NILE ACADEMY

FACULTY OF ENGINEERING

CIVIL ENGINEERING DEPARTEMENT

REINFORCED CONCRETE PROJECT

(مشروع تصميم منشآت خرسانية) (2023-2024)

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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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Project Definition

Structural design is an essential component of civil engineering that plays a pivotal role in the construction and maintenance of built environments. It involves the application of scientific principles to ensure that structures are capable of bearing anticipated loadings without failure during their intended lifespan. This discipline is crucial for the safety and functionality of various structures, including buildings, bridges, tunnels, and dams.

The importance of structural design lies in its ability to create frameworks that are not only strong and stable but also cost-effective and sustainable. By carefully selecting materials and designing efficient load paths, structural engineers minimize the risk of structural failure, which can lead to catastrophic consequences such as loss of life, economic disruption, and environmental damage.

Moreover, structural design is integral to achieving architectural vision and aesthetic goals. It enables architects to realize complex shapes and forms while ensuring that these structures are practical and safe. This collaboration between form and function is what allows for innovation in the field of construction.

In addition to these considerations, structural design also addresses environmental concerns. It seeks to reduce the carbon footprint of construction projects by optimizing material usage and promoting the use of renewable resources and energy-efficient designs. This approach not only conserves natural resources but also ensures that structures contribute positively to their surroundings.

Furthermore, in regions prone to natural disasters such as earthquakes, hurricanes, or floods, structural design becomes even more critical. Engineers must account for these additional stresses and design structures that can withstand such extreme conditions. This resilience is key to protecting communities and ensuring that infrastructure remains functional in the aftermath of a disaster.

In summary, structural design is a multifaceted discipline that underpins the safety, sustainability, and success of construction projects. Its importance cannot be overstated, as it directly impacts the well-being of individuals and the collective progress of society.

The field of structural design is a cornerstone of civil engineering, where the primary motivation lies in creating structures that are safe, efficient, and sustainable. The aim of this project is to explore advanced methodologies in structural design that can be applied to modern construction challenges. The contributions of this project include innovative design solutions that enhance structural integrity and cost-effectiveness

Economic Feasibility Study and Engineering Standards

Structural design is deeply intertwined with environmental considerations and economic benefits. An economic feasibility study is crucial to evaluate the viability of design choices, ensuring that projects are not only technically sound but also financially sustainable. Adherence to engineering standards guarantees that structures meet the required safety and performance criteria while being mindful of their environmental impact.

Study Objectives

The study objectives for concrete structural design are centered around understanding and optimizing the use of concrete, a versatile and widely used construction material. These objectives include:

1. **Material Properties**: To investigate the physical and mechanical properties of concrete, including its strength, durability, and behavior under various environmental conditions.
2. **Design Techniques**: To develop efficient design methodologies that maximize the load-bearing capacity of concrete structures while minimizing material usage and cost.
3. **Sustainability**: To explore sustainable practices in concrete production and usage, aiming to reduce the environmental impact of construction activities **Innovation**: To integrate new technologies such as high-performance concrete, fiber reinforcement, and smart sensors to enhance the functionality and resilience of concrete structures.
4. **Safety Standards**: To ensure that concrete structural designs comply with international safety standards and can withstand natural disasters like earthquakes and floods.
5. **Economic Analysis**: To perform cost-benefit analyses to determine the most economically feasible solutions for concrete structural design projects. These objectives

guide the continuous improvement and innovation within the field of concrete structural design.

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These objectives guide the continuous improvement and innovation within the field of concrete structural design.

Concrete Challenges: Appeal and Solutions

Reinforced concrete is a composite material that combines the high compressive strength of concrete with the high tensile strength of steel reinforcement. While it is one of the most popular materials used in construction due to its versatility, durability, and cost-effectiveness, it faces several challenges:

1. **Corrosion**: Steel reinforcement is susceptible to corrosion, especially in aggressive environments, leading to structural weakness.

2. **Cracking**: Concrete can develop cracks over time due to thermal expansion, shrinkage, or excessive loads.
3. **Sustainability**: The production of cement, a key ingredient in concrete, contributes significantly to carbon emissions.

The appeal of reinforced concrete lies in its adaptability to various shapes and forms, its inherent fire resistance, and its ability to be made with locally available materials.

Solutions to these challenges include:

1. **Corrosion-resistant Alloys**: Using stainless steel or other corrosion-resistant alloys for reinforcement.
2. **Advanced Admixtures**: Incorporating admixtures that improve the durability and reduce the permeability of concrete.
3. **Green Cement**: Developing alternative cementitious materials that reduce the carbon footprint of concrete production.

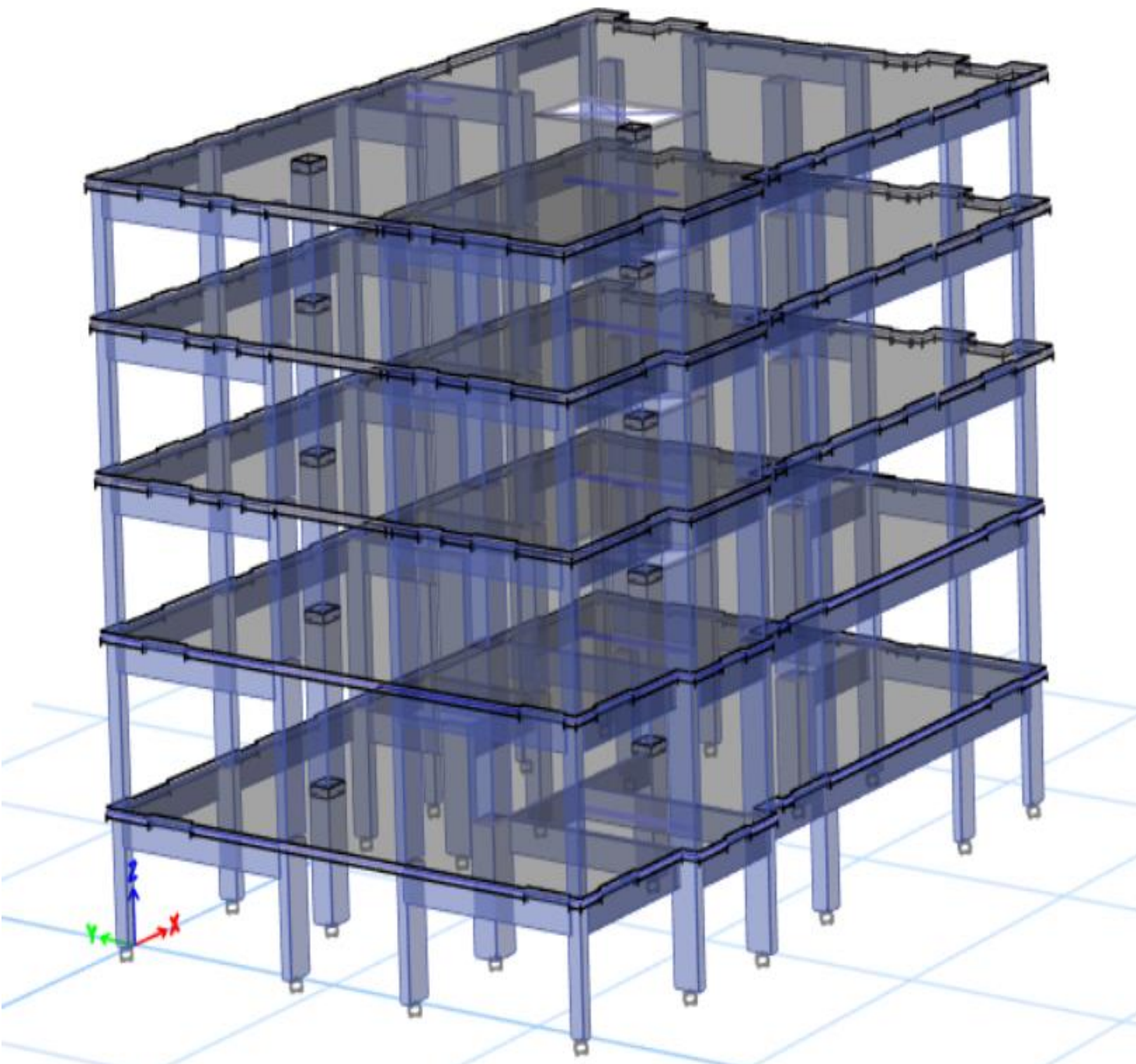
By addressing these challenges with innovative solutions, the long-term performance and sustainability of reinforced concrete can be significantly enhanced

The environmental challenges of concrete, its appeal, and how the designer can overcome them

1. **Efficient Design**: By optimizing the structural layout, engineers can reduce the amount of concrete required, leading to lower material costs.
2. **Durability**: Designing for longevity reduces the need for repairs and replacements, saving money over the structure's lifespan.
3. **Recycled Materials**: Using recycled aggregates in concrete mixtures can lower costs and reduce waste.
4. **Thermal Mass**: Concrete's thermal mass can improve energy efficiency in buildings, cutting heating and cooling expenses.
5. **Green Concrete**: Incorporating sustainable materials like fly ash or slag in concrete reduces the environmental impact and can be more economical than traditional cement.

By focusing on these aspects, concrete structural design not only becomes cost-effective but also environmentally friendly.

Uinit (1) Villa



1.1 INTRODUCTION

1.1.1 Villa Consists of:

- Basement of (2.90) m height
- Ground Floor of (3.80) m height
- First Floor (3.40) m height
- Second Floor (3.30) m height
- Third Floor of (3.00) m height

1.1.2 Material Properties Used:

- F_{cu}=350 kg/cm^2
- $F_{y(main\ steel)}$=3600 kg/cm^2
- $F_{y(stirrups)}$=2400 kg/cm^2
- Weight of used brick =1400 kg/m^3
- Bearing Capacity of Soil = 1.0 kg/m^2

1.1.3 Cover Thickness

- Slabs Cover = 2 cm
- Beams Cover = 2 cm
- Columns Cover = 2.5 cm
- Foundations Cover = 7 cm
- Stairs Cover = 2 cm
- Ramp Cover = 2 cm
- Semell Cover = 3 cm

1.1.4 Loads Used:

- L.L= According to every Floor
- Cover = 0.15 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

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)

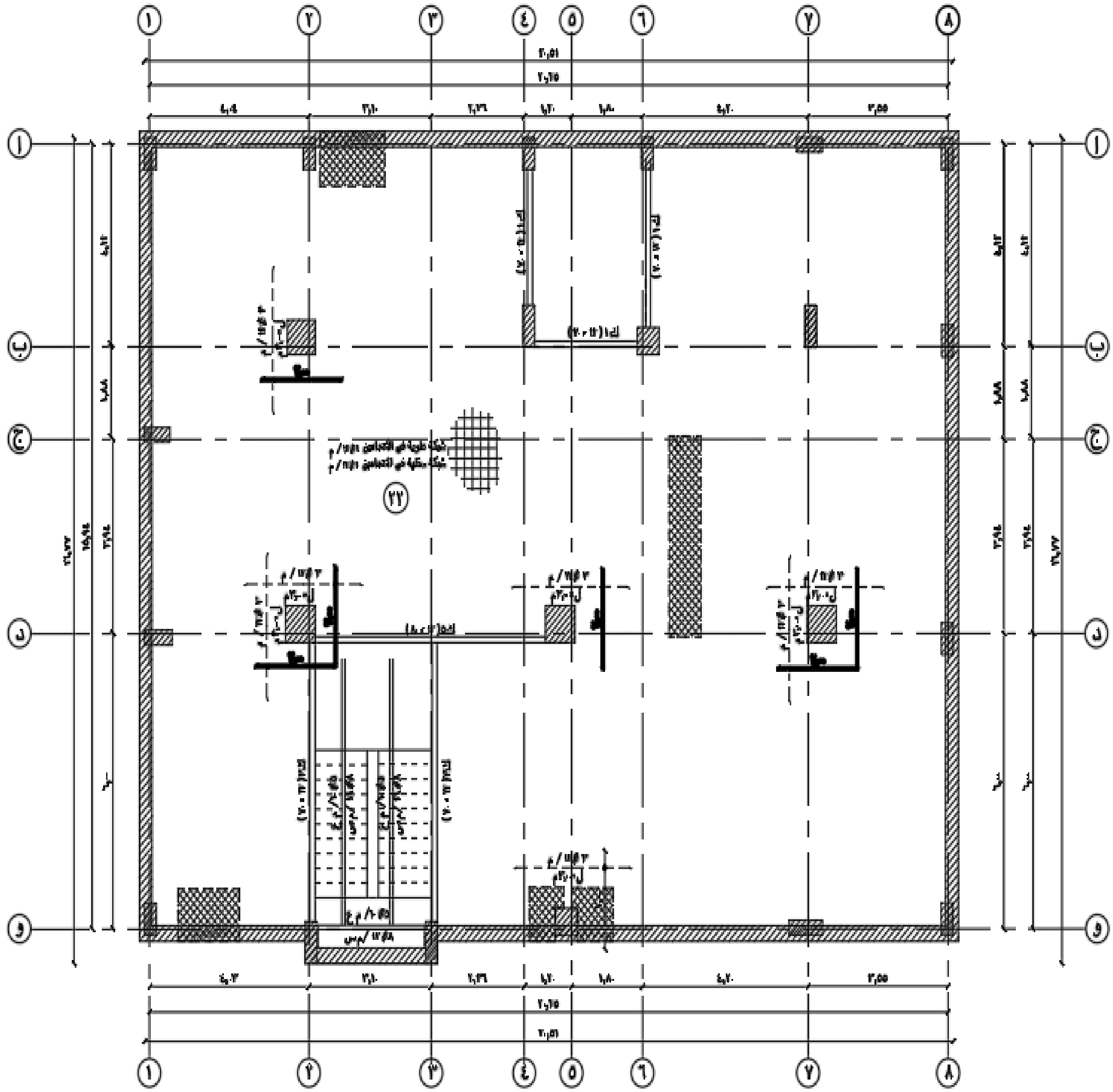


Figure 1.1 Statical System of Basement Roof

- 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 = 250 kg/m^2 = 0.25 t/m^2
- Wall load = 250 kg/m^2 = 0.25 t/m^2.

Solving This flat slab By Using CSI Safe program:

- D.L = O.W + W_{wall} + Covering material
  = 0.55 + 0.25 + 0.15 = 0.95 t/m^2
- L.L = 250 kg/cm^2 = 0.25 t/m^2
- W_u = 1.4 D.L + 1.6 L.L =1.4 (0.95) + 1.6 (0.25) = 1.73 t/m^2

For ultimate design:-

- $As = As = \left[\frac{Mu}{Fy*J*d}\right]$
-
- M_u = As * F_y *J *d = $6*\left(\frac{\pi*(1.2)^2}{4}\right)*3600*0.826*18*(10)^{-5}$
- M(r)=3.63 t.m ⇛ Use 6 ∯ 12 / m in each Direction
- Additional RFT (3 ∯ 12 /m) & 6 ∯ 12 / m) upper and lower

In X-Direction: (Upper and Lower)

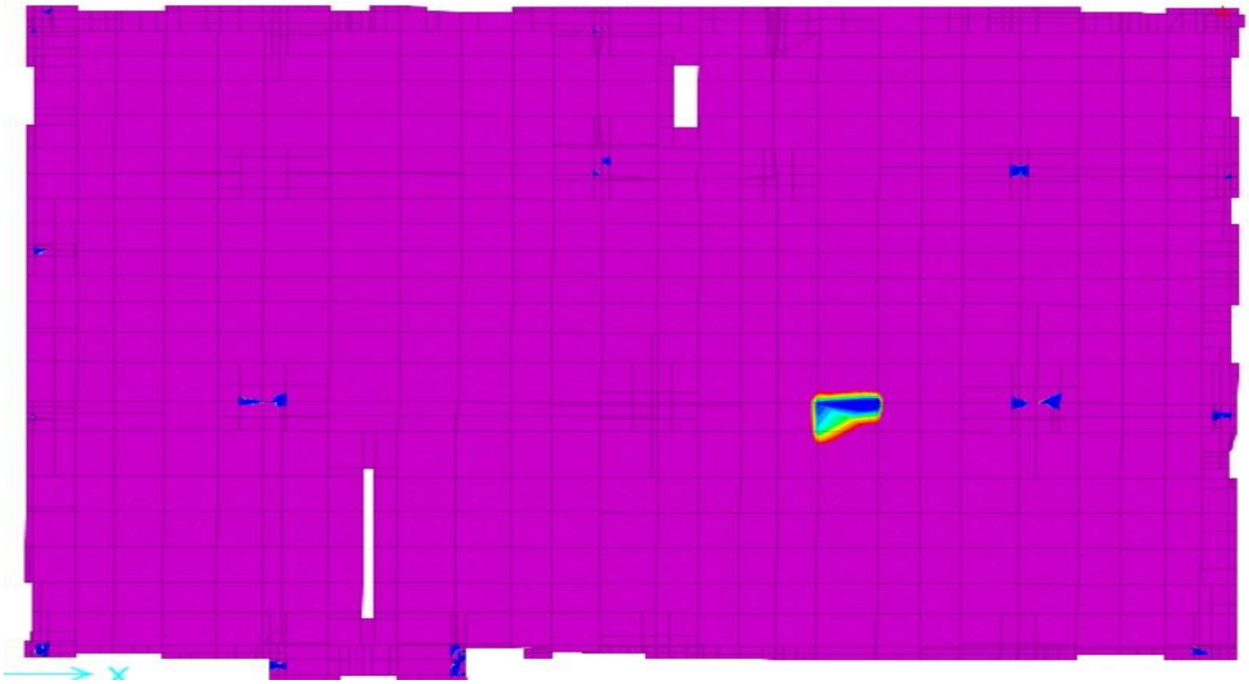


Figure 1.2 Additional Reinforcement in X-Direction (Upper and Lower)

In Y-Direction (Upper and Lower):

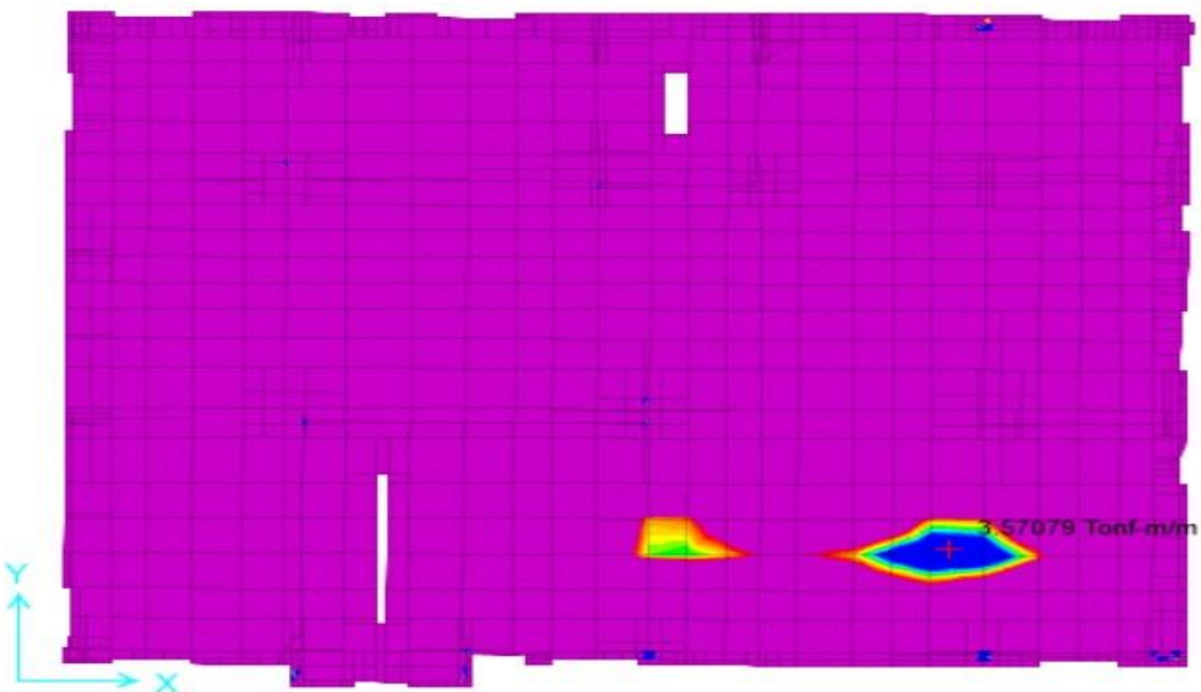


Figure 1.3 Additional Reinforcement in Y-Direction (Upper and Lower)

1.2.1.1 Check for Long Term Deflection:

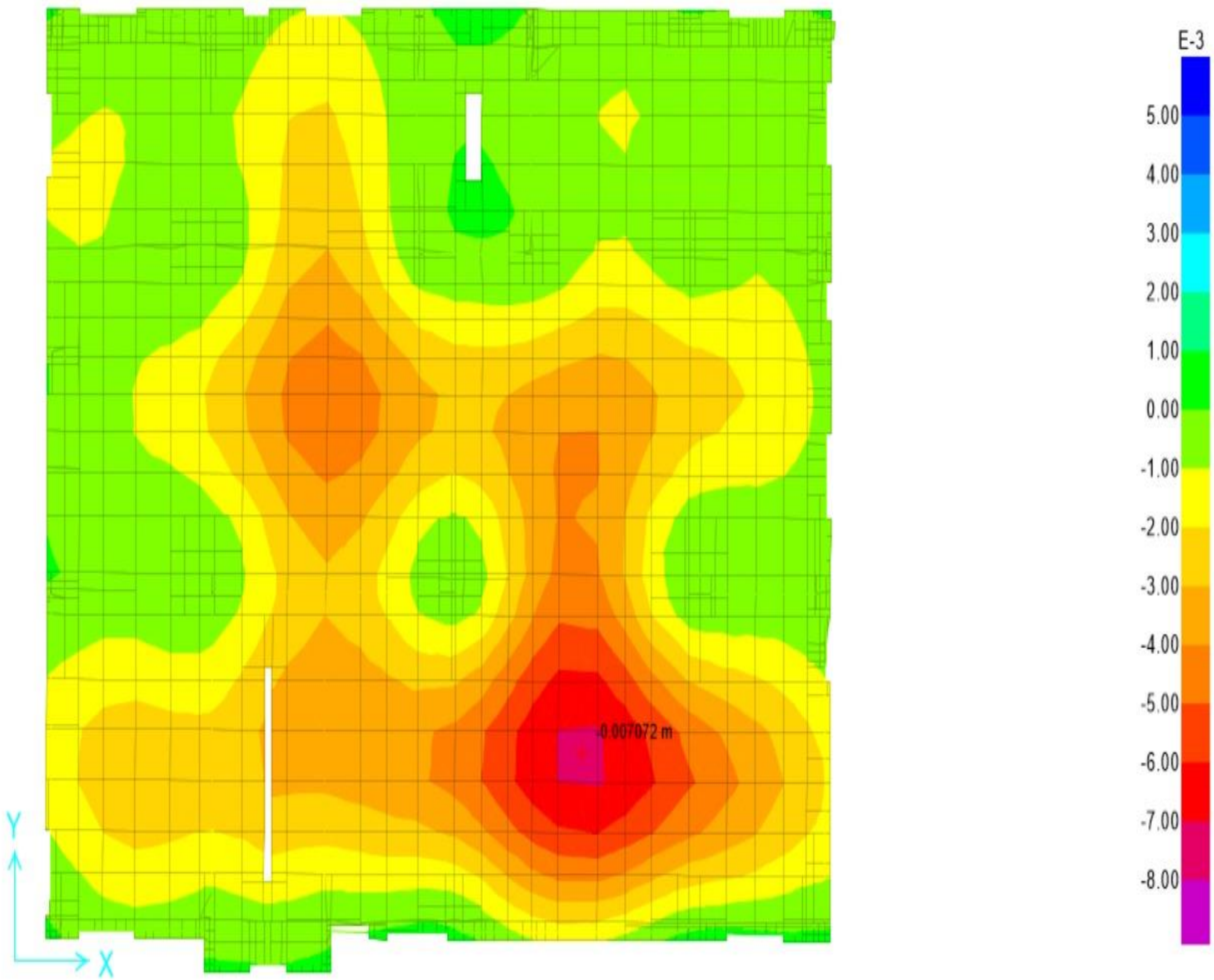


Figure 1.4 Long Term Deflection

- From Code Check = L/250
- Span for Check = 6.00 m
- Allowable Deflection = 6/250 = 0.024 m
- Maximum Deflection = 0.007072 m

1.2.2 Ground Slab (Flat Slab System)

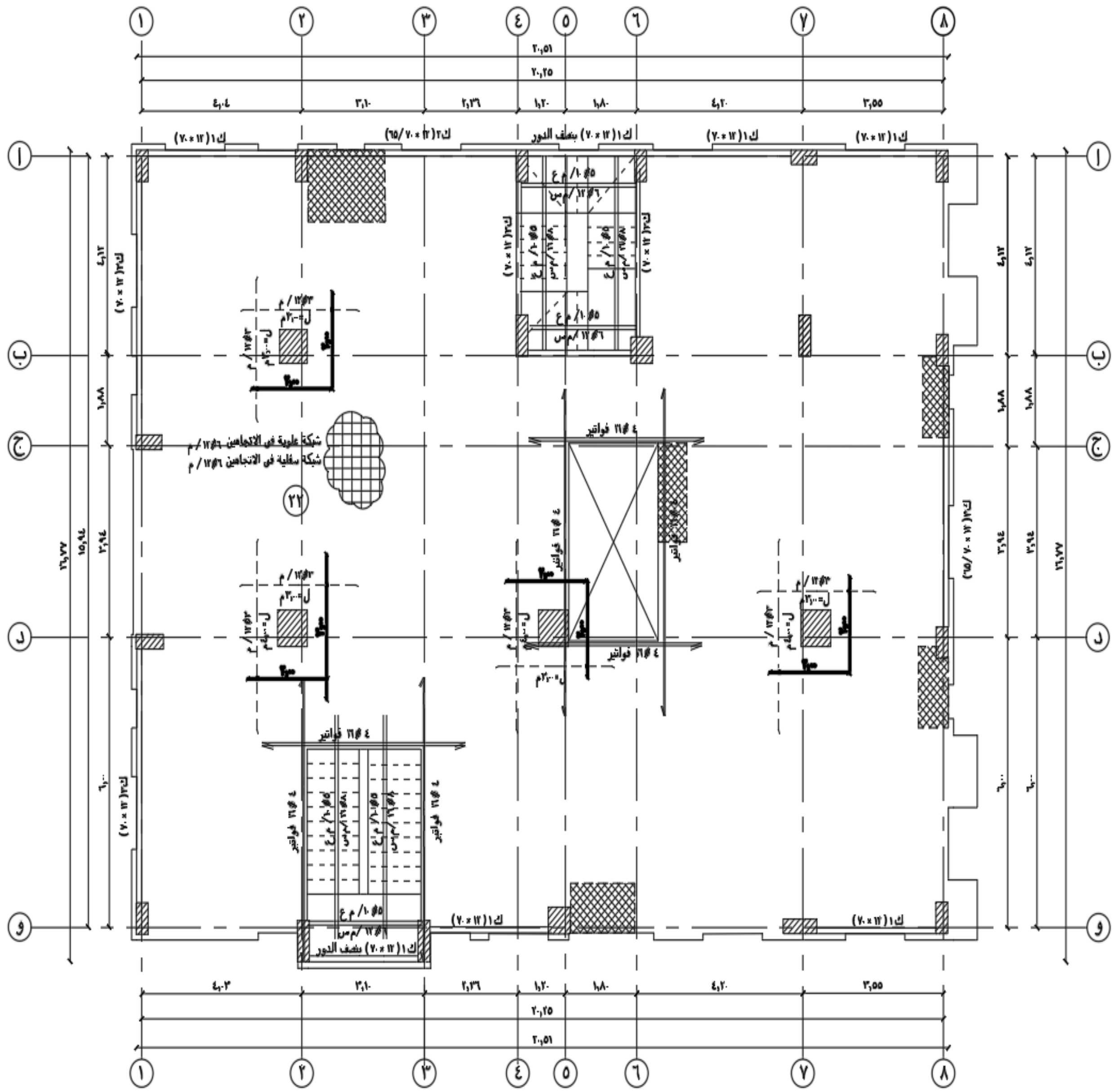


Figure 1.5 Statical System of Ground Roof

- Slab Thickness = 22 cm
- Own weight = 0.22* 2.5 =0.55 t/m^2.
- Covering = 150 kg/m^2 = 0.150 t/m^2.
- Live load = 250 kg/m^2 = 0.250 t/m^2
- Wall load =250 kg/m^2 = 0.250 t/m^2.

