NILE ACADEMY

FACULTY OF ENGINEERING

CIVIL ENGINEERING DEPARTEMENT

REINFORCED CONCRETE PROJECT (2023)

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Appreciation and thanking

Before we start to wright this book, we should express our deepest feelings of thanking and appreciation to our supervisors who helped us throughout this year to complete this project, there are no words that can describe their effort and time to help us in this project; so we thank:

Prof. Dr. / Ahmed M. Yousef

Professor of Concrete Structures & Head of Structural Engineering Department – Faculty of Engineering El-Mansoura University

At the end, we would like to thank our biggest supportive team - our parents - for their support through this year

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1 PROJECT DEFINITION

Reinforced concrete project is analyzing and designing

- residential villa consisting of 5 floors (basement + ground + 3Typical Floors) .
- residential towers consisting of 15 floors (basement + ground + 13 Typical Floors) .
- elevated water tank based on a circular concrete wall, a square hall 32 * 32 meters.

1.1 THE PROBLEM

Design This Projects by professionally

1.2 STUDY OBJECTIVES

The objectives of the project are that the design is safe and economic.

1.3 EXISTING SOLUTIONS

The solutions included listening through lectures and researching the university library to find out how to design these projects.

1.4 DESIGN CONSTRAINTS

The primary constrains faced during our research work are classified into these categories:

1.4.1 Economic

High prices for printing engineering Drawing.
Exorbitant software prices

1.4.2 Environmental

NO Environmental CONSTRAINTS.

1.4.3 Sustainability

NO Sustainability CONSTRAINTS.

1.4.4 Ethical

NO Ethical CONSTRAINTS.

1.4.5 Health and Safety

covid 19 pandemic.

1.4.6 Social and Political

NO Social and Political CONSTRAINTS.

1.4.7 Development

NO Development CONSTRAINTS.

2 CUSTOMER NEEDS AND BACKGROUND

The designs must meet the needs of customers according to the uses of this facility and its importance, with attention to safety and economy.

3 GENERATED CONCEPTS

We encountered some problems in some units of the project. We used the lecture of the doctor and the assistant teacher's assistance, in addition to researching the academic and professional offices .

FINAL CONCEPT

It was reached to know how to design the different units in the project using various engineering programs such as (SAP program - SAFE program - ETABS program - AutoCAD program) and using the Egyptian code for the implementation of concrete structures, taking into account the safety and economic factors.

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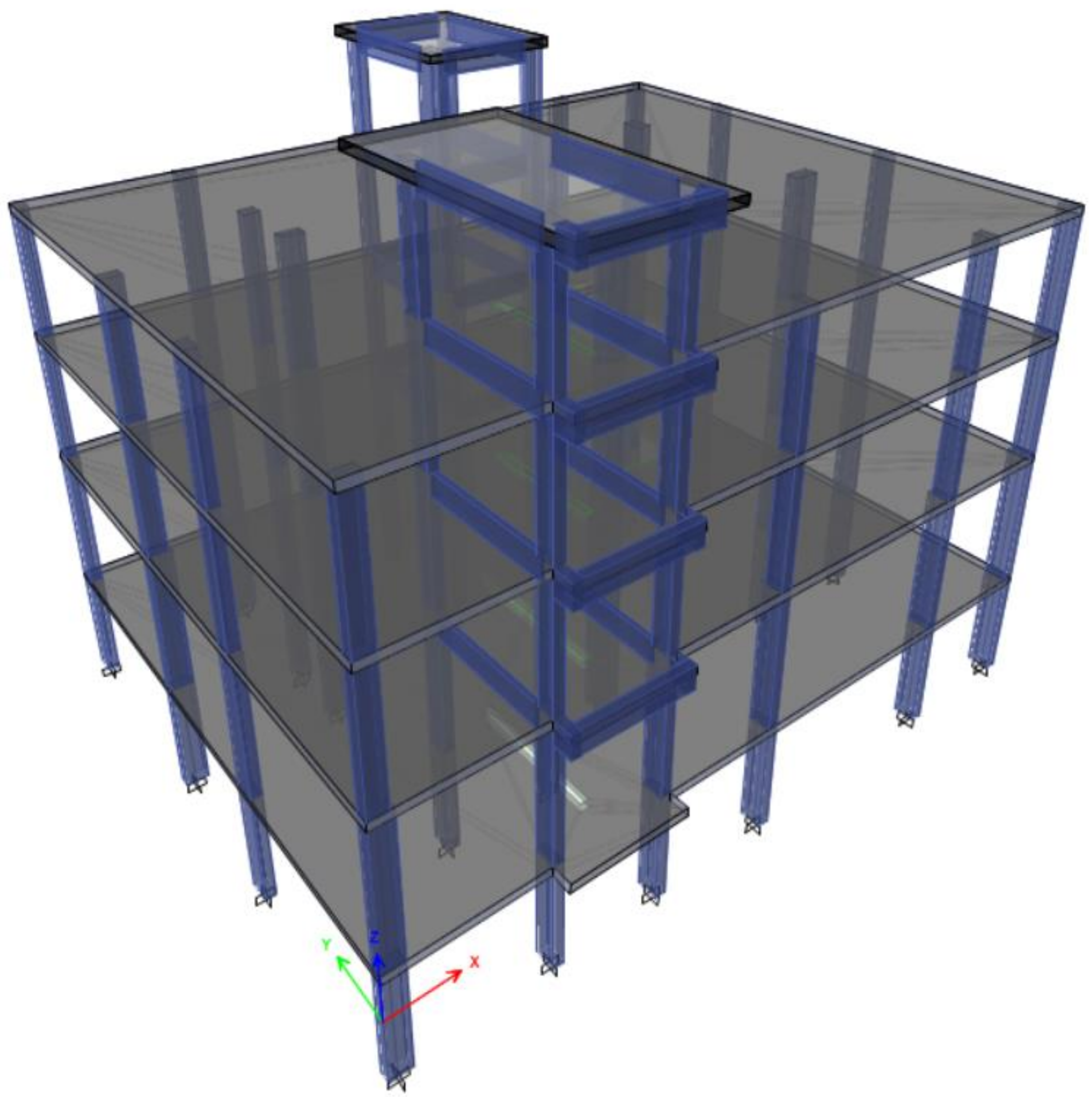
Unit (1)
Villa Project



1.1 INTRODUCTION

1.1.1 Villa Consists of:

- Beasment Floor of (3.00) m height
- Ground Floor (3.80) m height
- First Floor (3.40) m height
- Second Floor of (3.30) m height
- ROOF (3.00) m height

1.1.2 Material Properties Used:

- F_{cu} = 250 kg/cm^2
- $F_{y(main\ steel)}$=3500 kg/cm^2
- $F_{y(stirrups)}$=2400 kg/cm^2
- Weight of used brick =1500 kg/m^3
- Bearing Capacity of Soil = 1.5 kg/cm^2

1.1.3 Cover Thickness

- Slabs Cover = 2 cm
- Beams Cover = 2.5 cm
- Columns Cover = 2.5 cm
- Foundations Cover = 7 cm
- Stairs Cover = 2 cm
- Semell Cover = 2.5 cm

1.1.4 Loads Used:

- L.L= According to every Floor
- Cover = 0.18 ton
  - 0.05 * 1.5 رمل تسوية بسمك 5 سم
  - 0.01 * 2.1 مونة أسمنتيه بسمك 2 سم
  - 0.02 * 2.1 بلاط سيراميك بسمك 2 سم
  - 0.02 * 2.1 محارة أسفل البلاطة بسمك 1 سم
- Wall = According to every Floor
- D.L = Own weight + Covering Material + Wall Load

1.1.5 Design Method:

- Ultimate limit state design

1.1.6 Computer Programs Used in Analysis:

- (Etabs + Safe + SAP2000 + Excel)

1.1.7 Design Code:

Egyptian code of practice 2020

1.2 DESIGN OF SLABS:

1.2.1 Basement Slab: (Flat Slab System)

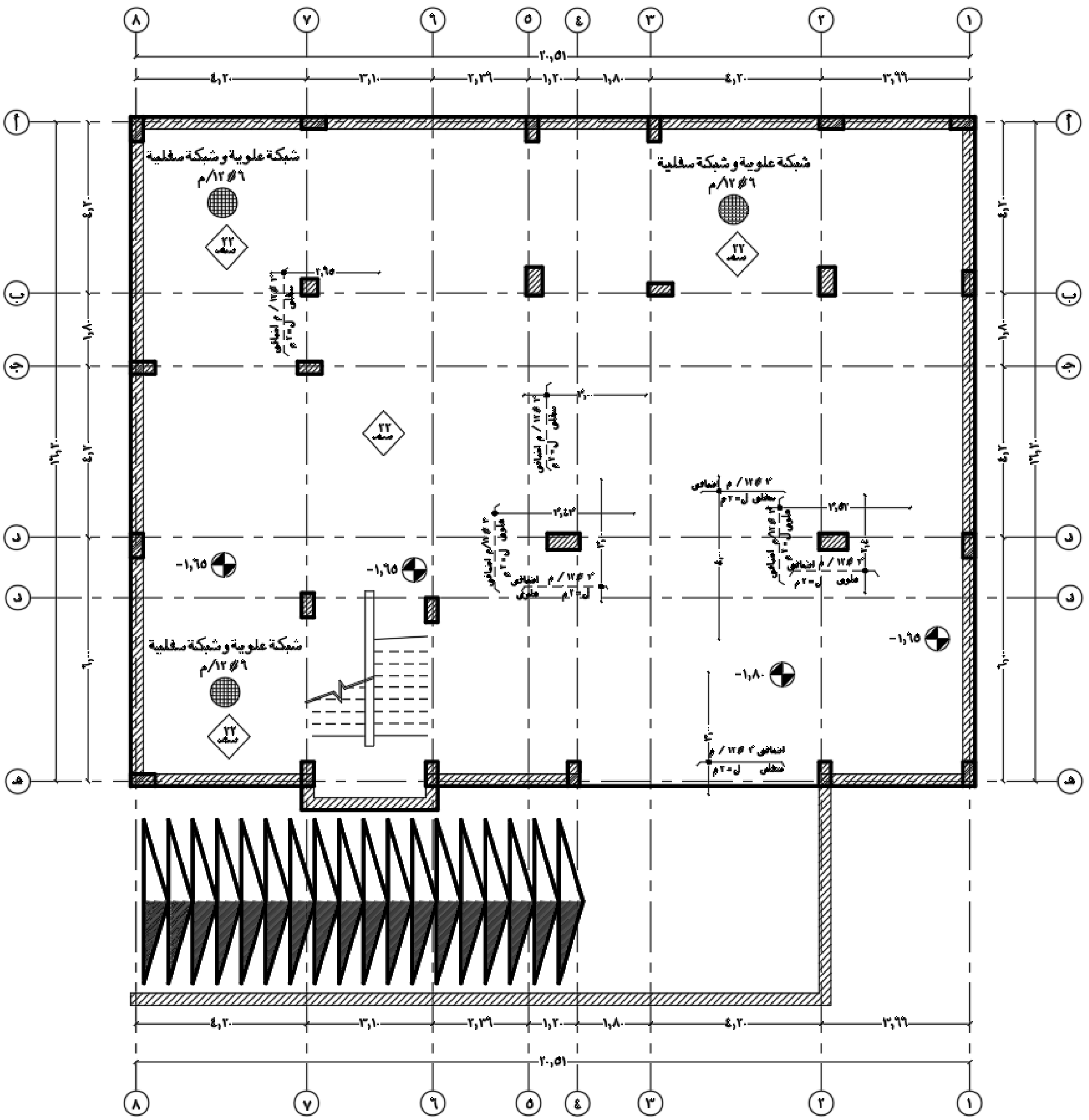


Figure 1.1 Statical System of GROUND FLOOR

- Slab Thickness = 22 cm
- Own weight = 0.22* 2.5 = 0.55 t/m^2.
- Covering = 200 kg/m^2 = 0.20 t/m^2.
- Live load = 300 kg/m^2 = 0.3 t/m^2 .
- Wall load = 150 kg/m^2 = 0.15 t/m^2.

Solving This flat slab By Using CSI Safe program:

- D.L = O.W +W_{wall} + Covering material
  = 0.55 + 0.15 + 0.2 = 0.9 t/m^2
- L.L = 300 kg/cm^2 = 0.3 t/m^2
- W_u = 1.4 D.L + 1.6 L.L = 1.74 t/m^2

For ultimate design: -

- $As = \left[\frac{Mu}{Fy*J*d}\right]$
- $M_u = As * F_y * J * d = 6*\left(\frac{\pi*(1.2)^2}{4}\right) * 3500 * 0.826 * 20 * (10)^{-5}$
- M(r)=3.92 m ⇛ Use 6 ∯ 12 / m in each Direction
- Additional RFT (3 ∯ 12 /m) upper and lower

In X-Direction: (Lower)

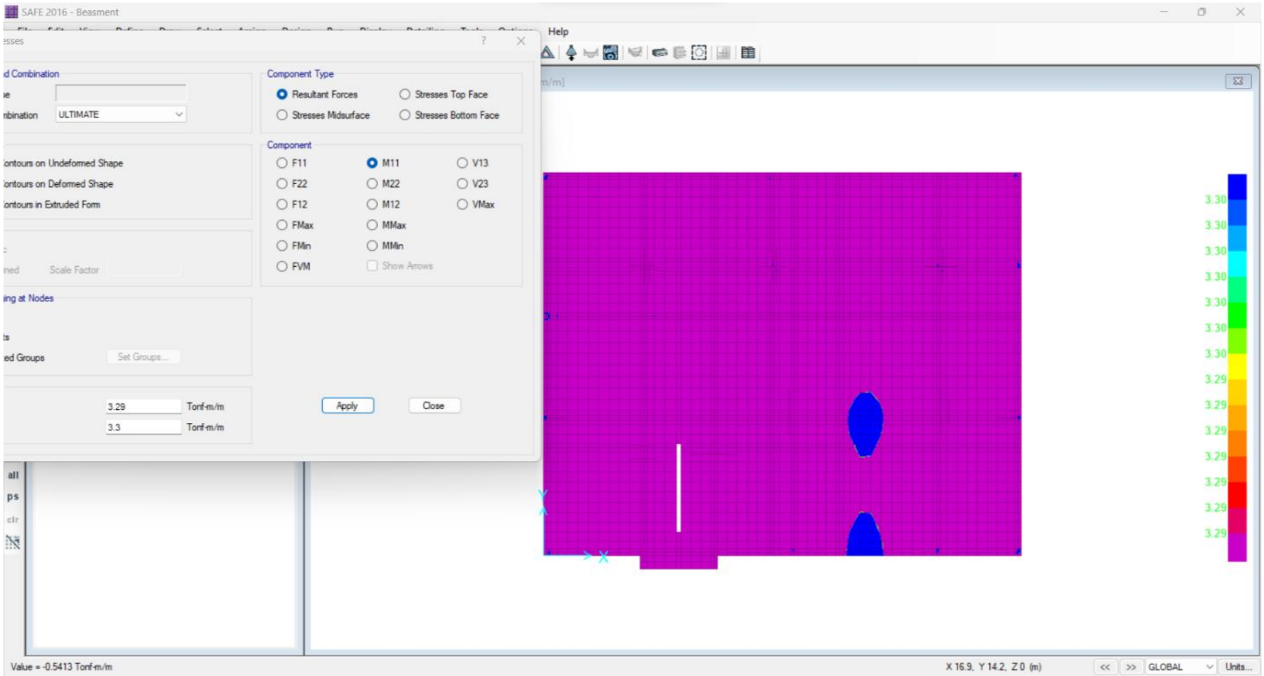


Figure 1.2 Additional Reinforcement in X-Direction (Lower)

In X-Direction: (Upper)

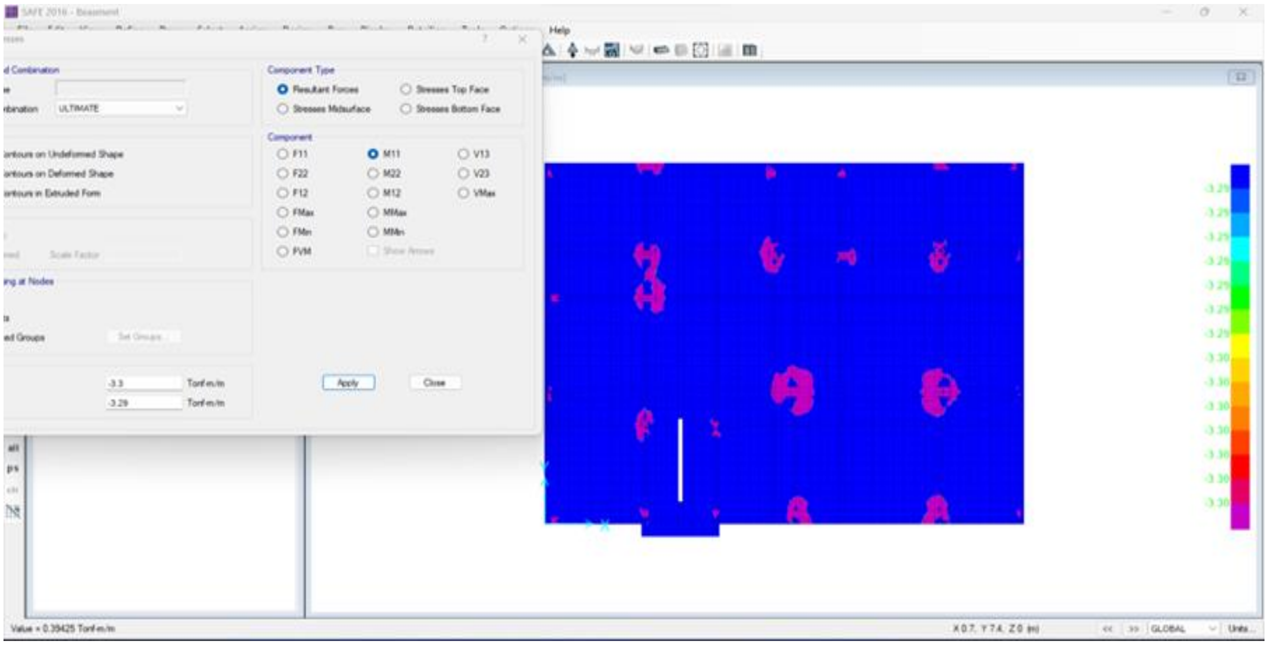


Figure 1.3 Additional Reinforcement in X-Direction (Upper)

In Y-Direction (Lower):

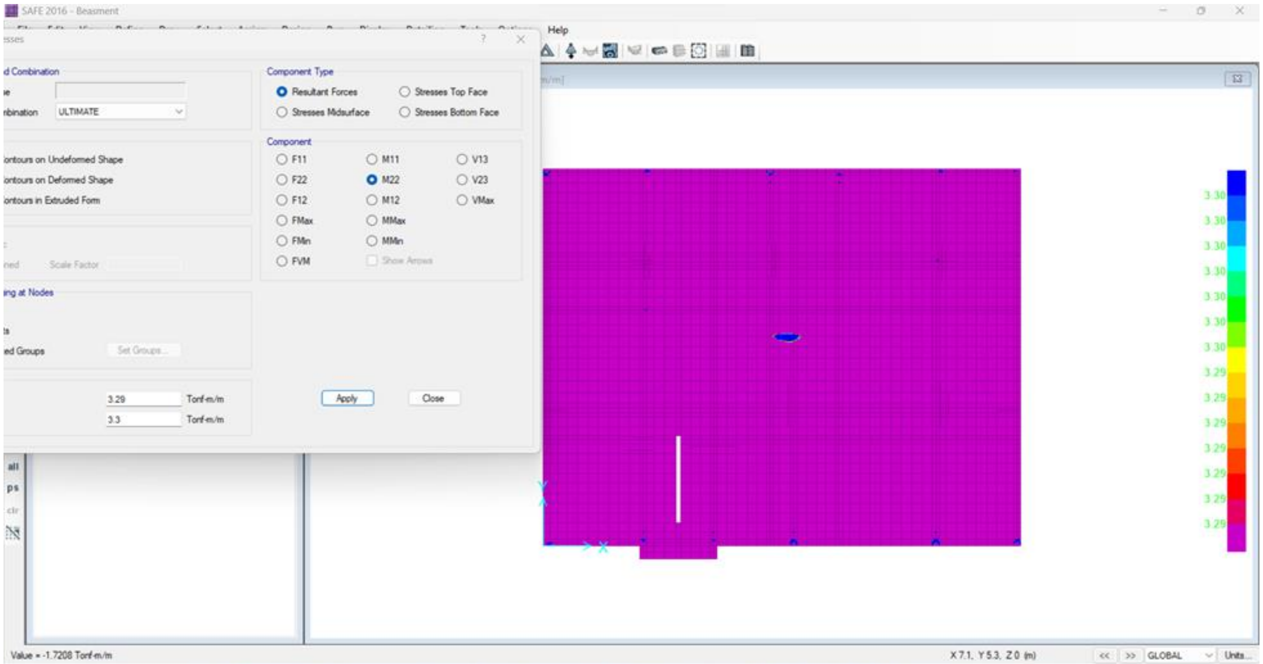


Figure 1.4 Additional Reinforcement in Y-Direction (Lower)

In Y-Direction (Upper):

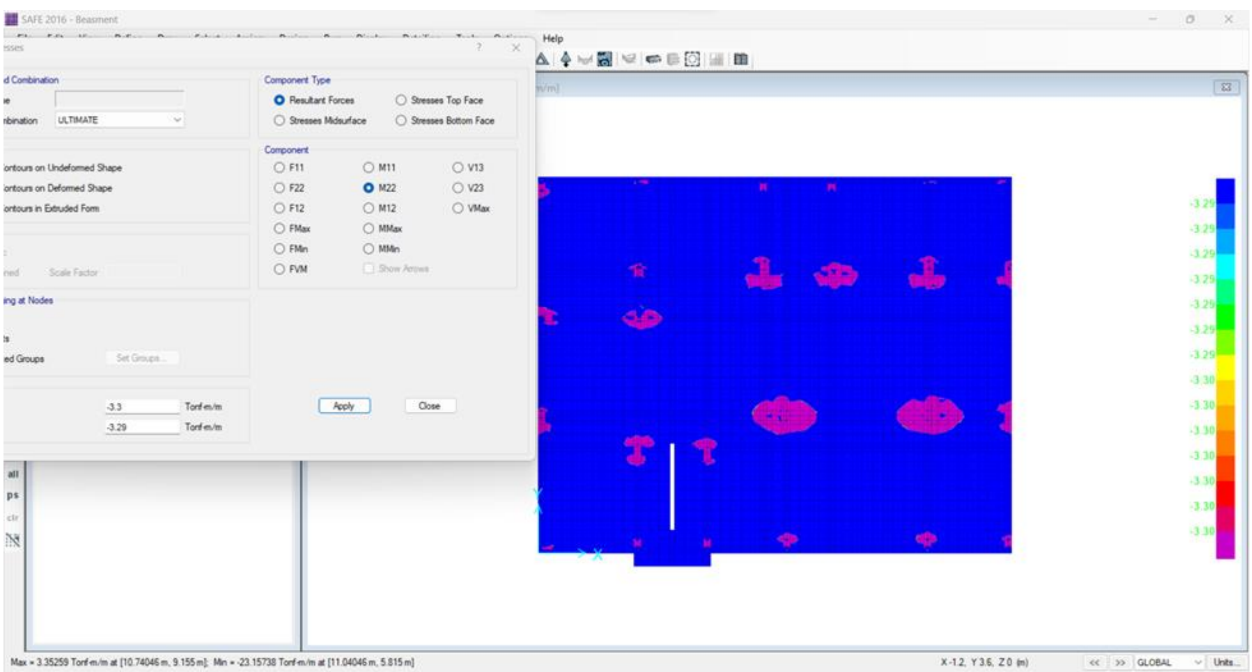


Figure 1.5 Additional Reinforcement in Y-Direction (Upper)

1.2.1.1 Check forAll Loads Deflection:

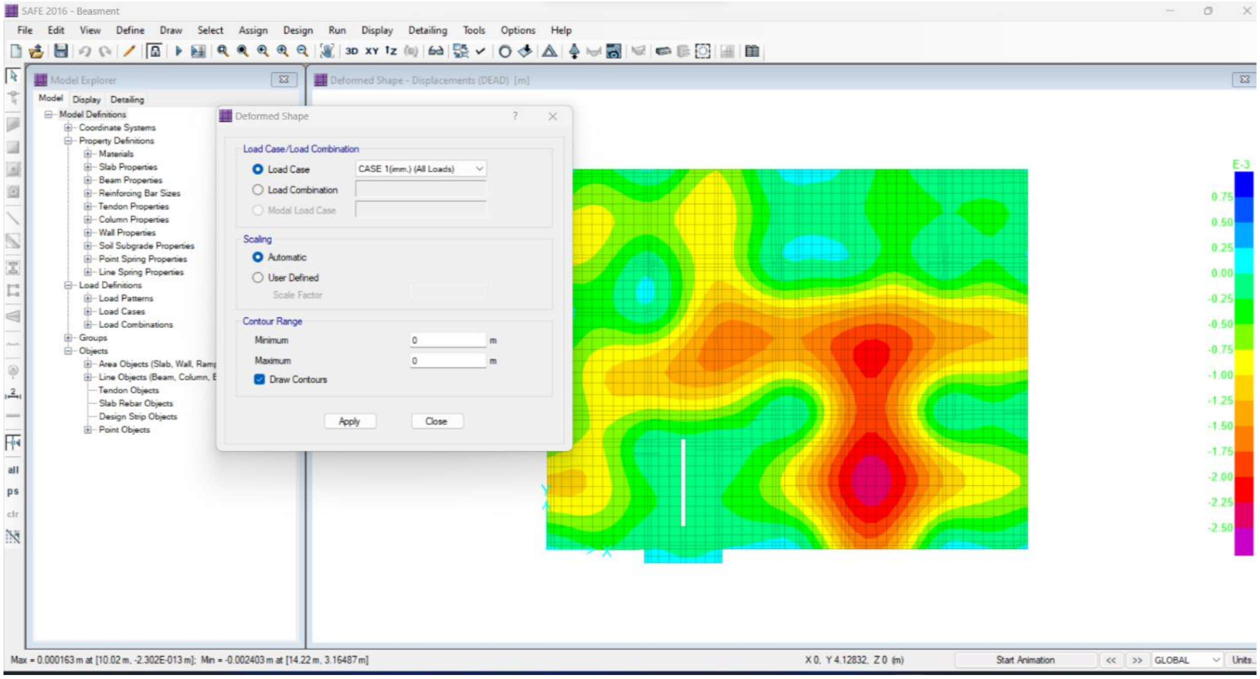


Figure 1.6 Deflection Due to All loads

- From Code Check = L/250
- Span for Check = 6.4m
- Allowable Deflection = 0.0256 m
- Maximum Deflection = 0.0024 m

1.2.1.2 Check for Long Term Deflection

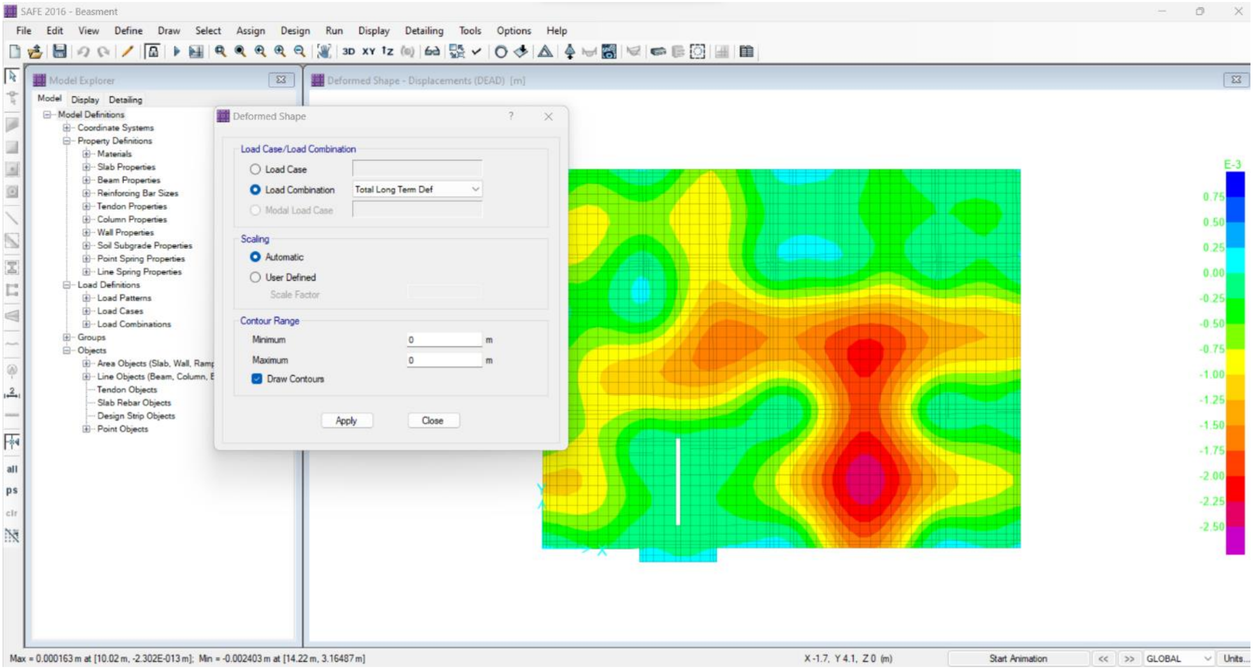


Figure 1.7 Total Long Term Deflection

- From Code Check = L/250
- Span for Check = 6.4 m
- Allowable Deflection = 0.0256 m
- Maximum Deflection = 0.002403 m

1.2.1.3 (Total Dead Loads)

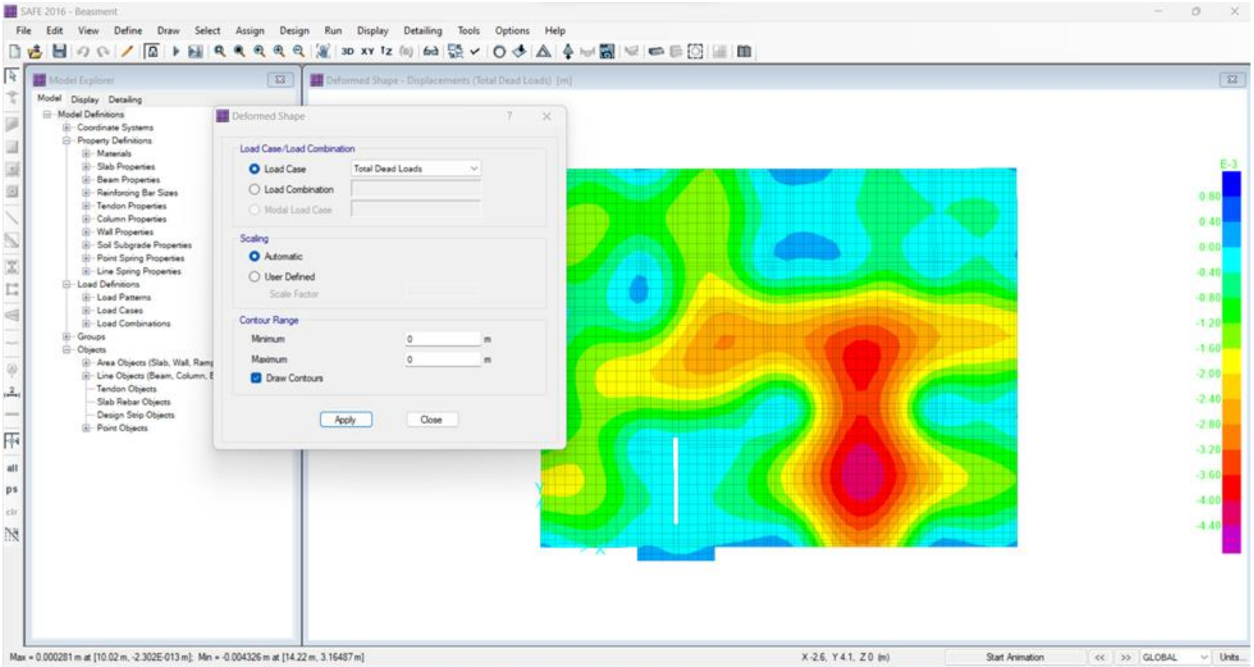


Figure 1.8 Deflection Due to Dead

- From Code Check = L/250
- Span for Check = 6.4 m
- Allowable Deflection = 0.0256 m
- Maximum Deflection = 0.00281 m

1.2.1.3 (Statical System of Ground)

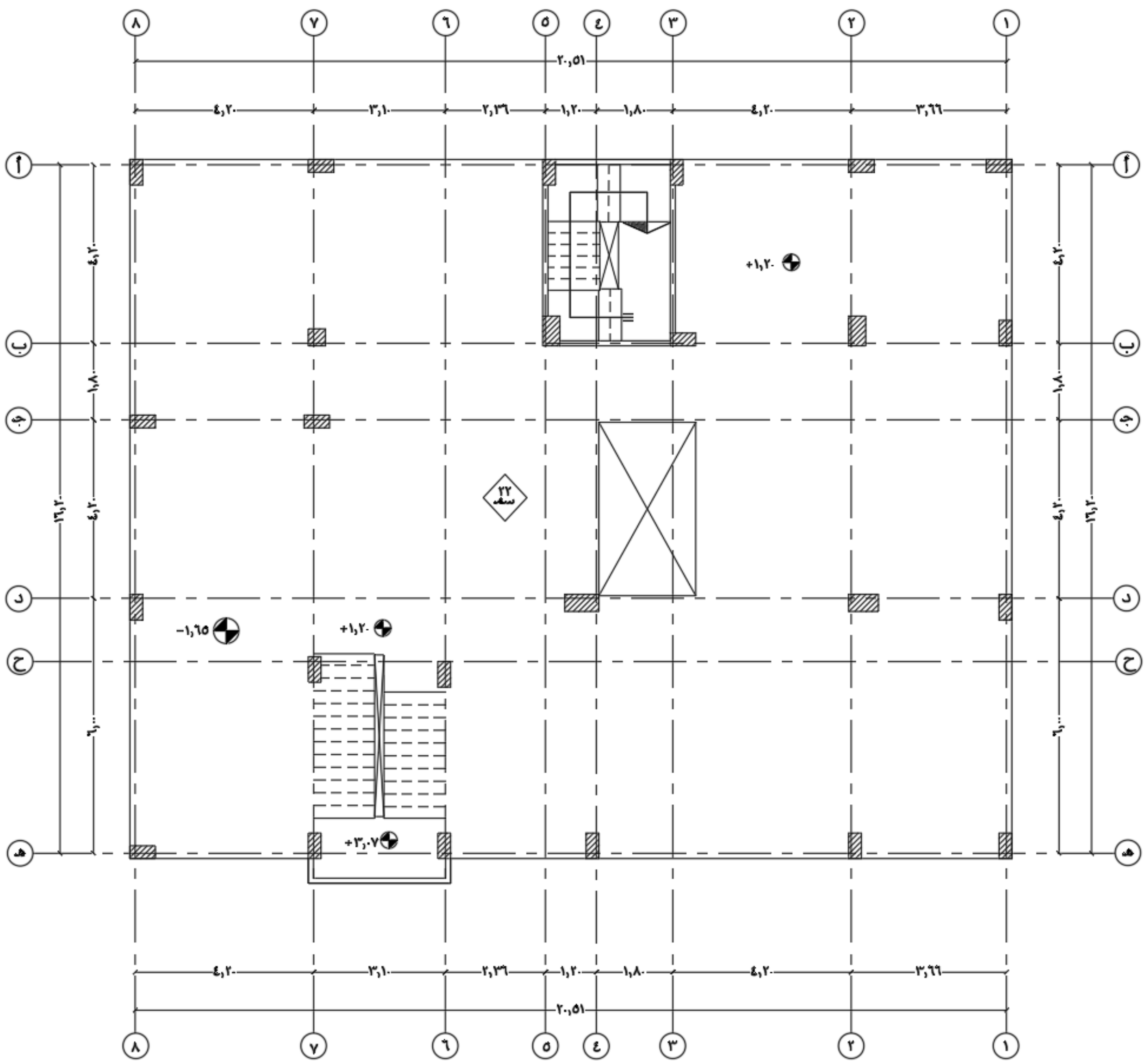


Figure 1.9 Statical System of Ground

- Slab Thickness = 22 cm
- Own weight = 0.22* 2.5= 0.55 t/m^2.
- Live load = 300 kg/m^2 = 0.3 t/m^2
- Wall load = 150 kg/m^2 = 0.15 t/m^2.

Solving This flat slab By Using CSI Safe program:

- D.L = O.W +W_{wall} + Covering material
  =0.55+0.15+0.2=0.9 t/m^2
- L.L = 300 kg/cm^2 = 0.3 t/m^2
- W_u = 1.4 D.L + 1.6 L.L = 1.4*0.9+1.6*0.3 = 1.74 t/m^2

For ultimate design: -

- $As = \left[\frac{Mu}{Fy*J*d}\right]$
- $M_u = As * F_y * J * d = 6*\left(\frac{\pi*(1.2)^2}{4}\right)*3500*0.826*20*(10)^{-5}$
- M(r)= 3.92 t.m ⇛ Use 6 ∯ 12 / m in each Direction
- Additional RFT (3 ∯ 12 /m) & (3 ∯12 / m) upper and lower

In X-Direction: (Lower)

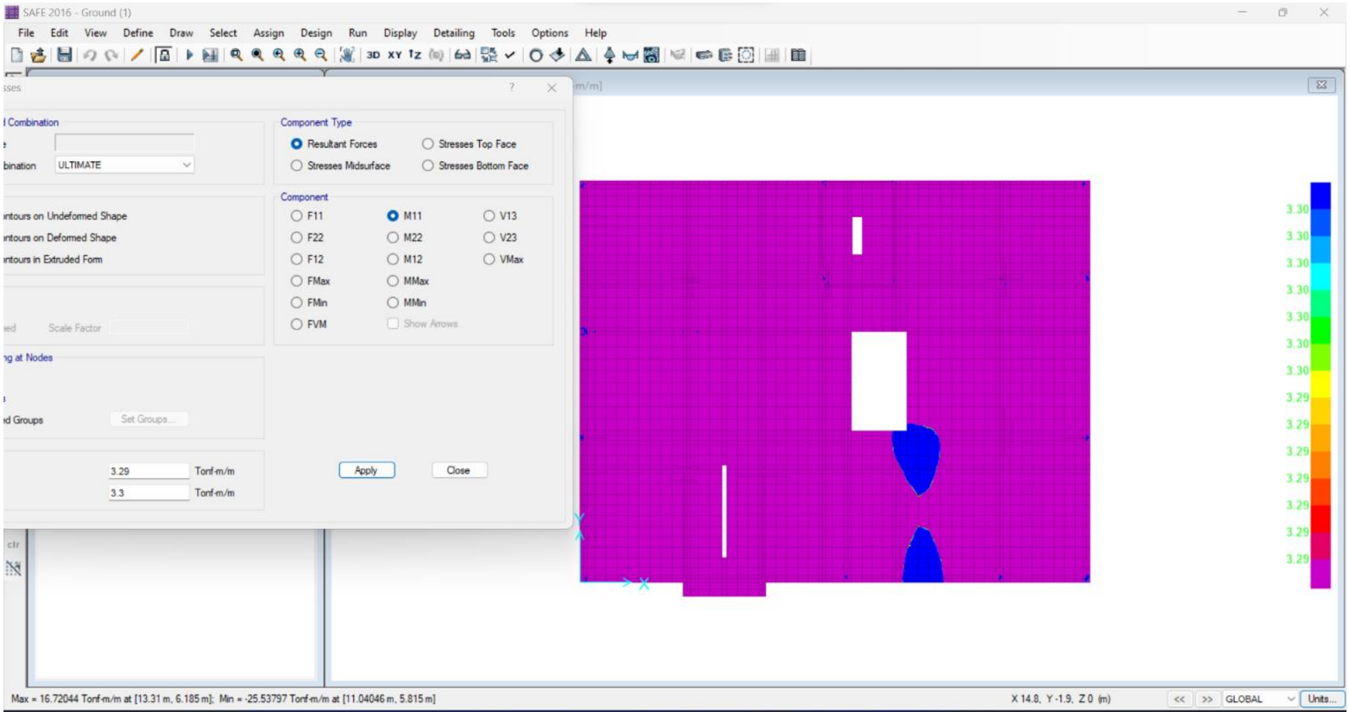


Figure 1.10 Additional Reinforcement in X-Direction (Lower)

In X-Direction: (Upper)

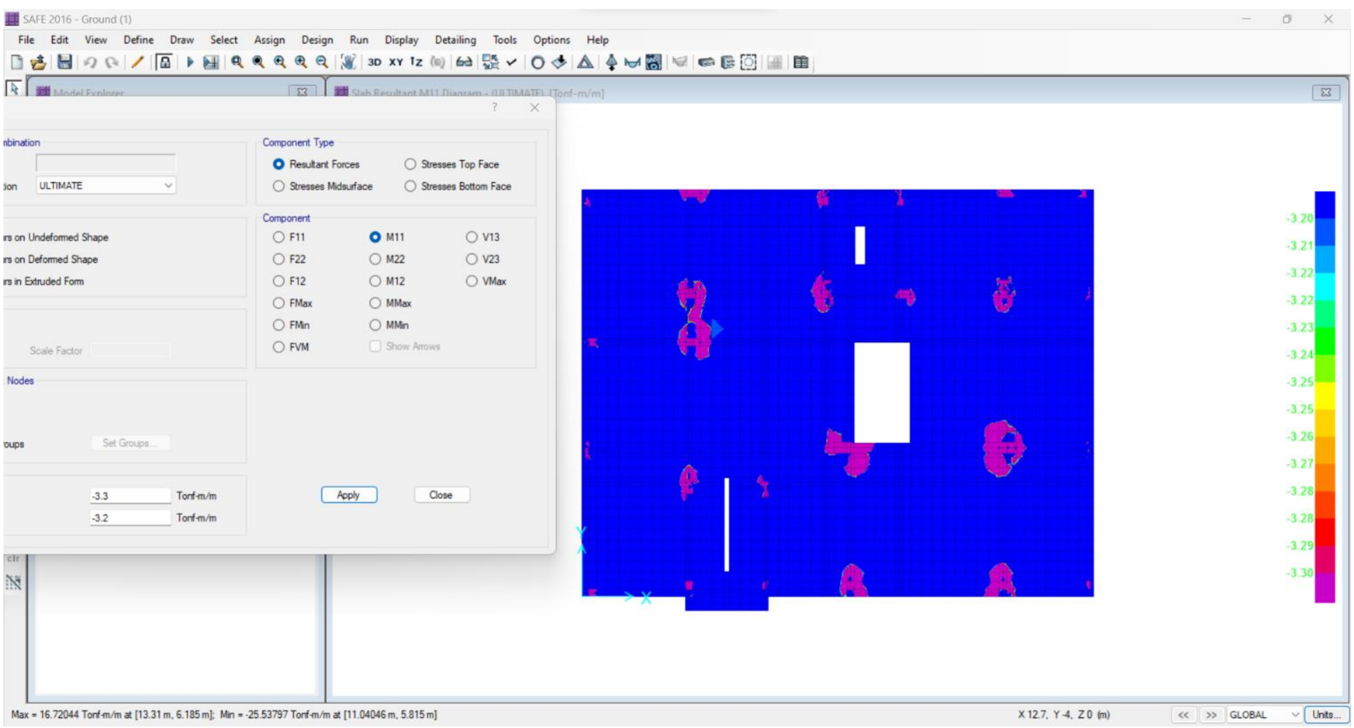


Figure 1.11 Additional Reinforcement in X-Direction (Upper)

In Y-Direction: (Lower)

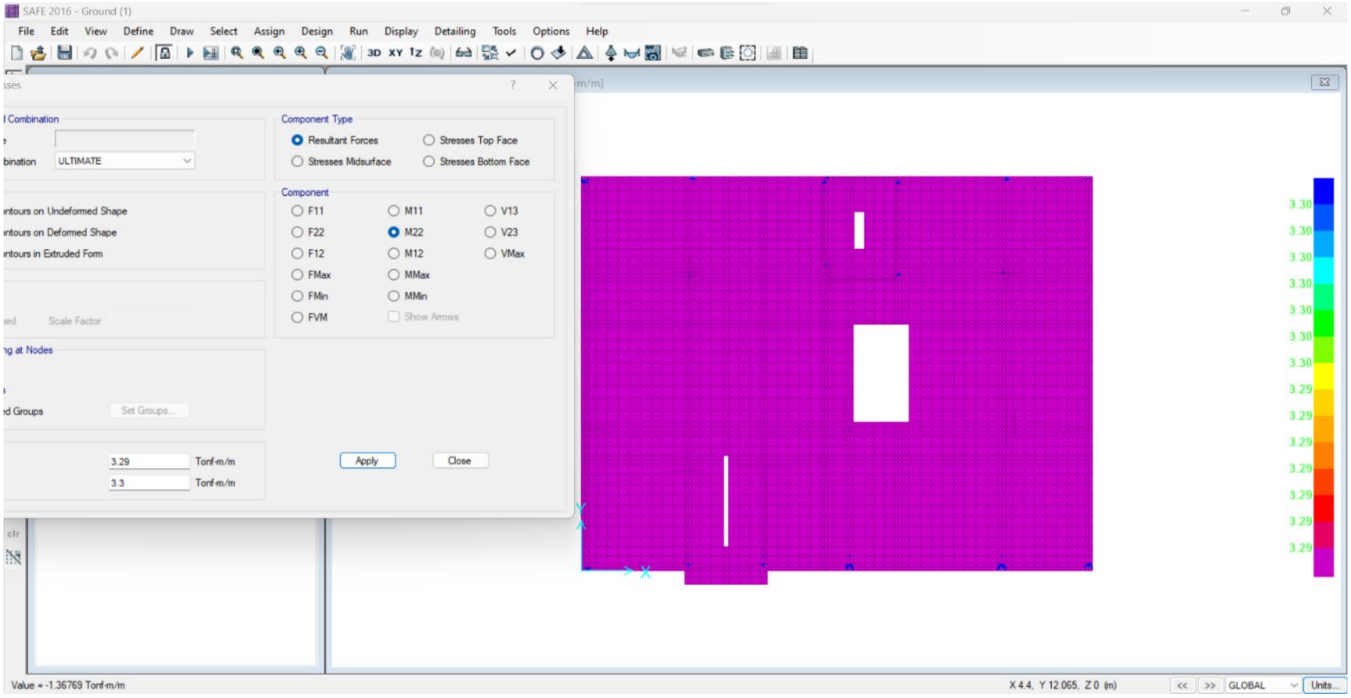


Figure 1.12 Additional Reinforcement in Y-Direction (Lower)

In Y-Direction: (Upper)

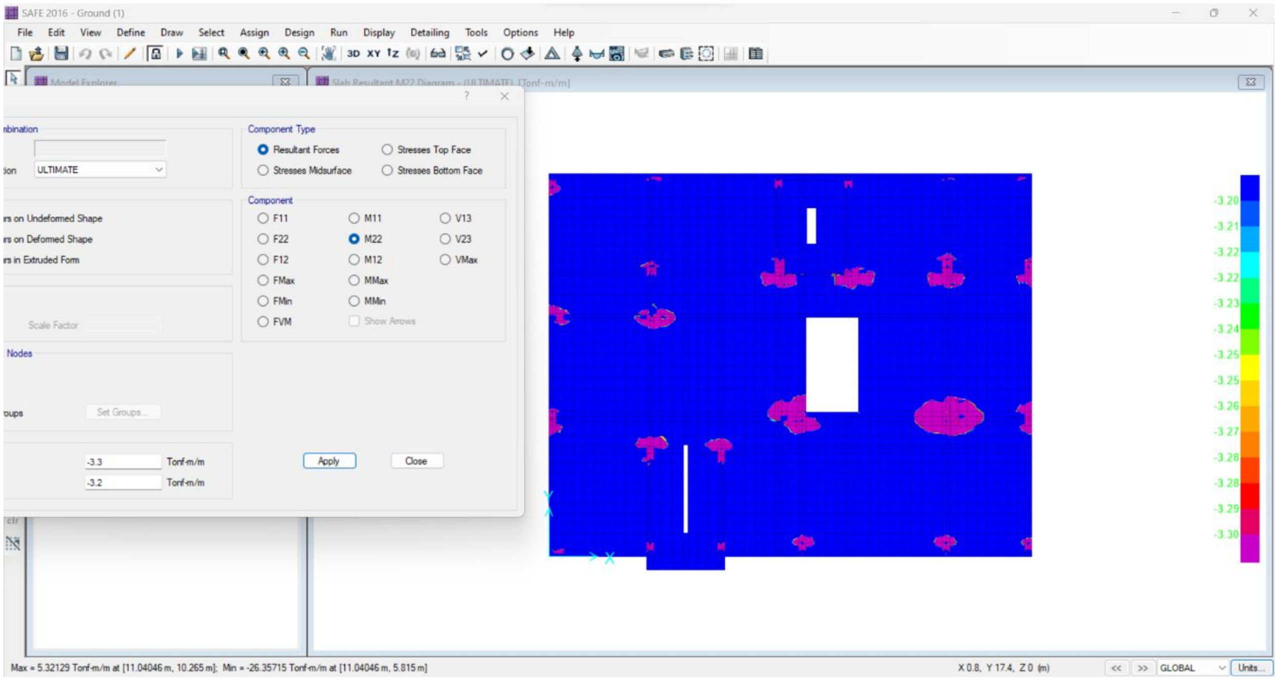


Figure 1.13 Additional Reinforcement in Y-Direction (Upper)

1.2.2.1 Check for All Loads Deflection:

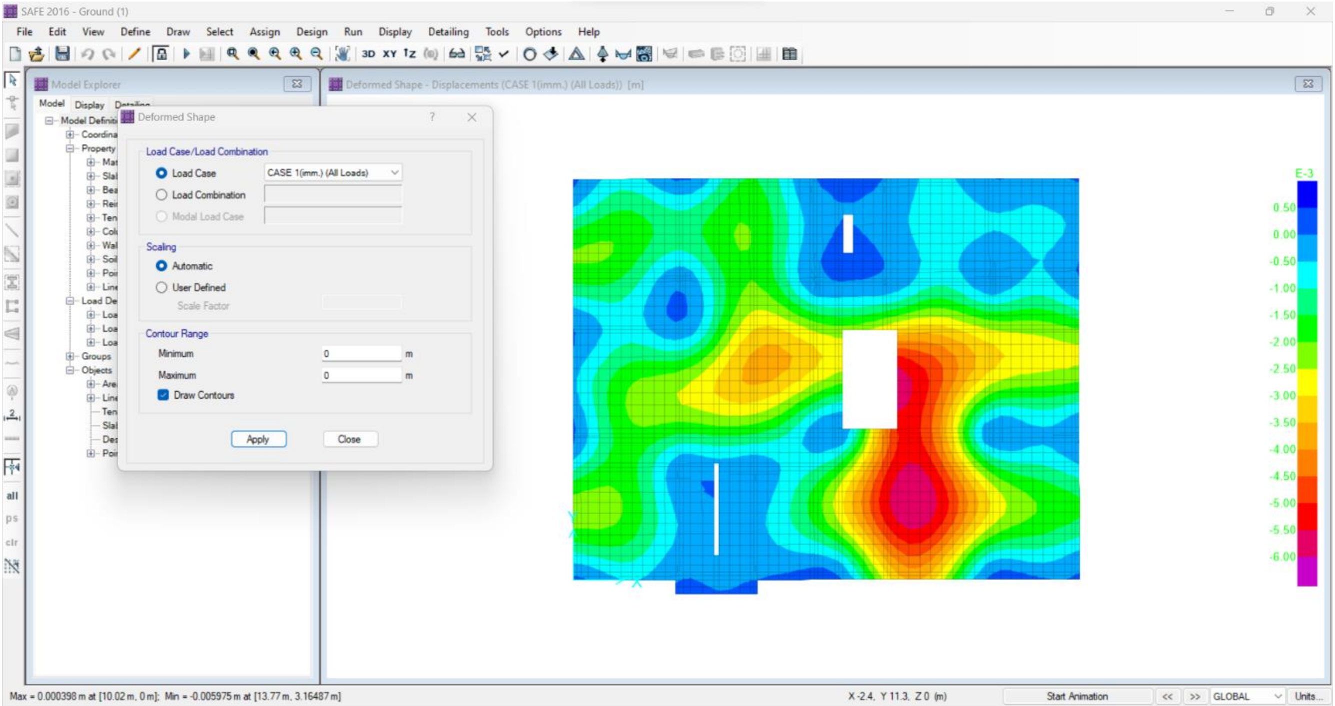


Figure 1.14 All Loads Deflection

- From Code Check = L/360
- Span for Check = 6.7 m
- Allowable Deflection = 0.02667 m
- Maximum Deflection = 0.00398 m

1.2.2.2 Check for Long Term Deflection:

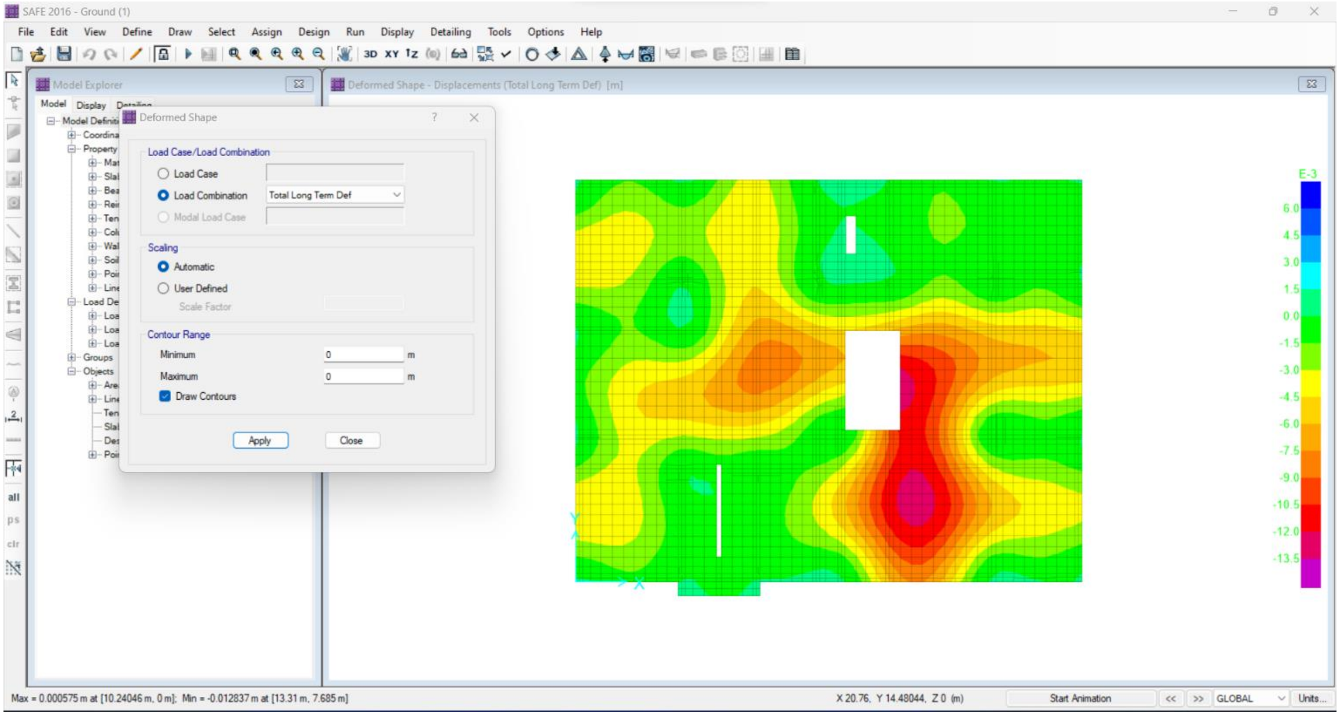


Figure 1.15 Long Term Deflection

- From Code Check = L/250
- Span for Check = 6.5 m
- Allowable Deflection = 0.0384 m
- Maximum Deflection = 0.000575 m

