Comb

EX1P=1.518DL+0.55LL+1.1EX1	EX1N=1.518DL+0.55LL-1.1EX1
EX2P=1.518DL+0.55LL+1.1EX2	EX2N=1.518DL+0.55LL-1.1EX2
EY1P= 1.518DL+0.55LL+1.1EY1	EY1N=1.518DL+0.55LL-1.1EY1
EY2P=1.518DL+0.55LL+1.1EY2	EY2N=1.518DL+0.55LL-1.1EY

☒ Secondary seismic combinations:

SX1P= 0.792DL+1.1EX1	SX1N= 0.792DL-1.1EX1
SX2P=0.792DL+1.1EX2	SX2N=0.792DL-1.1EX2
SY1P=0.792DL+1.1EY1	SY1N=0.792DL-1.1EY1
SY2P=0.792DL+1.1EY2	SY2N=0.792DL-1.1EY2

3. Wind combinations:

WIND-P=1.2DL+0.5LL+1.3Wind	S WIND-P=0.9DL+1.3Wind
WIND-N=1.2DL+0.5LL-1.3Wind	S WIND-N=0.9DL-1.3Wind

IMPORTANCE:

- We have now large number of combination load, but we need to do our design one Comb (the large one), but here with this large number of Comb it is impossible. So, we need combination load give me the highest term from every comb; for that we define new comb (Max-Max).

- All the previous combination load used for lateral load formed the terms in this Max-Max combination load.

- Every term in this combination given Factor = 1.

- As the following Fig shown, all the previous combination load (lateral load combination) multiply with Factor = 1.

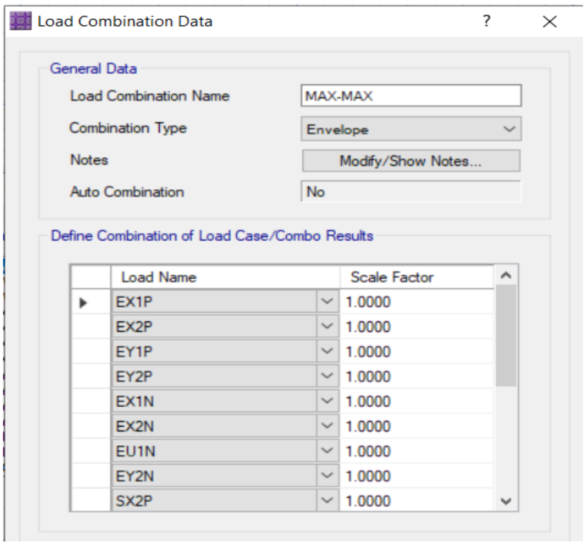


Figure 7.8: Factor of Max-Max combination

So:

Max-Max=1 Workimg+1 EX1P +1 EX2P +1 EY1P …..+1 S WIND-N

NOTE: All our design mainly depends on this load combination.

7.3 Design Procedure

❖ *Design of footing:*

- Related to choose the Raft foundation system, we need to check the deflection, Punching Shear and stress under footing then we will find the suitable design for the footing.

1. Deflection

- Deflection checked on Max-Max combination load on Min case.
- The permissible deflection that allowed to happen in all type of footing is 2.5 cm.
- Using Safe program, the actual deflection in footing depending on the load transfer from all stores to the foundation shown in this picture.

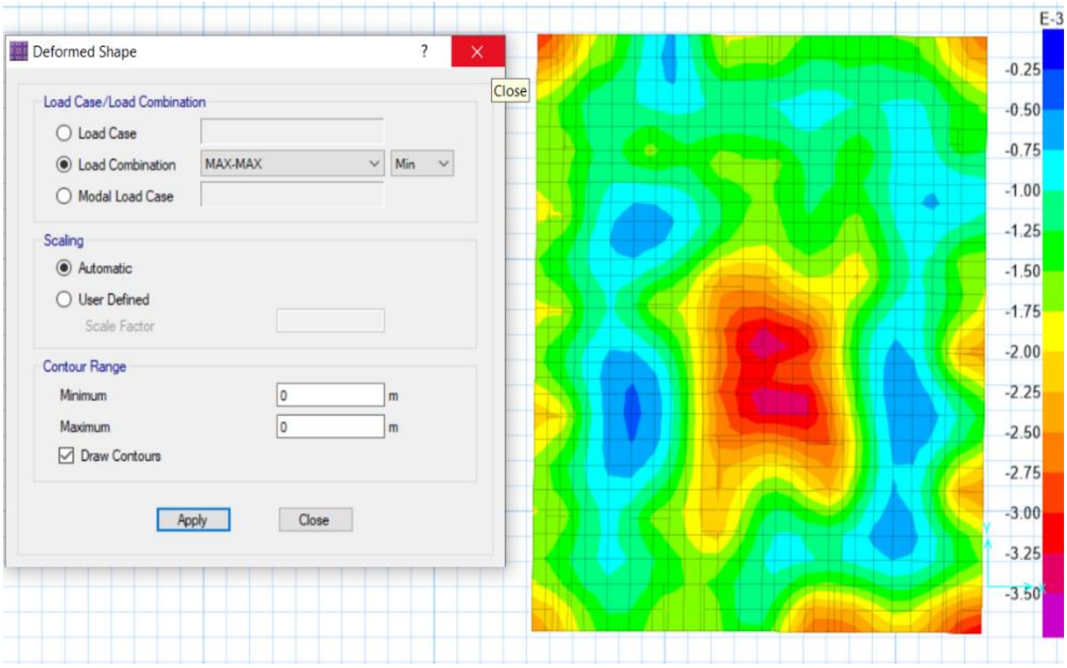


Figure 7.9: actual deflection in footing

- From picture as we see the max value of actual deflection = 0.35 cm

0.35cm < 2.5 cm ⇒ OK

2. Punching shear

- The results of Punching Shear are shown in the figure below, and we notice all the columns is OK so No Need of shear reinforcement.
- As noticed:

All factor < 1

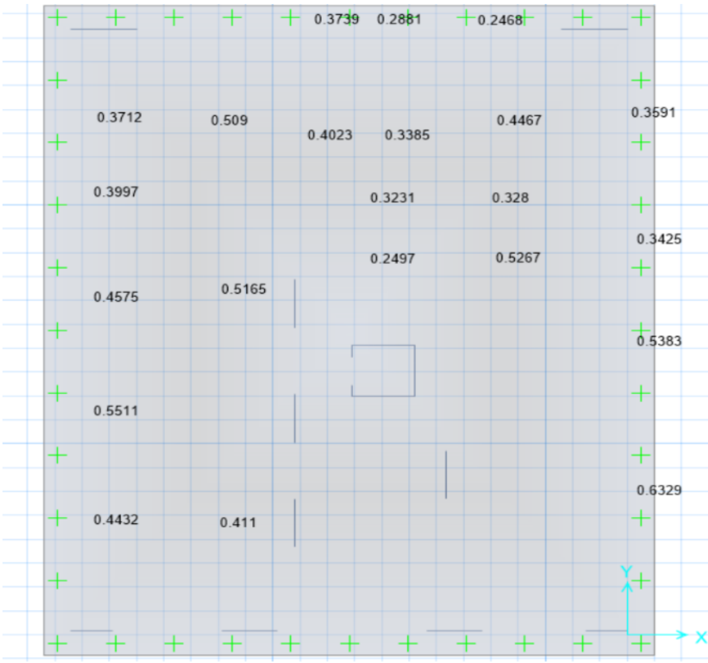


Figure 7.10: Punching Shear

3. Soil pressure (stress)

- All value of stress under the footing must be less than the value of bearing capacity (1700Kn/m^2)

Stress value < B.C (1700)

- The value of stress and soil pressure under footing In the attached Excel sheet file

All stress value < 1700 ⇒ OK

SOIL PRESSURE.xlsx

- If the stress value more than the value of bearing capacity; in this case you should do one of:
  - ⇒ Increasing the foundation thickness.
  - ⇒ Replacing the soil with another one that have grater B.C value.
  - ⇒ Using Pile foundation.

4. Steel Reinforcement

- The value of moment:

A) Positive moment:

X-direction:

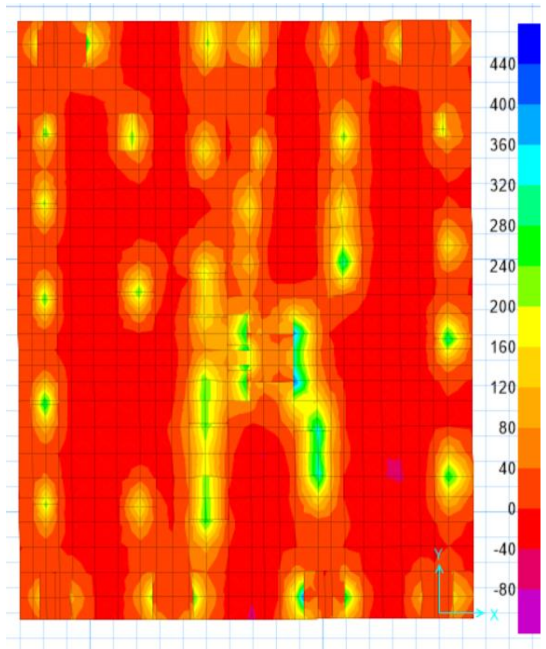


Figure 7.11: Value of moment (positive) in X axis

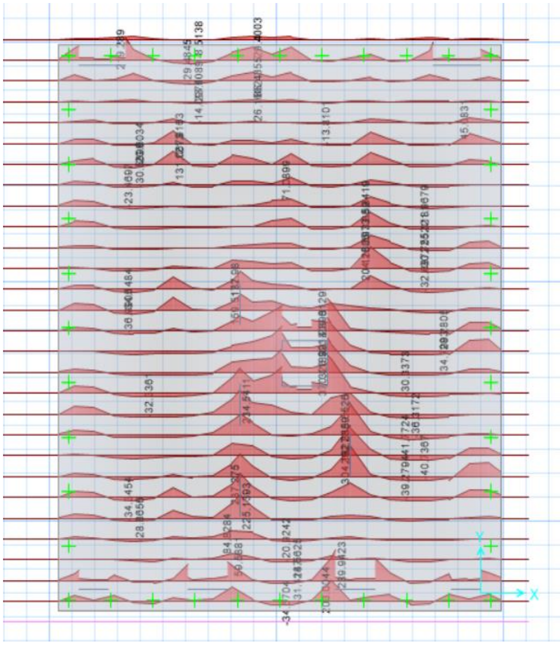


Figure 7.12: positive moment diagram in X axis

Y-direction:

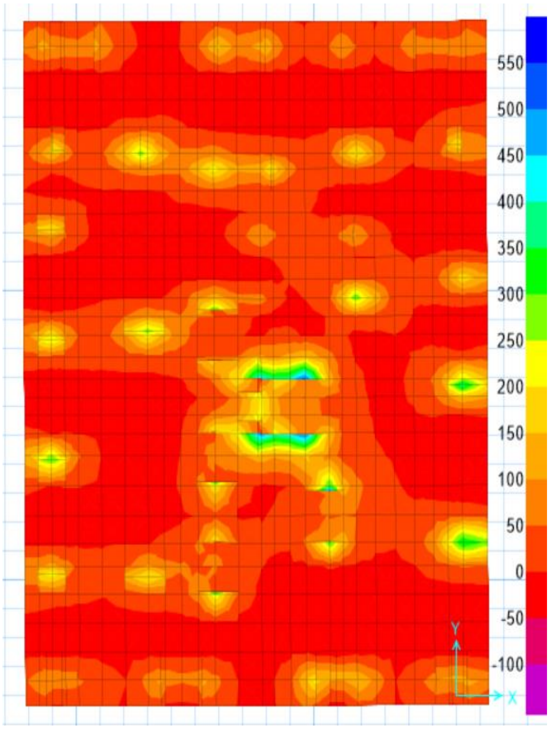


Figure 7.13: Value of moment (positive) in Y axis

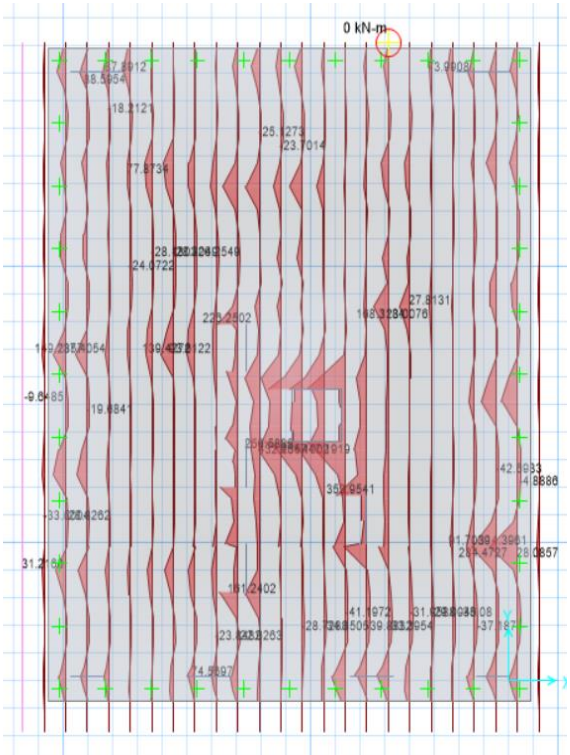


Figure 7.14: positive moment diagram in Y axis

B) Negative Moment:

X-direction:

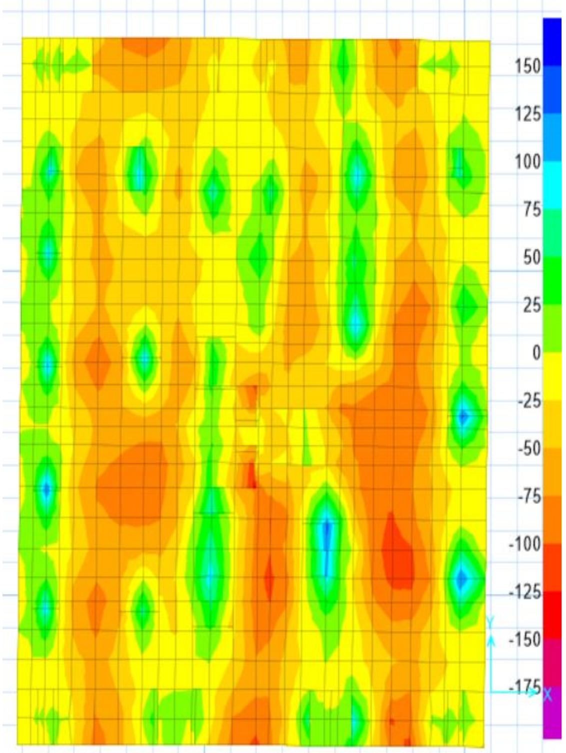


Figure 7.15: Value of moment (Negative) in X axis

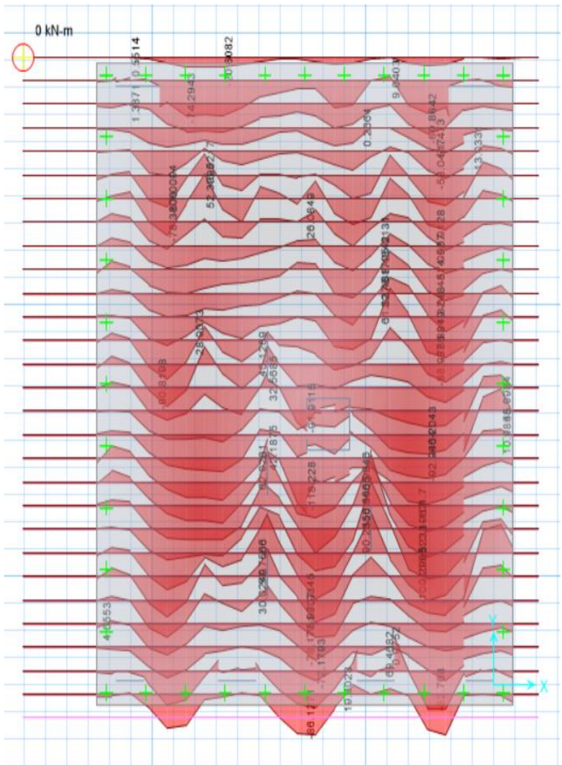


Figure 7.16: Negative moment diagram in X axis

Y-direction:

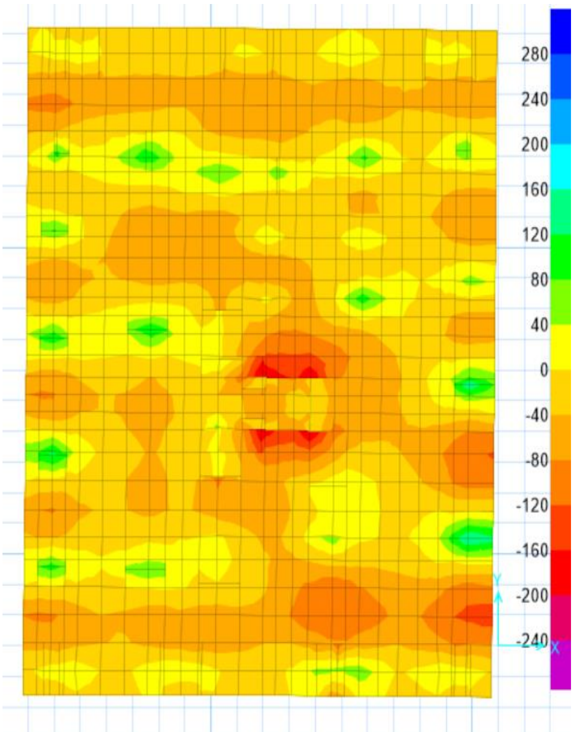


Figure 7.17: Value of moment (Negative) in Y axis

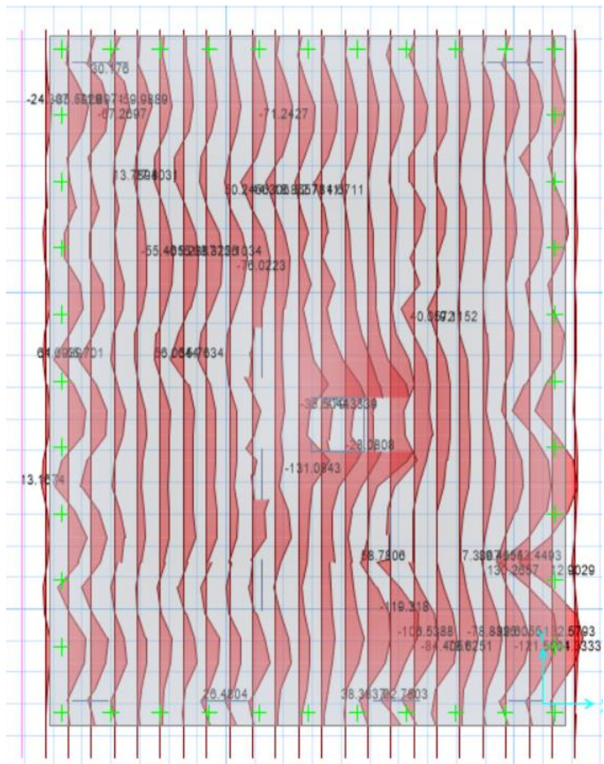


Figure 7.18: Negative moment diagram in Y axis

⇒Needed area of steel to resist the moment.

☒ Design of negative moment (top reinforcement)

- The cover of foundation is 7cm so the effective depth=43cm

- Start from Min area of steel:

$$As_{min} = 0.0018bh$$

$$= 0.0018*1000*500$$
$$= 900\ mm^2$$

✓ Try 5 Ø 16:

$As_{16} = 199$

So, Number needed steel bar = $\frac{900}{199} = 4.5$

So, use 5#16 ⇒ A= 995 mm^2

As = $\frac{M_u}{0.9*Fy*J_d}$ ⇒ 0.995=$\frac{M_U}{0.9*420*0.9*0.43}$

$M_u = 145\ Kn.m$

X-direction:

From the picture 5Ø16 is enough

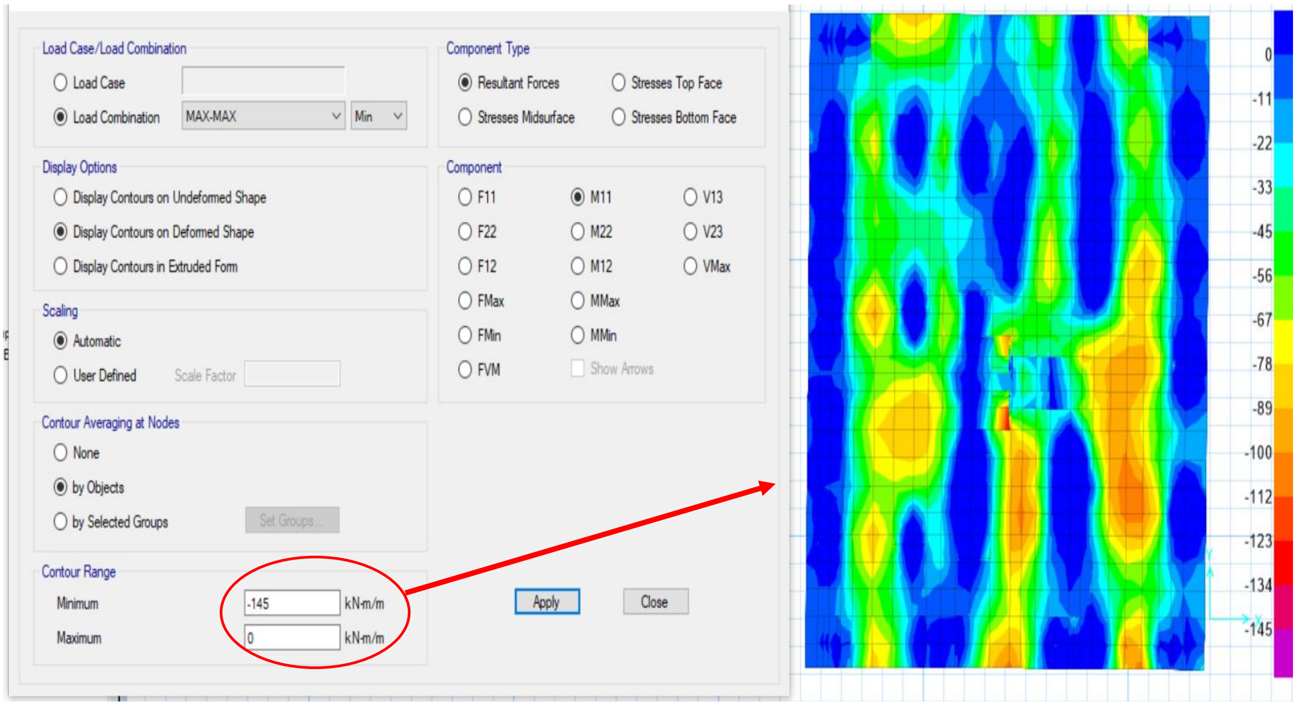


Figure 7.19: reinforcement Steel in X axis at the Negative moment

Y-direction

From the picture 5Ø16 enough

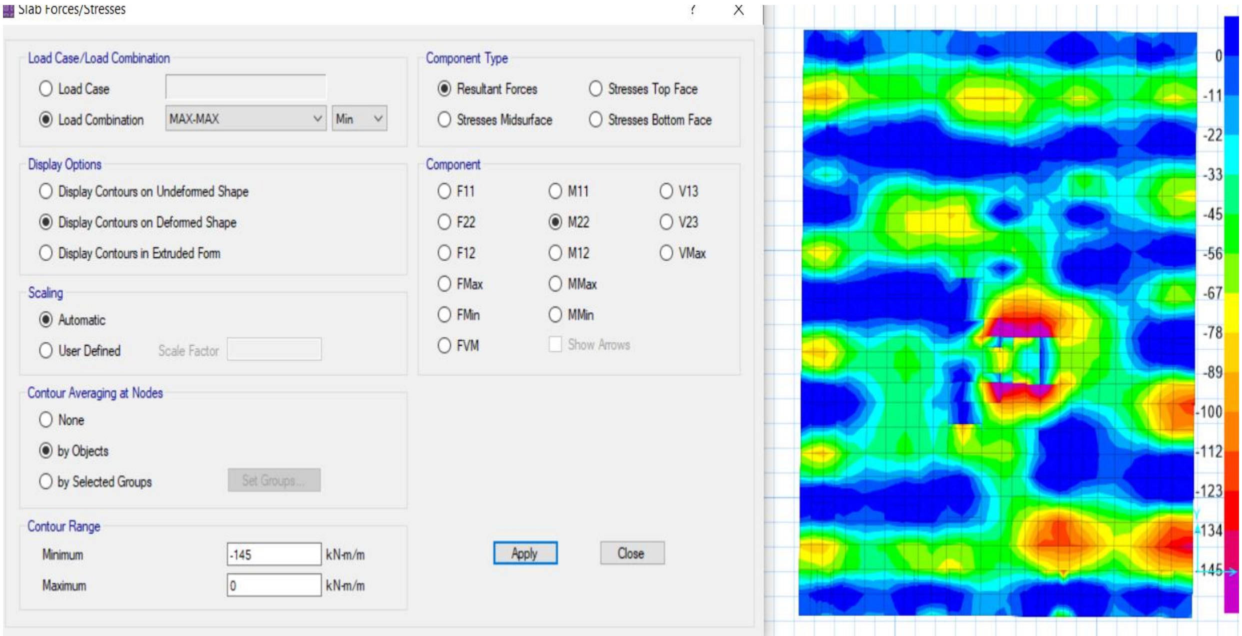


Figure 7.20: reinforcement Steel in Y axis at the Negative moment

⇒ So, the top reinforcement is:

5Ø16 ⇒ Ø16 at 200 mm

In X & Y direction (like a mesh).

☒ Design of positive moment (Bottom reinforcement)

➢ Let’s start with 5 Ø16 as the reinforcement of top

✓ Try 5 Ø 16:

$As_{16} = 199$

$\text{As} = \frac{M_u}{0.9 * Fy * J_d} \Rightarrow 0.995 = \frac{M_U}{0.9 * 420 * 0.9 * 0.43}$

$M_u = 145\ Kn.m$

X-direction:

From the picture 5Ø16 is enough

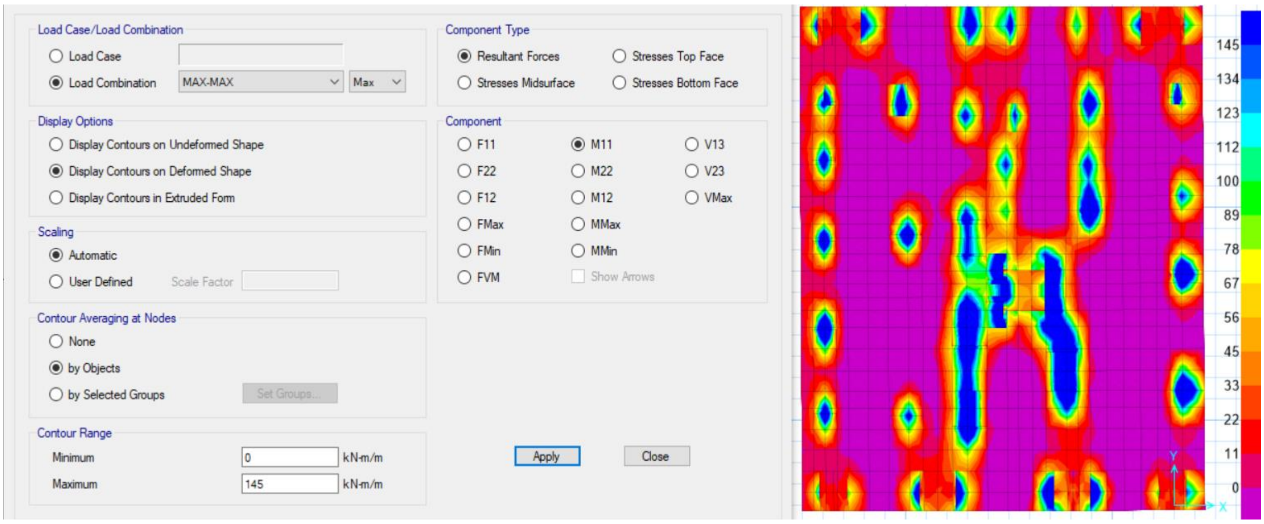


Figure 7.21: reinforcement Steel in X axis at the Positive moment

Y-direction

From the picture 5Ø16 enough

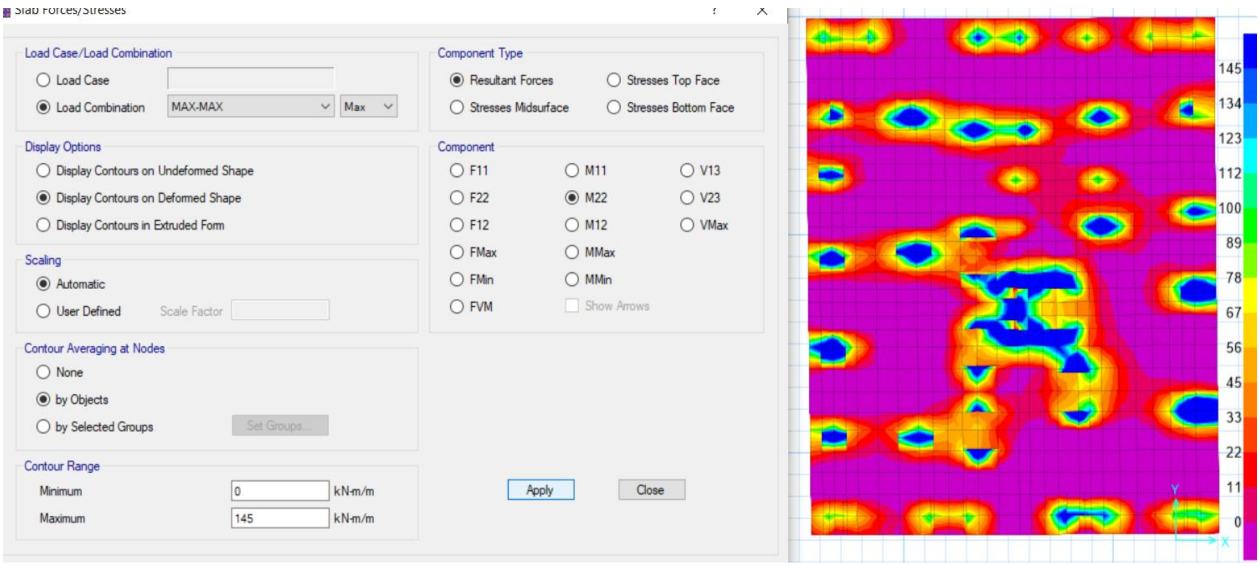


Figure 7.22: reinforcement Steel in Y axis at the Positive moment

⇒ So, the bottom reinforcement is:

5Ø16 ⇒ Ø16 at 200 mm

In X & Y direction (like a mesh).