Solving This flat slab By Using CSI Safe program:

- D.L = O.W + W_{wall} + Covering material
  = 0.55 + 0.25 + 0.15 = 0.95 t/m^2
- L.L = 250 kg/cm^2 = 0.25 t/m^2
- W_u = 1.4 D.L + 1.6 L.L =1.4 (0.95) + 1.6 (0.25) = 1.73 t/m^2

For ultimate design:-

- $As = \left[\frac{Mu}{Fy * J * d}\right]$
- $M_u = As * F_y * J * d = 6 * \left(\frac{\pi * (1.2)^2}{4}\right) * 3600 * 0.826 * 18 * (10)^{-5}$
- M(r)= 3.63 t.m ⇛ Use 6 ∯ 12 / m in each Direction
- Additional RFT (3 ∯ 12 / m) & (6 ∯ 12 / m) upper and lower

In X-Direction: (Upper and Lower)

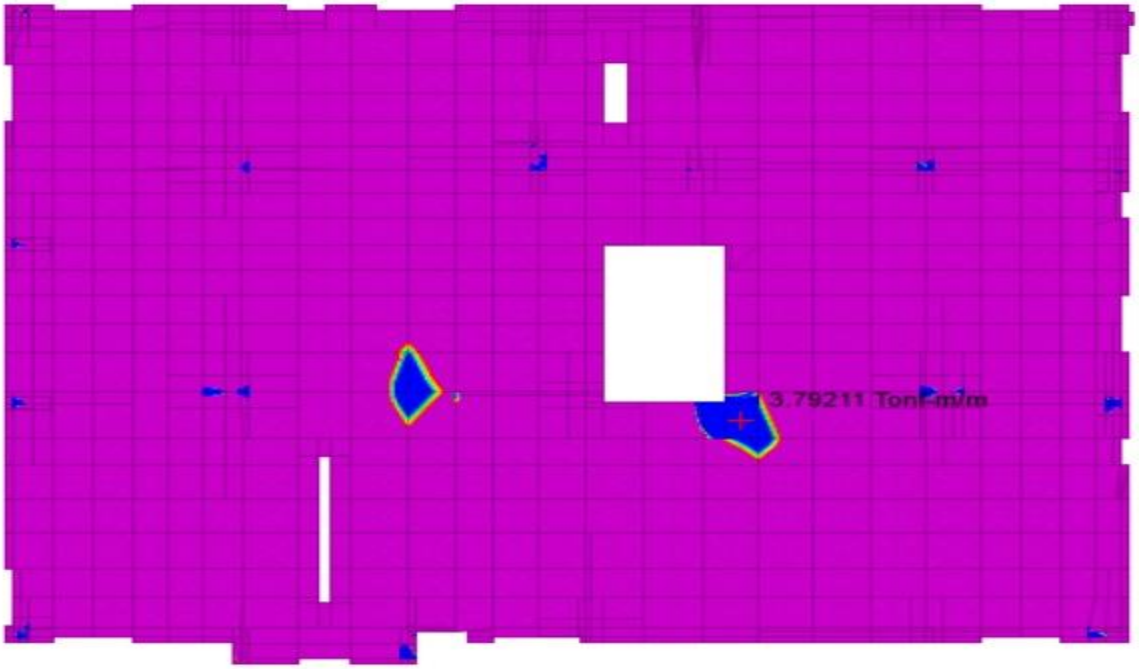


Figure 1.6 Additional Reinforcement in X-Direction (Upper and Lower)

In Y-Direction: (Upper and Lower)

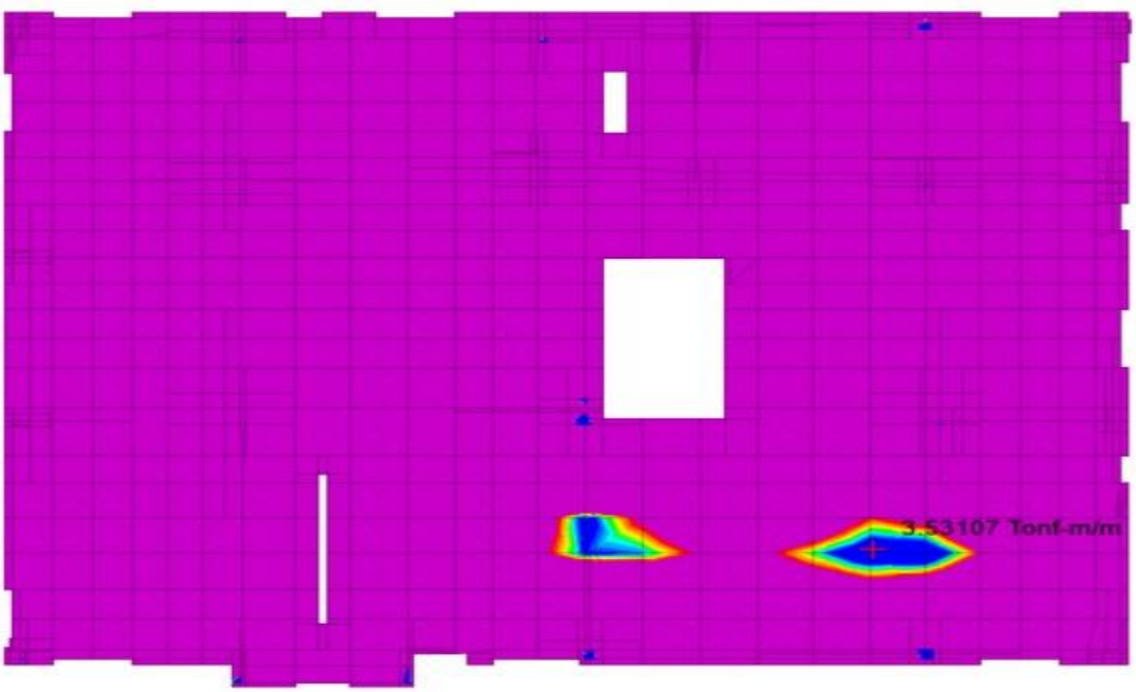


Figure 1.7 Additional Reinforcement in Y-Direction (Upper and Lower)

1.2.2.1 Check for Long Term Deflection:

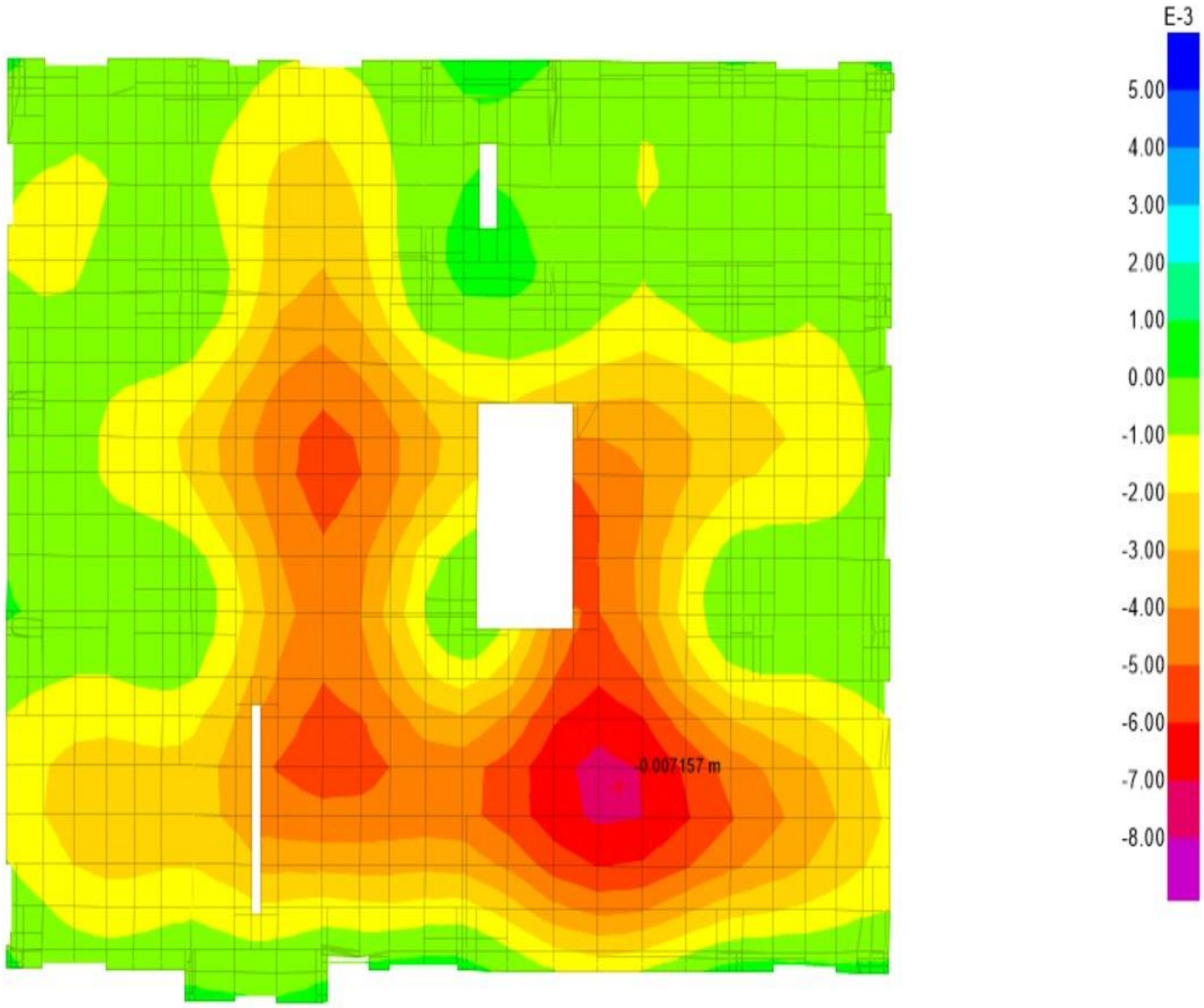


Figure 1.8 Long Term Deflection

- From Code Check = L/250
- Span for Check = 6.00 m
- Allowable Deflection = 6/250 = 0.024 m
- Maximum Deflection = 0.007157 m

1.2.3 First Slab (Flat Slab System)

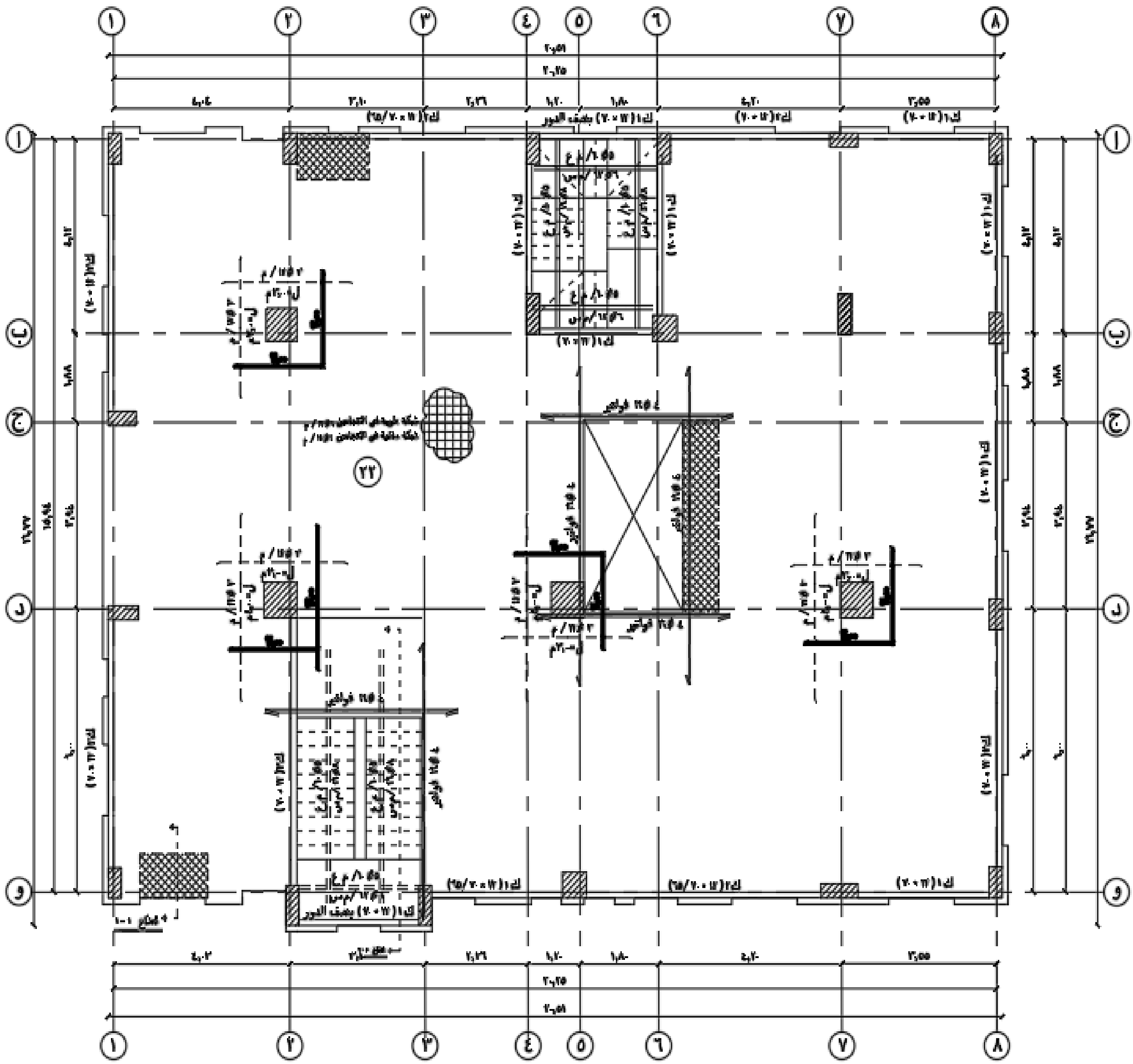


Figure 1.9 Statical System of First Roof

- 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 = 250 kg/m^2 = 0.25 t/m^2.

Solving This flat slab By Using CSI Safe program:

- D.L = O.W + W_{wall} + Covering material
  = 0.55 + 0.25 + 0.15 = 0.95 t/m^2
- L.L = 250 kg/cm^2 = 0.25 t/m^2
- W_u = 1.4 D.L + 1.6 L.L =1.4 (0.95) + 1.6 (0.25) = 1.73 t/m^2

For ultimate design:-

- $As = \left[\frac{Mu}{Fy*J*d}\right]$
- M_u = As * F_y *J *d = $6*\left(\frac{\pi*(1.2)^2}{4}\right)*3600*0.826*18*(10)^{-5}$
- M(r)= 3.63 t.m ⇛ Use 6 ∯ 12 / m in each Direction
- Additional RFT (3 ∯ 12 /m) & (6 ∯ 12 / m) upper and lower

In X-Direction: (upper and Lower)

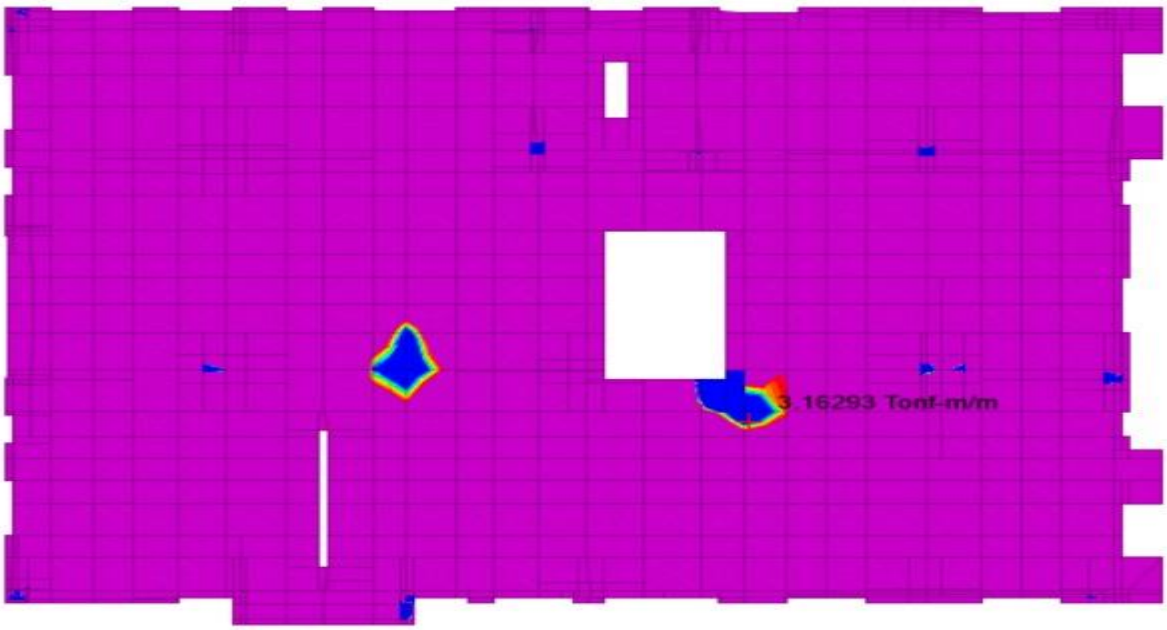


Figure 1.10 Additional Reinforcement in X-Direction (upper and Lower)

In Y-Direction: (upper and Lower)

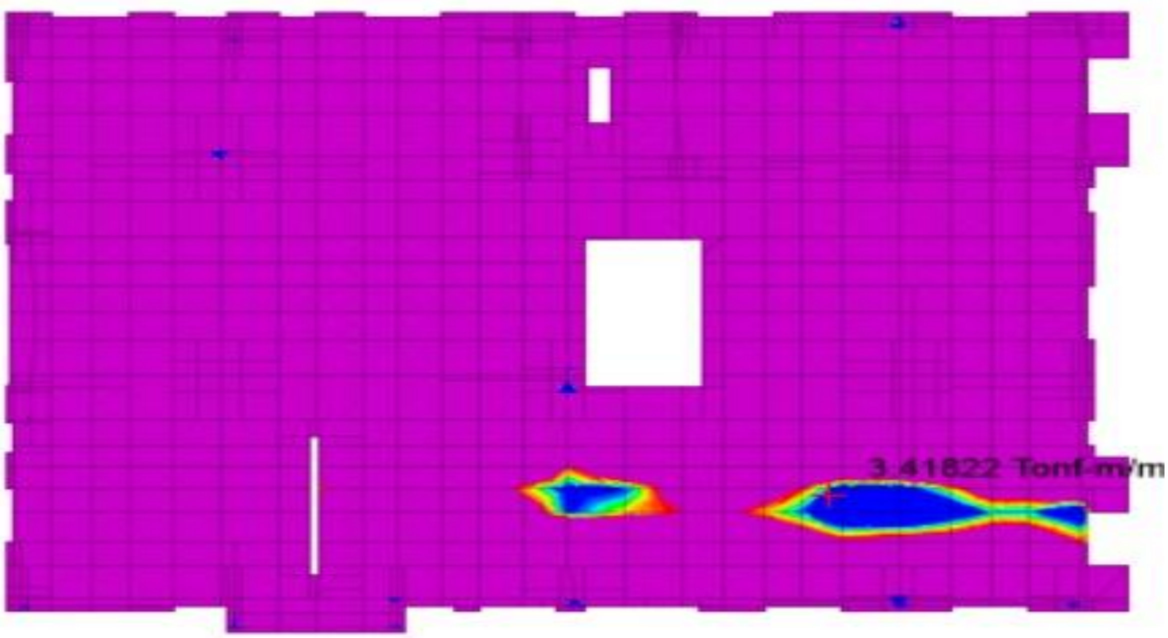


Figure 1.11 Additional Reinforcement in Y-Direction (upper and Lower)

1.2.3.1 Check for Long Term Deflection:

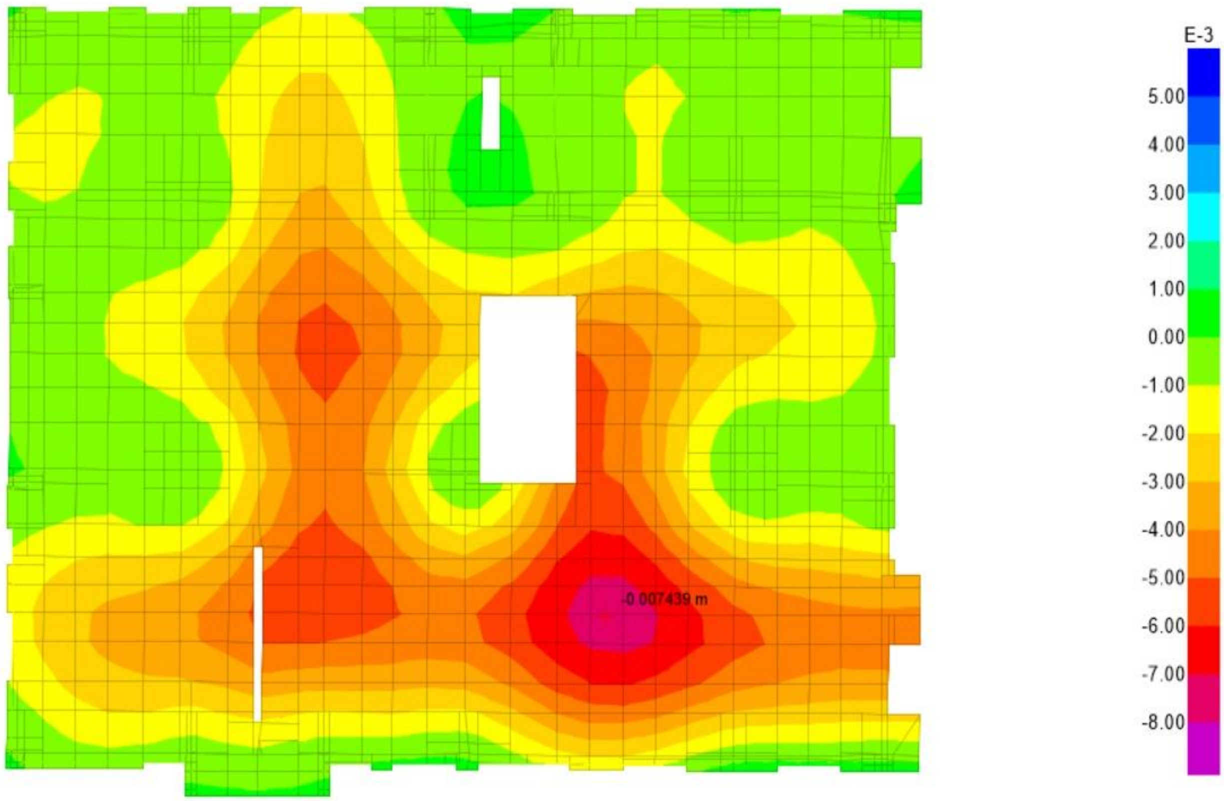
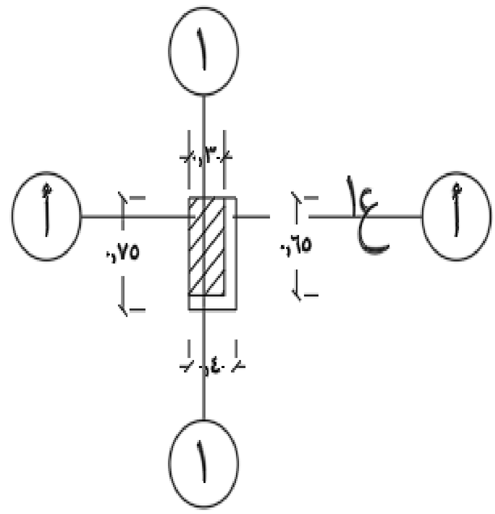


Figure 1.12 Long Term Deflection

- From Code Check = L/250
- Span for Check = 6.00 m
- Allowable Deflection = 6 / 250 = 0.024 m
- Maximum Deflection = 0.007439 m

1.2.4 Check of Punching Shear: (Basement Roof)

1.2.4.1 Corner Column (C_1=30*65) on (1 –أ) Axis



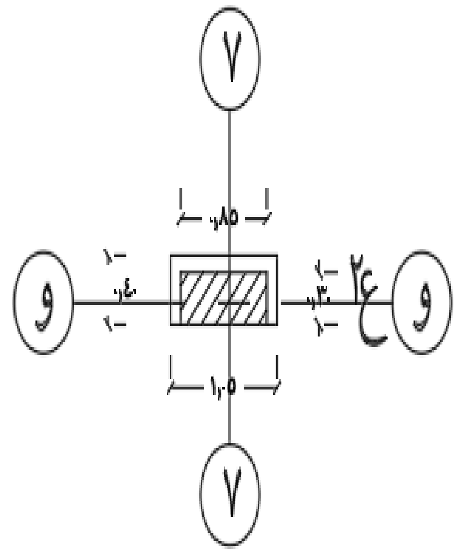
- Slab Thickness = 22 cm
- Own weight = 0.22* 2.5 = 0.55 t/m^2.
- F.Covering = 150 kg/m^2 = 0.15 t/m^2.
- Live load = 250 kg/m^2 = 0.25 t/m^2
- Wall load = 250 kg/m^2 = 0.25 t/m^2.

- D.L = O.W +W_{wall} + Covering material
- = 0.55 + 0.25 + 0.15 = 0.95 t/m^2
- L.L = 250 kg/cm^2 = 0.25 t/m^2

- $W_u = 1.4\,D.L + 1.6\,L.L = 1.4(0.95) + 1.6(0.25) = 1.73\ t/m2 = 17.3Kn/m2$
- $d = t_s$ – 20 mm = 220 – 20 = 200 mm = 0.200m
- b_o =400+750=1150 mm
- Q_{up}=Wu (L1 * L2 – Ap)= 17.3 (2.06 *2.02 – 0.40 * 0.75)= 66.79 KN/m^2
- $q_{up} = \frac{Q_{up}}{bo*d} * \beta = \frac{66.79 * 1000}{1150*200} * 1.5 = 0.33\ N/mm^2$
- q_{cup} = *the least of*:-
  - 1.7 N/mm^2
  - $0.316\sqrt{\frac{f_{cu}}{\gamma_c}} = 0.316\sqrt{\frac{35}{1.5}} = 1.53\ N/mm^2$
  - $0.316(\frac{a}{b} + 0.5)\sqrt{\frac{f_{cu}}{\gamma_c}} = 0.316\,(\frac{300}{650} + 0.5)\sqrt{\frac{35}{1.5}} = 1.46\ N/mm^2$
  - $0.8\,(\frac{\alpha*d}{b0} + 0.5)\sqrt{\frac{f_{cu}}{\gamma_c}} = 0.8(\frac{2*200}{1150} + 0.2)\sqrt{\frac{35}{1.5}} = 2.11\ N/mm^2$
- $q_{up} = 0.33\ N/mm^2 \le q_{cup} = 1.46\ N/mm^2$

OK , safe punching

1.2.4.2 Edge Column (C2 = 30*85) on (7-و) Axis:



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

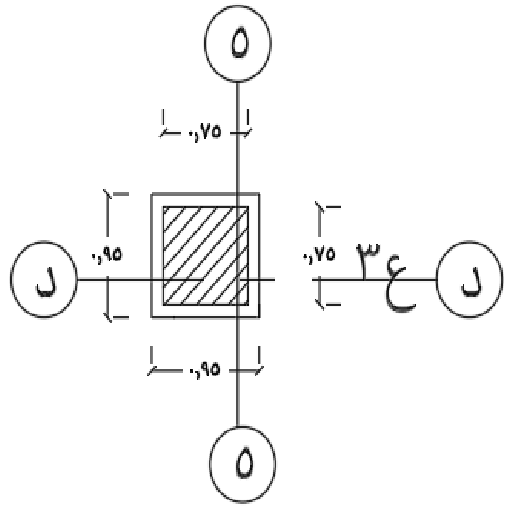
- D.L = O.W +W_{wall} + Covering material
  = 0.55 + 0.25 + 0.15 = 0.95 t/m^2
- L.L = 250 kg/cm^2 = 0.25 t/m^2

- $W_u = 1.4\,D.L + 1.6\,L.L = 1.4(0.95) + 1.6(0.25) =$ $1.73\text{t/m2} = 17.3Kn/m2$
- $d = t_s$ – 20 mm = 220 – 20 = 200 mm = 0.200m
- b_o = 1005 + 400 + 400 =1805 mm
- Q_{up}=Wu (L1 * L2 – Ap) = 17.3 (3.9 * 3 – 1.05 * 0.400) = 195.144 KN/m^2
- $q_{up} = \frac{Q_{up}}{bo*d} * \beta = \frac{195.144*1000}{1805*200} * 1.3 = 0.70\ \text{N}/mm^2$
- q_{cup} =*the least of*:-
  - 1.70 N/mm^2
  - $0.316\sqrt{\frac{f_{cu}}{\gamma_c}} = 0.316\sqrt{\frac{35}{1.5}} = 1.53\ \text{N}/mm^2$
  - $0.316(\frac{a}{b} + 0.5)\sqrt{\frac{f_{cu}}{\gamma_c}} = 0.316\,(\frac{300}{850} + 0.5\,)\sqrt{\frac{25}{1.5}} = 1.30\ \text{N}/mm^2$
  - $0.8(\frac{\alpha*d}{b0} + 0.5)\sqrt{\frac{f_{cu}}{\gamma_c}} = 0.8(\frac{3*200}{1805} + 0.2\,)\sqrt{\frac{35}{1.5}} = 2.06\ \text{N}/mm^2$
- $q_{up} = 0.70\ \text{N}/mm^2 \le q_{cup} = 1.30\ \text{N}/mm^2$

OK , safe punching

1.2.4.3 Interior Column (C3 = 75*75) on (5 – د)Axis:

- 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 = 250 kg/m^2 = 0.25 t/m^2.