1.2.2.2 Check for Long Term Deflection:

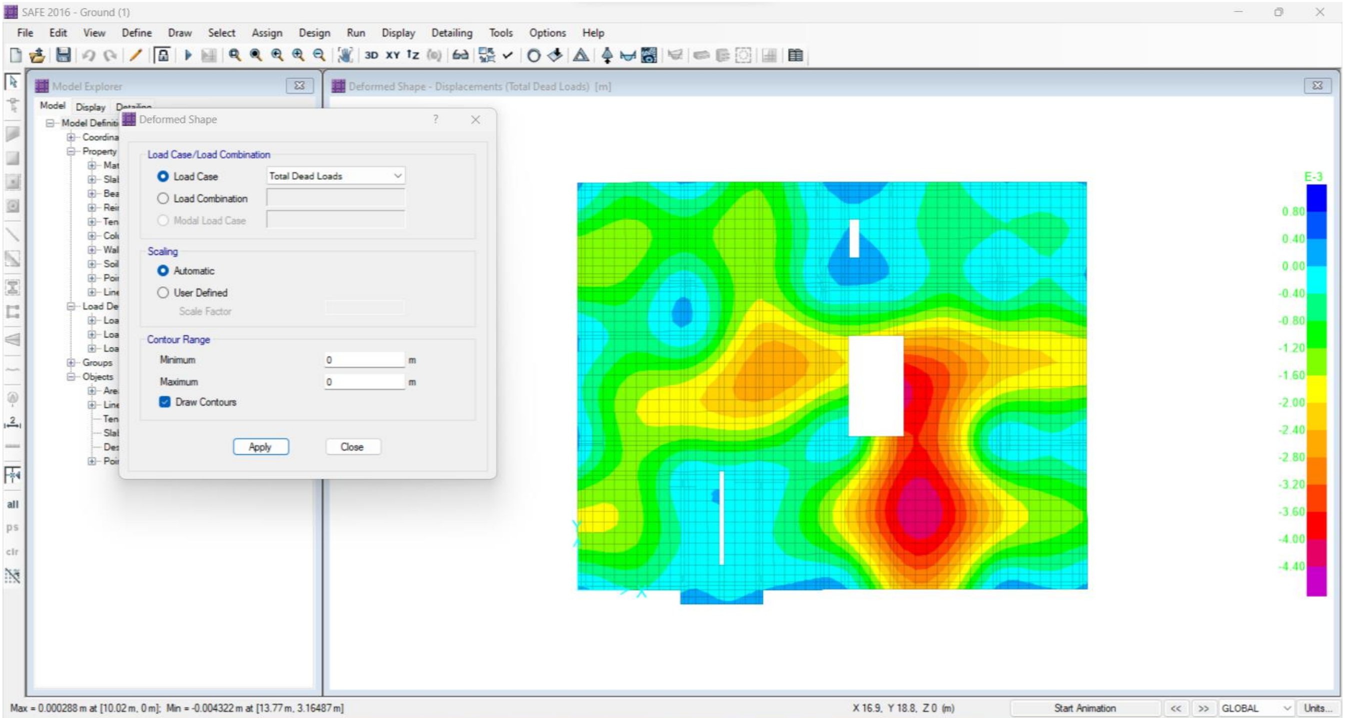


Figure 1.16 Long Term Deflection

- From Code Check = L/250
- Span for Check = 6.5 m
- Allowable Deflection = 0.0384 m
- Maximum Deflection = 0.000288 m

1.2.2.1 First and Second Slab:(Flat Slab System)

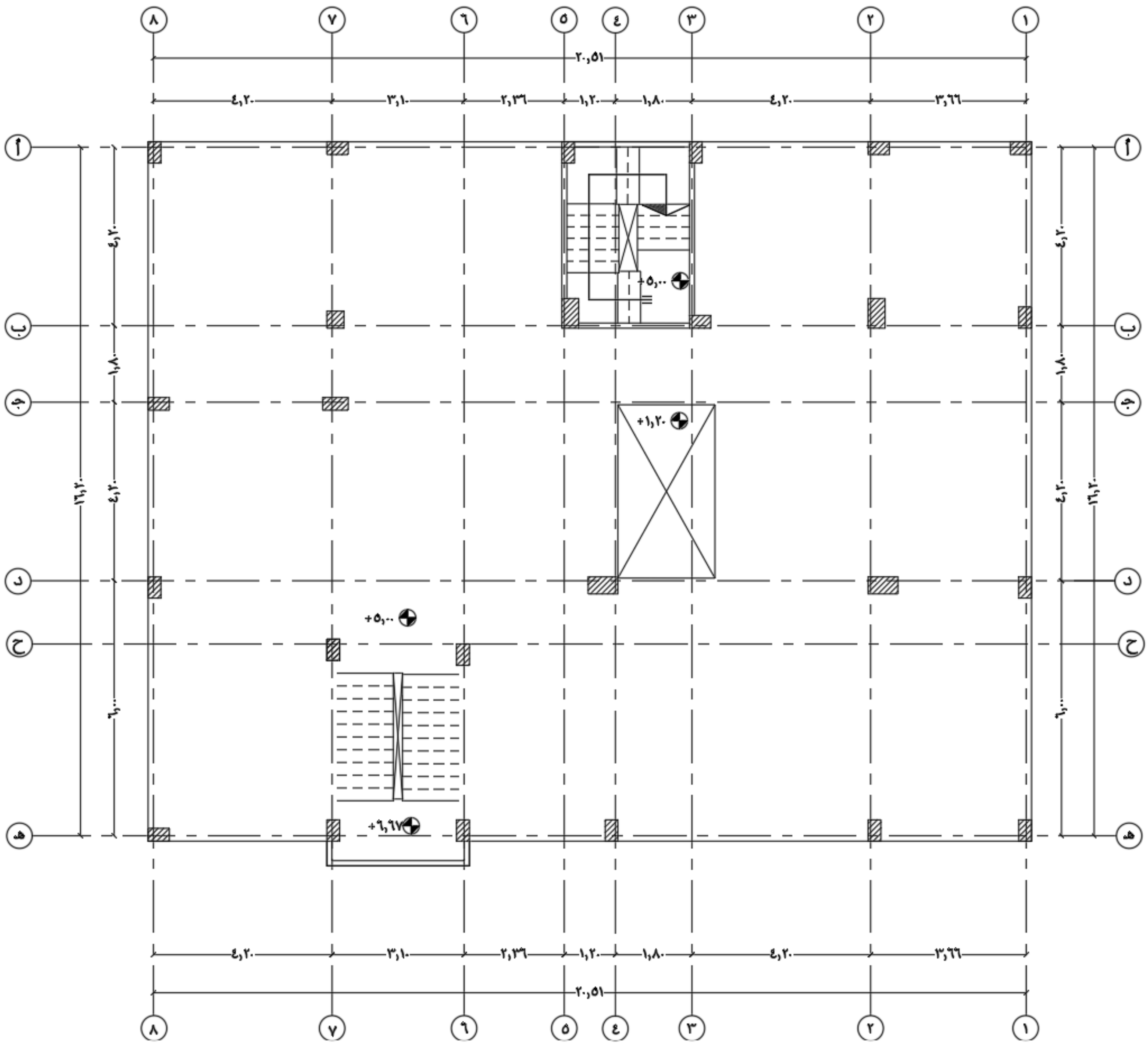


Figure 1.17 Statical System of First and Second

- Slab Thickness = 22 cm
- Own weight = 0.22* 2.5= 0.55 t/m^2.
- Live load = 300 kg/m^2 = 0.3 t/m^2
- Wall load = 150 kg/m^2 = 0.15 t/m^2.

Solving This flat slab By Using CSI Safe program:

- D.L = O.W + W_{wall} + Covering material
  =0.55+0.15+0.2=0.9 t/m^2
- L.L = 300 kg/cm^2 = 0.3 t/m^2
- W_u = 1.4 D.L + 1.6 L.L = 1.4*0.9+1.6*0.3 = 1.74 t/m^2

For ultimate design: -

- $As = \left[\frac{Mu}{Fy*J*d}\right]$
- $M_u = As * F_y * J * d = 6*\left(\frac{\pi*(1.2)^2}{4}\right)*3500*0.826*20*(10)^{-5}$
- M(r)= 3.92 t.m ⇛ Use 6 ∯ 12 / m in each Direction
- Additional RFT (3 ∯ 12 /m) & (3 ∯12 / m) upper and lower

In X-Direction: (Lower)

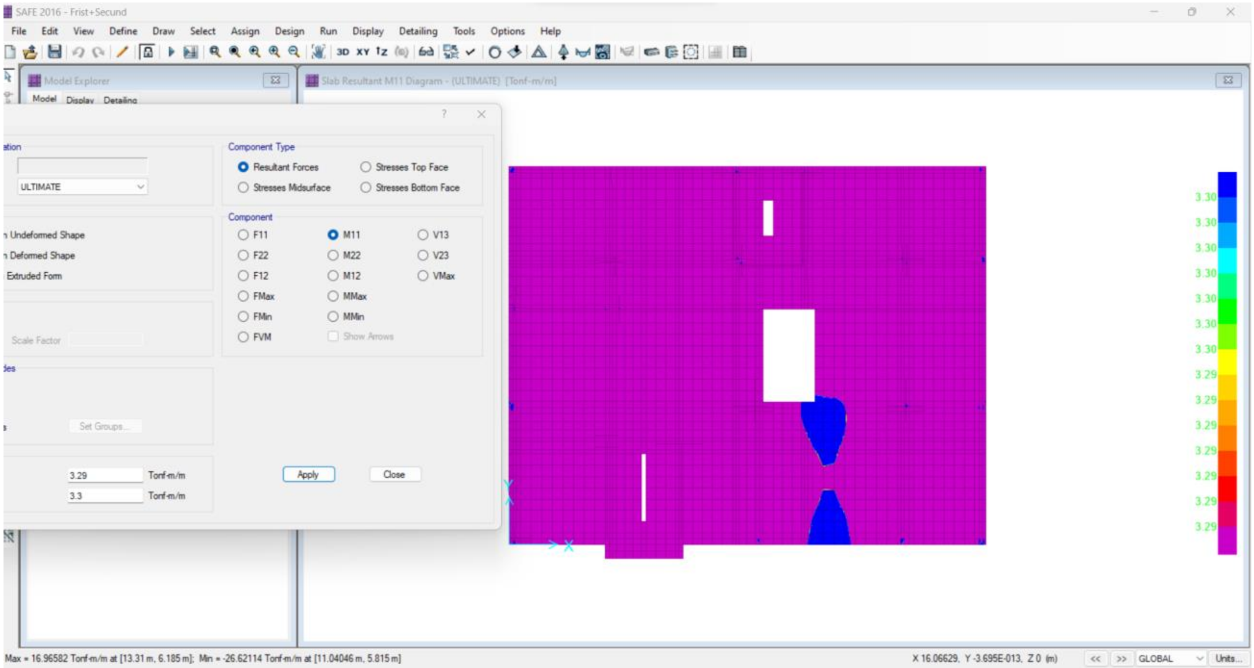


Figure 1.18 Additional Reinforcement in X-Direction (Lower)

In X-Direction: (Upper)

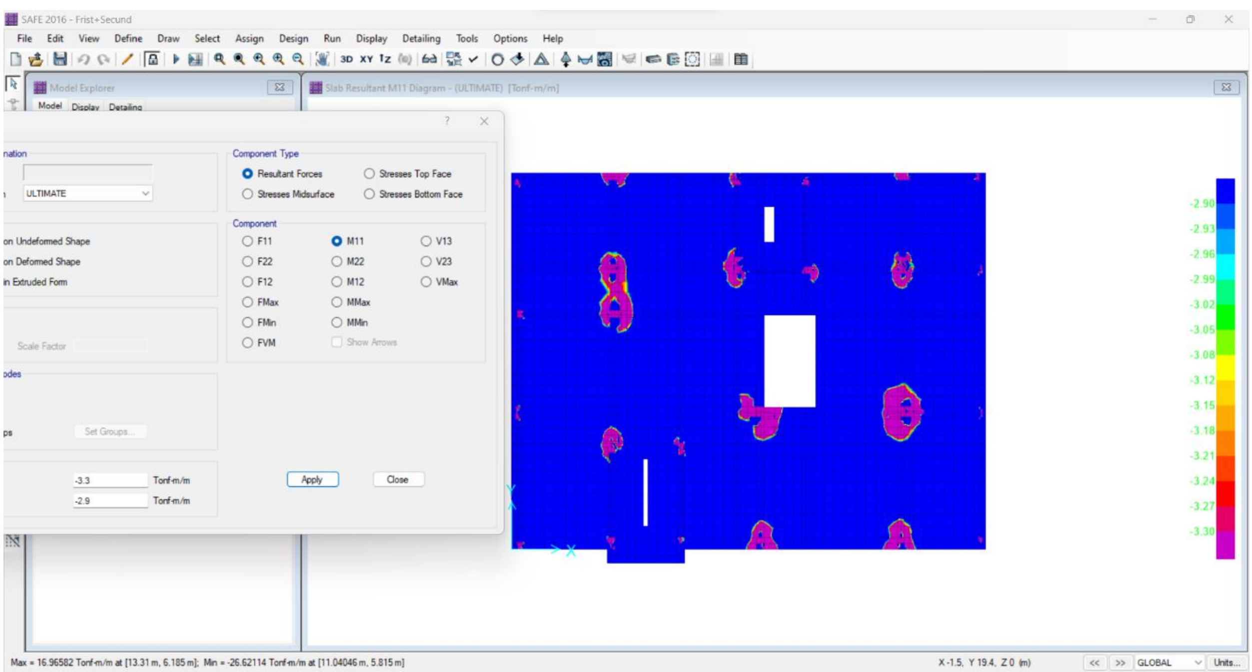


Figure 1.19 Additional Reinforcement in X-Direction (Upper)

In Y-Direction: (Lower)

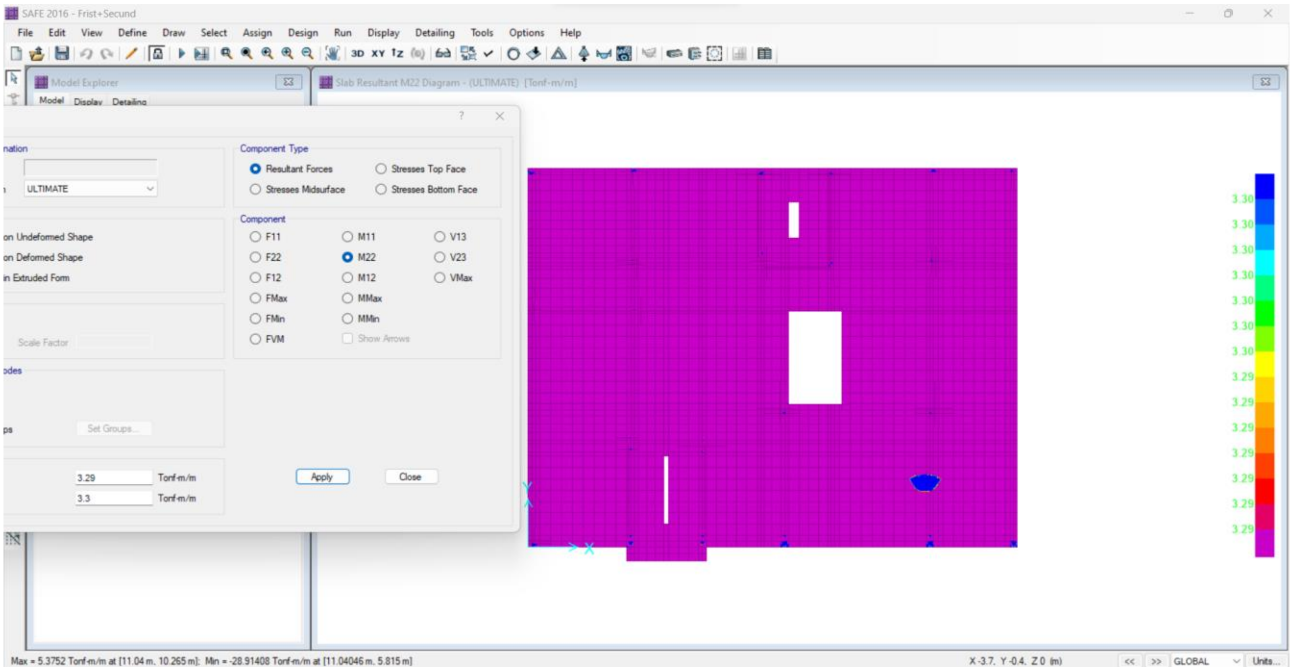


Figure 1.20 Additional Reinforcement in Y-Direction (Lower)

In Y-Direction: (Upper)

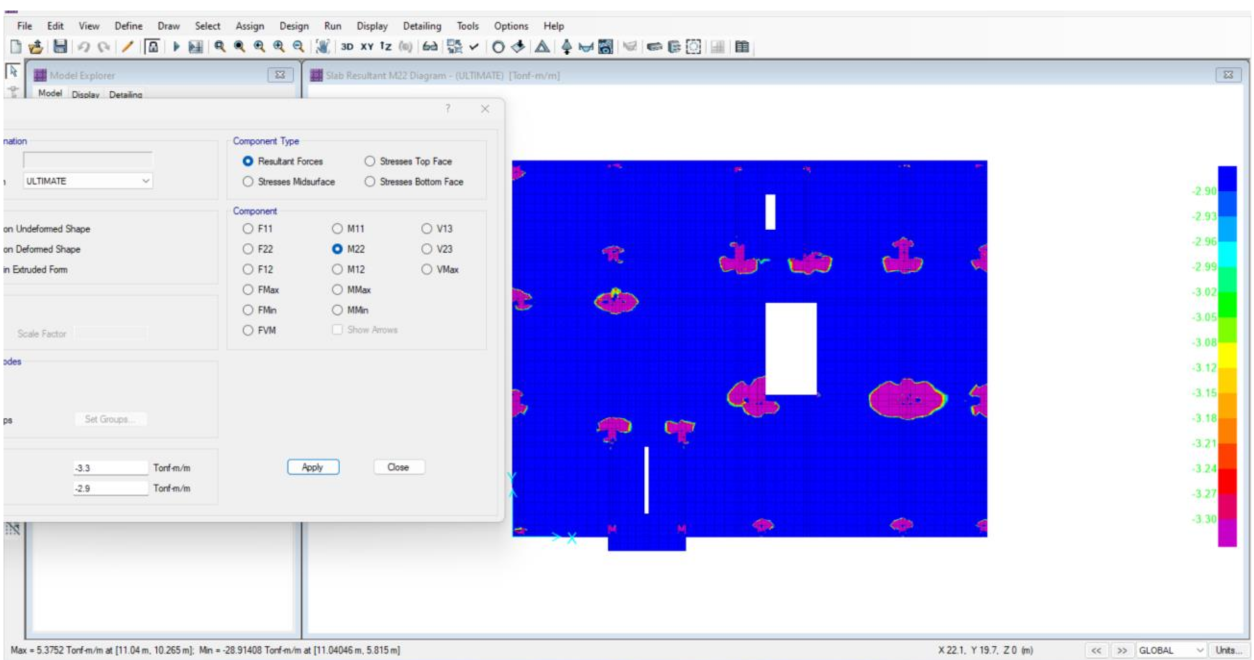


Figure 1.21 Additional Reinforcement in Y-Direction (Upper)

1.2.2.2 Check forAll Loads Deflection:

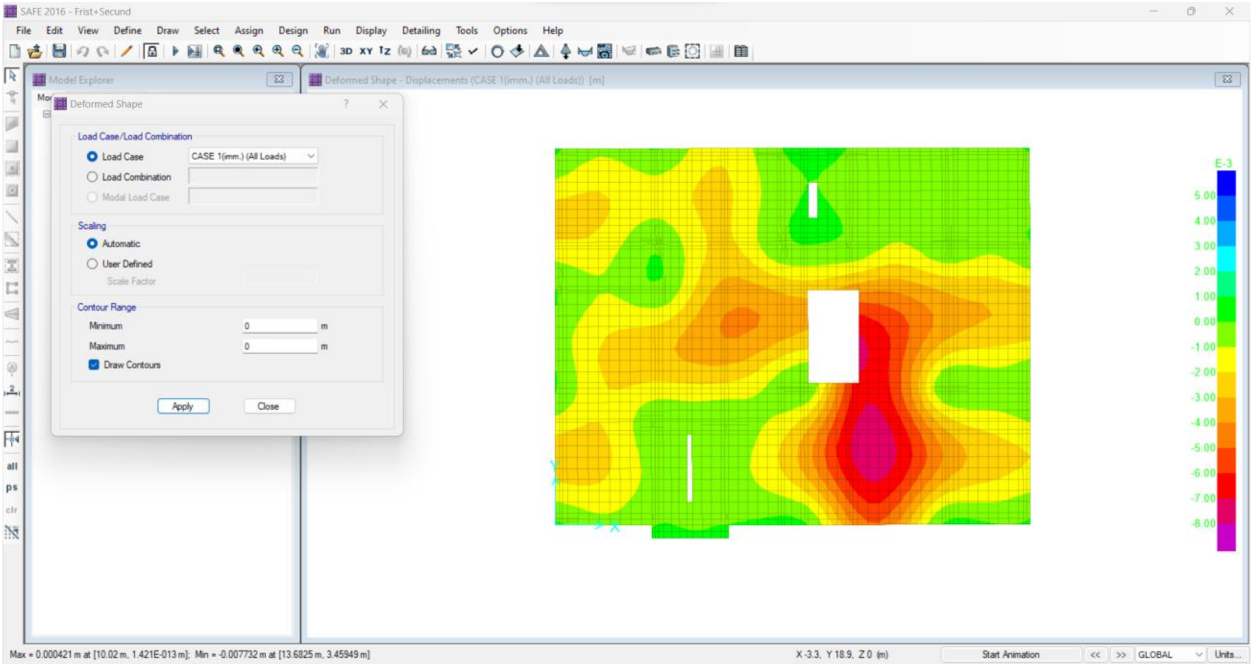


Figure 1.22 All Loads Deflection

- From Code Check = L/360
- Span for Check = 6 m
- Allowable Deflection = 0.0384 m
- Maximum Deflection = 0.000421 m

1.2.2.2 Check for Total Long Deflection:

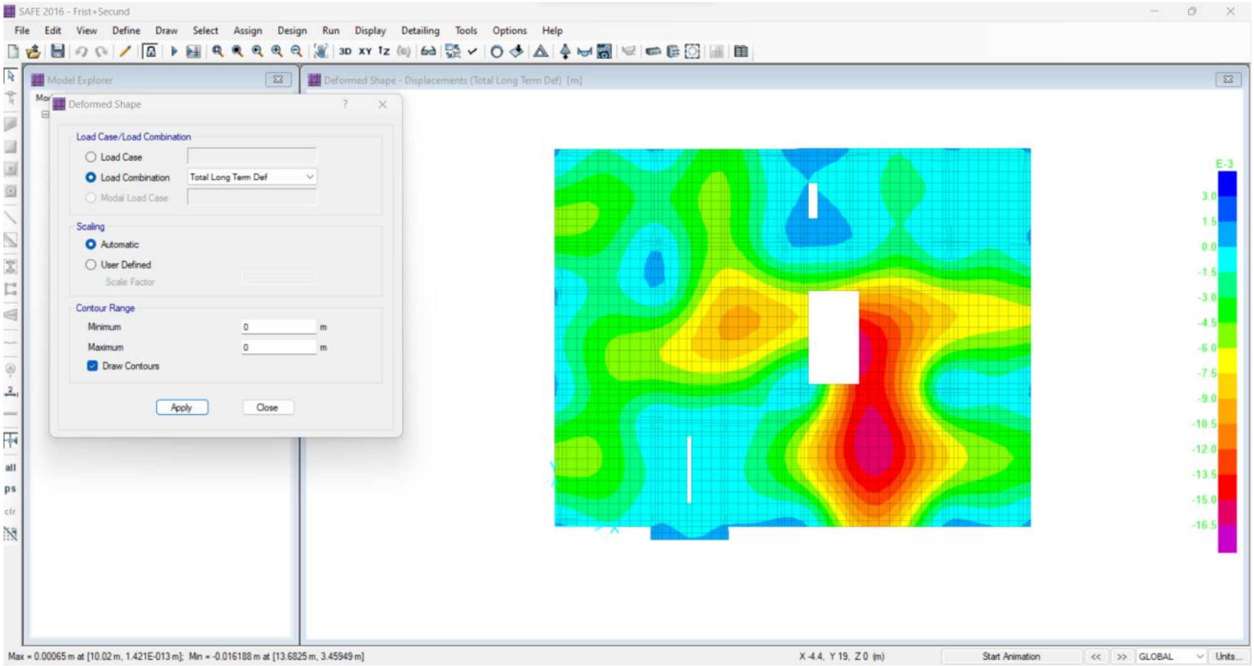


Figure 1.23 Total Long Deflection

- From Code Check = L/250
- Span for Check = 9.6 m
- Allowable Deflection = 0.0384 m
- Maximum Deflection = 0.00065 m

1.2.2.2 Check for Total Dead Loads Deflection:

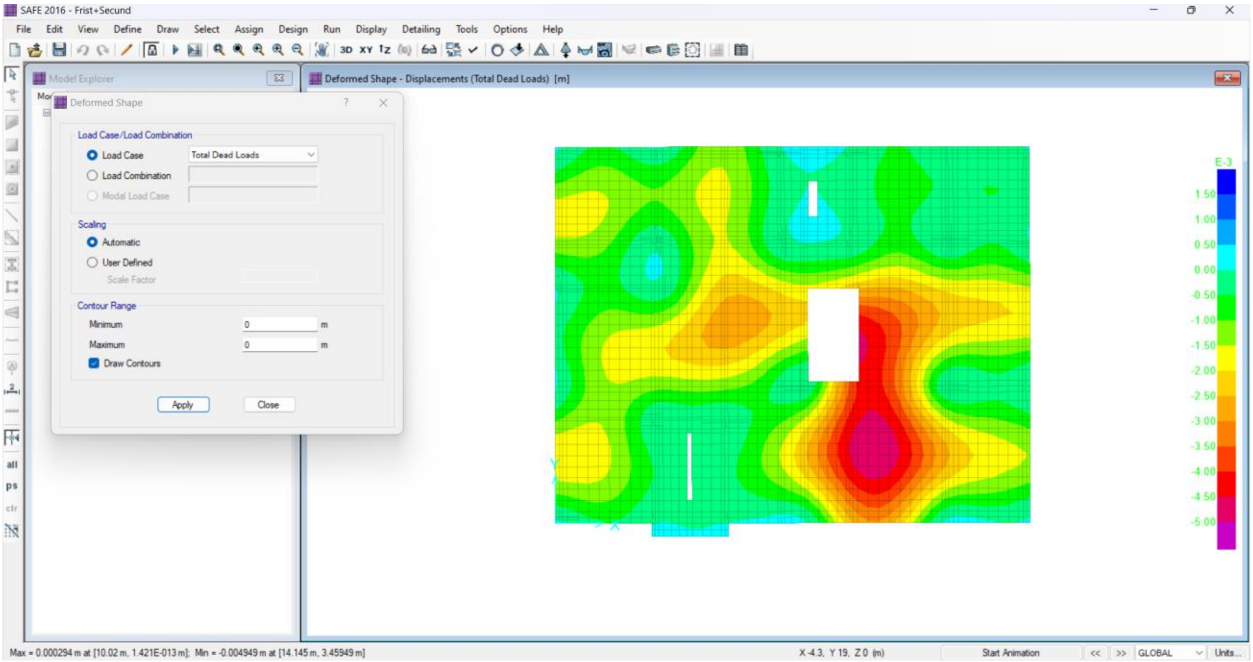


Figure 1.24 Total Dead Loads Deflection

- From Code Check = L/250
- Span for Check = 9.6 m
- Allowable Deflection = 0.0384 m
- Maximum Deflection = 0.000294 m

1.2.2.1 Roof Slab:(Flat Slab System)

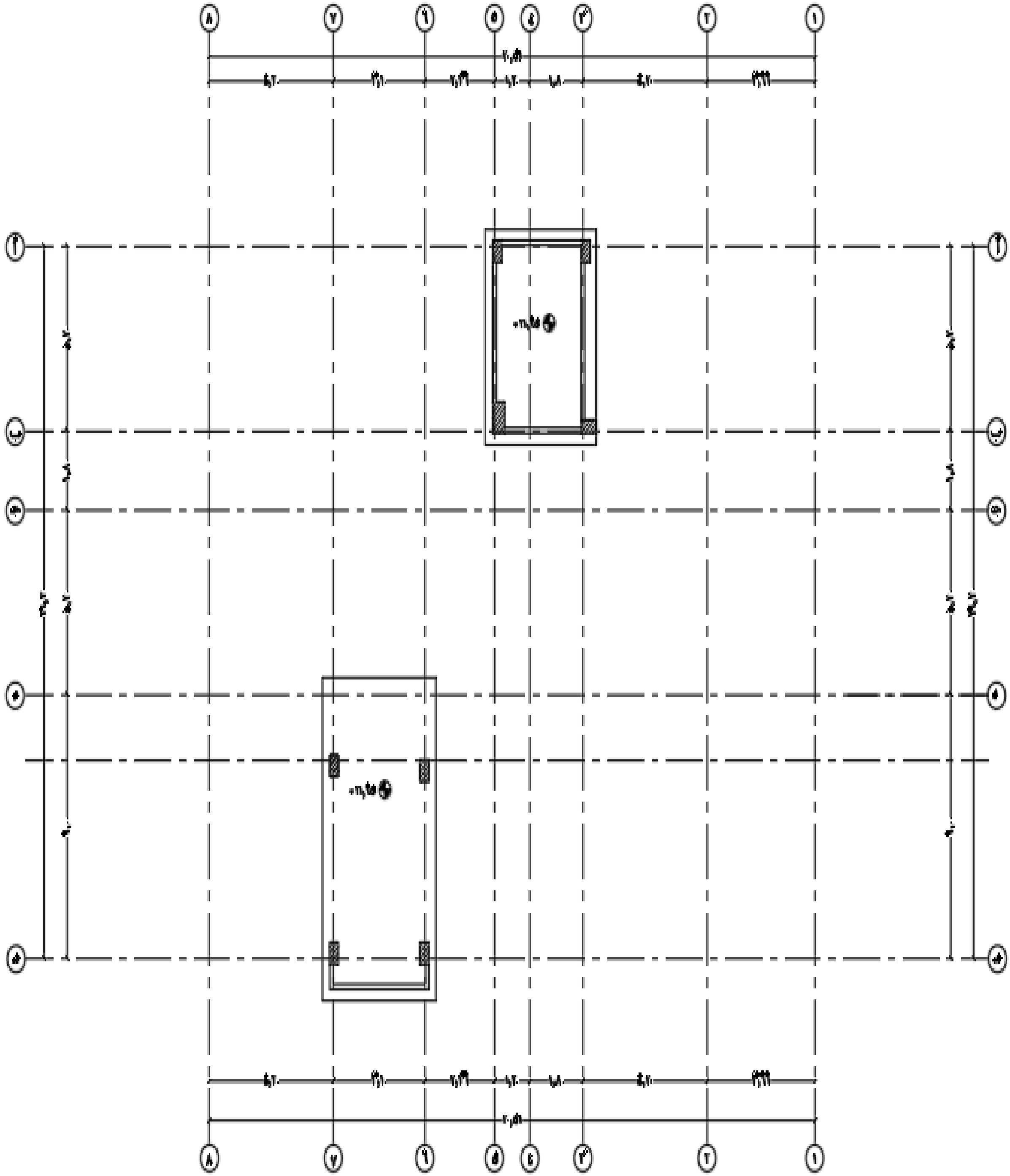


Figure 1.25 Statical System Roof

- Slab Thickness = 22 cm
- Own weight = 0.22* 2.5= 0.55 t/m^2.
- Live load = 300 kg/m^2 = 0.3 t/m^2
- Wall load = 150 kg/m^2 = 0.15 t/m^2.

Solving This flat slab By Using CSI Safe program:

- D.L = O.W +W_{wall} + Covering material
  =0.55+0.15+0.2=0.9 t/m^2
- L.L = 300 kg/cm^2 = 0.3 t/m^2
- W_u = 1.4 D.L + 1.6 L.L = 1.4*0.9+1.6*0.3 = 1.74 t/m^2

For ultimate design: -

- $As = \left[\frac{Mu}{Fy*J*d}\right]$
- M_u = As * F_y *J *d = $6*\left(\frac{\pi*(1.2)^2}{4}\right)*3500*0.826*20*(10)^{-5}$
- M(r)= 3.92 t.m ⇛ Use 6 ∯ 12 / m in each Direction
- Additional RFT (3 ∯ 12 /m) & (3 ∯12 / m) upper and lower

Figure 1.8 Statical System of Roof

In X-Direction: (Lower)

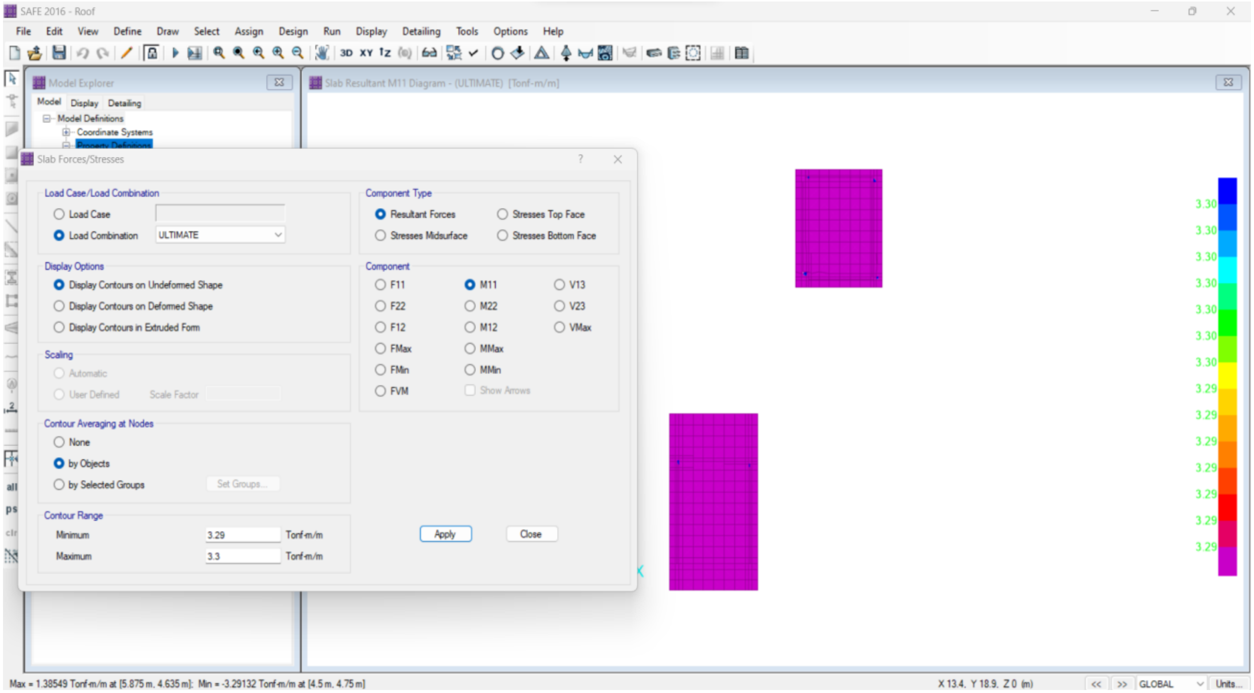


Figure 1.26 Additional Reinforcement in X-Direction (Lower)

In X-Direction: (Upper)

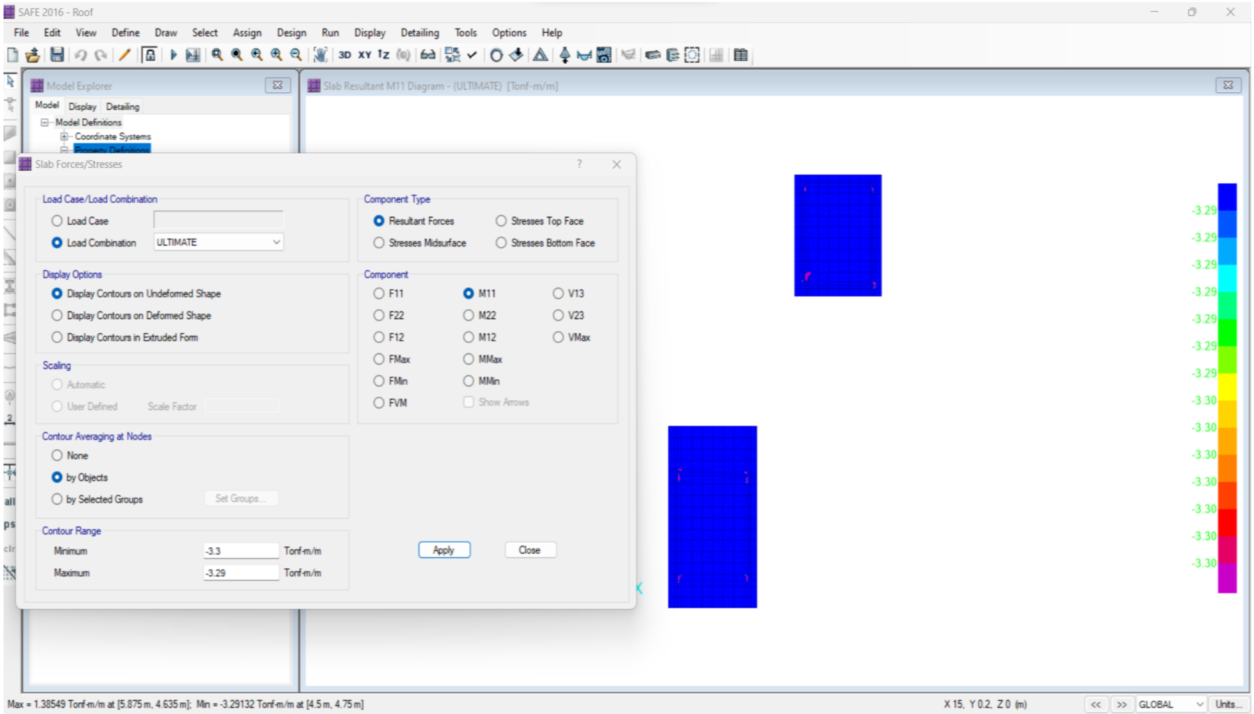


Figure 1.27 Additional Reinforcement in X-Direction (Upper)

In Y-Direction: (Lower)

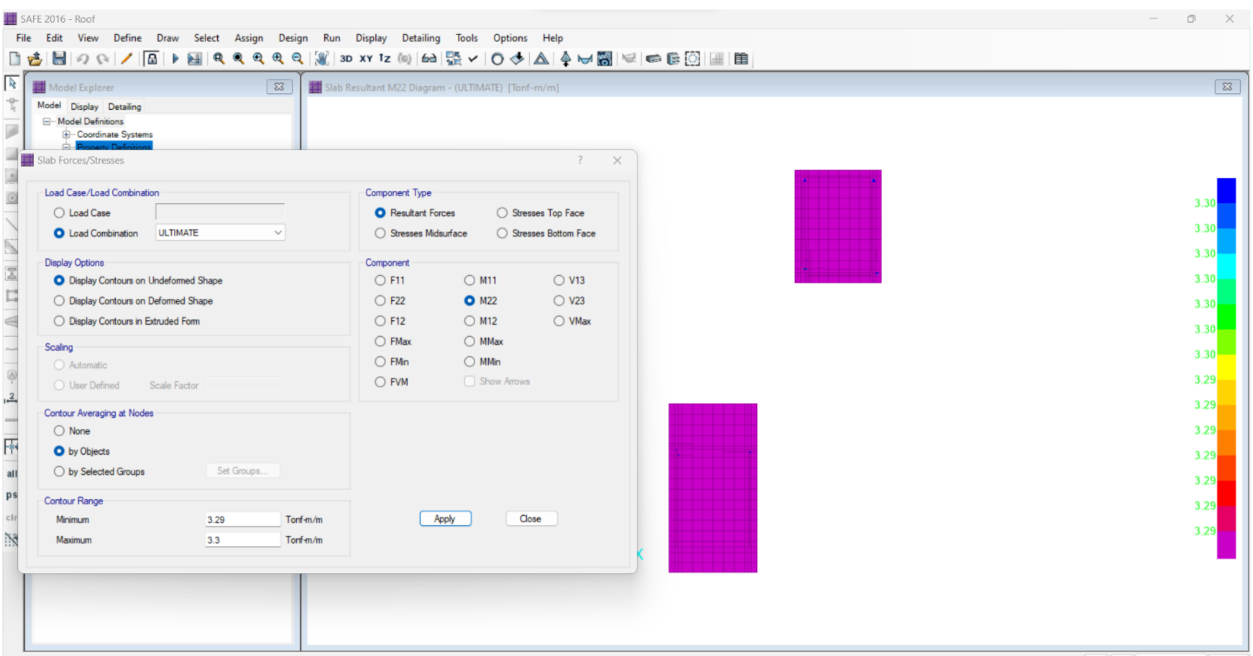


Figure 1.28 Additional Reinforcement in Y-Direction (Lower)

In Y-Direction: (Upper)

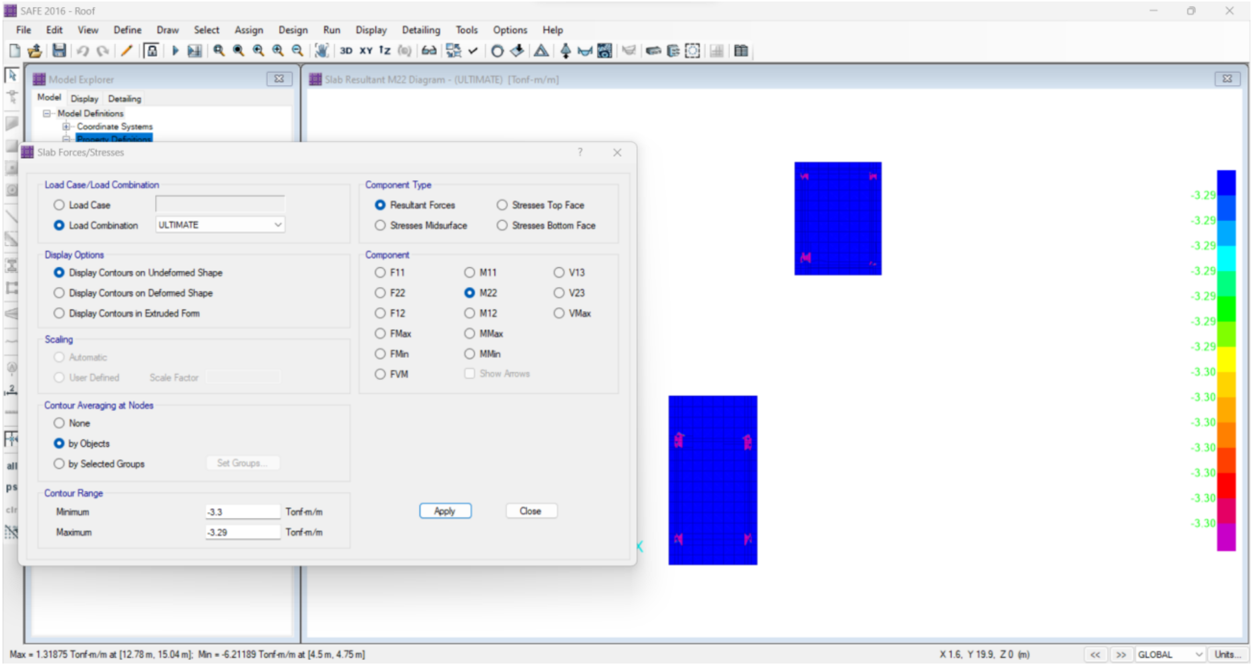


Figure 1.29 Additional Reinforcement in Y-Direction (Upper)

1.2.2.2 Check forAll Loads Deflection:

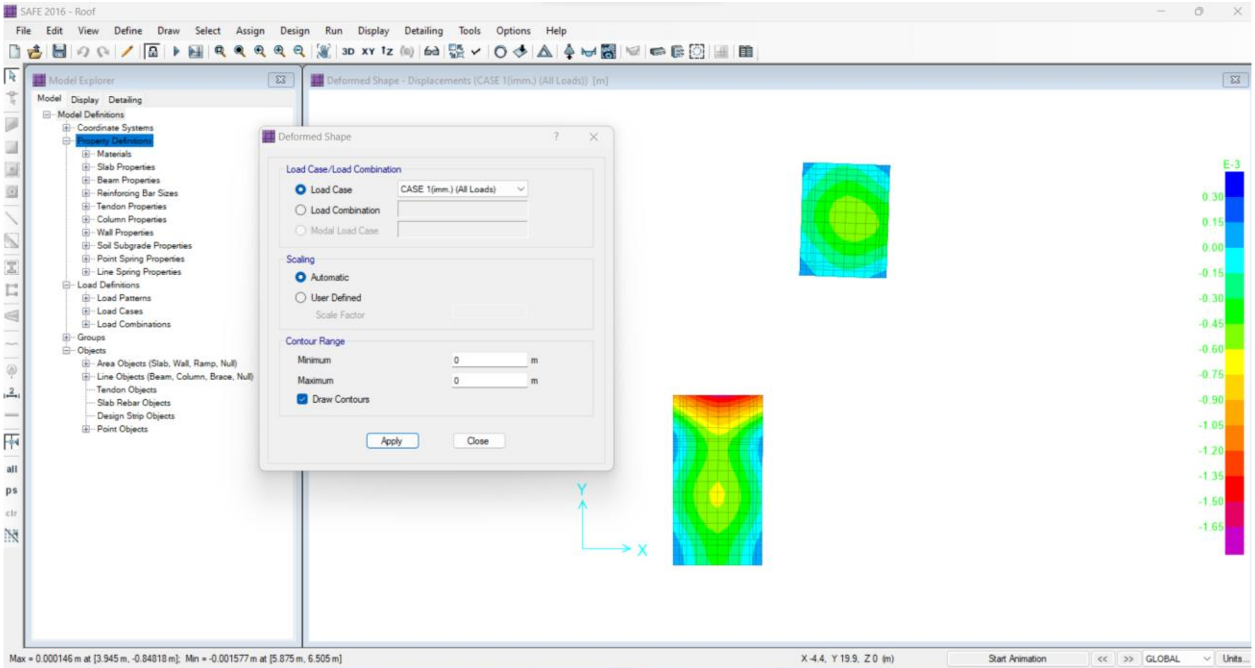


Figure 1.30 All Loads Deflection

- From Code Check = L/360
- Span for Check = 3 m
- Allowable Deflection = 0.0384 m
- Maximum Deflection = 0.000146 m

1.2.2.2 Check for Total Long Deflection:

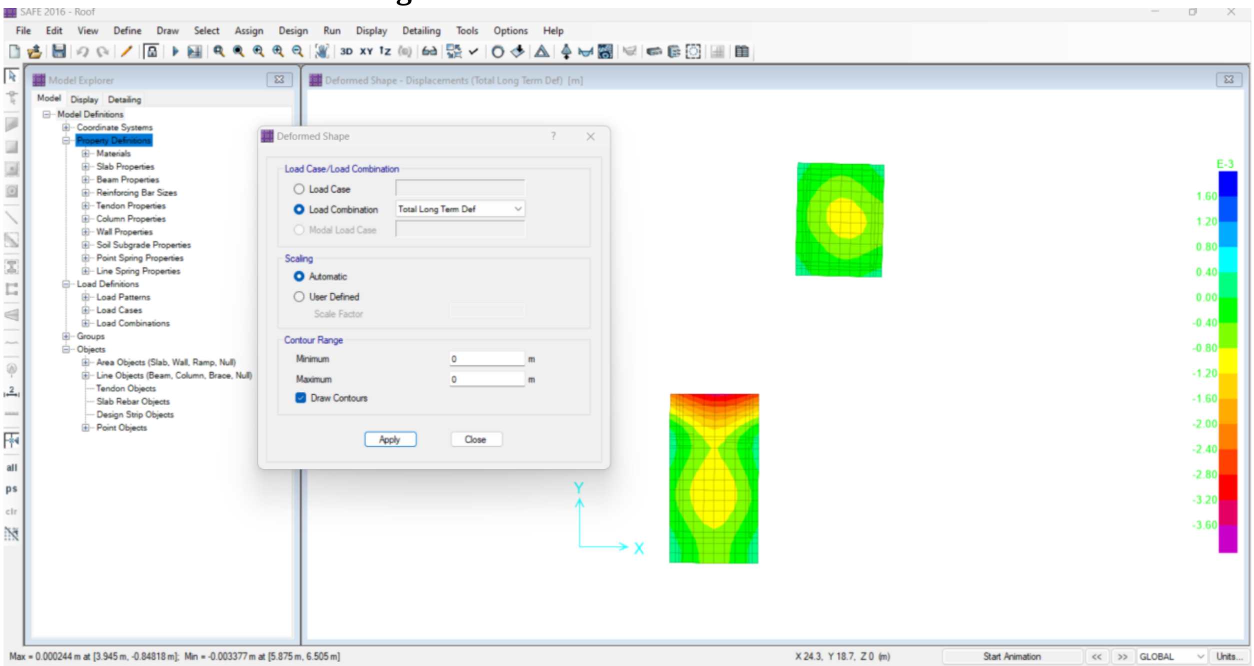


Figure 1.31 Total Long Deflection

- From Code Check = L/250
- Span for Check = 3 m
- Allowable Deflection = 0.0384 m
- Maximum Deflection = 0.000244 m

1.2.2.2 Check for Total Dead Loads Deflection:

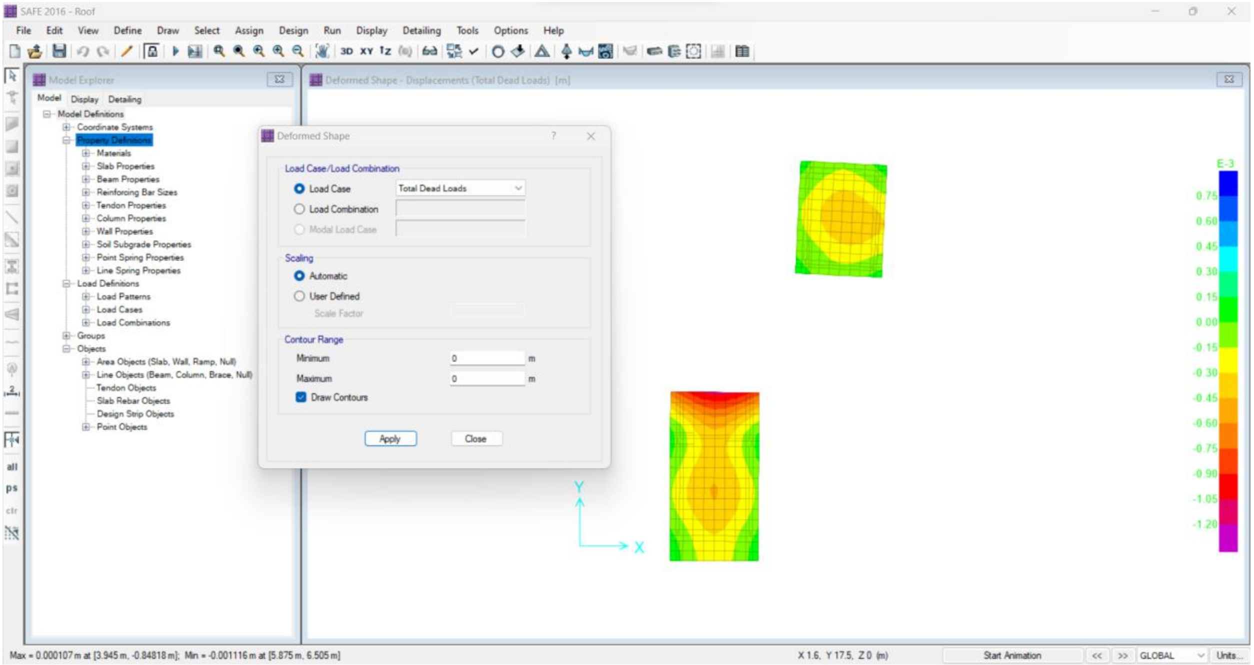
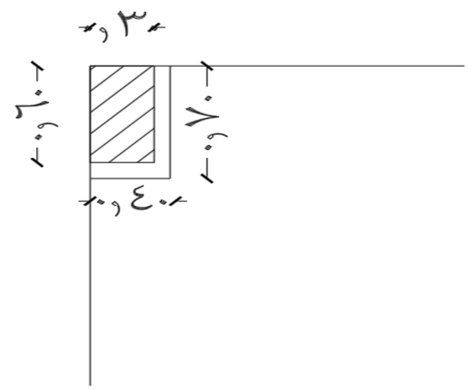


Figure 1.32 Total Dead Loads Deflection

- From Code Check = L/250
- Span for Check = 3 m
- Allowable Deflection = 0.0384 m
- Maximum Deflection = 0.000107 m

1.2.6 Check of Punching Shear:

1.2.6.1 corner Column (C_{25}=30*60) on (1- ا) Axis:



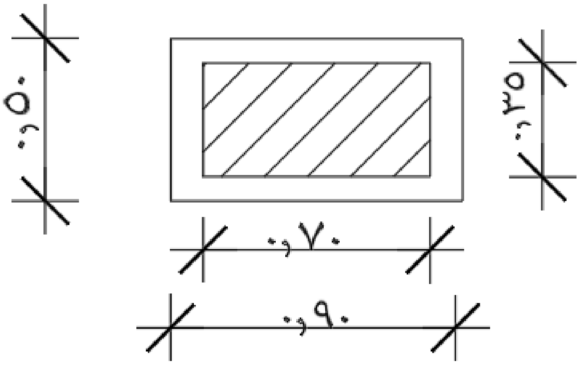
- Slab Thickness = 22 cm
- Own weight = 0.22* 2.5 = 0.55 t/m^2.
- Covering =200 kg/m^2 = 0.2 t/m^2.
- Live load = 300 kg/m^2 = 0.3 t/m^2
- Wall load = 150 kg/m^2 = 0.15 t/m^2.