☒ Design of positive moment (Under columns)

- There is a high value of positive moment under columns so we need to increase the number of steel bars or use another one with a larger diameter.
- After many iteration lets try 5 Ø22

A_s

✓ Try 5 Ø 22:

$As_{22} = 387$

So, use 5Ø 22 $\Rightarrow$ $A_s = 1935\, mm^2$

As = $\frac{M_u}{0.9*Fy*J_d} \Rightarrow 1.935 = \frac{M_U}{0.9*420*0.9*0.43}$

$M_u = 280\ Kn.m$

X-direction:

From the picture 5Ø22 is enough

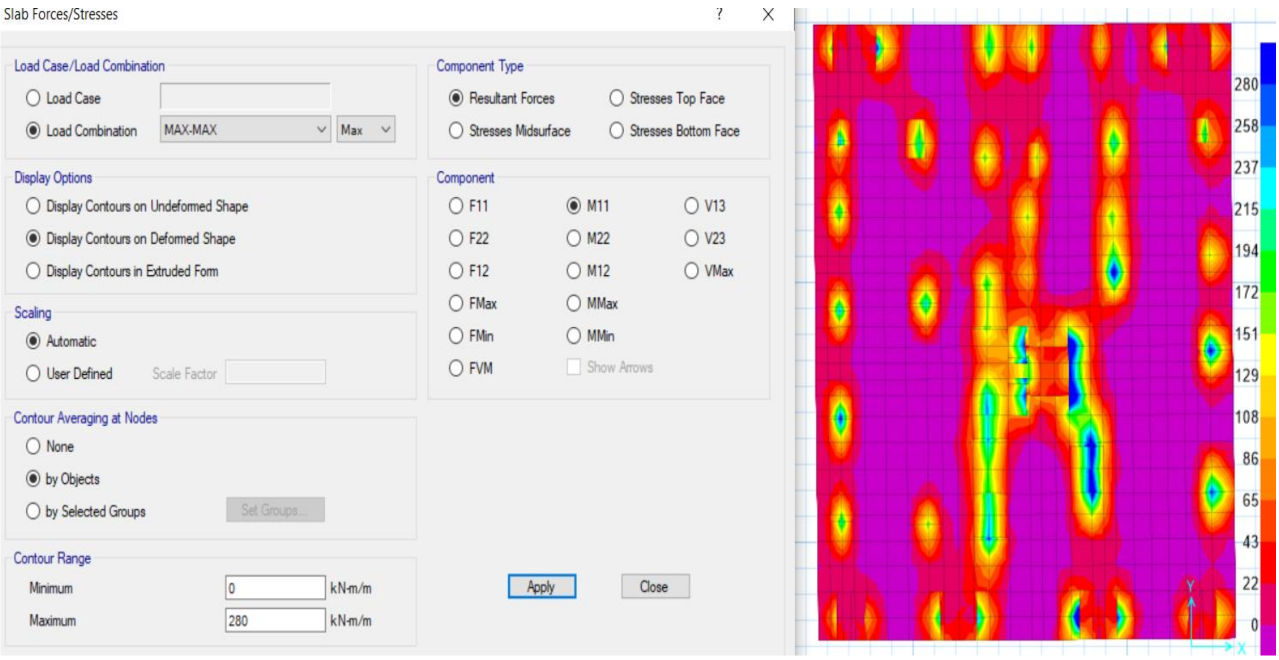


Figure 7.23: reinforcement Steel in X axis at the Positive moment (Under columns)

Y-direction

From the picture 5Ø22 enough

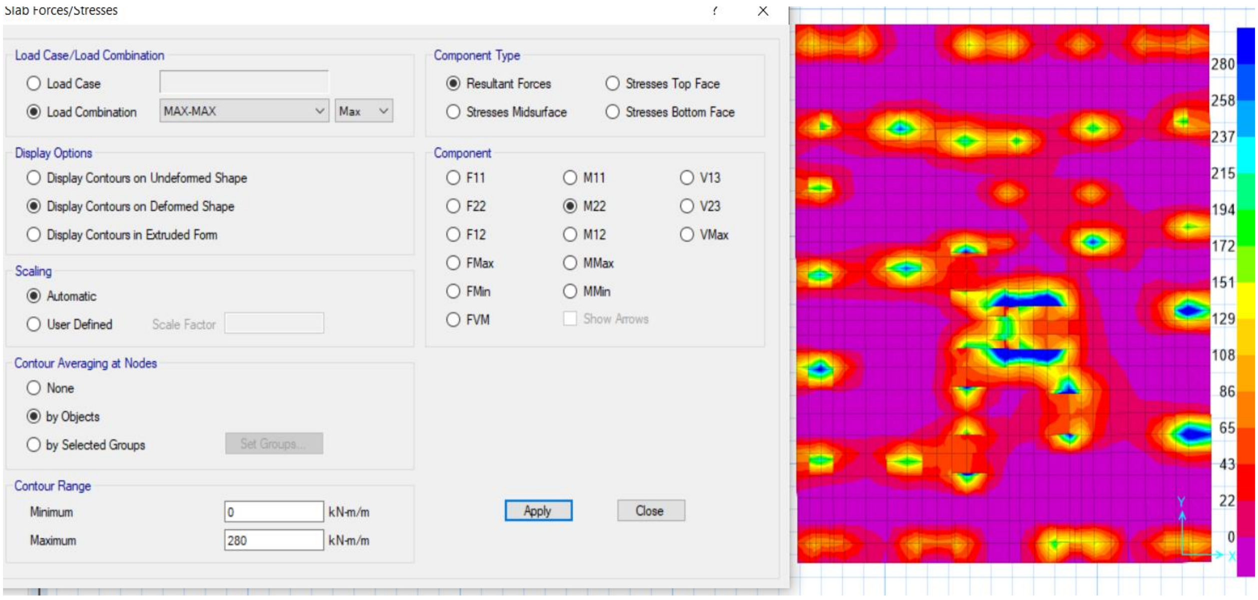


Figure 7.24: reinforcement Steel in Y axis at the Positive moment (Under columns)

$\Rightarrow$ So, the reinforcement under columns is:

5Ø22 $\Rightarrow$ Ø22 at 200 mm

In X & Y direction (like a mesh).

⇒Check S_{max}

S_{max} = smaller of $\begin{cases} 2\,h = 2 * 500 = 1000\,mm \\ \quad 500\,mm \quad \textcolor{red}{Control} \end{cases}$

actual spacing for 5 bar = 20 cm < 50 cm (OK)

⇒Check S_{min}

$$S_{min} = \max of \begin{cases} Bar\ diameter & = 16\,mm\,, 22\,mm \\ 25\,mm & = 25mm \quad \textcolor{red}{Control} \\ diameter\ of\ vibrat\ or & (unknown) \\ 1.33\max C.A\ size & (unknown) \end{cases}$$

Actual spacing for 5 bars = 200mm > 25mm (OK)

⇒Check As_{max}

$$As_{max} = 0.319\beta \frac{f'c}{Fy}\, bd = 5317\, mm^2$$

For 5 Φ 16

A_s= 565 mm^2 <5317mm^2 (OK)

For 5 Φ 22

A_s = 1935 mm^2 <5317mm^2 (OK)

➢ Therefore, and because all the checks are now OK and as we said previously that there is a bended

Cover = 7cm (top & bottom)

Length of bend:

50 – 2*7 = 36 cm

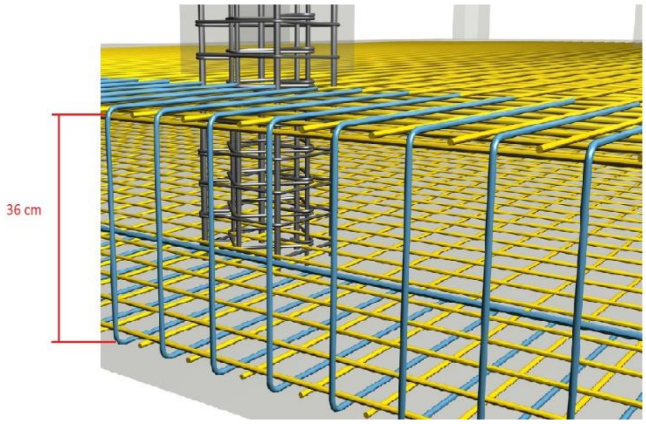


Figure 7.25: Length of bend steel in the foundation

☒ Summary of reinforcement (Positive & negative)

Table 7.1: Summary of reinforcement (Positive & negative)

	Design	X-direction		Y-direction	
		Positive steel	Negative steel	Positive steel	Negative steel
Steel	Foundation	5 Φ 16	5 Φ 16	5 Φ 16	5 Φ 16
	Columns	5 Φ 22	5 Φ 12	5 Φ 22	5 Φ 12

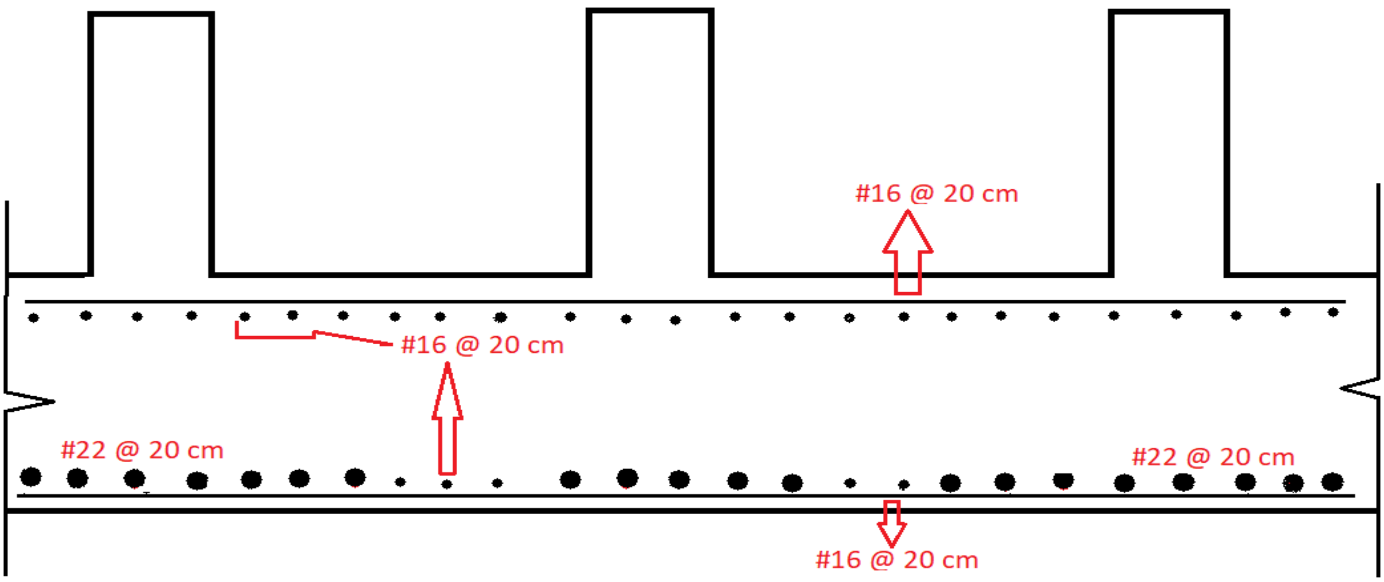


Figure 7.26: Summary of reinforcement (Positive & negative)

Chapter 8: Staircase

Stairs are constructed to provide access to different floor levels within buildings. They consist of several steps arranged in series. Most of stairs are designed as simply supported one-way solid slab.

The definition of some technical terms which are used in connection with the planning and design of stairs are shown in Figure

1. **Waist (h):** The thickness of the stair slab.
2. **Go (G):** The horizontal distance of the step.
3. **Rise (R)**: The vertical distance between two consecutive horizontal steps.
4. **Landing:** The horizontal platform which is usually provided at the beginning and the end of the series of steps**.**
5. **Flight:** The vertical distance between two consecutive landing.

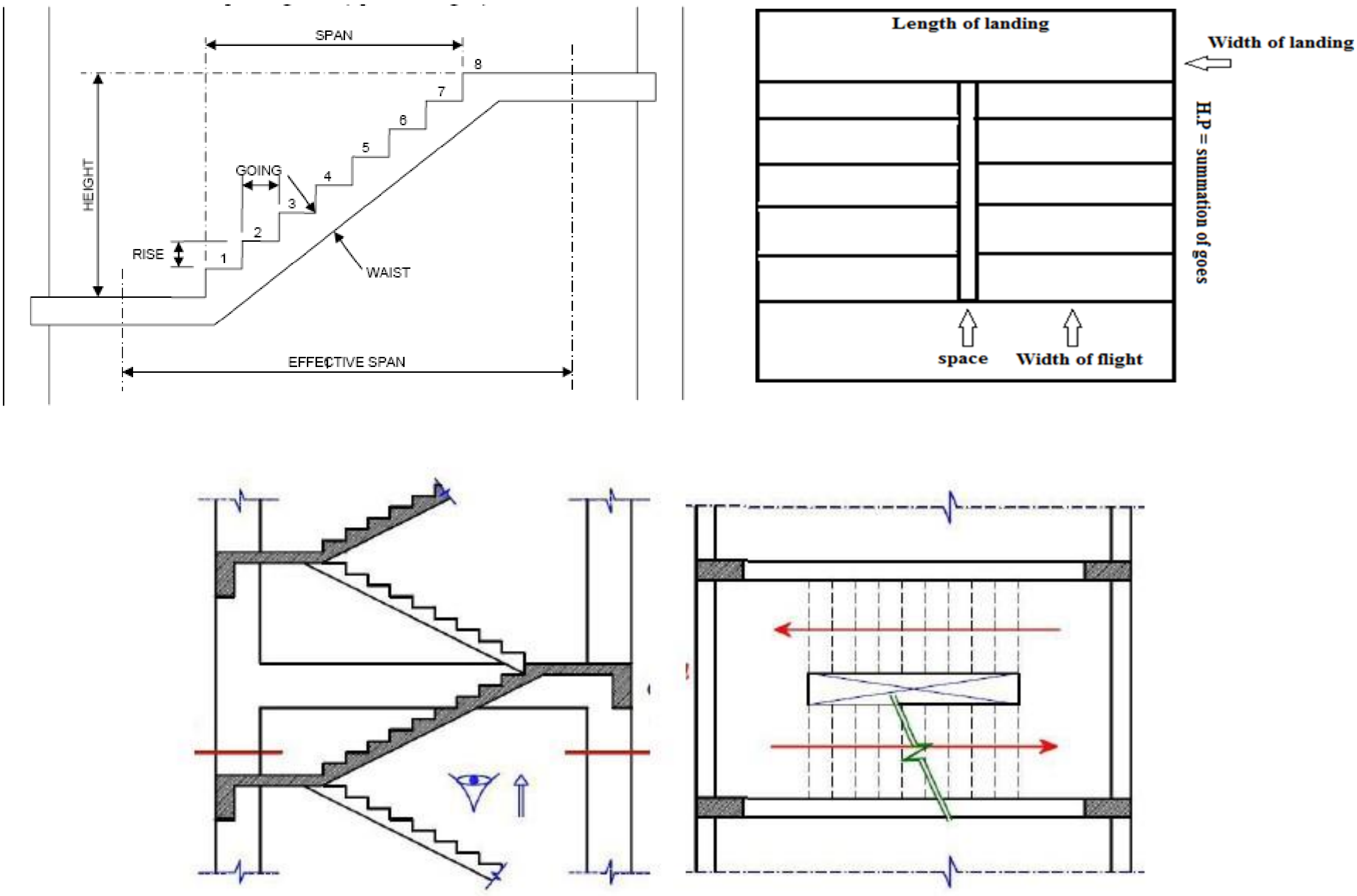


Figure8.1: some technical terms which are used in connection with the planning and design of stairs

8.1 Stair Design Procedure

The Stairway is designed by dividing it into 3 parts:(Flight and two Landings).
The Flight is simply supported while the Landings are Simply supported on beam.
Design of Flight and landing:

- Calculating (Wu) then (Mu) then get (Rn) and (ρ).
- Check if (ρ_{min} < ρ <ρ_{max})
- Find (As = ρ*b*d)
- For other direction, use (Temperature and Shrinkage Steel Reinforcement, As= 0.0018*b*h).

The following details will be used in the design:

- Goes length must be (27-30 cm), we take goes in all structure floors = 30cm.
- Rise height must be (15-17 cm), we take rises in all structure floors = 15cm.

⇒ In our project we have three different high typical stair 3.4m, roof stair 4m, and base floor stair 5.35m

In this section, we will show detailed calculation for roof floor stair

1. Dimension:

- **From architectural drawing, we got the stair component:**
  - Height of the floor = 4m.
  - Take Rise = 15 cm.
  - Take Go = 30 cm.
  - length of the landing = 2.7 m
  - Number of rises = $\frac{4}{0.15}$ =27 ⇒ 14 ≤ 14 OK, 13 < 14 OK, So use two flights, first flight has 14 rises and the second one has 13 rises.
  - Number of Go's = Number of rises -1 = 14- 1 = 13.
  - Horizontal Projection of each flight (H.P) = 13x0.3 = 3.9 m.
  - Space between two flight = 0.1m.
  - Length of the step = $\frac{2.7-0.1}{2}$ = 1.3 m
  - Width of the landing = at least length of one step = 1.3 m.