- D.L = O.W +W_{wall} + F.Covering material

  = 0.55 + 0.25 + 0.15 = 0.95 t/m^2

- L.L = 250 kg/cm^2 = 0.25 t/m^2

- W_u = 1.4 $D.L + 1.6\ L.L = 1.4(0.95) + 1.6(0.25) =$ $1.73 t/m2 = 17.3 Kn/m2$
- $dp = t_s$ – 20mm = 220 – 20 = 200 mm = 0.2m
- b_o = 2(a + b) =2* (750 + 750) = 3000 mm
- Q_{up}=Wu (L1 * L2 - Ap) =17.3 (6.3 * 5.91 – 0.75*0.75) = 634.4 Kn/m^2
- $q_{up} = \frac{Q_{up}}{bo*d} * \beta = \frac{634.4*1000}{3000*200} * 1.15 = 1.21\ N/mm^2$
- q_{cup} $= the\ least\ of$:-
  - 1.70 N/mm^2
  - $0.316\sqrt{\frac{f_{cu}}{\gamma_c}} == 0.316\sqrt{\frac{35}{1.5}} = 1.53\ N/mm^2$
  - $0.316(\frac{a}{b} + 0.5)\sqrt{\frac{f_{cu}}{\gamma_c}} = 0.316\ (\frac{75}{75} + 0.5)\sqrt{\frac{35}{1.5}} = 2.29\ N/mm^2$
  - $0.8(\frac{\alpha*d}{b0} + 0.5)\sqrt{\frac{f_{cu}}{\gamma_c}} = 0.8(\frac{4*180}{3000} + 0.2)\sqrt{\frac{35}{1.5}} = 1.80\ N/mm^2$
- $q_{up} = 1.21\ N/mm^2 \le q_{cup} = 1.53\ N/mm^2$

OK safe punching

1.3 Design of Stairs (TWO Flight Stair Axis د - و ١)

1.3.1 Manual solution

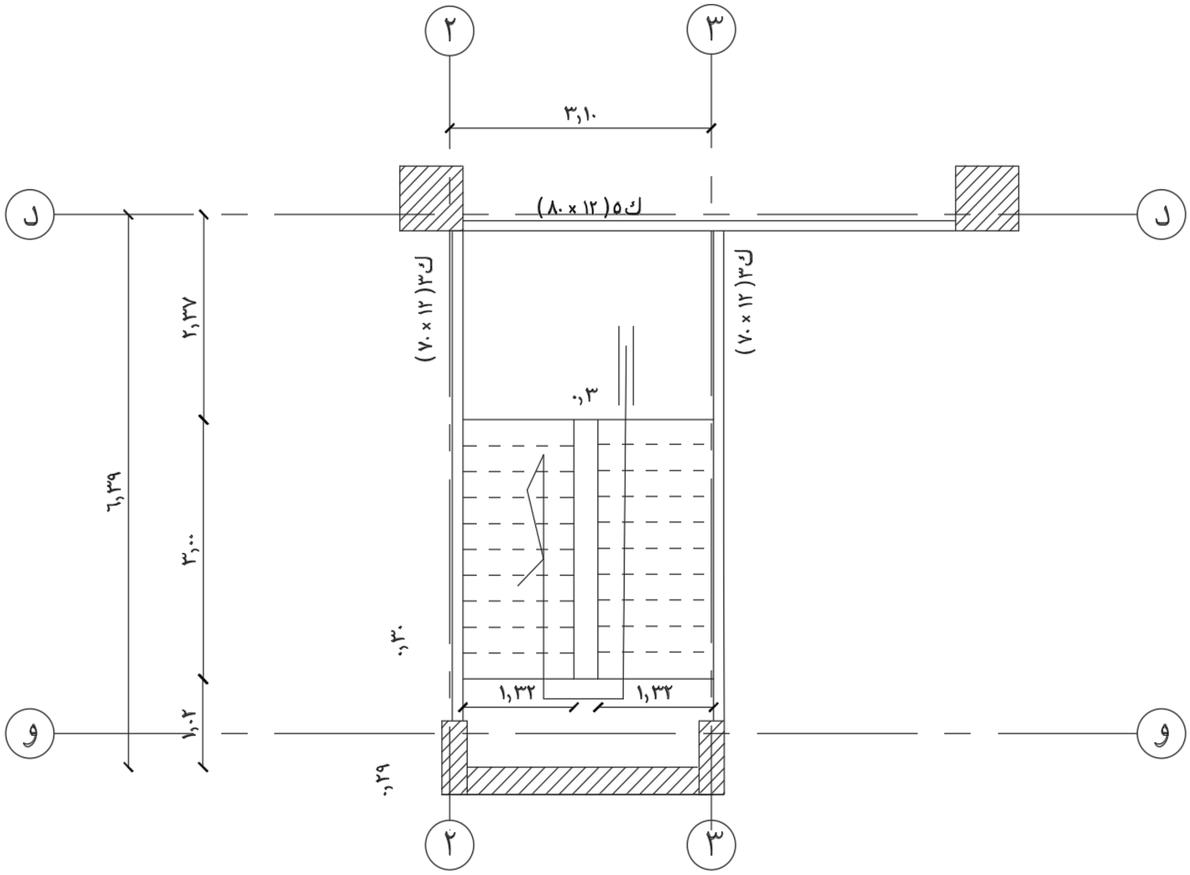


Figure 1.13 Stair Cross Section

1) Dimensions:

- $ts = \frac{Span}{24:30} = \frac{6.39\ m}{24:30} = 0.22\ m$
- $H_{Story=}\ 2.90m$
- $Rise = 0.15\ m$
- $Going = 0.30\ m$
- $\theta = \tan^{-1}(\frac{0.15}{0.30}) = 26.56\ °$
- $t^* = \frac{ts}{Cos\theta} = \frac{22}{Cos(26.56)} = 24.59\ cm$
- $t_{av} = t^* + \frac{Rise}{2} = 24.59 + 7.5 = 32$ cm

2) Loads:

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

  = 1.4 (25*0.32 + 1.0) + 1.6 (2.5)

  = 16.6 KN/m^2

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

  = 1.4 (25*0.22 + 1.5) + 1.6 (2.5)

  = 13.8 KN/m^2

3) For Strips
show in figure

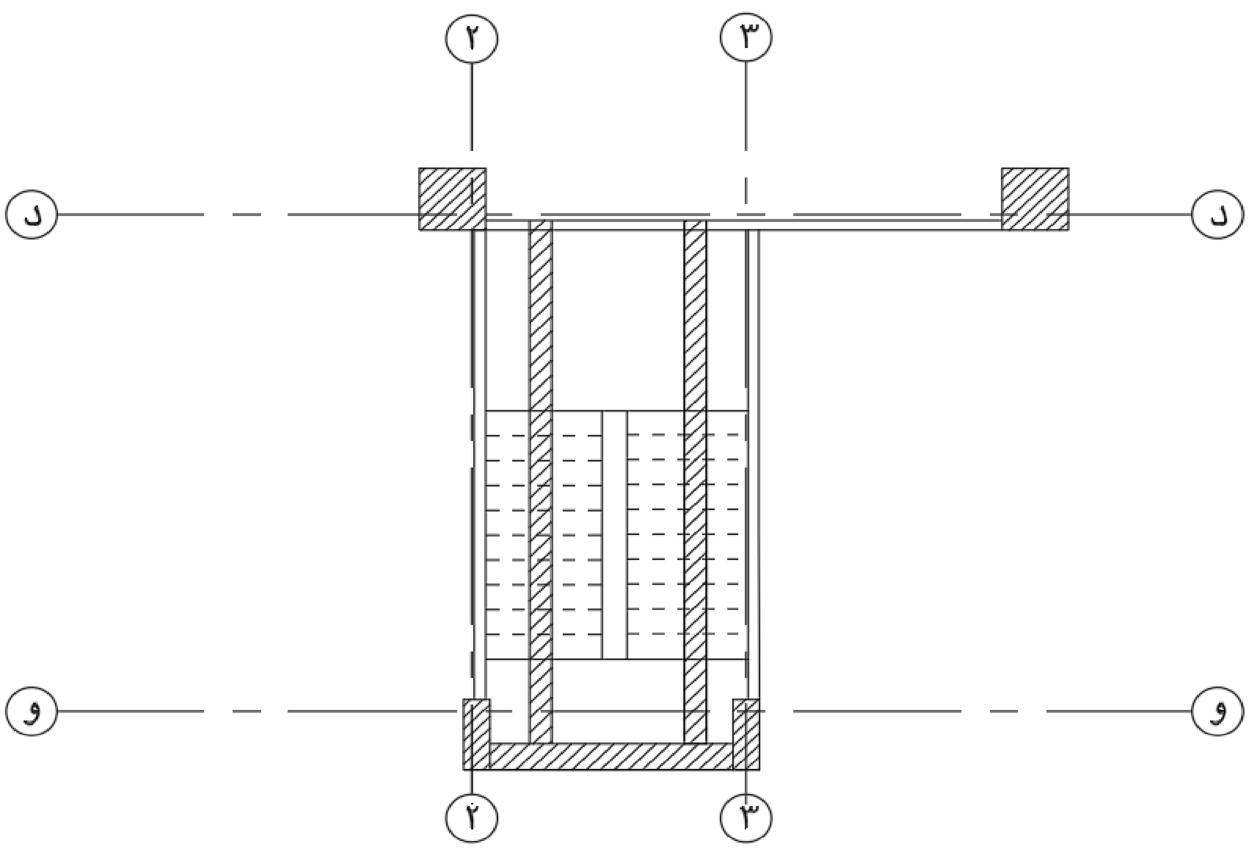


Figure 1.14 Strips Of Stair

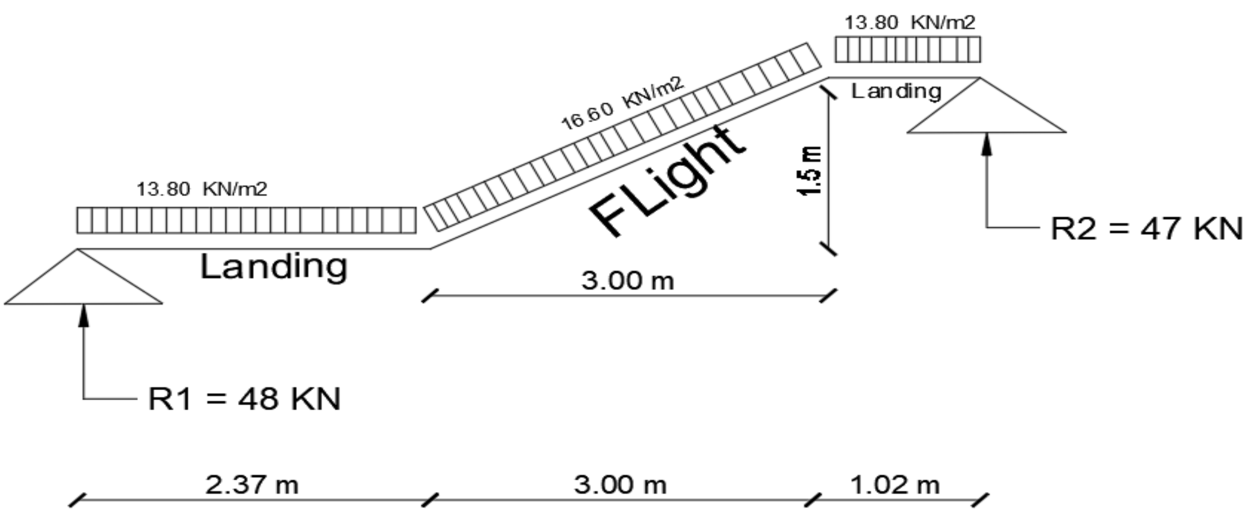
For Strip :-

R1 (6.39) = (13.8 * 2.37) ($\frac{2.37}{2}$ + 3 + 1.02) + (16.6 * 3) ($\frac{3}{2}$+ 1.02) + (13.8 * 1.02) ($\frac{1.02}{2}$)

= 302 .909

∴ R1 = $\frac{302.909}{6.39}$ = 48 KN

∴ R2 = (13.8 * 2.37) + (16.6 * 3) + (13.8 * 1.02) – 48 = 47 KN



Moment :-

. M1

R1(2.37) – 13.8 * 2.37 * $\frac{2.37}{2}$ = 48 (2.37) – 13.8 * 2.37 * $\frac{2.37}{2}$ = 75 KN.m

∴ M1 = 75 KN.m

. M2

R2 (4.02) – 13.8 * 1.02 ($\frac{1.02}{2}$ + 3) – 16.6 * 3 * $\frac{3}{2}$

= 47 (4.02) – 13.8 * 1.02 ($\frac{1.02}{2}$ + 3) – 16.6 * 3 * $\frac{3}{2}$ = 64.8 KN .m

∴ M2 = 64.8 KN.m

. M3

R1(5.37) – 13.8 * 2.37 ($\frac{2.37}{2}$ + 3) – 16.6 * 3 *$\frac{3}{2}$
= 48 (5.37) – 13.8 * 2.37 ($\frac{2.37}{2}$ + 3) – 16.6 * 3 *$\frac{3}{2}$ = 46.18 KN.m
∴ M3 = 46.18 KN.m

. M4
R2 (1.02) – 13.8 * 1.02 * $\frac{1.02}{2}$ = 47 (1.02) – 13.8 * 1.02 * $\frac{1.02}{2}$ = 40.76 KN.m

∴ M4 = 40.76 KN.m

∴ Moment of flight = 55.5 KN.m

Design of section

1- M = 55.5 KN.m

$R_1 = \frac{M_u}{F_{cu}\, b\, d^2} = \frac{55.5 * 10^6}{35*1000*200^2} = 0.04 < R_{x\,max} = 0.129$

ɯ = 0.05

∴ $A_s = ɯ \frac{Fcu}{Fy} bd = 0.05 * \frac{35}{360} * 1000 * 200 = 972$ mm^2 = 9.72 cm^2
Use A_S ⇛ 6 ∯ 16 / m ⇛ A_{sact} =1205.76 mm^2

2- M1 = 75 KN.m

$R_1 = \frac{M_u}{F_{cu}\, b\, d^2} = \frac{75 * 10^6}{35*1000*200^2} = 0.05 < R_{x\,max} = 0.129$

ɯ = 0.06

∴ $A_s = ɯ \frac{Fcu}{Fy} bd = 0.06 * \frac{35}{360} * 1000 * 200 = 1167$ mm^2 = 11.67 cm^2

Use A_S ⇛ 6 ∯ 16 / m ⇛ A_{sact} =1205.76 mm^2

Draw of moment and section and details of RFT

Moment

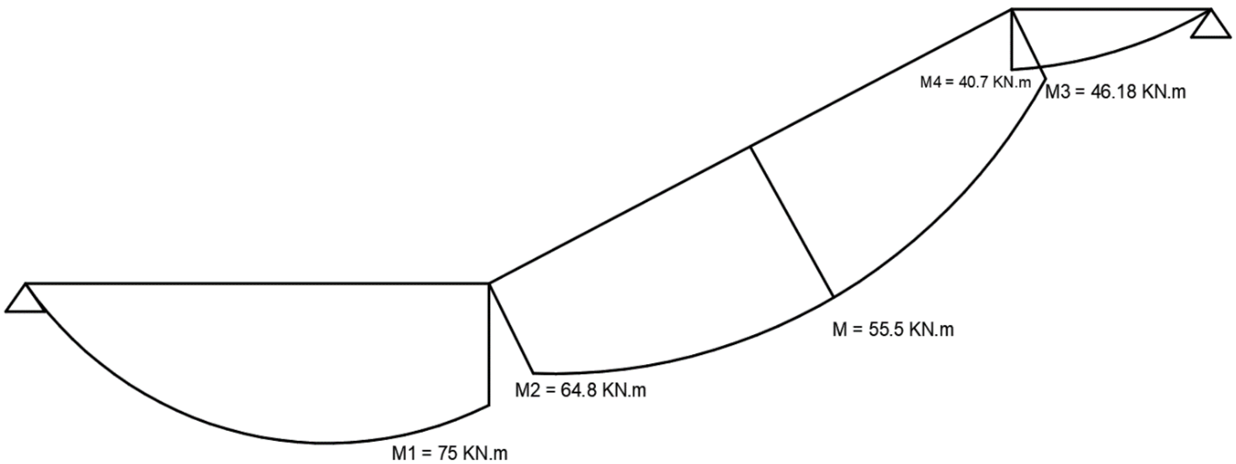


Figure 1.15 BMD

Section and details of RFT

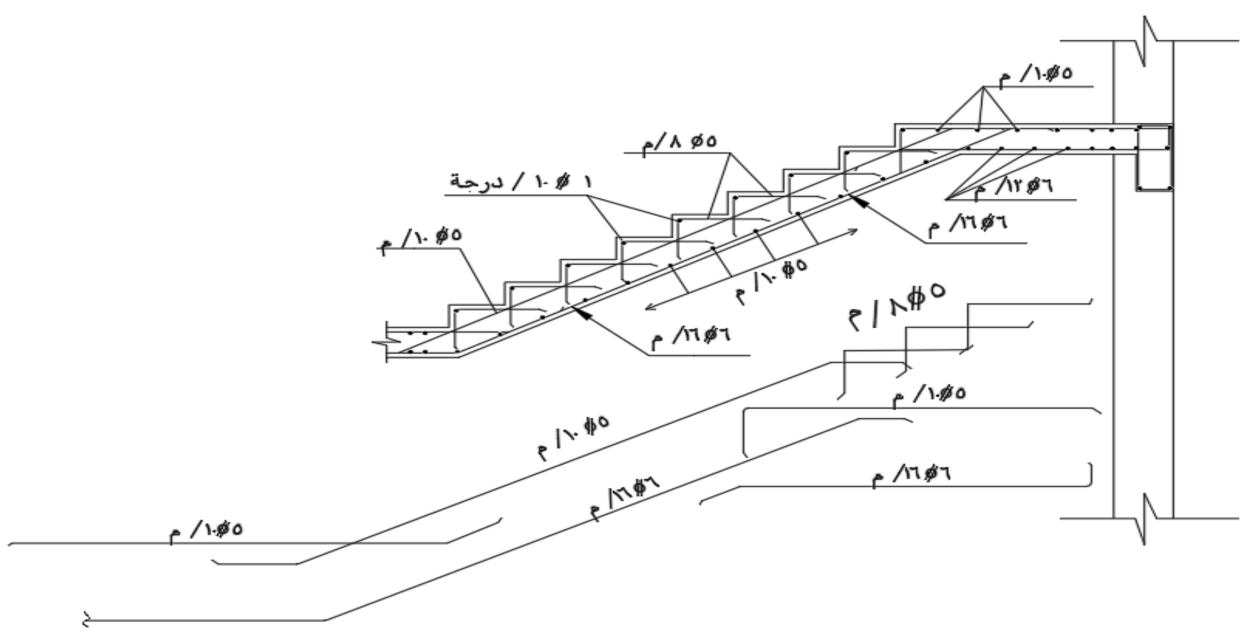


Figure 1.16 Section and details of RFT

1.3.2 Using Sap Program and Excel

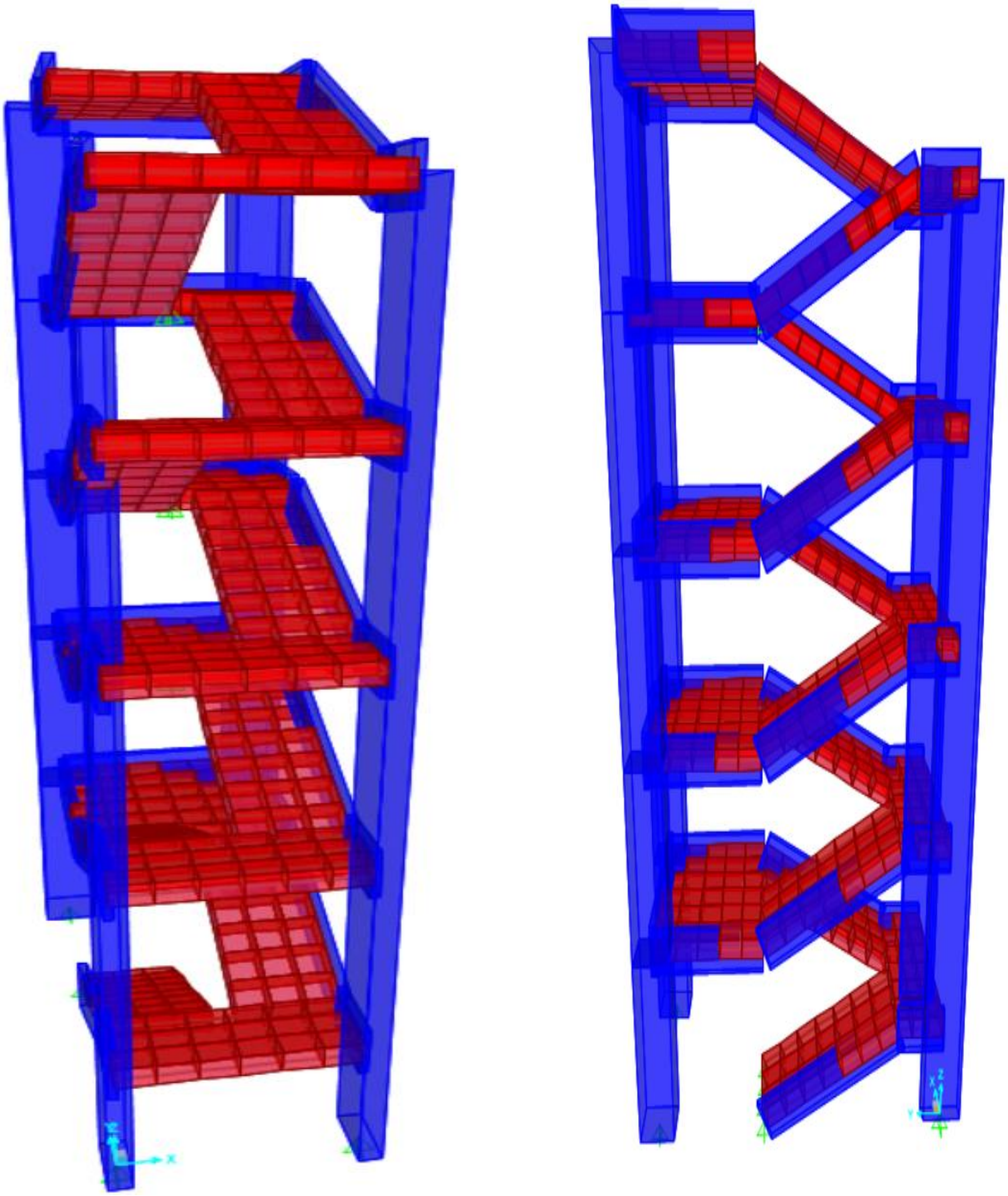


Figure 1.17 Stair 3D

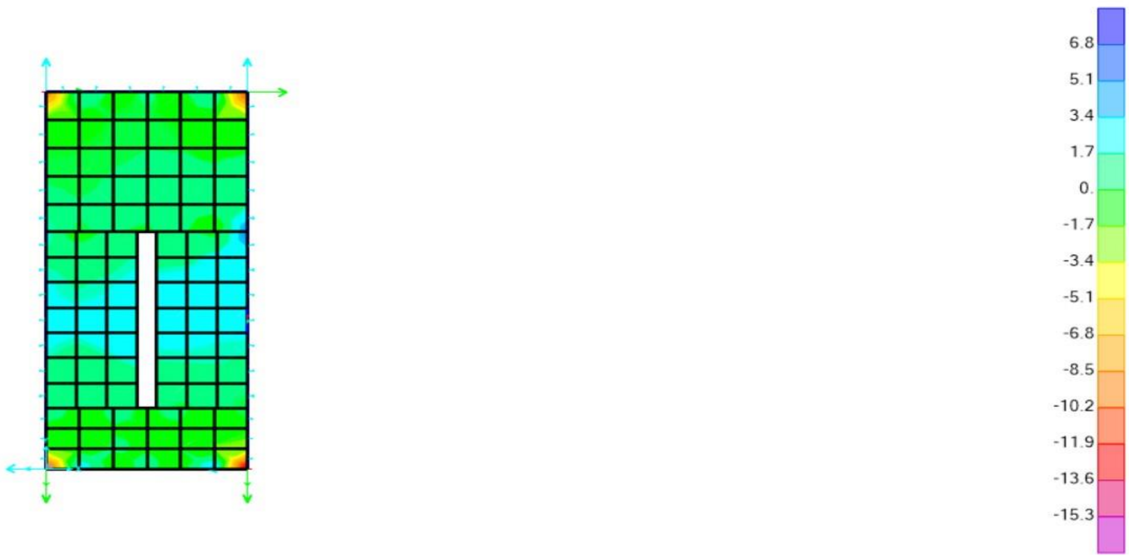


Figure 1.18 Moment axis xy

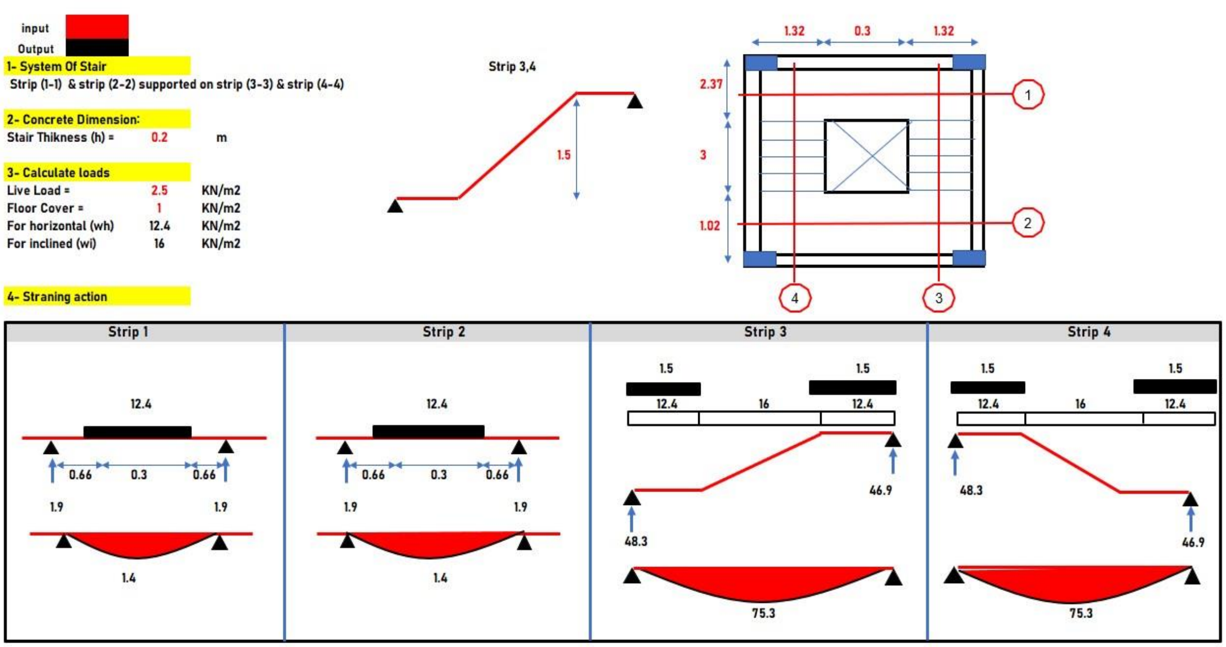


Figure 1.19 Sheet Excel of moment and BMD

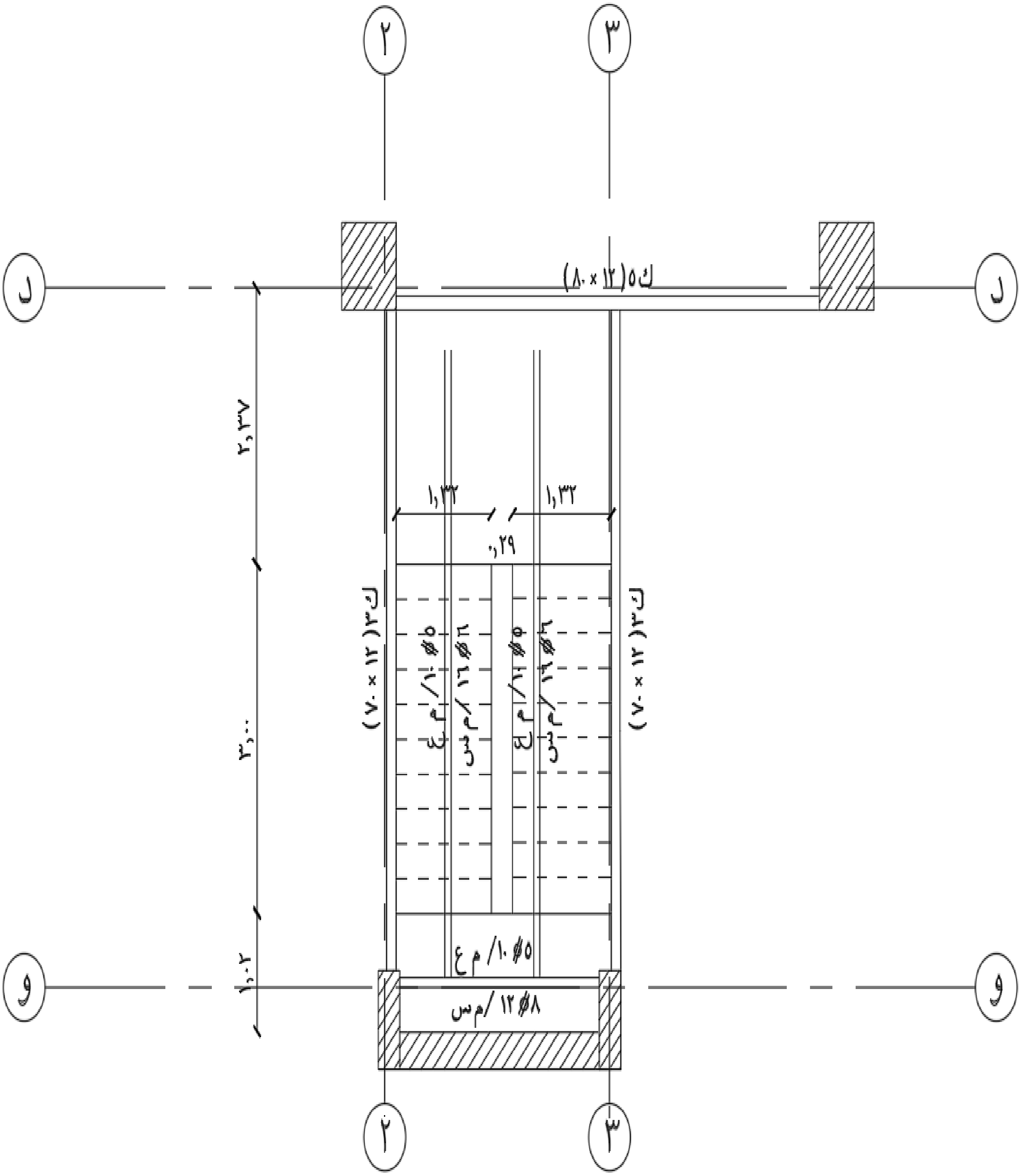


Figure 1.20 Stair Reinforcement in plan

1.3 Design of Stairs (Three Flight Stair Axis / أ – ب)
1.3.3 Manual solution

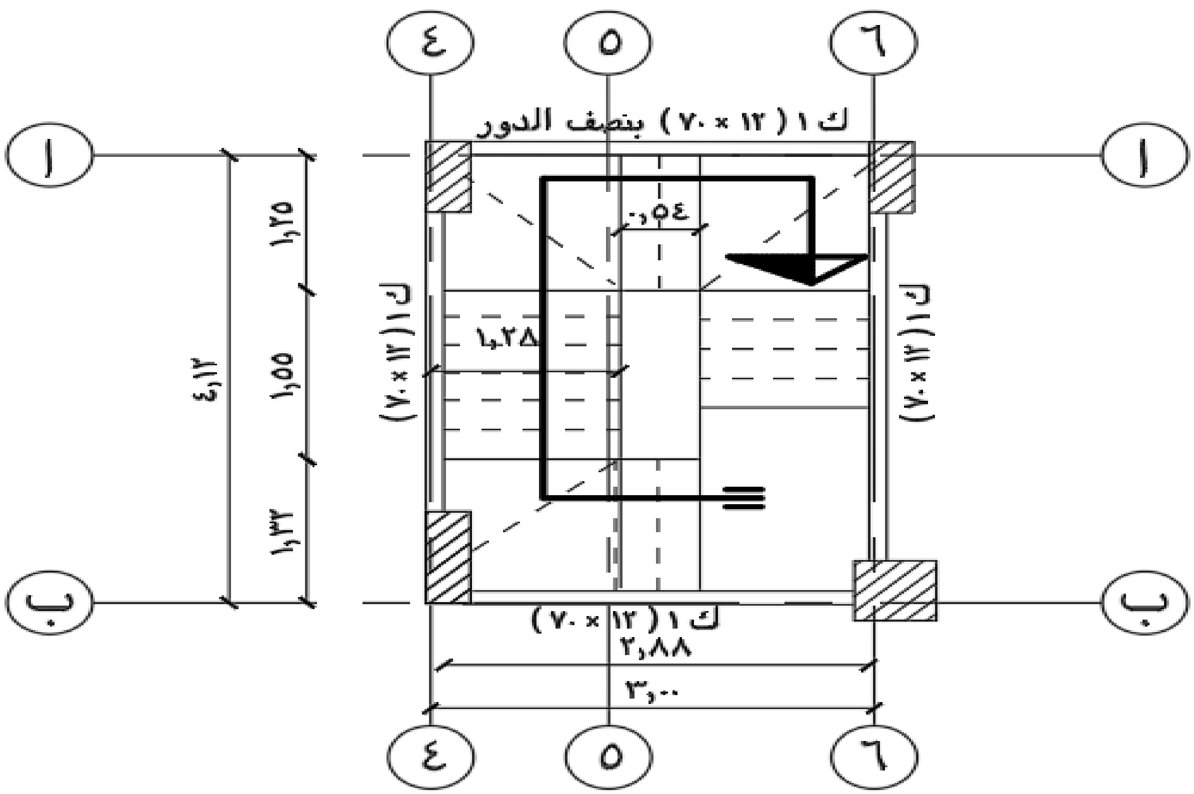


Figure 1.21 Stair Cross Section

1) Dimensions:

- $ts = \frac{Span}{24:30} = \frac{4.12m}{24:30} = 0.16\ m$
- $H_{Story} = 3.80\ m$
- $Rise = 0.15\ m$
- $Going = 0.30\ m$
- $\theta = \tan^{-1}(\frac{0.15}{0.30}) = 26.56\ °$