- D.L = O.W +W_{wall} + Covering material
- = .55 + 0.3 + 0.2 =0.9 t/m^2
- L.L = 300 kg/cm^2 = 0.3 t/m^2

- $W_u = 1.4\, D.L\ +\ 1.6\, L.L\ = 1.4 * 0.9 + 1.6 * .3 = 1.74\ t/m2\ = 17.4\ Kn/m2$
- $d\ =\ t_s -$ 20 mm = 220 – 20 = 200 mm = 0.2m
- b_o =700+400=1100 mm
- Q_{up}=Wu (L1 * L2 – Ap)=17.4*(($\frac{6}{2}$*$\frac{4.2}{2}$– 0.4*0.7)=104.748Kn/m^2
- $q_{up}\ = \frac{Q_{up}}{bo*d} * \beta = \frac{104.748*1000}{1100*200} * 1.5 = 0.714\ \text{N}/mm^2$
- q_{cup} =*the least of*:-
  - 1.7 N/mm^2
  - $0.316\sqrt{\frac{f_{cu}}{\gamma_c}} == 0.316\sqrt{\frac{25}{1.5}} = 1.29\ \text{N}/mm^2$
  - $0.316(\frac{a}{b} + 0.5)\sqrt{\frac{f_{cu}}{\gamma_c}} = 0.316\ (\frac{300}{600} + 0.5)\ \sqrt{\frac{25}{1.5}} = 1.29\ \text{N}/mm^2$
  - $0.8(\frac{\alpha * d}{b0} + 0.2)\sqrt{\frac{f_{cu}}{\gamma_c}} = 0.8(\frac{2*200}{1100} + 0.2)\ \sqrt{\frac{25}{1.5}} = 1.84\ \text{N}/mm^2$
- $q_{up}\ = 1.226 \frac{\text{N}}{mm^2} \leq q_{cup}\ = 1.29\ \text{N}/mm^2$

OK safe punching

1.2.6.2 interior Column (C8 = 35*70) on (2 – د) Axis:

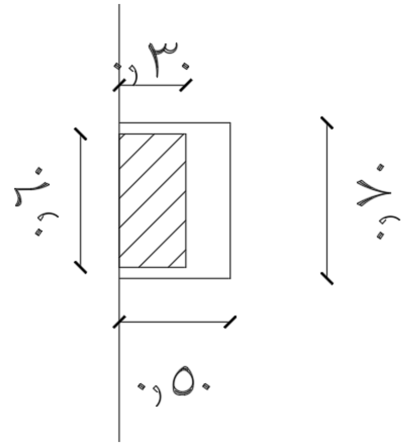


- Slab Thickness = 25 cm
- Own weight = 0.22* 2.5 = 0.55 t/m^2.
- Covering = 200 kg/m^2 = 0.20 t/m^2.
- Live load = 300 kg/m^2 = 0.3 t/m^2
- Wall load = 150 kg/m^2 = 0.15 t/m^2.

- D.L = O.W +W$_{wall}$ + Covering material
- = .55 + 0.15 + 0.20 =0.9 t/m^2
- L.L = 300 kg/cm^2 = 0.3 t/m^2

- $W_u = 1.4\,D.L + 1.6\,L.L = 1.4 * 0.9 + 1.6 * .3 = 1.74\ t/m2 = 17.4 Kn/m2$
- $d = t_s -$ 20 mm = 220 – 20 = 200 mm = 0.2m
- b_o =2*(900+500) = 2800 mm
- Q_{up}=Wu (L1 * L2 – Ap) = 17.4*(6*4.20 – 0.9*0.5) = 430 Kn/m^2
- $q_{up} = \frac{Q_{up}}{bo*d} * \beta = \frac{430*1000}{2800*200} * 1.15 = 0.164\ \text{N}/mm^2$
- q_{cup} =*the least of*:-
  - 1.70 N/mm^2
  - $0.316 \sqrt{\frac{f_{cu}}{\gamma_c}} == 0.316 \sqrt{\frac{25}{1.5}} = 1.29\ \text{N}/mm^2$
  - $0.316(\frac{a}{b} + 0.5)\sqrt{\frac{f_{cu}}{\gamma_c}} = 0.316\ (\frac{350}{700} + 0.5)\sqrt{\frac{25}{1.5}} = 1.38\ \text{N}/mm^2$
  - $0.8(\frac{\alpha * d}{bo} + 0.5)\sqrt{\frac{f_{cu}}{\gamma_c}} = 0.8(\frac{2*200}{2800} + 0.2)\sqrt{\frac{25}{1.5}} = 1.08\ \text{N}/mm^2$
- $q_{up} = 0.164 \frac{\text{N}}{mm^2} \leq q_{cup} = 1.29\ \text{N}/mm^2$

OK safe punching



1.2.6.1 Edge Column (C_{12}=30*60) on (7-د) Axis:

- Slab Thickness = 22 cm
- Own weight = 0.22* 2.5 = 0.55 t/m^2.
- Covering =200 kg/m^2 = 0.2 t/m^2.
- Live load = 300 kg/m^2 = 0.3 t/m^2
- Wall load = 150 kg/m^2 = 0.15 t/m^2.

- D.L = O.W +W_{wall} + Covering material
- = .55 + 0.3 + 0.2 =0.9 t/m^2
- L.L = 300 kg/cm^2 = 0.3 t/m^2

- $W_u = 1.4\,D.L + 1.6\,L.L = 1.4 * 0.9 + 1.6 * .3 = 1.74\,t/m2 = 17.4\,Kn/m2$
- $d = t_s -$ 20 mm = 220 – 20 = 200 mm = 0.2m
- b_o =(300+200) + 2*(600+$\frac{200}{2}$)=1900 mm
- Q_{up}=Wu (L1 * L2 – Ap)=17.4*(6*$\frac{4.2}{2}$ – 0.5*0.7)=213.15 Kn/m^2
- $q_{up} = \frac{Q_{up}}{bo*d} * \beta = \frac{213.15*1000}{1900*200} * 1.3 = 0.729\ N/mm^2$
- q_{cup} =*the least of*:-
  - 1.7 N/mm^2
  - $0.316\sqrt{\frac{f_{cu}}{\gamma_c}} == 0.316\sqrt{\frac{25}{1.5}} = 1.29\ N/mm^2$
  - $0.316(\frac{a}{b} + 0.5)\sqrt{\frac{f_{cu}}{\gamma_c}} = 0.316\,(\frac{300}{600} + 0.5)\sqrt{\frac{25}{1.5}} = 1.29\ N/mm^2$
  - $0.8(\frac{\alpha*d}{b0} + 0.2)\sqrt{\frac{f_{cu}}{\gamma_c}} = 0.8(\frac{2*200}{1900} + 0.2)\sqrt{\frac{25}{1.5}} = 1.34\ N/mm^2$
- $q_{up} = 1.226\frac{N}{mm^2} \le q_{cup} = 1.29\ N/mm^2$

OK safe punching

1.3 Design of Stairs (Three Flight Stair Axis ١د – ح)

1.3.1 Manual solution

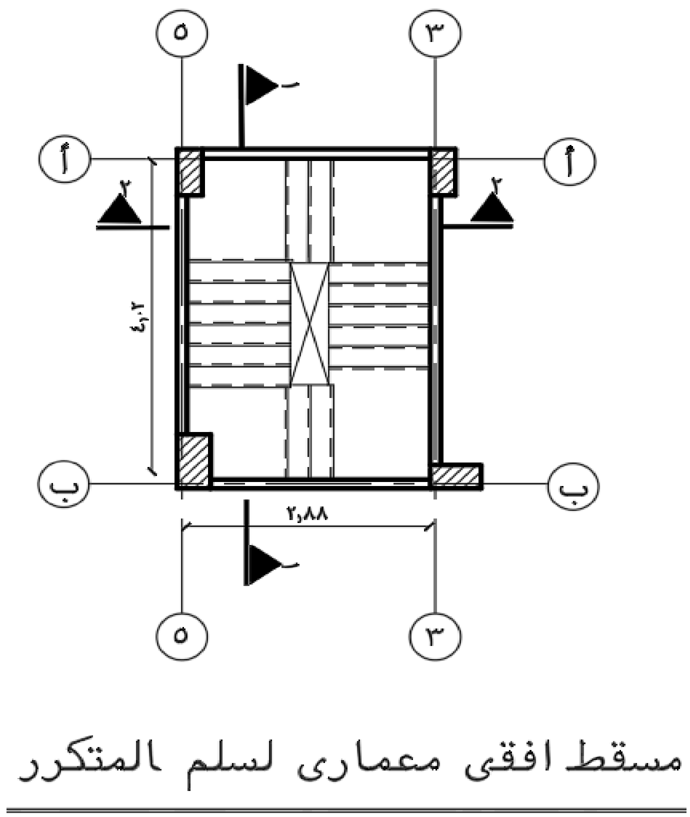


Figure 1.33 Stair Cross Section

1) Dimensions:

- $ts = \frac{Span}{24:30} = \frac{4.2m}{24:30} = .22\ m$
- $H_{Story} = 3.8m$
- $Rise = 0.146\ m$
- $Going = 0.30\ m$
- $\theta = \tan^{-1}(\frac{0.146}{0.30}) = 26.56\ ^{\circ}$
- $t^* = \frac{ts}{Cos\theta} = \frac{22}{Cos(25.96)} = 24.46\ cm$
- $t_{av} = t^* + \frac{Rise}{2} = 24.46 + \frac{14.6}{2} = 31.76$ cm

2) Loads:

- Wfu = 1.4 D. L + 1.6 L.L

  = 1.4 (25*.3176 + 2.0) + 1.6(3)

  = 18.716 KN/m^2

- Wu $_{landing}$= 1.4 D. L + 1.6 L.L

  = 1.4 (25*0.22 + 0.2) + 1.6(3)

  = 15.3KN/m^2

3) For Strips

Shown In The figure

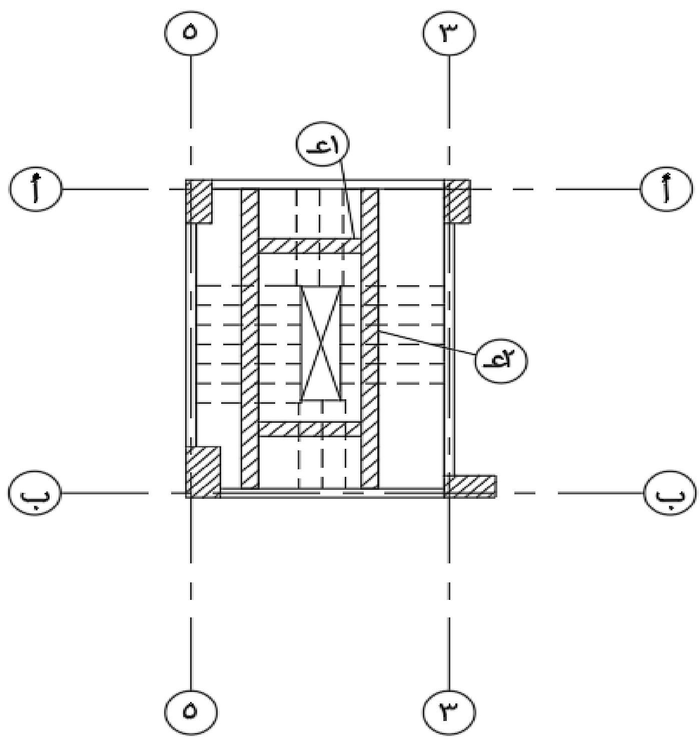


Figure 1.37 Strips of Stair

For Strip (S1):

- $M_{u1} = 0.6*0.27 - \frac{15.3*.27^2}{2} = 0.271$ KN.m
- $A_s = \frac{.271*10^6}{350*.826*(321-20)} = 6 ∯ 12$ ➡ ok

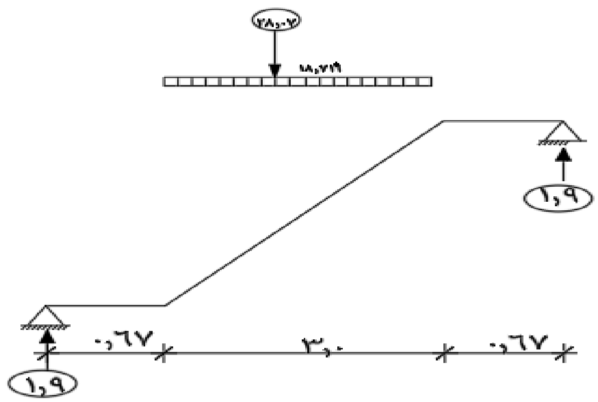


Figure 1.38 Strip (S1) of Stair

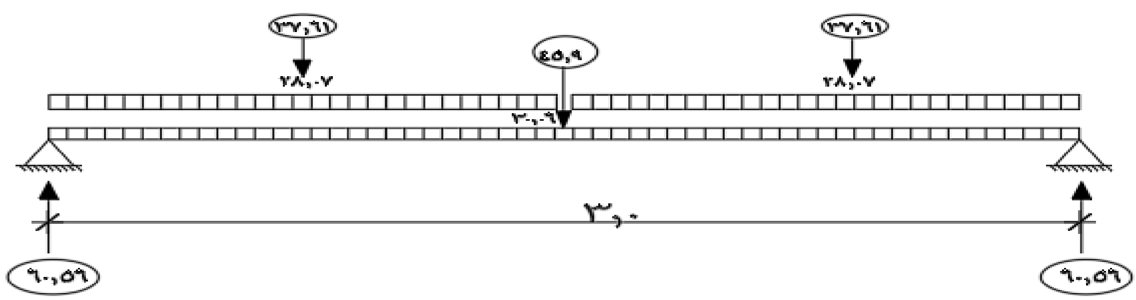


Figure 1.39 Strip (S2) of Stair

For Strip (S2):

- $R1 = \frac{(18.835+3)}{2} =$

  $R_1 = 28.07$Kn

- $M_U = 28.07x(1.5+0.67) - 14.03\ (\frac{1.5}{2} + 0.67)$ $M_U = 40.98$ Kn

  $A_S = \frac{MU}{F_{cu}*b*d} = \frac{40.98*10^6}{250*1000*(190^2)}$

  $A_S = 678\ mm^2$

  $A_S = 6 ∯ 12/\text{m}$

1.3.2 Using Sap Prgram

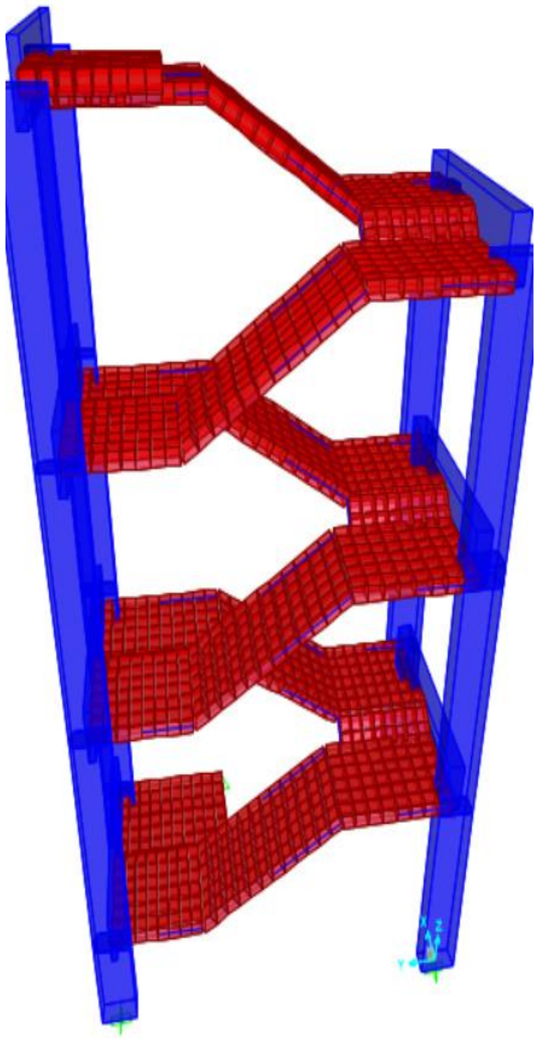


Figure 1.40 St air 3D

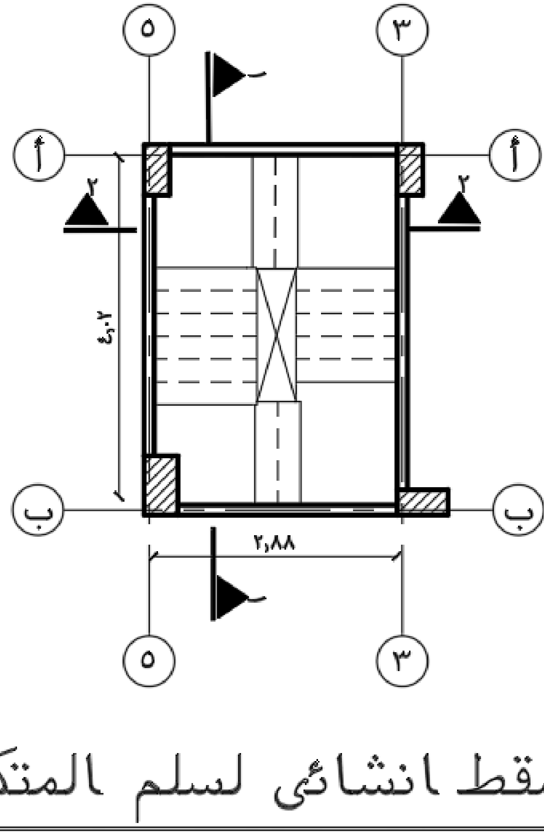


Figure 1.43 Stair Reinforcement in plan

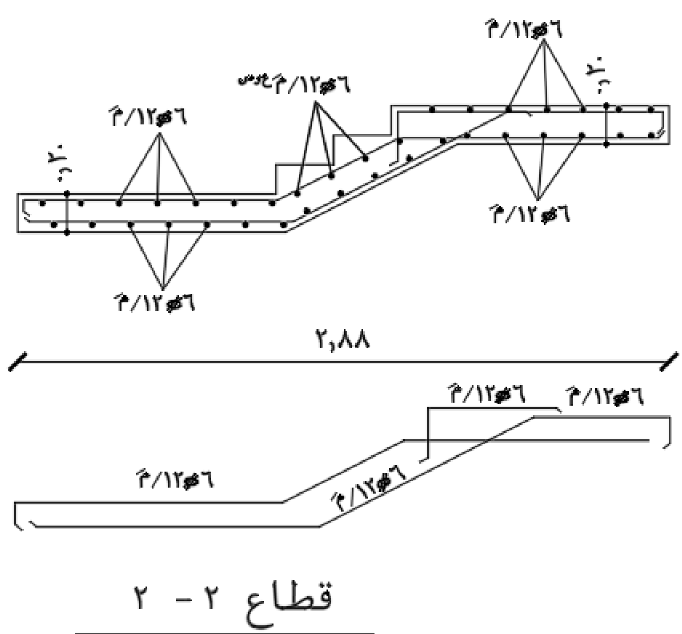
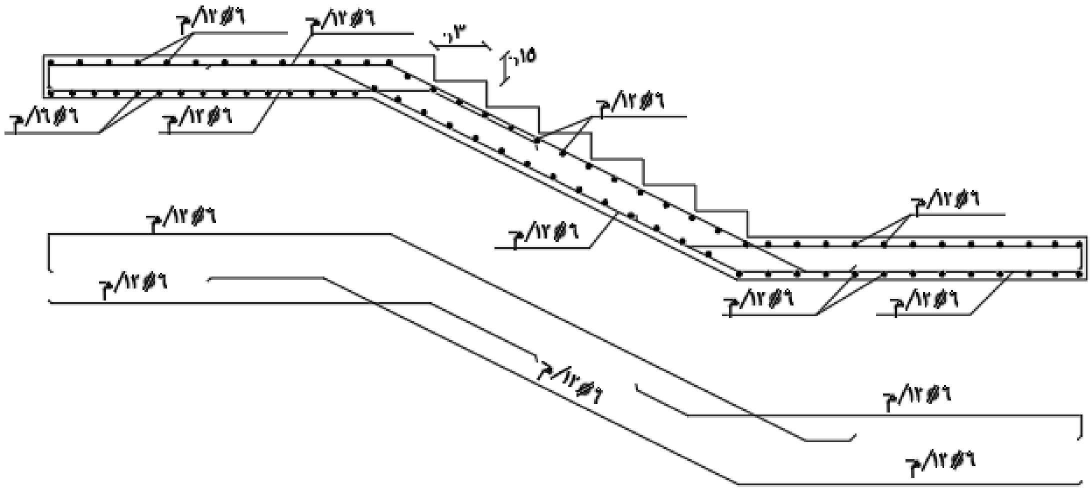


Figure 1.45 Reinforcement in sec 2-2



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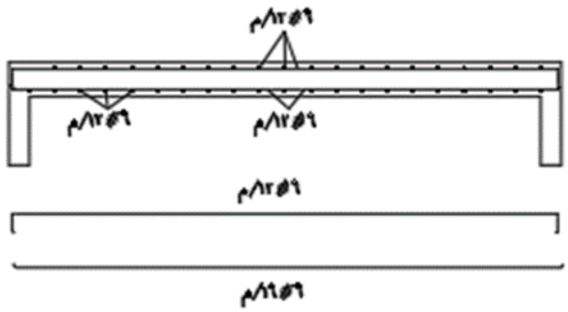
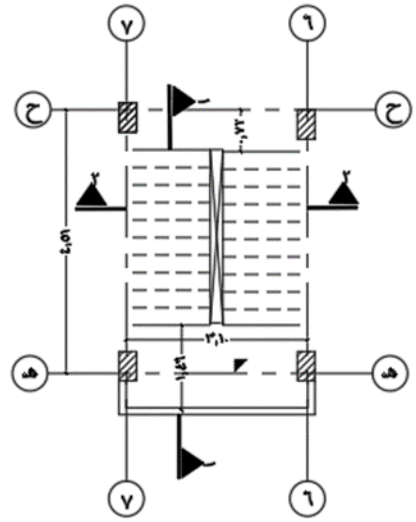
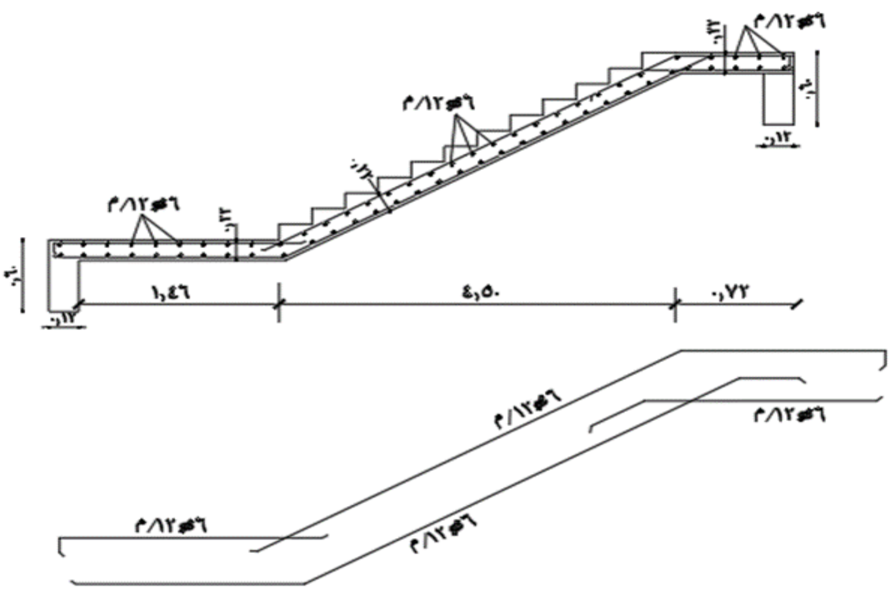
Reinforcement in sec 1-1



قطاع ٢ - ٢



مسقط انشائى لسلم المتكرر



قطاع (١ - ١)

1.5 Design of Beams

Concrete dimensions

- ❖ Assume b = 12 cm
- ❖ t = $\frac{L}{10}$=60 cm
- ❖ **Loads on beams**
  - ❖ Own weight
  - ❖ Load from slab
  - ❖ Load from wall
- ❖ As=$\frac{Mu}{Fy*j*d}$

1.5.1 from All Slab Beams:

جدول تسليح الكمرات :

ملاحظات		كانات / م		تسليح علوي		تسليح سفلي	قطاع	نموذج
		منتصف البحر	حتى ل/٤ من وجه الركيزة	منتصف البحر	فوق الركيزه	عدل		
	كانات مغلقة	٥ ∅ ٨	٥ ∅ ٨	٢ ∅ ١٢	٢ ∅ ١٢	٢ ∅ ١٢	٠,١٢ × ٠,٦٠	ك١
	كانات مغلقة	٥ ∅ ٨	٥ ∅ ٨	٢ ∅ ١٢	٢ ∅ ١٢	٢ ∅ ١٢	٠,١٢ × ٠,٦٠	ك٢

Table 1.1 RFT Of All Slab Beams

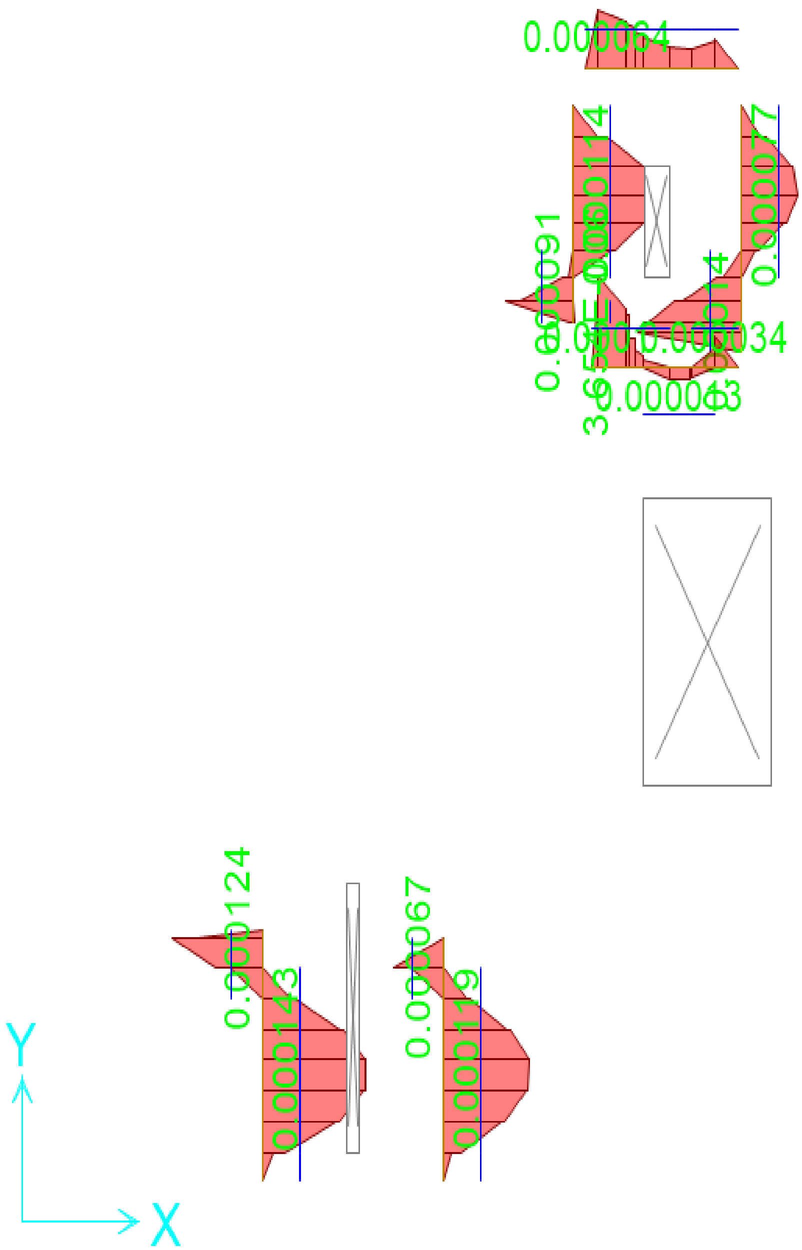


Figure 1.49 Moment of Slab Beams

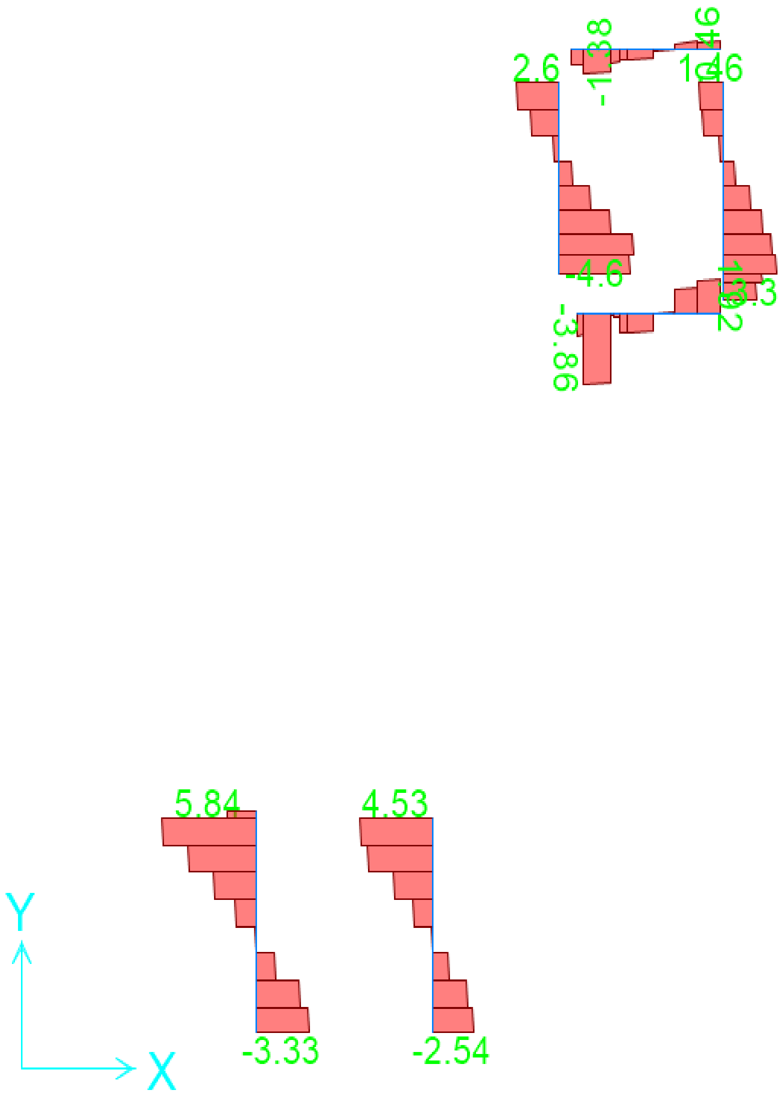


Figure 1.50 Shear on Basement Slab Beams

1.5.6 Check of Shear on Beam Section:

Max shear on floor o= 10.61 t

- $q_{cu}(uncracked) = 0.16\sqrt{\frac{Fcu}{\gamma c}} = 0.16\sqrt{\frac{25}{1.5}} = 0.65$ N/mm^2
- $q_{max} = 0.7\sqrt{\frac{Fcu}{\gamma c}} = 0.7\sqrt{\frac{25}{1.5}} = 2.86$ N/mm^2
- $q_u = \frac{Qmax}{b*d} = \frac{12.4*10^4}{250*650} = 0..763$ N/mm^2

$q_{max} > q_u > q_{cu}(uncracked)$

$q_{cu(cracked)} = .12*\sqrt{\frac{Fcu}{\gamma c}} = .12*\sqrt{\frac{25}{1.5}} = .489$

$q_{su} = q_u - q_{cu(cracked)} = .763 - .489 = .265$

$A_{st} = \frac{q_{su*S*b}}{\frac{Fy}{\gamma s}*n} = \frac{.265*s*250}{\frac{350}{1.15}*2} = 50.24$ ➡ S = 200 ➡ OK

Use Stirrups 5 ∯ 8 / m as minimum
increase stirrups around support as shown in tables

1.6 Design of Columns

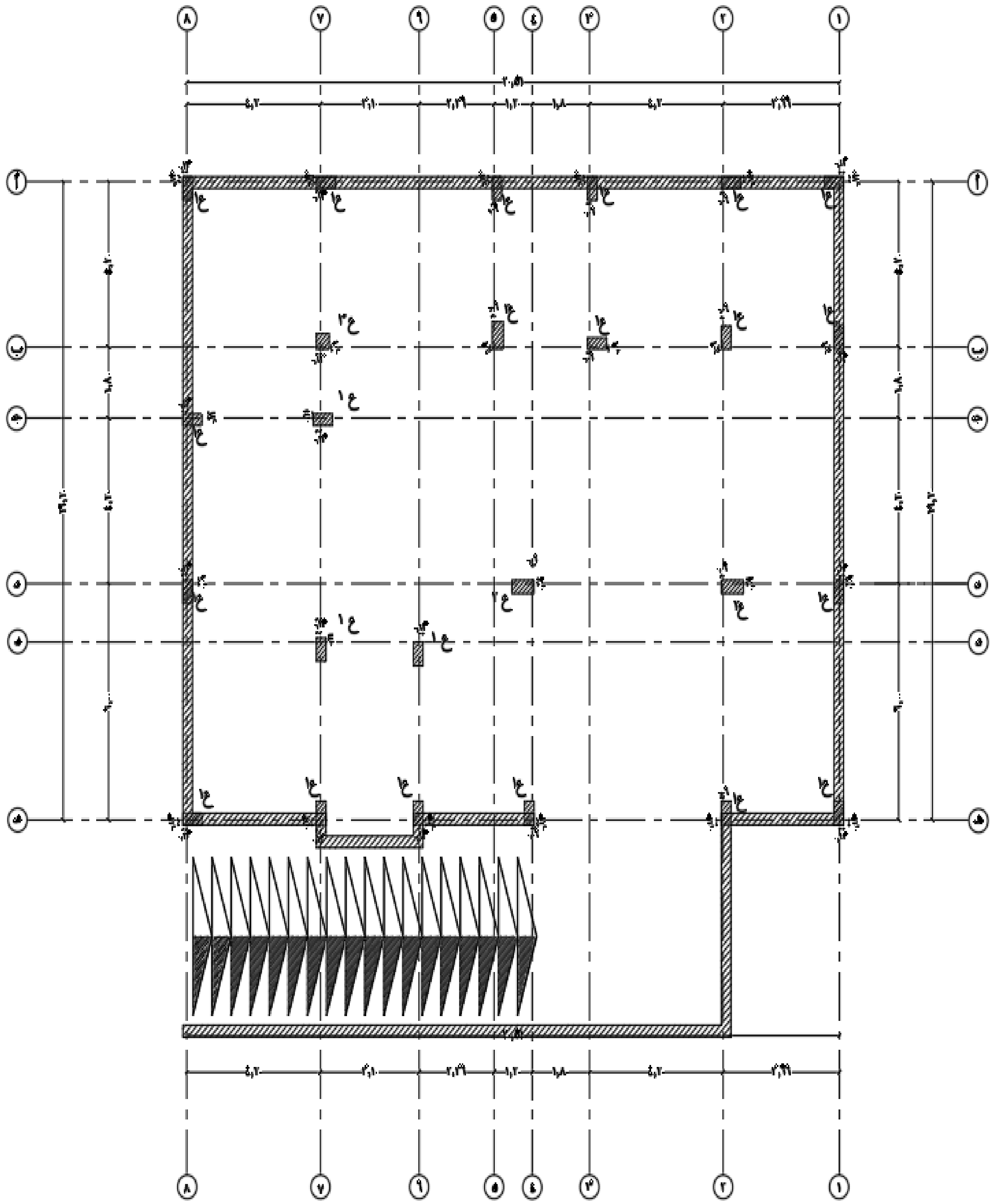


Figure 1.59 Columns A

1.6.1 Design of Column Section (subjected to axial compression force)

- **For 2ع on axis (2-2), (د - د)**
- **DEGSION OF COLUMNS (C_8)**

$(P_u = 236N)$

Solution:

$Pu_{act} = 1.1 * PU = 1.1 * 236$= 260 N

Pu_{act}=.35*Fcu*(Ac-As) +.67*Fy*As

260*10^4=.35*25*(Ac-.0085Ac) +.67*350*.0085Ac

Ac=$\frac{260*10^4}{10.6688}$ = 112476.71 mm^2

Ac=b*t ➡ t= $\frac{243699.54}{350}$ =609.24mm ➔ t= 700mm

Ac=35*70 cm

As=.0.0085*Ac= .01* 40*70 = 28 CM^2 ➡ As=14Φ16 ➡ نتيجة قوة الضغط المحورية

<u>Check of buckling:</u>

λ_b=$\frac{K*H_0}{b}$=$\frac{1.3*3.8}{0.35}$=12.35➡ 23 >12.35 >10

<u>SO,</u> Columns is cylinder

About 2-2

M_{add} 2 =$\frac{\lambda_b{}^2*b}{2000}$*$Pu_{act}$= $\frac{12.35^2*.35}{2000}$ $*$ 260= 7.93 ton.m

M_{min} 2 = .05*.4*56.79= 1.135 ton.m

λ_t=$\frac{K*H_0}{t}$ = $\frac{1.3*3.80}{0.7}$=7.057 ➡ 10 >7.057

SO, Columns is SHORT

About 3-3

M_{min} **3 = .05*0.7*56.79= 1.98 ton.m**

μ_{min}**=.25+.052***λ_b**=.25+.052*12.35=0.1➡**

choose $\mu_{min} = 1.1\%$

As = $\frac{1.1}{100}$***35*70=30.8** cm^2

As =14 Φ**16**

Use 14 ∯ 16

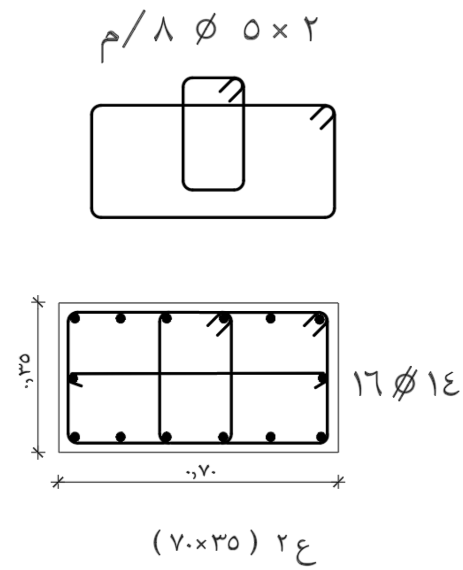


Figure 1.60 Column Cross Section

❖ **DEGSION OF STIRAPS:**

1- DIMETER:

A. 8mm ➡ $\boxed{\phi = 8\ mm}$

B. $\frac{\phi}{4} = \frac{16}{4} = 4$

2- spacing between stirps in the end and start

$S_0 =$
- → $8\phi_L = 8*16 = 128$
- → $24\phi_{str} = 24*8 = 192$ ➡ $\boxed{S_0 = 128}$
- → **.5b= .5* 400= 200 mm**
- → **150mm**

3- Length of stirps in the end and start (l_0)

$(l_0) =$
- → $\frac{H_0}{6} = \frac{3800}{6} = 633.33$ **mm**
- → **T= 700 mm**
- → **.5 m= 500 mm** ➡ $\boxed{l_0 = 1000mm}$
- → **1000mm**

4- The other spacing (S)

S =
- → $15\phi_L = 15*16 = 240mm$
- → **B = 400** ➡ $\boxed{S = 200mm}$
- → $2*S_0 = 2*128 = 256mm$
- → **200mm**

5- L= 3.5-2= $\boxed{1.5}$

➔ *Check volume of stiraps in (1 m)*

$V_{str} \geq .25\% V_{conc}$

$A_{str} * P_{str} * n = \frac{.25}{100} * b * t * 1000$

P_{str} = 2*(642+242) + 4*(107+242) = 3164 mm

$50.3*3164*n = 300*700*\frac{.25}{100}*1000$

n = 3.29 ➔ n = 4 < 5 ϕ 8 Ok

								No . of Bars			
Joint	P_u etabs (T)	$P_{u\ act}$ (T)	$\mu = A_s/A_c$	$A_{c\ act}$ (cm^2)	b (cm)	t (cm)	t_{act} (cm)	A_s (cm^2)	Ø	16	Sample
1	51.0126	57	1.00%	518	30	60	60	18	9		C1
2	83.936	93	1.00%	845	30	60	60	18	9		
3	70.7364	78	1.00%	709	30	60	60	18	9		
4	108.7637	120	1.00%	1091	30	60	60	18	9		
5	115.2356	127	1.00%	1154	30	60	60	18	14		C
6	40.3447	45	1.00%	409	30	60	60	18	9		C1
7	72.2649	80	1.00%	727	30	60	60	18	9		
8	235.9987	260	1.00%	2363	35	70	70	24.5	13		C2
9	219.3214	242	1.00%	2199	35	70	70	24.5	13		
10	131.5191	145	1.00%	1318	30	60	60	18	9		C1
11	160.3483	177	1.00%	1608	30	60	60	18	9		
12	87.8578	97	1.00%	882	30	60	60	18	9		
13	85.239	94	1.00%	854	30	60	60	18	9		
14	151.4225	167	1.00%	1518	30	60	60	18	9		
15	115.4239	127	1.00%	1154	40	40	40	16	8		C3
16	187.4857	207	1.00%	1881	35	70	70	24.5	13		C2
17	134.8535	149	1.00%	1354	30	60	60	18	9		C1
18	169.0257	186	1.00%	1690	30	60	60	18	9		
19	68.7709	76	1.00%	691	30	60	60	18	9		
20	32.0141	36	1.00%	328	30	60	60	18	9		
21	55.1243	61	1.00%	555	30	60	60	18	9		
22	66.807	74	1.00%	673	30	60	60	18	9		
23	75.707	84	1.00%	764	30	60	60	18	9		
24	79.2586	88	1.00%	800	30	60	60	18	9		
25	51.9222	58	1.00%	527	30	60	60	18	9		

Table 16 Columns Load And Section (Ultimate)

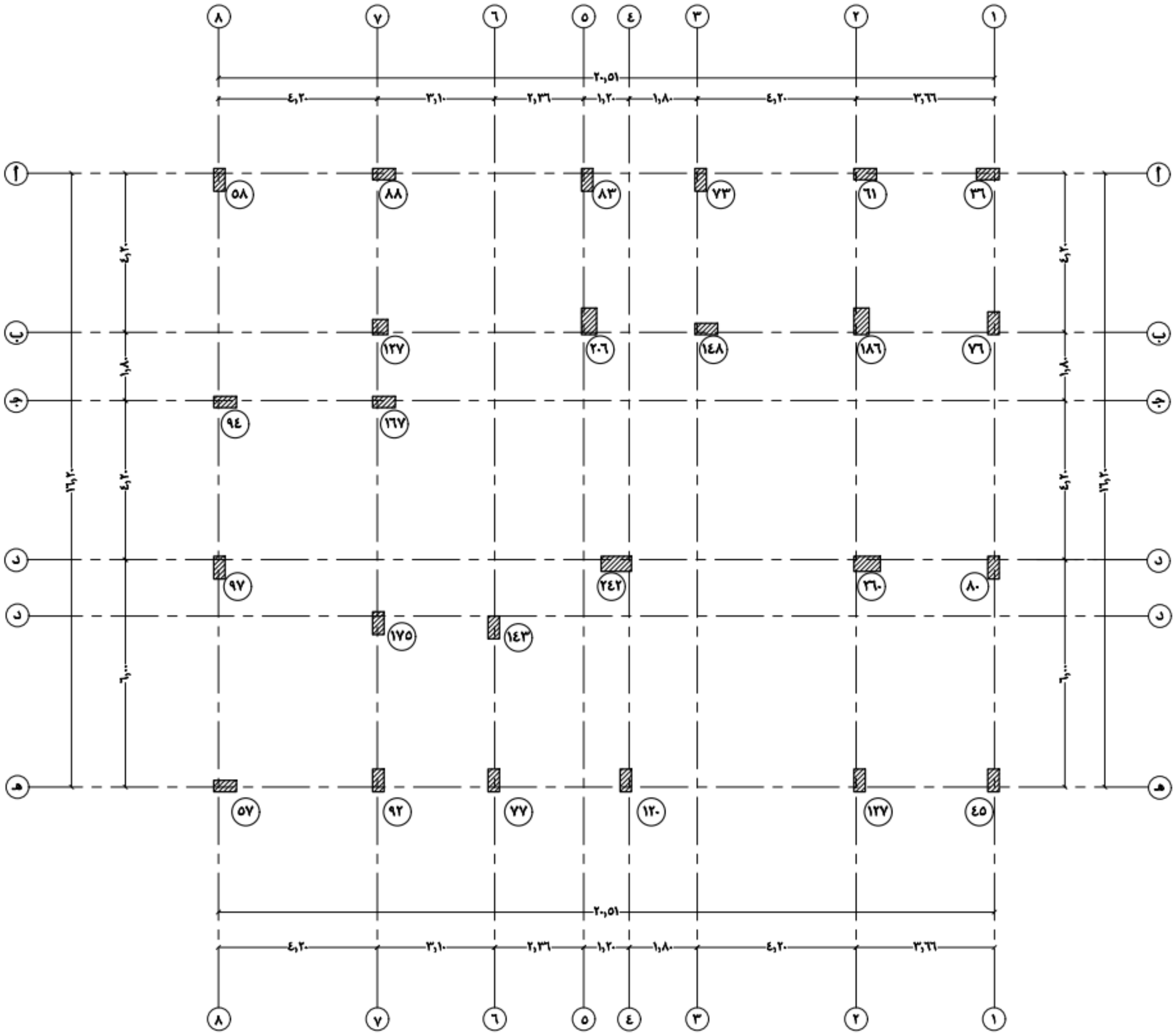


Figure 1.61 Column Reaction (Ultimate)

1.7 Design of Foundation

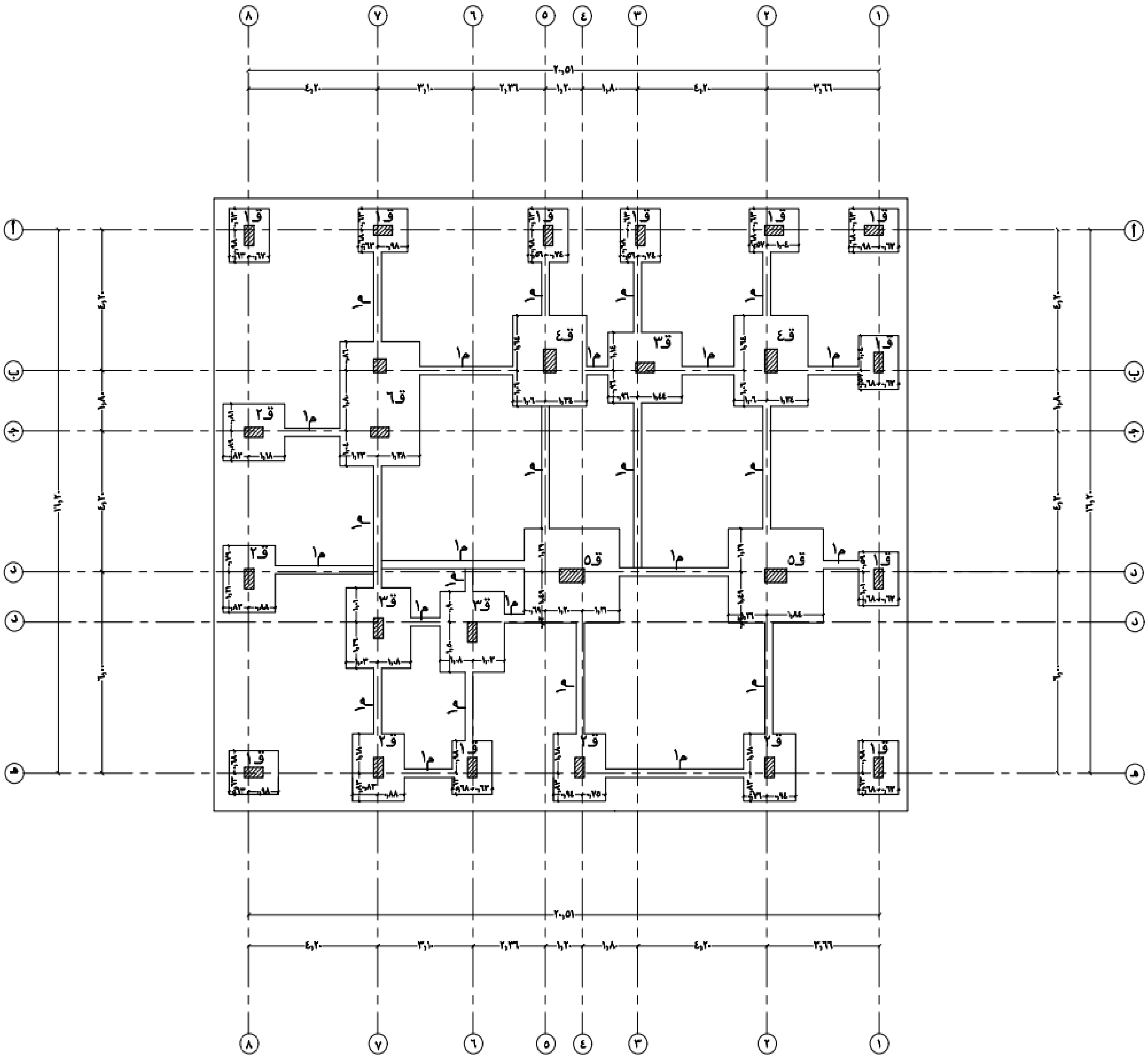


Figure 1.62 Foundations

1) Bearing Capacity for Soil = 15 t/m^2 = 1.5 kg/cm^2
2) Tensile Steel Stress Fy = 350
3) Compressive Concrete Stress Fcu = 25 N/mm^2

Foundations designed by reactions of columns from etabs program as shown in table:

ULTMATE Joint Reaction FZ Tonf	WORKING Joint Reaction FZ tonf	Joint Label	footing
57	39	1	ق1
92	64	2	ق2
77	54	3	ق1
120	83	4	ق2
127	88	5	ق2
45	31	6	ق1
80	56	7	ق1
260	180	8	ق5
242	167	9	ق5
143	99	10	ق3
175	121	11	ق3
97	67	12	ق2
94	65	13	ق2
167	116	14	ق6
127	88	15	ق6
206	142	16	ق4
148	103	17	ق3
186	129	18	ق4
76	53	19	ق1
36	25	20	ق1
61	42	21	ق1
73	51	22	ق1
83	58	23	ق1
88	61	24	ق1
58	40	25	ق1

Table 1.8 Actual columns loads and sections for foundations

For (4 ق)

1.1 Input Data:

- Column Working Load = 125 ton
- Column Dimension a = 40 cm
  b = 70 cm
- Plain Concrete Depth = 0.10 m
- Plain Concrete Extension = 0..10 m

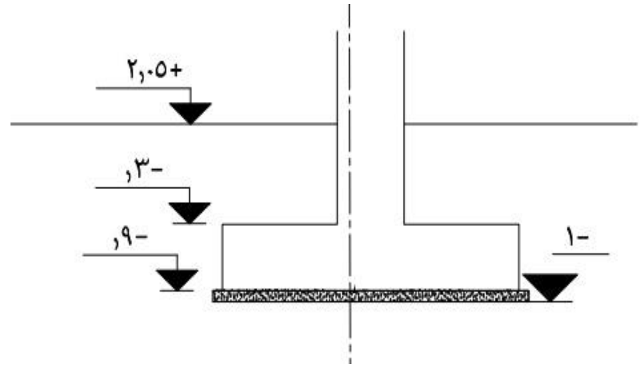
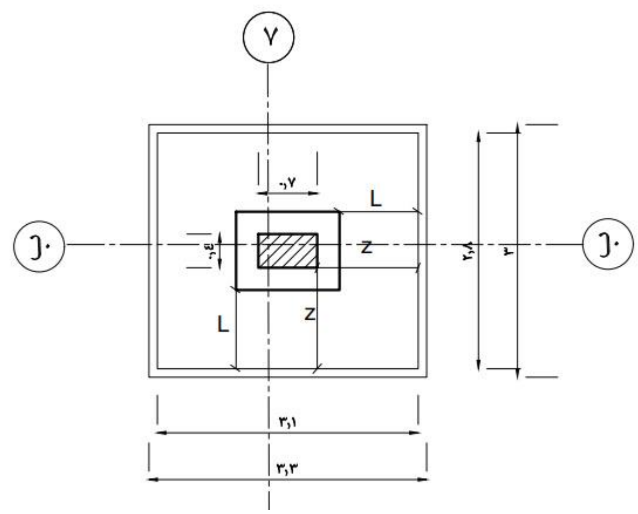


Figure 1.64 Section of footing (elevation)

1.2 Concrete Dimension :

- B.C = $\frac{Pw}{A_{pc}}$
- 150 = $\frac{1250}{A_{Rc}}$
- A_{Rc}= 8.333 m^2
- $L_{R.C} = \sqrt{A_{RC}} + \frac{b-a}{2} = \sqrt{8.33} + \frac{.7-.4}{2} = 3.1$
- $B_{R.C} = \sqrt{A_{RC}} - \frac{b-a}{2} = = \sqrt{8.33} - \frac{.7-.4}{2} = 2.8$
- $B_{R.C}$=5.5 m ⇨ $B_{R.C} = B_{R.C} + 2\ t_{P.C}$ =3 m
- $L_{R.C}$=6.3 m ⇨ $L_{R.C} = L_{R.C} + 2\ t_{P.C}$ =3.3 m



1.3 Check of Stress:

- $q_{act} = \frac{P_w}{A_{R.C}}$
- $q_{act} = \frac{1250}{3.1-2.8}$ = 144 KN/m^2 < B.C =150 KN/m^2 , Ok

$$q_{act} = \frac{P_U}{A_{R.C}} \frac{181}{3.1*2.8} = 208.255$$

Figure 1.65 Sections of footing (plan)

$$Z= \frac{3.1-.7}{2} = 1.2$$

$$M_U = \frac{q_{act} * Z^2}{2} = \frac{208.255*1.2^2}{2} = 149.943 \text{ KN.M}$$

d = $\sqrt{\frac{M_U}{F_{CU}*b}}$ = $\sqrt{\frac{149.943*10^6}{25*1000}}$ =387.22 ➡ d = 530 ➡ t = 600

1.4 Check of Punching:

Qp = Pco- q_{act} *(a+d) * (b+d)

- **Q**p = 1810 – 208.255*(.4+.6)*(.7+.6) = 1539.268
- qp = $\frac{Qp}{2(\,(a+d)+(a+d))*d}$
- qp = $\frac{1539.268*10^3}{2(\,(400+530)+(700+530))*530}$ = .67228

$q_{p\ (allow)}$= 1- 1.7

2- .316*(.5+$\frac{a}{b}$) * $\sqrt{\frac{F_{CU}}{\gamma_C}}$ = .316*(.5+$\frac{.4}{.7}$) * $\sqrt{\frac{25}{1.5}}$ = 1.3822

3- .316* $\sqrt{\frac{F_{CU}}{\gamma_C}}$ = .316* $\sqrt{\frac{25}{1.5}}$ = 1.29

4- .8*($\frac{\propto*d}{2(a+d+b+d)}$ +.2)* $\sqrt{\frac{F_{CU}}{\gamma_C}}$ =

= .8*($\frac{4*530}{2(400+530+700+530)}$ +.2)* $\sqrt{\frac{25}{1.5}}$ = 2.2559

qp < $q_{p\ (allow)}$ ➡ **OK, safe**

1.5 Check of Shear

- **For Sec (1-1)**
  L= 1.2 - $\frac{.53}{2}$ = .935 m

Qsh1 = q_{act} * L = 208.25 * .935 = 194.71 ton

qsh = $\frac{Qsh}{B*d}$ =$\frac{194.71*10^3}{1000*530}$ = .3673 KN / mm^2

$q_{sh(allow)}$ =.16* $\sqrt{\frac{F_{CU}}{\gamma_C}}$ = .16* $\sqrt{\frac{25}{1.5}}$ = .6532 KN / mm^2

$q_{sh} < q_{sh(allow)}$ OK .Safe

1.6 Design for Moment

$$A_S = \frac{M_U}{F_Y * J * d} = \frac{149.943 * 10^6}{350 * .76 * 530} = 1063.57 \text{ mm}^2$$

A_S = **8 ∯ 16**

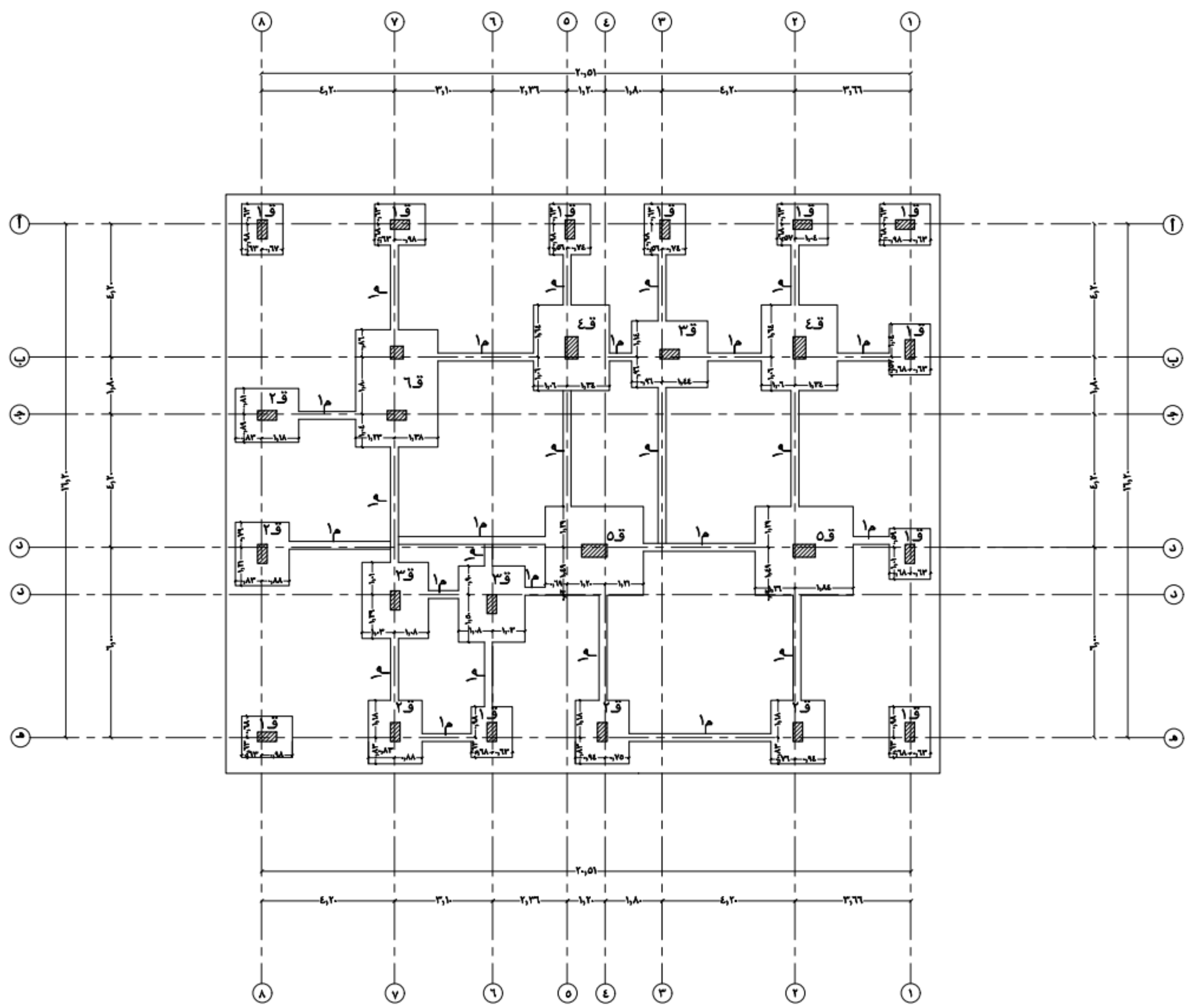


Figure 1.70 Foundations

جدول القواعد ...