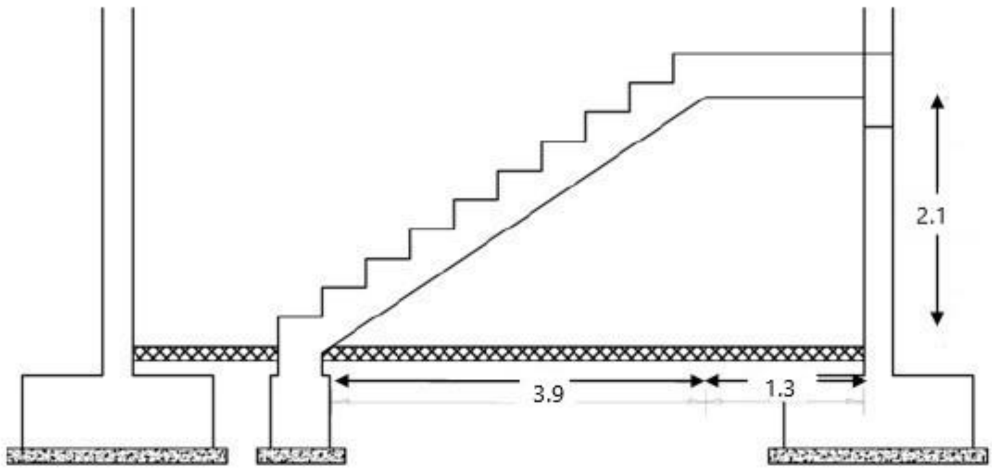


Figure 8.2:roof stair section

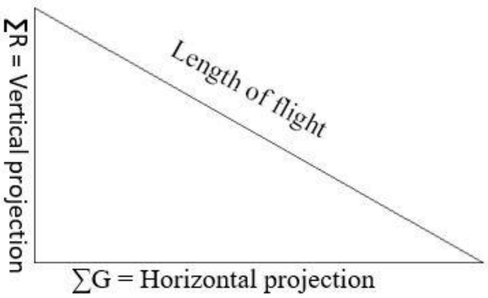


Figure 8.3: detailed stair section 1

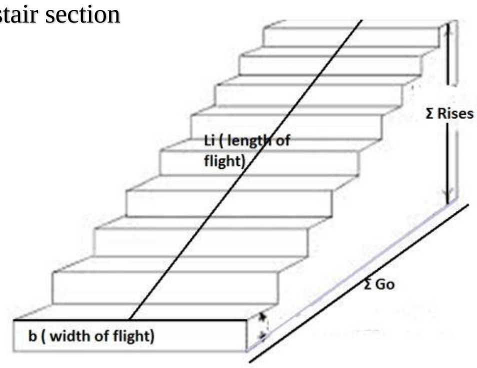


Figure 8.4: detailed stair section 2

- Length of flight = $\sqrt{3.9^2 + 2.1^2}$ = 4.43m
- Thickness of stair:

For control the deflection (h). Using ACI Code Deflection Conditions.

Table 8.1: minimum thickness of solid one way slab, beams and ribbed slab

Member	Minimum Thickness *h*			
	Simply Supported	One End Continuous	Both Ends Continuous	Cantilever
	Members Not Supporting or Attached to Partitions or Other Construction Likely to Be Damaged by Large Deflections			
Solid one-way slabs	$l/20$	$l/24$	$l/28$	$l/10$
Beams or ribbed one-way slabs	$l/16$	$l/18.5$	$l/21$	$l/8$

$\boldsymbol{L_{min}} = \frac{\boldsymbol{L}}{\boldsymbol{24}} = \frac{\boldsymbol{4430}}{\boldsymbol{24}} = 184.58\ mm\ \ldots\ Use\ \boldsymbol{L_{min}} = 0.185m \Rightarrow \text{thickness} = 0.185m = 185mm$

Thickness of landing = thickness of stair = 185mm

8.2 Loads

❈ **Load calculation:**

❈ **Load on the stair slab:**

- own weight of steps = $\frac{0.15*0.3*24*14}{2}$ =7.56KN/m^2

- Finishing:

Tiles= $\frac{30mm}{10mm}$*20*14*$\frac{1}{1000}$=0.84 KN/m^2

Cement mortar=0.03*21*1= 0.63 KN/m^2

Plaster== $\frac{20mm}{10mm}$*21*14*$\frac{1}{1000}$ =0.588 KN/m^2

Σ DL= 9.618 KN/m^2

Σ LL= 3 KN/m^2 (Jordanian code)

- Design load:

W_u= 1.2DL + 1.6 LL = 16.342 KN/m^2

❈ **Load on stair landing:**

-own weight of landing=0.19*24=4.56 kN/m^2

- Additional load from flight= = $\frac{36.2}{4.43} = 8.172$kN/$m^2$

- Finishing:

Tiles= 0.84 KN/m^2

Cement mortar= 0.63 kN/m^2

Plaster= 0.588 KN/m^2

Σ DL= 14.79 KN/m^2

ΣLL=3KN/m^2(Jordanian code)

- Design load:

W_u= 1.2DL + 1.6 LL = 22.55 kN/m^2

Table 8.2: Live Load from Jordanian code

الجدول (٣–١–ب)

الأحمال الحية للأرضيات والعقدات

الحمل المركز البديل	الحمل الموزع	الاستعمال	نوع المبنى	
كن	كن/م٢	الاشغال	خاص	عام
1.4	2.0	جميع الغرف بما في ذلك غرف النوم والمطابخ وغرف الغسيل وما شابه ذلك (All Usages).	النوع الأول : مباني الشقق السكنية التي لا يزيد ارتفاعها عن ثلاثة طوابق ولا يزيد عدد الشقق التي يمكن الوصول إليها من خلال درج مشترك عن أربع شقق للطابق الواحد.	المباني الخاصة والسكنية.
1.8	2.0	غرف النوم.	النوع الثاني : المباني التي لا ينطبق عليها ما ورد في النوع الأول والبنسيونات والمباني المخصصة لاقامة الضيوف.	
–	2.0	الحمامات.		
2.7	2.0	الطعام وردهات الاستراحة والبلياردو.		
4.5	3.0	الممرات والمداخل والأدراج وبسطات الأدراج والممرات المرتفعة الموصلة بين المباني.		
4.5	3.0	المطابخ وغرف الغسيل.		

8.3Analysis and Reinforcements Design

❋ Stair Flight Analysis and Design:

$$M_u = \frac{W_u * L^2}{8} = \frac{16.342 * 4.43^2}{8} = 40.10 \text{ kN.m}$$

$$V_u = \frac{W_u * L}{2} = \frac{16.342 * 4.43}{2} = 36.20 \text{ kN}$$

❋ Design for main reinforcement

$M_u = 40.10\ kN.m$

b=1m = 1000mm

$d = h - cover = 185 - 30 = 155mm$

$$R_n = \frac{M_u}{\Phi * b * d^2} = \frac{40.10 * 10^6}{0.9 * 1000 * 155^2} = 1.85$$

Percentage of required tension reinforcement:

$$\rho = 0.85 * \frac{\sqrt{f'_c}}{f_y}\left(1 - \sqrt{1 - \frac{2.36 * Rn}{f'_c}}\right)$$

$$\rho = 0.85 * \frac{\sqrt{28}}{420}\left(1 - \sqrt{1 - \frac{2.36 * 1.85}{28}}\right) = 0.0009$$

$$\rho_{min} = \frac{0.25 * \sqrt{f'_c}}{f_y} > \frac{1.4}{fy} = 0.00315 > 0.0033 \Rightarrow \rho_{min} = 0.0033$$

$$\rho_{max} = 0.85 * \beta * \frac{f'_c}{f_y} * \frac{\zeta}{\zeta + 0.005} = 0.85 * 0.85 * \frac{28}{420} * \frac{0.003}{0.003 + 0.005} = 0.018$$

$\rho < \rho_{min} < \rho_{max} \quad \Rightarrow 0.0009 < 0.0033 < 0.018 \Rightarrow$ use $\rho = 0.0033$

As= ρ*b*d = 0.0033*1000*155=511.5 mm^2

As_{min} = 0.0018*b*h = 0.0018 x 1000 x 185 = 333 mm^2

As > As_{min} ⇒ OK

Use ((ϕ16@150mm))

S_{max}=3h= 555mm > S=150 mm ((OK))

- **Check Shear:**

ϕVc= ϕ*$\frac{f'c}{6}$* b *d = 0.75*$\frac{28}{6}$*1000*155=54.25kN

ϕVc > Vu $\Rightarrow$ 54.25 > 36.2 ((OK))

- **Landing Analysis and Design:**

$-M_u = \frac{W_u * L^2}{12} = \frac{22.55 * 2.7^2}{12}$ = 13.7 kN.m

$+M_u = \frac{W_u * L}{24} = \frac{22.55 * 2.7^2}{24}$ = 6.85 kN.m

$V_u = \frac{W_u * L}{2} = \frac{22.55 * 2.7}{2}$ = 30.44 kN

- **Design for main reinforcement**

  - Negative moment:

$M_u = 13.7\ kN.m$

b=1m = 1000mm

$d = h - cover = 185 - 30 = 155mm$

$R_n = \frac{M_u}{\Phi * b * d^2} = \frac{13.7 * 10^6}{0.9 * 1000 * 155^2}$ = 0.634

Percentage of required tension reinforcement:

ρ=0.85 * $\frac{\sqrt{f'_c}}{f_y}$(1-$\sqrt{1 - \frac{2.36 * Rn}{f'_c}}$)

ρ=0.85 * $\frac{\sqrt{28}}{420}$(1-$\sqrt{1 - \frac{2.36 * 0.634}{28}}$)=0.00029

ρ_{min}=$\frac{0.25 * \sqrt{f'_c}}{f_y} > \frac{1.4}{f_y}$ = 0.00315<0.0033 $\Rightarrow$ ρ_{min}=0.0033

ρ_{max}=0.85*β*$\frac{f'_c}{f_y}$*$\frac{\zeta}{\zeta + 0.005}$=0.85*0.85*$\frac{28}{420}$*$\frac{0.003}{0.003+0.005}$=0.018

ρ <ρ_{min}<ρ_{max} $\Rightarrow$ 0.00029<0.0033<0.018 $\Rightarrow$ use ρ=0.0033

As= ρ*b*d = 0.0033*1000*155=511.5 mm^2

As_{min} = 0.0018*b*h = 0.0018 x 1000 x 185 = 333 mm^2

As > As_{min} $\Rightarrow$ OK

Use ((ϕ16@150mm))

S_{max}=3h= 555mm > S=150 mm ((OK))

- Positive moment:

$M_u = 6.85\ kN.m$

b=1m = 1000mm

$d = h - cover = 185 - 30 = 155mm$

$R_n = \frac{M_u}{\Phi * b * d^2} = \frac{6.85*10^6}{0.9*1000*155^2} = 0.318$

Percentage of required tension reinforcement:

$\rho = 0.85 * \frac{\sqrt{f'_c}}{f_y}(1-\sqrt{1 - \frac{2.36*Rn}{f'_c}})$

$\rho = 0.85 * \frac{\sqrt{28}}{420}(1-\sqrt{1 - \frac{2.36*0.318}{28}})$=0.00014

$\rho_{min} = \frac{0.25*\sqrt{f'_c}}{f_y} > \frac{1.4}{fy}$ = 0.00315<0.0033 ⇒ ρ_{min}=0.0033

ρ_{max}=0.85*β*$\frac{f'_c}{f_y} * \frac{\varsigma}{\varsigma+0.005}$=0.85*0.85*$\frac{28}{420} * \frac{0.003}{0.003+0.005}$=0.018

ρ <ρ_{min}<ρ_{max} ⇒ 0.00014<0.0033<0.018 ⇒use ρ=0.0033

As= ρ*b*d = 0.0033*1000*155=511.5 mm^2

As_{min} = 0.0018*b*h = 0.0018 x 1000 x 185 = 333 mm^2

As > As_{min} ⇒ OK

Use ((ϕ16@150mm))

S_{max}=3h= 555mm > S=150 mm ((OK))

❋ Additional reinforcement for Stair:

- Provide steel for shrinkage and temperature:

  As = 0.0018*b*h = 0.0018 x 1000 x 185 = 333 mm^2

  Use ((ϕ10@200mm))

  S_{max}=3h= 555mm > S=150 mm ((OK))

- Provide steel for step reinforcement (anti crack):

  Use ((ϕ8@150mm)) in both direction

  S_{max}=3h= 555mm > S=150 mm ((OK))

  And **Use ((ϕ8@ each step))**

❋ Beam with stair Design:

We use Prokon to design this beam

✓ Deflection:

Using ACI-Code, Table 9.5 (b) ⟹ Case 4.

Permissible deflection = $\frac{2.3}{240}$ = 0.96 cm

Actual deflection = 0.069 cm < 0.96 (okay)

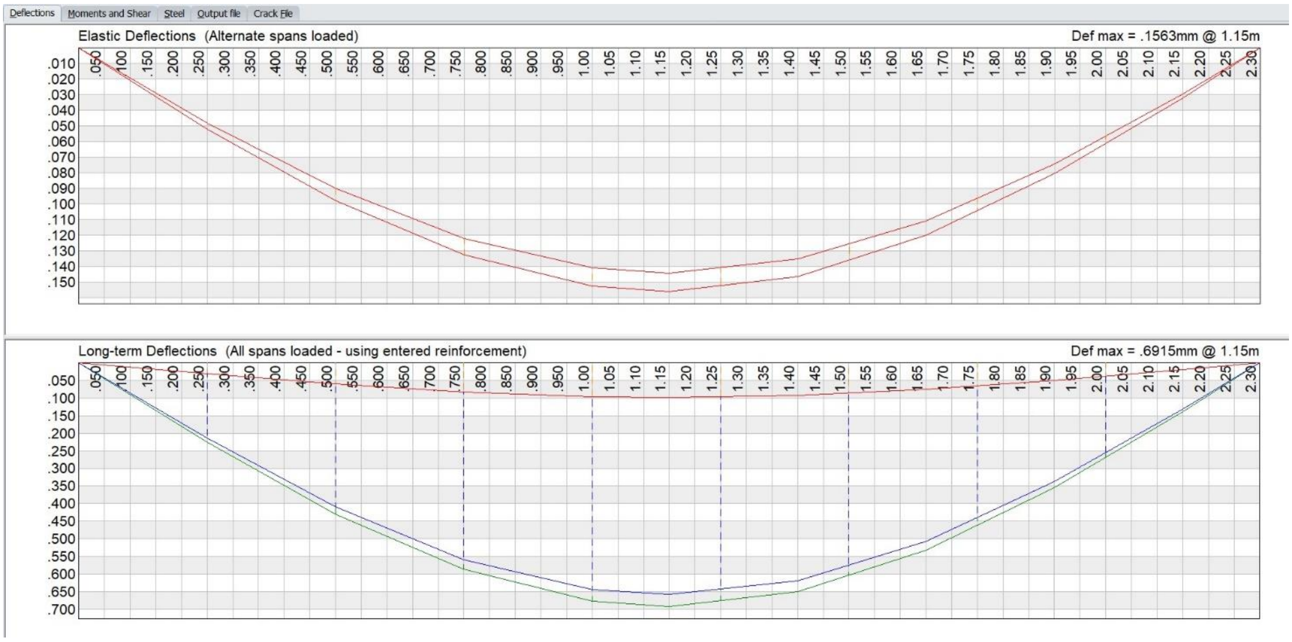


Figure 8.5: Prokon Defletion in stair beam

✓ Load value:

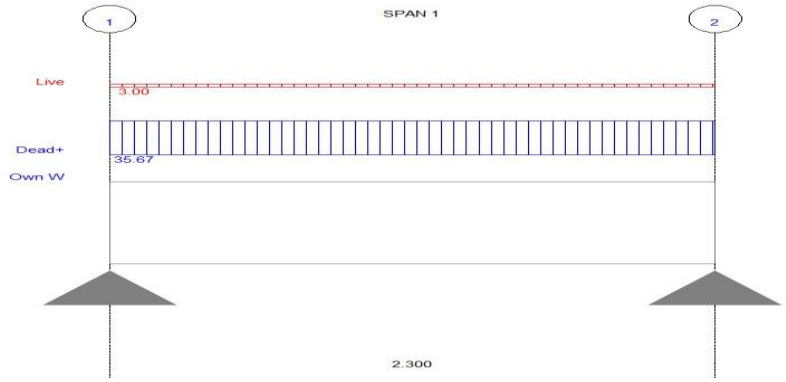


Figure 8.6: Prokon load value in stair beam

Moment & Shear value

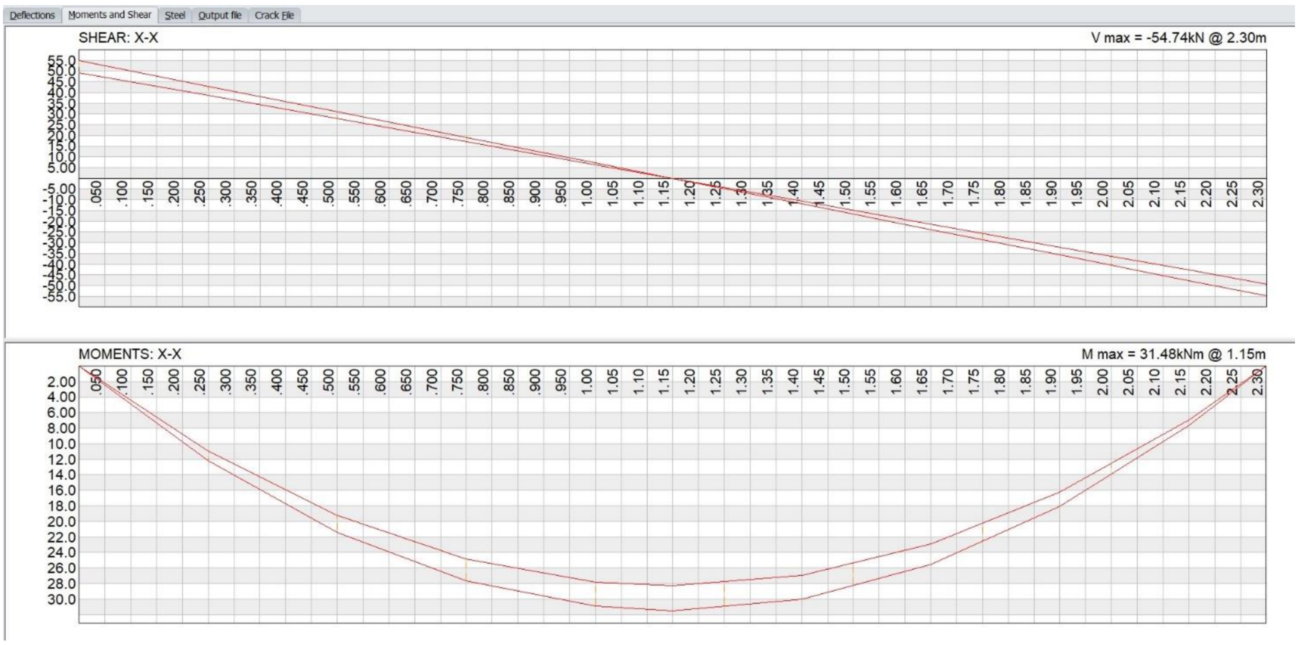


Figure 8.7: Prokon moment and shear in stair beam

✓ Reinforcement (Moment):

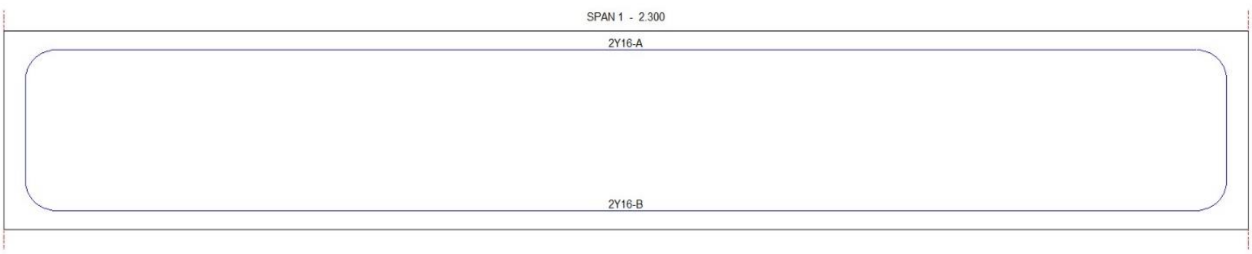


Figure 8.8: Prokon reinforcement moment in stair beam

✓ Reinforcement (Shear):

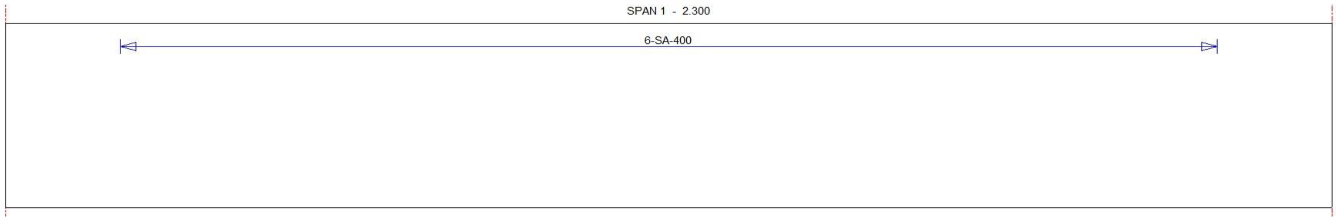


Figure 8.9: Prokon reinforcement Shear in stair beam

✓ Section of Stair Beam:

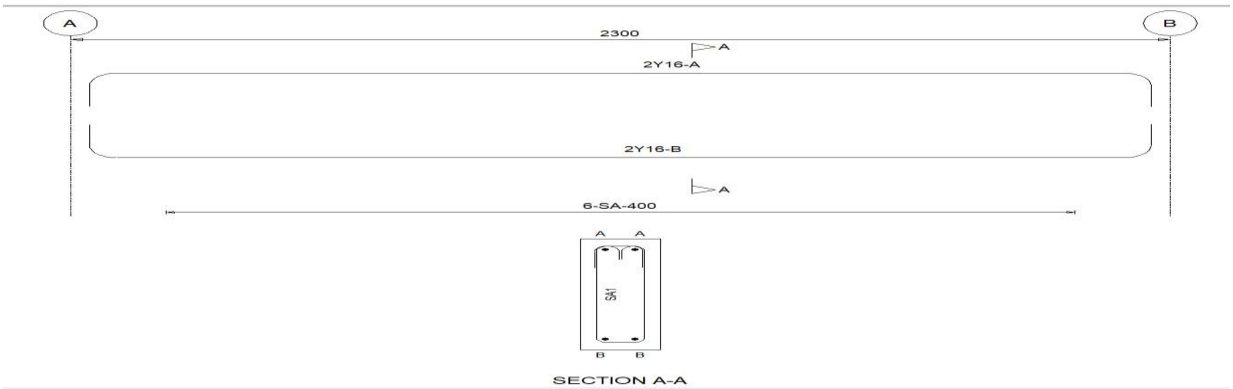


Figure 8.10: Section of Stair beam

First floor detailed

The height of the first floor is 6m, Height of the stair = 5.35m.

Number of rises = $\frac{5.35-(0.185+0.2+0.2+0.2)}{0.15}$ =31 ⇒ 16 > 14 not OK.

so, we use three flights, first flight has 3 rises, the second one has 14 rises and the third one has 14 rises.

▪

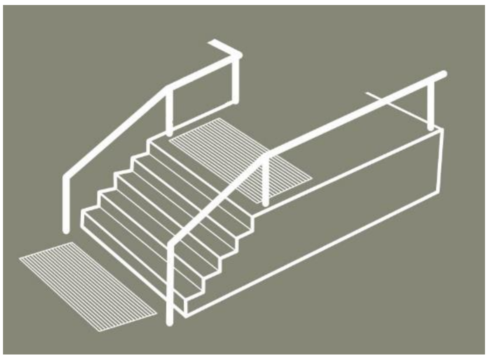


Figure 8.11: entrance stairs

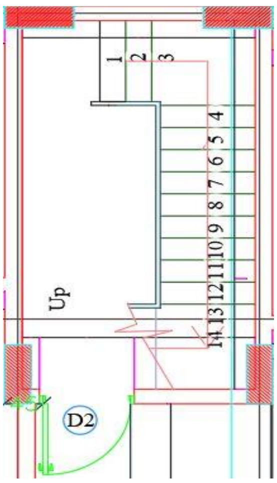


Figure 8.12: autocad Clarification for first flight in first floor stairs

Table 8.3: Summary of reinforcement

Floor Name	Hight (m)	No of rises in first flight	No of rises in Second flight	No of rises in third flight	Main reinforcement	Landing negative moment reinforcement	Landing positive moment reinforcement	Additional reinforcement
First Floor	5.35	3	14	14	ϕ16@150mm	ϕ16@150mm	ϕ16@150mm	ϕ10@200mm ϕ8@150mm ϕ8@step
Typical	3.4	11	11	———	ϕ16@150mm	ϕ16@150mm	ϕ16@150mm	ϕ10@200mm ϕ8@150mm ϕ8@step
Roof	4	14	13	———	ϕ16@150mm	ϕ16@150mm	ϕ16@150mm	ϕ10@200mm ϕ8@150mm ϕ8@step
Entrance	0.45	3	———	———	ϕ16@150mm	ϕ16@150mm	ϕ16@150mm	ϕ10@200mm ϕ8@150mm ϕ8@step

❈ Roof cross section for Stairs:

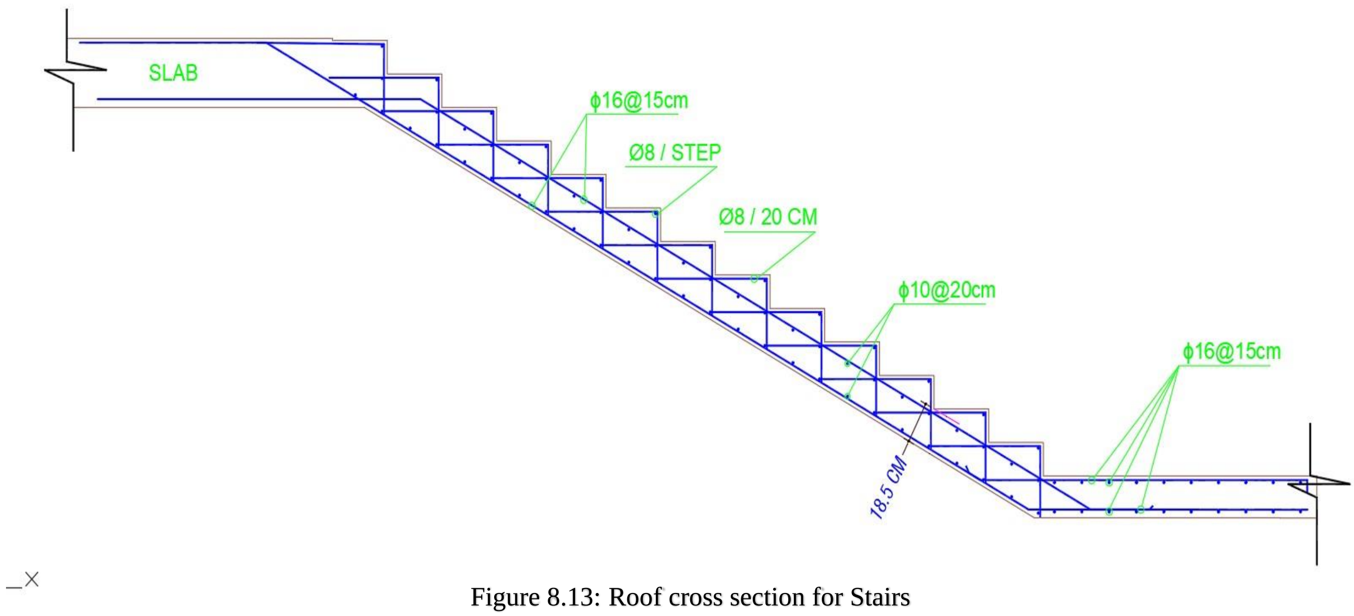


Figure 8.13: Roof cross section for Stairs

Chapter 9: QUANTITY SURVEY AND COST ESTIMATE

Quantity survey is essential to estimate before the construction starts the probable cost of construction for the complete work. The estimate is required in inviting tenders for the works and to arrange contract for a complete project.

Quantity survey is required to estimate the quantities of the various materials required and the labor involved for satisfactory completion of a construction project.

We use (Unit weight of standard reinforcing steel bars) table to determine weight of rebar in ton.

Table 9.1: Unit weight of standard reinforcing steel bars

Diameter (mm)	Cross Sectional Area (m2)	Mass Per Meter (kg/m)	Mass Per 12 Meter (kg/12m)
6	28.3	0.222	2.664
9	63.6	0.499	5.988
10	78.5	0.616	7.392
12	113.1	0.888	10.656
13	132.7	1.042	12.504
14	153.9	1.208	14.496
16	201.1	1.579	18.948
18	254.5	1.998	23.976
20	314.2	2.466	29.592
22	380.1	2.984	35.808
25	490.9	3.854	46.248
28	615.8	4.834	53.008
32	804.2	6.313	75.755
35	962.1	7.553	90.636
38	1134.1	8.903	106.836

Prices applied are given by local suppliers

- CONCRETE

Table 9.2: Offer prices of Concrete by local suppliers

F′c , Supplier	Kingdom Concrete	Nuqul Concrete	ARMCOO
28 MPa	52 JOD/m^3	54 JOD/m^3	53 JOD/m^3
30 MPa	53 JOD/m^3	56 JOD/m^3	57 JOD/m^3

Based on the prices provided for the project, we choose Kingdom Concrete As a Ready Mixed Concrete Supplier.

- REBAR

Table 9.3: Offer prices of Rebar by local suppliers

Φ , Supplier	Eamar Jordan for steel	Ali abu Shamaa steel and cement Est	Emirates Jordanian Company for steel
Φ8	630 JOD/ton	690 JOD/ton	610 JOD/ton
Φ10	620 JOD/ton	640 JOD/ton	618 JOD/ton
Φ12	600 JOD/ton	620 JOD/ton	590 JOD/ton
Φ14	600 JOD/ton	620 JOD/ton	590 JOD/ton
Φ16	600 JOD/ton	620 JOD/ton	590 JOD/ton
Φ18	600 JOD/ton	620 JOD/ton	590 JOD/ton
Φ20	600 JOD/ton	620 JOD/ton	590 JOD/ton
Φ22	600 JOD/ton	620 JOD/ton	590 JOD/ton
Φ25	600 JOD/ton	620 JOD/ton	600 JOD/ton

Based on the prices provided for the project, we choose Emirates Jordanian Company for steel as a Rebar Supplier.

9.1 Quantities and Costs of Beams

In this section, we will calculate the Quantities and Cost of Concrete and Rebar, that used in Beams.

Figure 9.1: Detailed of ground tie beams

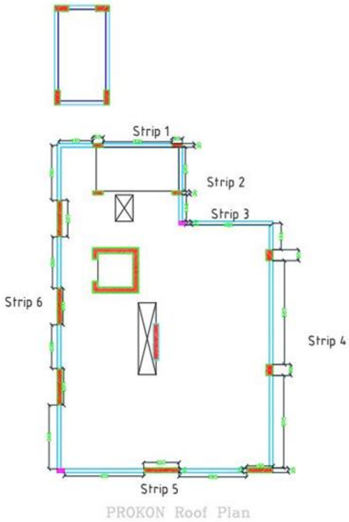


Figure 9.2: Detailed of roof beams

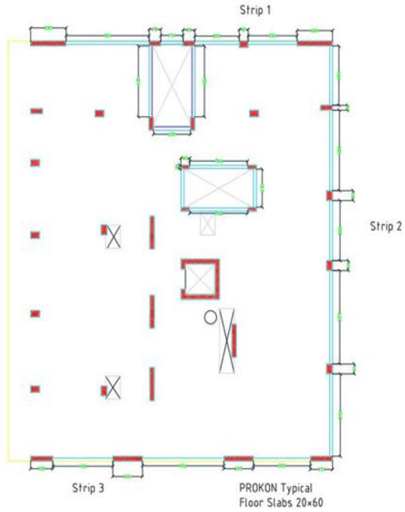


Figure 9.3: Detailed of typical beams

all beams in the project have same dimension and have same section of Stirrups

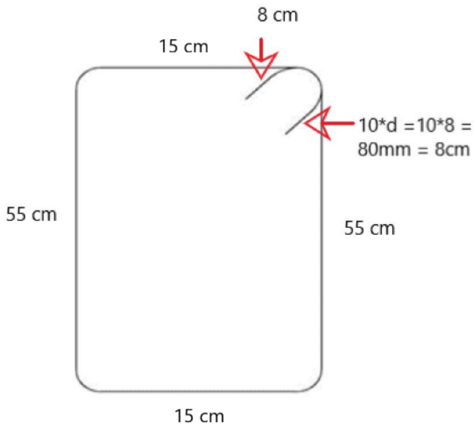


Figure 4: beams dimension and section of stirrups in beams

❋ Quantity and cost of Rebar and Stirrups:

Table 9.4: Quantity and cost of Rebar for beams

Location	d(mm)	Number	Wt/m(kg/m)	Length (m)	Total weight (kg)
TYPICAL					
Strip 1	12	2	0.888	6.5	11.5
Strip 1	16	2	1.579	7.5	23.69
Strip 1	16	2	1.579	4.5	14.2
Strip 1	12	1	0.888	1.9	1.69
Strip 1	16	2	1.579	3.3	10.42
Strip 1	16	3	1.579	2.1	6.632
Strip 1	12	2	0.888	11	19.54
Strip 1	12	2	0.888	10	17.76
Strip 1	12	2	0.888	1.5	2.66
Strip 1	16	1	1.579	3	4.74
NUMBER OF STIRRUPS = 47					
Strip 2	12	2	0.888	9.5	30
Strip 2	12	2	0.888	10	17.76
Strip 2	12	4	0.888	1.5	5.33
Strip 2	12	1	0.888	2.5	2.22
Strip 2	12	2	0.888	11	19.54
Strip 2	12	5	0.888	3.5	15.54
Strip 2	16	2	1.579	2.1	6.63
Strip 2	16	3	1.579	2	9.47
Strip 2	12	2	0.888	5.5	9.77
Strip 2	16	2	1.579	2.6	8.21
Strip 2	12	2	0.888	6	10.66
Strip 2	12	2	0.888	5	8.88
Strip 2	16	1	1.579	3	4.74
NUMBER OF STIRRUPS = 62					
Strip 3	12	4	0.888	5	17.76
Strip 3	12	6	0.888	6	31.97

Strip 3	12	2	0.888	3.1	5.51
Strip 3	16	2	1.579	3.2	10.11
Strip 3	16	6	1.579	2	18.95
Strip 3	12	2	0.888	8	14.21
Strip 3	16	2	1.579	5	15.79
Strip 3	12	1	0.888	2	1.78
Strip 3	16	2	1.579	3.5	11.053
Strip 3	16	1	1.579	3.1	4.89
NUMBER OF STIRRUPS = 52					
ROOF					
Strip 1	12	4	0.888	6.5	23.088
Strip 1	12	2	0.888	1.5	2.66
Strip 1	12	1	0.888	2.5	2.22
NUMBER OF STIRRUPS =16					
Strip 2	12	4	0.888	4.1	14.56
NUMBER OF STIRRUPS =10					
Strip 3	12	4	0.888	5	17.76
Strip 3	16	1	1.579	3.1	4.89
NUMBER OF STIRRUPS =13					
Strip 4	12	2	0.888	7.5	13.32
Strip 4	12	2	0.888	8.1	14.36
Strip 4	12	2	0.888	1.5	2.66
Strip 4	16	1	1.579	3.5	5.53
Strip 4	16	2	1.579	2.5	7.89
Strip 4	16	1	1.579	2	3.16
Strip 4	12	2	0.888	5.6	9.95
Strip 4	12	2	0.888	5	8.88
Strip 4	16	1	1.579	3.1	4.89
NUMBER OF STIRRUPS =34					
Strip 5	12	4	0.888	5	17.76
Strip 5	12	4	0.888	6	21.31
Strip 5	16	1	1.579	3.5	5.53

Strip 5	16	2	1.579	3	4.74
Strip 5	16	2	1.579	2	6.32
Strip 5	12	2	0.888	3.5	6.22
NUMBER OF STIRRUPS =28					
Strip 6	12	6	0.888	9	47.95
Strip 6	12	2	0.888	9.5	16.87
Strip 6	12	4	0.888	1.5	5.33
Strip 6	12	1	0.888	2.1	1.86
NUMBER OF STIRRUPS =44					
STAIRS BEAMS					
Strip 1	16	4	1.579	2.3	14.53
NUMBER OF STIRRUPS =6					
Strip 2	12	2	0.888	5.2	9.24
Strip 2	16	2	1.579	5.2	16.42
Strip 2	12	1	0.888	4	3.55
Strip 2	16	1	1.579	3.5	5.53
NUMBER OF STIRRUPS =13					
Strip 3	16	4	1.579	2.3	14.53
NUMBER OF STIRRUPS =6					
Strip 4	12	2	0.888	5.2	9.24
Strip 4	16	2	1.579	5.2	16.42
Strip 4	12	1	0.888	4	3.55
Strip 4	16	1	1.579	3.5	5.53
NUMBER OF STIRRUPS =13					
GROUND TIE BEAMS					
Strip 1	12	2	0.888	8	14.21
Strip 1	12	2	0.888	9.5	16.87
Strip 1	12	2	0.888	4.6	8.17
Strip 1	12	2	0.888	9.7	17.23
Strip 1	12	2	0.888	4.8	8.52
Strip 1	16	2	1.579	8	25.26
Strip 1	16	2	1.579	9.5	30

Strip 1	16	2	1.579	9.7	30.63
NUMBER OF STIRRUPS = 70					
Strip 2	16	2	1.579	8	25.26
Strip 2	16	2	1.579	9.5	30
Strip 2	16	2	1.579	9.7	30.63
Strip 2	12	2	0.888	8	14.21
Strip 2	12	2	0.888	9.5	16.87
Strip 2	12	2	0.888	4.6	8.17
Strip 2	12	2	0.888	9.7	17.23
Strip 2	12	2	0.888	4.8	8.52
NUMBER OF STIRRUPS = 54					
Strip 3	16	2	1.579	7	22.11
Strip 3	16	2	1.579	6.5	20.53
Strip 3	16	2	1.579	5	15.79
Strip 3	12	2	0.888	9	15.98
Strip 3	12	2	0.888	4	7.10
Strip 3	12	2	0.888	6	10.66
Strip 3	12	2	0.888	9.5	16.87
Strip 3	16	2	1.579	7	22.11
NUMBER OF STIRRUPS = 44					
Strip 4	16	2	1.579	7	22.11
Strip 4	16	2	1.579	6.5	20.53
Strip 4	16	2	1.579	5	15.79
Strip 4	12	2	0.888	9	15.98
Strip 4	12	2	0.888	4	7.10
Strip 4	12	2	0.888	6	10.66
Strip 4	12	2	0.888	9.5	16.87
NUMBER OF STIRRUPS = 32					
Strip 5	16	2	1.579	6	18.95
Strip 5	12	2	0.888	6	10.66
NUMBER OF STIRRUPS = 24					

Strip 6	12	2	0.888	2	3.55
Strip 6	16	2	1.579	2	6.32
NUMBER OF STIRRUPS = 8					
TOTAL WEIGHT OF Φ 12 WITHOUT TIE GROUND BEAM (ONE STORY TYPICAL) =403.01 kg					
TOTAL WEIGHT OF Φ 16 WITHOUT TIE GROUND BEAM (ONE STORY TYPICAL) =193.43kg					
TOTAL WEIGHT OF Φ 12 FOR ROOF =281.48 kg					
TOTAL WEIGHT OF Φ 16 FOR ROOF =86.85 kg					
TOTAL WEIGHT OF Φ 12 TIE GROUND BEAMS =245.43 kg					
TOTAL WEIGHT OF Φ 16 (KG) TIE GROUND BEAM=336.02KG					
TOTAL NUMBER OF STIRRUPS Φ 8 WITHOUT TIE GROUND BEAM (ONE STORY TYPICAL) =211					
TOTAL NUMBER OF STIRRUPS Φ 8 ROOFS =183					
TOTAL NUMBER OF STIRRUPS Φ 8 TIE GROUND BEAMS=232					
TOTAL WEIGHT OF Φ 12 (KG) WITHOUT TIE GROUND BEAM (TYPICAL ALL BUILDING WITH STAIRS) =1612.04 kg = 1.61 ton					
TOTAL WEIGHT OF Φ 16 (KG) WITHOUT TIE GROUND BEAM (TYPICAL ALL BUILDING WITH STAIRS) =773.72 kg					
TOTAL NUMBER OF STIRRUPS Φ 8 WITHOUT TIE GROUND BEAM (TYPICAL ALL BUILDING WITH STAIRS) =844					
TOTAL WEIGHT OF Φ 16 FOR ALL BUILDING BEAMS=1109.74 kg = 1.11 ton					
TOTAL WEIGHT OF Φ 12 FOR ALL BUILDING BEAMS=1857.47 kg = 1.86 ton					
TOTAL NUMBER OF STIRRUPS Φ 8 FOR ALL BUILDING BEAMS=1076					
Length of the Stirrups (Φ8) = 15+55+15+55+8+8= 156 cm = 1.56 m					
Wt/m (kg/m) for (Φ8) = 0.395 kg/m, 0.395*1.56= 0.6162 kg, 0.6162*1076 = 663.04 kg					
TOTAL WEIGHT OF Φ 8 FOR ALL BUILDING BEAMS=663.04 kg					
COST OF Φ 16 FOR BEAMS = 1.1 *590 = 649 JOD					
COST OF Φ 12 FOR BEAMS =1.86*590 = 1097.4 JOD					
COST OF Φ 8 FOR BEAMS =0.66304*610 = 404.45 JOD					

❋ Quantity and Cost of Concrete:

Table 9.5: Quantity and Cost of Concrete for beams

Strip	B_w**(m)**	**D(m)**	**Length (m)**	**Volume of Concrete (m^3)**
TYPICAL				
Strip 1	0.2	0.6	18	2.16
Strip 2	0.2	0.6	25.27	3.03
Strip 3	0.2	0.6	18.9	2.27
ROOF				
Strip 1	0.2	0.6	6.7	0.8
Strip 2	0.2	0.6	4.15	0.5
Strip 3	0.2	0.6	5.1	0.6
Strip 4	0.2	0.6	13.54	1.63
Strip 5	0.2	0.6	11.3	1.36
Strip 6	0.2	0.6	17.8	2.14
STAIRS BEAMS				
Strip 1	0.2	0.6	2.3	0.28
Strip 2	0.2	0.6	5.2	0.62
Strip 3	0.2	0.6	2.3	0.28
Strip 4	0.2	0.6	5.2	0.62
GROUND TIE BEAMS				
Strip 1	0.2	0.6	25.6	3.07
Strip 2	0.2	0.6	25.6	3.07
Strip 3	0.2	0.6	19	2.28
Strip 4	0.2	0.6	19	2.28
Strip 5	0.2	0.6	6	0.72
Strip 6	0.2	0.6	2	0.24
VOLUME OF CONCRETE FOR ALL TYPICAL AND STAIRS BEAMS = 29.28 m^3				
VOLUME OF CONCRETE FOR ROOF BEAMS = 8.83 m^3				
VOLUME OF CONCRETE FOR ALL GROUND TIE BEAMS = 21.4 m^3				
VOLUME OF CONCRETE FOR ALL BUILDING BEAMS = 59.51 m^3				
COST OF CONCRETE FOR BEAMS = 59.51*52=3094.52 JOD				

9.2 Quantities and Costs of Slabs

In this section, we will calculate the quantities and cost of concrete and Rebar, that used in Slabs.

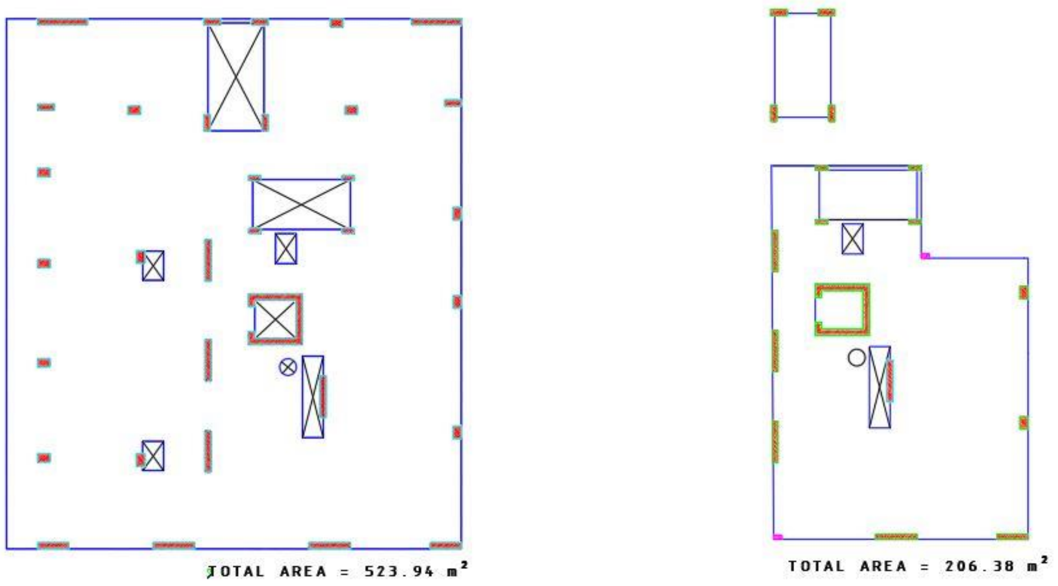


Figure 9.5: top view of slab in typical and roof

❈ Quantity and Cost of Concrete:

Table 9.6: Quantity and Cost of Concrete for slabs

NUMBER OF SLAB	TOTAL AREA (m^2)	THICKNESS	Volume of Concrete (m^3)
TYPICAL SLABS			
4	523.94	0.25	523.94
ROOF SLAB			
1	206.38	0.25	51.595
VOLUME OF CONCRETE FOR ALL BUILDING SLABS = 575.535 m^3			
COST OF CONCRETE FOR ALL BUILDING SLABS = 575.535*52=29927.82 JOD			

❋ Quantity and Cost of Rebar:

Table 9.7: Quantity and Cost of Rebar for slabs

SLAB	TOTAL AREA (m^2)	NO OF Bar/m	NUMBER	Wt/m(kg/m)	Total Weight (kg)
Top (-ve) Reinforcement					
TYPICAL	523.94	6Φ12	4	0.888	22332.419
ROOF	206.38	5Φ12	1	0.888	1832.655
Bottom (+ve) Reinforcement					
TYPICAL	523.94	6Φ12	4	0.888	22332.419
ROOF	206.38	5Φ12	1	0.888	1832.655
Around Columns, Top (-ve)					
ALL	6.25	9Φ14	37	1.208	2514.15
Around opening					
SLAB	**LENGTH (m)**	**RINFORCMENT**	**NUMBER**	**Wt/m(kg/m)**	**Total Weight (kg)**
ALL	3	40Φ20	4	2.466	1183.68
ALL	3	25Φ20	5	2.466	924.75
ALL	5.3	10Φ20	5	2.466	653.49
CHAIR REBAR					
SLAB	**LENGTH (m)**	**RINFORCMENT**	**NUMBER**	**Wt/m(kg/m)**	**Total Weight (kg)**
TYPICAL	0.75	524Φ12	4	0.888	1395.936
ROOF	0.75	207Φ12	1	0.888	137.862
TOTAL WEIGHT OF Φ 12 FOR ALL SLABS=49863.946 kg=49.87 ton					
TOTAL WEIGHT OF Φ 14 FOR ALL SLABS =2514.15kg=2.52 ton					
TOTAL WEIGHT OF Φ 20 FOR ALL SLABS =2762.92kg=2.77ton					
TOTAL COST OF Φ 12 FOR ALL SLABS= 49.87*590 = 29423.3 JOD					
TOTAL COST OF Φ 14 FOR ALL SLABS=2.52*590= 1486.8 JOD					
TOTAL COST OF Φ 20 FOR ALL SLABS = 2.77*590= 1634.3 JOD					

Length of chair rebar= 40cm + 2h

h= thickness of slab - 4d – cover -curve = 25-4*1.2-2-1 = 17.2 cm

Length of chair rebar= 40 + (2*17.2) = 74.4 cm ≅ 75 cm

Length of chair rebar= 75 cm = 0.75 m

<u>9.3</u> Quantities and Costs of Columns & Shear wall

In this section, we will calculate the Quantities and Cost of Concrete and Rebar, that used in Columns and shear wall.