$t^* = \frac{ts}{Cos\theta} = \frac{16}{Cos(26.56)} = 17.88\ cm$

$t_{av} = t^* + \frac{Rise}{2} = 17.88 + \frac{15}{2} = 25.38$ cm

2) Loads:

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

  = 1.4 (25*0.2538 + 1.0) + 1.6 (2.5)

  = 14.283 KN/m^2

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

  = 1.4 (25*0.16 + 1.5) + 1.6(2.5)

  = 11.7 KN/m^2

3) For Strips :-

Shown In The figure

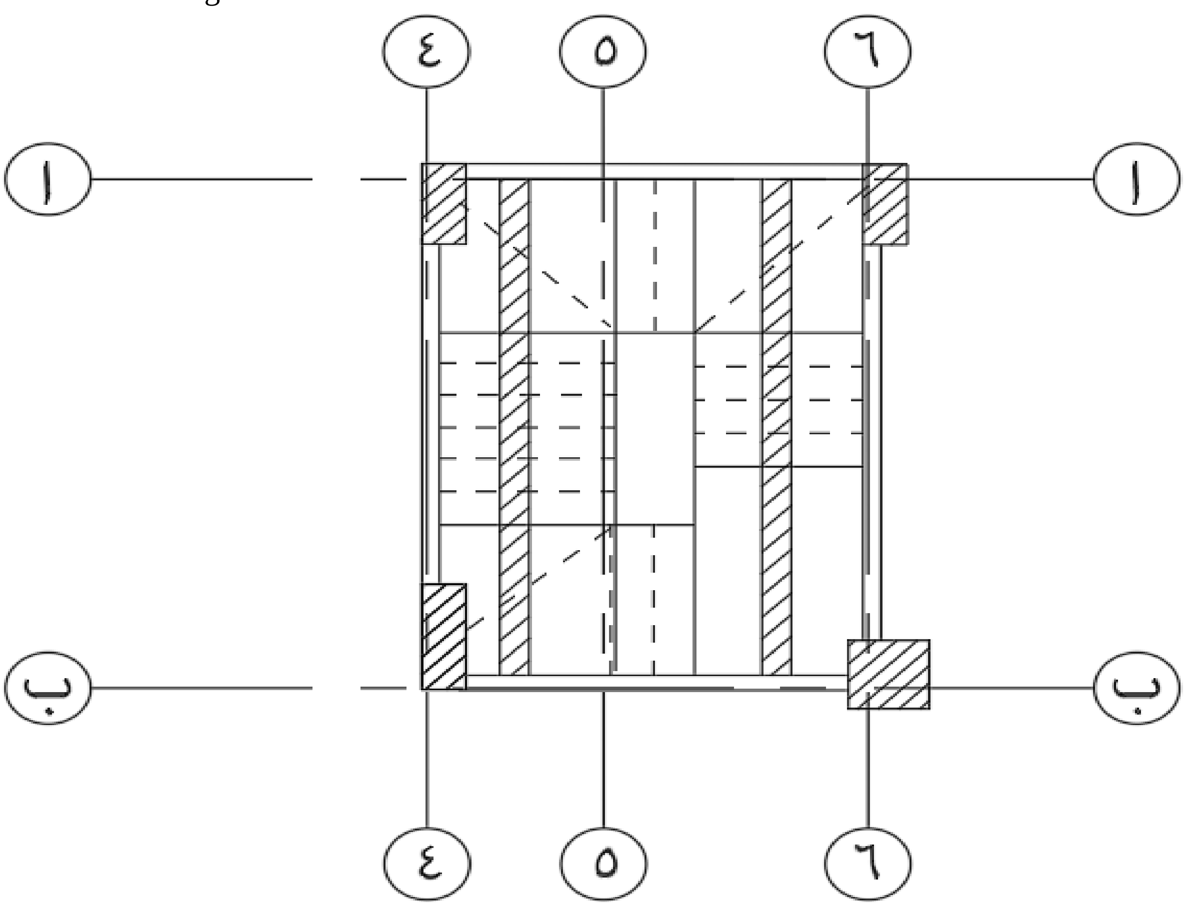


Figure 1.22 Strips Of Stair

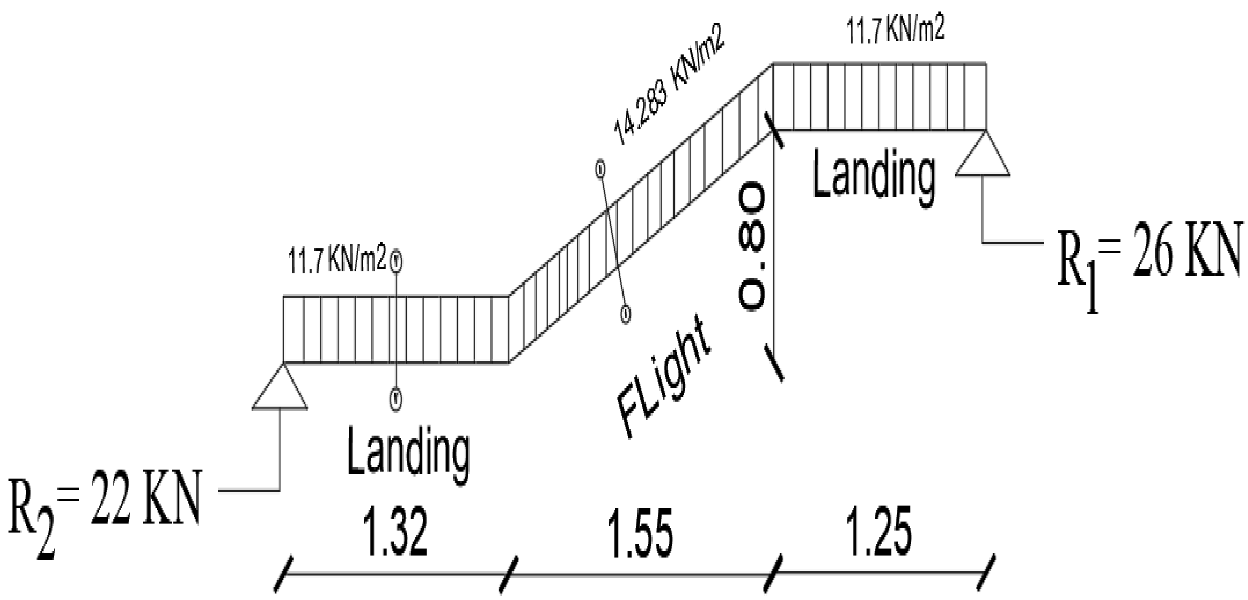
For Strip :-

- $R_1 * 4.12 = (11.7 * 1.32)\left(\frac{1.32}{2} + 1.55 + 1.25\right) + (14.283 * 1.55) * \left(\frac{1.55}{2} + 1.32\right) + (11.7 * 1.25)\left(\frac{1.25}{2}\right)$

  $\therefore R_1 = 13.73$ KN

- $R_2 = (11.7 * 1.32) + (14.283 * 1.55) + (11.7 * 1.25) - 26$

  $\therefore R_2 = 22$ KN



Moment :-

. M1

$R2\,(1.32) - 11.7 * 1.32 * \frac{1.32}{2} = 22\,(1.32) - 11.7 * 1.32 * \frac{1.32}{2} = 18.8$ KN.m

$\therefore M1 = 18.8$ KN.m

. M2

$R1\,(1.25 + 1.55) - 11.7 * 1.25\left(\frac{1.25}{2} + 1.55\right) - 14.283 * 1.55 * \frac{1.55}{2}$

$= 26\,(2.8) - 11.7 * 1.25\left(\frac{1.25}{2} + 1.55\right) - 14.283 * 1.55 * \frac{1.55}{2} = 23.8$ KN.m

$\therefore M2 = 23.8$ KN.m

. M3

R2(1.32 + 1.55) – 11.7 * 1.32 ($\frac{1.32}{2}$ + 1.55) – 14.283 * 1.55 *$\frac{1.55}{2}$

= 22 (2.87) – 11.7 * 1.32 ($\frac{1.32}{2}$ + 1.55) – 14.283 * 1.55 *$\frac{1.55}{2}$ = 11.85 KN.m

∴ M3 = 11.85 KN.m

. M4

R1 (1.25) – 11.7 * 1.25 * $\frac{1.25}{2}$ = 26 (1.25) – 11.7 * 1.25 * $\frac{1.25}{2}$ = 23.35 KN.m

∴ M4 = 23.35 KN.m

∴ Moment of flight = 23.57 KN.m

4) . Design of Section

1- M = 23.57 KN.m

$R_1 = \frac{M_u}{F_{cu}\, b\, d^2} = \frac{23.57 * 10^6}{35*1000*200^2} = 0.02 < R_{x\,max} = 0.129$

ɯ = 0.03

A_s = ɯ $\frac{Fcu}{Fy} bd = 0.03 * \frac{35}{360} * 1000 * 200 = 583.33$ mm^2 = 5.833 cm^2

Use A_S ⇛ 6 ∯ 16 / m ⇛ A_{sact} =1205.76 mm^2

2- M1 = 18.8 KN.m

$R_1 = \frac{M_u}{F_{cu}\, b\, d^2} = \frac{18.8 * 10^6}{35*1000*200^2} = 0.02 < R_{x\,max} = 0.129$

ɯ = 0.03

∴ A_s = ɯ $\frac{Fcu}{Fy} bd = 0.03 * \frac{35}{360} * 1000 * 200 = 583.33$ mm^2 = 5.833 cm^2

Use A_S ⇛ 6 ∯ 16 / m ⇛ A_{sact} =1205.76 mm^2

Draw of moment and section and details of RFT

Moment

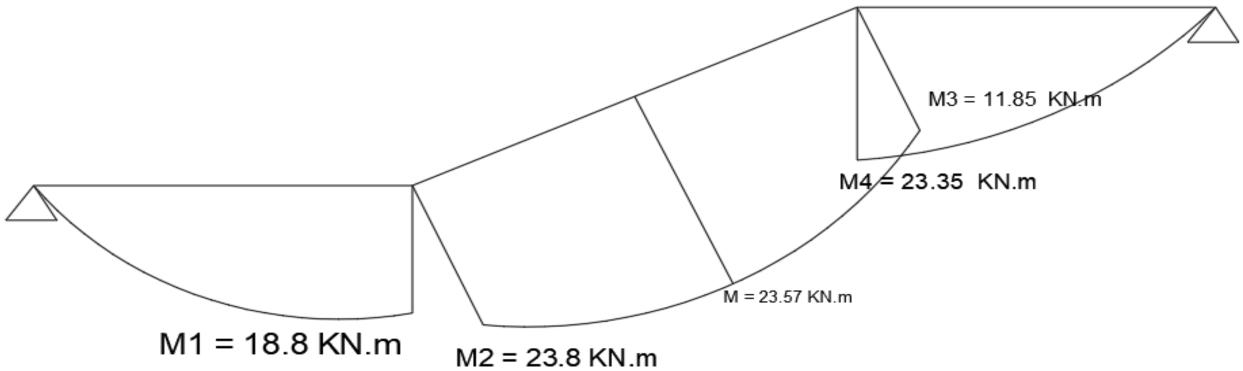


Figure 1.23 BMD

Section and details of RFT

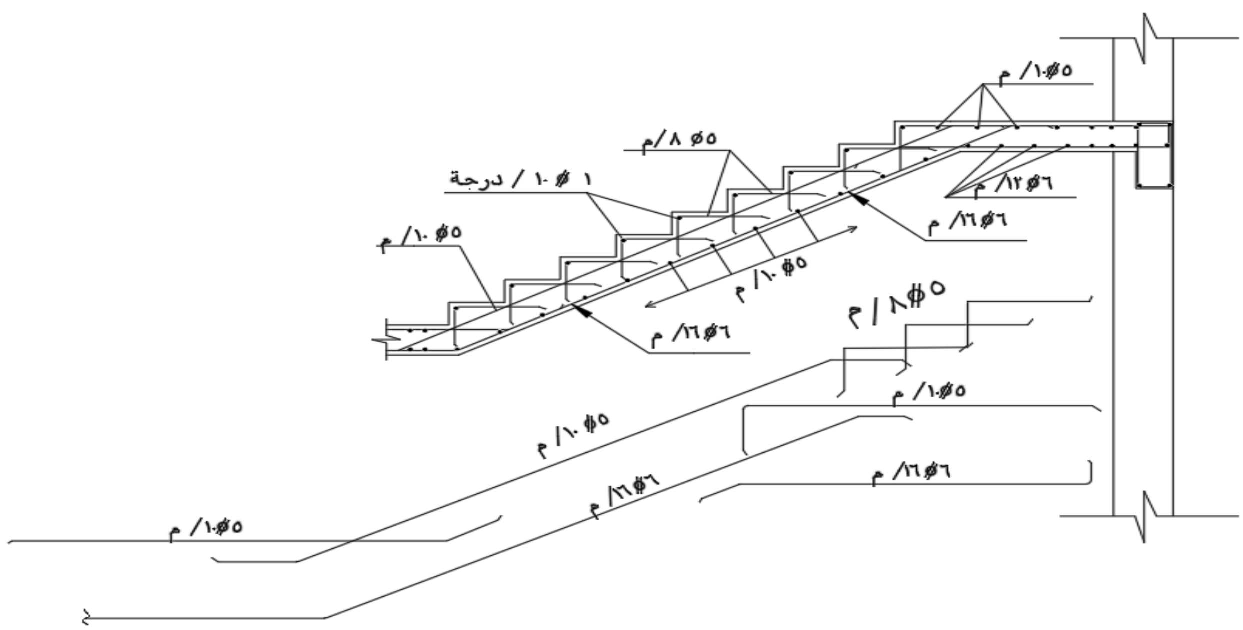


Figure 1.24 Section and details of RFT

1.3.4 Using Sap Program

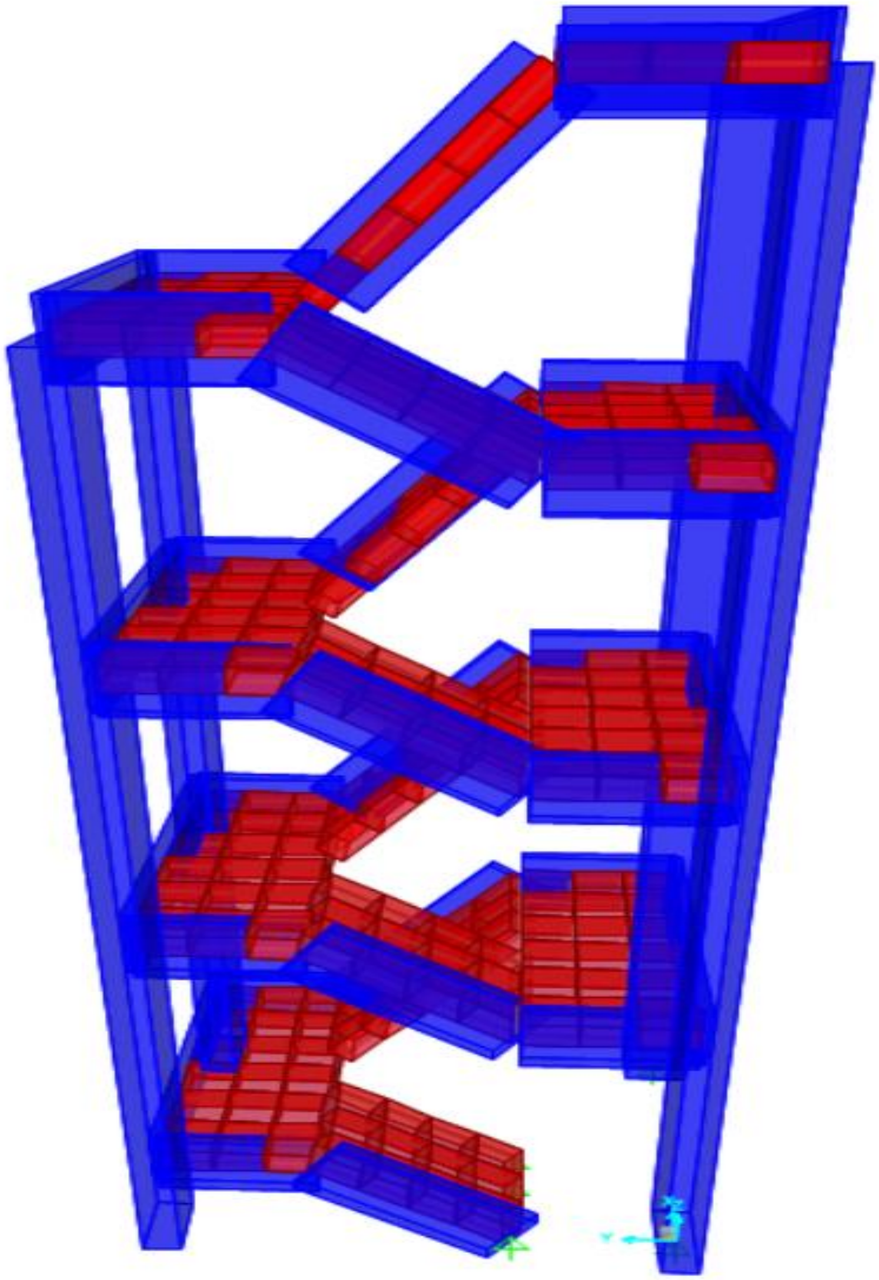


Figure 1.25 3D Stair

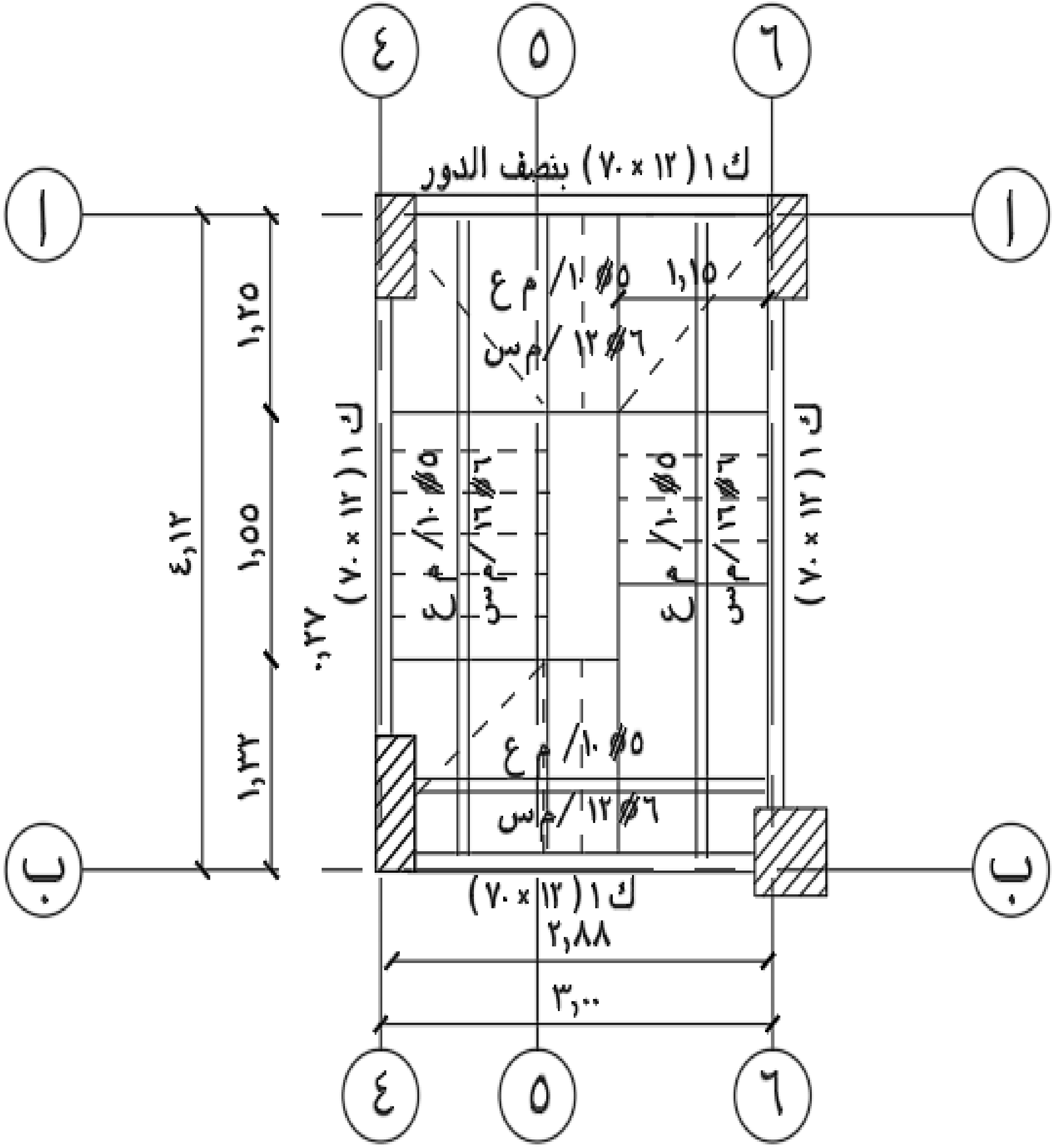


Figure 1.26 Stair Reinforcement in plan

1.4 Design of Beams

Concrete dimensions

- Assume b = 12 cm
- d = $\frac{L}{12}$ = 80 cm , d = $\frac{L}{12}$ = 70 cm
- **Loads on beams**
    - Own weight
    - Load from slab
    - Load from wall