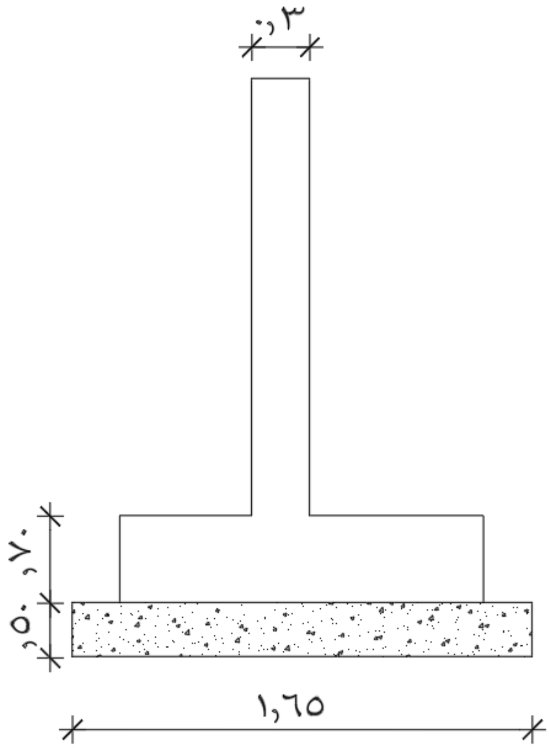
نموذج	ابعاد الخرسانة العادية			ابعاد الخرسانة المسلحة			التسليح السفلى		التسليح العلوى		ملاحظـــــات
	طول	عرض	سمك	طول	عرض	سمك	طويل	قصير	طويل	قصير	
ق ١	لبشة من الخرسانة العادية ٣٠ سم			١,٦	١,٣	٠,٦٠	١٢Ø٨	١٢Ø٨			
ق٢				٢	١,٧	٠,٦٠	١٢Ø٨	١٢Ø٨	—	—	
ق٣				٢,٤	٢,١	٠,٦٠	١٢Ø٩	١٢Ø٩	—	—	
ق٤				٢,٧	٢,٤	٠,٦٠	١٦Ø٦	١٦Ø٦	—	—	
ق٥				٣,١	٢,٨	٠,٦٠	١٦Ø٧	١٦Ø٧	—	—	
ق٦				٣,٧	٢,٦	٠,٦٠	١٢Ø٨	١٢Ø٨	١٦Ø٨	١٦Ø٨	

جدول نماذج و تسليح الميد والسملات ...

نموذج	قطاع	تسليح سفلى	تسليح علوى	كانات	ملاحظات
م	٦٠×٢٥	١٦Ø٤	١٦Ø٤	م /٨Ø٥	

1.8 Design of Retaining Wall

1.8.1 Loads



- Thicknees of wall= 30 cm
- Use Fill sand Φ_{30}
- $\gamma_{soil} = 1.5\ t/m^2$
- $K_a = \frac{1-\sin\emptyset}{1+\sin\emptyset} = 0.33$
- $q_{(sur\ charge)} = 0.1\ t/m^2$
- $F_1 = 0.1 * 0.33 * 2.1 = 0.77 t/m^2 \rightarrow 1.05m$
- $F_2 = 0.5 * 1 * 2.1 = 1.05 t/m^2 \rightarrow 0.7m$

1.8.2 Design of critical section (as uncracked section)

- $\mathrm{M}_{over} = 1.05 * 0.7 + 0.07 = 0.81\ t.m/m$
- $t = 30cm \qquad d = 27\ cm$
- $d = K_1\sqrt{\frac{M}{b}} \qquad 27 = K_1\sqrt{\frac{0.81*10^5}{100}} \qquad \rightarrow K_1 = 0.95$
- $K_2 = 1832$
- $A_s = \frac{M}{K^2 * d} = \frac{10.81*10^5}{1832*27} = 1.64\ cm^2$
- $UseA_{s\,min}\ 6\emptyset 12/m$ (Vertical) in both side
- $UseA_{s\,min}\ 6\emptyset 12/m$ (Horizontal) in both side

1.9 Design of Strip footing (under retaining wall)

1.9.1 Concrete dimensions

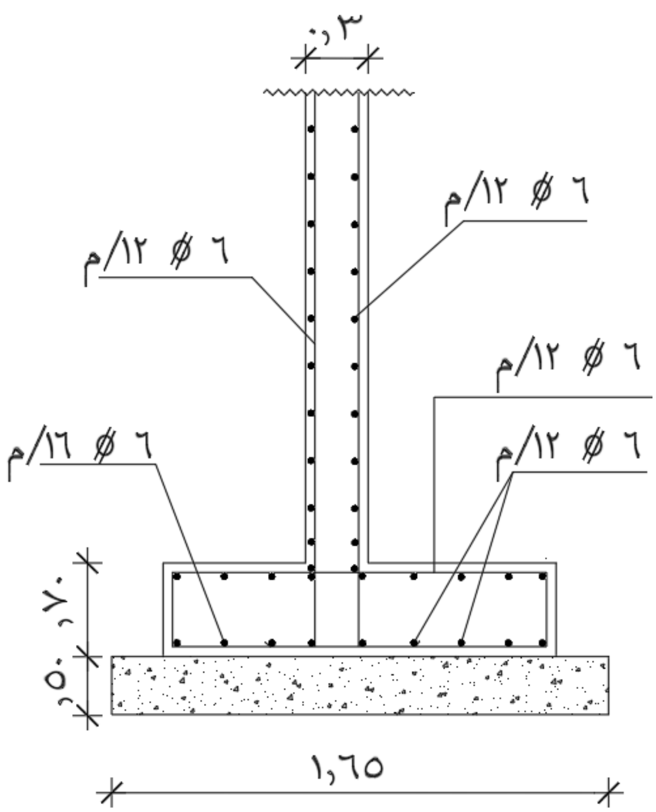
- H=2.30+0.7=3 m
- B=0.4H→ $0.7\mathrm{H} = 1.65\ m$

1.9.2 Loads

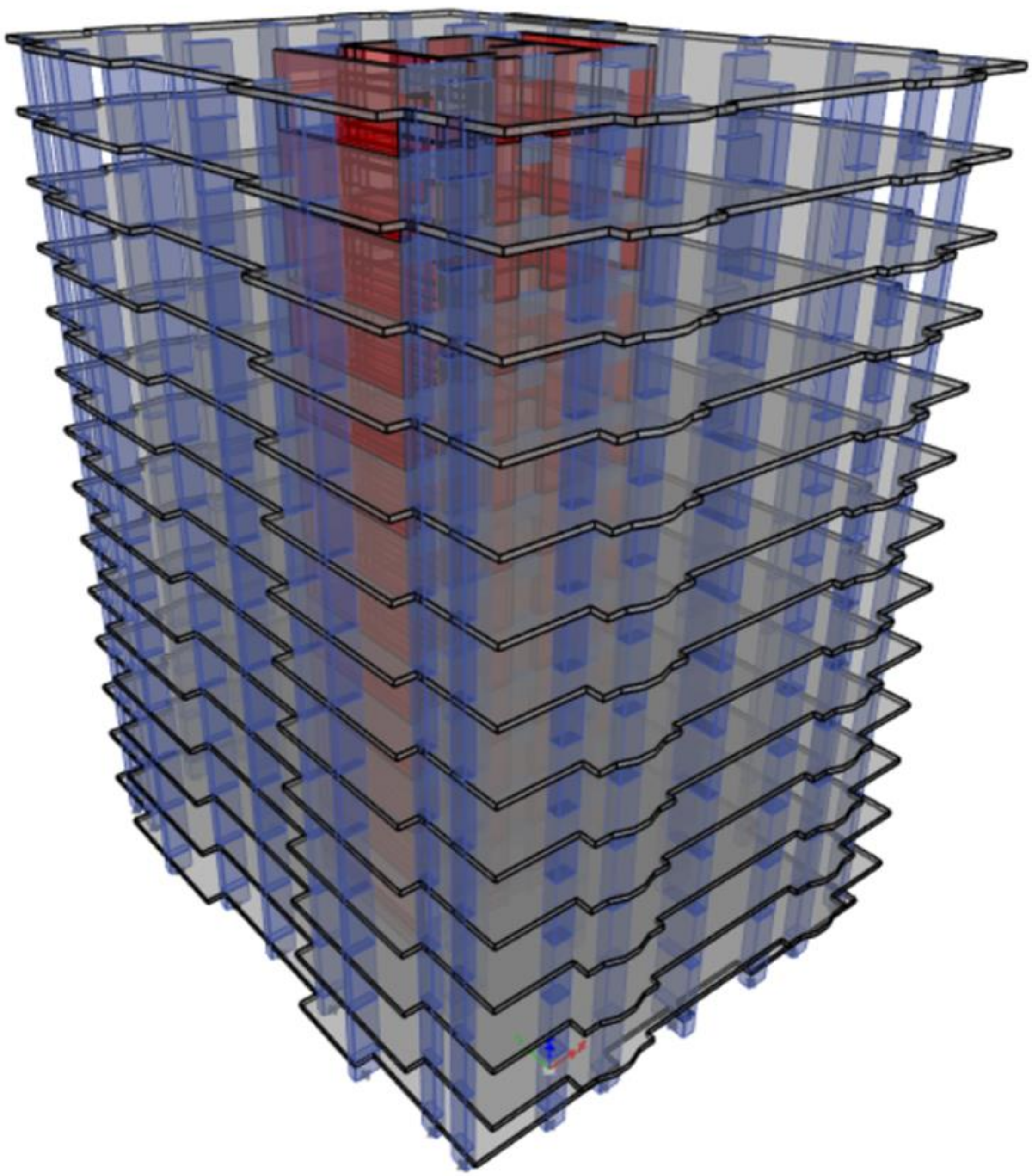
- ✓ $\mathrm{P_{wall}} = (2.5 * 0.3 * 2.3) + (1.8 * 2.1 * 0.525) + (2.5 + * \, 0.7 * 1.65) = 6.597 \; \mathrm{t/m}$
- ✓ $q_{act} = \frac{P_{wall}}{B_{R.W}} = \frac{6.597}{1.65} = 3.994 \frac{t}{m} < B.C = 10t/m^2$ OK Safe

1.9.3 Moment

- ➢ X=0.525 m
- ➢ $M_{max} = q_{act}\frac{x^2}{2} = 9.42 * \frac{0.525^2}{2} = 1.3 \; t/m$
- ➢ $d = K_1\sqrt{\frac{M_{max}}{b}}$
- ➢ $70 - 7 = K_1\sqrt{\frac{1.3*10^5}{100}}$
- ➢ $K_1 = 1.75$ $\qquad$ $K_2 = 1832$
- ➢ $A_s/M = \frac{M_{max}}{K^2 * d} = \frac{1.3*10^5}{1832*65} = 1.09 \; cm^2$
- ➢ $UseA_{s\,min} \; 6\emptyset 12/m$



Uinit (2)
High Rise Building



2.1 INTRODUCTION

2.1.1 High Rise Building Consists of:

- Basement of (2.55) m height
- Ground Floor of (3) m height
- 13 Typical Floors each (3) m height

2.1.2 Material Properties Used:

- F_{cu}=300 kg/cm^2
- $F_{y(main\ steel)}$=3500 kg/cm^2
- $F_{y(stirrups)}$=2400 kg/cm^2
- Weight of used brick =1500 kg/m^3
- Bearing Capacity of Soil = 1.5 kg/cm^2

2.1.3 Cover Thickness

- Slabs Cover = 2 cm
- Beams Cover = 2 cm
- Columns Cover = 2.5 cm
- Foundations Cover = 5 cm
- Stairs Cover = 2 cm
- Ramp Cover = 2 cm

2.1.4 Loads Used:

- L.L= According to every Floor
- Cover = 0.2 ton
  - 0.05 * 1.5 رمل تسوية بسمك 5 سم
  - 0.01 * 2.1 مونة أسمنتيه بسمك 1 سم
  - 0.02 * 2.1 بلاط سيراميك بسمك 2 سم
  - 0.02 * 2.1 محارة أسفل البلاطه بسمك 2 سم
- Wall = According to every Floor
- D.L = Own weight + Covering Material + Wall Load

2.1.5 Design Method:

- Ultimate limit state design

2.1.6 Computer Programs Used in Analysis :

- (Etabs + Safe + SAP2000 + CSI Column + Excel)

2.1.7 Design Code:

- Egyptian code of practice 201

2.2 DESIGN OF SLABS:

2.2.1 Basement Slab: (Flat Slab System)

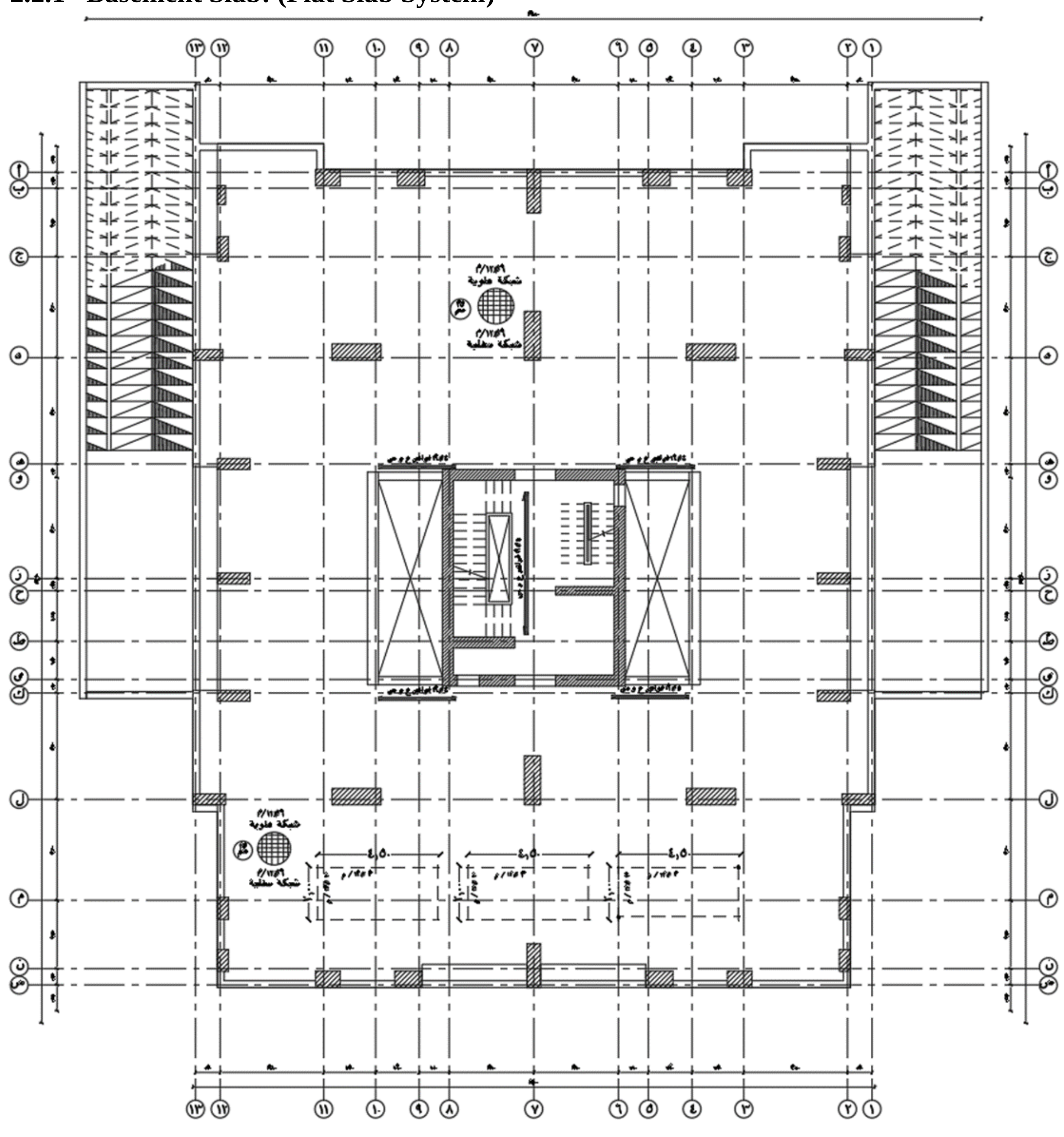


Figure 2.1 Statical System of Basement Roof

- Slab Thickness = 22 cm
- Own weight = 0.22* 2.5 = 0.55 t/m^2.
- Covering = 150 kg/m^2 = 0.15 t/m^2.
- Live load = 300 kg/m^2 = 0.30 t/m^2
- Wall load = 200 kg/m^2 = 0.20 t/m^2.

Solving This flat slab By Using CSI Safe program:

- D.L = O.W + W_{wall} + Covering material
  = 0.55+ 0.20 + 0.15 = 0.90 t/m^2
- L.L = 300 kg/cm^2 = 0.30 t/m^2
- W_u = 1.4 D.L + 1.6 L.L = 1.74 t/m^2

For ultimate design:-

- $As = \left[\frac{Mu}{Fy*J*d}\right]$
- $M_u = As * F_y * J * d = 6*\left(\frac{\pi*(1.2)^2}{4}\right) * 3500 * 0.826 * 20 * (10)^{-5}$
- M(r)= 3.92 t.m ⇛ Use 6 ∯ 12 / m in each Direction
- Additional RFT (3 ∯ 12 /m) & (6 ∯ 12 / m) upper and lower

In X-Direction: (Lower)

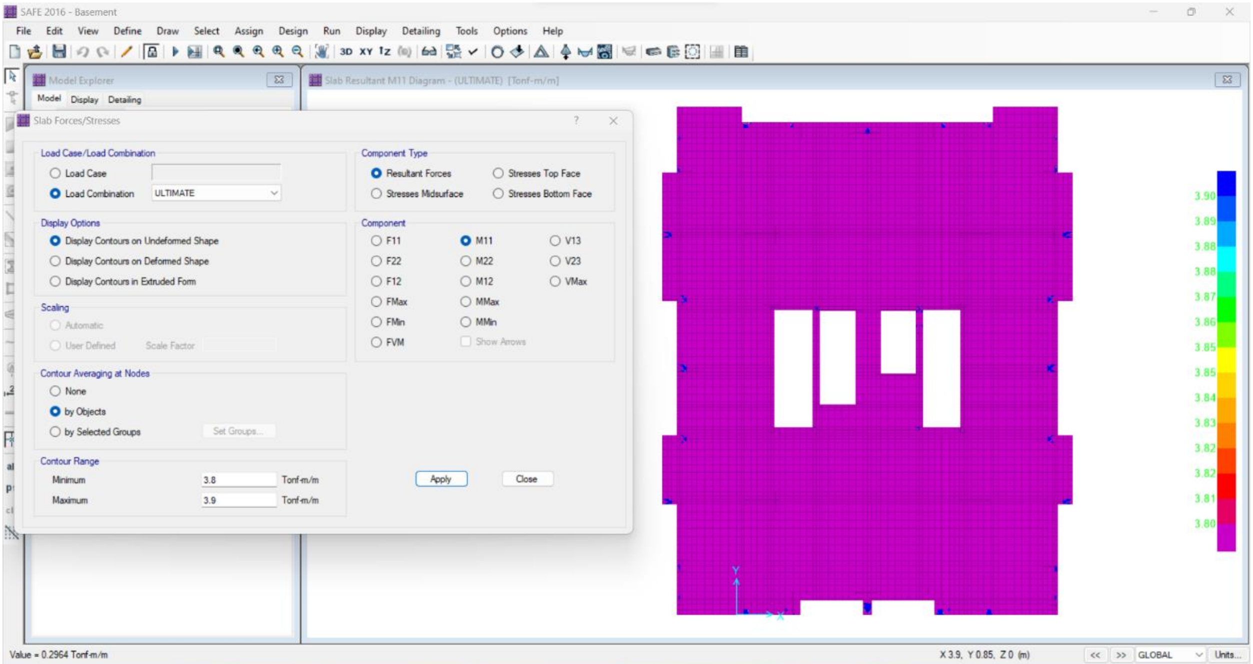


Figure 2.2 Additional Reinforcement in X-Direction (Lower)

In X-Direction: (Upper)

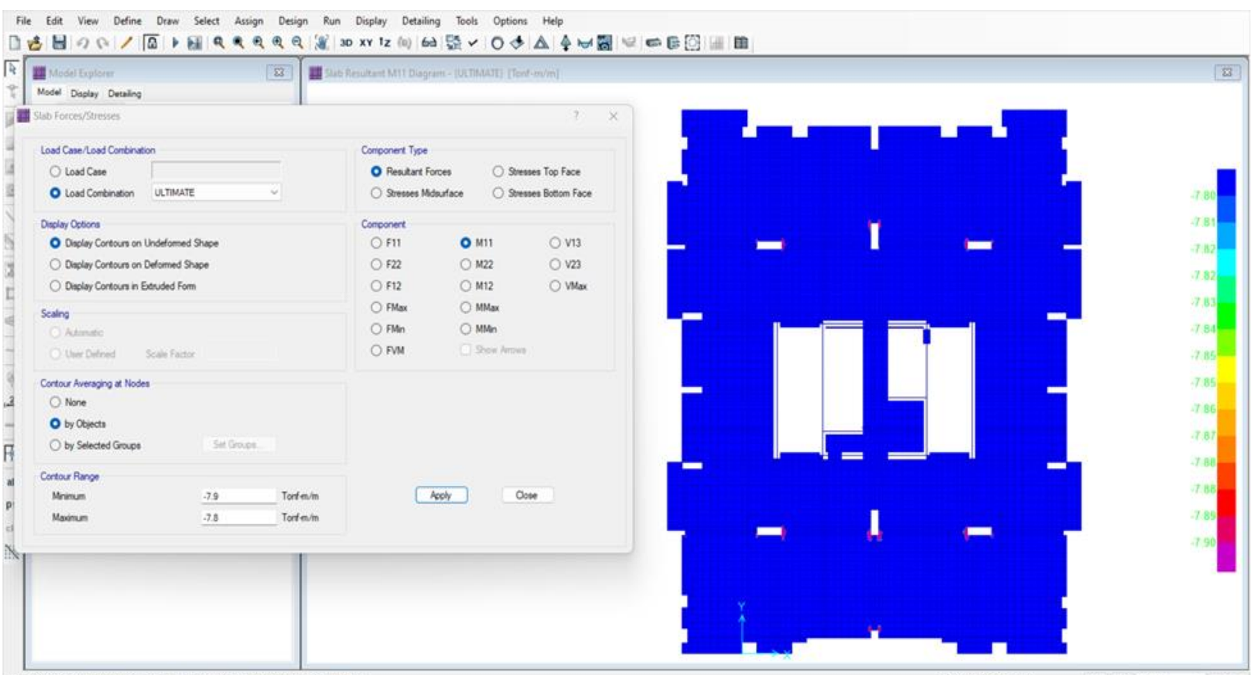


Figure2.3 Additional Reinforcement in X-Direction (Upper)

In Y-Direction (Lower):

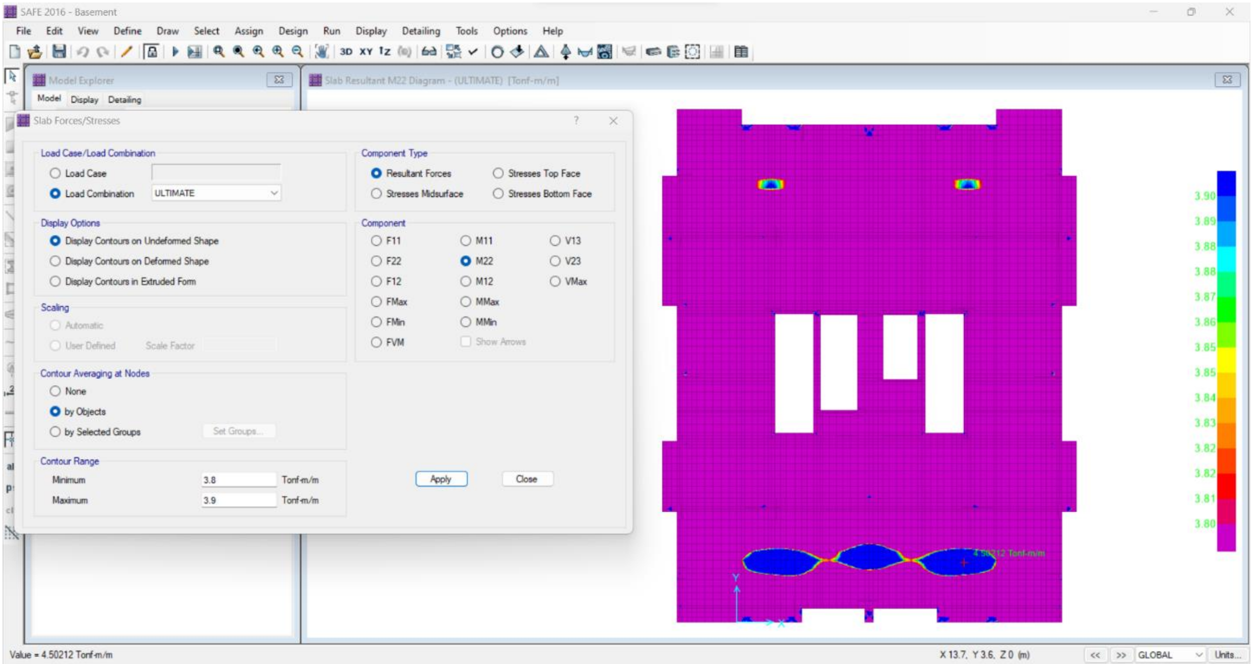


Figure 2.4 Additional Reinforcement in Y-Direction (Lower)

In Y-Direction (Upper):

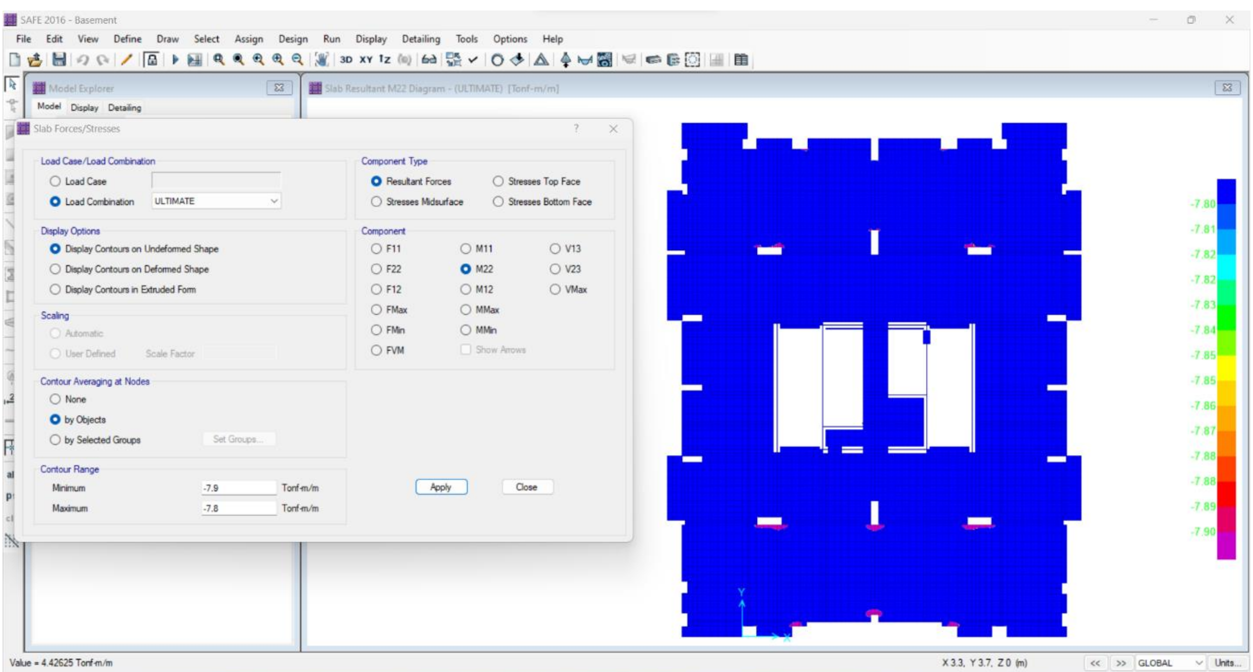


Figure 2.5 Additional Reinforcement in Y-Direction (Upper)

2.2.1.1 Check for All Loads

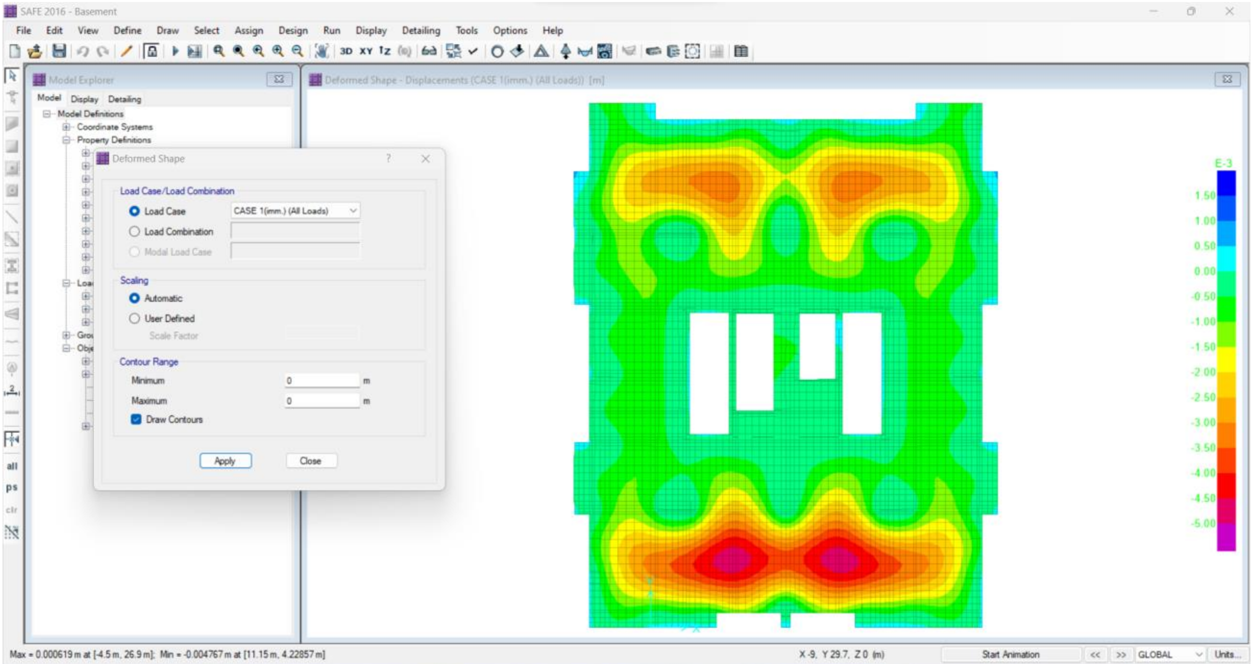


Figure2.6 All Loads

- From Code Check = L/250
- Span for Check =4.13 m
- Allowable Deflection =0.0165 m
- Maximum Deflection = 0.000179 m

2.2.1.2 Check for Total Long Term Deflection

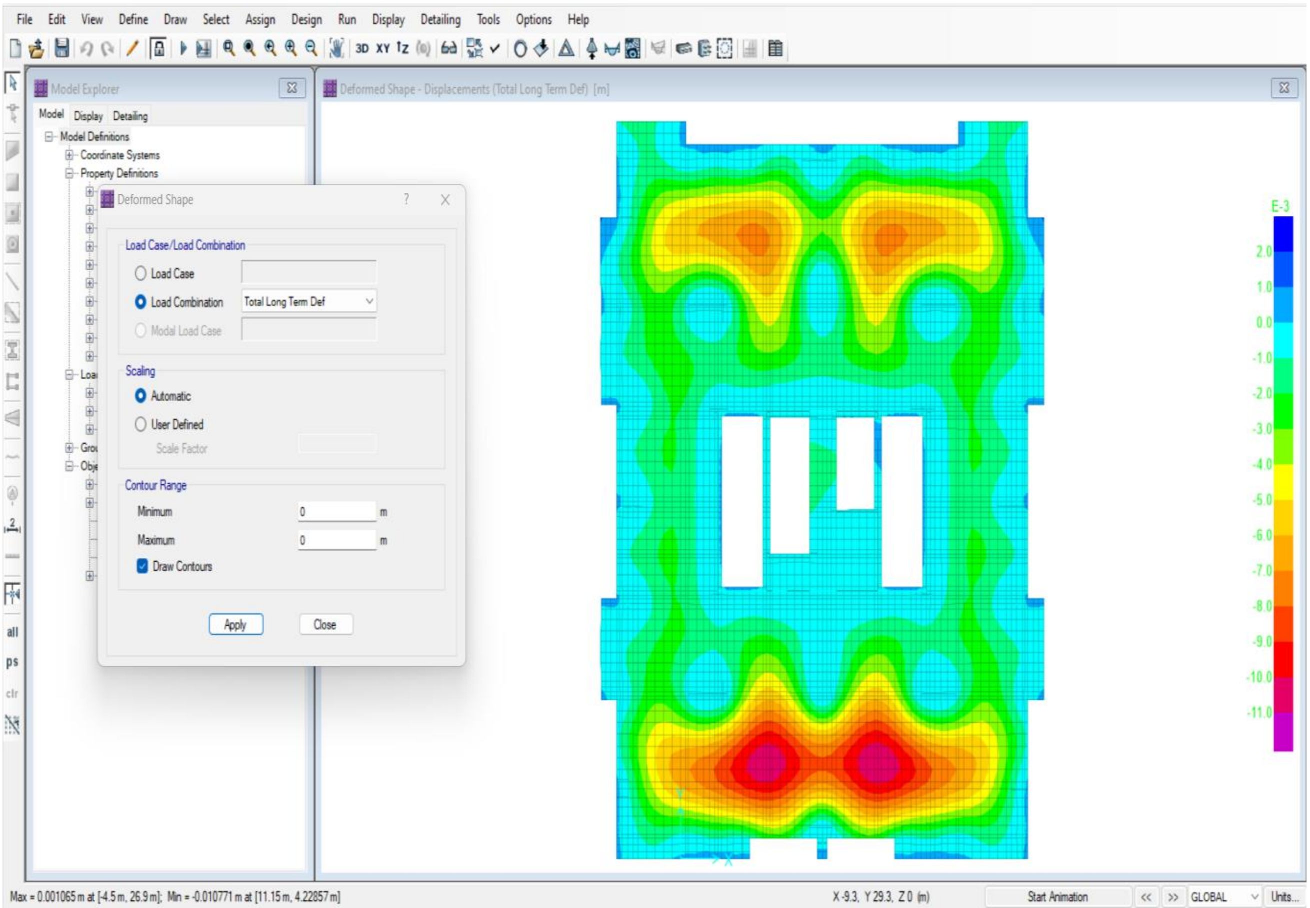


Figure 2.7 Total Long Term Deflection

- From Code Check = L/250
- Span for Check =4.13 m
- Allowable Deflection =0.0165
- Maximum Deflection = 0.000154 m

2.2.1.3 Check for Total Dead Loads

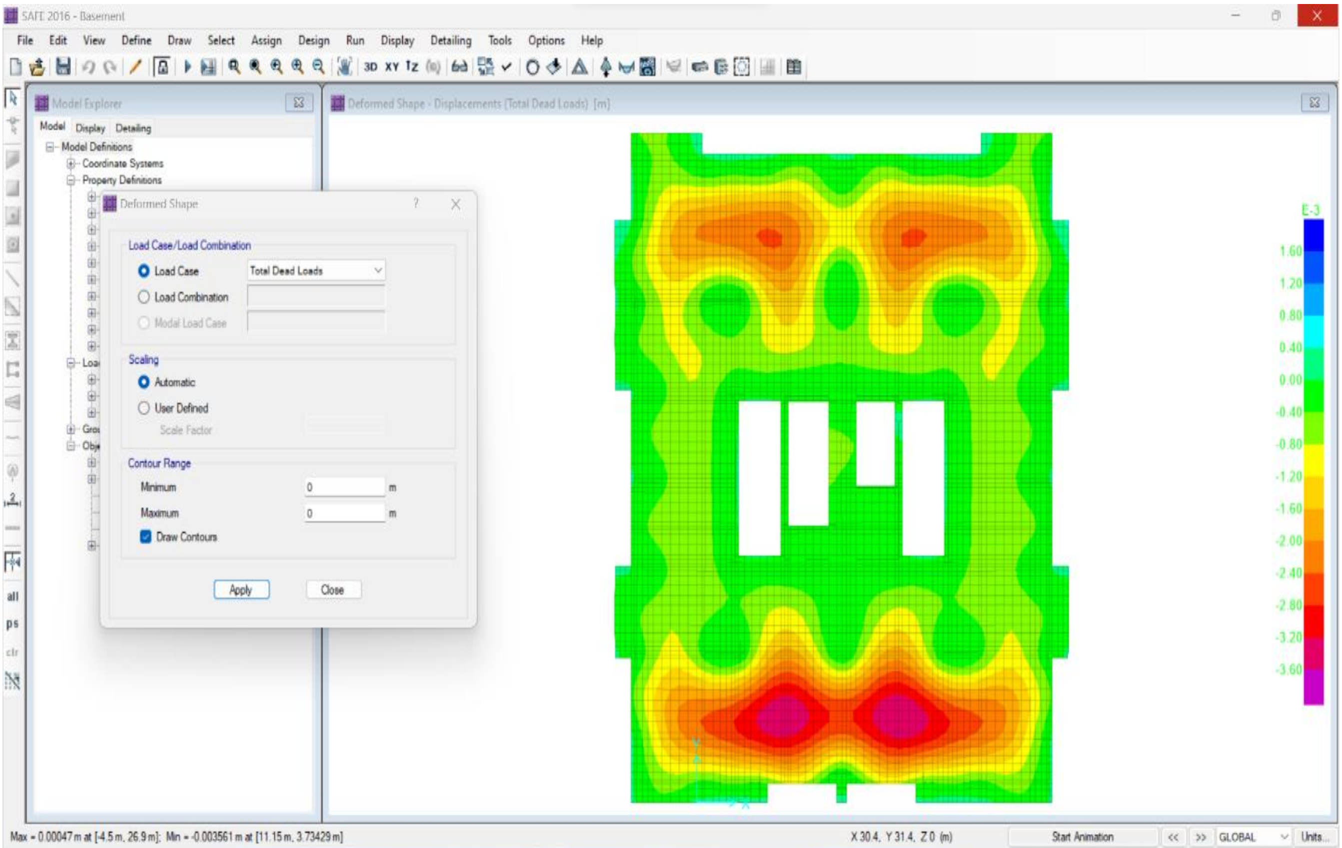


Figure 2.7 Total Dead Loads

- From Code Check = L/250
- Span for Check =4.13 m
- Allowable Deflection =0.0165
- Maximum Deflection = 0.000154 m

2.2.2 Ground Slab (Flat Slab System)

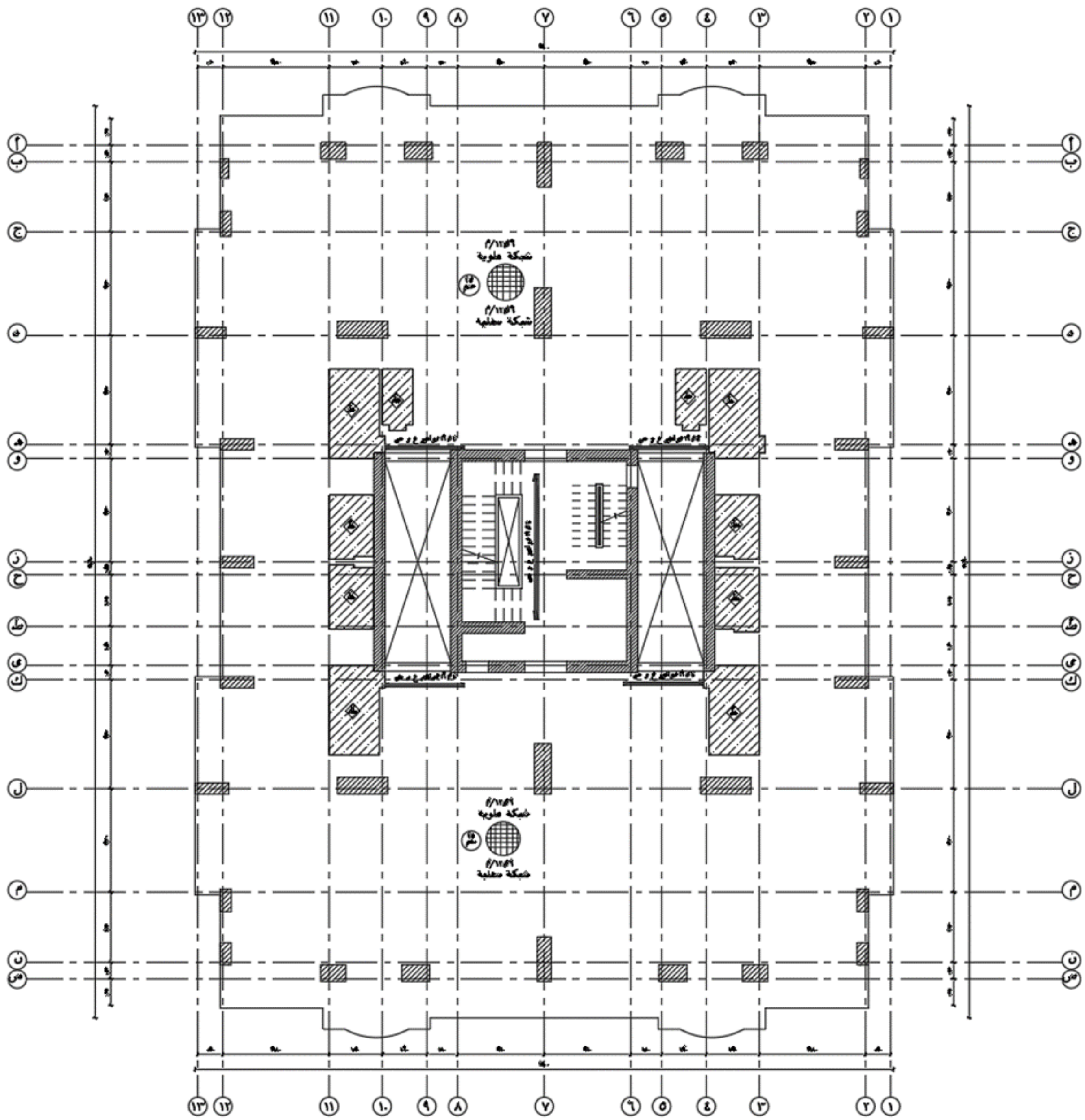


Figure 2.8 Statical System of Ground Roof

- Slab Thickness = 22 cm
- Own weight = 0.22* 2.5 = 0.55 t/m^2.
- Covering = 150 kg/m^2 = 0.15 t/m^2.
- Live load = 250 kg/m^2 = 0.25 t/m^2
- Wall load = 200 kg/m^2 = 0.20 t/m^2.