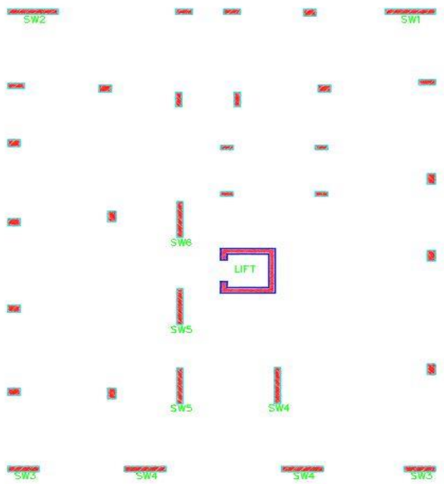


Figure 9.6: Structure columns and shear walls in typical

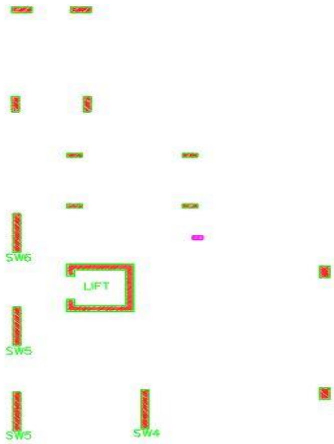


Figure 9.7: Structure columns and shear walls in roof

✵ Quantity and Cost of Concrete, Rebar & Stirrups:

⇒ There are two types of column Ties:

- 6 legged Ties

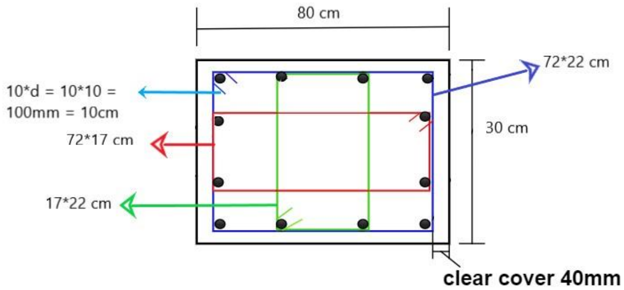


Figure 9.8: column Ties (6 legged Ties)

- Standard Ties

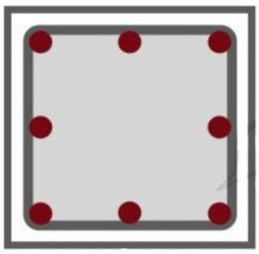


Figure 9.9: column Ties (Standard Ties)

Table 9.8: Quantities and Cost of Concrete and Rebar for columns and shear walls

CONCRETE				
COLUNMS				
STORY NAME: FIRST STORY				
COLUNM DIMENTION (cmXcm)	**LENGTH (M)**	**NUMBER**	**CONCRETE VOLUM (m^3)**	
80X30	6	6	8.64	
60X40	6	12	17.28	
60X25	6	4	3.6	
STORY NAME: TYPICAL STORYES				
80X30	3.5	18	15.12	
60X40	3.5	36	30.24	
60X25	3.5	12	6.3	
STORY NAME: FINAL STORY (ROOF)				
80X30	4	4	3.84	
60X20	4	4	1.92	
40X20	4	2	0.64	
60X40	4	2	1.92	
TOTAL CONCRETE VOLUM (m^3) FOR ALL BILDUING COLUMNS = 89.5 m^3				
TOTAL COST OF CONCRETE FOR COLUMNS = 89.5*52 = 4654 JOD				
SHEAR WALL				
STORY NAME: FIRST STORY				
SHEAR WALL DIMENTION (cmXcm)	**LENGTH (M)**	**NUMBER**	**CONCRETE VOLUM (m^3)**	
240X30	6	2	8.64	
200X30	6	6	21.6	
150X30	6	4	10.8	
STORY NAME: TYPICAL STORYES				
240X30	3.5	6	15.12	
200X30	3.5	36	75.6	
150X30	3.5	16	25.2	
STORY NAME: FINAL STORY (ROOF)				
200X30	4	5	12	
150X30	4	1	1.8	
LIFT				
890X30	20.5	———	54.735	
TOTAL CONCRETE VOLUM (m^3) FOR ALL BILDUING SHEAR WALL = 225.495 m^3				
TOTAL COST OF CONCRETE FOR SHEAR WALL = 225.495*52=11725.74 JOD				
REBAR				
COLUNMS				
REAINFORCMENT				
REAINFORCMENT FOR EACH COLOUMN	**LENGTH(M)**	**NUMBER**	**Wt/m(kg/m)**	**Total Weight (kg)**
12Φ16	20.5	45	1.579	17479.5
8Φ16	20.5	15	1.579	3884.34
12Φ16	16.5	40	1.579	12505.68
TOTAL WEIGHT OF Φ 16 FOR ALL BUILDING COLUNMS=33869.52kg=33.87ton				
TOTAL COST OF Φ 16 FOR ALL BUILDING COLUNMS =33.87*590=19983.3 JOD				

TIES				
TIES DIMENTION (cmXcm)	**LENGTH(M)**	**NUMBER OF TIES**	**Wt/m(kg/m) (Φ10)**	**Total Weight (kg)**
6 LEGGED TIES				
STORY NAME: 1st STORY				
72X22	2.08	180	0.617	231
52X32	1.88	360	0.617	417.5856
52X17	1.58	120	0.617	116.9832
STORY NAME: TYPICAL STORYES				
72X22	2.08	324	0.617	415.8086
52X32	1.88	648	0.617	751.65
52X17	1.58	216	0.617	134.852
STORY NAME: FINAL STORY (ROOF)				
72X22	2.08	80	0.617	102.67
52X12	1.48	80	0.617	73.053
32X12	1.08	40	0.617	26.65
52X32	1.88	40	0.617	26.65
STORY NAME: 1st STORY				
72X17	1.78	180	0.617	197.687
52X17	1.38	360	0.617	306.526
STORY NAME: TYPICAL STORYES				
72X17	1.78	324	0.617	355.84
52X17	1.38	648	0.617	551.77
STORY NAME: FINAL STORY (ROOF)				
72X17	1.78	80	0.617	87.86
52X17	1.38	40	0.617	34.06
STORY NAME: 1st STORY				
22X17	0.78	180	0.617	86.627
32X17	0.98	360	0.617	217.678
STORY NAME: TYPICAL STORYES				
22X17	0.78	324	0.617	155.931
32X17	0.98	648	0.617	391.82
STORY NAME: FINAL STORY (ROOF)				
22X17	0.78	80	0.617	38.50
32X17	0.98	40	0.617	24.187
STANDARD TIES				
STORY NAME: 1st STORY				
52X17	1.38	120	0.617	102.18
STORY NAME: TYPICAL STORYES				
52X17	1.38	216	0.617	183.92
TOTAL WEIGHT OF Φ 10 FOR ALL BUILDING TIES FOR COLUMNS =5031.48 kg=5.04 ton				
COST OF Φ 10 FOR COLUMNS = 5.04*618 = 3109.46 JOD				

SHEAR WALL					
HORIZONTAL STEEL					
STORY NAME: 1st STORY					
NAME OF SHEAR WALL	**REAINFORCMENT FOR EACH SHEAR WALL**	**LENGTH(M)**	**NUMBER**	**Wt/m(kg/m)**	**Total Weight (kg)**
SW1	80Φ10	2.4	1	0.617	118.464

SW2	80Φ12	2.4	1	0.888	170.496
SW3	80Φ12	1.5	2	0.888	213.12
SW4	80Φ12	2	3	0.888	426.24
SW5	80Φ12	2	2	0.888	284.16
SW6	80Φ12	2	1	0.888	142.08
STORY NAME:2nd & 3rd STORYES					
SW1	47Φ10	2.4	1	0.617	69.598
SW2	47Φ12	2.4	1	0.888	100.167
SW3	47Φ12	1.5	2	0.888	125.208
SW4	47Φ12	2	3	0.888	250.416
SW5	47Φ12	2	2	0.888	166.944
SW6	47Φ12	2	1	0.888	83.472
STORY NAME: 4th STORY					
SW1	35Φ10	2.4	1	0.617	51.828
SW2	35Φ12	2.4	1	0.888	74.592
SW3	35Φ12	1.5	2	0.888	93.24
SW4	35Φ12	2	3	0.888	186.48
SW5	35Φ12	2	2	0.888	124.32
SW6	35Φ12	2	1	0.888	62.16
ROOF STORY					
SW3	40Φ12	1.5	1	0.888	53.28
SW4	40Φ12	2	2	0.888	142.08
SW5	40Φ12	2	2	0.888	142.08
SW6	40Φ12	2	1	0.888	71.04
LIFT					
SW7	144Φ18	8.9	1	1.998	2560.64
TOTAL WEIGHT OF Φ 10 FOR SHEAR WALL HORIZONTAL STEEL =239.89 kg					
TOTAL WEIGHT OF Φ 12 FOR SHEAR WALL HORIZONTAL STEEL =2911.548 kg					
TOTAL WEIGHT OF Φ 18 FOR SHEAR WALL HORIZONTAL STEEL =2560.64 kg					
TOTAL COST OF Φ 10 FOR SHEAR WALL HORIZONTAL STEEL=148.252 JOD					
TOTAL COST OF Φ 12 FOR SHEAR WALL HORIZONTAL STEEL =1717.814 JOD					
TOTAL COST OF Φ 18 FOR SHEAR WALL HORIZONTAL STEEL =1510.778 JOD					
VERTICAL STEEL					
STORY NAME: 1st STORY					
SW1	32Φ20	6	1	2.466	473.472
SW2	32Φ20	6	1	2.466	473.472
SW3	20Φ20	6	2	2.466	591.84
SW4	28Φ20	6	3	2.466	1242.864
SW5	28Φ20	6	2	2.466	828.576
SW6	28Φ20	6	1	2.466	414.288
SW7 (LIFT)	20Φ25	6	1	3.854	462.48
SW7 (LIFT)	70Φ20	6	1	2.466	1035.72
STORY NAME:2nd & 3rd STORYES					
SW1	32Φ16	3.5	2	1.579	353.696
SW2	32Φ16	3.5	2	1.579	353.696
SW3	20Φ16	3.5	4	1.579	442.12

SW4	28Φ18	3.5	9	1.998	1762.236
SW5	28Φ20	3.5	4	2.466	966.672
SW6	28Φ18	3.5	2	1.998	391.608
SW7 (LIFT)	20Φ25	3.5	2	3.854	539.56
SW7 (LIFT)	70Φ18	3.5	2	1.998	979.02
STORY NAME: 4th STORY					
SW1	32Φ14	3.5	1	1.208	135.296
SW2	32Φ16	3.5	1	1.579	176.848
SW3	20Φ16	3.5	2	1.579	221.06
SW4	28Φ14	3.5	3	1.208	355.152
SW5	28Φ14	3.5	2	1.208	236.768
SW6	28Φ14	3.5	1	1.208	118.384
SW7 (LIFT)	20Φ25	3.5	1	3.854	269.78
SW7 (LIFT)	70Φ14	3.5	1	1.208	295.96
ROOF STORY					
SW3	20Φ16	4	1	1.579	126.32
SW4	28Φ14	4	2	1.208	270.592
SW5	28Φ14	4	2	1.208	270.592
SW6	28Φ14	4	1	1.208	135.296
SW7 (LIFT)	20Φ25	4	1	3.854	308.32
SW7 (LIFT)	70Φ14	4	1	1.208	338.24
TOTAL WEIGHT OF Φ 14 FOR SHEAR WALL VERTICAL STEEL =2156.27 kg					
TOTAL WEIGHT OF Φ 16 FOR SHEAR WALL VERTICAL STEEL =1673.74 kg					
TOTAL WEIGHT OF Φ 20 FOR SHEAR WALL VERTICAL STEEL =6026.904 kg					
TOTAL WEIGHT OF Φ 25 FOR SHEAR WALL VERTICAL STEEL =1580.14 kg					
TOTAL COST OF Φ 14 FOR SHEAR WALL VERTICAL STEEL =1272.20 JOD					
TOTAL COST OF Φ 16 FOR SHEAR WALL VERTICAL STEEL =987.51 JOD					
TOTAL COST OF Φ 20 FOR SHEAR WALL VERTICAL STEEL =3555.88 JOD					
TOTAL COST OF Φ 25 FOR SHEAR WALL VERTICAL STEEL =948.24 JOD					

9.4 Quantities and Costs of Foundation

In this section, we will calculate the Quantities and Cost of Concrete and Rebar, that used in Foundation.

❈ Quantity and Cost of Concrete:

Table 9.9: Quantity and Cost of Concrete for foundation

TOTAL AREA (m^2)	**THICKNESS**	**Volume of Concrete (m^3)**
633.589	0.5	316.80
VOLUME OF CONCRETE FOR FOUNDATION = 316.80 m^3		
COST OF CONCRETE FOR FOUNDATION = 316.8*53 = 16790.4 JOD		

❈ Quantity and Cost of Rebar:

Table 9.10: Quantity and Cost of Rebar for foundation

(X/Y) Direction	**Length**	**Reinforcement**	**Wt/m(kg/m)**	**Total weight (kg)**
Top (-ve) Reinforcement				
X	23.9	133Φ16	1.579	5019.17
Y	26.51	120Φ16	1.579	5023.12
Bottom (+ve) Reinforcement				
X	23.9	133Φ16	1.579	5019.17
Y	26.51	120Φ16	1.579	5023.12
UNDER THE COLOUMN				
X	1	150Φ22	2.984	447.6
Y	1	150Φ22	2.984	447.6
BEND				
LENGTH (m)	**RINFORCMENT**	**Wt/m(kg/m)**	**Total weight (kg)**	
0.72	948Φ16	1.579	1077.77	
TOTAL WEIGHT OF Φ 16 FOR FOUNDATION =21163kg=21.163ton				
TOTAL WEIGHT OF Φ 22 FOR FOUNDATION =895.2 kg				
TOTAL COST OF Φ 16 FOR FOUNDATION =21.163*590=12487 JOD				
TOTAL COST OF Φ 22 FOR FOUNDATION =0.8952*590=528.2 JOD				

9.5 Quantities and Costs of Stairs

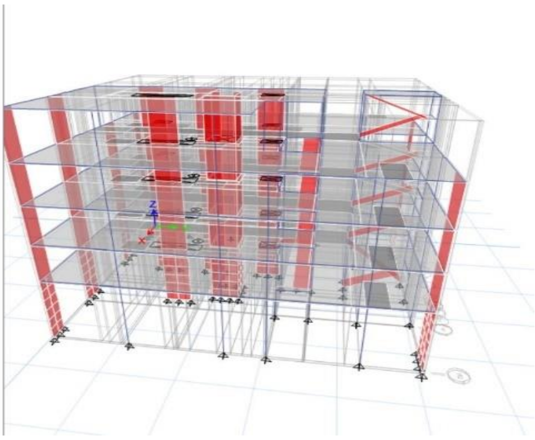


Figure 9.10: staircase modeling using ETABS 1

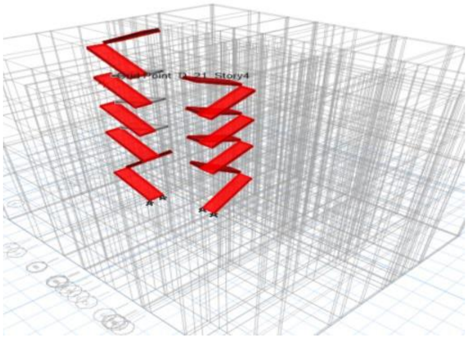


Figure 9.11: staircase modeling using ETABS 2

In this section, we will calculate the Quantities and Cost of Concrete and Rebar, that used in Stairs.