- As=$\frac{Mu}{Fy*j*d}$

1.4.1 DESGION OF Beam

Moment and shear

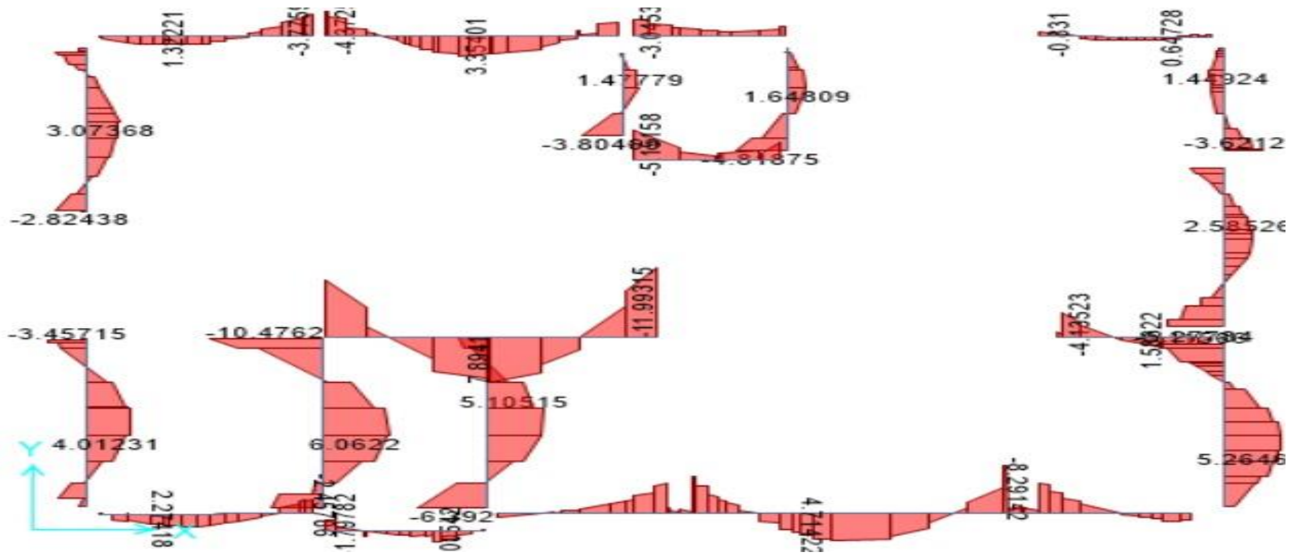


Figure 1.27 Moment Basement Slab

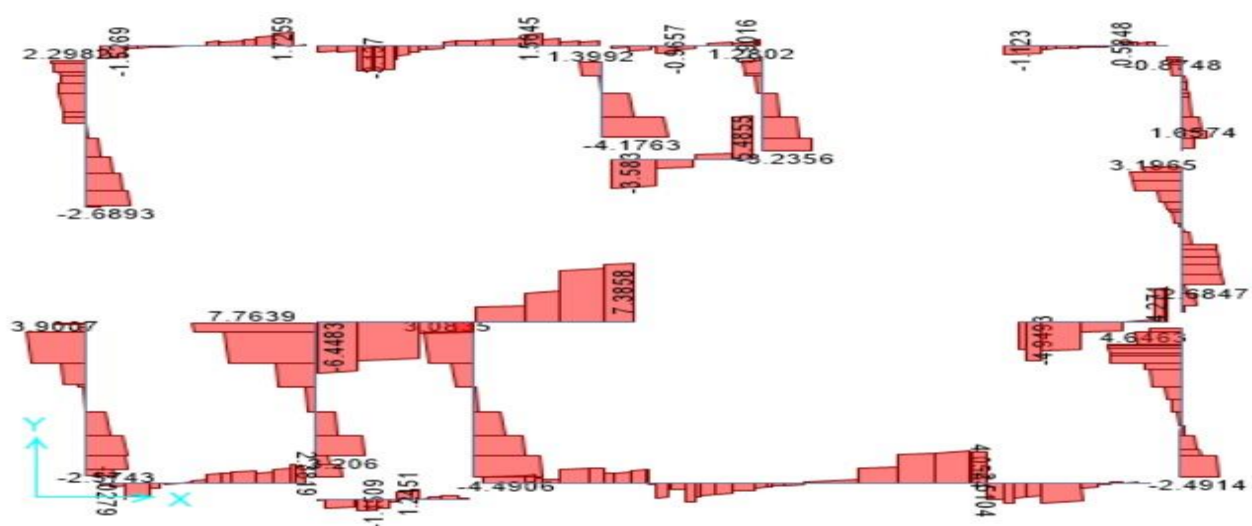


Figure 1.28 Shear Basement Slab

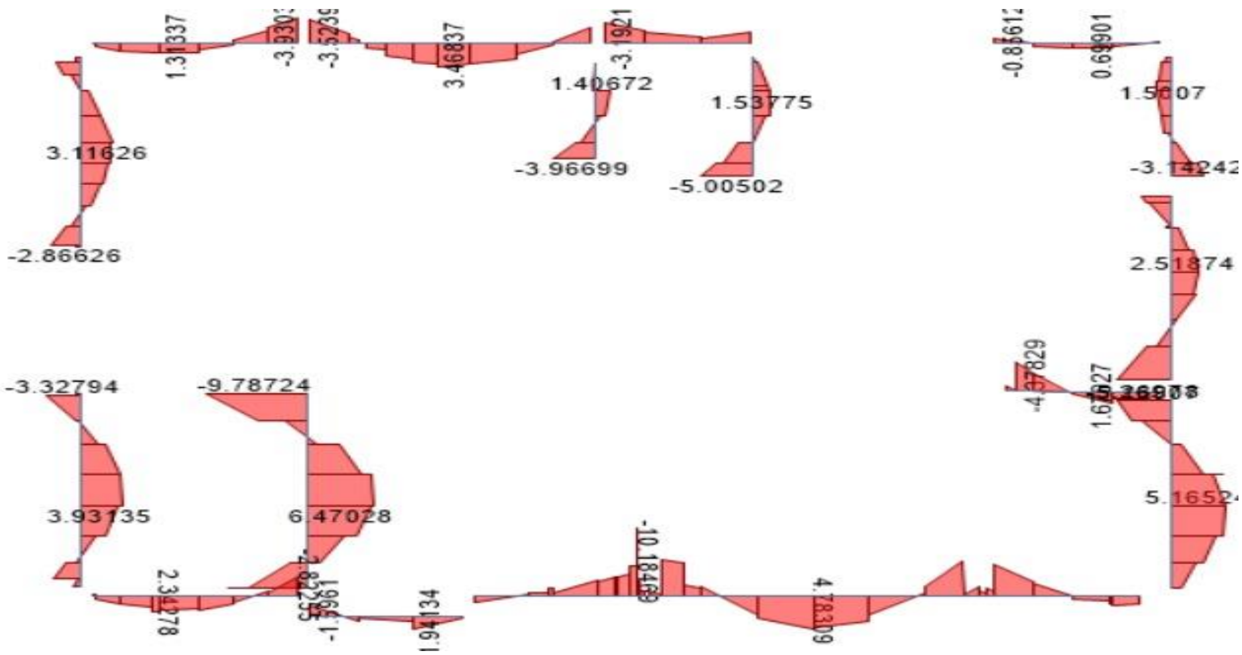


Figure 1.29 Moment Ground Slab

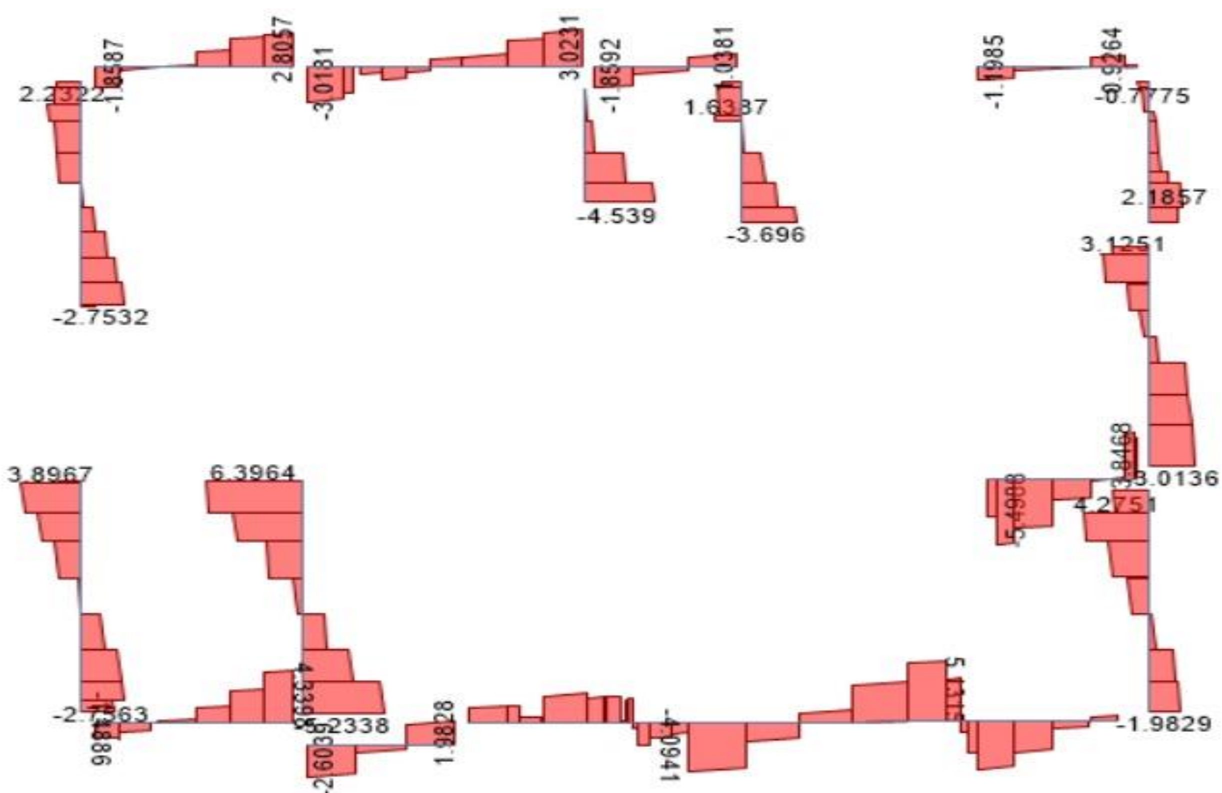


Figure 1.30 Shear Beam Ground Slab

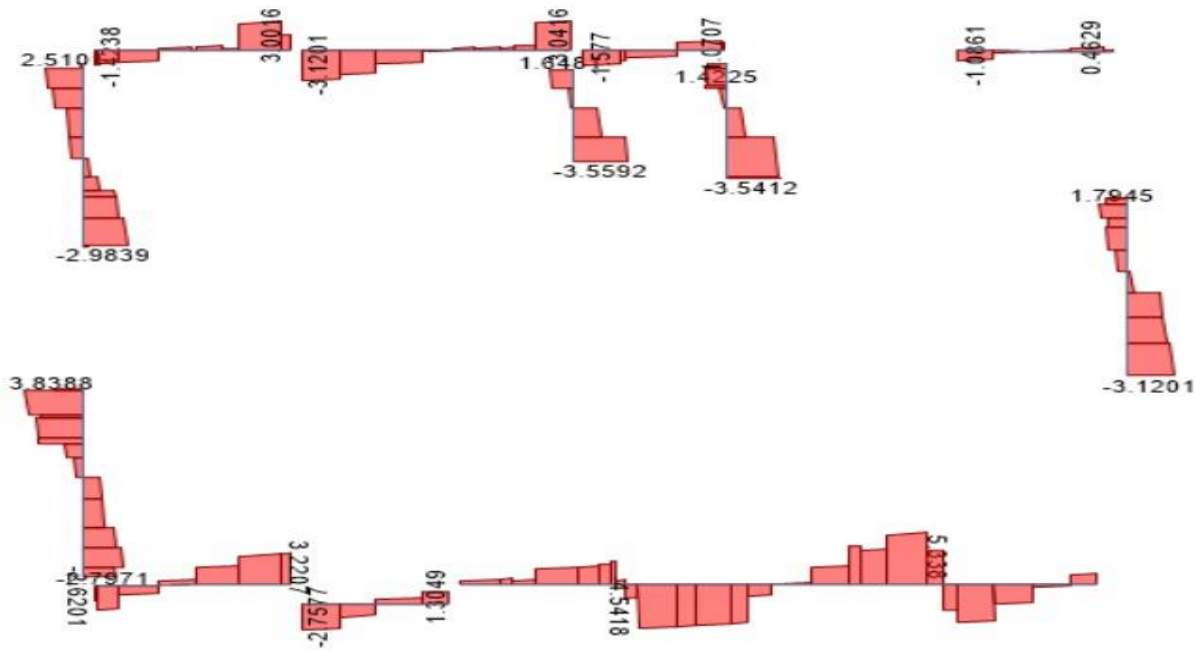


Figure 1.31 Moment Beam First Slab

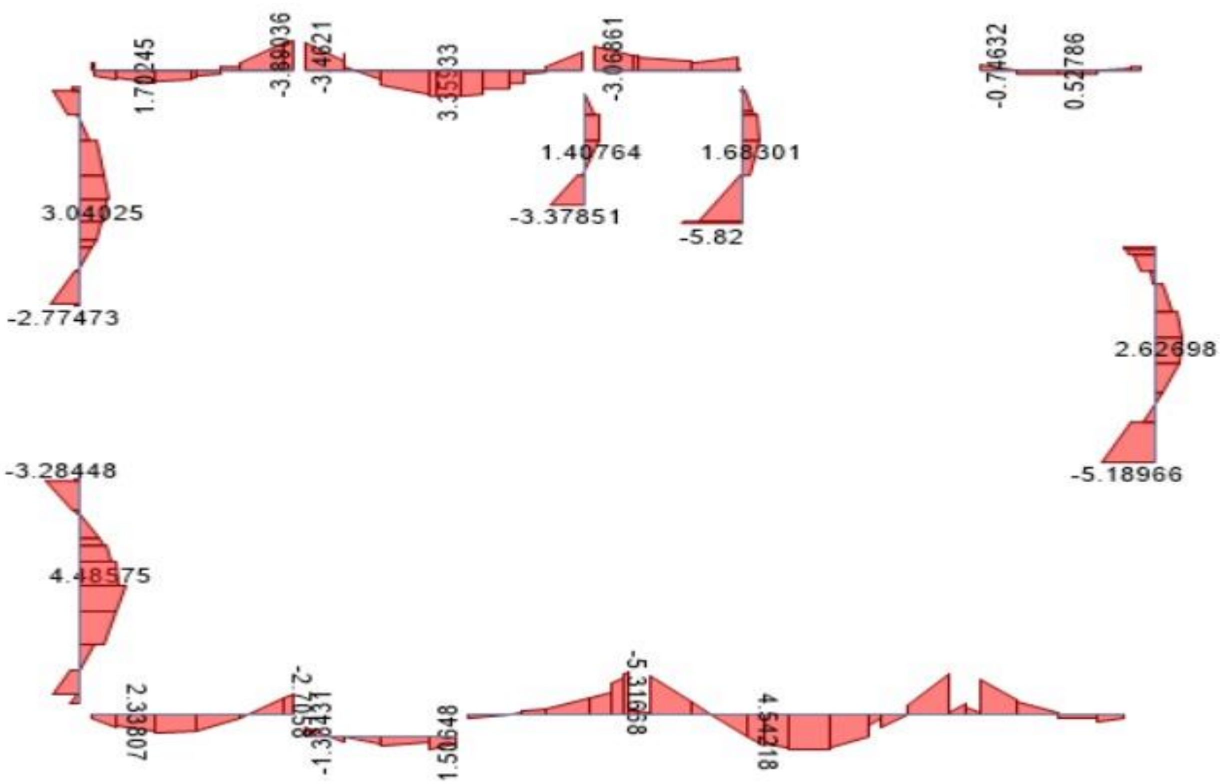


Figure 1.32 Shear Beam First Slab

جدول تسليح الكمرات :

نموذج	تسليح سفلي	تسليح علوي		كانات/ م	ملاحظات
	عدل	فوق الركيزه	منتصف البحر		
ك ١	٢ Ø ١٢	٤ Ø ١٢	-------	٥ Ø ٨	
ك ٢	٤ Ø ١٢	٤ Ø ١٢	-------	٥ Ø ٨	
ك ٣	٢ Ø ١٦	٢ Ø ١٦	-------	٥ Ø ٨	
ك ٤	٤ Ø ١٦	٤ Ø ١٦	-------	٥ Ø ٨	
ك ٥	٦ Ø ١٦	٦ Ø ١٦	-------	٦ Ø ٨	

Figure 1.33 Table RFT beam

1.4.2 Check of Shear on Beam Section:

Max shear on Basement Slab Qmax = 7.76 t

- $q_{cu}(uncracked) = 0.16\sqrt{\frac{Fcu}{\gamma c}} = 0.16\sqrt{\frac{35}{1.5}} = 0.77$ N/mm²
- $q_{max} = 0.7\sqrt{\frac{Fcu}{\gamma c}} = 0.7\sqrt{\frac{35}{1.5}} = 3.38$ N/mm²
- $q_u = \frac{Qmax}{b*d} = \frac{7.76*10^4}{250*780} = 0.4$ N/mm²

$qcu > qu$, Use Stirrups 6 Ø 8 / m as minimum

Max shear on Ground Slab Qmax = 6.4 t

- $q_{cu}(uncracked) = 0.16\sqrt{\frac{Fcu}{\gamma c}} = 0.16\sqrt{\frac{35}{1.5}} = 0.77$ N/mm²
- $q_{max} = 0.7\sqrt{\frac{Fcu}{\gamma c}} = 0.7\sqrt{\frac{35}{1.5}} = 3.38$ N/mm²
- $q_u = \frac{Qmax}{b*d} = \frac{6.4*10^4}{250*680} = 0.46$ N/mm²

$qcu > qu$, Use Stirrups 5 Ø 8 / m as minimum

Max shear on First Slab Qmax = 5.3 t

- $q_{cu}(uncracked) = 0.16\sqrt{\frac{Fcu}{\gamma c}} = 0.16\sqrt{\frac{35}{1.5}} = 0.77$ N/mm²
- $q_{max} = 0.7\sqrt{\frac{Fcu}{\gamma c}} = 0.7\sqrt{\frac{35}{1.5}} = 3.38$ N/mm²

$q_u = \frac{Qmax}{b*d} = \frac{5.3*10^4}{250*680} = 0.31$ N/mm²

$qcu > qu$, Use Stirrups 5 Ø 8 / m as minimum

1.5 Design of Columns

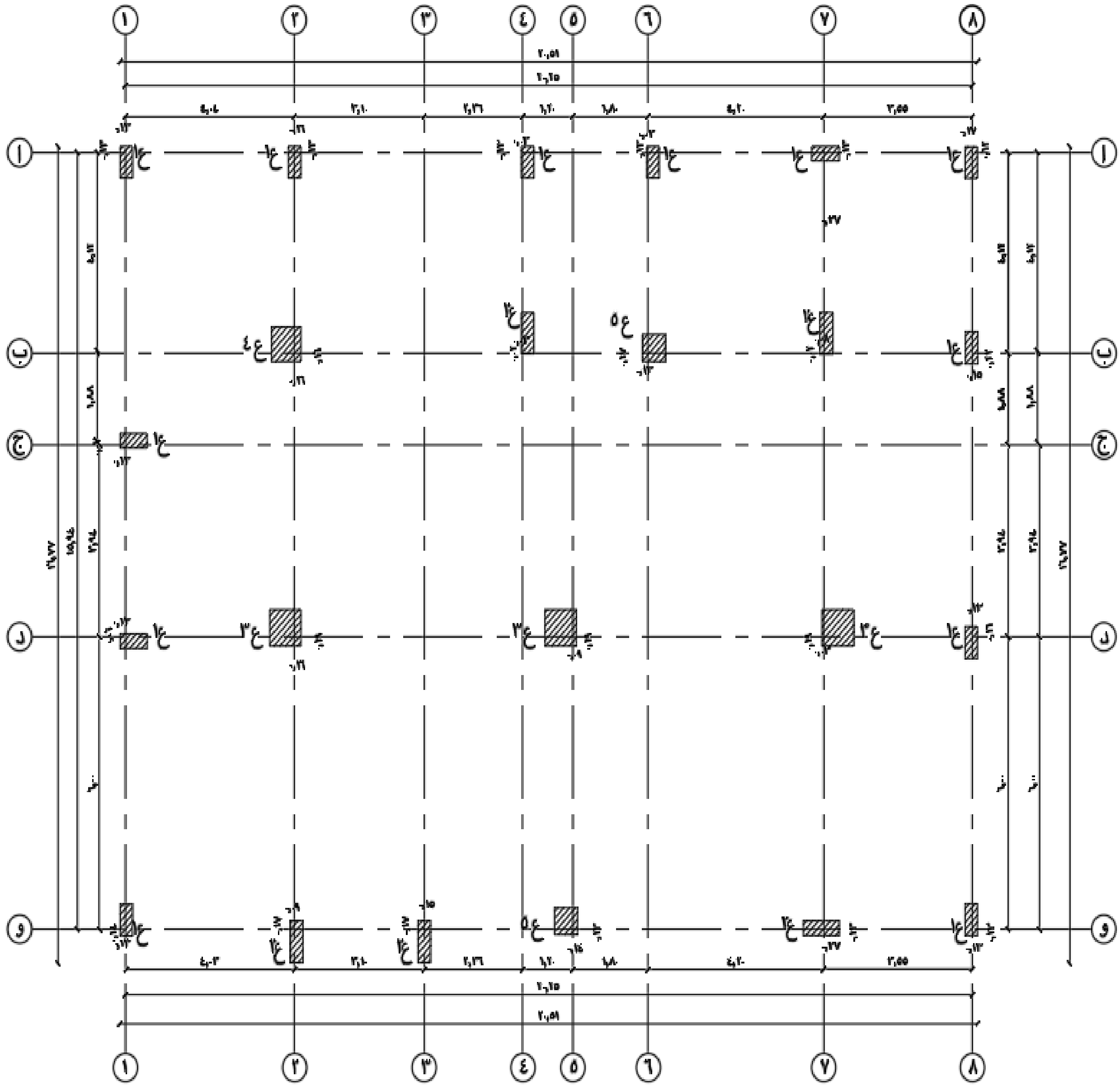


Figure 1.34 Columns Axis

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

❖ **For 1ع on axis (8 -د)**

➢ **Concrete Dimension**

Use the largest Of

- $\frac{L}{20} = \frac{6}{20} = 0.3$ m = 30 cm
- $\frac{h}{15} = \frac{3}{15} = 0.20$ m = 20 cm
- $30\ cm$

Use b = 30 cm

- P_U = 150 ton (From Etabs Program)
- $P_{u\ actual}$ = 150 ton
- $P_{u\ actual} = 0.35\ (A_c - A_s)F_{cu} + 0.67\ A_s\ F_y \ \rightarrow$ assume $A_s = 0.01\ A_c$
- $150 * 10^4 = 0.35 * (0.99\ A_c) * 35 + 0.67 * 360 * 0.01\ A_c$
- $A_c = 103167.23\ mm^2$
- $t = \frac{Ac}{b} = \frac{103167.23}{300} = 343.89$ mm $\sim$ 650 mm
- $A_{cact} = 30 * 65 = 1950\ mm^2$

❖ Check of Buckling (braced column)

- in short direction

$\lambda = \frac{k*Ho}{b} = \frac{.8*3}{0.3} = 8 < 15 \Rightarrow$ Short Column

- in Long direction

$\lambda = \frac{k*Ho}{t} = \frac{0.8*3}{.650} = 3.69 < 15 \Rightarrow$ Short Column

- $A_s = 0.01*30*65 = 19.5\ cm^2$ Use 10 ∯ 16

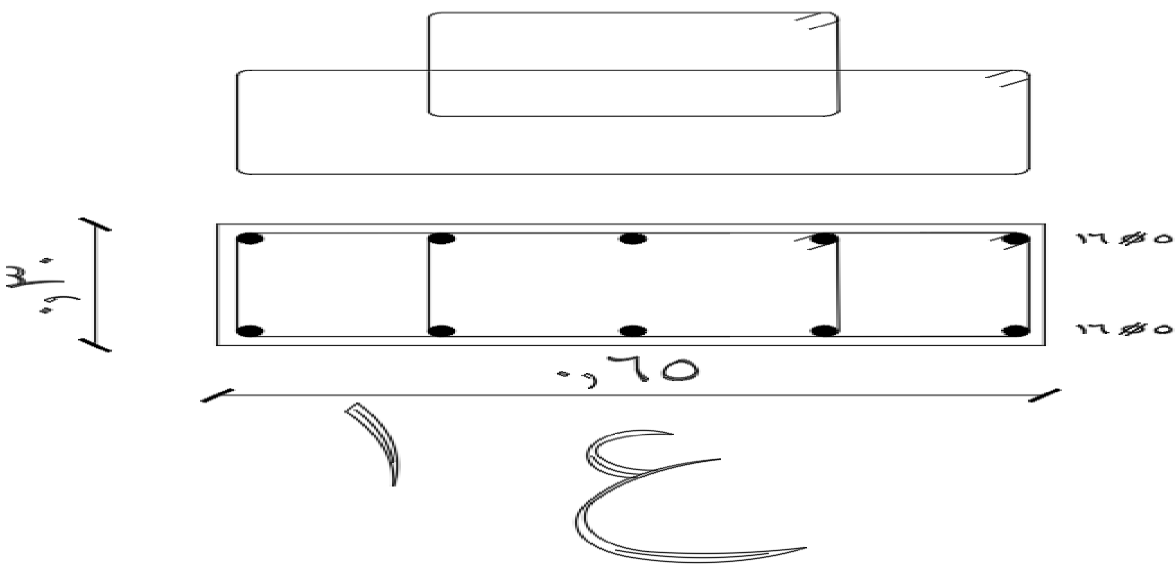


Figure 1.35 Column Cross Section

1.5.2 Table of Columns

Joint	P_u etabs (T)	$P_{u\ act}$ (T)	$\mu = A_s/A_c$	b (cm)	t (cm)	∯	Sample
1	150.01	150	1.00%	30	65	10∯16	C1
2	135.355	135	1.00%	30	65	10∯16	C1
3	135.355	135	1.00%	30	65	10∯16	C1
4	134.09	134	1.00%	30	65	10∯16	C1
5	144.21	144	1.00%	30	65	10∯16	C1
6	129.03	129	1.00%	30	65	10∯16	C1
7	112.585	113	1.00%	30	65	10∯16	C1
8	108.79	109	1.00%	30	65	10∯16	C1
9	98.67	99	1.00%	30	65	10∯16	C1
10	98.67	99	1.00%	30	65	10∯16	C1
11	92.345	92	1.00%	30	65	10∯16	C1
12	56.925	57	1.00%	30	65	10∯16	C1
13	228.8	229	1.00%	30	85	12∯16	C2
14	214.5	215	1.00%	30	85	12∯16	C2
15	187.22	187	1.00%	30	85	12∯16	C2
16	165.715	166	1.00%	30	85	12∯16	C2
17	120.175	120	1.00%	30	85	12∯16	C2
18	440.22	440	1.00%	75	75	28∯16	C3
19	400	440	1.00%	75	75	28∯16	C3
20	431.365	431	1.00%	75	75	28∯16	C3
21	333.96	334	1.00%	70	70	22∯16	C4
22	222.64	223	1.00%	55	55	20∯16	C5
23	211.255	211	1.00%	55	55	20∯16	C5

Table 1.36 Columns Load And Section (Ultimate)

1.6 Design of Foundation

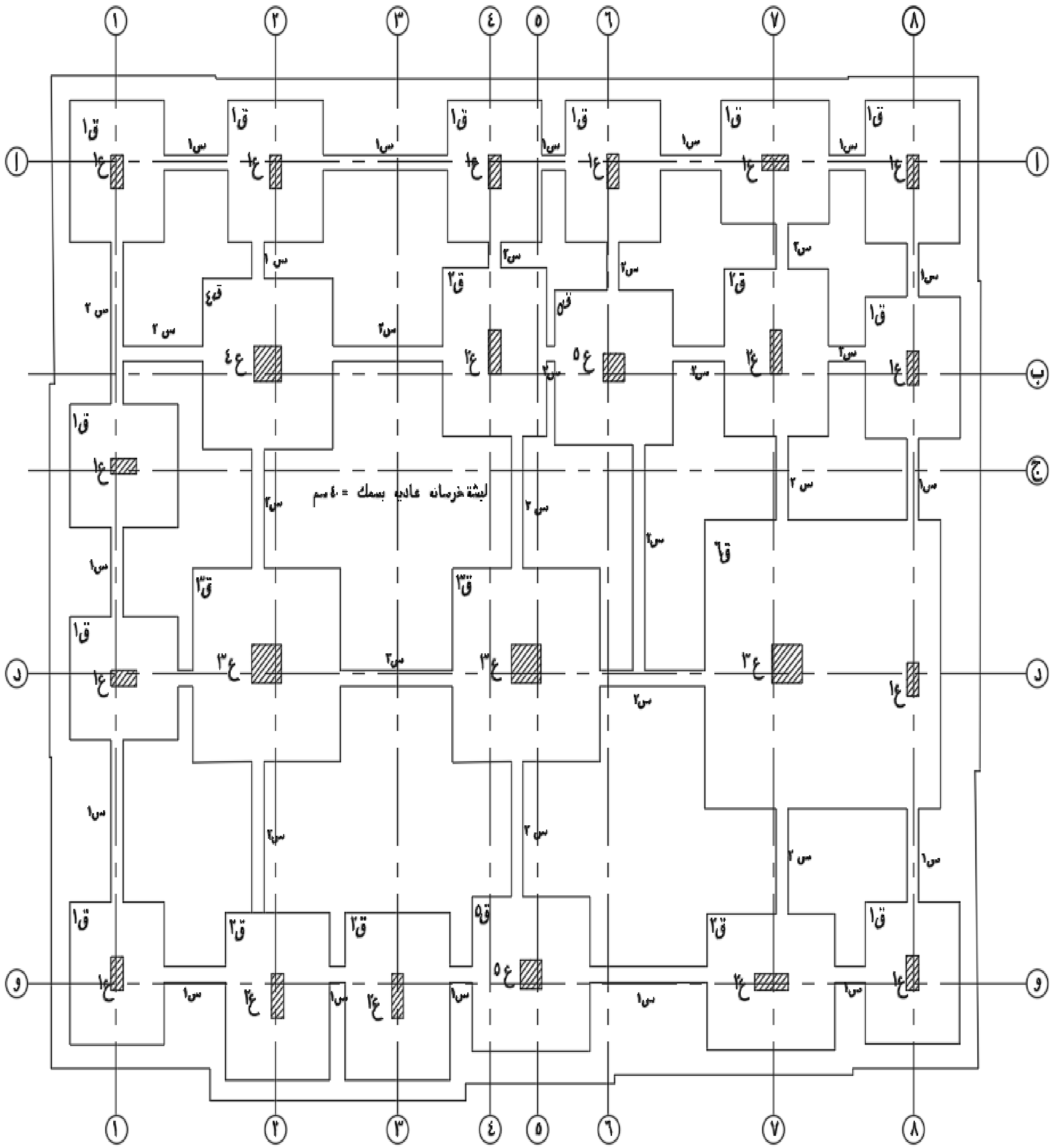


Figure 1.37 Foundations

1) Bearing Capacity for Soil = 1 t/m^2 = 10 kg/cm^2
2) Tensile Steel Stress Fy = 360
3) Compressive Concrete Stress Fcu = 35

1.6.1 For (ق1)

1.1 Input Data:

- Column Working Load = 106.71 ton
- Column Dimension a = 30 cm
  b = 65 cm
- Plain Concrete Depth = 0.4 m
- Plain Concrete Extension = 0.40 m

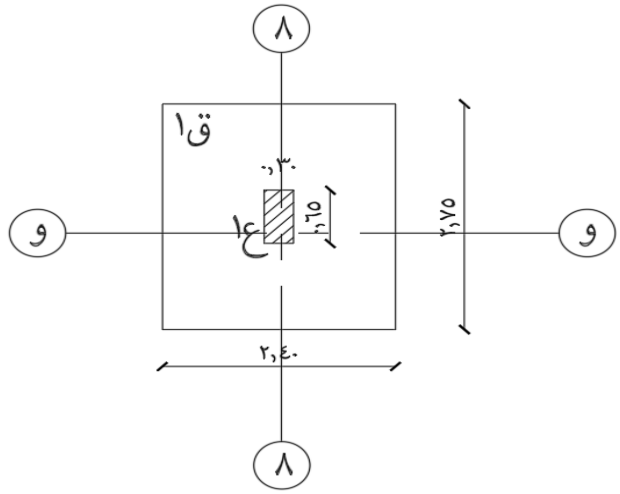


Figure 1.38. Sections of footing

1.2 Concrete Dimension :

Figure 1.64 Section of footing (elevation)

- B.C = $\frac{106.71}{A_{pc}}$
- $10 = \frac{106.7}{A_{Pc}}$
- A_{Pc}= 10.671 m^2 = Lpc *Bpc
  = (0.65 + 2c) * (0.30 + 2c)

  C = 1.441 m

- Lpc = 0.65 + 2 *1.441 = 3.54 m
- Bpc = 0.30 + 2 * 1.441 = 3.18 m

∴ $L_{RC} = L_{pc} - 2t_{pc}$ = 3.54 – 2 * 0.40 = 2.75 m

$B_{RC} = B_{PC} - 2t_{pc}$ = 3.18 – 2* 0.40 = 2.4 m

1.3 Check of Stress:

- $q_{act} = \frac{106.71}{A_{PC}}$
- $q_{act} = \frac{106.71}{3.54*3.18}$ = 9.47 t/m^2 < B.C = 10 t/m^2, Ok

$q_{act} = \frac{P_U}{A_{R.C}} \frac{106.71}{2.75*2.4}$ = 161.68 KN

$Z = \frac{2.75-0.65}{2} = 1$

$M_U = \frac{q_{act} * Z^2}{2} = \frac{161.68 * 1^2}{2} = 85$ KN.m

d $= c1\sqrt{\frac{M_U}{FCU*b}} = C1\sqrt{\frac{85*10^6}{35*1000}}$ = 246.40 ➡ d = 600 ➡ t = 700

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

- Qp = 1067 – 161.68 * (0.3 + 0.6)*(0.64 + 0.6) = 887 KN
- $q_p = \frac{Qp}{2(\ (a+d)+(a+d))*d}$
- $q_p = \frac{887 * 10^3}{2(\ (300 + 600\ )+(\ 650 + 600))*600}$ = 0.35 N/mm^2

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

2- $0.316*(.5+\frac{a}{b}) * \sqrt{\frac{F_{CU}}{\gamma_C}}$ = $0.316*(\ 0.5 + \frac{0.3}{0.65}\ ) * \sqrt{\frac{35}{1.5}}$ = 1.46 Nlmm^2

3- $0.316* \sqrt{\frac{F_{CU}}{\gamma_C}}$ = $0.316* \sqrt{\frac{35}{1.5}}$ = 1.53 Nlmm^2

4- $0.8*(\frac{\propto * d}{2(a+d+b+d)} +.2)* \sqrt{\frac{F_{CU}}{\gamma_C}}$ =

= $0.8\ (\frac{4*600}{2(\ 300 + 600 + 650 + 600\ )} + 0.2\ ) * \sqrt{\frac{35}{1.5}}$ = 2.93 N/mm^2

$q_p < q_{p\ (allow)}$ ➡ OK, safe

1.4 Check of Shear

$Qsh_1 = q_{act}$ * c = 161.68 * 1.441 = 232.98 ton

$qs_h = \frac{Qsh}{B*d} = \frac{232.98 * 10^3}{2400 * 600}$ = 0.161 KN / mm^2

$q_{sh(allow)}$ = $0.16 * \sqrt{\frac{F_{CU}}{\gamma_C}}$ = $0.16 * \sqrt{\frac{35}{1.5}}$ = 0.773 KN / mm^2

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

1.5 RFT

$$A_S = \frac{M_U}{F_Y*J*d} = \frac{85*10^6}{360*0.826*600} = 476.41 \text{ mm}^2$$

$$A_S = 9 \oiint 12 / m$$

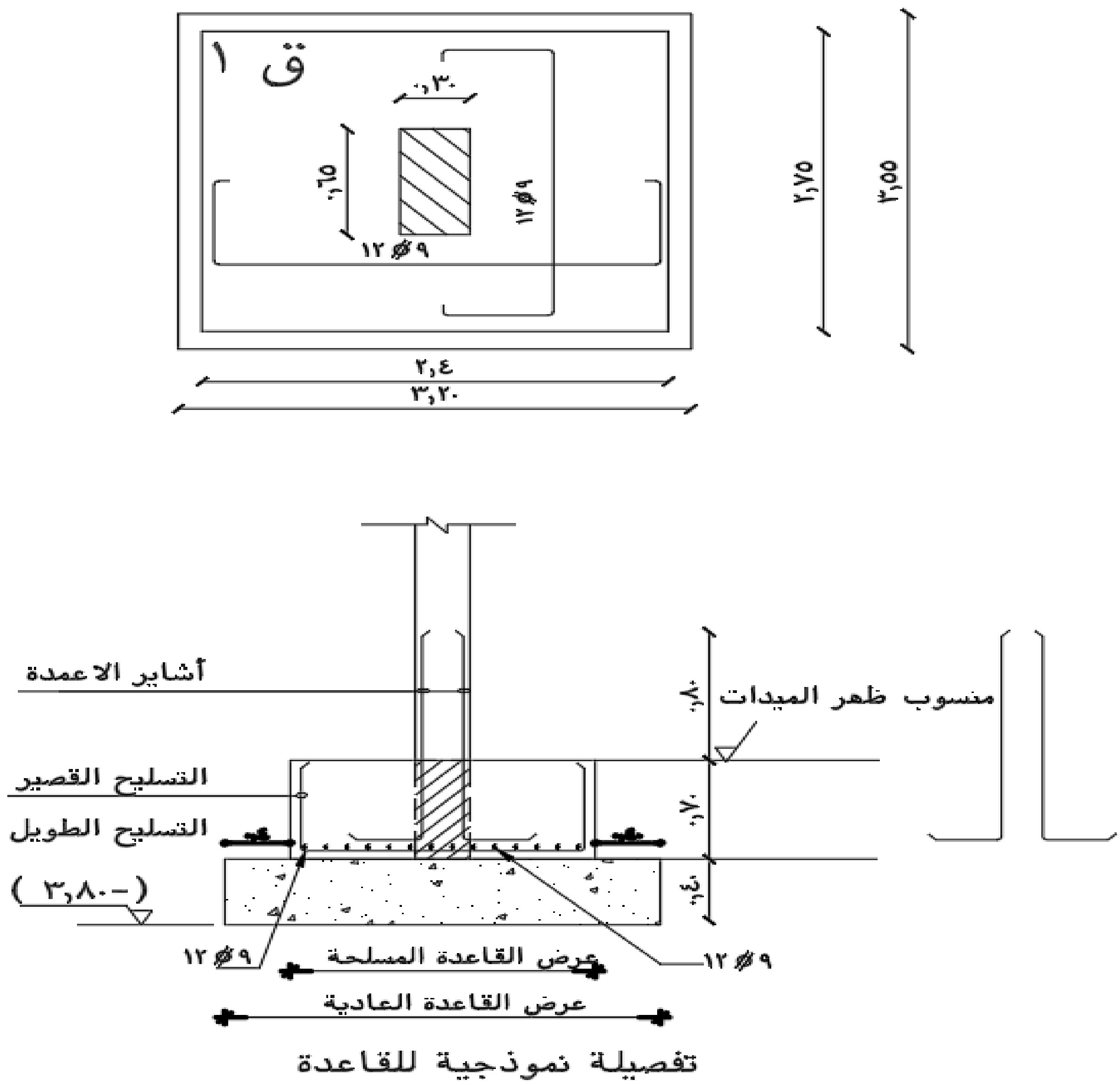


Figure 1.39. Sections of footing and details' of RFT

1.6.2 For (ق 6)

2.1 Input Data:

- Column 1 Working Load P1 = 118 ton
  Column 3 Working Load P2 = 316 ton
- Column Dimension a1 = 30 cm
  B1 = 65 cm
  Column Dimension a2 = 75 cm
  B2 = 75 cm
- Plain Concrete Depth = 0.4 m
- Plain Concrete Extension = 0.40 m