Solving This flat slab By Using CSI Safe program:

- D.L = O.W + W_{wall} + Covering material
  = 0.55 + 0.20 + 0.15 = .90 t/m^2
- L.L = 250 kg/cm^2 = 0.25 t/m^2
- W_u = 1.4 D.L + 1.6 L.L = 1.66 t/m^2

For ultimate design:-

- $As = \left[\frac{Mu}{Fy*J*d}\right]$
- $M_u = As * F_y * J * d = 5*\left(\frac{\pi*(1.2)^2}{4}\right) * 3500 * 0.826 * 20 * (10)^{-5}$
- M(r)= 3.27 t.m ⇛ Use 5 ∯ 12 / m in each Direction
- Additional RFT (2.5 ∯ 12 /m) & (5 ∯ 12 / m) upper and lower

In X-Direction: (Lower)

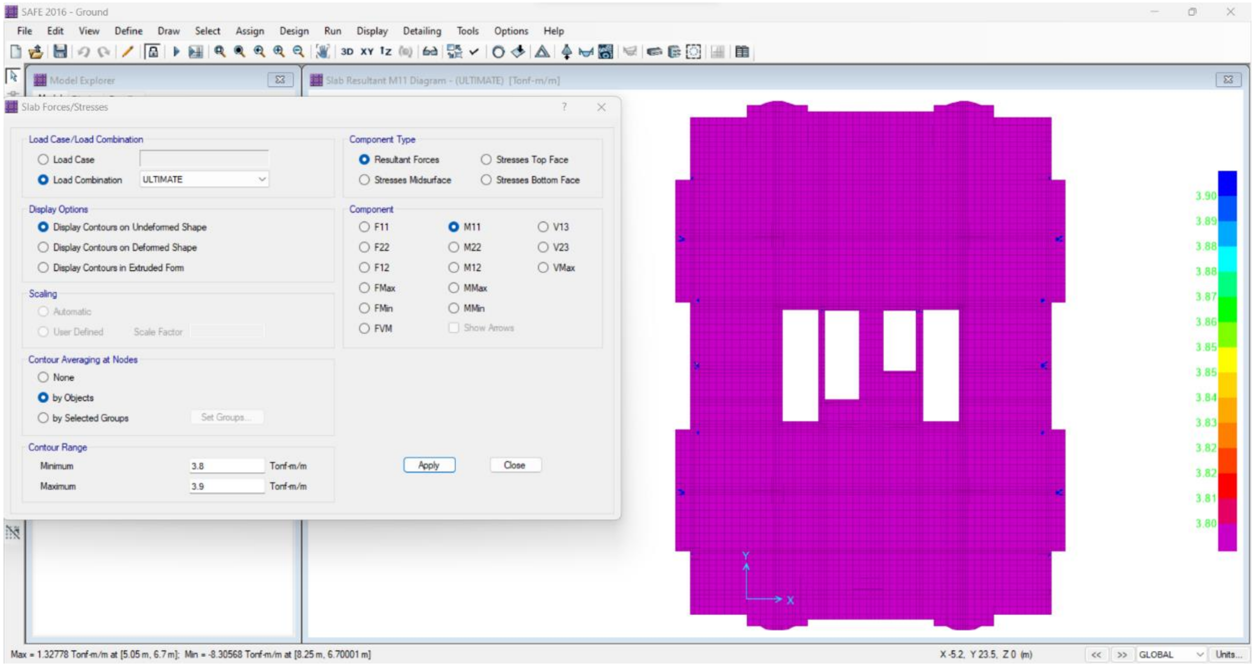


Figure 2.9 Additional Reinforcement in X-Direction (Lower)

In X-Direction: (Upper)

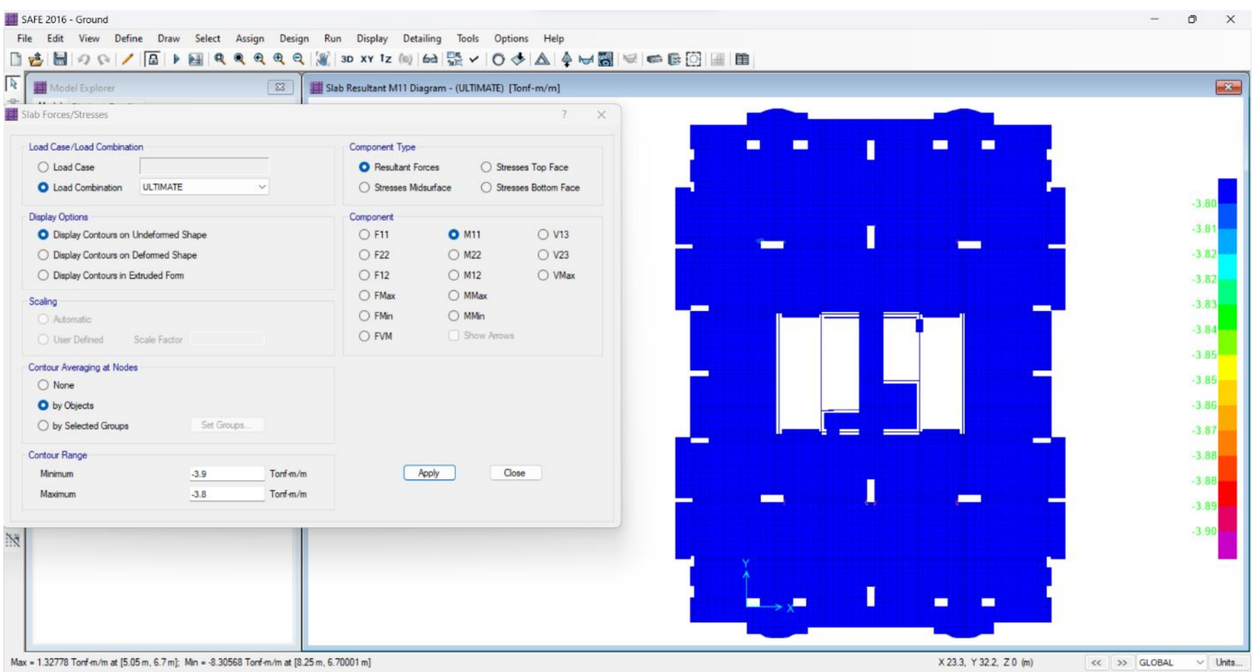


Figure 2.10 Additional Reinforcement in X-Direction (Upper)

In Y-Direction: (Lower)

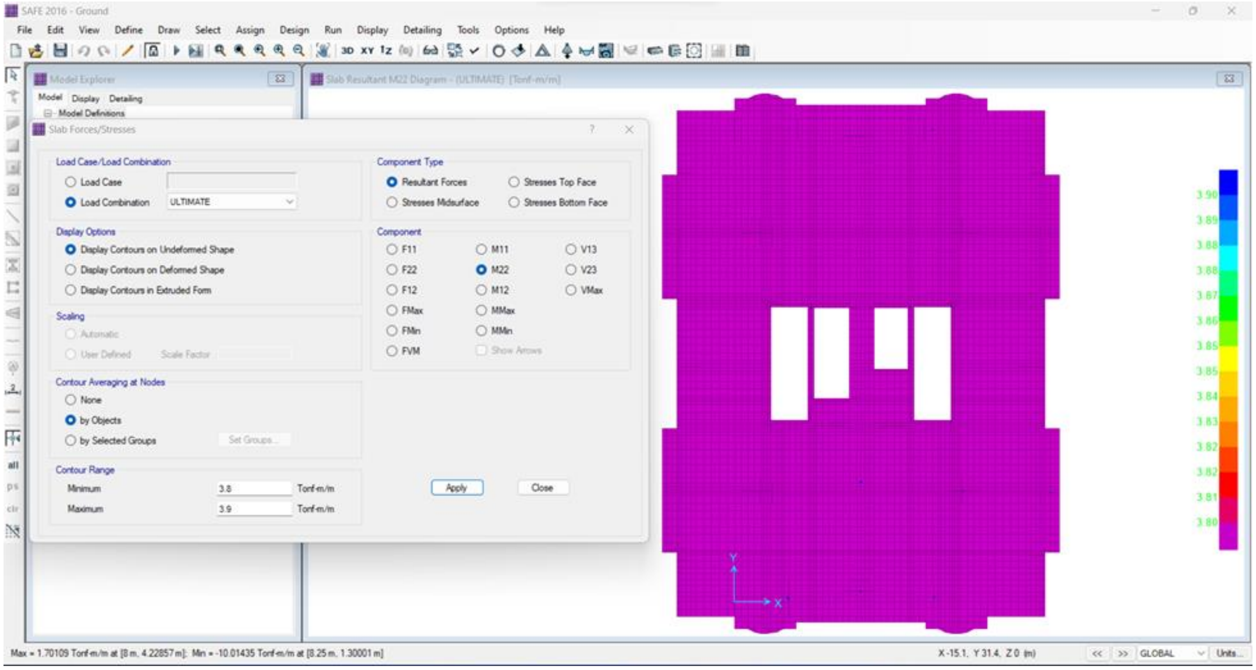


Figure2.11 Additional Reinforcement in Y-Direction (Lower)

In Y-Direction: (Upper)

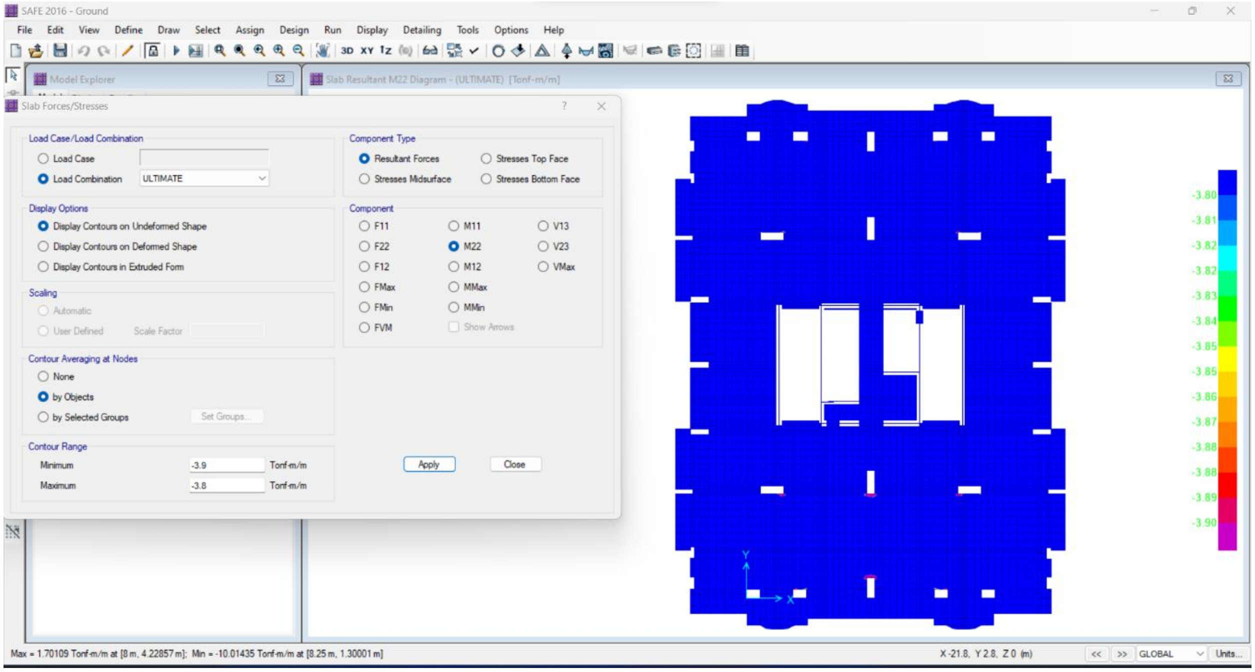


Figure 2.12 Additional Reinforcement in Y-Direction (Upper)

2.2.2.1 Check for All Loads Deflection:

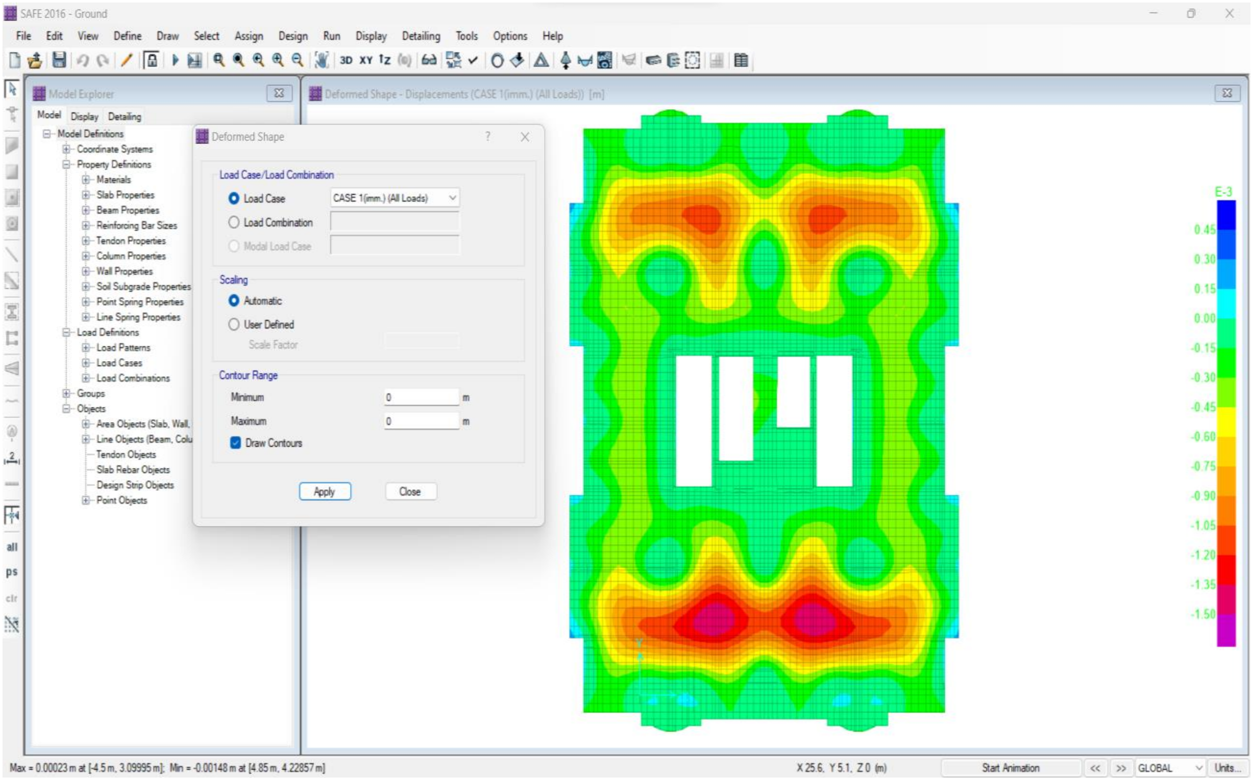


Figure 2.13 All Loads Deflection

- From Code Check = L/250
- Span for Check =4.53 m
- Allowable Deflection =0.0181 m
- Maximum Deflection = 0.000382 m

2.2.2.2 Check for Total Long Term Deflection:

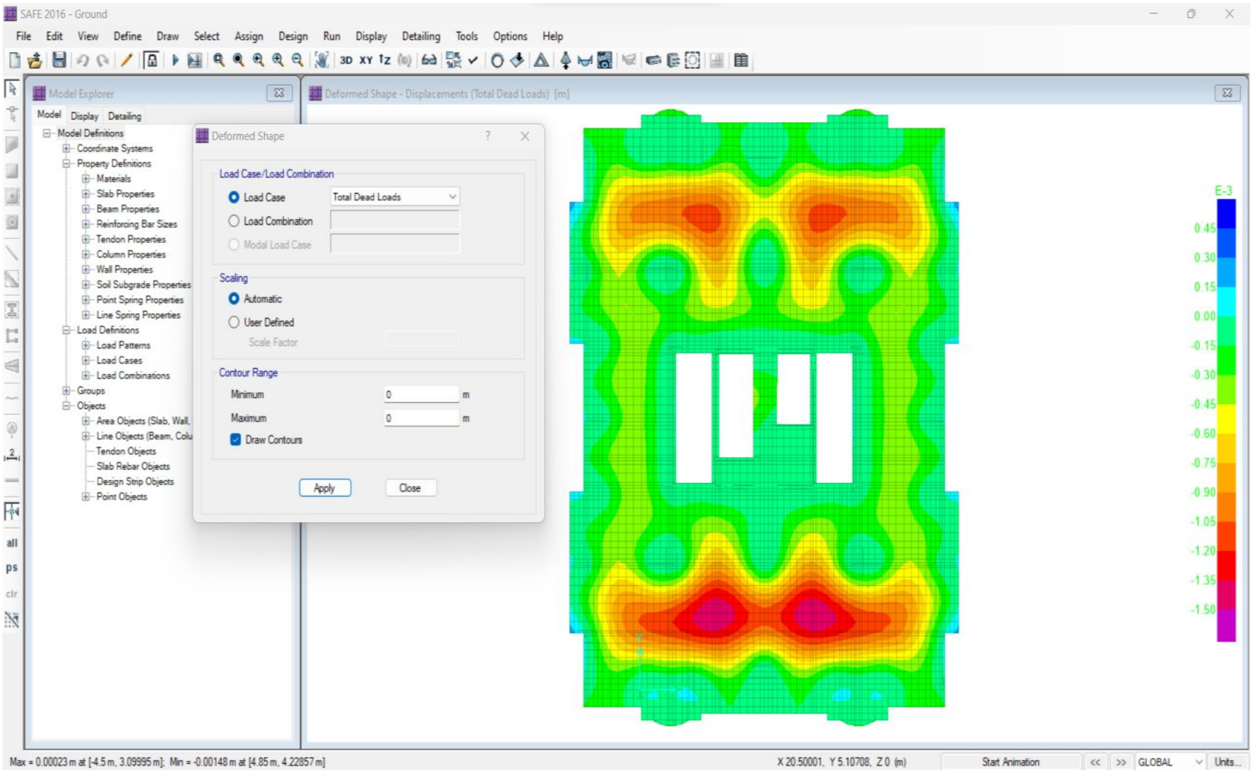


Figure 2.14 Total Long Term Deflection

- From Code Check = L/250
- Span for Check =4.53 m
- Allowable Deflection =0.0181 m
- Maximum Deflection = 0.000359m

2.2.2.3 Check for Total Dead Loads Deflection:

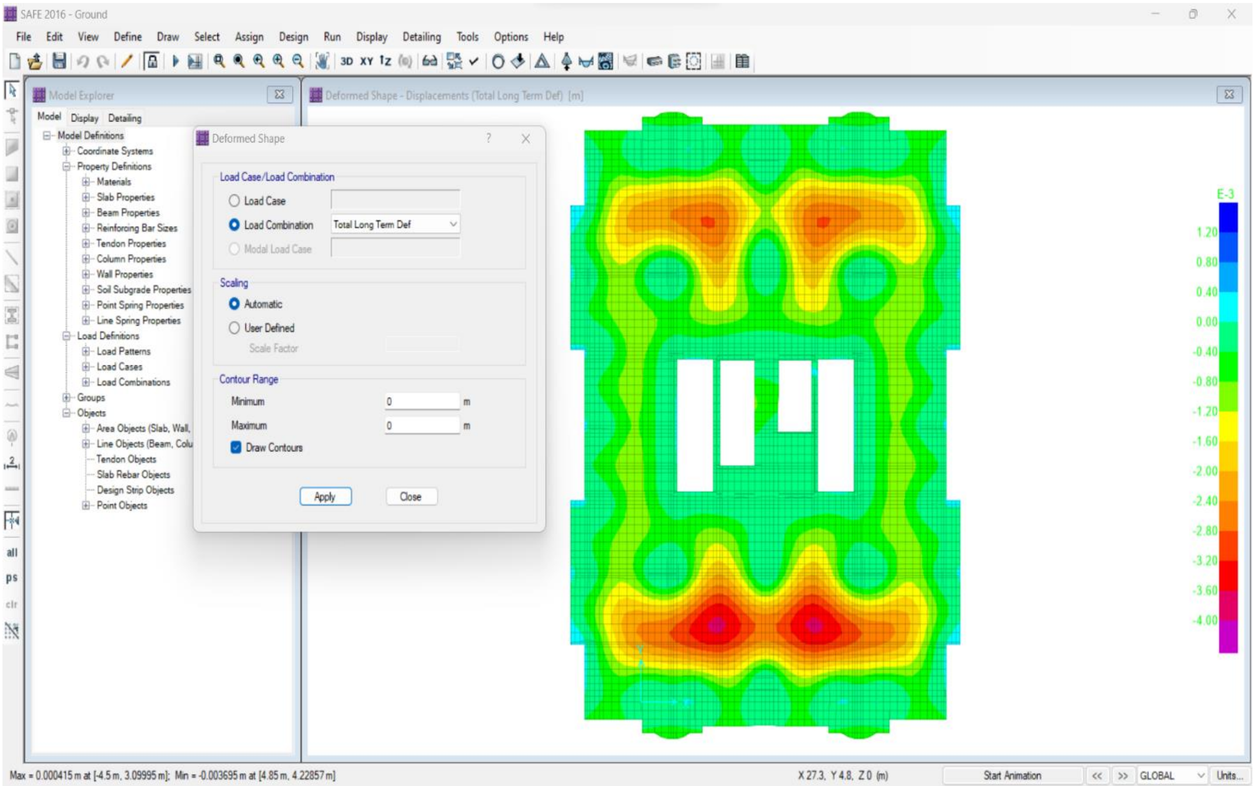


Figure 2.14 Total Dead Loads Deflection

- From Code Check = L/250
- Span for Check =4.53 m
- Allowable Deflection =0.0181 m
- Maximum Deflection = 0.000359m

2.2.3 Repeated Slab (Flat Slab System)

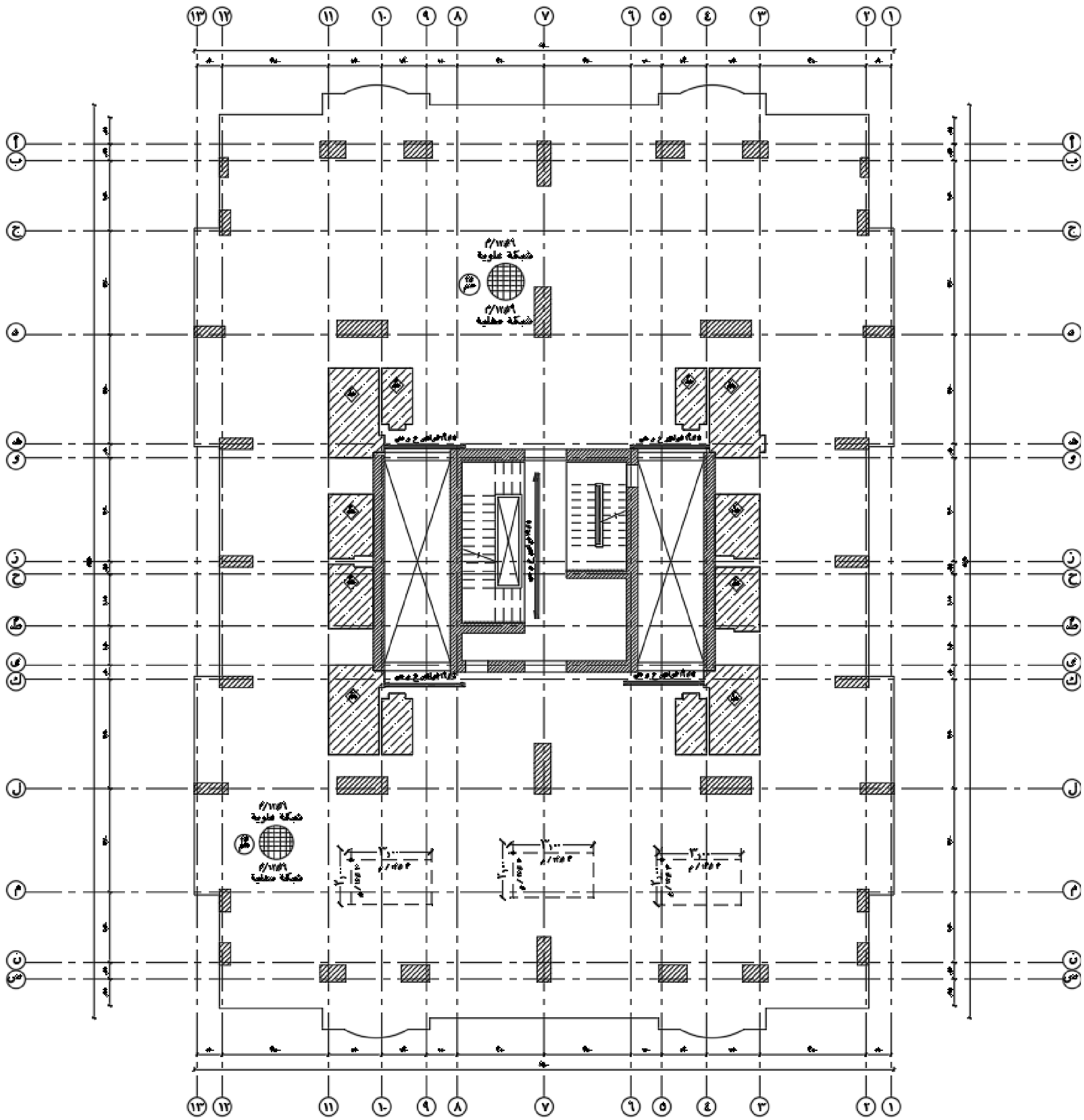


Figure 2.15 Statical System of Rppeated Roof

- Slab Thickness = 22 cm
- Own weight = 0.22* 2.5 = 0.55 t/m^2.
- Covering = 150 kg/m^2 = 0.15 t/m^2.
- Live load = 250 kg/m^2 = 0.25 t/m^2
- Wall load = 200 kg/m^2 = 0.2 t/m^2.

Solving This flat slab By Using CSI Safe program:

- D.L = O.W + W_{wall} + Covering material
  = 0.55 + 0.20 + 0.15 = 0.9 t/m^2
- L.L = 250 kg/cm^2 = 0.25 t/m^2
- W_u = 1.4 D.L + 1.6 L.L = 1.66 t/m^2

For ultimate design:-

- $As = \left[\frac{Mu}{Fy*J*d}\right]$
- $M_u = As * F_y * J * d = 5*\left(\frac{\pi*(1.2)^2}{4}\right)*3500*0.826*20*(10)^{-5}$
- M(r)= 3.27 t.m ⇛ Use 5 ∯ 12 / m in each Direction
- Additional RFT (2.5 ∯ 12 /m) & (5 ∯ 12 / m) upper and lower

In X-Direction: (Lower)

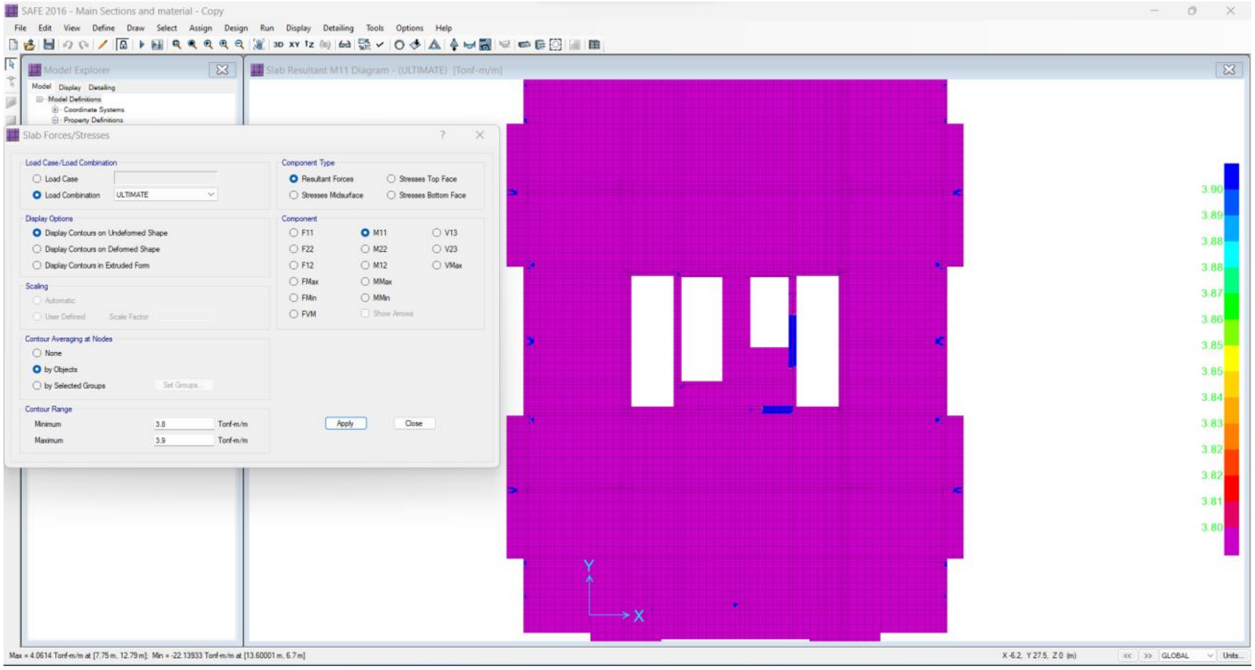


Figure 2.16 Additional Reinforcement in X-Direction (Lower)

In X-Direction: (Upper)

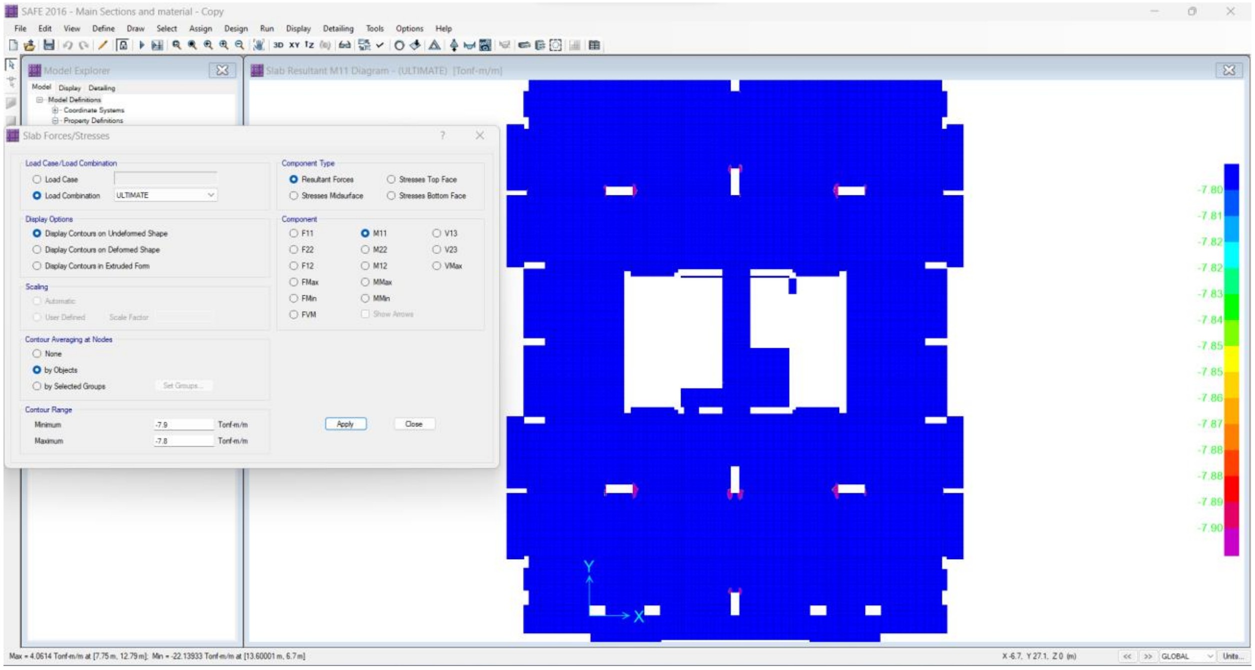


Figure 2.17 Additional Reinforcement in X-Direction (Upper)

In Y-Direction: (Lower)

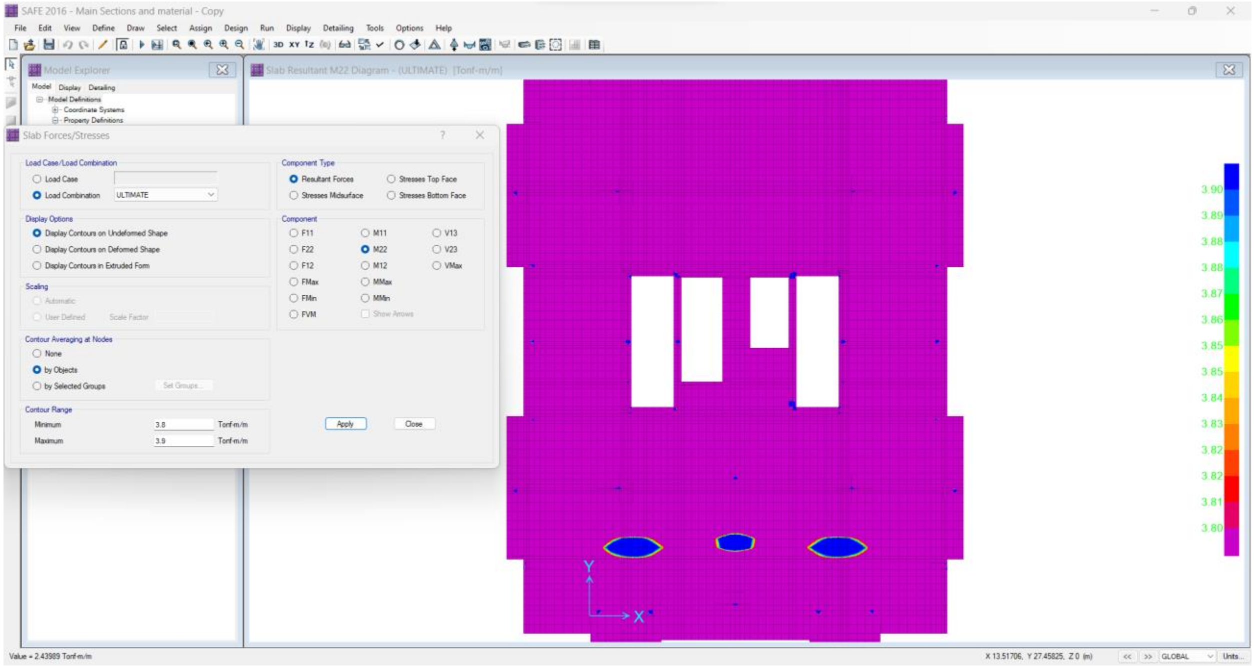


Figure2.18 Additional Reinforcement in Y-Direction (Lower)

In Y-Direction: (Upper)

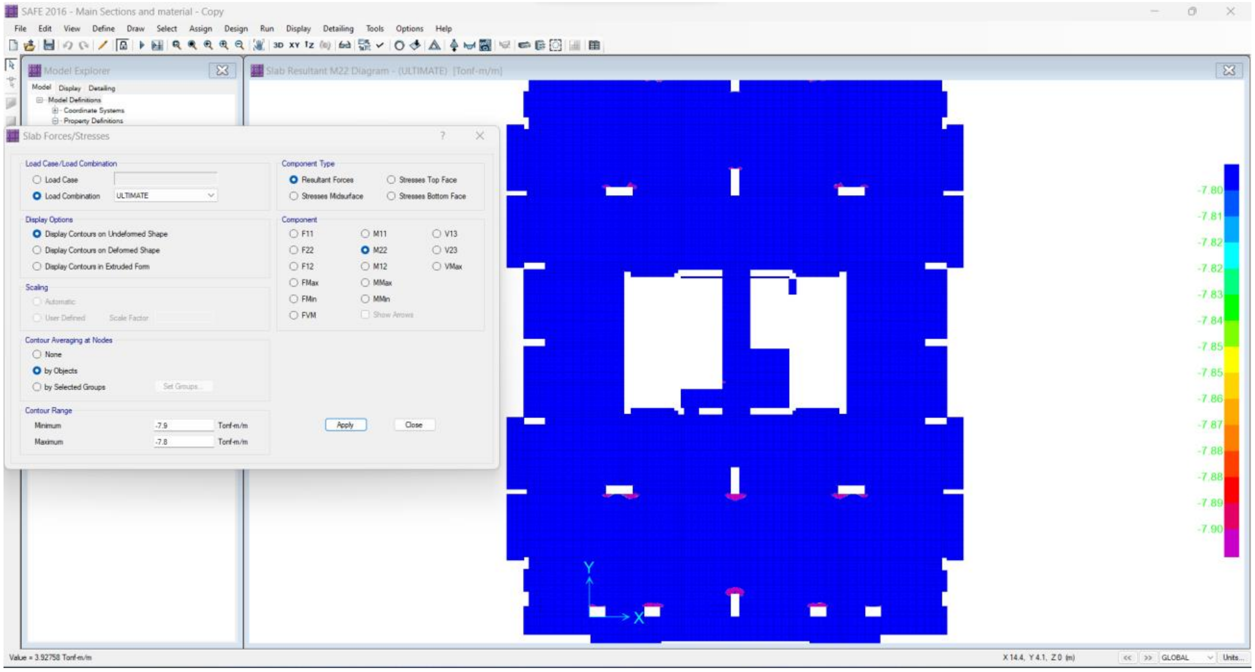


Figure 2.19 Additional Reinforcement in Y-Direction (Upper)

2.2.2.1 Check for All Loads Deflection:

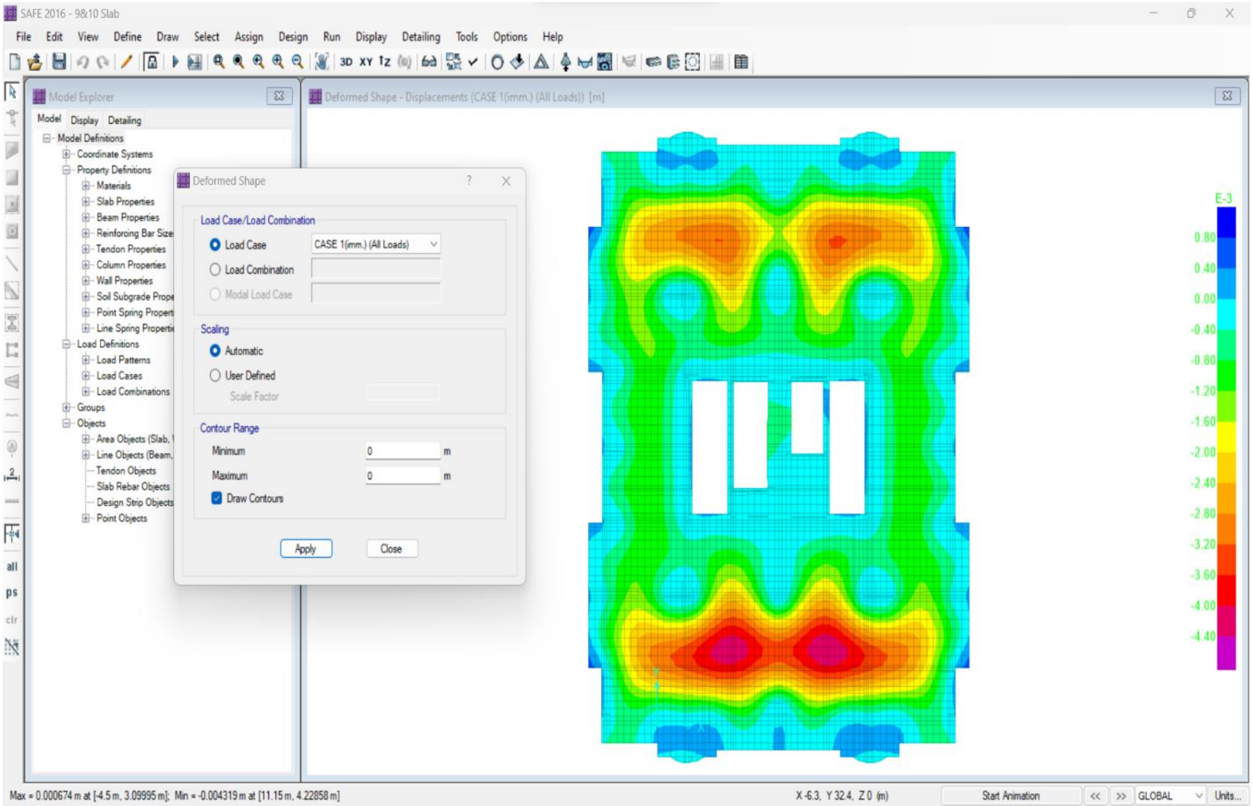


Figure 2.20 All Loads Deflection

- From Code Check = L/250
- Span for Check = 4.53m
- Allowable Deflection =0.0181 m
- Maximum Deflection = 0.000382 m

2.2.2.2 Check for Total Dead Loads Deflection:

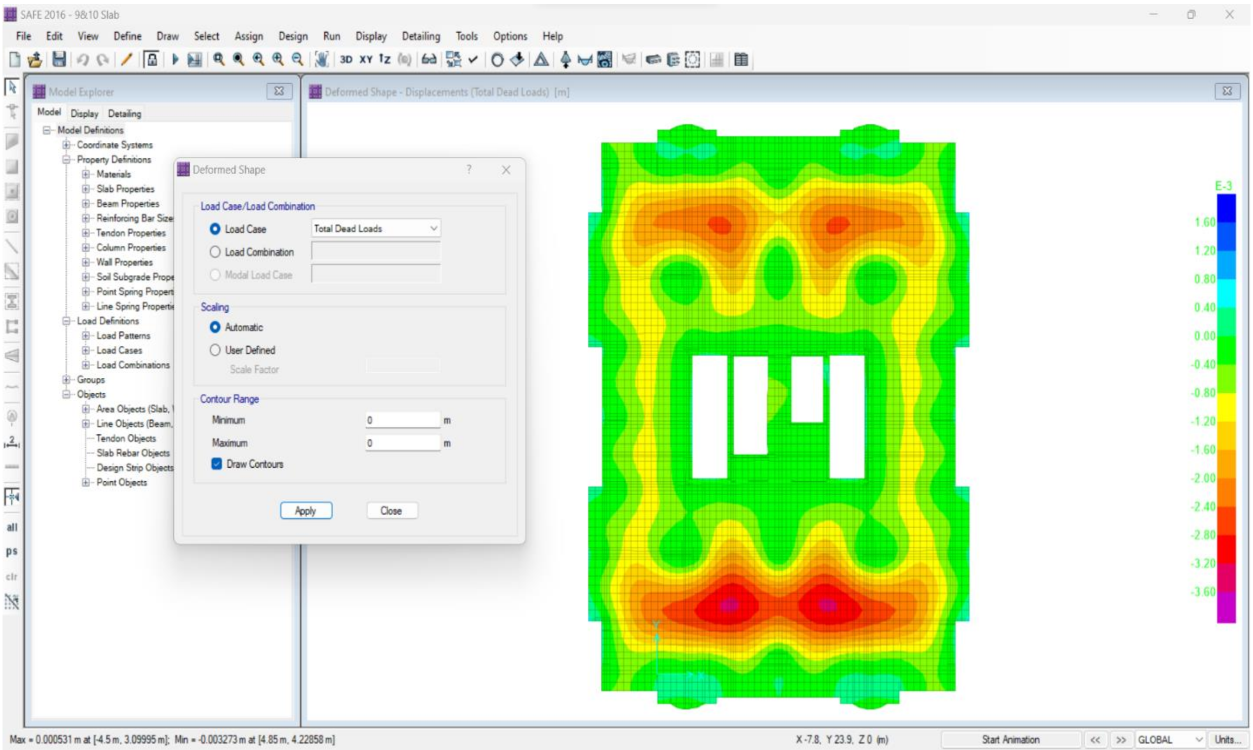


Figure 2.21 Total Dead Loads Deflection

- From Code Check = L/250
- Span for Check = 4.53m
- Allowable Deflection = 0.0181m
- Maximum Deflection = 0.00359 m

2.2.2.2 Check for Total Long Term Deflection:

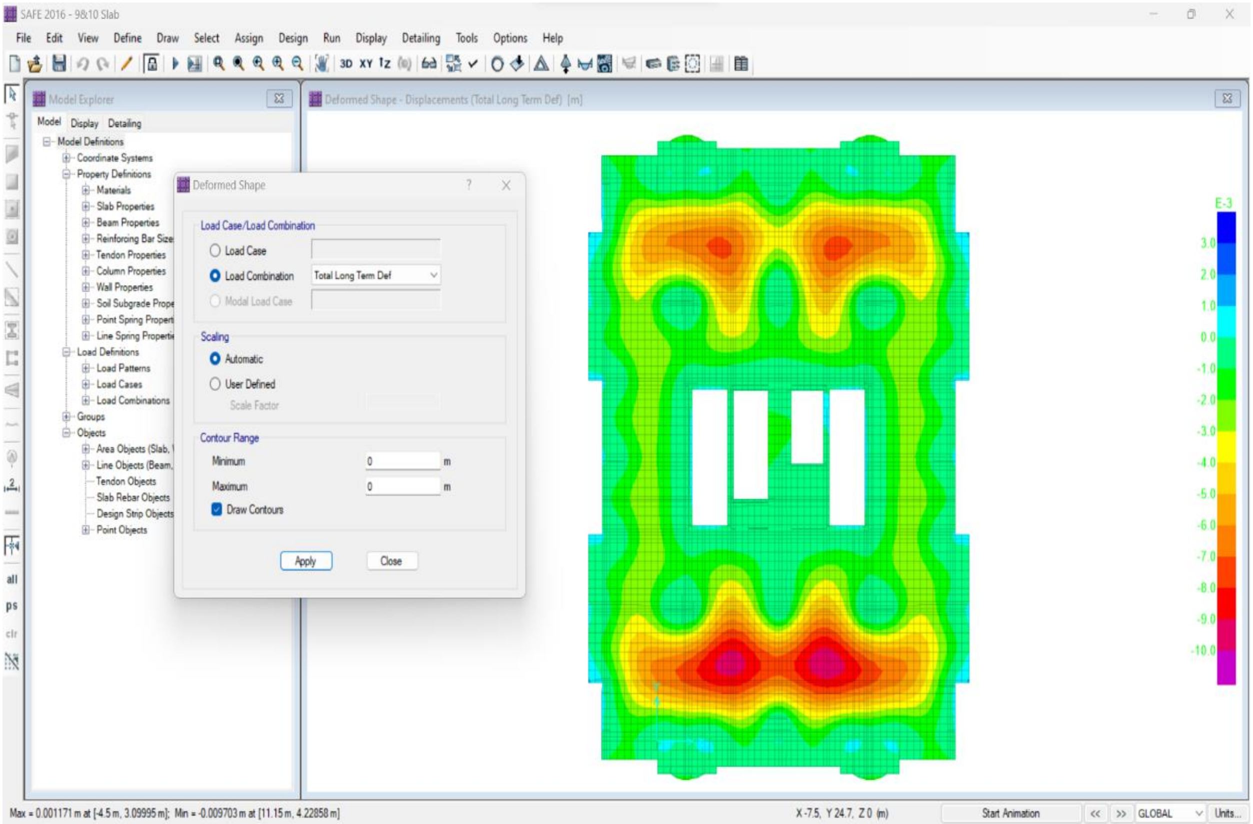
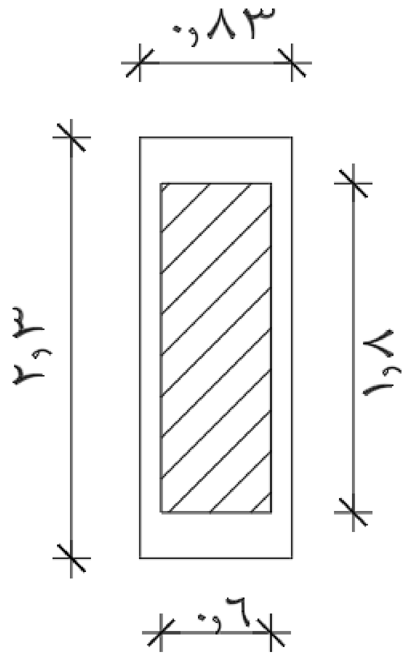


Figure 2.21 Total Long Term Deflection

- From Code Check = L/250
- Span for Check = 4.53m
- Allowable Deflection = 0.0181m
- Maximum Deflection = 0.00359 m



2.2.4 Check of Punching Shear: (Basement Roof)

2.2.4.1 Interior Column (C10 = 60*180) on (2 – ب) Axis:

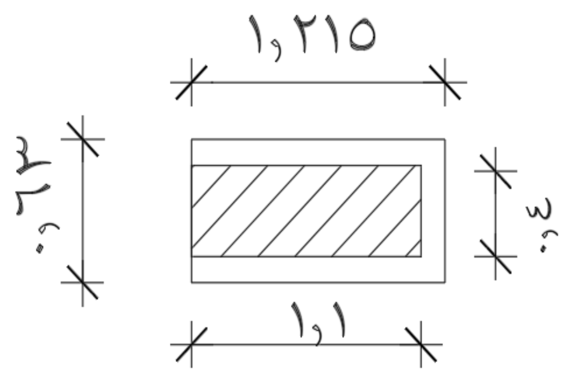
- Slab Thickness = 25 cm
- Own weight = 0.25* 2.5 = 0.62 t/m^2.
- Covering = 200 kg/m^2 = 0.2 t/m^2.
- Live load = 300 kg/m^2 = 0.30 t/m^2
- Wall load = 390 kg/m^2 = 0.39 t/m^2.

- D.L = O.W +W_{wall} + Covering material
- = 0.62 + 0.39+ 0.2 = 1.215 t/m^2
- L.L = 300 kg/cm^2 = 0.30 t/m^2

- W_u = 1.4 $D.L$ + 1.6 $L.L$ = 2.2 $t/m2$ = 22 $Kn/m2$
- d = t_s – 20 mm = 250 – 20 = 230 mm = 0.23
- b_o =2*(230+1800)+(230+600))=4860mm
- Q_{up}=Wu (L1 * L2 – Ap)=22 *(6.5*6.5 – ((0.23+0.6)*(0.23+1.80))=892.5Kn/m^2
- $q_{up} = \frac{Q_{up}}{bo*d} * \beta = \frac{892.5*1000}{4860*230} * 1.15 = 0.65 \text{ N}/mm^2$
- q_{cup} =*the least of*:-
  - 1.70 N/mm^2
  - $0.316\sqrt{\frac{f_{cu}}{\gamma_c}} == 0.316\sqrt{\frac{30}{1.5}} = 1.41\text{N}/mm^2$
  - $0.316(\frac{a}{b}+0.5)\sqrt{\frac{f_{cu}}{\gamma_c}} = 0.316(\frac{600}{1800}+0.5)\sqrt{\frac{30}{1.5}} = 1.17 \text{ N}/mm^2$
  - $0.8(\frac{\alpha*d}{b0}+0.2)\sqrt{\frac{f_{cu}}{\gamma_c}} = 0.8(\frac{4*230}{4860}+0.2)\sqrt{\frac{30}{1.5}} = 1.05 \text{ N}/mm^2$
- $q_{up} = 0.65\frac{\text{N}}{mm^2} \leq q_{cup} = 1.05 \text{ N}/mm^2$

OK safe punching

1.2.6.1 Edge Column (C_5=40*110) on (7-د) Axis:



- Slab Thickness = 25 cm
- Own weight = 0.25* 2.5 = 0.62 t/m^2.
- Covering =200 kg/m^2 = 0.2 t/m^2.
- Live load = 300 kg/m^2 = 0.3 t/m^2
- Wall load = 390 kg/m^2 = 0.39 t/m^2.

- D.L = O.W +W_{wall} + Covering material
- = .62 + 0.39 + 0.2 =1.215 t/m^2
- L.L = 300 kg/cm^2 = 0.3 t/m^2

- $W_u = 1.4\,D.L \;+\; 1.6\,L.L \;=1.4 * 1.215 + 1.6 * .3 = 2.2\; t/m2 \;= 22\; Kn/m2$
- $d \;=\; t_s - 20$ mm = 250 – 20 = 230 mm = 0.23m
- b_o =(400+230) + 2*(1100+$\frac{230}{2}$)=3060 mm
- Q_{up}=Wu (L1 * L2 – Ap)=22*(5.45*$\frac{4}{2}$– 0.63*1.215)=222.96 Kn/m^2
- $q_{up} \;= \frac{Q_{up}}{bo*d} * \beta = \frac{222.96*1000}{3060*230} * 1.3 = 0.41\ \text{N}/mm^2$
- q_{cup} =*the least of*:-
  - 1.7 N/mm^2
  - $0.316\sqrt{\frac{f_{cu}}{\gamma_c}} == 0.316\sqrt{\frac{30}{1.5}} = 1.41\ \text{N}/mm^2$
  - $0.316(\frac{a}{b}+0.5)\sqrt{\frac{f_{cu}}{\gamma_c}} = 0.316\,(\frac{400}{1100}+0.5)\sqrt{\frac{30}{1.5}} = 1.22\ \text{N}/mm^2$
  - $0.8(\frac{\alpha*d}{b0}+0.2)\sqrt{\frac{f_{cu}}{\gamma_c}} = 0.8(\frac{2*230}{3060}+0.2)\sqrt{\frac{30}{1.5}} = 1.25\ \text{N}/mm^2$
- $q_{up} = 0.41\frac{\text{N}}{mm^2} \le q_{cup} = 1.22\ \text{N}/mm^2$

OK safe punching

2.3 Design of Stairs (Three Flight Stair Axis \ج – \ه)

2.3.1 Manual solution

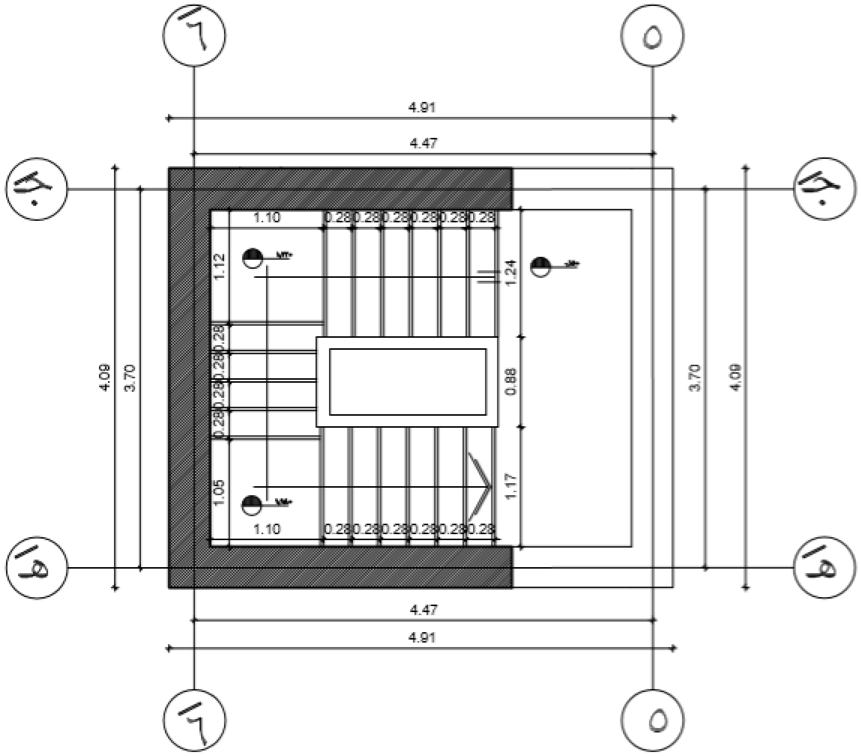


Figure 2.22 Stair Cross Section

1) Dimensions:

- $ts = \frac{Span}{24:30} = \frac{4.2m}{24:30} = 0.14\ m$
- $H_{Story} = 2.90m$
- $Rise = 0.15\ m$
- $Going = 0.30\ m$
- $\theta = \tan^{-1}(\frac{0.15}{0.30}) = 26.56\ °$
- $t^* = \frac{ts}{Cos\theta} = \frac{14}{Cos(26.56)} = 15.65\ cm$
- $t_{av} = t^* + \frac{Rise}{2} = 23.15$ cm

2) Loads:

- W_{su} = 1.4 D.L + 1.6 L.L

  = 1.4 (25*0.2315 + 1.0) + 1.6(3)

  = 14.30 KN/m^2

- $Wu_{\,landing}$ = 1.4 D.L + 1.6 L.L

  = 1.4 (25*0.22 + 1.5) + 1.6(3)

  = 14.60 KN/m^2

3) For Strips

Shown In The figure

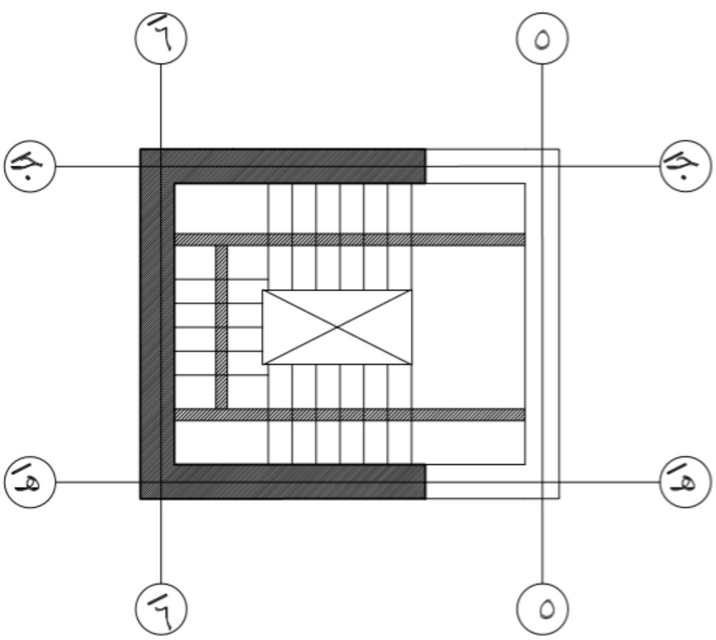


Figure 2.23 Strips Of Stair

For Strip (F1):

- R_1 = 4 KN
- M_{u1} = 4*0.84 -14.30*0.28^2/2= 2.80 KN.m

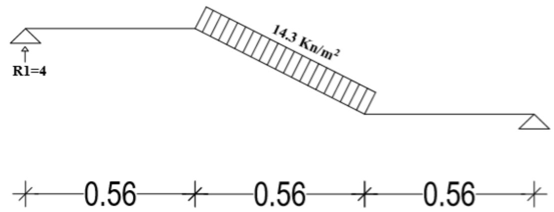


Figure 2.24 Strip (F1) of Stair

For Strip (F2):

- R_3 *2.84 = 23.73* 1.1*2.29 + (14.3 * $\frac{1.74^2}{2}$)

  R_3 = 28.67 Kn

- R_2=(23.73*1.1) + (14.3*1.74) – 28.67

  R_3 = 22.30 Kn

- M_{u2} =(22.3 * $\frac{1.74}{2.00}$) – ($\frac{14.3*0.87^2}{2}$) = 13.99 Kn.m
- M_{u3} =(28.67 *1.1) – ($\frac{23.73*1.1^2}{2}$) = 17.18 Kn.m

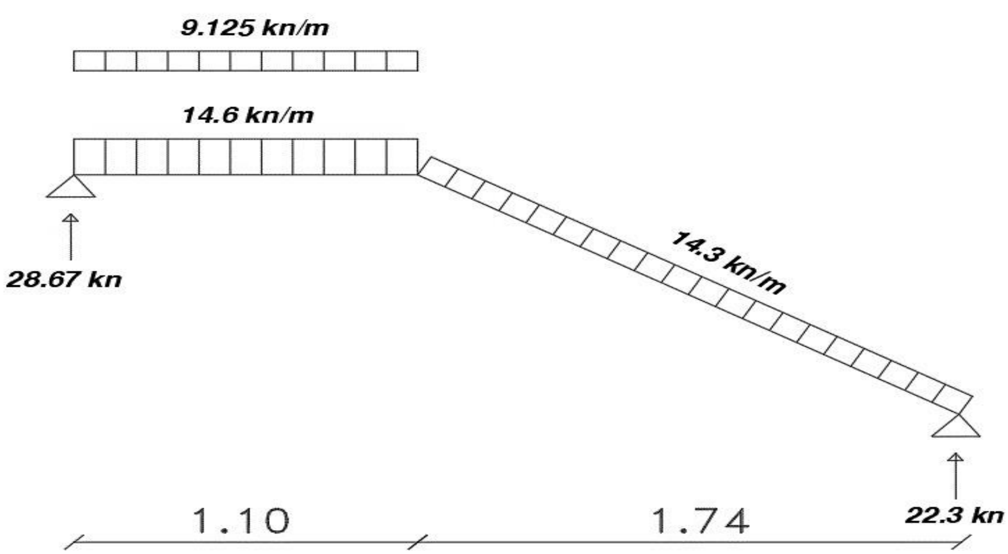


Figure 2.25 Strip (F2) of Stair

4) Design of Section of Flight :p

- M_{u2} = 13.99 KN.m
- $R_1 = \frac{M_u}{F_{cu}\, b\, d^2} = \frac{13.99*10^6}{30*1000*210^2} = 0.011 < R_{x\,max}$=0.129

  ɯ = 0.02

  A_s = $ɯ \frac{Fcu}{Fy} bd = 0.02 * \frac{30}{350} * 100 * 21 = 3.6$ cm^2
- Use A_S ⇛ 5 ∯ 12 / m ⇛ A_{sact} =5.65 cm^2

5) Design of Section of Landing :

- M_{u3} = 17.18 KN.m
- $R_1 = \frac{M_u}{F_{cu}\, b\, d^2} = \frac{17.18*10^6}{30*1000*210^2} = 0.013 < R_{x\,max}$=0.129

  ɯ = 0.02

  A_s = $ɯ \frac{Fcu}{Fy} bd = 0.02 * \frac{30}{350} * 100 * 21 = 3.6$ cm^2
- Use A_S ⇛ 5 ∯ 12 / m ⇛ A_{sact} =5.65 cm^2