❋ Quantity and Cost of Concrete:

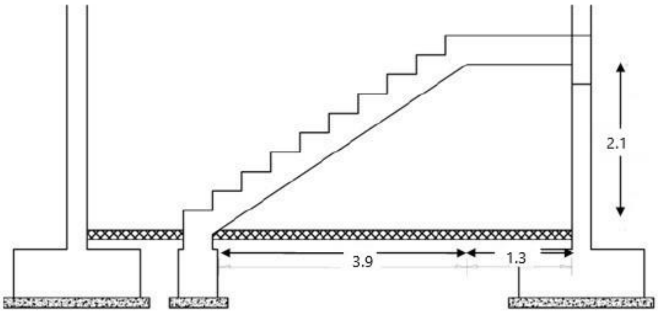


Figure 9.12: roof stair section

Sample of Calculation:

we will take a roof as a sample of calculation

- Waist Length = $\sqrt{3.9^2 + 2.1^2}$ = 4.43m
- Thickness of Waist (we showed detailed of how to find thickness in stair section) = $0.185m$
- Volume of concrete for Roof Waist = b*L*h = 0.185*4.43*1.2 = 0.98 m^3
- Volume of concrete for Typical Waist = b*L*h = 0.185*3.11*1.2 = 0.69 m^3
- Volume of concrete for first story, first flight Waist =b*L*h=0.185*0.45*1.2= 0.1 m^3
- Volume of concrete for First story Waist = b*L*h = 0.185*3.04*1.2 = 0.67 m^3
- Volume of concrete for entrance Waist = b*L*h = 0.185*0.45*2.5 = 0.21 m^3

- Volume of concrete for 1 step = $\frac{1}{2}$ * 0.30*0.15*1.2= 0.027m^3
- volume of concrete for each flight = 11*0.027 = 0.297 m^3
- volume of concrete for each landing = 0.96*1.2*0.185 = 0.213 m^3

Table 9.11: Quantity and Cost of concrete for staircases

NUMBER IN ALL PROJECT		**VOLUME OF ONE ELEMNT (m^3)**	**TOTAL VOLUME (m^3)**
LANDING			
22		0.475	10.45
WAIST			
ROOF	4	0.98	3.92
Typical	12	0.69	8.28
first flight	2	0.1	0.2
First story	4	0.67	2.68
Entrance	1	0.21	0.21
STEPS			
ROOF	54	0.027	1.458
Typical	132	0.027	3.564
First flight	6	0.027	0.162
First story	56	0.027	1.512
Entrance	3	0.08	0.24
TOTAL VOLUME OF CONCRETE FOR STAIRS = 26.912 m^3			
TOTAL COST OF CONCRETE FOR STAIRS = 26.912*52= 1399.424 JOD			

❉ Quantity and Cost of Rebar

Table 9.12: Quantity and Cost of Rebar for staircases

REAINFORCMENT		**LENGTH(m)**	**NUMBER**	**Wt/m(kg/m)**	**Total Weight (kg)**
LANDING					
18Φ16		2.7	22	1.579	1688.27
7Φ16		0.96	22	1.579	233.44
WAIST & STEPS					
DISTRIBUTION REAINFORCMENT					
ROOF	22Φ10	1.2	4	0.617	65.16
Typical	16Φ10	1.2	12	0.617	142.16
First flight	3Φ10	1.2	2	0.617	4.44
First story	15Φ10	1.2	4	0.617	44.42
Entrance	3Φ10	2.5	1	0.617	4.63
ADDITIONAL REAINFORCMENT					
Φ8 **FOR EACH STEP**		1.2	251	0.395	118.97
Φ8 **FOR EACH STEP**		0.9	251*8= 2008	0.395	713.844
MAIN REAINFORCMENT					
ROOF	8Φ16	4.43	4	1.579	223.839
Typical	8Φ16	3.11	12	1.579	471.426
First flight	8Φ16	0.45	2	1.579	11.369
First story	8Φ16	3.04	4	1.579	153.605
Entrance	8Φ16	0.45	1	1.579	5.685

TOTAL WEIGHT OF Φ 16 FOR ALL STAIRCASE = 566.92 kg
TOTAL COST OF Φ 16 FOR ALL STAIRCASE = 0.56692*590 = 334.483 JOD
TOTAL WEIGHT OF Φ 10 FOR ALL STAIRCASE = 514.59 kg
TOTAL COST OF Φ 10 FOR ALL STAIRCASE = 0.51459*618= 318.02 JOD
TOTAL WEIGHT OF Φ 8 FOR ALL STAIRCASE = 654.35 kg
TOTAL COST OF Φ 8 FOR ALL STAIRCASE = 0.65435*610 = 399.154 JOD

9.6 Final Cost and Quantities Estimating

Table 9.13: Final Cost and Quantities

Rebar (Φ)	**Total Weight (ton)**	**TOTAL COST (JOD)**
Φ8	1.32	805.2
Φ10	6.02	3719.94
Φ12	58.68	34640.29
Φ14	3.99	2354.758
Φ16	57.19	33732.173
Φ18	4.77	2814.552
Φ20	4.76	2812.23
Φ22	0.9	528.2
Φ25	1.58	948.084
TOTAL	139.21	**82355.427**
Concrete (F′c)	**Total Volume (m^3)**	**TOTAL COST (JOD)**
28	976.952	50801.504
30	316.8	16790.4
TOTAL	1293.752	**67591.904**
Total Estimate Cost of the Rebar & Concrete of the Project = 149947.331 JOD		

REFERENCES

[1] James K; MacGregor, James G, “Reinforced Concrete: Mechanics and Design”

[2] Jordan National Building Codes

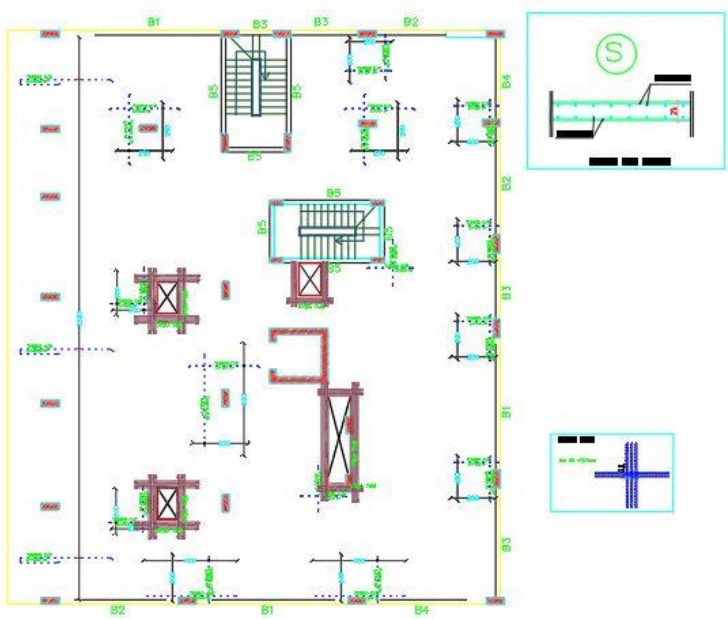
[3] ACI 318-14

[4] Web sites:

a) www.picbasic.org/articles/ultrasonic/ultrasonic_experiments
b) https://www.concrete.org/tools/318buildingcodeportal.aspx.aspx
c) https://www.jea.org.jo/portal/jordanian-national-building-codes/
d) https://www.azimuthbuilders.com/blog/flat-slab-construction
e) https://www.graitec.com/design-of-flat-slab-structures-advance-design/
f) http://www.jcca.org.jo/SiteContent.aspx?id=97
g) https://dailycivil.com/types-rcc-beam/
h) https://www.concretecentre.com/Codes/Eurocode-2/Punching-Shear.aspx
i) https://www.ancon.com.au/products/punching-shear-reinforcement
j) https://civildigital.com/punching-shear-punching-shear-flat-slabs/
k) https://user.eng.umd.edu/~ccfu/ref/AISC_ExamI1&2&3.pdf
l) https://www.steelconstruction.info/Shear_connection_in_composite_bridge_beams
m) https://smdltd.co.uk/shear-studs-and-composite-decking/
n) https://www.phd.eng.br/wp-content/uploads/2015/12/en.1992.1.1.2004.pdf
o) http://www.zuj.edu.jo/portal/hesham-s-ahmad/wp-content/uploads/sites/308/QS-_-Civil-Engineering.pdf
p) https://www.hkis.org.hk/ufiles/QS-costplans2016.pdf
q) https://theconstructor.org/geotechnical/methods-of-raft-footing-design/3747/
r) https://structurepoint.org/pdfs/Reinforced-Concrete-Shear-Wall-Analysis-Design-ACI318-14.pdf

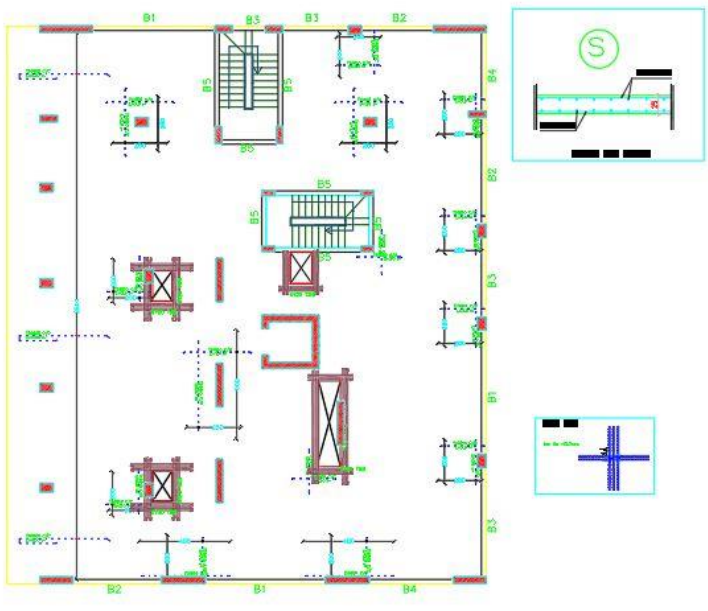
Appendix A : Detailed drawings

Structural Plan



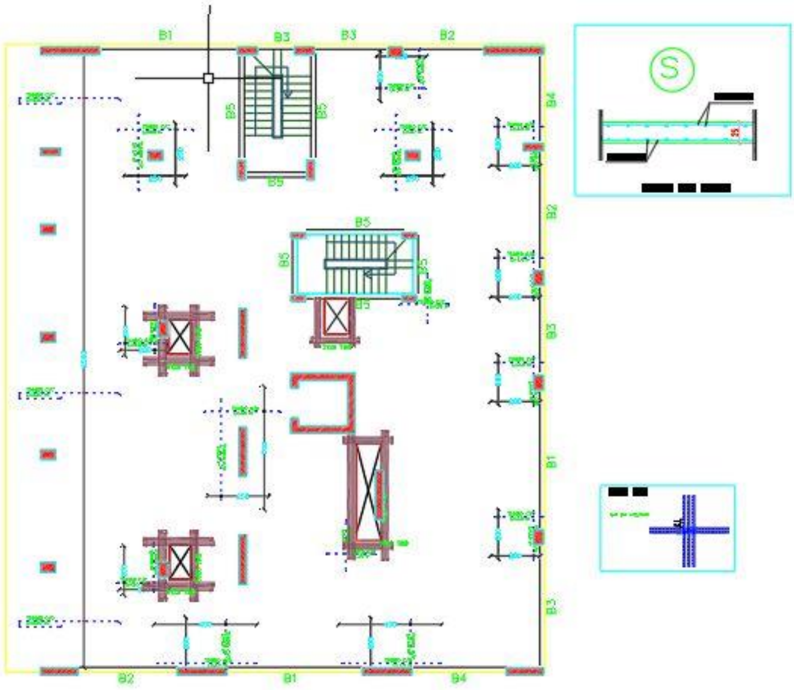
1ST FLOOR PLAN

Figure A.1: Structure Plans 1



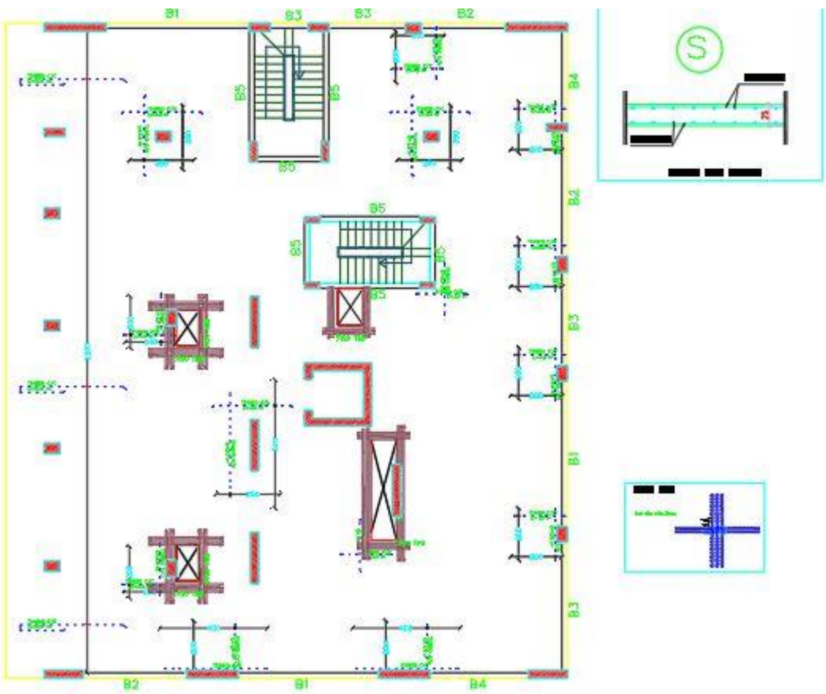
2nd.FLOOR Plan

Figure A.2: Structure Plans 2



3rd.FLOOR Plan

Figure A.3: Structure Plans 3



4th.FLOOR Plan

Figure A.4: Structure Plans 4

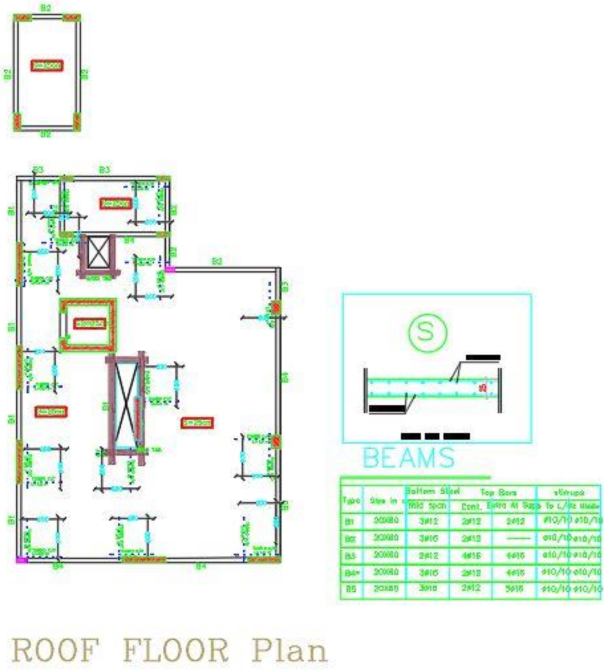


Figure A.5: Structure Plans roof

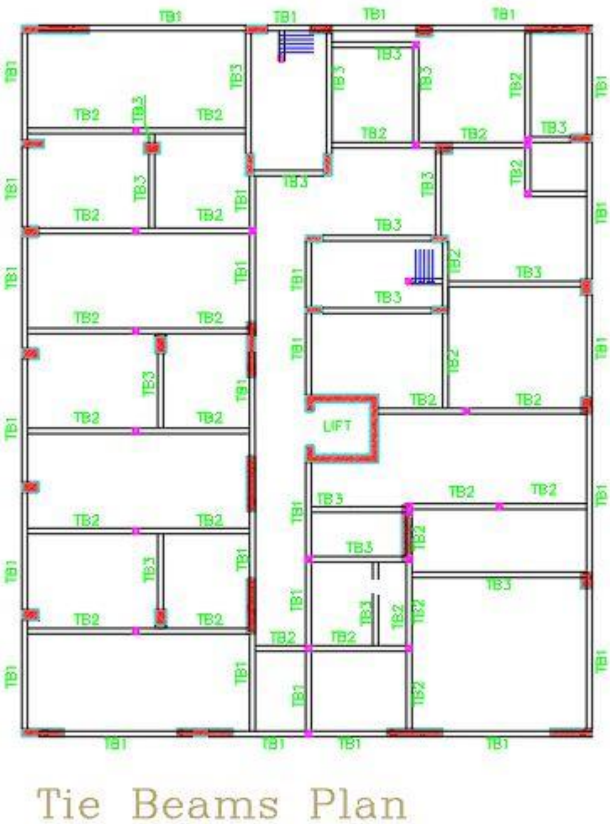


Figure A.6: Structure Plans tie beam

Appendix B : Design Aids

The **Direct Design method** is proposed by concrete institute for the analysis of design of one-way slab as well as two-way slabs. This technique uses various coefficients to determine the positive and negative moment acting on the slab. This method is analogous to the method that is used for the design of beams under loading.

❈ **The conditions that must be fulfilled before applying direct design method are as follows:**

- The shape of the panels must be rectangular,
- The dead load must be at least two times greater than the live load,
- The length of the consecutive span should be not vary more than the one third of the longest span amount, and The loading must be of uniformly distributed nature.

❈ **The direct design method has following limitations**:

- It is necessary to have minimum three continuous spans. In case of dissatisfaction of the conditions, the interior moment will tend to zero.
- Panels used must be rectangular in shape. The value of longer span divided by shorter span should not be more than 2.
- Successive span lengths should not reduce or increase, more than the value given by one third of the longest span.
- Column offset must not be more than 10% of the span.
- The direct design method is only implemented, when the gravity loads case are acting on the slab.