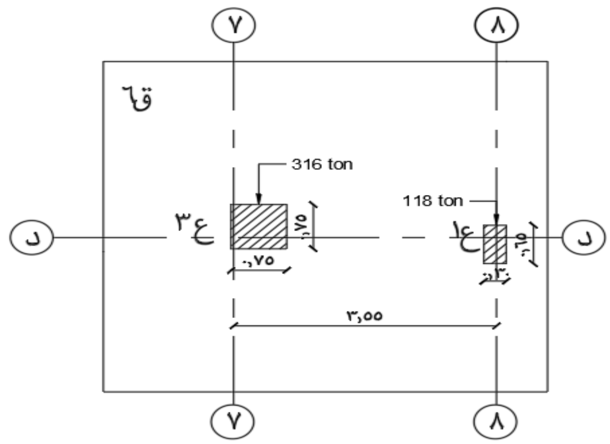


Figure 1.40. Sections of footing

SOLUTION :-

2.2 - Get R Location

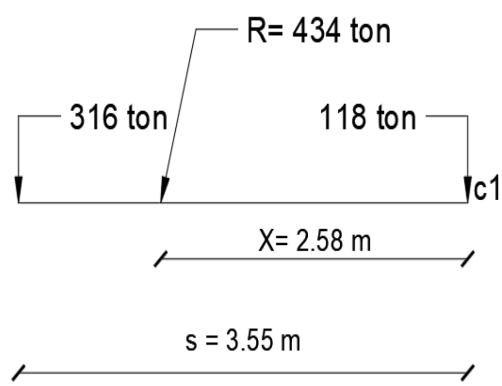
R = P1+ P2 = 118 + 316 = 434 ton

$\varepsilon m@\ c1 = 0.0$

R * x = P2 * s

434 * x = 316 * 3.55

∴ X = 2.58 m



2.3 - Concrete Dimension

B.C = Pw/Apc

$10 = \frac{470}{A_{Pc}}$

$A_{PC} = 47\ m^2$

$L_{RC} = 2\ (\ X + \frac{a1}{2}\ ) = 2\ (\ 2.58 + \frac{0.09}{2} + 0.5\ ) = 6.2$ m

∴ $B_{RC} = 5.45$ m

∴ $B_{PC} = 5.45 + 2t_{PC} = 5.45 + 2 * 0.40 = 6.25$ m

$L_{PC} = 6 + 2t_{PC} = 6.2 + 2 * 0.40 = 7$ m

2.4 - Check of Stress

- $q_{act} = \frac{434}{A_{PC}}$
- $q_{act} = \frac{434}{6.25*7} = 9.98\ t/m^2 < B.C = 10\ t/m^2$, Ok safe

2.5 Design of Longitudinal direction

Calculate of actual uniform load on Rc Footing (U.L) as a beam :-

$P2_{UL} = 316 * 1.5 = 474$ ton

$P1_{UL} = 118 * 1.5 = 177$ ton

$Pt_{UL} = 434 * 1.5 = 651$ ton

$W_{UL} = \frac{Pt\ UL}{L_{RC}} = \frac{651}{6.2} = 105$ ton / m

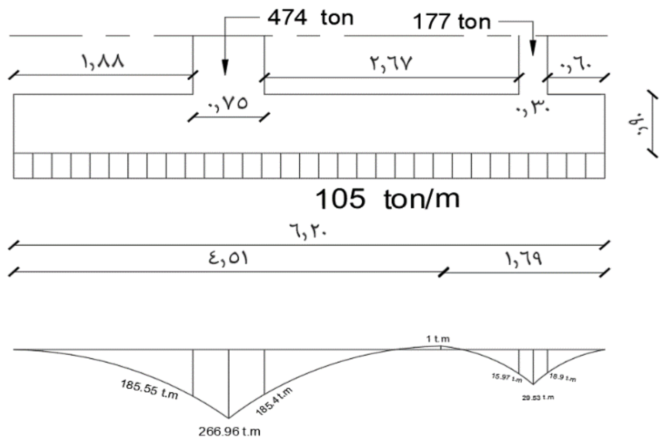


Figure 1.41 BMD

point of zero shear and drawing (BMD) :-

zero shear : $105 * y - 474 = 0.0$

$y = 4.51$ m

$M1 = \frac{w*L*L}{2} = \frac{105*1.88*1.88}{2} = 185.55$ t.m

$M2 = \frac{w*L*L}{2} = \frac{105*2.255*2.2.255}{2} = 266.96$ t.m

$M3 = 105 * \frac{L*L}{2} - P2 * \frac{0.75}{2} = 105 * \frac{2.63*2.63}{2} - 474 * 0.375 = 185.4$ t.m

$M_{zero\ shear} = (105 * \frac{Y*Y}{2}) - (P2\ (0.375 + 1.83)) = (105 * \frac{4.51*4.51}{2}) - (474 * 2.255) = -1.014$ t.m

$M4 = \frac{w*L*L}{2} - P1 * 0.15 = \frac{105*0.90*0.90}{2} - 177 * 0.15 = 15.975$ t.m

$M5 = \frac{w*L*L}{2} = \frac{105*0.75*0.75}{2} = 29.53$ t.m

$M6 = \frac{w*L*L}{2} = \frac{105*0.60*0.60}{2} = 18.9$ t.m

Use M max = 266.96 t.m = 2669.6 KN.m

$$d = K1 \sqrt{\frac{M_U}{B}} = 0.33 * \sqrt{\frac{266.96 * 10^5}{545}} = 73.05 \text{ cm} \Rightarrow d = 80 \text{ cm} \Rightarrow t = 90 \text{ cm}$$

2.6 - Check of Shear :-

Q1 = 105 * 1.88 = 197.4 ton
Q2 = 105 (1.88 + 0.75) - 474 = -197.85 ton
Q3 = 105 (1.88 + 0.75 + 2.67) – 474 = 82.5 ton
Q 4 = 105 (1.88 + 0.75 + 0.30 + 2.67) – 474 – 177 = -63 ton

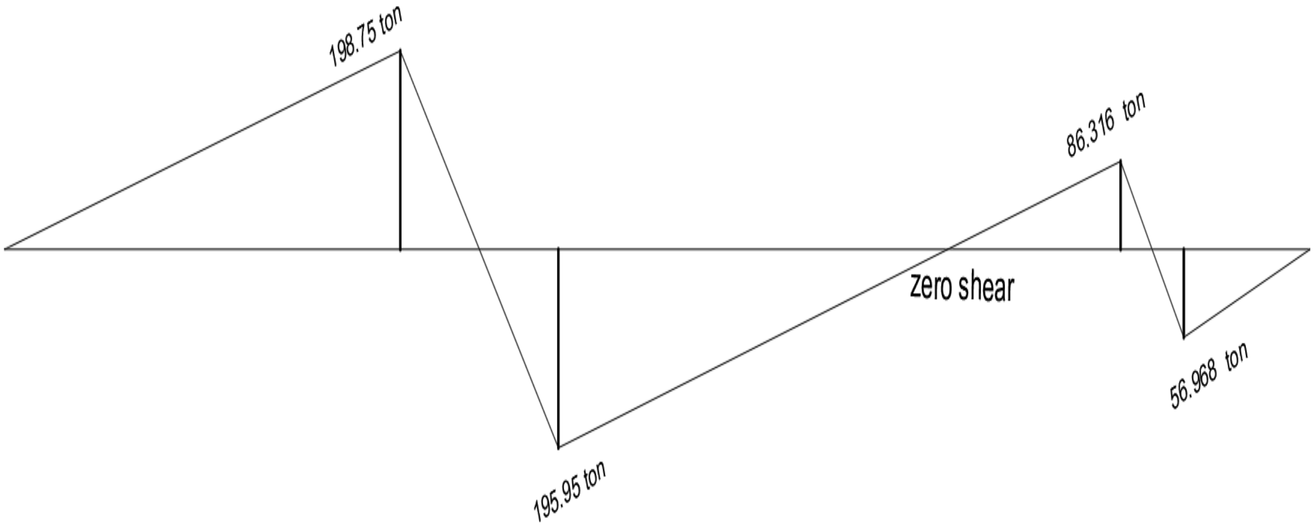


Figure 1.42. SFD

USE Q Max = 197.85 ton
OR

Q = 197.85 – $W_{UL} * \frac{d}{2}$ = 197.85 – 105 * $\frac{0.80}{2}$ = 155.85 ton = 1558.5 KN

$q_{sh} = \frac{Qsh}{B*d} = \frac{1558.4 * 10^3}{5450 * 800}$ = 0.35 KN / mm^2

$q_{sh(allow)}$ = 0.16 * $\sqrt{\frac{F_{CU}}{\gamma_C}}$ = 0.16 * $\sqrt{\frac{35}{1.5}}$ = 0.773 KN / mm^2

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

OR

$q_{sh} = \frac{Q*1000}{B*d} = \frac{155.85*1000}{545*80}$ = 3.58 Kg/cm^2 < 4.5 Kg/cm^2 , ok safe

2.7 - Check of punching :-

$q_{act} = \frac{PUL}{B\ L} = \frac{6551}{6.2*5.45}$ = 19.26 t / m^2 = 192.6 KN/m^2

Column 1

Pu = 474 ton = 4740 KN

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

- Qp = 4740 – 192.9 * (0.75 + 0.8) * (0.75 + 0.8) = 4276.5 KN
- $q_p = \frac{Qp}{2(\,(a+d)+(a+d))*d}$

$q_p = \frac{4276.5 * 10^3}{2(\ (750+800)+(750+800))*800}$ = 0.86 N/mm^2

$q_{p\ (allow)}$ = 1- 1.7 N/mm^2

2- $0.316*(.5+\frac{a}{b})*\sqrt{\frac{F_{CU}}{\gamma_C}} = 0.316*(0.5+\frac{0.3}{0.65})*\sqrt{\frac{35}{1.5}}$ = 1.46 N/mm^2

3- $0.316*\sqrt{\frac{F_{CU}}{\gamma_C}} = 0.316*\sqrt{\frac{35}{1.5}}$ = 1.53 N/mm^2

4- $0.8*(\frac{\propto*d}{2(a+d+b+d)}+.2)*\sqrt{\frac{F_{CU}}{\gamma_C}}$ =

$= 0.8\ (\frac{4*800}{2(300+800+650+800)}+0.2)*\sqrt{\frac{35}{1.5}}$ = 3.2 N/mm^2

$q_p < q_{p\ (allow)}$ ➨ OK , safe

Column 2

Pu = 177 ton = 1770 KN

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

- Qp = 1770 – 192.9 * (0.30 + 0.8) * (0.65 + 0.8) = 1462.32 KN
- $q_p = \frac{Qp}{2(\,(a+d)+(a+d))*d}$
- $q_p = \frac{1462.32 * 10^3}{2(\ (300+800)+(650+800))*800}$ = 0.358 N/mm^2

$q_{p\ (allow)}$ = 1- 1.7 N/mm^2

2- $0.316 * (0.5 + \frac{a}{b}) * \sqrt{\frac{F_{CU}}{\gamma_C}} = 0.316 * (0.5 + \frac{0.75}{0.75}) * \sqrt{\frac{35}{1.5}} = 2.289$ N/mm^2

3- $0.316 * \sqrt{\frac{F_{CU}}{\gamma_C}} = 0.316 * \sqrt{\frac{35}{1.5}} = 1.53$ N/mm^2

4- $0.8 * \left(\frac{\propto * d}{2(a+d+b+d)} + .2\right) * \sqrt{\frac{F_{CU}}{\gamma_C}} =$

$= 0.8 \left(\frac{4*800}{2(750+800+750+800)} + 0.2\right) * \sqrt{\frac{35}{1.5}} = 2.76$ N/mm^2 , $q_p < q_{p\ (allow)}$ OK, safe

2.8 – RFT

$A_S = \frac{M_U}{F_Y * J * d} = \frac{266.96 * 10^6}{360 * 0.826 * 800} = 1122.20$ mm^2 , 8 ∯ 16 / m

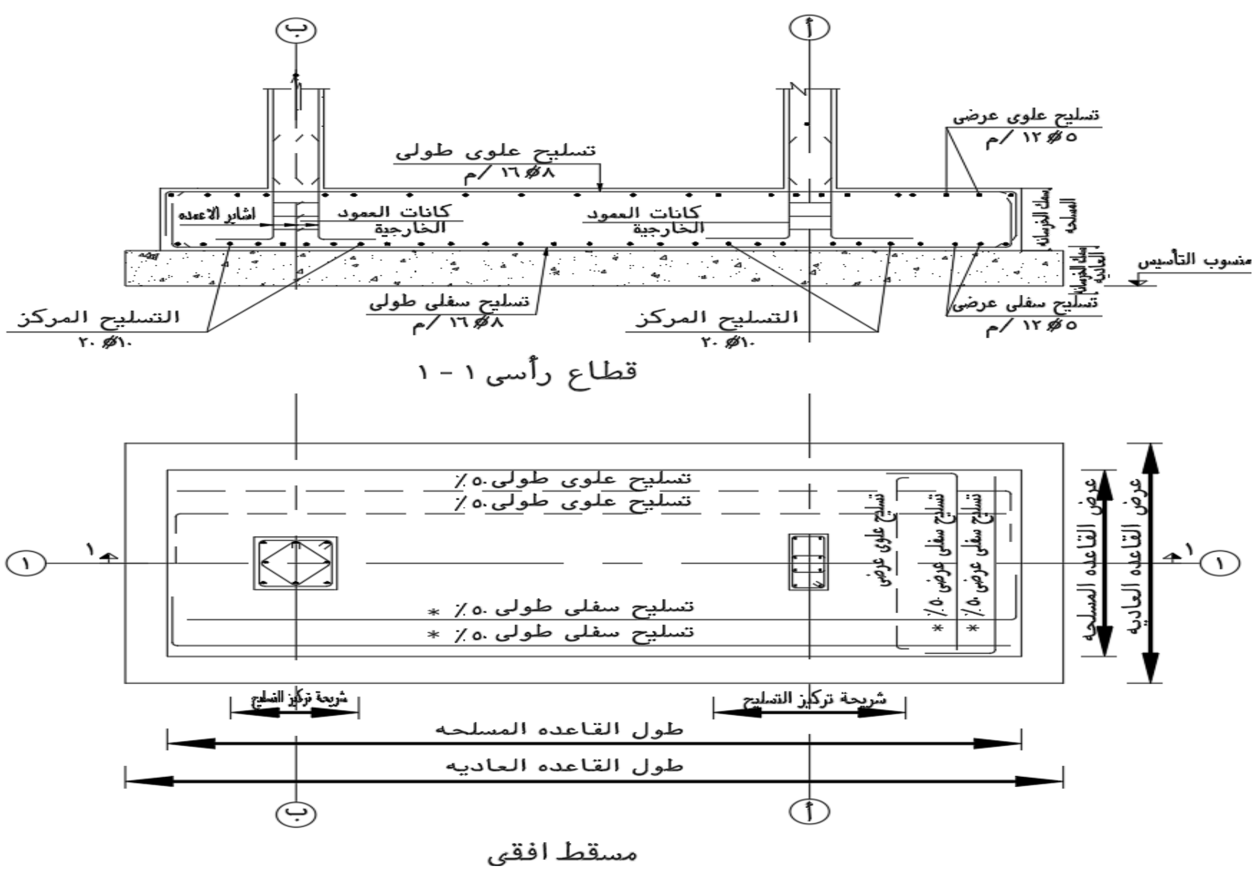


Figure 1.43. Sections of footing and details' of RFT

1.6.3 Table foundation

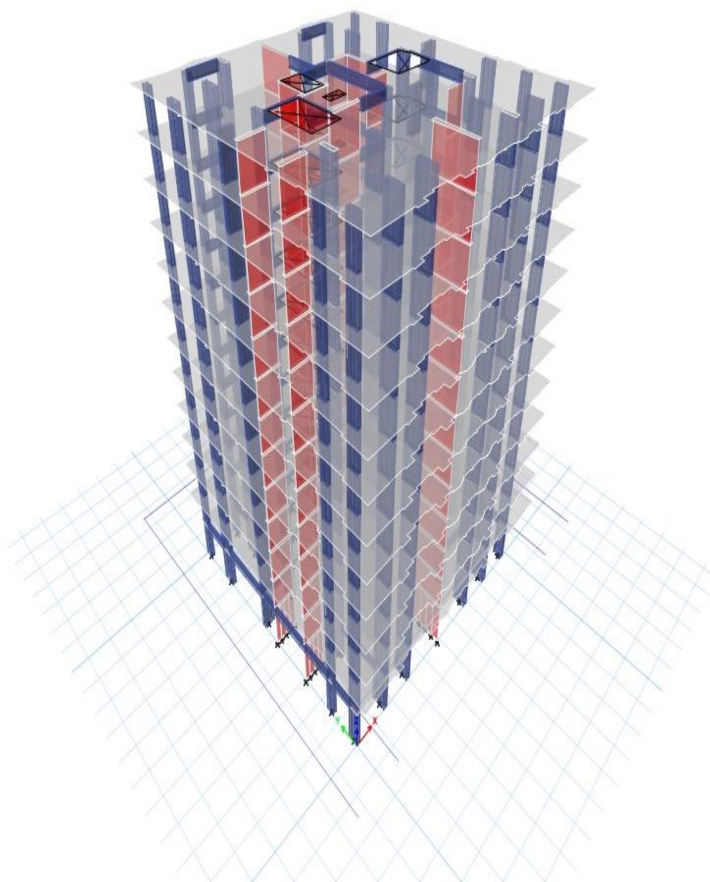
جـدول القـواعــد على اجهاد لايقل عن ١,٠٠ كجم/سم٢

النموزج	الابعاد	الابعاد	التسليح السفلى		التسليح العلوى	
	عادية	مسلحة	قصير	طويل	قصير	طويل
ق١	خرسانه عاديه بسمك = ٤٠سم	٠,٧× ٢,٤ × ٢,٧٥	٩ Ø١٢ / م	٩ Ø١٢ / م	--------	--------
ق٢		٠,٧ ×٢,٦٥ × ٣,٢٥	٥ Ø١٦ / م	٥ Ø١٦ / م	--------	--------
ق٣		٠,٧×٣,٧٥×٣,٧٥	٩ Ø١٦ / م	٩ Ø١٦ / م	--------	--------
ق٤		٠,٧×٣,٣ ×٣,٣	٧ Ø١٦ / م	٧ Ø١٦ / م	--------	--------
ق٥		٠,٧× ٣,٠ × ٣,٠	٩ Ø١٢ / م	٩ Ø١٢ / م	--------	--------
ق٦		٠,٩×٥,٤٥×٦,٢	٨ Ø١٦ / م	٨ Ø١٦ / م	٥ Ø١٢ / م	٨ Ø١٦ / م

نموذج	ابعاد (م)		التسليح السفلى		التسليح العلوى		كانات
	عرض	ارتفاع	عدل	مكسح	عدل	مكسح	
س١	٠,٣٠	٠,٧٠	٣Ø١٦	--	٣Ø١٦	--	٦Ø٨/م
س٢	٠,٣٠	٠,٧٠	٤Ø١٦	--	٤Ø١٦	--	٦Ø٨/م

Figure 1.44 . Table foundation

Unit (2) High Rise Building



2.1 INTRODUCTION

2.1.1 High Rise Building Consists of:

- Basement of (2.90) m height
- Ground Floor of (2.90) m height
- 11 Typical Floors each (2.90) m height

2.1.2 Material Properties Used:

- F_{cu}= 250 kg/cm^2
- $F_{y(main\ steel)}$=3600 kg/cm^2
- $F_{y(stirrups)}$=2400 kg/cm^2
- Weight of used brick =1500 kg/m^3
- Bearing Capacity of Soil = 10 kg/cm^2

2.1.3 Cover Thickness

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

2.1.4 Loads Used:

- L.L= According to every Floor
- Cover = 0.15 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 2020

2.2 DESIGN OF SLABS:

2.2.1 Basement Slab: (Flat Slab System)

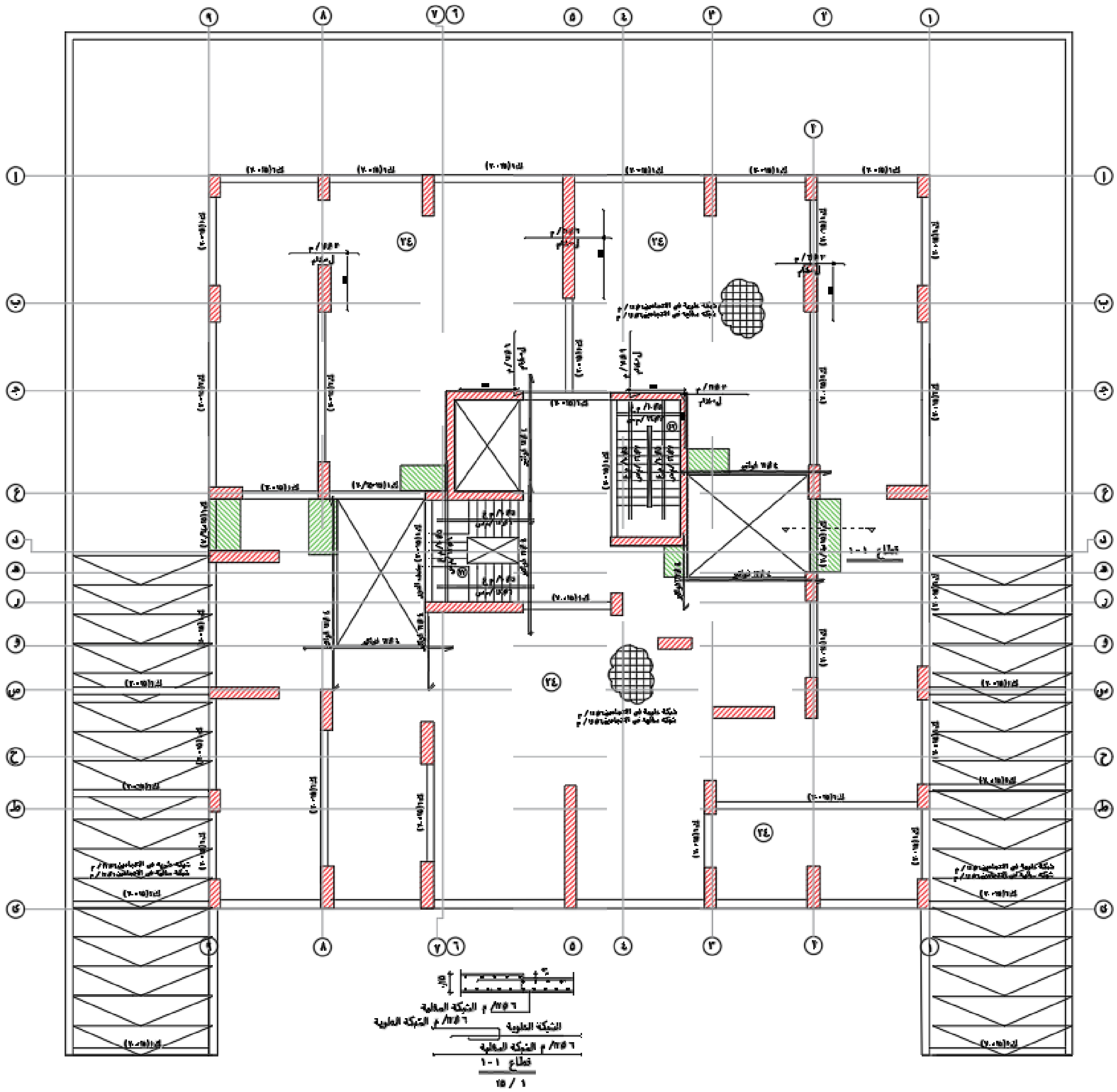


Figure 2.1 Statical System of Basement Roof

- Slab Thickness = 24 cm
- Own weight = 0.24* 2.5 = 0.6 t/m^2.
- Covering = 150 kg/m^2 = 0.15 t/m^2.
- Live load = 300 kg/m^2 = 0.3 t/m^2
- Wall load = 300 kg/m^2 = 0.3 t/m^2.

Solving This flat slab By Using CSI Safe program:

- D.L = O.W +W_{wall} + Covering material
  = 0.6 + 0.3 + 0.15 = 1.05 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 (1.05) + (1.6 * 0.3) = 1.95 t/m^2

For ultimate design:-

- $As = \left[\frac{Mu}{Fy*J*d}\right]$
- M_u = As * F_y *J *d = $6*\left(\frac{\pi*(1.2)^2}{4}\right)*3600*0.85*23*(10)^{-5}$
- M(r) = 4.775 t.m ⇛ Use 6 ∯ 12 / m in each Direction
- Additional RFT (3 ∯ 12 /m) and (6 ∯ 12 / m) upper and lower

In X-Direction: (Upper and Lower) :

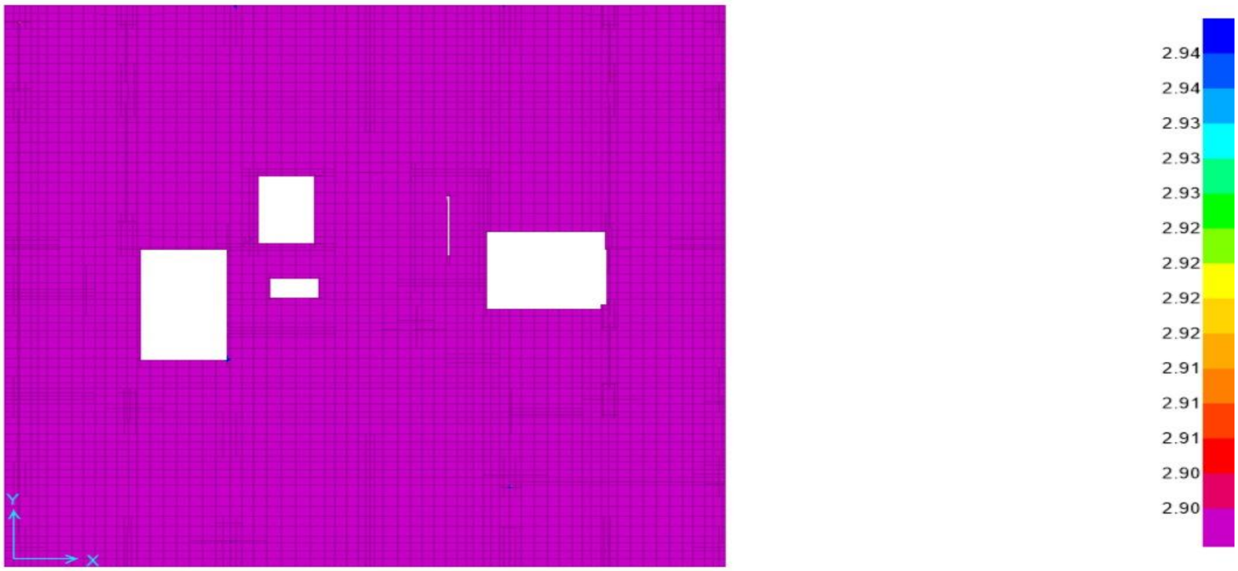


Figure 2.2 Additional Reinforcement in X-Direction (Upper and Lower)

In Y-Direction (Upper and Lower)

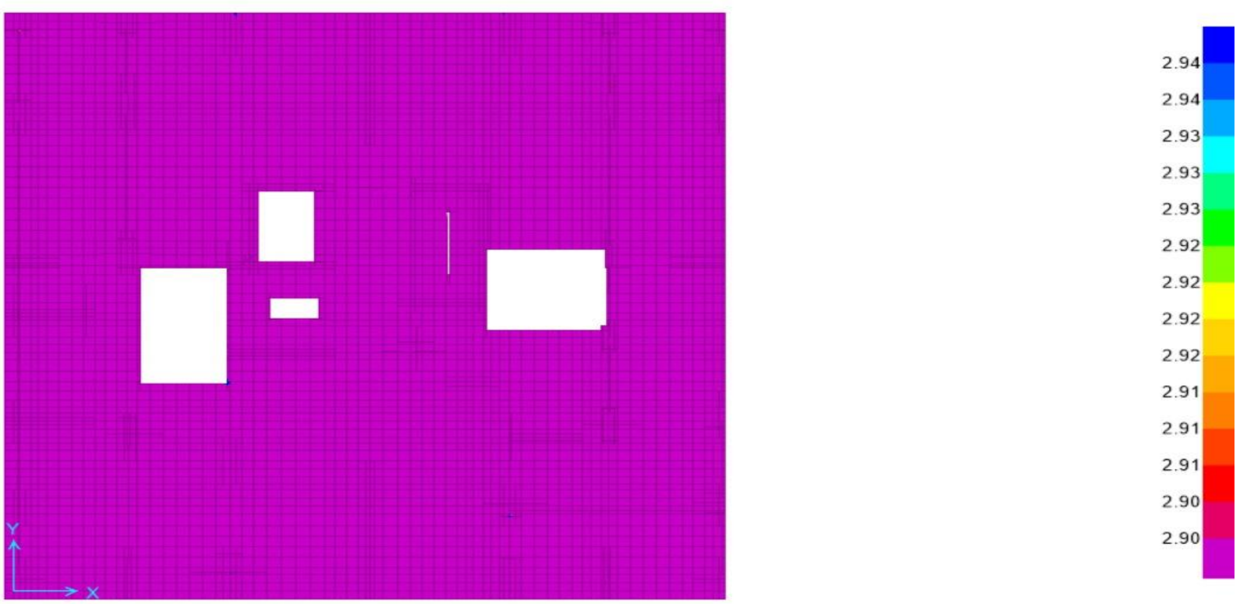


Figure 2.3 Additional Reinforcement in Y-Direction (Upper and Lower)