2.3.2 Using Sap Program

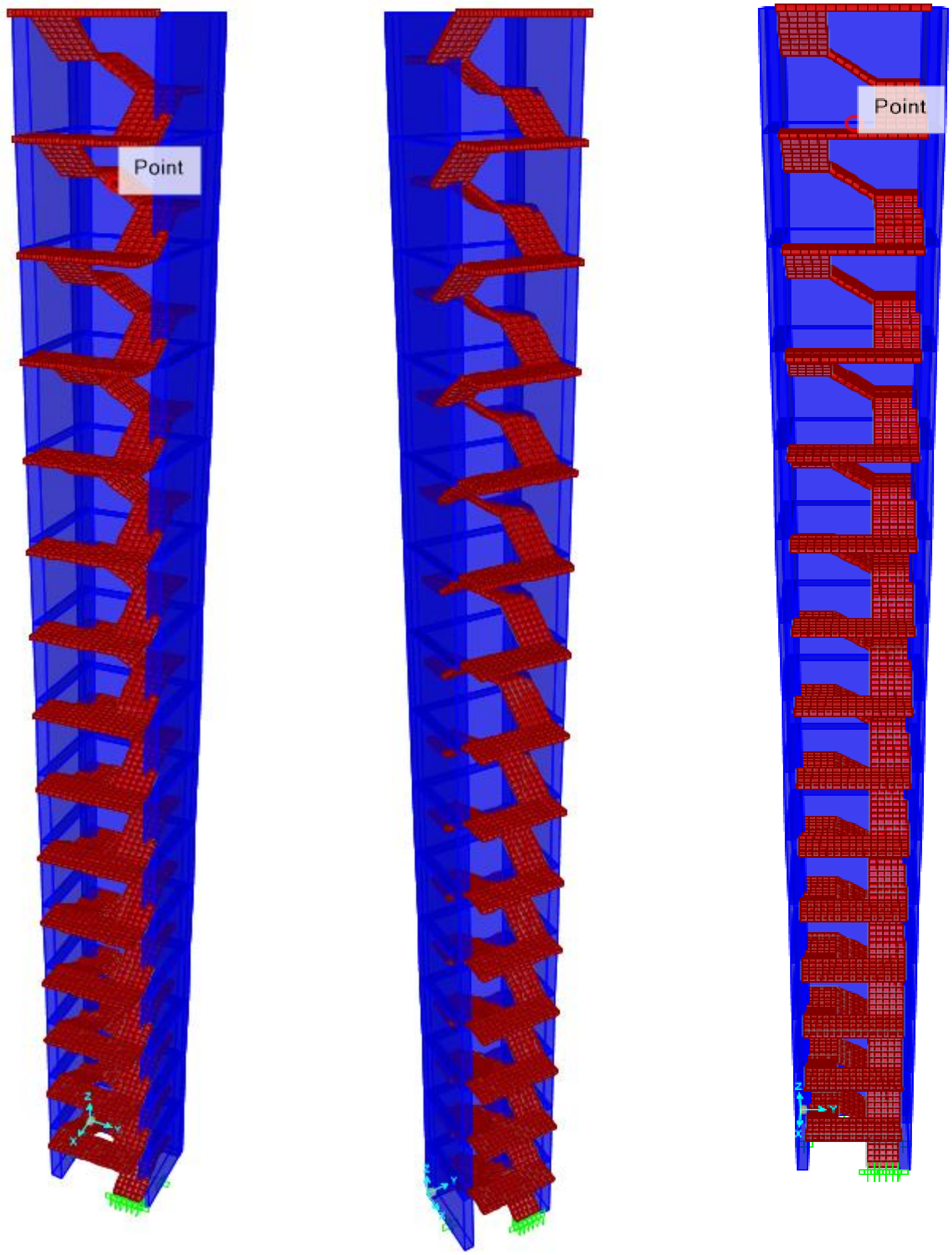


Figure 2.26 Stair 3D

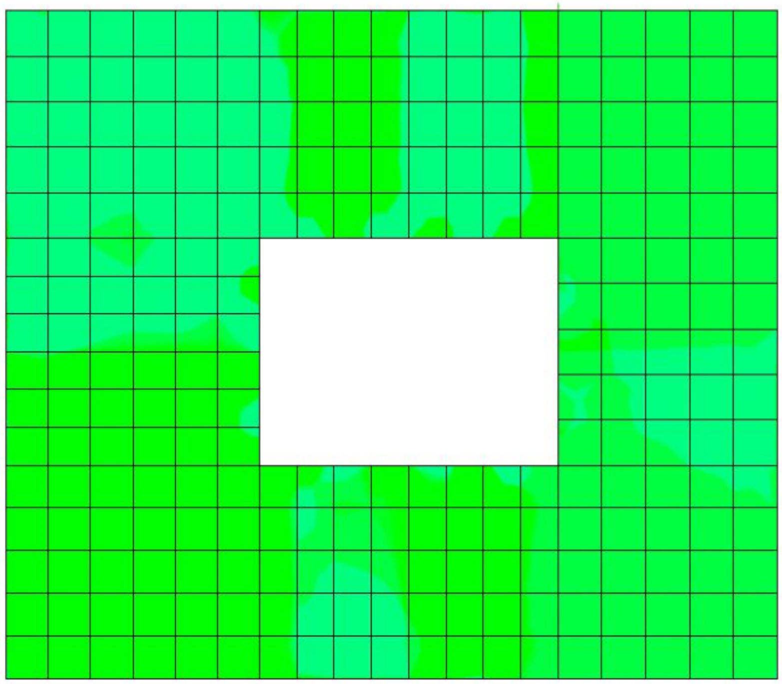


Figure 2.27 Bending Moment In X-Direction

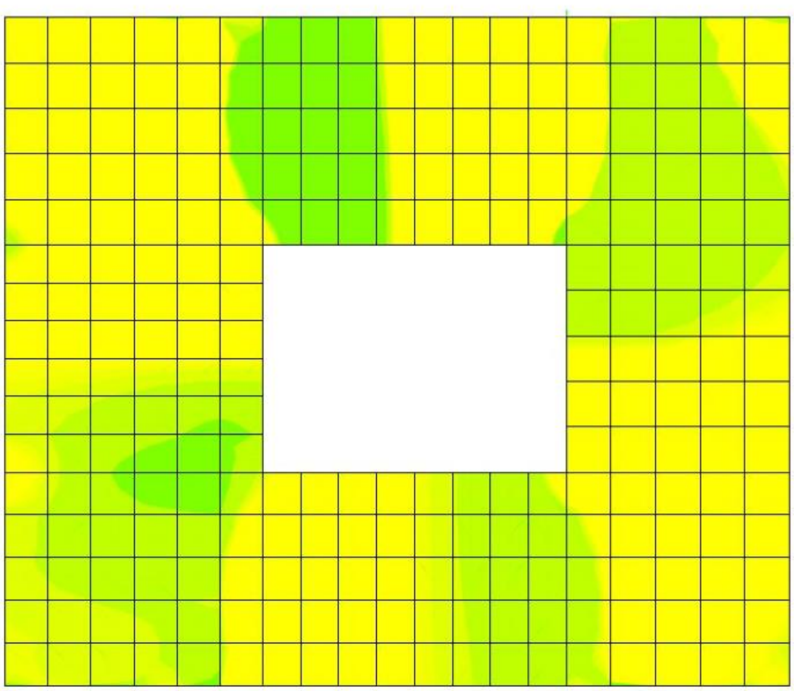


Figure 2.28 Bending Moment In Y-Direction

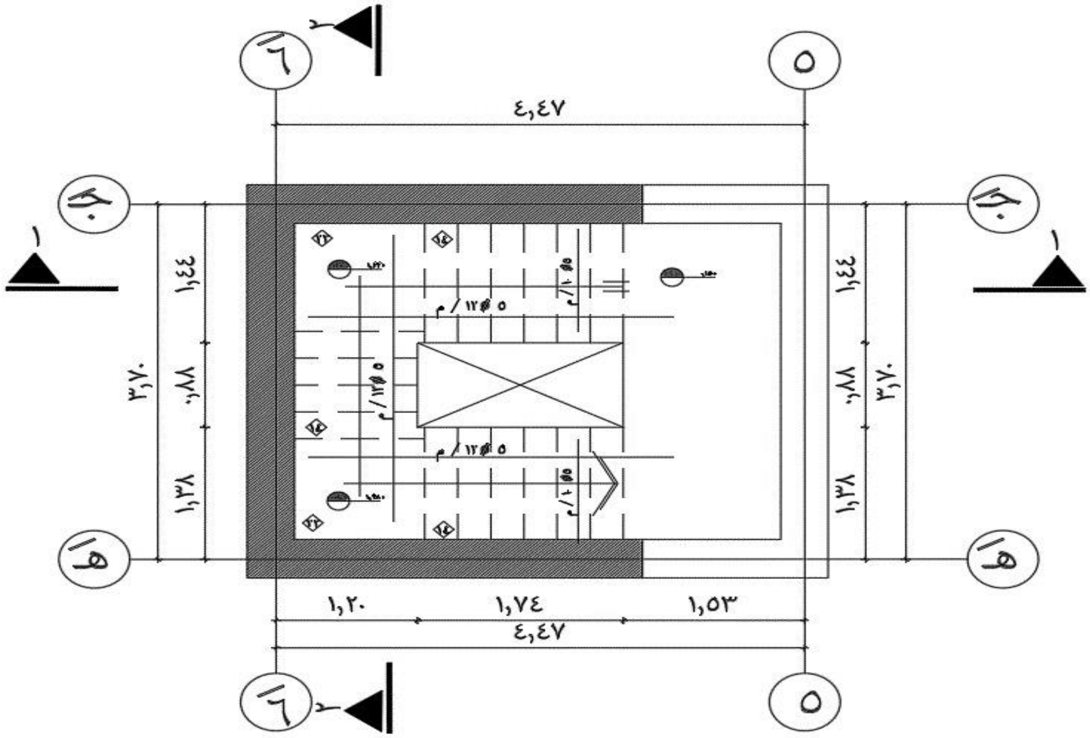


Figure 2.29 Stair Reinforcement in plan

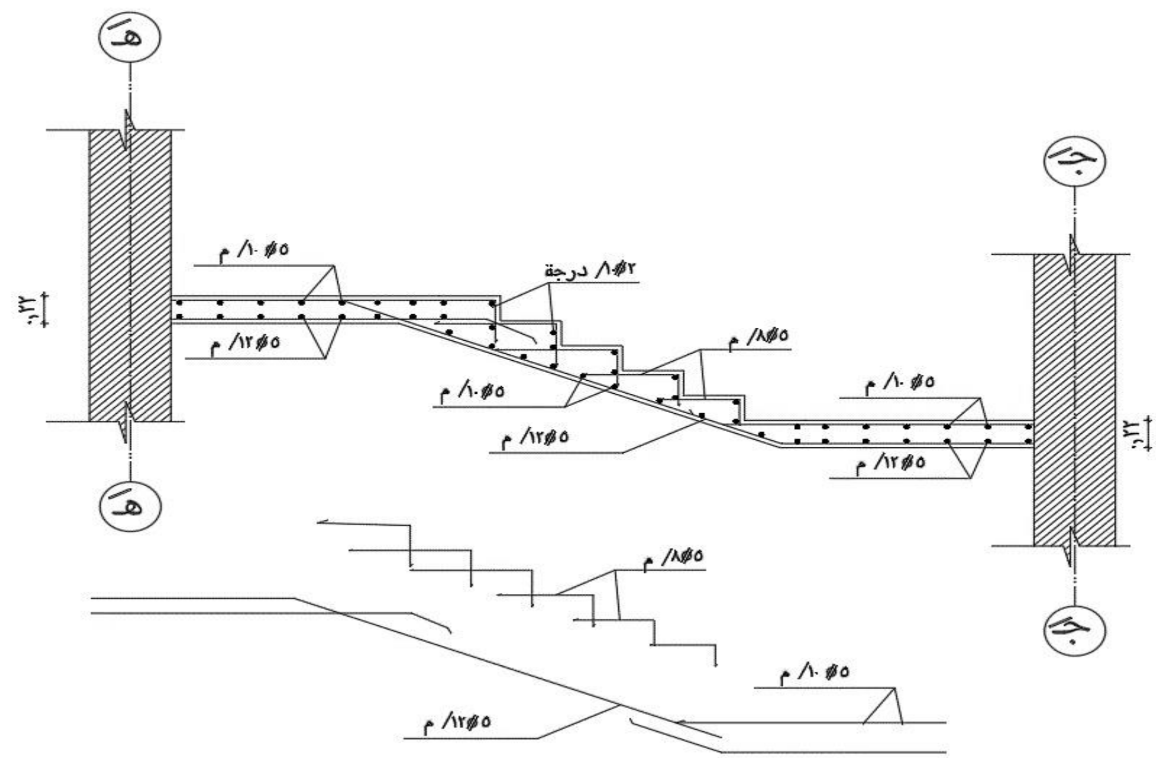


Figure 2.30 Reinforcement in sec 2-2

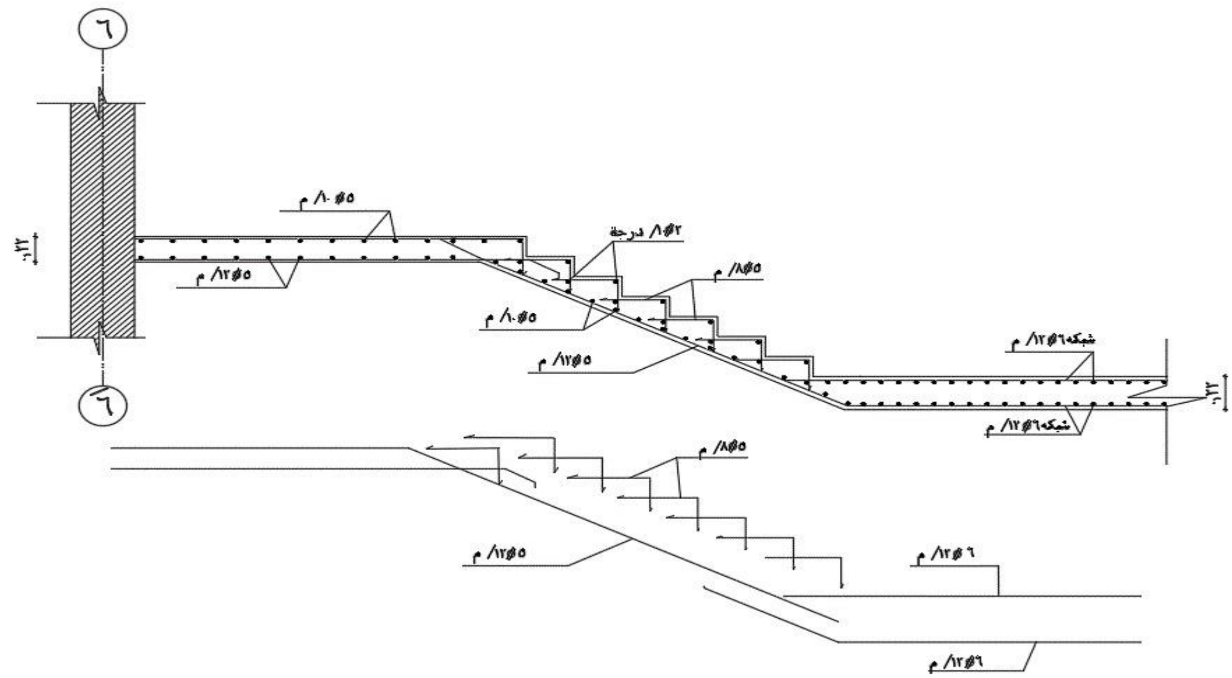


Figure 2.31 Reinforcement in sec 1-1

2.4 Design of Stairs (Two Flight Stair Axis ج – د)

2.4.1 Manual solution

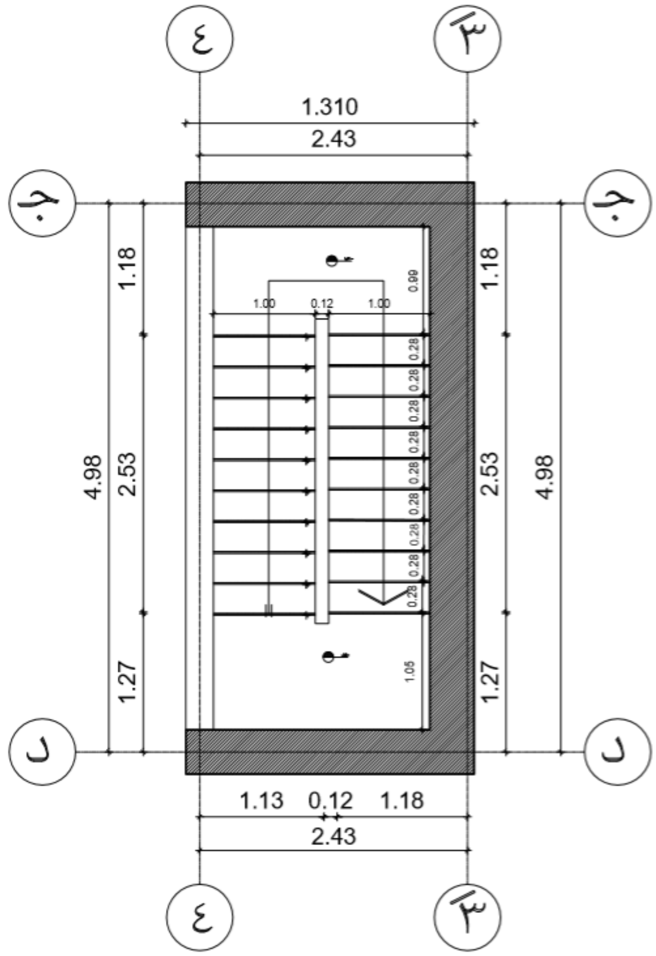


Figure 2.32 Stair Cross Section

1) Dimensions:

- $ts = \frac{Span}{24:30} = \frac{4.2m}{24:30} = 0.14\ m$
- $H_{Story=}\ 2.90m$
- $Rise = 0.15\ m$
- $Going = 0.30\ m$
- $\theta = \tan^{-1}(\frac{0.15}{0.30}) = 26.56\ °$
- $t^{*} = \frac{ts}{Cos\theta} = \frac{14}{Cos(26.56)} = 15.65\ cm$
- $t_{av} = t^{*} + \frac{Rise}{2} = 23.15$ cm

2) Loads:

- Wsu = 1.4 D.L + 1.6 L.L

  = 1.4 (25*0.2315 + 1.0) + 1.6(3)

  = 14.30 KN/m^2

3) For Strips

Shown In The figure

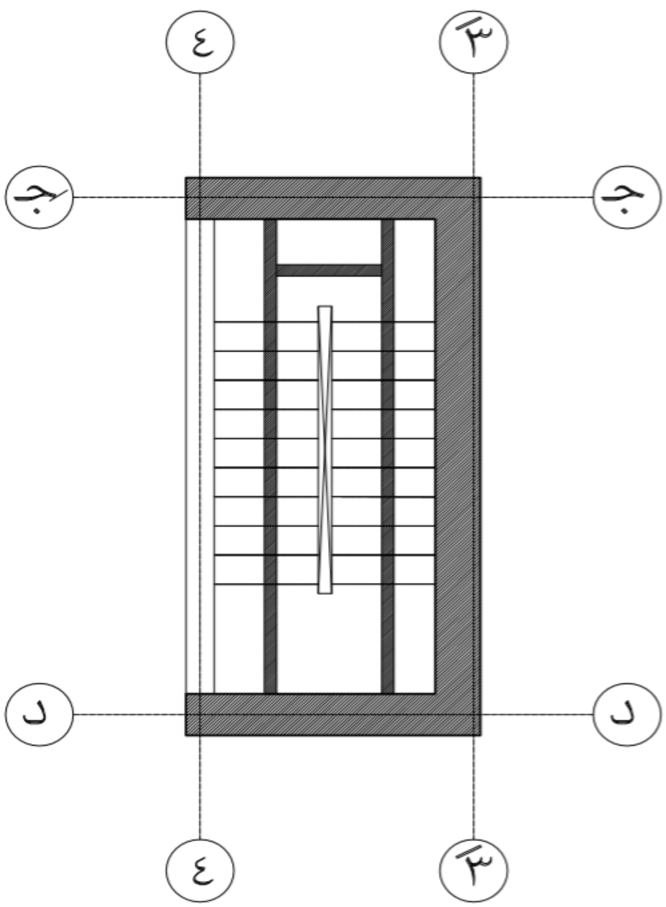


Figure 2.33 Strips Of Stair

For Strip (F1):

- R_1 = 18.1 KN
- M_{u1} = 18.1*1.76 -14.30*(1.76)^2/2= 9.71 KN.m

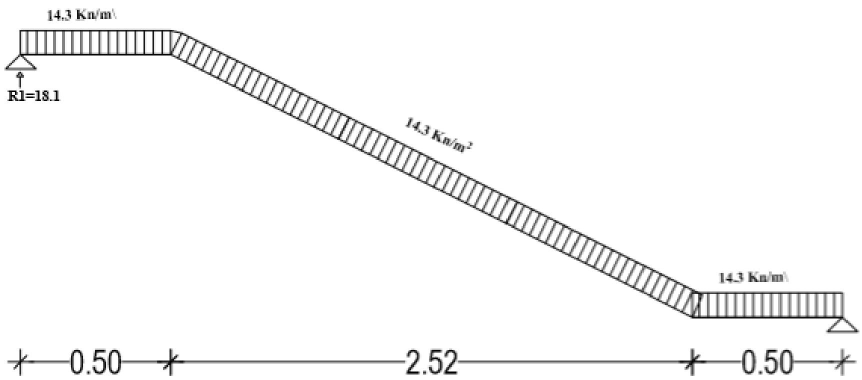


Figure 2.34 Strip (F1) of Stair

4) Design of Section of Flight :

- M_{u2} = 9.71 KN.m
- $R_1 = \frac{M_u}{F_{cu}\, b\, d^2} = \frac{9.71*10^6}{30*1000*210^2} = 0.02 < R_{x\ max}$=0.129

  ɯ = 0.02

  $A_s = ɯ \frac{Fcu}{Fy} bd = 0.02 * \frac{30}{350} * 100 * 21 = 3.6\ cm^2$
- Use A_S ⇛ 5 ∯ 12 / m ⇛ A_{sact} =5.65 cm^2

2.4.2 Using Sap Program

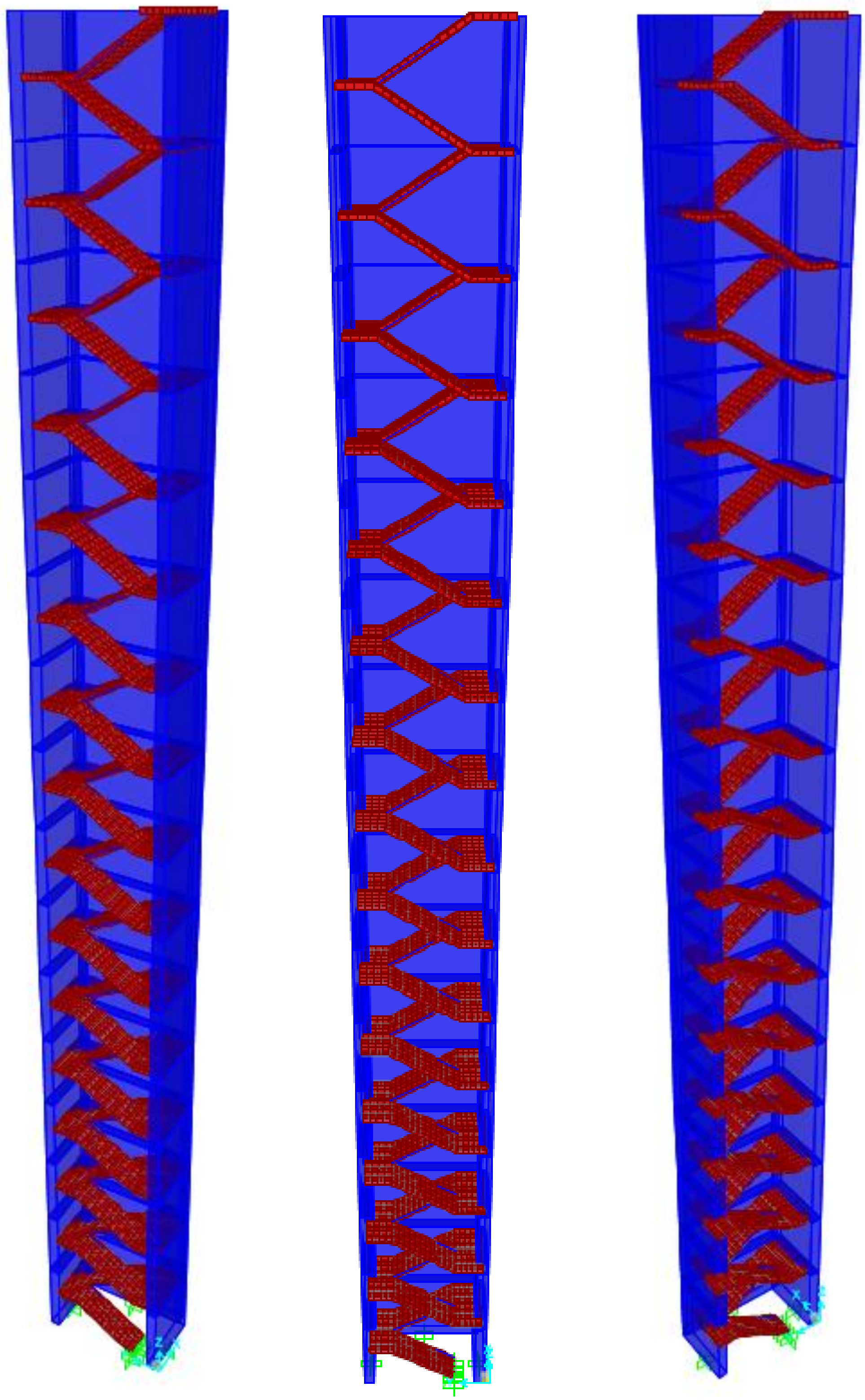


Figure 2.35 Stair 3D

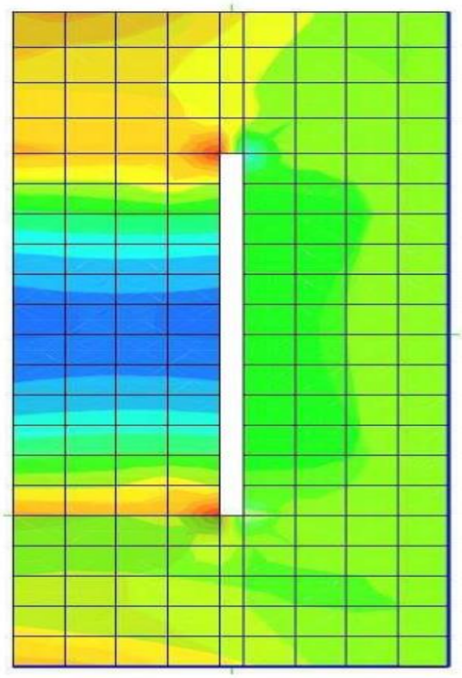


Figure 2.36 Bending Moment In X-Direction

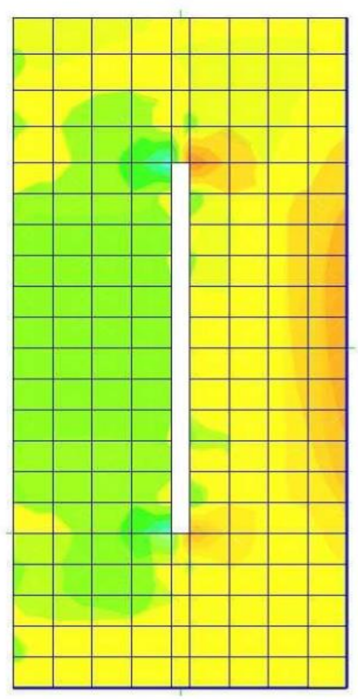


Figure 2.37 Bending Moment In Y-Direction

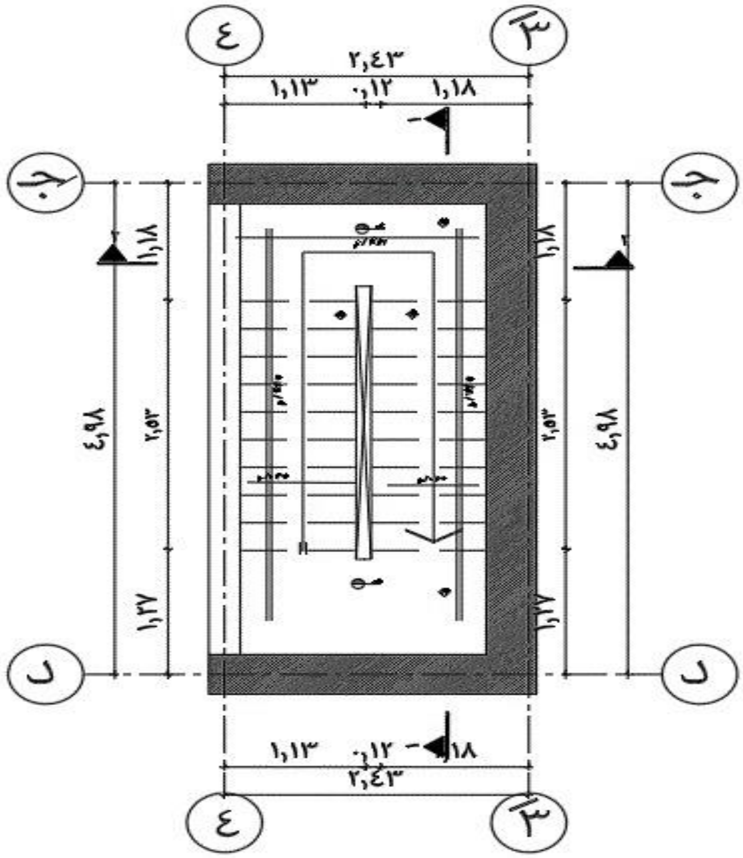


Figure 2.38 Stair Reinforcement in plan

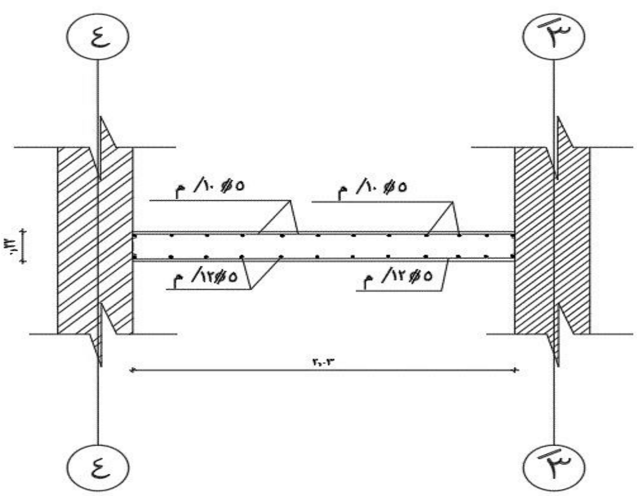


Figure 2.39 Reinforcement in sec 2-2

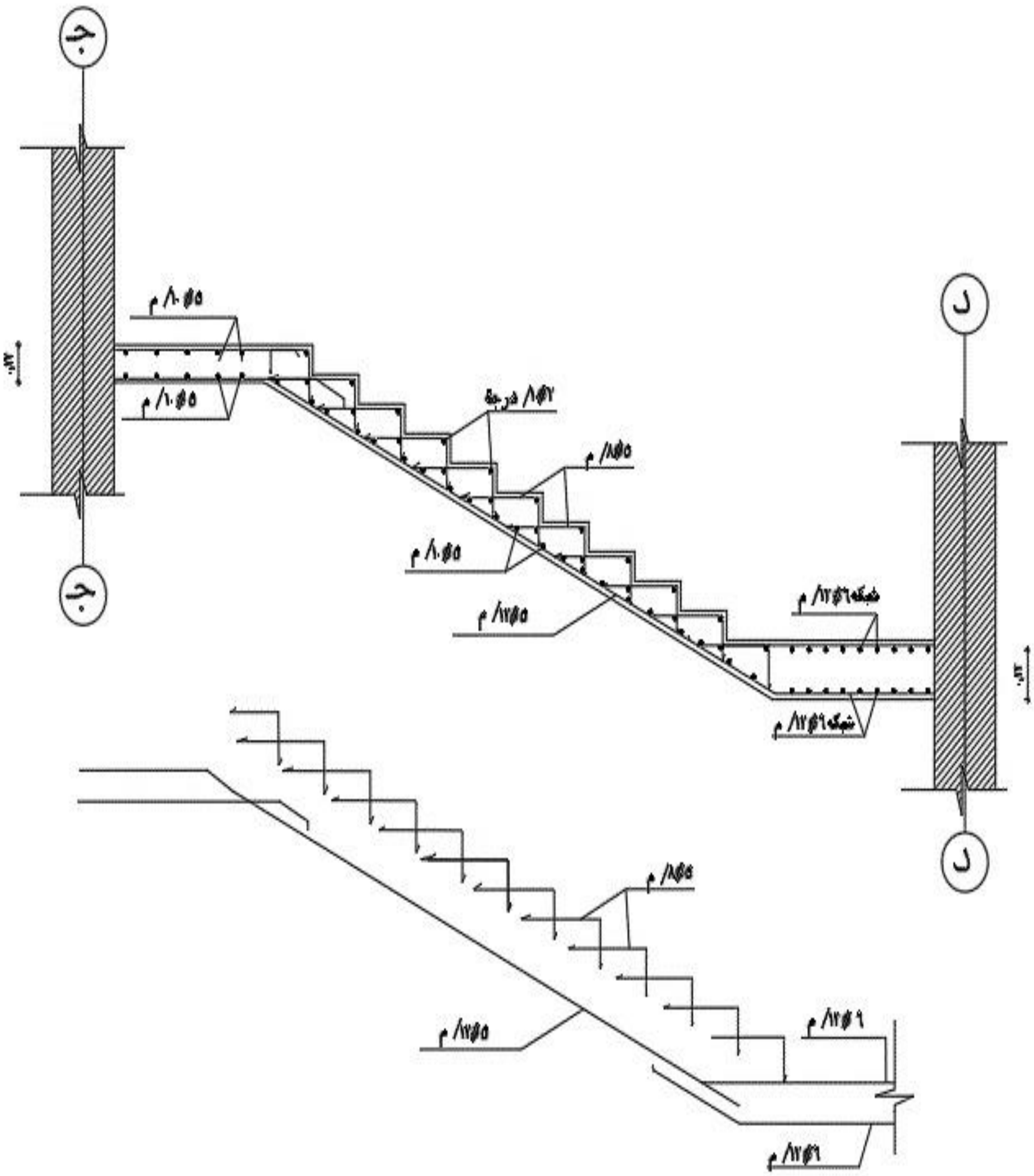


Figure 2.40 Reinforcement in sec 1-1

2.6 Design of Columns

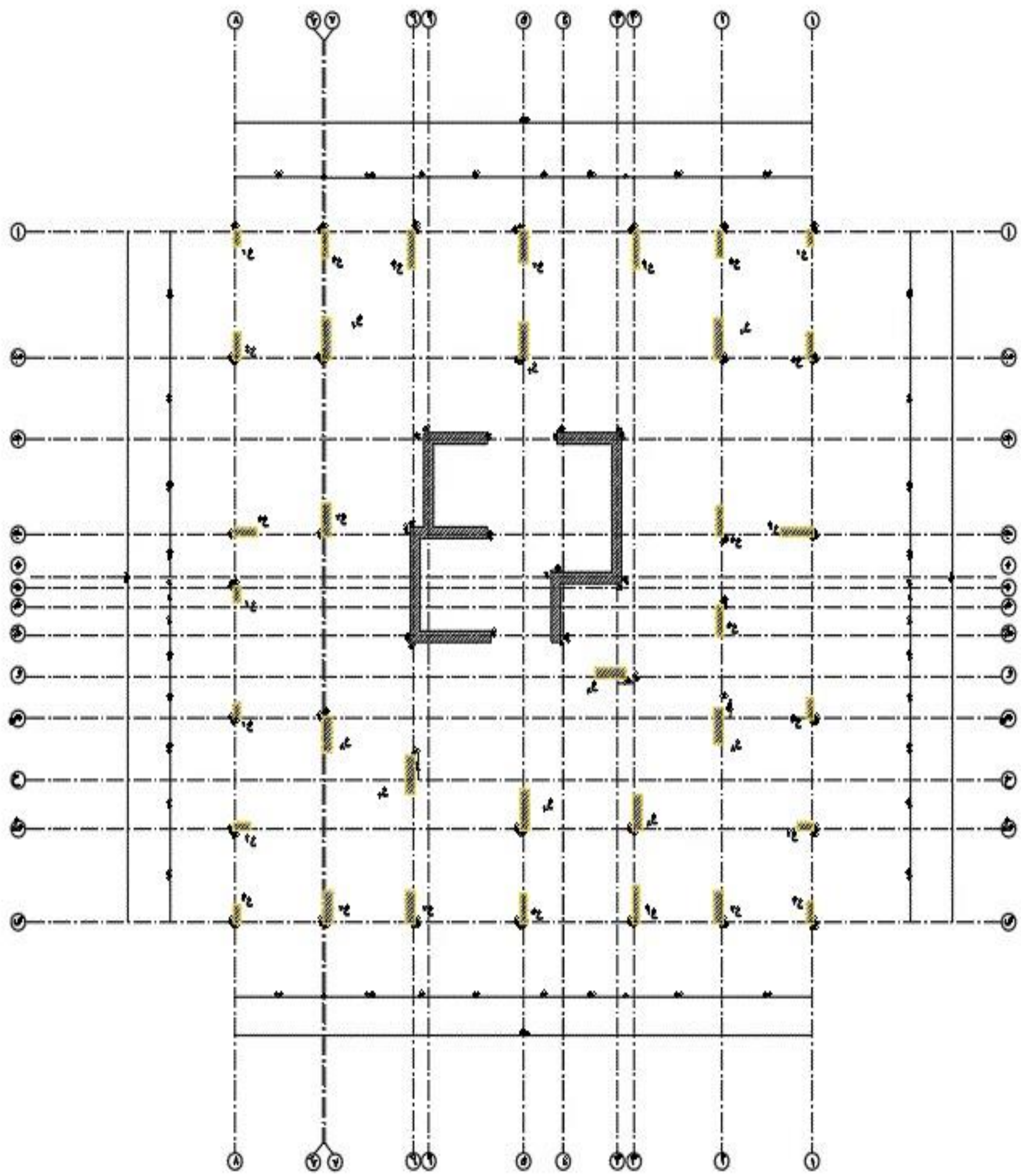


Figure 2.41 Columns Axis

2.6.1 Design of Column Section (subjected to axial compression force)

- **For ع1 on axis (2 - 2) (ب - ب)**
  - **Concrete Dimension**
  - P_U = 690.535 ton (From Etabs Program)
  - $P_{u\ actual}$ = 690.535 *1.1 =759.589 ton
  - $P_{u\ actual} = 0.35(A_c - A_s)F_{cu} + 0.67\ A_s\ F_y \rightarrow$ assume $A_s = 0.01\ A_c$
  - $759.589 * 10^3 = 0.35*(0.99A_c)*300 + 0.67*3500*0.01A_c$
  - $A_c = 5913.5\ cm^2$
  - Assume b=40 cm
  - $t = \frac{Ac}{b}$ = 150 cm
  - A_{cact}=40*150 =6000 cm^2
- Check of Buckling (braced column)
  - in short direction
  - k = 0.85 (1-2)

$\lambda = \frac{k*Ho}{b} = \frac{0.85*2.70}{0.40} = 5.74 < 15 \Rightarrow$ Short Column

  - in Long direction
  - k = 0.85 (1-2)

$\lambda = \frac{k*Ho}{t} = \frac{0.85*2.70}{1.50} = 1.53 < 15 \Rightarrow$ Short Column

Neglected buckling in short and long direction

  - A_s = 0.01*40*150 = 60 cm^2

Use 30 ∅ 16

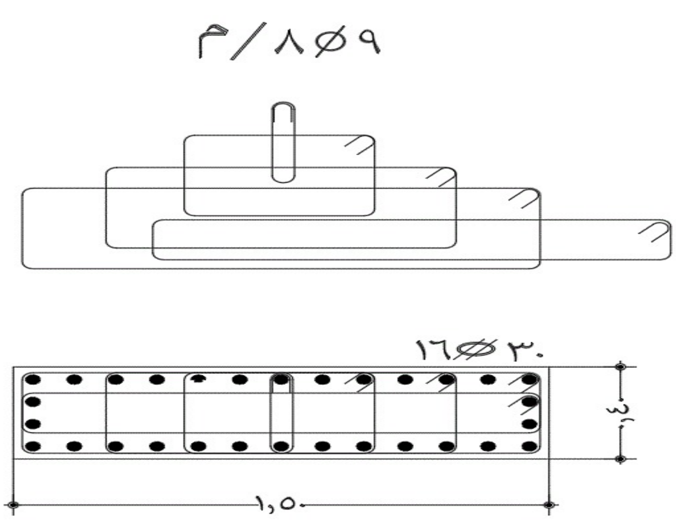


Figure 2.42 Column Cross Section

2.6.2 Table of Columns

								No . of Bars			
Joint	P_u etabs (T)	$P_{u\ act}$ (T)	$\mu = A_s/A_c$	$A_{c\ act}$ (cm^2)	b (cm)	t (cm)	t_{act} (cm)	A_s (cm^2)	Ø	16	Sample
1	473.6665	522	1.00%	4743	60	90	90	54	28		C_4
2	551.8681	608	1.00%	5524	60	100	100	60	30		C2
3	871.01	959	1.00%	8713	60	180	180	108	54		C5
4	553.9098	610	1.00%	5542	60	100	100	60	30		C2
5	473.6777	522	1.00%	4743	60	90	90	54	14		C4
6	196.7841	217	1.00%	1972	40	80	80	32	16		C_1
7	272.8153	301	1.00%	2735	40	80	80	32	16		
8	362.4781	399	1.00%	3625	40	120	120	48	24		C_3
9	390.2502	430	1.00%	3907	40	120	120	48	24		
10	353.3961	389	1.00%	3534	40	120	120	48	24		
11	403.153	444	1.00%	4034	40	120	120	48	24		
12	353.5324	389	1.00%	3534	40	110	110	44	22		C7
13	299.0935	330	1.00%	2998	40	90	90	36	18		
14	162.5646	179	1.00%	1627	30	70	70	21	12		C6
15	486.8731	536	1.00%	4870	60	90	90	54	28		C4
16	549.2442	605	1.00%	5497	60	100	100	60	30		C2
17	699.6955	770	1.00%	6996	50	160	160	80	40		C8
18	547.4504	603	1.00%	5479	60	100	100	60	30		C2
19	487.014	536	1.00%	4870	60	90	90	54	28		C4
20	162.7105	179	1.00%	1627	30	70	70	21	12		C6
21	299.2239	330	1.00%	2998	40	90	90	36	18		
22	353.382	389	1.00%	3534	40	110	110	44	22		C7
23	404.8916	446	1.00%	4052	40	120	120	48	24		C_3
24	354.0691	390	1.00%	3544	40	120	120	48	24		
25	390.6017	430	1.00%	3907	40	120	120	48	24		
26	362.5723	399	1.00%	3625	40	120	120	33	18		
27	274.0616	302	1.00%	2744	40	80	80	21	12		C_1
28	197.1372	217	1.00%	1972	40	80	80	24	12		
29	979.1146	1078	1.00%	9794	60	180	180	48	24		C_5
30	946.8372	1042	1.00%	9467	60	180	180	48	24		
31	984.0634	1083	1.00%	9839	60	180	180	48	24		
32	965.2035	1062	1.00%	9648	60	180	180	48	24		
33	974.6014	1073	1.00%	9748	60	180	180	48	24		
34	959.6157	1056	1.00%	9594	60	180	180	48	24		

Table 2.2 Columns Load And Section (Ultimate)

Table 2.3 Columns Section

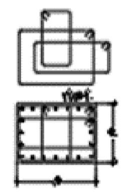
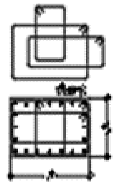
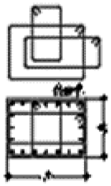
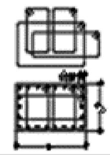
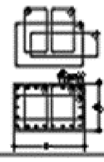
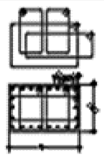
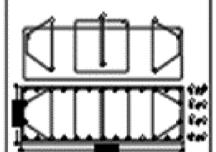
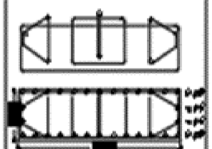
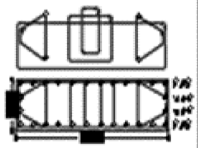
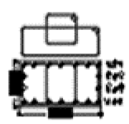
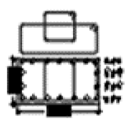
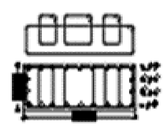
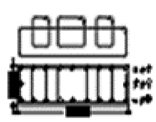
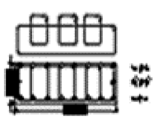
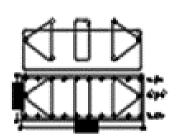
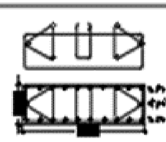
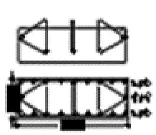
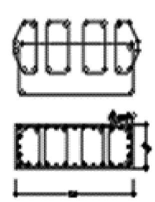
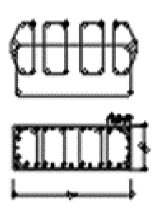
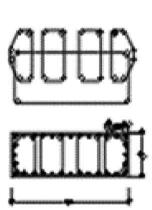
النموذج	القطاع (بدروم + ارضي + اول + ثاني) الابعاد	تسليح	تفاصيل	القطاع (ثالث + رابع + خامس + سادس) الابعاد	تسليح	تفاصيل	القطاع (سابع + ثامن + تاسع + عاشر) الابعاد	تسليح	تفاصيل	كانات عادية	تكثيف
ع١	٩٠×٦٠	١٦ Ø ٢٠		٩٠×٦٠	١٦ Ø ٢٠		٩٠×٦٠	١٦ Ø ٢٠		[illegible]	[illegible]
ع٢	١٠٠×٦٠	١٦ Ø ٢٢		١٠٠×٦٠	١٦ Ø ٢٢		١٠٠×٦٠	١٦ Ø ٢٢		[illegible]	[illegible]
ع٣	١٢٠×٥٠	١٦ Ø ٢٠		[illegible]	[illegible]		١٥٠×٥٥	٢٠ Ø ٢٤ ١٨ Ø ٤٠		[illegible]	[illegible]
ع	٨٠×٤٠	[illegible]		٨٠×٤٠	١٨ Ø ١٦ ١٦ Ø ٤٠		٨٠×٤٠	١٨ Ø ١٢ ١٦ Ø ٤٠		[illegible]	[illegible]
ع٥	١١٠×٤٠	١٨ Ø ١٨ ١٦ Ø ٤٠		١١٠×٢٥	١٨ Ø ١٨ ١٦ Ø ٢٠		١٠٠×٢٥	١٨ Ø ١٢ ١٦ Ø ٢٠		[illegible]	[illegible]
ع٦	١٢٠×٤٠	١٨ Ø ٢٠ ١٦ Ø ٢٠		١٢٠×٢٥	١٨ Ø ٢٠ ١٦ Ø ٢٠		١١٠×٢٥	١٨ Ø ١٨ ١٦ Ø ٢٠		[illegible]	[illegible]
ع٧	١٨٠×٦٠	١٦ Ø ٢٢		١٨٠×٦٠	١٦ Ø ٢٢		١٨٠×٦٠	١٦ Ø ٢٢		[illegible]	[illegible]

Table 2.4 Columns Section

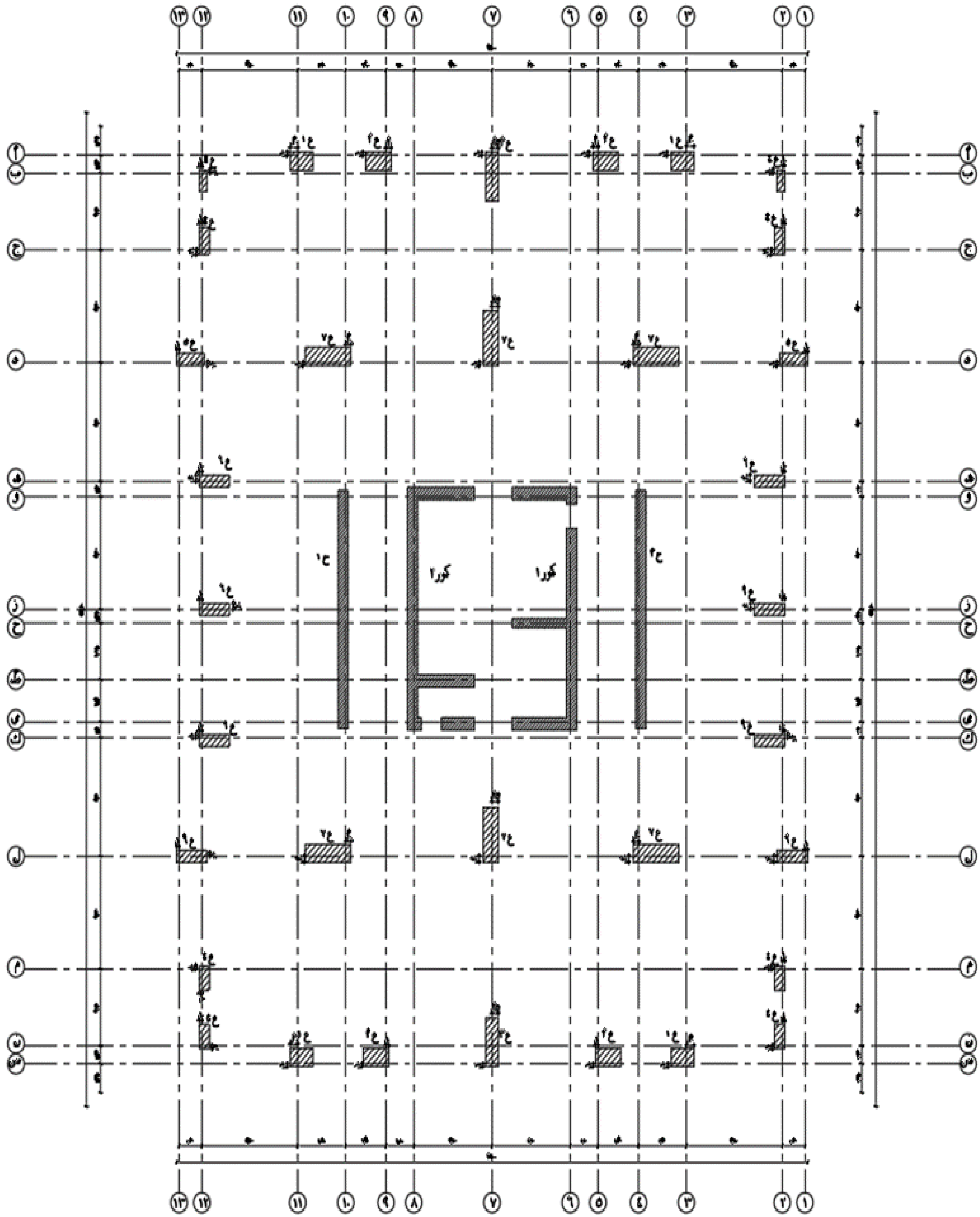


Figure 2.43 Column Reaction (Ultimate)

2.7 Design of ShearWall

(W1=0.4m*7.80m) on asix (7-7)

Case (1):

- Pu = 994.24 ton
- Mu = 1596 ton
- Qu = 17.052 ton

1. K = $\frac{\text{Pu}}{\text{Fcu}*\text{b}*\text{t}} = \frac{994.24*10^4}{30*400*7800}$= 0.1

2. K$\frac{e}{t}$ = $\frac{\text{Mu}}{\text{Fcu}*\text{b}*t^2} = \frac{1596*10^7}{30*400*7800^2}$ =0.02

3. $\rho = 1$

4. µ = ρ * $\text{Fcu} * 10^{-4}$ ==1*30*10^{-4}= 0.003

6. $A_{s\ total=} m * b * t = 3 * 10^{-3} * 400 * 7800$

$A_{s\ total=}$9360 mm^2

Minimum RFT As = 0.01bt = 0.01 *300 * 2500 =7500 mm2

Use As min (Use 46 ∯ 16)

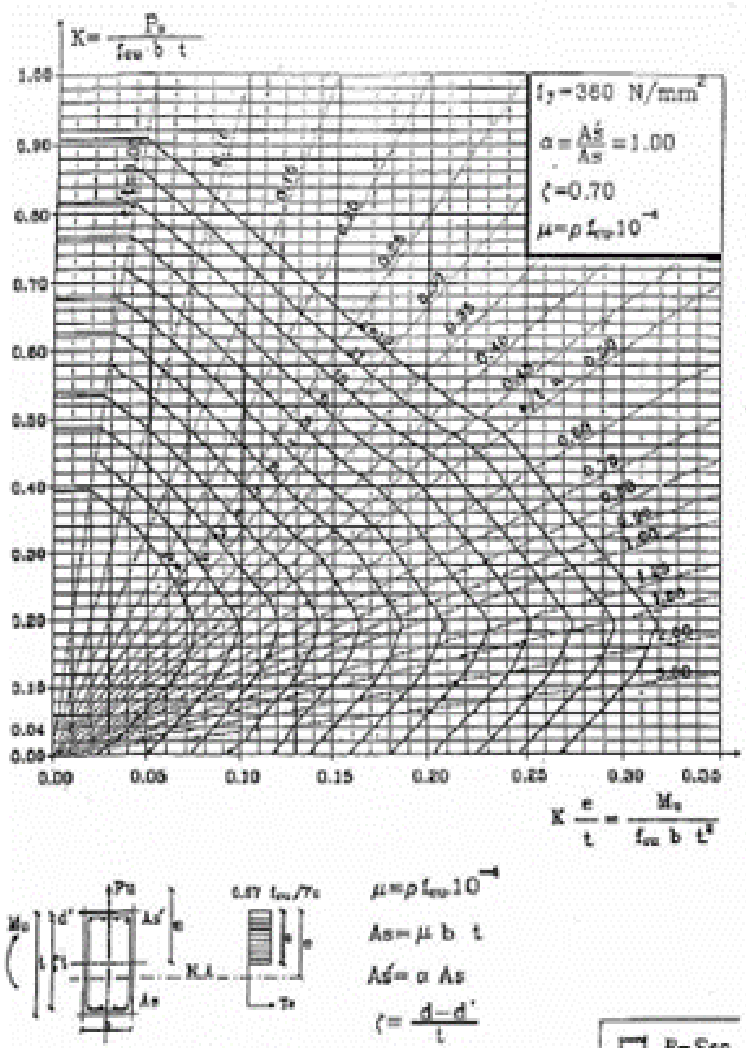


Figure 2.49 Interaction Diagram for Biaxial Loaded Wall

2.8 Design of Core 1 (using CSI Column program)

Case (1):

- P_u = 1363.61 t
- M_{ux} = 314.82t.m
- M_{uy} = 2651.69 t.m

Case (2):

- P_u = 729.18 t
- M_{ux} = 294.61 t.m
- M_{uy} = 1086.72 t.m

Case (3):