2.2.1.1 Check for Long Term Deflection:

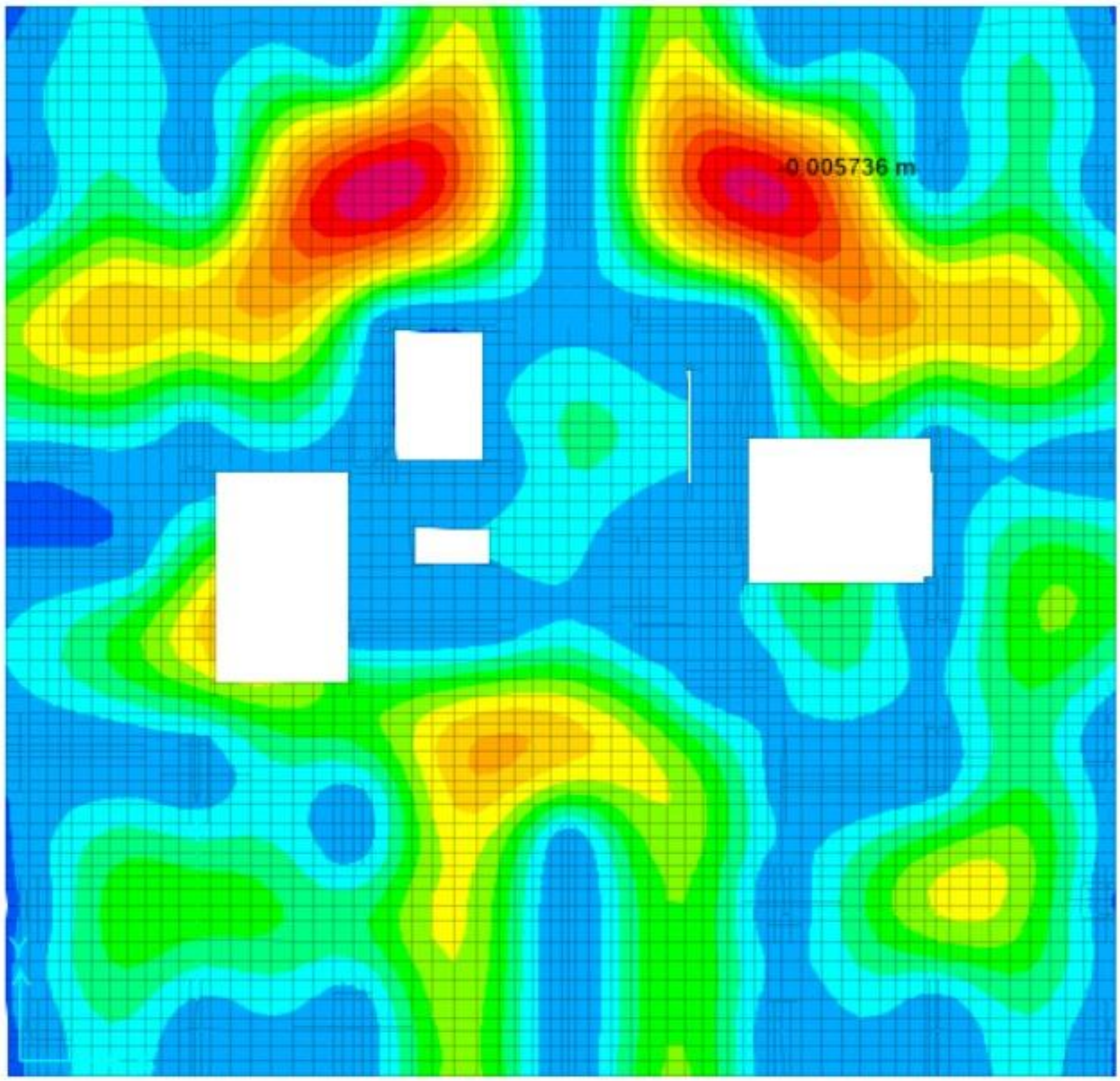


Figure2.4 Long Term Deflection

- From Code Check = L/250
- Span for Check = 4.32 m
- Allowable Deflection = 0.017 m
- Maximum Deflection = - 0.00573

2.2.2 Ground Slab & Rebated (Flat Slab System)

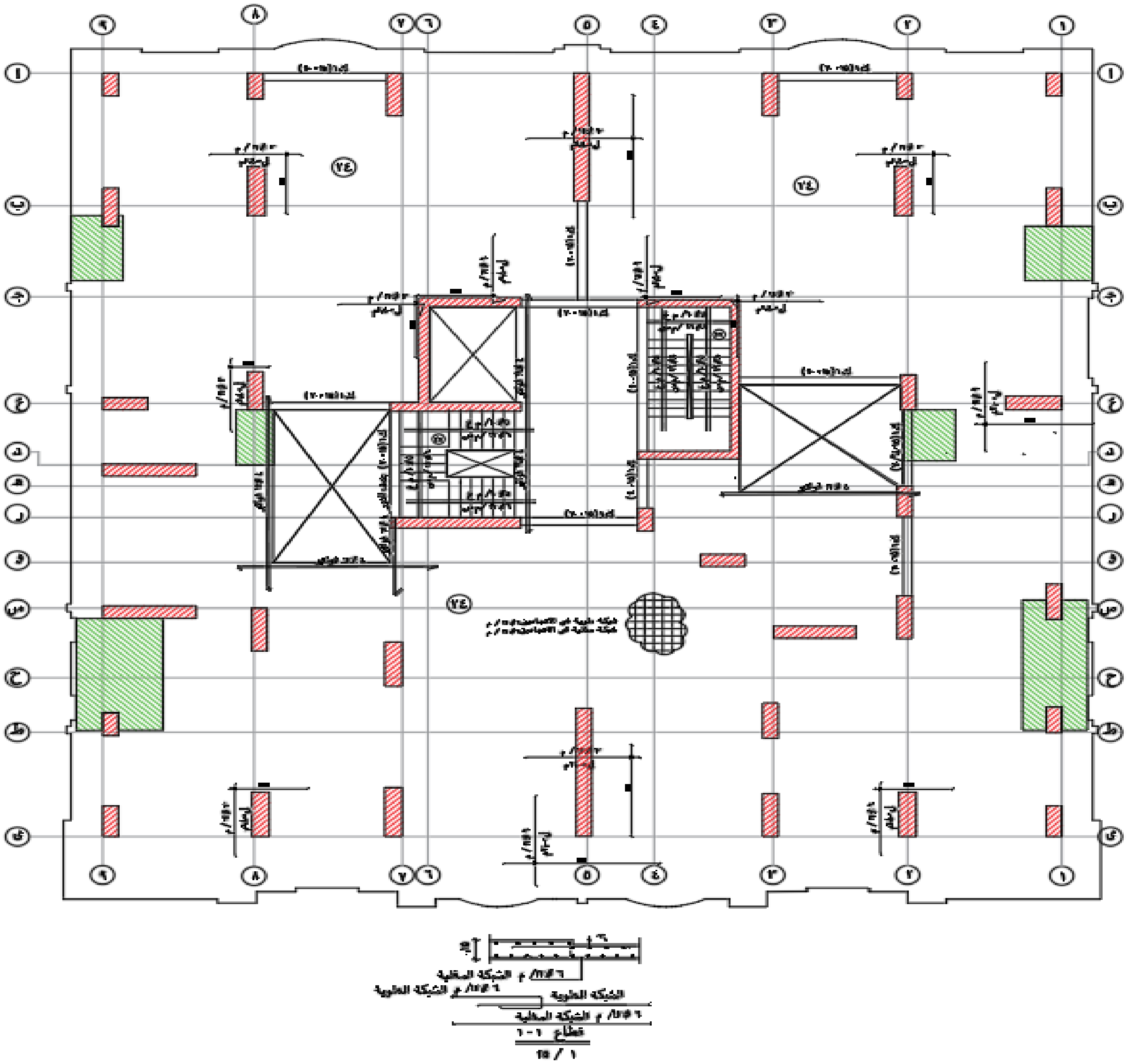


Figure 2.5 Statical System of Ground Roof

- Slab Thickness = 24 cm
- Own weight = 0.24* 2.5 = 0.6 t/m^2.
- Covering = 150 kg/m^2 = 0.15 t/m^2.
- Live load = 300 kg/m^2 = 0.3 t/m^2
- Wall load = 300 kg/m^2 = 0.3 t/m^2.

Solving This flat slab By Using CSI Safe program:

- D.L = O.W + W_{wall} + Covering material
  = 0.6 + 0.3 + 0.15 = 1.05 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 (1.05) + (1.6 * 0.3) = 1.95 t/m^2

For ultimate design:-

- $As = \left[\frac{Mu}{Fy*J*d}\right]$
- $M_u = As * F_y * J * d = 6*\left(\frac{\pi*(1.2)^2}{4}\right) * 3600 * 0.85 * 23 * (10)^{-5}$
- M(r)= 4.775 t.m ⇛ Use 6 ∯ 12 / m in each Direction
- Additional RFT (3 ∯ 12 /m) & (6 ∯ 12 / m) upper and lower

In X-Direction (Upper and Lower) :

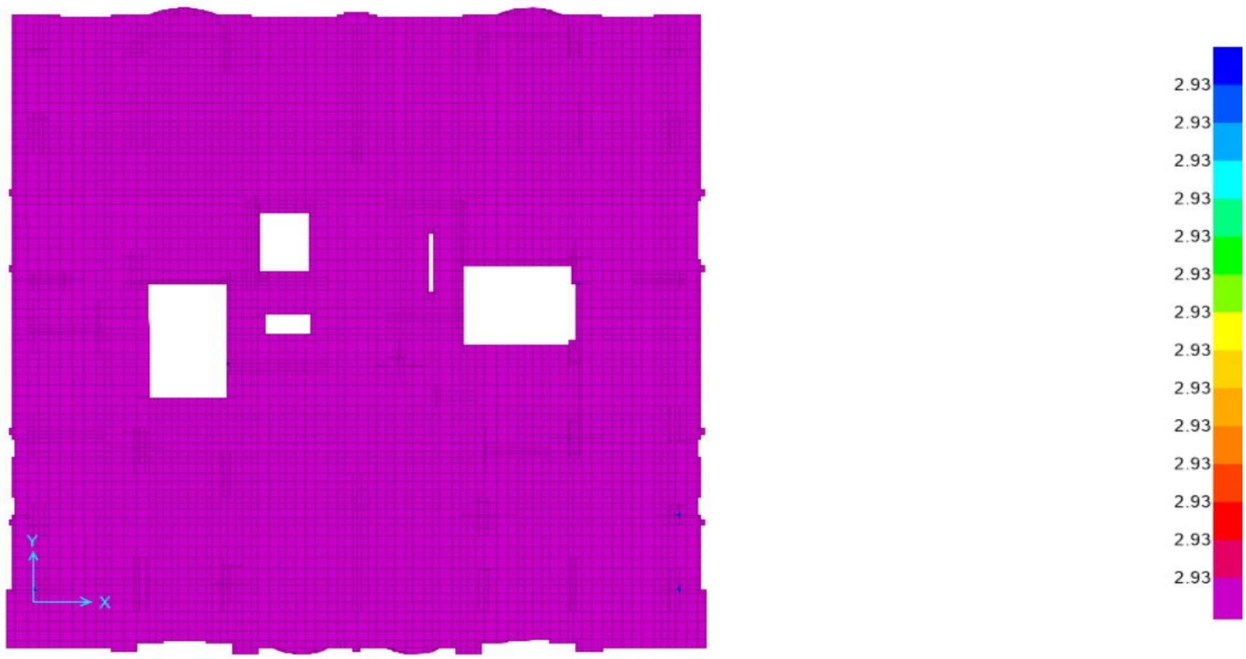


Figure 2.6 Additional Reinforcement in X-Direction (Upper and Lower)

In Y-Direction (Upper and Lower)

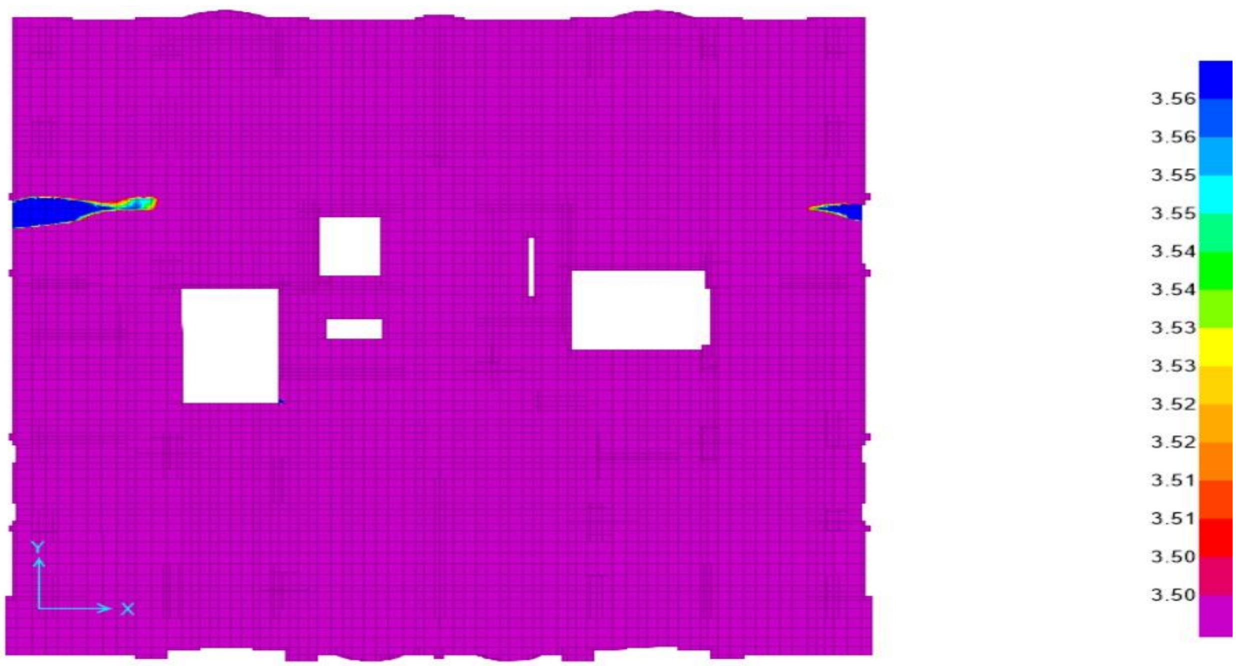


Figure2.7 Additional Reinforcement in Y-Direction (Upper and Lower)

2.2.2.1 Check for Long Term Deflection:

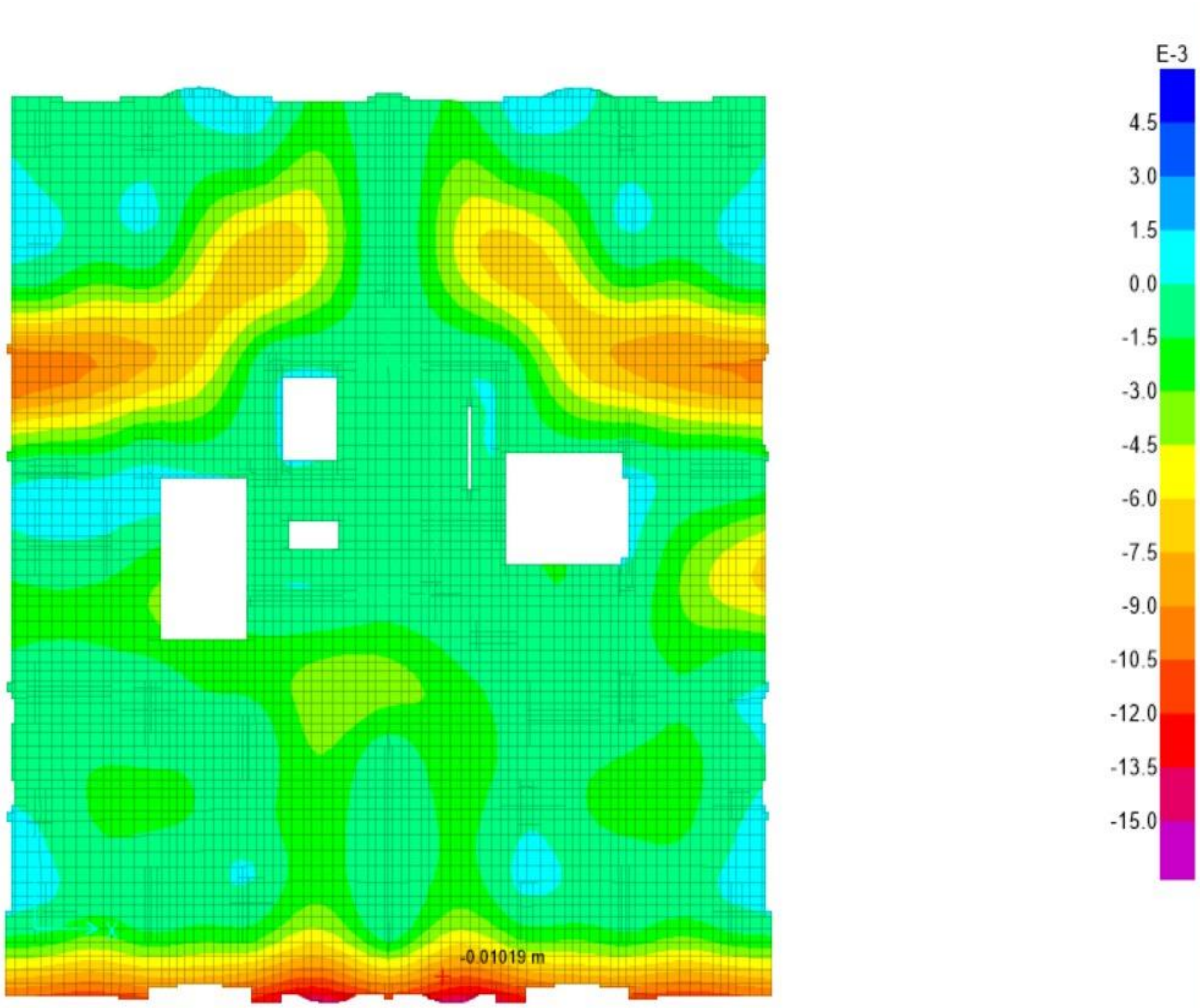


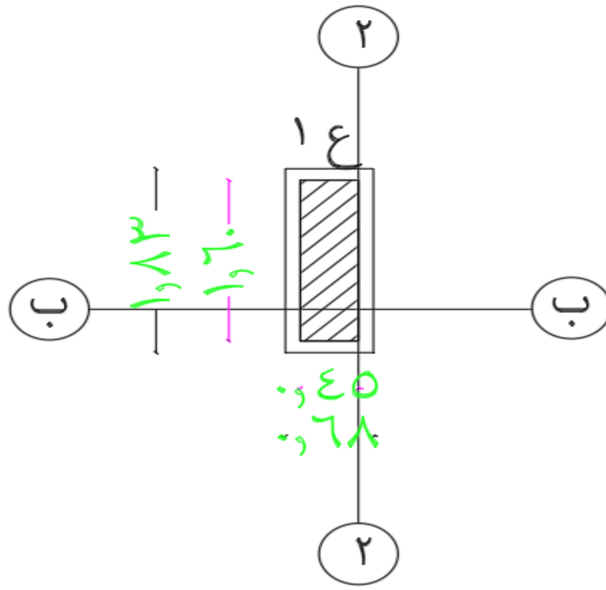
Figure 2.8 Long Term Deflection

- From Code Check = L/400 **for cantiliver**
- From Code Check = L/250
- Span for Check = 2.05 m **for cantiliver**
- Span for Check = 2 m
- Allowable Deflection =0.01812 m
- Allowable Deflection = 0.005 m **for cantiliver**
- Maximum Deflection = -0.01019 m

Ok use camper At cantilever **1 cm** each **1 m**

2.2.3 Check of Punching Shear: (Basement Roof)

2.2.3.1 Interior Column (C1 = 45*160) on (2 – ب) Axis:



- Slab Thickness = 24 cm
- Own weight = 0.24* 2.5 = 0.6 t/m^2.
- Covering = 150 kg/m^2 = 0.15 t/m^2.
- Live load = 300 kg/m^2 = 0.3 t/m^2
- Wall load = 300 kg/m^2 = 0.3 t/m^2.

Solving This flat slab

- D.L = O.W +W_{wall} + Covering material
  = 0.6 + 0.3 + 0.15 = 1.05 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 (1.05) + (1.6 * 0.3) = 1.95 t/m^2 = 19.5 $KN/m2$
-
- $d = t_s -$ 20 mm = 250 – 20 = 230 mm = 0.23 m
- b_o = 2 * (1830 + 680) = 5020 mm
- Q_{up}= Wu (L1 * L2 – Ap) = 1.95 (2.2 * 1.8 – 1.83 * 0.68) = 5.295 t/m^2
- $q_{up} = \frac{Q_{up}}{bo*d} * \beta = \frac{52.95 * 1000}{5020 * 230} * 1.15 = 0.052\ N/mm^2$
- q_{cup} $= the\ least\ of$:-
  - 1.70 N/mm^2
  - $0.316 \sqrt{\frac{f_{cu}}{\gamma_c}} = 0.316 \sqrt{\frac{30}{1.5}} = 1.41 N/mm^2$
  - $0.316(\frac{a}{b} + 0.5)\sqrt{\frac{f_{cu}}{\gamma_c}} = 0.316\ (\frac{50}{160} + 0.5) \sqrt{\frac{30}{1.5}} = 1.148\ N/mm^2$
  - $0.8\ (\frac{\alpha * d}{b0} + 0.2)\sqrt{\frac{f_{cu}}{\gamma_c}} = 0.8\ (\frac{4*230}{5020} + 0.2) \sqrt{\frac{30}{1.5}} = 1.371\ N/mm^2$
- $q_{up} = 0.052\ N/mm^2 \leq q_{cup} = 1.148\ N/mm^2$

OK safe punching

2.4 Design of Stairs Three Flight Stair Axis (ع – 5)

2.4.1 Manual solution

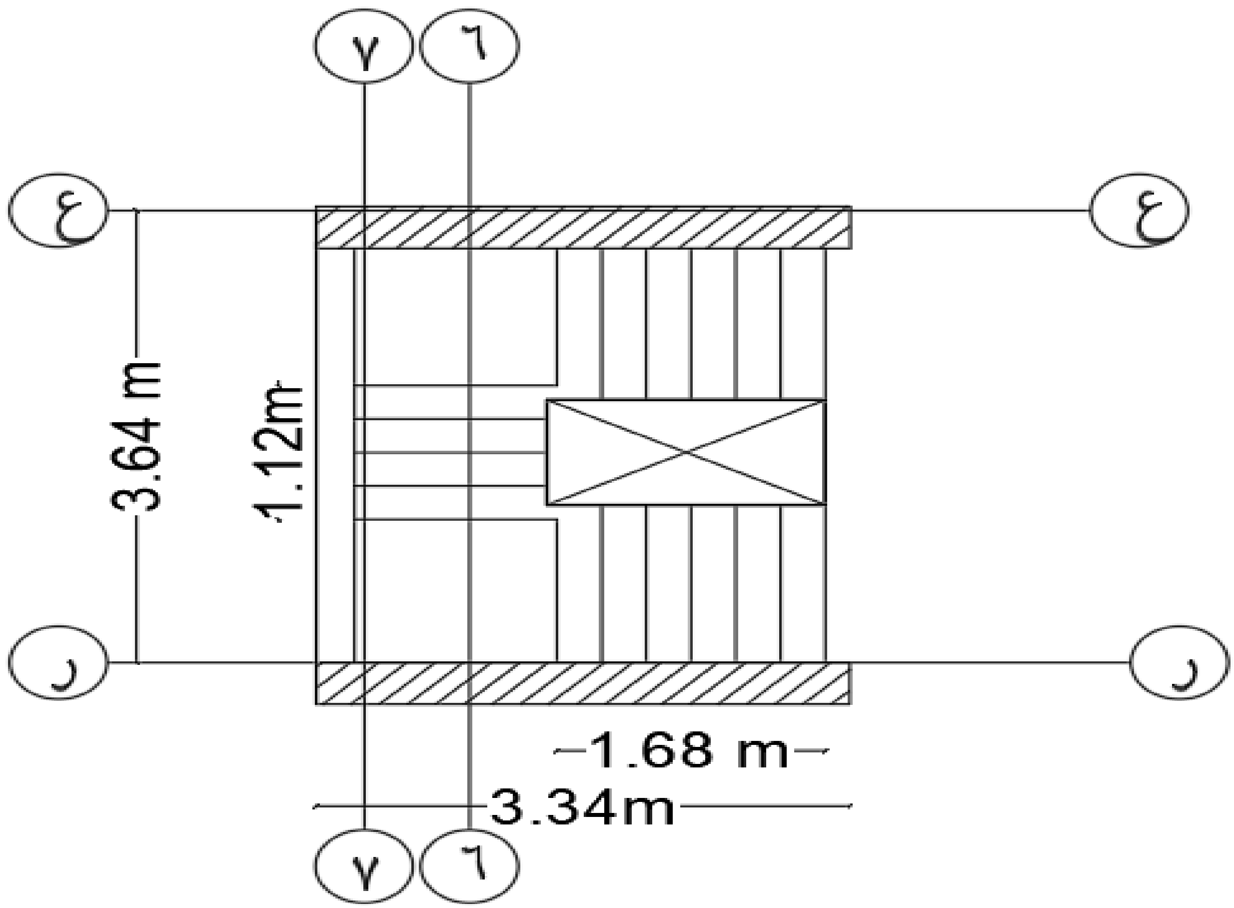


Figure 2.9 Stair Cross Section

1) Dimensions:

- $ts = \frac{Span}{24:30} = 0.14\ m$
- $H_{Story} = 2.90m$
- $Rise = 0.15\ m$
- $Going = 0.30\ m$
- $\theta = \tan^{-1}(\frac{0.15}{0.30}) = 26.56\ °$
- $t^* = \frac{ts}{Cos\theta} = \frac{14}{Cos(26.56)} = 15.6\ cm$
- $t_{av} = t^* + \frac{Rise}{2} = 23.1$ cm

2) Loads:

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

  = 1.4 (25 * 0.231 + 1.0) + 1.6 (3)

  = 14.30 KN/m^2

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

  = 1.4 (25 * 0.14 + 1.5) + 1.6 (3)

  = 11.80 KN/m^2

3) For Strips

Shown In The figure

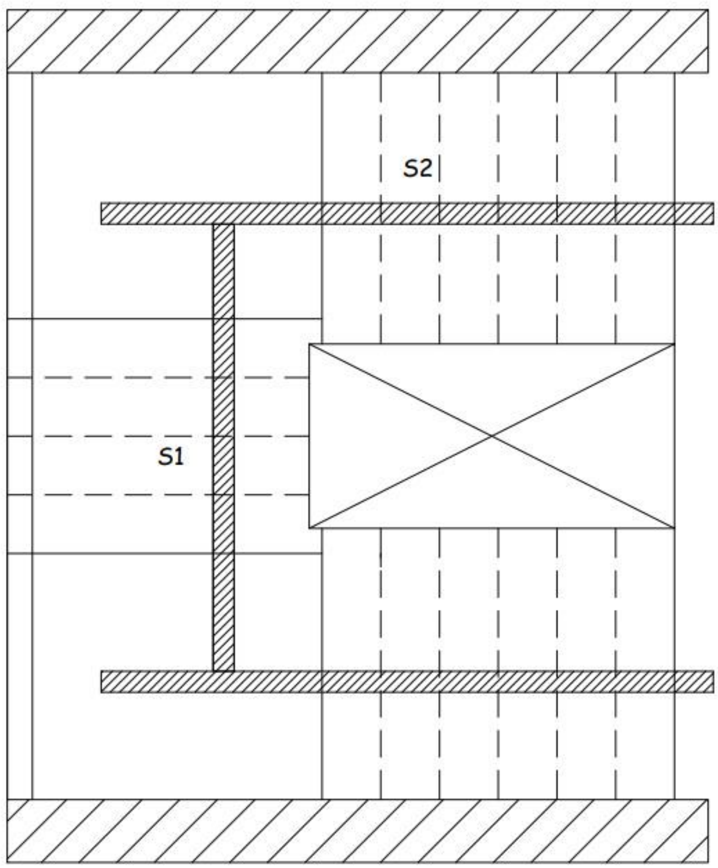


Figure 2.10 Strips Of Stair

For Strip (F1):

- R_1 = 8.01 KN
- M_{u1} = 8.01*1.15 -14.30*0.56^2/2= 6.69 KN.m

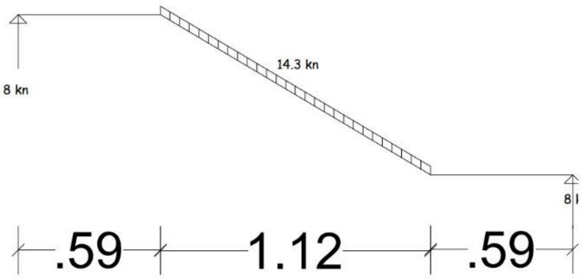


Figure 2.11 Strip (F1) of Stair

For Strip (F2):

- R_2 *3.93 =11.8*1.5*(.75+1.68+.75) +10.17*1.5*(.75+1.68+.75) + (14.3 * 1.68*1.59)

  R_2 = 36.38 Kn

- R_2 *3.93=(10.17*1.5*.75) + (11.8*1.5*.75) + (14.3*1.68*1.59)

  R_3 = 20.6Kn

- M_{u2} =36.38*1.5-10.17*1.5*.75+11.8*1.5*.75 = 29.85k.m
- M_{u3} =20.6*0.75= 15.45Kn.m
- M_{u4} = $\frac{29.85+15.45}{2}$ = 22.65

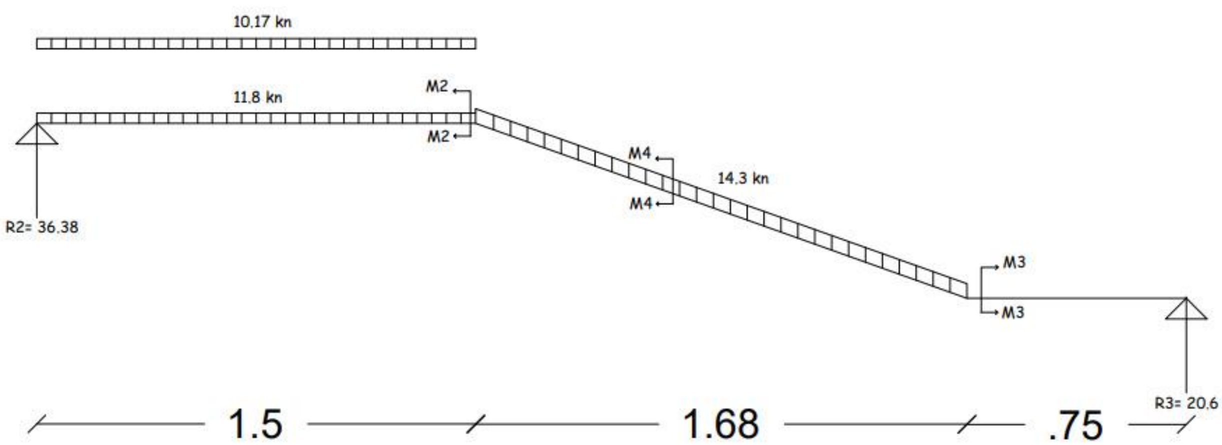


Figure 2.12 Strip (F2) of Stair

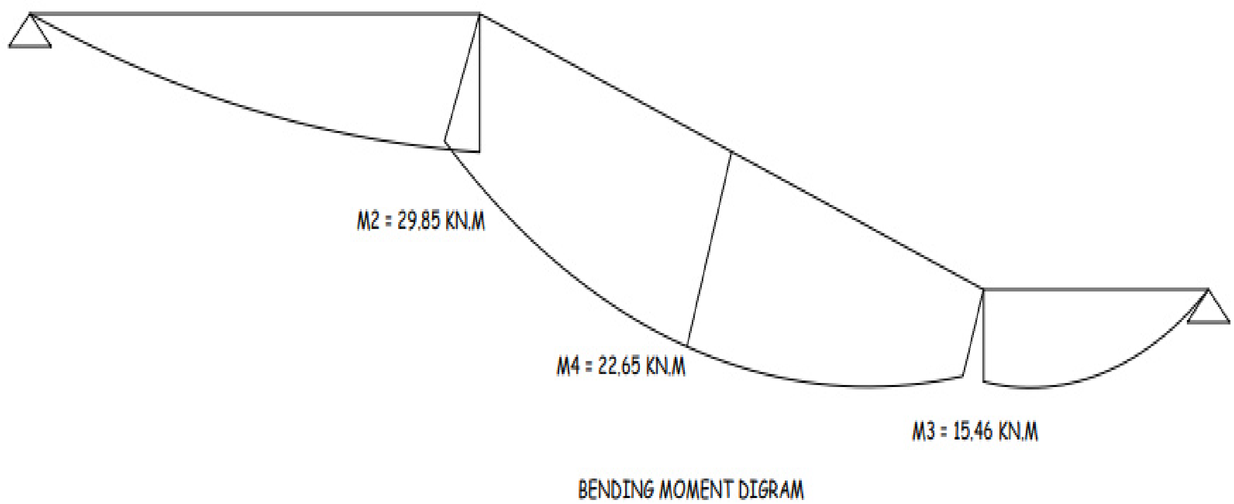


Figure 2.13 BMD

4) Design of Section of Flight :