- P_u = 989.6 t
- M_{ux} = 118.7 t.m
- M_{uy} = 1584.37 t.m

Use As_{total} =184 Φ 12 + (112 Φ 16) add

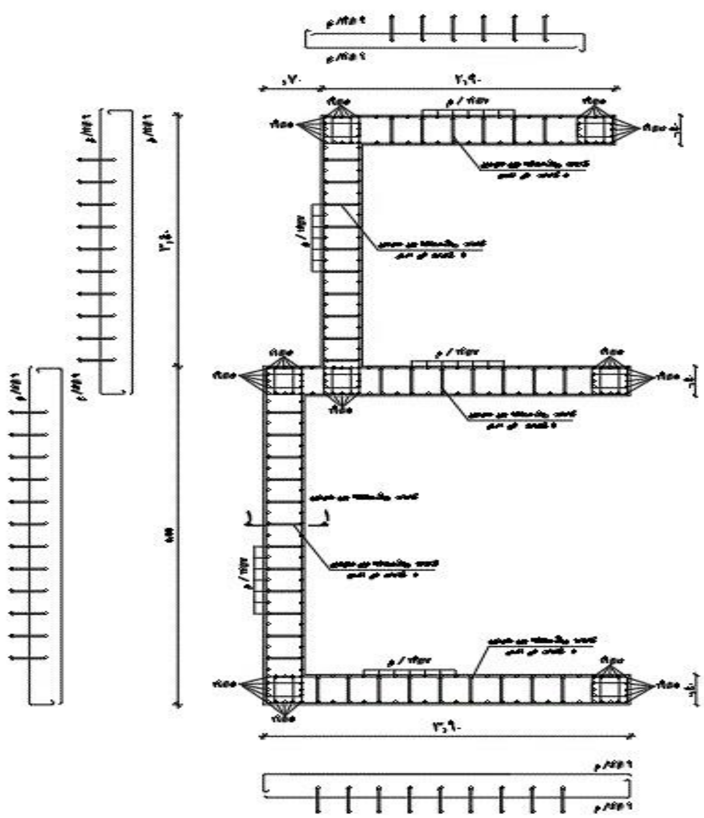


Figure 2.44 Core Cross Section

2.9 Design of Core 2 (using CSI Column program)

Case (1):

- P_u = 2020 t
- M_{ux} = 777.59 t.m
- M_{uy} = 1760.36 t.m

Case (2):

- P_u = 1089.9 t
- M_{ux} = 236.32t.m
- M_{uy} = 395.95 t.m

Case (3):

- P_u = 1073.41 t
- M_{ux} = 295.72 t.m
- M_{uy} = 265.2 t.m

Use As_{total} =214 Φ 12 + (60 Φ 18) add

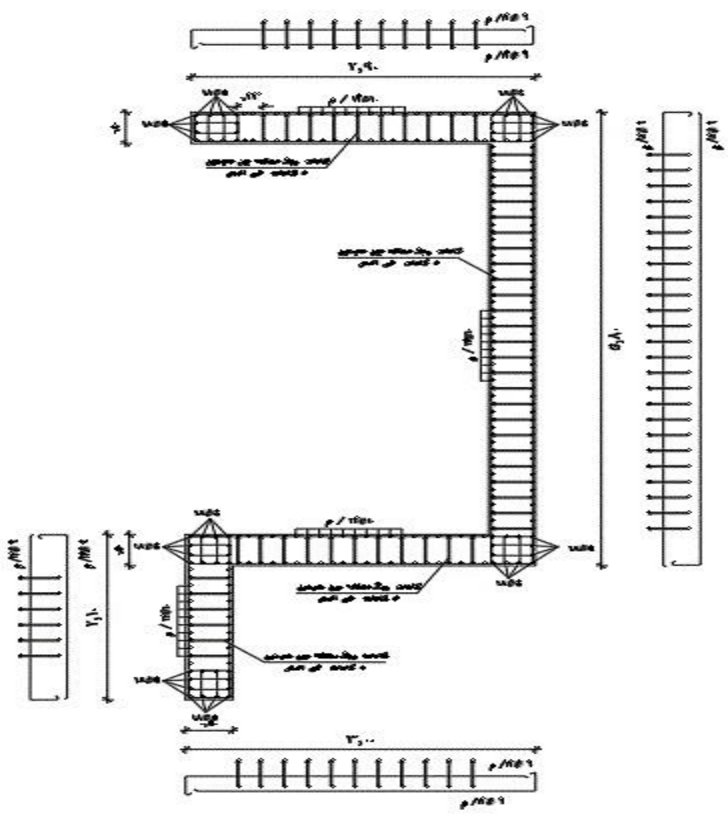


Figure 2.45 Core Cross Section

2.10 Effect of Earthquake on tower

2.10.1 Manual solution

- Zone (2) ⇛ $a_g = 0.125g = 1.25 m/sec^2$
- Type Soil (c) ⇛ S =1.5 , $T_B = 0.1$, $T_C = 0.25$, $T_D = 1.20$
- Importance Factor (Υ_I) = 1.0
- Damping Correction Factor (η) = 1.0
- Reduction facto r(R) = 6

$$F_b = Sd(T_1) * \lambda * \frac{w}{g}$$

1. $T_1 = C_t H^{3/4} = 0.05 * 43.50^{3/4} = 0.847$ sec
2. $T_C < T_I < T_D$ ⇛ $Sd(T_1) = ag * \Upsilon_I * S * \frac{2.5\,\eta}{R} * [\frac{TC}{T_1}] = 0.28 > 0.2\ a_g * \Upsilon_I = 0.24$
3. $\lambda = 1$ ⇛ as $T_I > 2\ T_C$
4. W_i = D.L +Ψ L.L
   a. D.L = O.w + cover + W_{wall} = 2.50 * 0.25 + 0.2 + 0.27= 1.095 t/m^2
   b. W = 1.095 + 0.25 * 0. 3= 1.17 t/m^2
5. W_{total} = W_1 5Rep.=(1.1*1.095* 682.7) x (1.17*751.7 *1.1*13) =12180.91 ton
6. $F_b = 0.28 * 1 * \frac{12180.91}{9.81}$ = **348.91 ton**

story	level	H	wight	wi * hi	fi	vi	mot	m torsional
14	41.55	3	879.48	36542.4	46.96	46.96	0	58.7
13	38.5	3	879.48	33903.95	43.57	90.53	140.88	113.16
12	35.55	3	879.48	31265.51	40.18	130.71	412.47	163.38
11	32.55	3	879.48	28627.1	36.78	167.49	804.6	209.36
10	29.55	3	879.48	25988.64	33.39	200.88	137.07	251.58
9	26.55	3	879.48	2350.19	30	230.88	1909.71	289.08
8	23.55	3	879.48	20711.75	26.62	257.5	2602.35	322.36
7	20.55	3	879.48	18073.31	23.22	280.72	3374.85	351.38
6	17.55	3	879.48	15434.87	19.83	300.55	4217.01	376.17
5	14.55	3	879.48	12796.43	16.44	316.99	5118.66	396.72
4	11.55	3	879.48	10157.99	13.1	330.09	6069.63	413.1
3	8.55	3	879.48	7519.55	9.66	339.75	7059.9	425.17
2	5.55	3	879.48	4881.11	6.27	346.02	8079.15	433.01
1	2.55	2.55	747.56	2242.67	2.88	348.9	9117.21	436.01
Sum				271495.47	∑fi= 348.91		10163.91	

Table 2.5 Over Turning Momnet And Torsional Moment

$M_{\text{overturning}}$ = 10163.91 t.m

$M_{\text{torsional}} = V_i * e_{min}$ = 348.91 *0.05 *25 =436.01 t.m

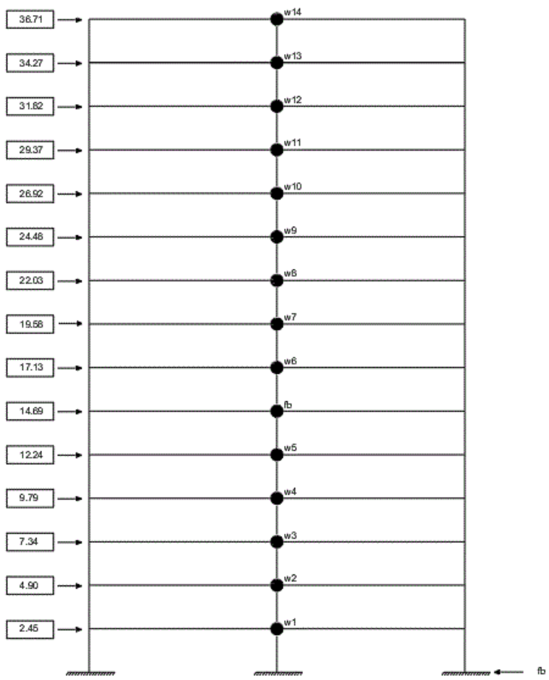


Figure 2.46 force at basement

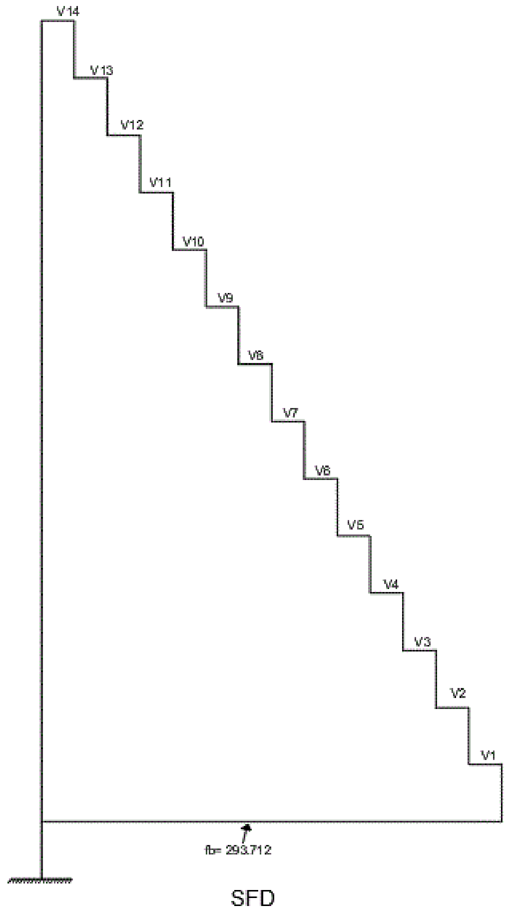


Figure 2.47 shear force diagram

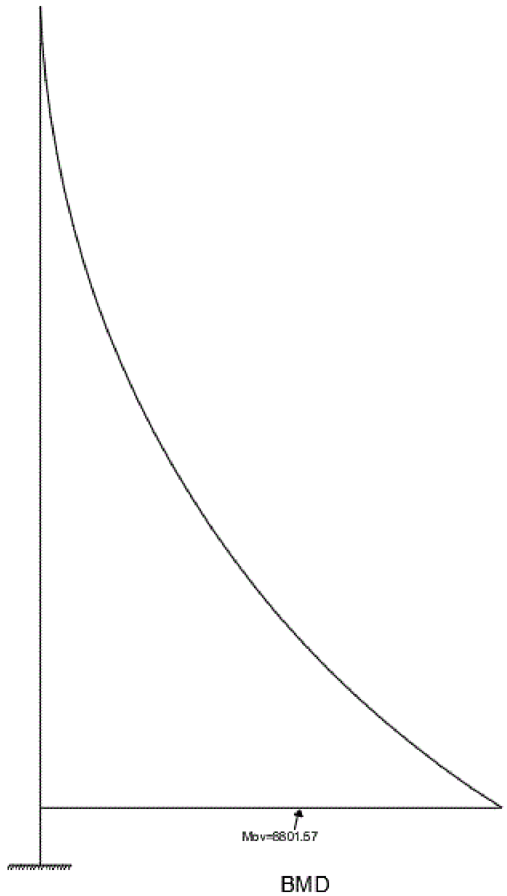


Figure 2.48 bending moment diagram

Check of stresses between cores and Foundation

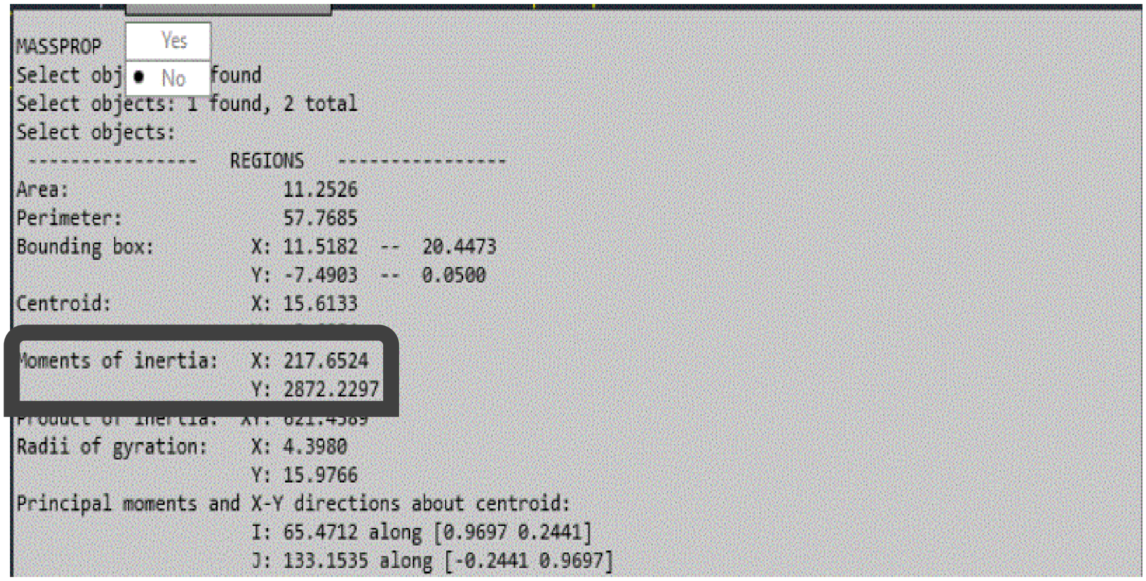
Inertia of Walls

- **Core 1 & 2**

A = 11.2526 m^2

I_X = 217.6524 m^4

I_Y = 2872.2297 m^4



```
MASSPROP                Yes
Select obj  ● No   found
Select objects: 1 found, 2 total
Select objects:
 ----------------    REGIONS    ----------------
Area:                    11.2526
Perimeter:               57.7685
Bounding box:         X: 11.5182  --  20.4473
                      Y: -7.4903  --  0.0500
Centroid:             X: 15.6133
                      Y: [illegible]
Moments of inertia:   X: 217.6524
                      Y: 2872.2297
Product of inertia:  XY: [illegible]
Radii of gyration:    X: 4.3980
                      Y: 15.9766
Principal moments and X-Y directions about centroid:
                      I: 65.4712 along [0.9697 0.2441]
                      J: 133.1535 along [-0.2441 0.9697]
```

Figure 2.49 Moment of inertia (Ix) , (Iy)

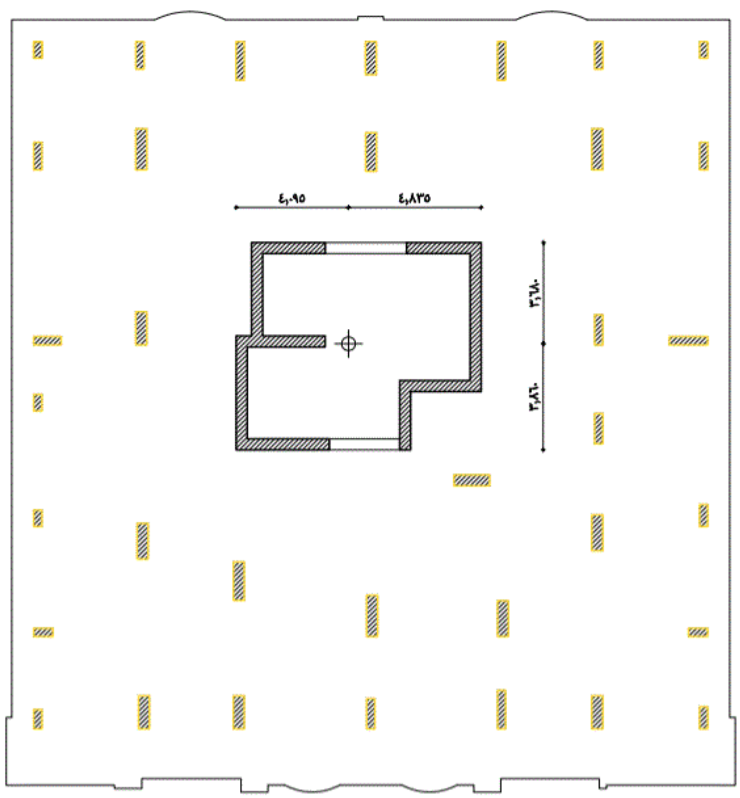


Figure 2.50 center of mass

In Y Direction

$F_{1,2} = \frac{-N}{A} \pm \frac{M_x * Y}{I_x} \leq 1.25\ Fc$

$A_{core} =$ 11.2526 m^2 (From CAD Program)

N_{wall} =2826.03 (From ETAP Program)

M_x = 8801.57 t.m

Y = 3.86 m

$I_{x\ group}$ = 217.6524 m^4

$F_{1,2} = \frac{-2826.03}{11.2526} \pm \frac{8801.57 * 3.86}{217.6524}$

$F_1 = \frac{-2826.03}{11.2526} - \frac{8801.57 * 3.86}{217.6524}$ = -40.72 Kg/Cm^2 <1.25 * 105 = 131.25 Kg/Cm^2

$F_2 = \frac{-2826.03}{11.2526} + \frac{8801.57 * 3.86}{217.6524}$ = - 1.64 Kg/Cm^2 <1.25 * 105 = 131.25 Kg/Cm^2

There is no tension OK Safe

In X Direction

$F_{1,2} = \frac{-N}{A} \pm \frac{M_x * Y}{I_x} \leq 1.25\ Fc$

$A_{core} =$ 11.2526 m^2 (From CAD Program)

N_{wall} =2826.03 (From ETAP Program)

M_x = 8801.57 t.m

X = 4.83 m

$I_{Y\ group}$ = 2872.2297 m^4

$F_{1,2} = \frac{-2826.03}{11.2526} \pm \frac{8801.57 * 4.83}{217.6524}$

$F_1 = \frac{-2826.03}{11.2526} - \frac{8801.57 * 4.83}{217.6524}$ = - 44.65 Kg/Cm^2 <1.25 * 105 = 131.25 Kg/Cm^2

$F_2 = \frac{-2826.03}{11.2526} + \frac{8801.57 * 4.83}{217.6524}$ = - 5.60 Kg/Cm^2 <1.25 * 105 = 131.25 Kg/Cm^2

There is no tension OK Safe

2.10.2 Check of Drifts

Story Response - Maximum Story Drifts

Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

Input Data

Name	StoryResp1		
Display Type	Max story drifts	Story Range	All Stories
Load Case	R.Spec X +ecc	Top Story	12
Output Type	Not Applicable	Bottom Story	footing

Plot

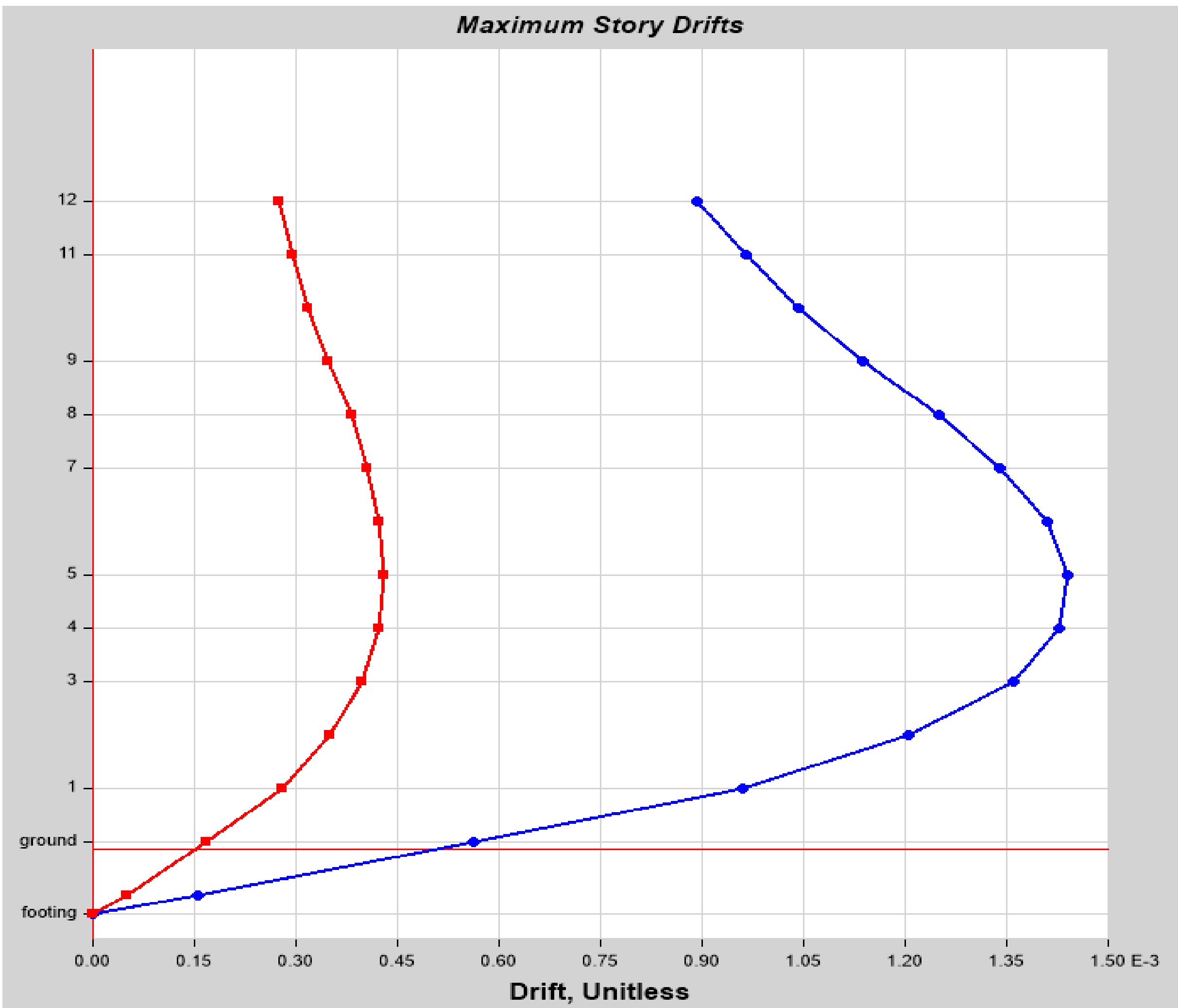


Figure 2.51 Maximum Story Drifts

Tabulated Plot Coordinates

Story Response Values

Story	Elevation	Location	X-Dir	Y-Dir
	m			
12	36.45	Top	0.000892	0.000274
11	33.45	Top	0.000964	0.000294
10	30.45	Top	0.001043	0.000319
9	27.45	Top	0.001138	0.000348
8	24.45	Top	0.001251	0.000381
7	21.45	Top	0.001339	0.000406
6	18.45	Top	0.001409	0.000422
5	15.45	Top	0.00144	0.000429
4	12.45	Top	0.001427	0.000422
3	9.45	Top	0.001359	0.000398
2	6.45	Top	0.001204	0.000349
1	3.45	Top	0.000961	0.00028
ground	0.45	Top	0.000562	0.000167
Beasment	-2.55	Top	0.000155	0.00005
footing	-3.6	Top	0	0

Table 2.6 Story Response Data

Max drift at X-direction = 0.000892 m

Max drift at Y-direction = 0.000274m

Allowable Drift according to ECP $=\frac{H}{500}=\frac{3}{500}$ =0.006 m Working Method

Ultimate Method = 0.006 *1.5 = 0.009 m

OK , Safe

Story Response - Maximum Story Drifts

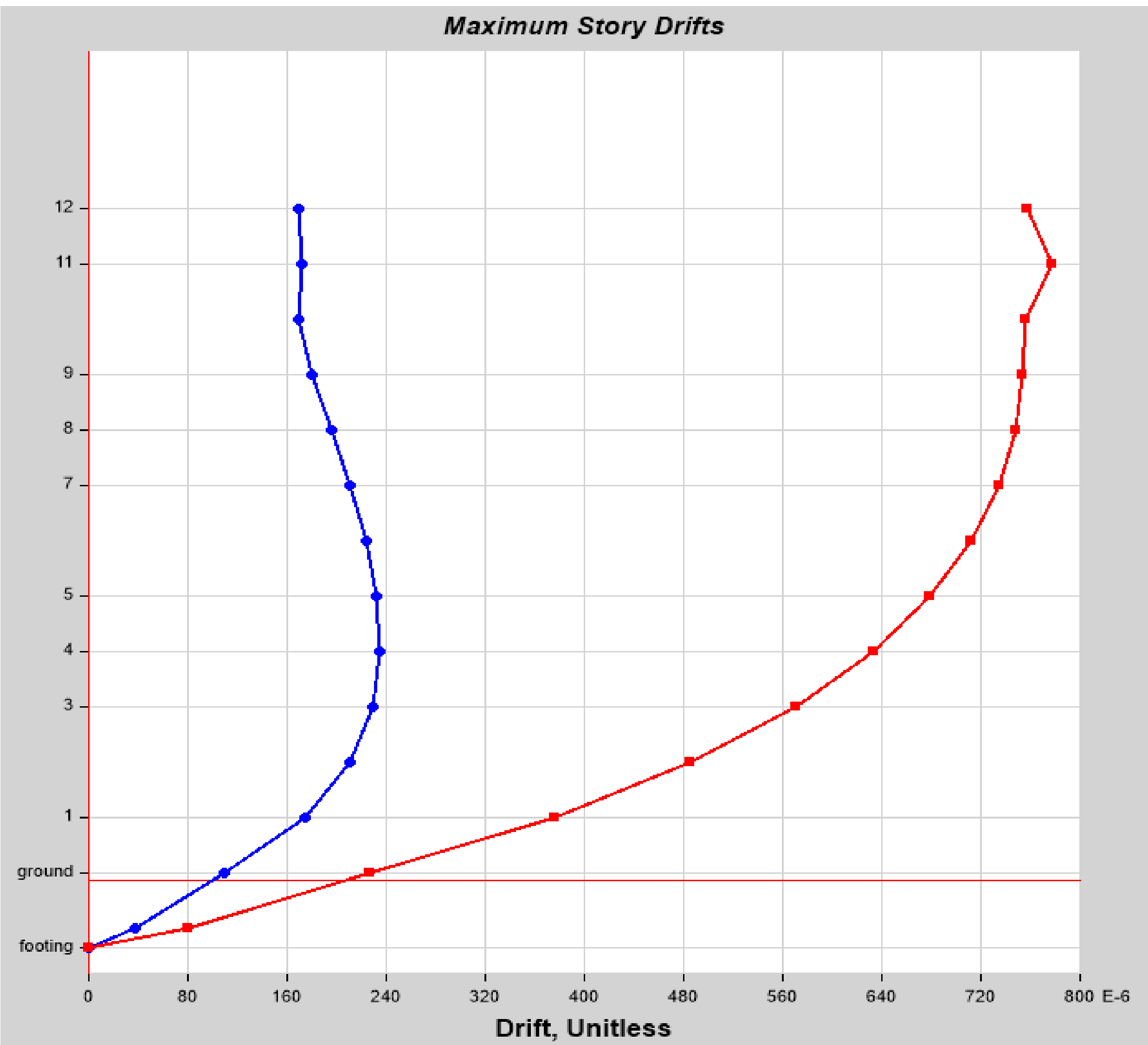
Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

Input Data

Name	StoryResp1		
Display Type	Max story drifts	Story Range	All Stories
Load Case	R.Spec Y+ecc	Top Story	12
Output Type	Not Applicable	Bottom Story	footing

Plot



Tabulated Plot Coordinates

Story Response Values

Story	Elevation	Location	X-Dir	Y-Dir
	m			
12	36.45	Top	0.000169	0.000757
11	33.45	Top	0.000172	0.000777
10	30.45	Top	0.000169	0.000756
9	27.45	Top	0.000181	0.000753
8	24.45	Top	0.000196	0.000747
7	21.45	Top	0.00021	0.000734
6	18.45	Top	0.000224	0.000712
5	15.45	Top	0.000232	0.000679
4	12.45	Top	0.000235	0.000633
3	9.45	Top	0.000229	0.000571
2	6.45	Top	0.000211	0.000486
1	3.45	Top	0.000175	0.000376
ground	0.45	Top	0.00011	0.000226
Beasment	-2.55	Top	0.000038	0.00008
footing	-3.6	Top	0	0

2.10.3 Check of Displacement

Story Response - Maximum Story Displacement

Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

Input Data

Name	StoryResp1		
Display Type	Max story displ	Story Range	All Stories
Load Case	R.Spec X +ecc	Top Story	12
Output Type	Not Applicable	Bottom Story	footing

Plot

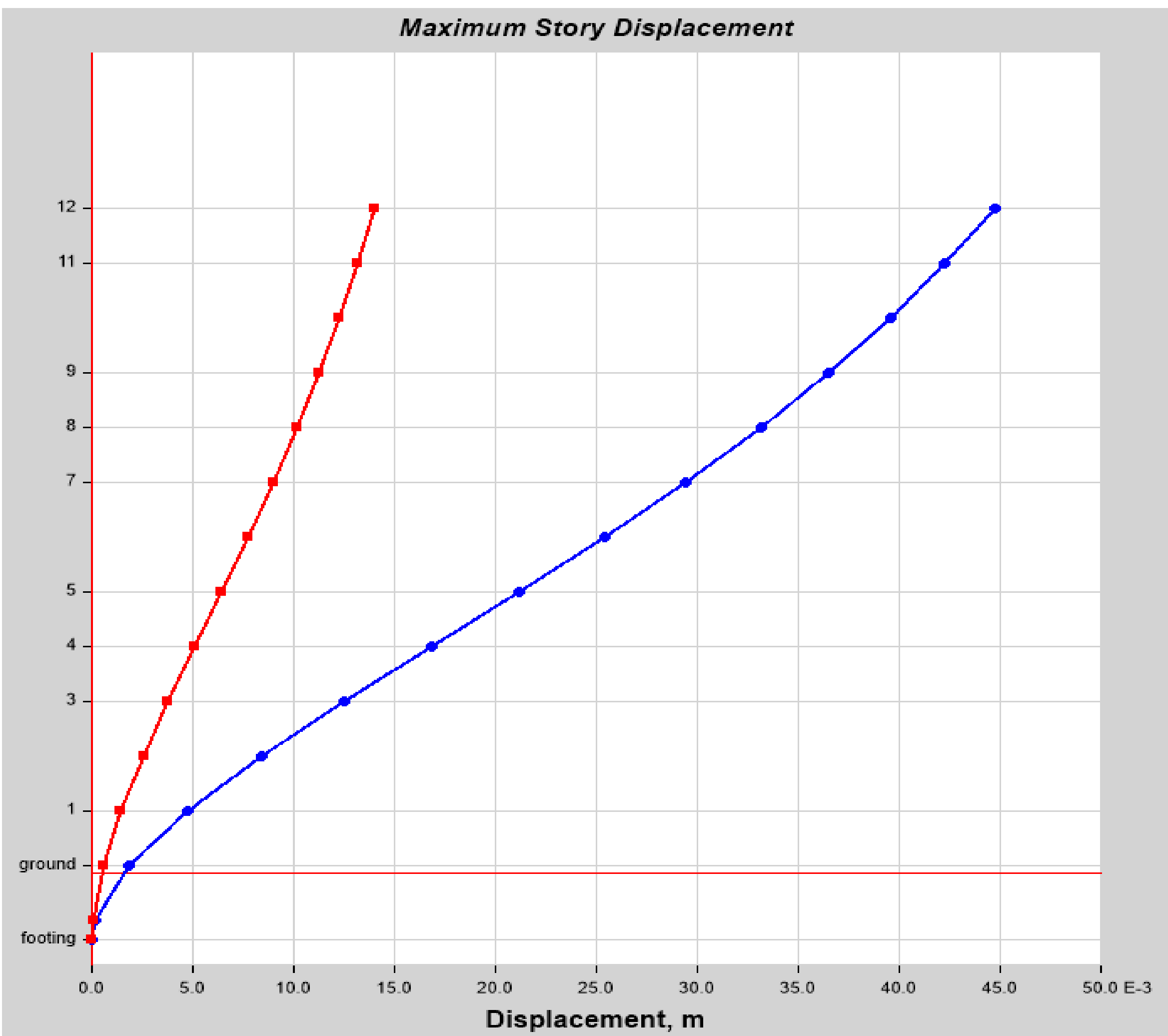


Figure 2.52 Maximum Story Displacement

Tabulated Plot Coordinates

Story Response Values

Story	Elevation	Location	X-Dir	Y-Dir
	m		m	m
12	36.45	Top	0.044779	0.013998
11	33.45	Top	0.042287	0.013153
10	30.45	Top	0.039548	0.012246
9	27.45	Top	0.036508	0.011263
8	24.45	Top	0.033136	0.010189
7	21.45	Top	0.029408	0.009009
6	18.45	Top	0.025404	0.007752
5	15.45	Top	0.021176	0.006438
4	12.45	Top	0.016844	0.005101
3	9.45	Top	0.012539	0.003787
2	6.45	Top	0.008432	0.002546
1	3.45	Top	0.004787	0.001453
ground	0.45	Top	0.001874	0.000578
Beasment	-2.55	Top	0.000167	0.000055
footing	-3.6	Top	0	0

Table 2.7 Story Response Values

Max Displacement at 13th Floor

- X-Direction = 0.044779
- Y-Direction = 0.013998

Height of Building = 36.45 m

Allowable Displacement according to ECP = $\frac{H}{500}$ =$\frac{36.45}{500}$ =0.0729 m Working Method

Ultimate Method = 0.0729*1.5 = 0.10935 m

OK , Safe

Story Response - Maximum Story Displacement

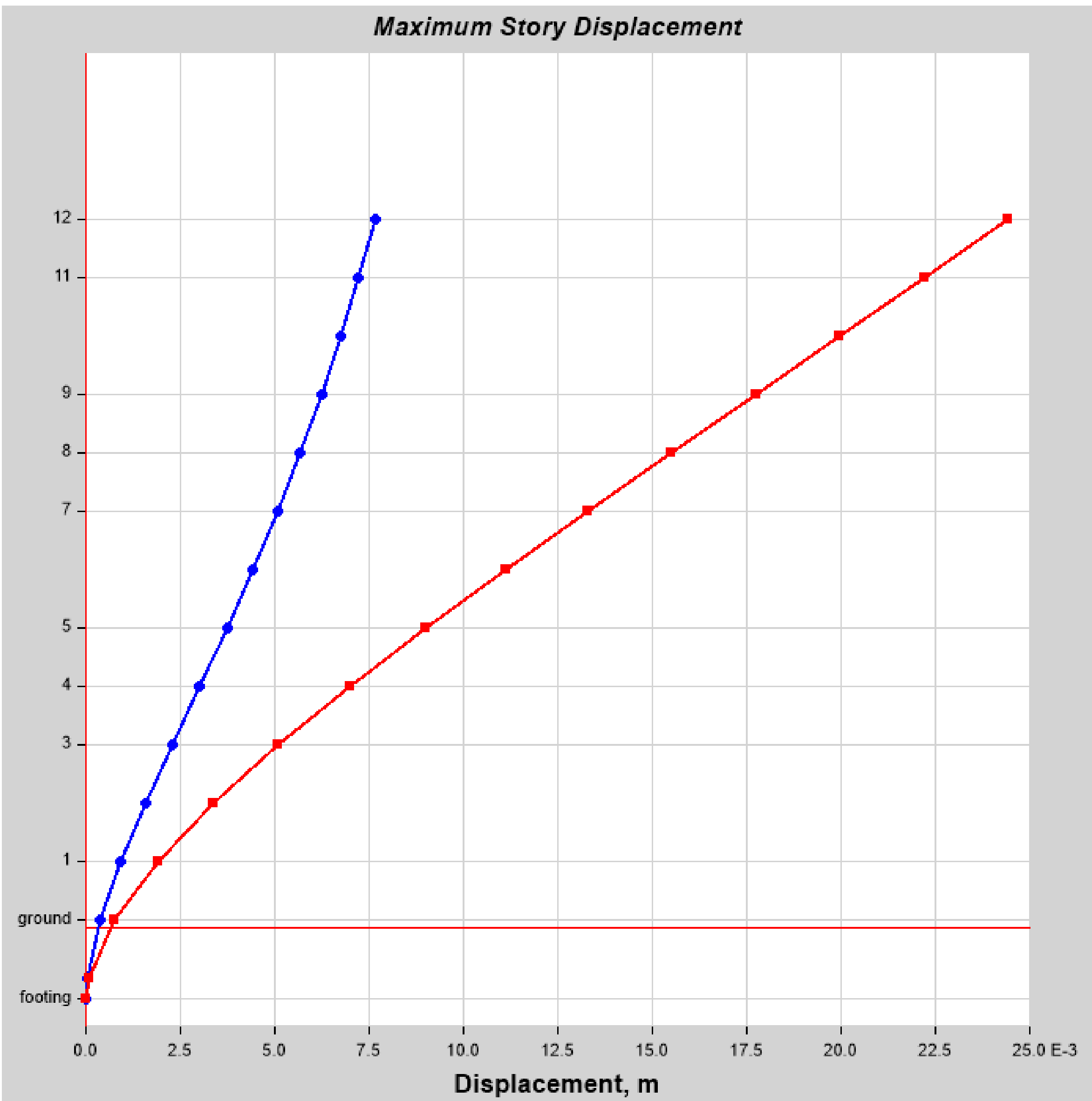
Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

Input Data

Name	StoryResp1		
Display Type	Max story displ	Story Range	All Stories
Load Case	R.Spec Y+ecc	Top Story	12
Output Type	Not Applicable	Bottom Story	footing

Plot



Tabulated Plot Coordinates

Story Response Values

Story	Elevation	Location	X-Dir	Y-Dir
	m		m	m
12	36.45	Top	0.007678	0.024425
11	33.45	Top	0.007222	0.02221
10	30.45	Top	0.006742	0.019948
9	27.45	Top	0.006233	0.017732
8	24.45	Top	0.005679	0.015518
7	21.45	Top	0.005072	0.013312
6	18.45	Top	0.004421	0.011137
5	15.45	Top	0.003731	0.009018
4	12.45	Top	0.003016	0.006988
3	9.45	Top	0.00229	0.005089
2	6.45	Top	0.00158	0.003371
1	3.45	Top	0.000925	0.001906
ground	0.45	Top	0.000385	0.00077
Beasment	-2.55	Top	0.000042	0.000085
footing	-3.6	Top	0	0

2.10.4 Check of Maximum Distance Between C.M , C.R

TABLE: Centers Of Mass And Rigidity											
Story	**Diaphragm**	**Mass X**	**Mass Y**	**XCM**	**YCM**	**Cum Mass X**	**Cum Mass Y**	**XCCM**	**YCCM**	**XCR**	**YCR**
		tonf-s²/m	tonf-s²/m	m	m	tonf-s²/m	tonf-s²/m	m	m	m	m
Beasment	Basement	104.25508	104.25508	8.0122	15.3068	104.25508	104.25508	8.0122	15.3068	7.9253	14.8189
ground	Ground	119.94695	119.94695	8.0075	14.9531	119.94695	119.94695	8.0075	14.9531	7.9497	14.8855
1	1	119.94695	119.94695	8.0075	14.9531	119.94695	119.94695	8.0075	14.9531	7.8843	14.9093
2	2	119.70222	119.70222	8.0075	14.9564	119.70222	119.70222	8.0075	14.9564	7.8339	14.9576
3	3	119.4575	119.4575	8.0075	14.9598	119.4575	119.4575	8.0075	14.9598	7.8014	15.0111
4	4	119.31219	119.31219	8.0075	14.9598	119.31219	119.31219	8.0075	14.9598	7.7849	15.0666
5	5	119.16688	119.16688	8.0075	14.9597	119.16688	119.16688	8.0075	14.9597	7.7759	15.1236
6	6	118.98333	118.98333	8.0075	14.9565	118.98333	118.98333	8.0075	14.9565	7.7741	15.1749
7	7	118.79978	118.79978	8.0075	14.9532	118.79978	118.79978	8.0075	14.9532	7.7762	15.2229
8	8	118.65447	118.65447	8.0076	14.9532	118.65447	118.65447	8.0076	14.9532	7.781	15.2667
9	9	118.50915	118.50915	8.0076	14.9531	118.50915	118.50915	8.0076	14.9531	7.7867	15.3103
10	10	118.15764	118.15764	8.017	14.9565	118.15764	118.15764	8.017	14.9565	7.795	15.3514
11	11	118.05588	118.05588	8.0197	14.9583	118.05588	118.05588	8.0197	14.9583	7.8273	15.3898
12	12	109.0312	109.0312	8.0098	14.9761	109.0312	109.0312	8.0098	14.9761	7.8831	15.4161

Table 2.8 Story Response Values

Max C.M

- at X = -2.51 m
- at Y = - 3.79 m

Max C.R

- at X = -2.4557 m
- at Y = -4.3578 m

Difference Between C.M , C.R

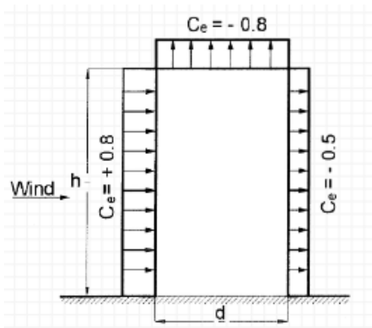
- at X = 2.51 - 2.4557 = 0.0543 m
- at Y = 3.79 – 4.3578 = -0.5678 m

Allowable Difference ≮ 0.15 b = 0.15*24.57 = 3.6855 m ⇛ OK Safe
≮ 0.15 t = 0.15*25 = 3.75 m ⇛ OK Safe

2.11 Effect of Wind on tower

2.11.1 Manual solution

- **Ct = 1** ⇛ معامل طبوغرافيه الارض
- **Cs = 1** ⇛ معامل المنشأ
- **ρ** = 1.25 Kg/m^3 ⇛ كثافه الهواء
- V =30 m/sec^2 ⇛ سرعه الرياح الاساسيه
- K ⇛ معامل التعرض ويتغير حسب الارتفاع عن سطح الارض
- Ce = 0.8 + 0.5 = 1.30 ⇛ معامل ضغط الرياح الخارجي علي سطح المبني



$F = q * Ce * K * A_{story} = Pe * A_{story}$

- $q = 0.5 * 10^{-3} * \rho * V^2 * Ct * Cs = 56.25\ Kg/m^2$
- $Pe = q * Ce * K = 56.25 * 10^{-3} * 1.3 * k = 0.073\ K$
  - $Pe1 = 0.073 * 1 = 0.073\ ton/m^2$
  - $Pe2 = 0.073 * 1.15 = 0.084\ ton/m^2$
  - $Pe3 = 0.073 * 1.4 = 0.102\ ton/m^2$
  - $Pe4 = 0.073 * 1.6 = 0.117\ ton/m^2$

F1 = 0.073 * 10 * 27.85 = 20.33 ton

F2 = 0.084 * 10 * 27.85 = 23.39 ton

F3 = 0.102 * 10 * 27.85 = 28.41 ton

F1 = 0.117 * 10 * 27.85 = 34.54 ton

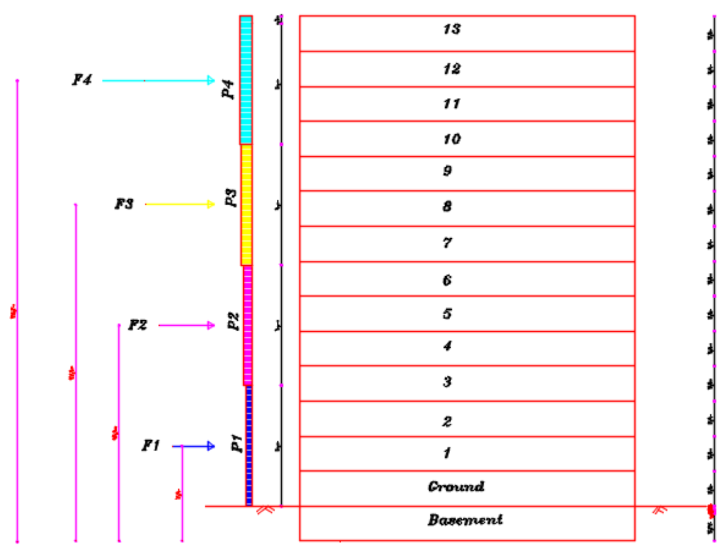


Figure 2.53 Wind Load

Check Of Over Turning Moment

Wt = 16667.85 ton

Resisting Moment = 16667.85 * $\frac{27.85}{2}$ =232099.8 ton.m

Over Turning Moment =20.33 * 7.9 + 23.39 * 17.90 + 28.41 * 27.90 + 34.54 * 38.30 = 2691.355 ton.m

Factor Of Safety = $\frac{\text{Resisting Moment}}{\text{Over Turning Moment}} = \frac{232099.8}{2691.355}$ = 86.24 > 1.50

Ok, Safe

Check Of Sliding Force

Ft =F1+F2+F3+F4 =106.67 ton

Resisting Force = **μ** * Wt = 0.3 *16667.85 =5000.36 ton

Factor Of Safety = $\frac{\text{Resisting Force}}{\text{Ft}}$ = 46.88 > 1.50

Ok, Safe

2.12 Design of Deep Foundation (Raft on Piles)

Use Thickness of Raft = 100 cm
Pile diameter = 60 cm
Pile capacity = 100 ton

Total building weight (working method) = reactions of column , core+$O.w_{R.w}$ + $O.w_{raft}$

- Reactions of columns (Etabs program) = 11111.9 ton
- $O.W_{R.W} = c*t_w*h_w*L_w$ = 2.5*0.4*2.9*28.29 = 82.04 ton
- $O.W_{raft}$ = ($o.w_{slab}$+ cover +W_{wall}+ car weight) *area

  = [(2.5*1)+0.15+0.5)*664.82 = 2094.18 ton

Total building weight = 11111.9+82.04+2094.18 = 13288.12 ton

No. of piles = total building / pile capacity

=13288.12/100 =133 piles

Increase no. of piles by 20% to carry lateral loads generated by earthquakes.

No, of piles = 133 *1.25 = 167 ~ 169 piles

Piles spacing (s) = (3→7) D

s = 2 m “ s = 3.33 D “

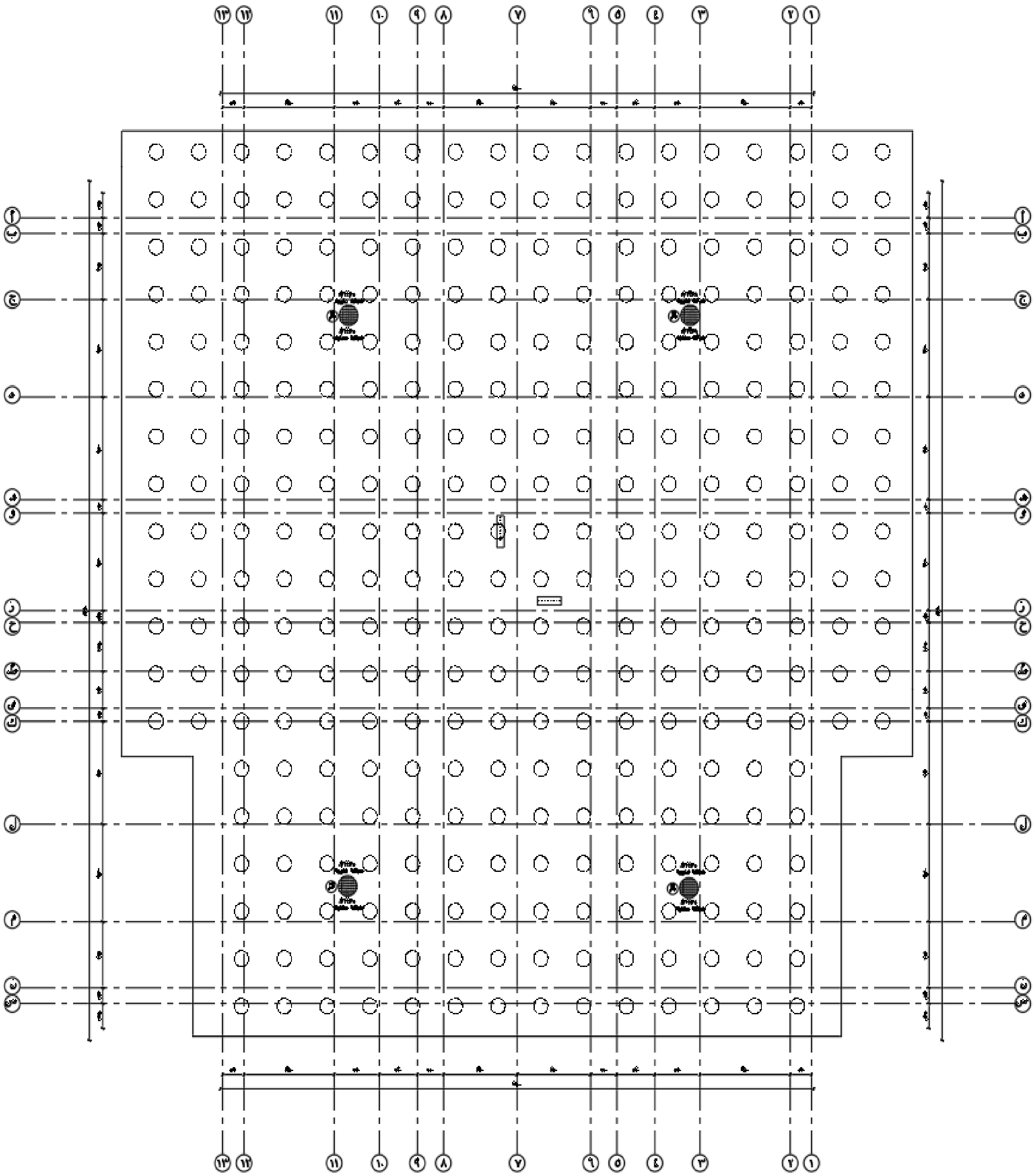


Figure 2.54 Piles arrangement

2.12.1 Design of Raft on Piles (Safe program)

$t_{raft} = 100$ cm

$d_{pile} = 60$ cm

$P = K\,\Delta \rightarrow P = 100$ ton , $\Delta = 0.6$ cm

$K_{stiffness} = p/\Delta = 100/0.006 = 16666.67$ t/m

Use 85% $K_{stiffness} = 0.85 * 16666.67 = 14166.67$ t/m

- For raft

  $A_s = (Mu)/ f_y J d$

$M_u = As * F_y * J * d = 8 * (\pi/4*2^2) * 3500 * 95 * 0.829*10^{-5} = 70$ t.m

- Use 9 ∯ 25 Upper and Lower Reinforcement

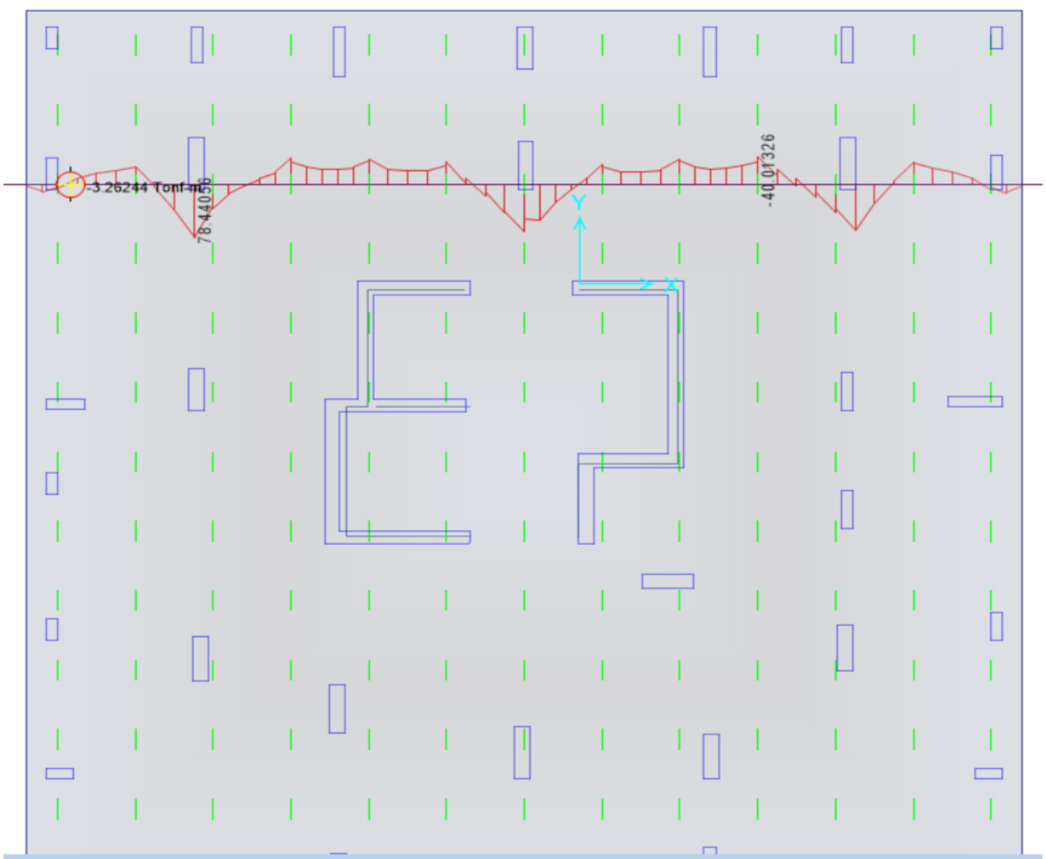


Figure 2.55 Piles Strip

In X–Direction (Upper)

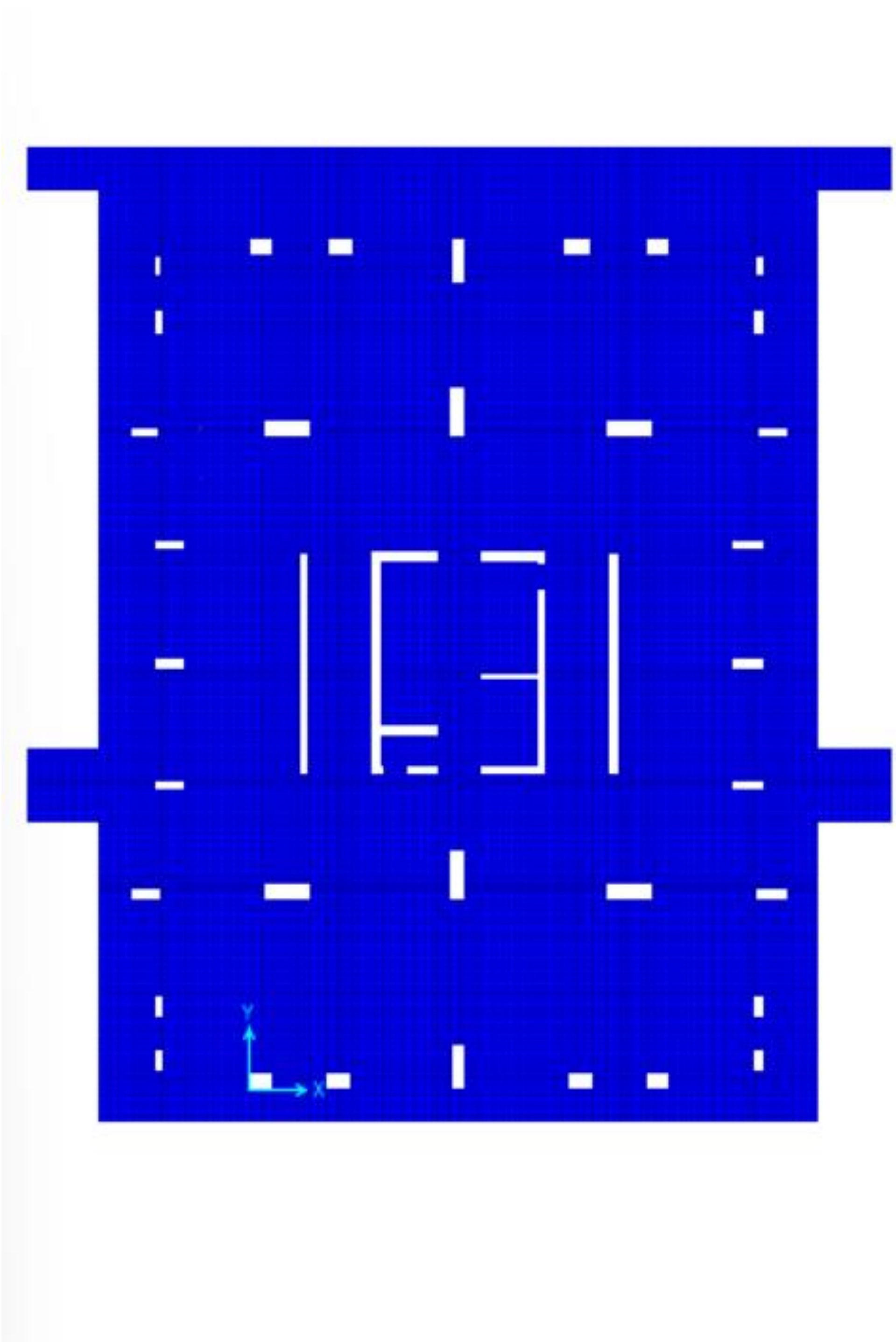


Figure 2.56 Reinforcement in X-Direction (Upper)

In X–Direction (Lower)

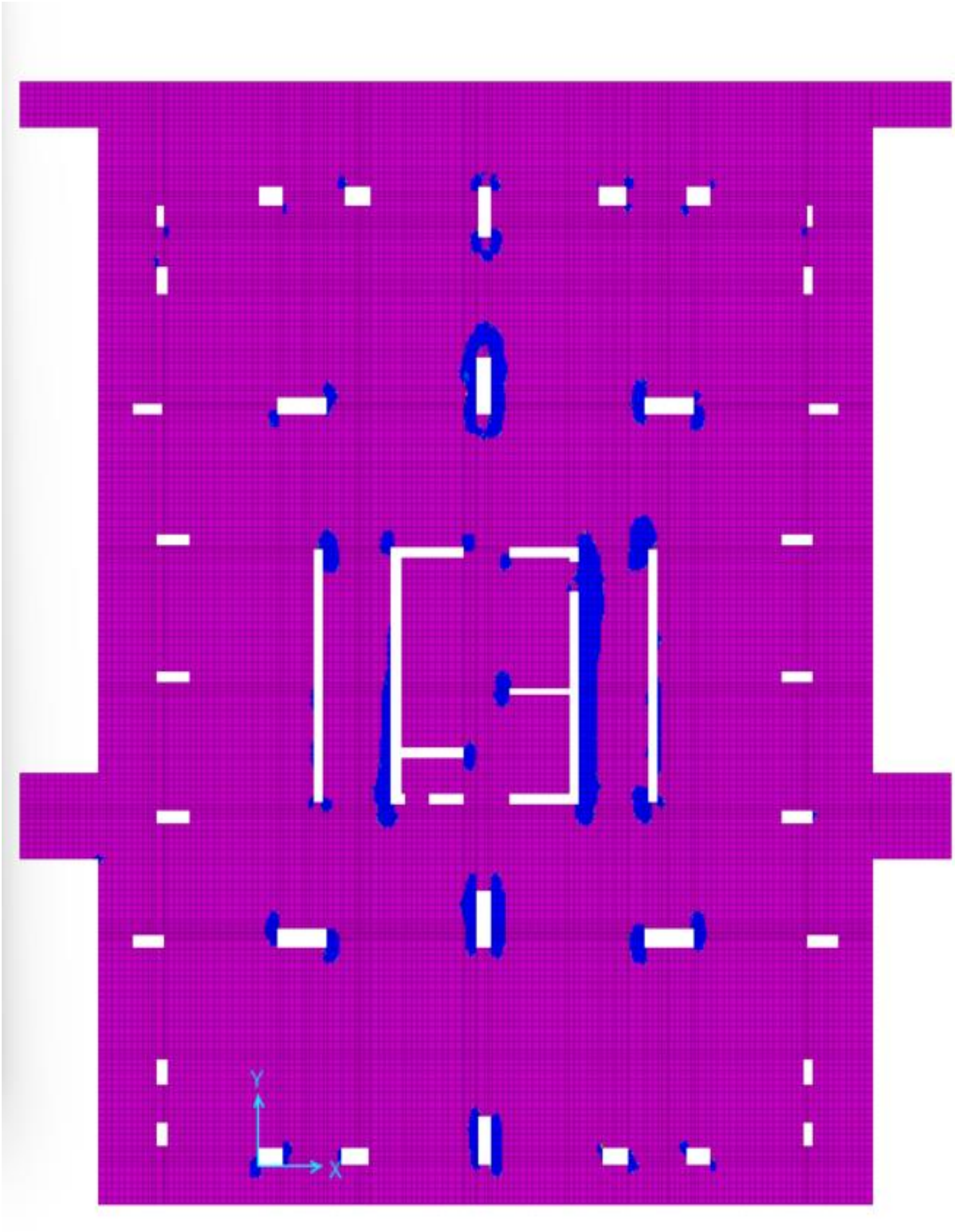


Figure 2.57 Reinforcement in X-Direction (Lowe

In Y–Direction: (Upper)

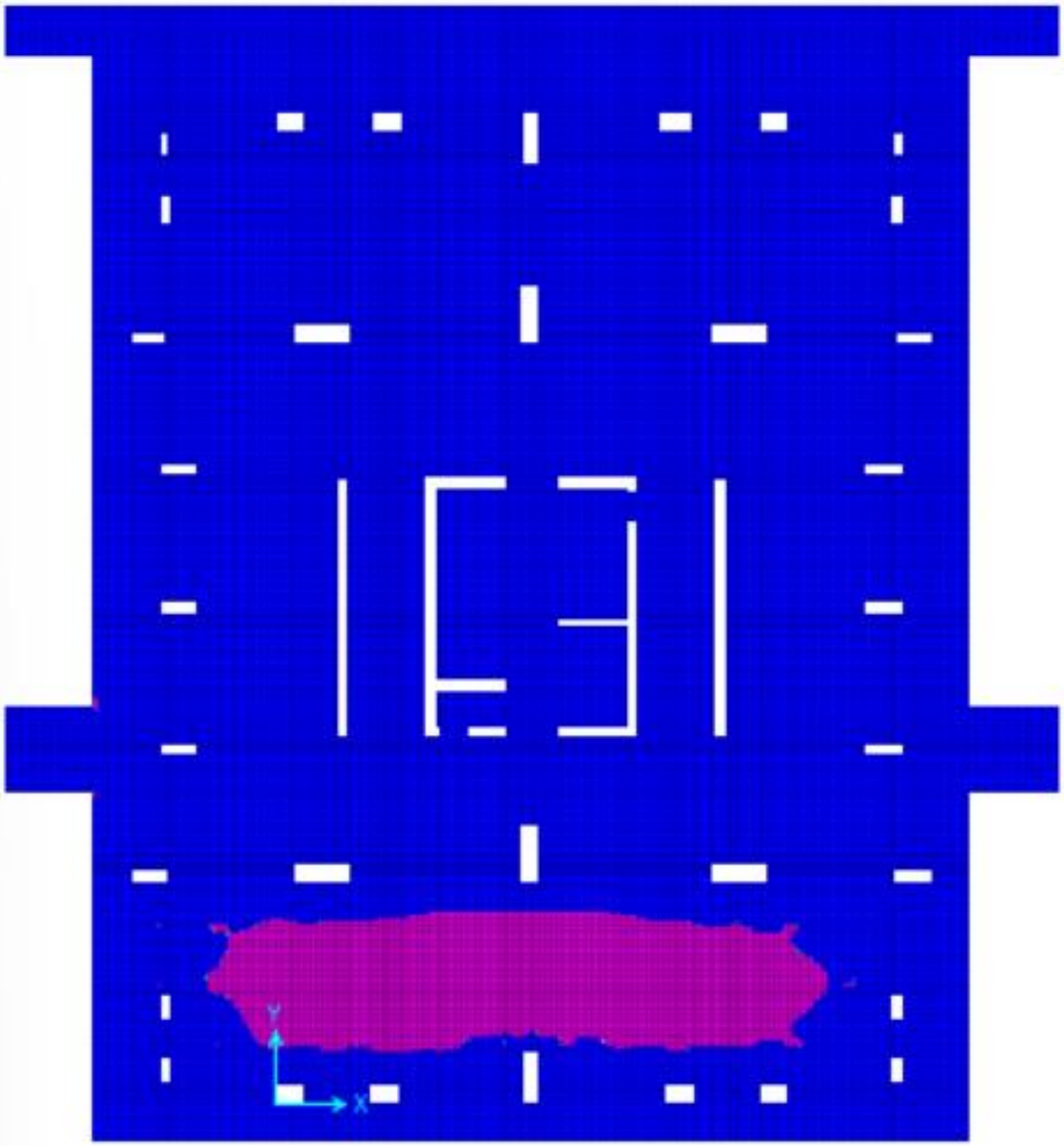


Figure 2.58 Reinforcement in Y-Direction (Upper)

In Y–Direction (Lower)

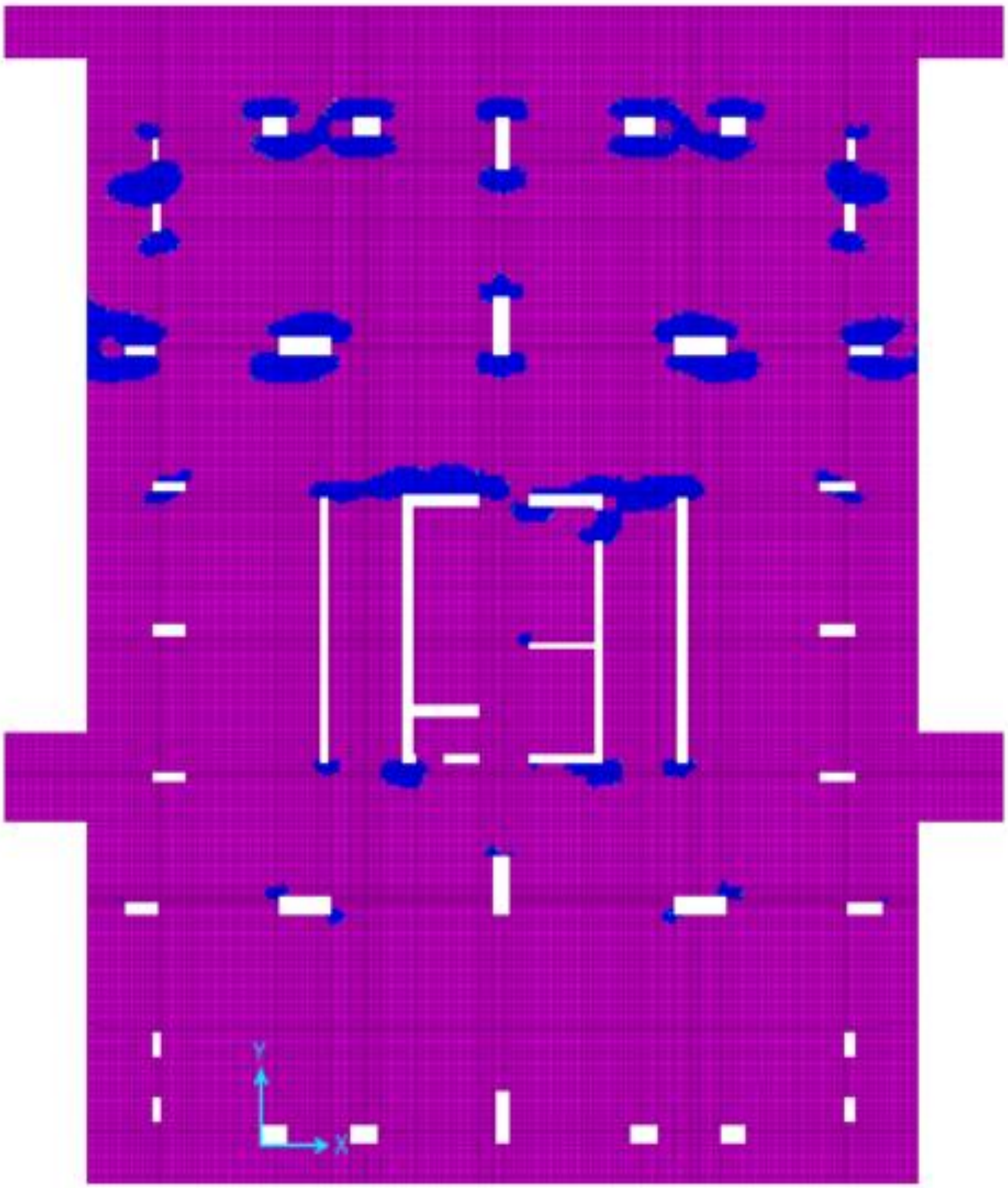


Figure 2.59 Reinforcement in Y-Direction (Lower)

Table of piles forces :-

pile No	FORCE
1	83.3763
2	87.4334
3	98.4253
4	97.1011
5	95.2695
6	80.2419
7	93.7744
8	88.4238
9	93.4233
10	95.5839
11	97.9515
12	88.8143
13	89.385
14	62.2983
15	66.599
16	72.6417
17	76.8144
18	77.3228
19	76.6828
20	78.939
21	77.4807
22	80.0425
23	79.1054
24	73.5027
25	68.4754
26	66.9886
27	48.9535
28	54.3195
29	61.3608
30	68.1932
31	70.2892
32	68.8381
33	74.2002
34	69.4782
35	71.9346
36	69.1988
37	61.7921

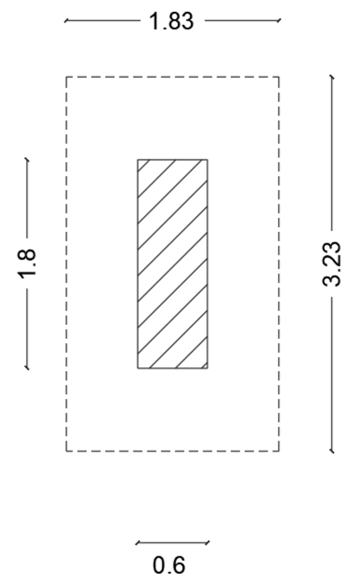
38	58.071
39	53.0247
40	42.372
41	50.1951
42	62.1989
43	64.3448
44	65.5534
45	62.8536
46	63.2888
47	64.3004
48	65.4
49	63.728
50	61.8118
51	52.715
52	48.5802
53	40.6443
54	48.7168
55	55.1731
56	61.402
57	65.2308
58	65.1405
59	65.5345
60	69.1048
61	71.9708
62	64.7704
63	59.8839
64	51.2712
65	44.6586
66	42.1811
67	46.604
68	54.4404
69	65.9011
70	75.9834
71	76.2798
72	76.3354
73	78.5918
74	76.7523
75	67.9029
76	61.0663
77	51.2208
78	42.6702

79	48.608
80	52.0447
81	60.3988
82	72.6688
83	83.3125
84	82.6082
85	82.3127
86	86.4791
87	83.7589
88	70.9253
89	62.185
90	54.1646
91	47.6424
92	56.0051
93	56.5648
94	67.5524
95	76.4997
96	87.7379
97	85.9871
98	82.6637
99	83.4986
100	83.7074
101	70.469
102	63.0346
103	56.3735
104	52.9948
105	43.5024
106	51.1874
107	60.2088
108	70.304
109	82.6168
110	81.7463
111	79.4102
112	80.5481
113	80.0031
114	66.2558
115	58.8243
116	50.6784
117	48.4384
118	44.429
119	50.991

120	57.564
121	63.6728
122	70.9001
123	74.2763
124	74.8329
125	74.1561
126	70.0266
127	61.9885
128	56.3249
129	51.7102
130	48.0173
131	56.0825
132	58.9045
133	65.6508
134	62.9049
135	64.2608
136	69.1499
137	75.7127
138	69.4689
139	64.3792
140	63.088
141	66.6042
142	61.0578
143	59.7899
144	59.2994
145	65.1368
146	70.3508
147	69.267
148	69.0611
149	71.7798
150	76.1903
151	72.1636
152	69.5983
153	70.2623
154	72.0768
155	68.1336
156	65.1124
157	67.6513
158	71.987
159	78.6392
160	80.8032

161	81.9058
162	80.6596
163	87.946
164	80.967
165	82.4468
166	81.9967
167	80.7885
168	76.097
169	73.5667

2.12.1.1Check of punching shear on raft



- for 1 ع (60 * 180)

Col load = 769 ton

t_{raft} = 150 cm

d = t_{raft} – cover = 150 -7 =143 cm

$q_{up} = P_{col} / 2((a+d)+(b+d))*d$

$=769 * 10^3 / 2((183+323))*143 = 5.31\ kg/cm^2 < 10\ kg/cm^2$

Figure 2.60 Punching shear of column

OK . Safe

- **For pile no. (13)**

Pile load = 99 ton

d_{pile} = 60 cm

$q_{up} = Q_{up}/\pi D d = (99 * 10^3) / (\pi * 203 * 143)$

$= 1.08\ kg/cm^2 < 10\ kg/cm^2$ OK . Safe

Figure 2.61 Punching shear of pile

Uinit (3)
Elevated Tank

3.1 INTRODUCTION

3.1.1 Intze tank consists of:

- Covering floor
- posts
- Covering Cone of Tank
- Horizontal Ring Beam
- Circular Wall
- Design of Wet Cone
- Effect of earthquakes on tank
- Check of stresses between shaft and foundation
- Design of Foundation

3.1.2 Material Properties Used:

- F_{cu}=30 N/mm^2
- $F_{y(main\ steel)}$=350 N/mm^2
- $F_{y(stirrups)}$=240 N/mm^2
- Bearing Capacity of Soil = 10 t/m^2

3.1.3 Cover Thickness ≮ 3 cm

3.1.4 Loads Used:

- L.L= 0.1 t/m$_2$
- Cover = 0.05 t/m$_2$

3.1.5 Design Method:

- Ultimate Limit State Design $ Working Method Design

3.2 Dimensions

Figure 3.1 Elevated Tank Dimensions

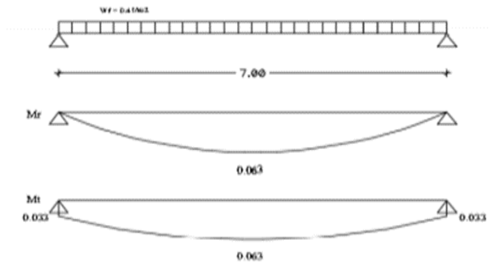
- Capicity Of Water (V) =800 m^3
- W_{water} = $\left[\left(\frac{\pi*7^2}{4}*7.5\right)-\left(\frac{\pi*(1.5)^2}{4}*7.5\right)\right]+\left[\frac{D-7}{2}*2.5*2\,\pi\left(\frac{D-7}{4}+3.5\right)\right]+\left[\frac{D-7}{2}*\ 0.5*5*\ 2\,\pi*\left(\frac{D-7}{4}+3.5\right)\right]$
- 800 = 275.4 + 1.96 $(D-7)^2$ + 27.5(D -7) + 1.3$(D-7)^2$ + 27.5(D -7)
- 800 = 275.4 + 3.26 $(D-7)^2$ + 55(D -7)
- D1 = 14 m

3.3.1 Covering Floor

- assume ts = 10 cm
- L.L = 100 kg/m^2 = 0.1 t/m^2
- Covering = 50 kg/m^2 = 0.05 t/m^2
- W = $[\gamma_c . ts + Fc] + L.L$
- W = 0.1*2.5+0.05+0.1 = 0.4 t/m^2
- M = coeff * W *r^2

  = 0.2*0.4*$(3.5)^2$= 0.98 t.m

  Use 5 Ø 10 /m



(Neglect) Figure 3.2 Covering Floor

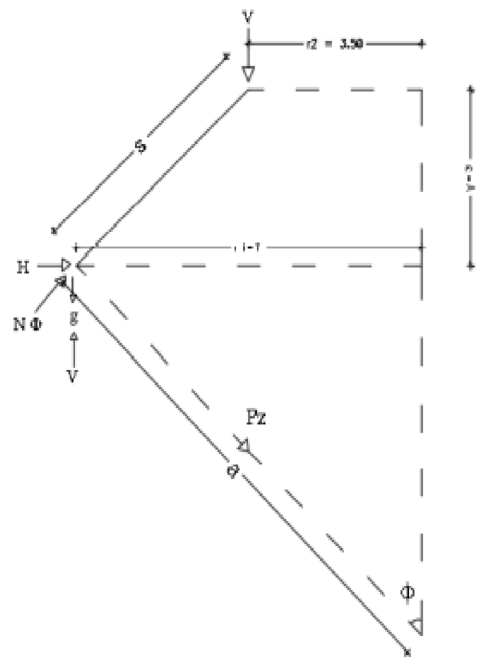
3.3.3 Covering Cone

V =$\frac{\pi * D2}{4} * H_O$

Assume $H_O = 0.6 D_s$ $D_s = 10\ m$ $H_O = 6\ m$

(1) Geometric Design

- r = $\frac{D}{2}$ = $\frac{10}{2}$ = 5 m
- y = $\frac{r}{2}$ = 2.5 m
- S = $\sqrt{5^2 + 2.5^2}$ = 5.59 m
- Sin Φ = $\frac{2.5}{5.59}$ = 0.45 m
- Cos Φ = $\frac{5}{5.59}$ = 0.89 m
- tan Φ = $\frac{2.5}{5}$ = 0.5 m
- a = $\frac{r*s}{y}$ = $\frac{5*5.59}{2.5}$ =11.18 m
- Assume That:
  - t = 0.12 m
  - Covering = 0.05 t/m^2
  - L.L = 0.1 t/m^2



(2) Loads

assume ts = 10 cm

g = $[\gamma_c . ts + Fc]$ = (0.1*2.5) + 0.05 = 0.3 t/m^2

L.L =0.1 t/m

W_{total} = g π s * r + L.L π r^2

= 0.3 * π * 5.59*5 + (0.1* π * 5^2) = 34.2 t

Pz = g cos Φ + L.L cos^2 Φ = 0.3*0.89 + 0.1*$(0.89)^2$ = 0.35 ton

(3) Internal Forces (At Footring)

- $V = \frac{W}{\pi * D} = \frac{34.2}{\pi * 10} = 1.09\ t/m$
- $N\Phi_{\ comprission} = \frac{V}{\text{Sin}\ \Phi} = \frac{1.09}{0.45} = 2.44$ t/m
- $H = \frac{V}{\tan \Phi} = \frac{1.09}{0.5} = 2.18$ t/m
- $N\theta_{\ comprission} =$ Pz * a = 0.35 * 11.18 = 3.913 t/m

(4) Check Of Stress

- For Nθ $Fc = \frac{N\theta}{b.t} = \frac{3.913*1000}{100*10} = 3.913\ kg/cm^2 \leq$ Fc all = 70 kg/cm^2
- For NΦ $Fc = \frac{N\Phi}{b.t} = \frac{2.44*1000}{100*10} = 2.44\ kg/cm^2 \leq$ Fc all = 70 kg/cm^2

(5) Moment At Edge

- $X = 0.6\sqrt{a\ .t} = 0.6\sqrt{11.18 * 0.15} = 0.78\ m$
- $M = \frac{W\ .\ X^2}{2} = \frac{0.4*(0.78)^2}{2} = 0.122$ m.t

Neglected

Use 6 Ø 10 /m for NΦ , Nθ

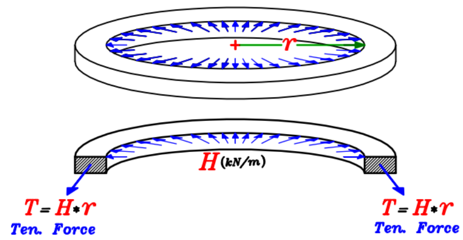


Figure 3.5 Horizontal Ring Beam

3.3.4 Horizontal R-Ring Beam

- Assume Beam (30 cm * 60 cm)
- T = H * r = 2.18 *5 = 10.9 ton
- $Fct = \frac{T}{A} = \frac{10.9*10000}{300*600} = 0.61\ N/mm^2$
- $Fctr = \frac{fct}{\mu} = \frac{0.6\sqrt{30}}{1.5} = 2.2\ N/mm^2$

Fct < Fctr OK . Safe

- $As = \frac{T_u}{\frac{F_y}{\Upsilon_s}} = \frac{1.4*10.9*10^3}{\frac{3500}{1.15}} = 5.014\ cm^2$
- $\frac{As}{each\ side} = \frac{5.014}{2} = 2.507\ cm^2$
- Use 4 ∯ 12 / *each side*

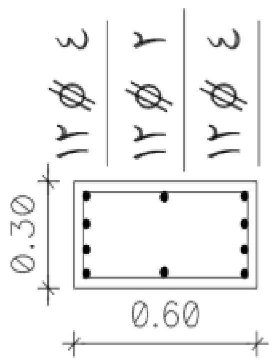


Figure 3.6 RFT Of Horizontal Ring Beam

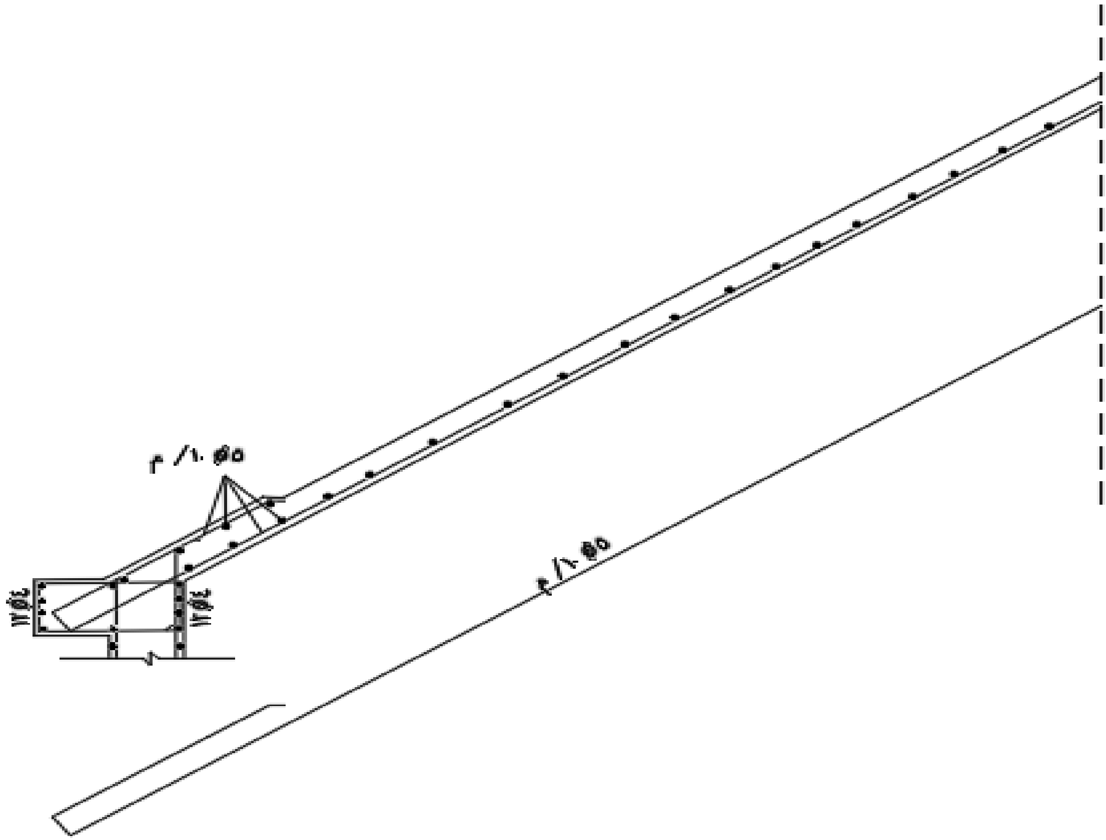


Figure 3.7 RFT Of Covering Cone

3.3.6 Design of Conical Wall

(1) Geometric Design

- S =3.25 m
- $\text{Sin}\ \Phi = \frac{3}{3.5}$ =0.923
- $\text{Cos}\ \Phi = \frac{2.5}{6.25} = 0.4$
- $\tan\Phi = \frac{6}{2.5} =$
- $a = \frac{r1*s}{h2} = \frac{5*3.25}{3}$ =5.42 m
- $a = \frac{r2*s}{h2} = \frac{3.75*3.25}{3}$ =4.06 m
- $a = \frac{r3*s}{h2} = \frac{2.5*3.25}{3}$ =2.71 m

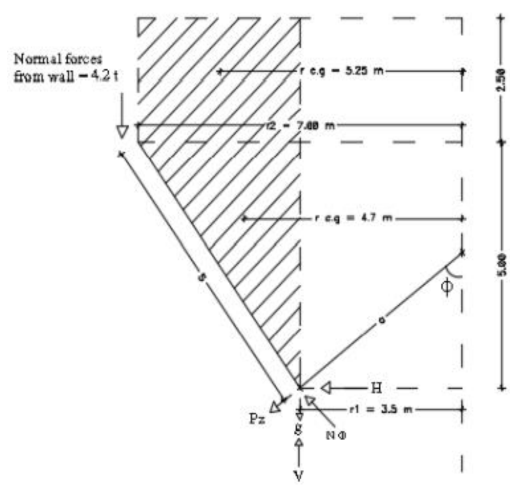


Figure 3.12 Conical Wall

Design for Section 1-1 :-

- $V_1 = 1.09\ t/m$
- $N\Phi_1$ compression $= \frac{V_1}{\text{Sin }\Phi} = \frac{1.09}{0.923} = 1.18$ t/m
- $N\theta_1$ Tension $=$ W Cos Φ_1 * a_1 = 1.5*0.4*5.42= 3.252 t/m

Design for Section 2-2 :-

- $W_{\Phi_2} = wt\ of\ cone\ above\ \text{sec} + wt\ of\ covering\ cone\ if\ any + wt\ of\ water\ above\ sec$
- $W_{cone} = w * A_{sur} = w * \pi * (r_1 + r_2)s = 1.5 * \pi * (5 + 3.75) * 3.25 = 134$ ton
- $W_{cover} = V_1 * \pi D = 1.09 * \pi * 10 = 34.2$ ton
- $W_{water} = \gamma w * Vw = \gamma w * A_1 * 2\pi r * cg_1 = 1 * 0.5 * 3 * 1.25 * 2\pi * \left(3.75 + \frac{1.25}{3}\right) =$ 49.90 ton
- $W_{\Phi_2} = 134 + 34.24 + 49.10 = 217.34\ ton$
- $V_2 = \frac{W_{\Phi_2}}{\pi D_1} = \frac{217.34}{2\pi * 3.75} = 9.22\ t/m$
- $N\Phi_2$ compression $= \frac{V_2}{\text{Sin }\Phi} = \frac{9.22}{0.923} = 9.99$ t/m
- $N\theta_2$ Tension $=$ (W Cos $\Phi_1 + \gamma w$ *h_2) a_2 = $\left(1.5 * \frac{2.5}{6.25} + 1 * 3\right)$*4.06= 14.62 t/m

Design for Section 3-3 :-

- $W_{cone} = w * A_{sur} = w * \pi * (r_2 + r_3)s = 1.5 * \pi * (3.75 + 2.5) * 3.25 = 95.72$ ton
- $W_{cover} = V_1 * \pi D = 1.09 * \pi * 10 = 34.2$ ton
- $W_{water} = \gamma w * Vw = \gamma w * A_2 * 2\pi r * cg_2 = 1 * 0.5 * 2.5 * 6 * 2\pi\left(2.5 + \frac{2.5}{3}\right) = 157.1$ ton
- $W_{\Phi_3} = 95.72 + 34.24 + 157.1 = 287.06\ ton$
- $V_3 = \frac{W_{\Phi_3}}{\pi D_3} = \frac{287.06}{\pi * 5} = 18.27\ t/m$
- $N\Phi_3$ compression $= \frac{V_2}{\text{Sin }\Phi} = \frac{18.27}{0.923} = 19.90$ t/m
- $N\theta_3$ Tension $=$ (W Cos $\Phi_1 + \gamma w$ *h_3) a_2 = $\left(1.5 * \frac{2.5}{6.25} + 1 * 6\right)$*2.71= 17.89 t/m

$$H = \frac{V}{\tan \Phi} = \frac{18.27}{\frac{6}{2.5}} = 7.61\ t/m$$

- As = $\frac{1.4 * N\theta}{Bcr\frac{Fy}{\gamma s}} = \frac{1.4 * 17.89 * 10^3}{0.93 * \frac{3500}{1.15}} = 8.85\ cm^2$
- As / side = $\frac{8.85}{2} = 4.424\ cm^2$

Use 5 Ø 12 / m /side

Check of Stresses :

Fct = $\frac{T}{Ac+nAs} = \frac{17.89*1000}{(60*100)+(10*2*5.65)}$ =2.93 kg / cm^2

Fc=$\frac{N\Phi_{max}}{Ac} = \frac{19.90*1000}{100*60}$ =3.30 kg / cm^2

Fct < Fctr OK . Safe

3.3.8 Circular floor with Circular hole

Hinged connection

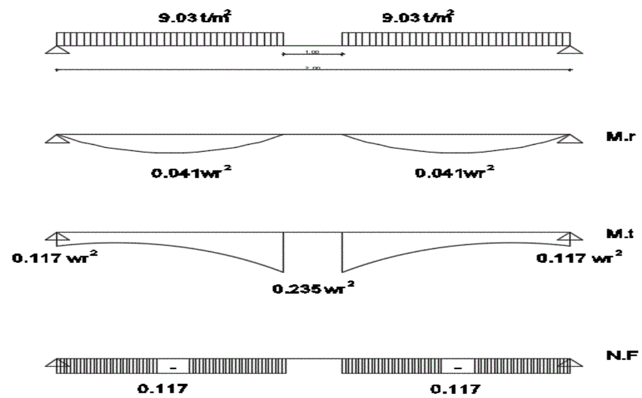


Figure 3.14 Mr & Mt & N f

Assume tf = 60 cm

Wf = ɣc * tf + ɣw * hw= (2.5*0.6) + (1*6)

=7.5 t/m2

O.W of area above floor= ɣc * tw * hw * π *D

=2.5*0.25*6.65* π *1.25=16.32 ton

Wt =7.5+$\frac{16.23}{\frac{\pi*(5^2-1^2)}{4}}$=8.37 t/m2

Mr = 0.041*8.37*(2.5)2=2.14 m.t

mt =0.117*8.37*(2.5)2= 6.12m.t

mt =0.235*8.37*(2.5)2=12.29 m.t

for sec 1-1

M=12.29 m.t , N= 7.61t

Ft=$\frac{-N}{b*t} + \frac{6M}{b*t^2} = \frac{-7.61*10^3}{100*60} + \frac{6*12.29*10^5}{100*60^2}$=19.215 kg/cm2>20 kg/cm2

e=$\frac{M}{N} = \frac{12.29}{7.61}$ =1.68 m > $\frac{t}{2}$ = 0.5 (big ecc)

es = e + $\frac{t}{2}$ – cover =1.68+0.3 - 0.05=1.93m

Mus = 1.4*7.61*1.93=20.56 m.t

d=c1 $\sqrt{\frac{M_u}{f_{cu}*b}}$

c1 > 5 ……………. J=0.826

As=$\frac{M_{us}}{f_y*j*d} = \frac{20.56*10^5}{0.826*3500*55} = 12.93\ cm2$

As/side = $\frac{12.93}{2}$ =6.47 cm2

As min = $\frac{0.15}{100}$ b*t = $\frac{0.15}{100}$ *100*60 =9 cm2

As/side = $\frac{13.5}{2}$ =6.75 cm2

Use 5Ø16/side

3.3.9 Effect of earthquakes on tank

Fb = sd(T1) * λ *$\frac{W}{g}$

T1= ct $(\mathrm{H})^{3/4}$ =0.05*$(20)^{3/4}$=0.473 sec

Seismic zone …… second zone……ag= 0.125*g= 1.23 m/sec2

Soil type B ……..s=1.35 ……TB=0.05 …….Tc=0.25 ……TD=1.2

Tc < T1 < TD

sd(T1) =ag* ɣI *s*$\frac{2.5\,\eta}{R}$ *($\frac{T_c}{T_1}$) ⩾ 0.2ag * ɣI

1.23*1.2*1.35*$\frac{2.5*1}{6}$ * $\frac{0.25}{0.473}$ =0.44⩾0.2*1.23*1.2=0.3

sd(T1) = 0.55

λ = 0.85 ……… T1 > 2

W = total weight of tank= Wtank + Wstair + Wshaft

For (W)tank=

$W_{cone} = 34.2\ ton$

$W_{HZB} = 2.5 * 0.3 * 0.6 * \pi * 10 = 14.14\ ton$

$W_{Wall} = 1.5 * \pi * (5 * 2.5) * 6.5 = 229.73\ ton$

$W_{Water} = \text{ɣw} * \text{Vw} = 1 * 400 = 400\ \text{ton}$

$W_{Floor} = 2.5 * 0.6 * \left(\frac{\pi * (5)^2}{4}\right) = 29.45\ ton$

Wtank=707.52 ton

Wstair=30 t

Wshaft=2.5*20*$\frac{\pi*(4^2-3^2)}{4}$=275 ton

W=1080.72 +30+275=1013 ton

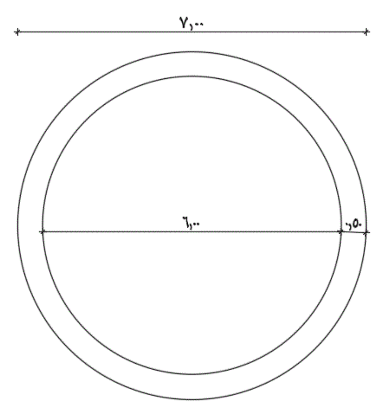


Figure 3.15 Dimension Of Shaft

Fb = 0.44*0.85*$\frac{1013}{9.81}$ =38.62 ton

O.T.M=38.62*20=772.4 m.t

3.3.10 Check of stresses between shaft and foundation

F1,2 =$\frac{-N}{A} \pm \frac{M_x}{I_x} * y$ ⩽1.25*Fc all =1.25*105=131.25kg/cm2

N=1013 ton

A =$\frac{\pi*(4^2-3^2)}{4}$ =4.27 m^2

Mx =772.4 m.t

Ix = $\frac{\pi*(4^2-3^2)}{64}$ = 6.26 m^4

Y =1.9 m

F1,2 =$\frac{-1013}{4.27} \pm \frac{772.4}{6.26} * 1.9$

F1= - 0.28 kg/cm^2 <131.25kg/cm^2

F2= -47.17 kg/cm^2 <131.25kg/cm^2

Ok,safe,No tension

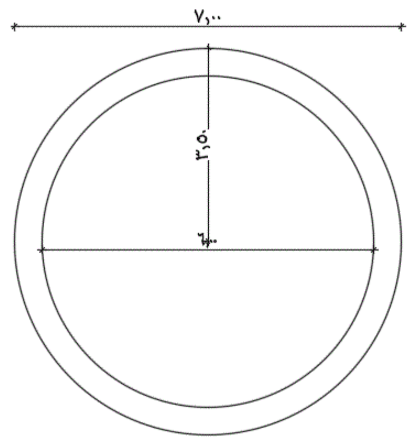


Figure 3.16 Dimension Of Shaft

3.3.12 Design of foundation

Assume tf= 1.6 m

Wt = Wtank + O.Wraft=1876.48+2.5*1.6*$\frac{\pi*(11.5)^2}{4}$ =2292 t

NO.of piles =(1.2:1.4)* $\frac{W_t}{pile\ capacity}$ =1.4*$\frac{2292}{100}$ =36 piles

F1,2 =$\frac{-N}{n} \pm \frac{M}{\sum r^2} * r_i$

N = 2292 t

M = 1859.7 m.t

n = 36 pile

$\sum r^2$=(18*4.89^2)+ (12 * 3.26^2) +(6 * 1.63^2) =573.9

For point (1)

F1,2 $=\frac{-2292}{40} \pm \frac{1859.7}{643} * 4.89$

F1= -47.82 t < 100 t

F2= -79.5 t < 100 t

For point (2)

F1,2 $=\frac{-2292}{40} \pm \frac{1859.7}{643} * 3.26$

F1= -53.1 t < 100 t

F2= -74.23 t < 100 t

For point (3)

F1,2 $=\frac{-2292}{40} \pm \frac{1859.7}{643} * 1.63$

F1= -58.38 t < 100 t

F2= -68.95 t < 100 t

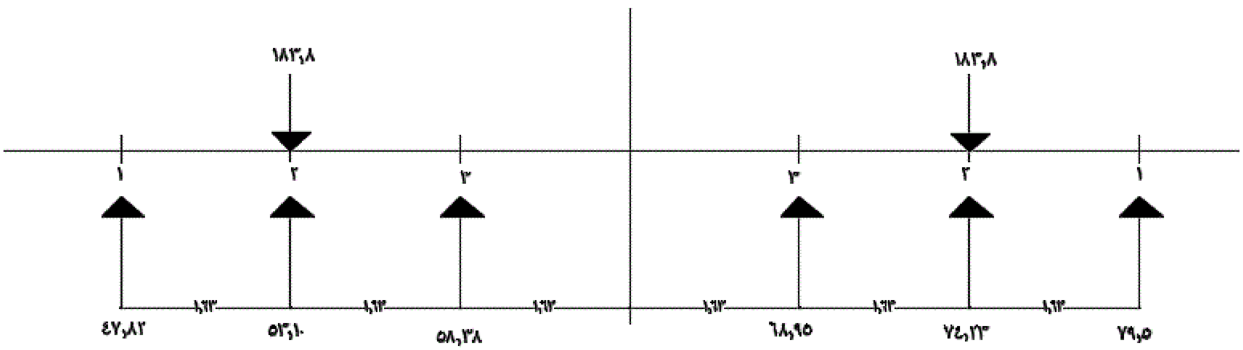


Figure 3.19 Reaction Of Piles

In X–Direction (Upper)

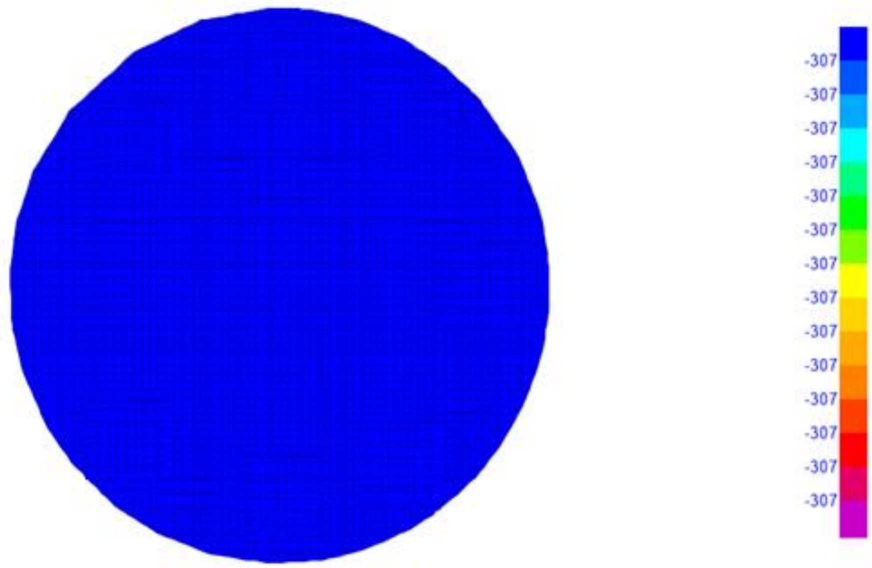


Figure 3.20 Reinforcement in X-Direction (Upper)

In X–Direction (Lower)

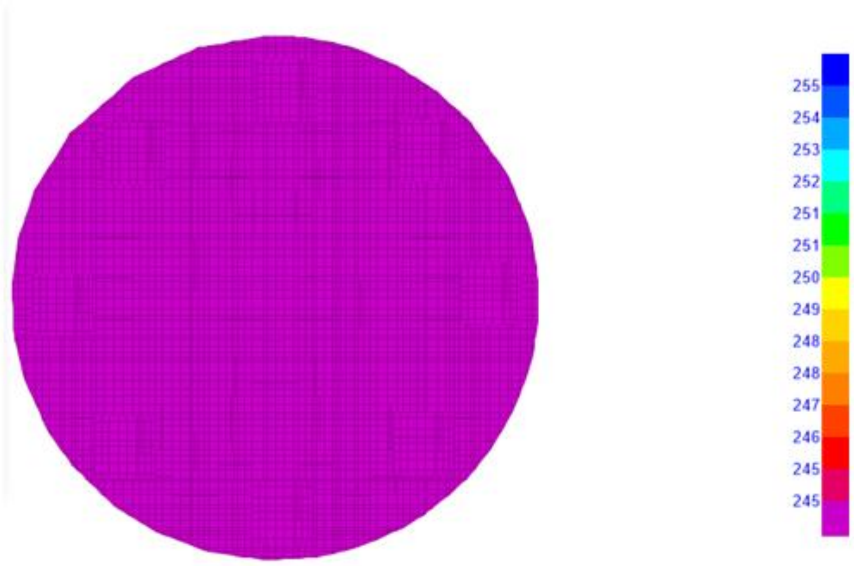


Figure 3.21 Reinforcement in X-Direction (Lower)

In Y–Direction: (Upper)

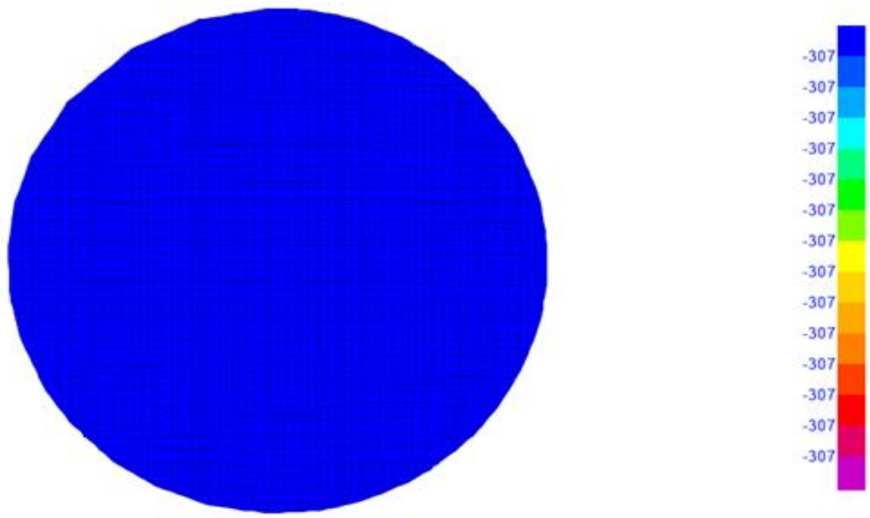


Figure 3.22 Reinforcement in Y-Direction (Upper)

In Y–Direction (Lower)

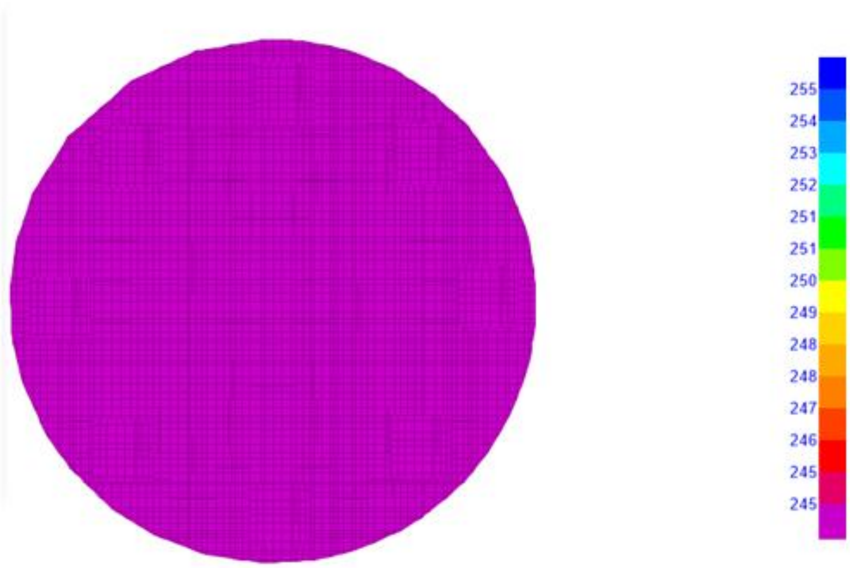


Figure 3.23 Reinforcement in Y-Direction (Lower)

From Safe Program

Mu = 224.3 Kn / m^2

$$d = c_1 \sqrt{\frac{M_u * 10^6}{F_{cu*B}}}$$

$$1530 = c_1 \sqrt{\frac{224.3 * 10^6}{25 * 1000}}$$

C1> 4.86 J = 0.826

$$A_S = \frac{M_u * 10^6}{F_{y*j\ d}}$$

$$A_S = \frac{224.3 * 10^6}{350 * 0.826 * 1530} = 507.1\ mm^2$$

$$A^{s`} = \frac{0.15}{100} * b * d = \frac{0.15}{100} * 1000 * 1530 = 2295\ mm^2$$

Use 8$Ø20/m^{`}$

Tangential (Top , Bottom)

Radial (Top , Bottom)

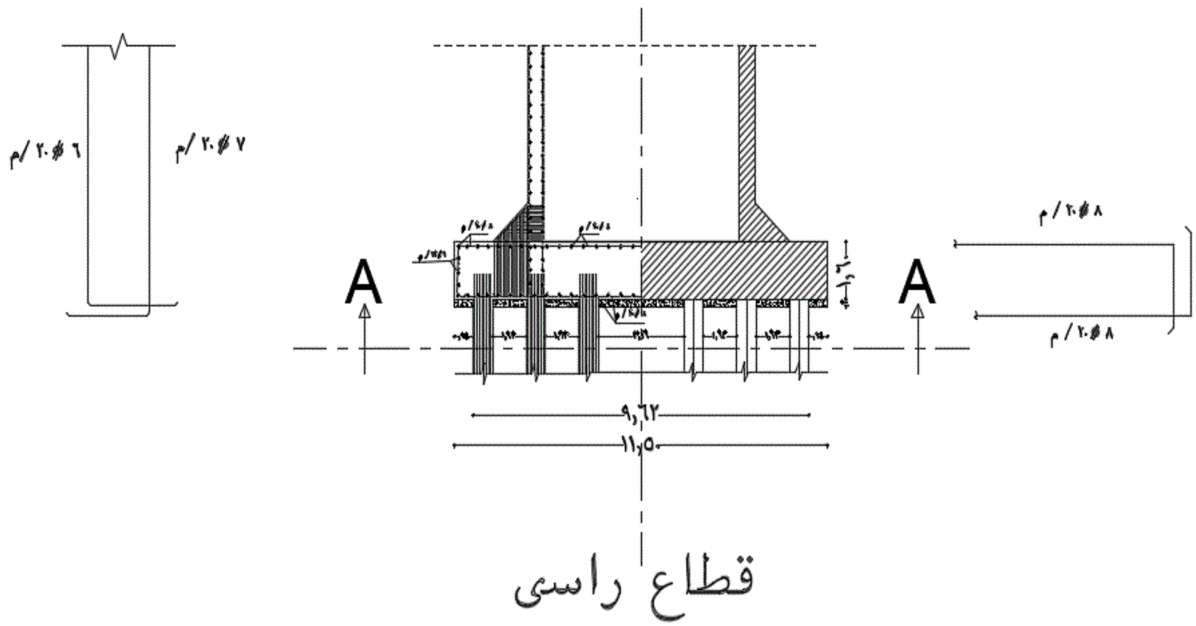


Figure 3.24 Details Of Reinforcement

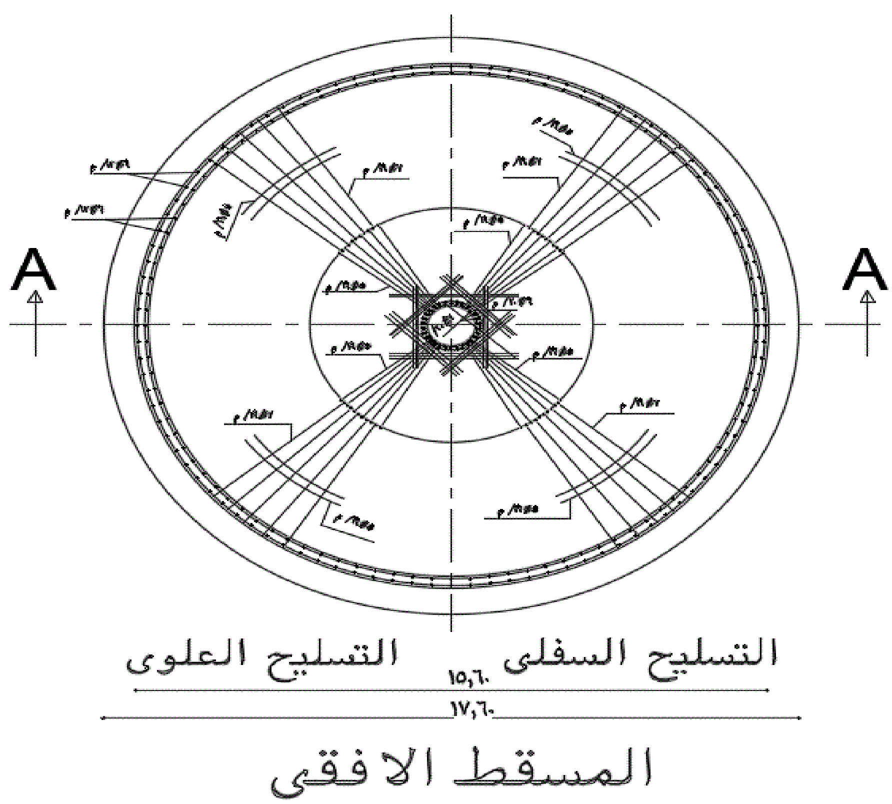


Figure 3.25 Details Of Reinforcement