- M_{u4} = 22.65 KN.m
- $d = c_1\sqrt{\frac{MU}{FCU*b}} \Rightarrow 120 = c_1\sqrt{\frac{22.65*10^6}{30*1000}} \Rightarrow c1 = 4.36 \Rightarrow J = .826$
- $A_s = \frac{M_u}{F_{y*} j*d} = \frac{22.65*10^6}{350*.826*120} = 652.88\ mm^2$
  - Use A_S ⇛ 6 ∯ 12 / m

4) Design of Section of STRIP 1 :

$A_s = \frac{M_u}{F_{y*} j*d} = \frac{6.69*10^6}{350*.826*120} = 192.83\ mm^2$

5 ∯ 12 / m

2.3.2 Using Sap Program

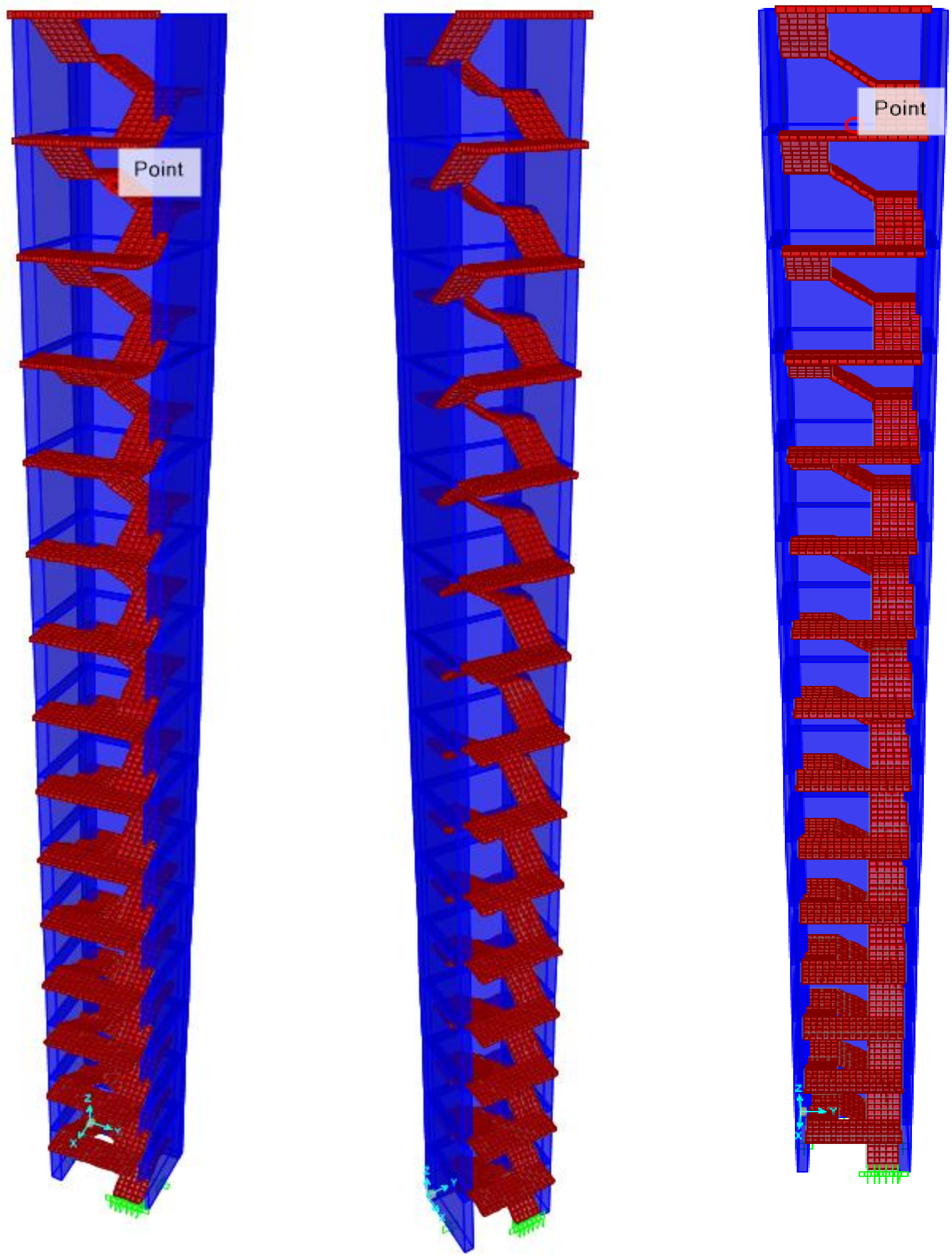


Figure 2.14 Stair 3D

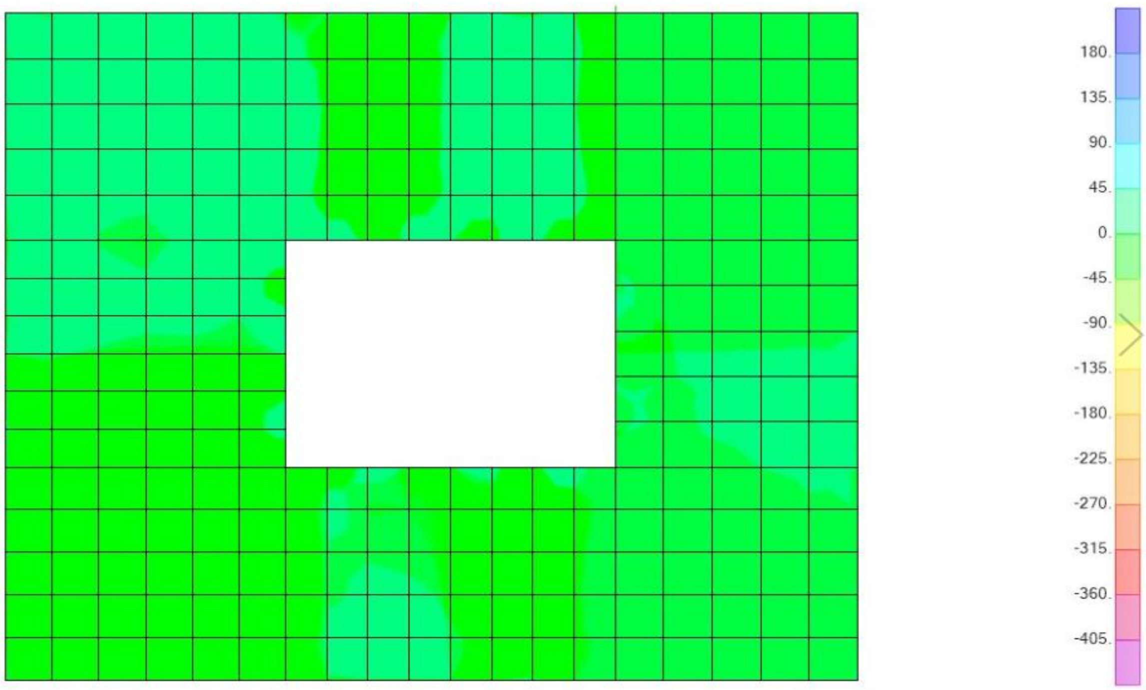


Figure 2.15 Bending Moment In X-Direction

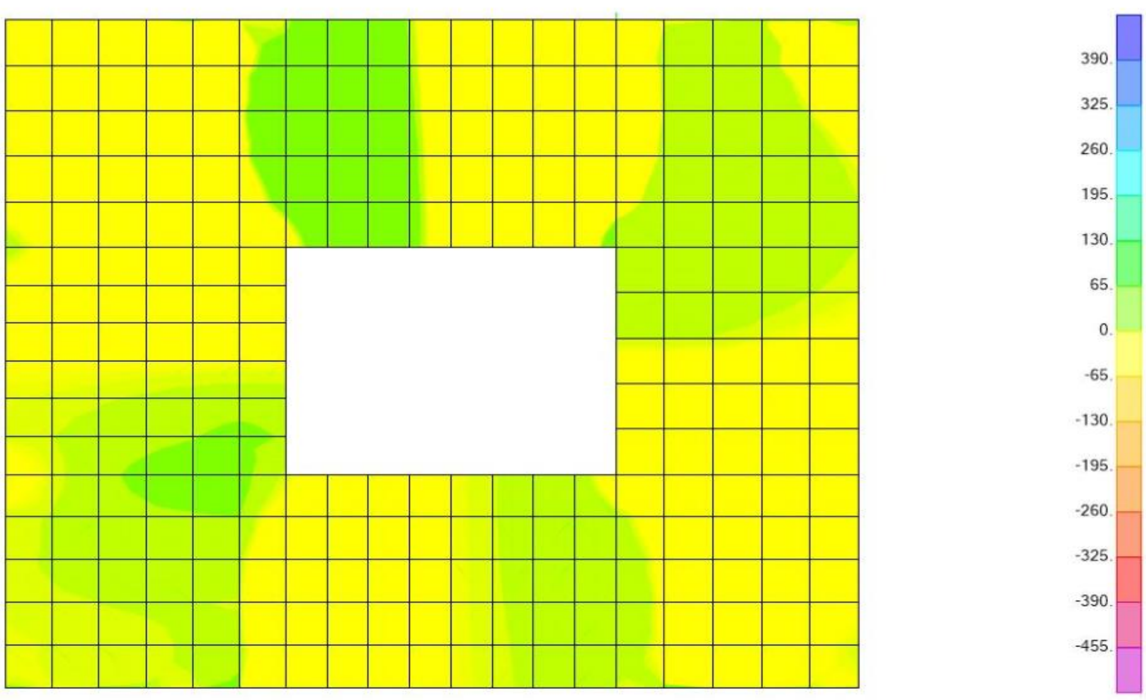


Figure 2.16 Bending Moment In Y-Direction

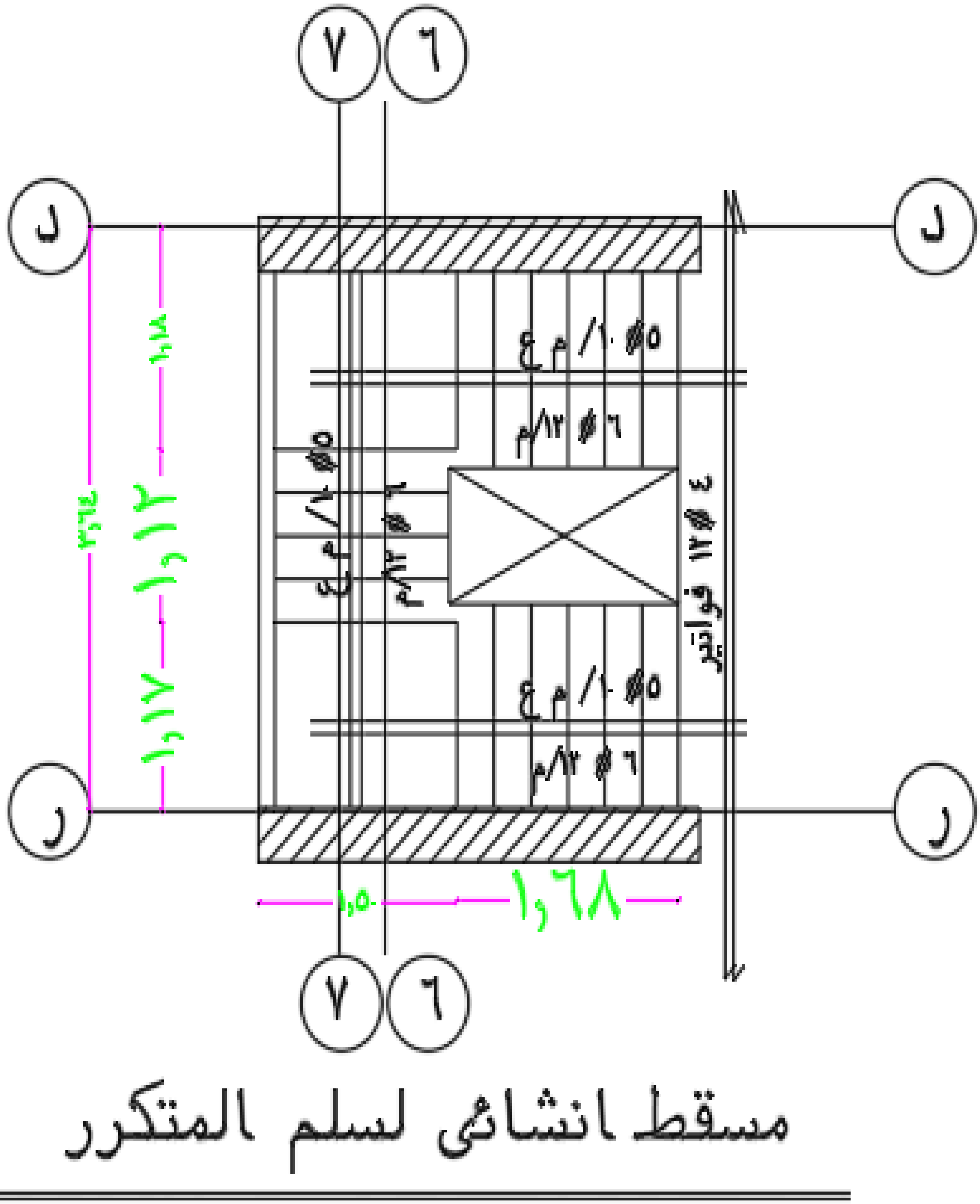


Figure 2.17 Stair Reinforcement in plan

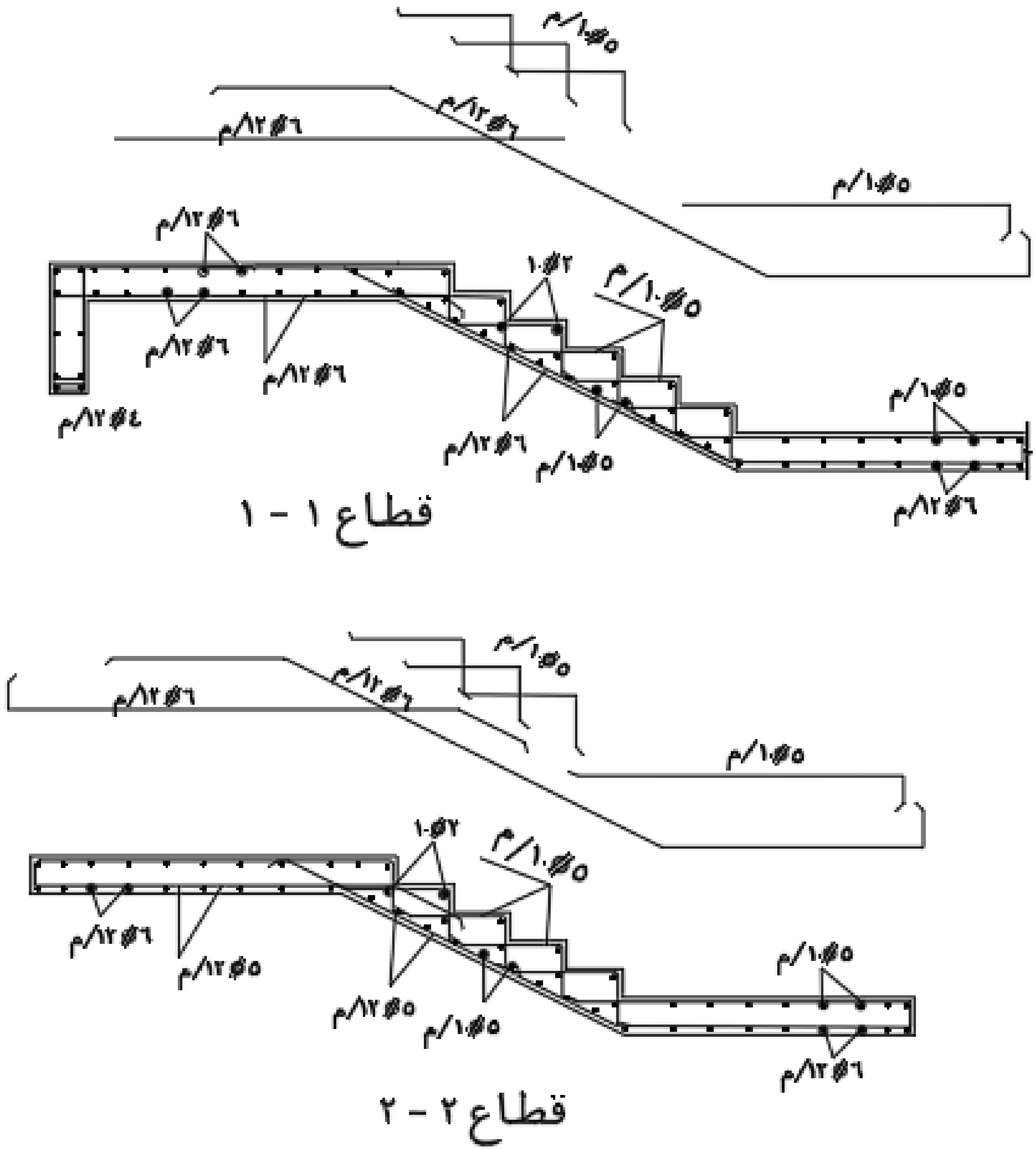


Figure 2.18 Reinforcement in sec1-1 and sec 2-2

2.4 Design of Stairs (Two Flight Stair Axis 3- ع)

2.4.2 Manual solution

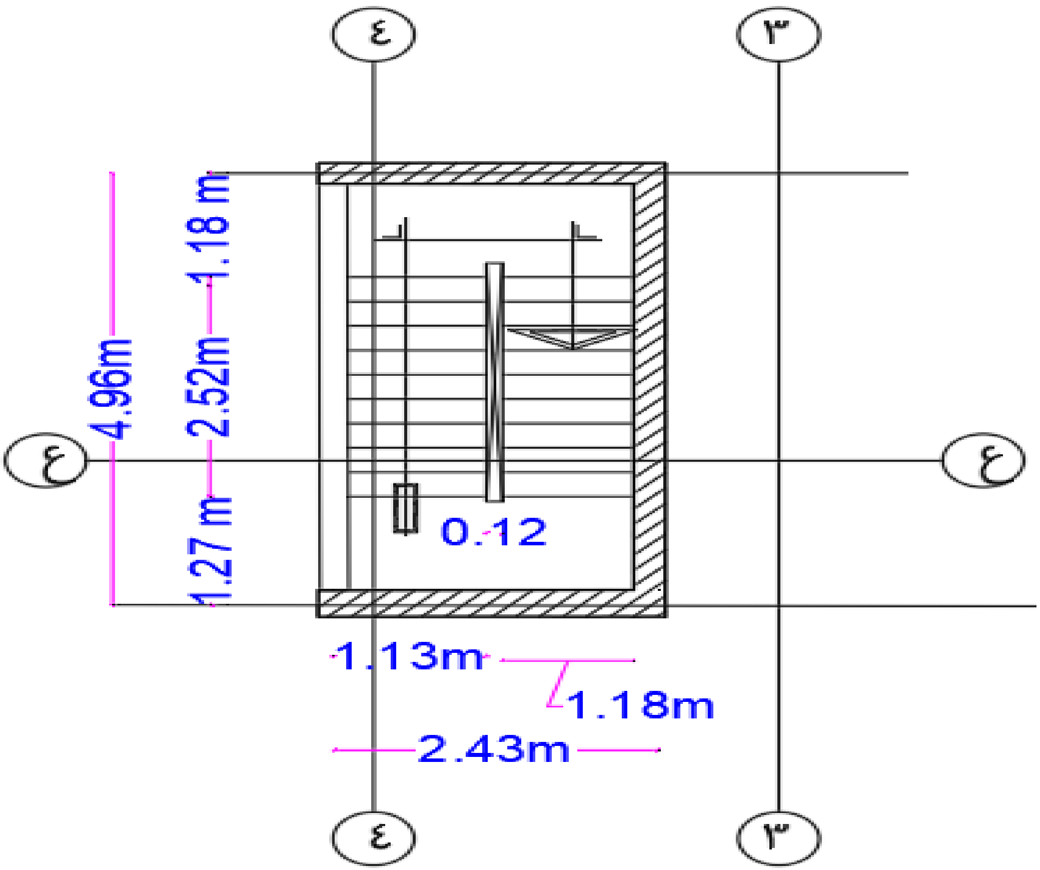


Figure 2.19 Stair Cross Section

1) Dimensions:

- $ts = \frac{Span}{24:30} = 0.22\ m$
- $H_{Story} = 2.90m$
- $Rise = 0.15\ m$
- $Going = 0.30\ m$
- $\theta = \tan^{-1}(\frac{0.15}{0.30}) = 26.56\ ^\circ$
- $t^* = \frac{ts}{Cos\theta} = \frac{16}{Cos(26.56)} = 17.88\ cm$
- $t_{av} = t^* + \frac{Rise}{2} = 25.38$ cm

2) Loads:

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

  = 1.4 (25*0.2538 + 1.0) + 1.6(3)

  = 15 KN/m^2

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

  = 1.4 (25 * 0.16 + 1.5) + 1.6 (3)

  = 12.5KN/m^2

3) For Strips

Shown In The figure

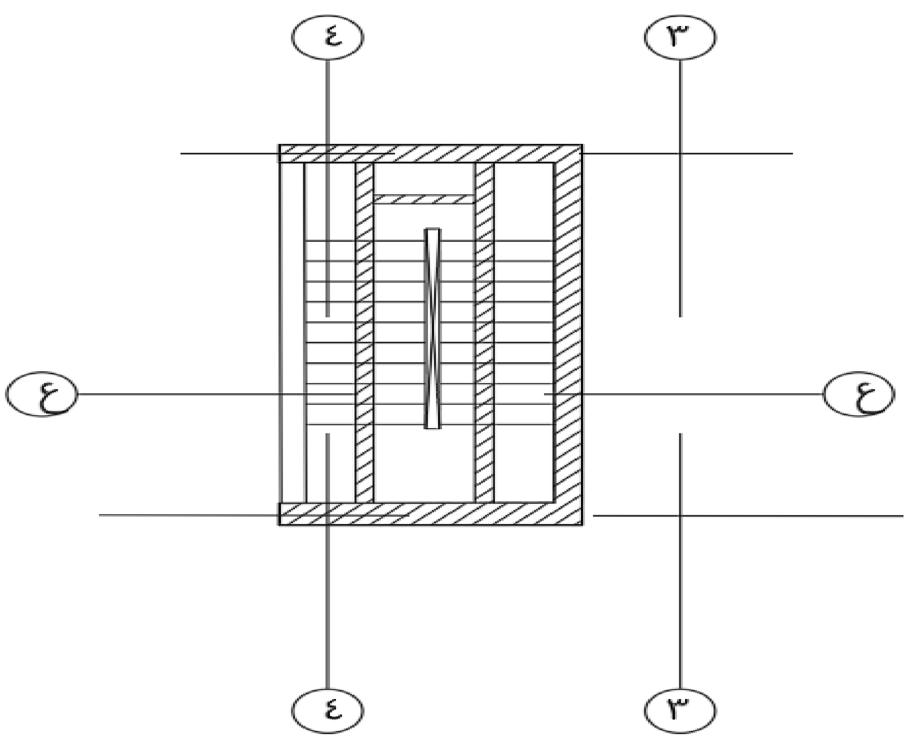


Figure 2.20 Strips Of Stair

For Strip (F1):

- $R_1 = 12.5 * \frac{1.25}{2} * 2 + 15 * \frac{2.52}{2} = 34.52$ KN
- $M_{u1} = 34.52 * 2.52 - 15*1.26^2/2 - 12.5*1.25*1.885$
  $= 45.28$ KN.m

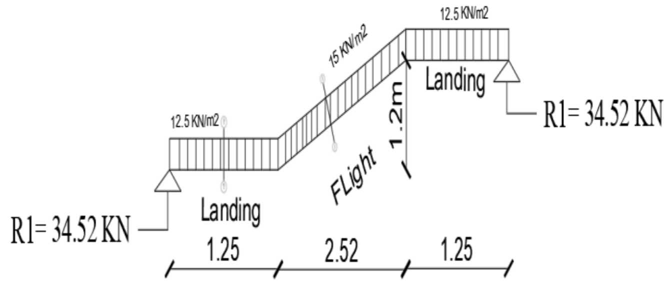


Figure 2.21 Strip (F1) of Stair

Moment :-

. M1 = M3

$R1 * 1.25 - 12.5 * \frac{1.25*1.25}{2} = 34.52 * 1.25 - 12.5 * \frac{1.25*1.25}{2} = 33.38$ KN.m

. M2 = M4

$R1\ (1.25 + 2.52) - 12.5 * 1.25\ (\frac{1.25}{2} + 2.52) - 15 * 2.52 * \frac{2.52}{2}$

$= 34.52\ (3.77) - 12.5 * 1.25\ (\frac{1.25}{2} + 2.52) - 15 * 2.52 * \frac{2.52}{2} = 33.37$ KN.m

. Mu1

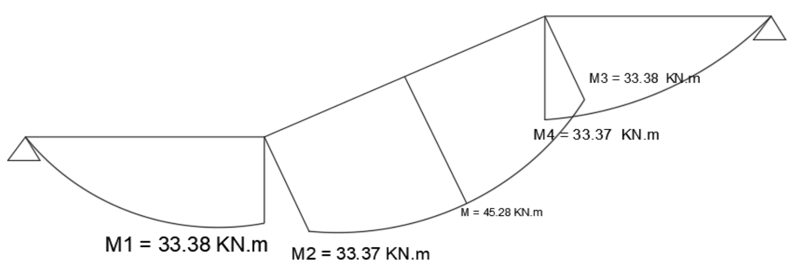
$M_{u1} = 34.52 * 2.52 - 15*1.26^2/2 - 12.5*1.25*1.885 = 45.28$ KN.m

4) Design of Section of Flight :

- $M_{u1} = 45.28$ KN.m
- $d = c_1 \sqrt{\frac{MU}{FCU*b}} \Rightarrow 140 = c_1 \sqrt{\frac{45.28*10^6}{30*1000}}$
- $c1 = 3.60 \Rightarrow J = 0.786$

$A_s = \frac{M_u}{F_y * j * d} = \frac{45.28*10^6}{360*0.786*140} = 1156.14\ mm^2$

- Use A_S ⇒ 8 ∯ 16 / m



2.4.3 Using Sap Program

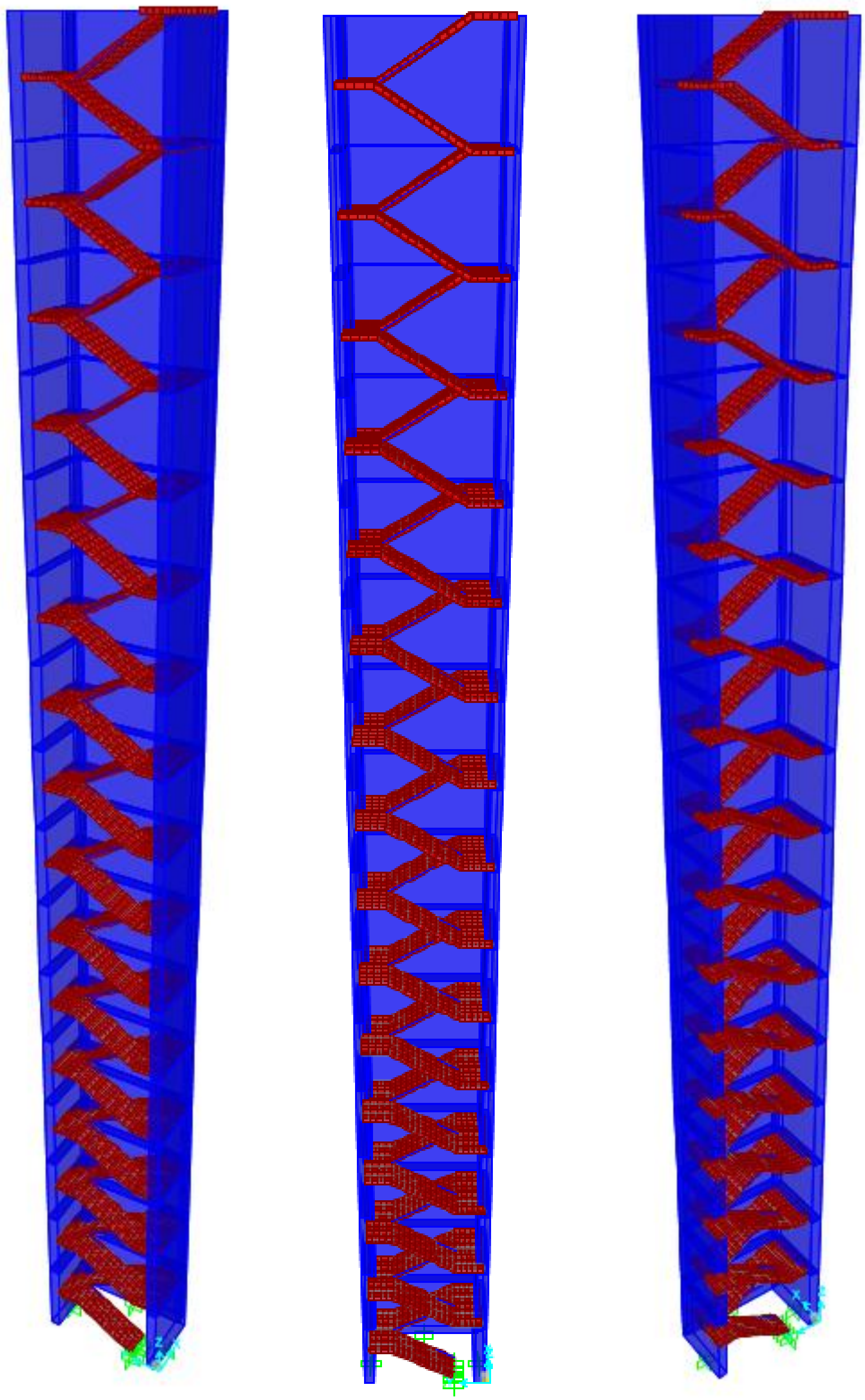


Figure 2.22 Stair 3D

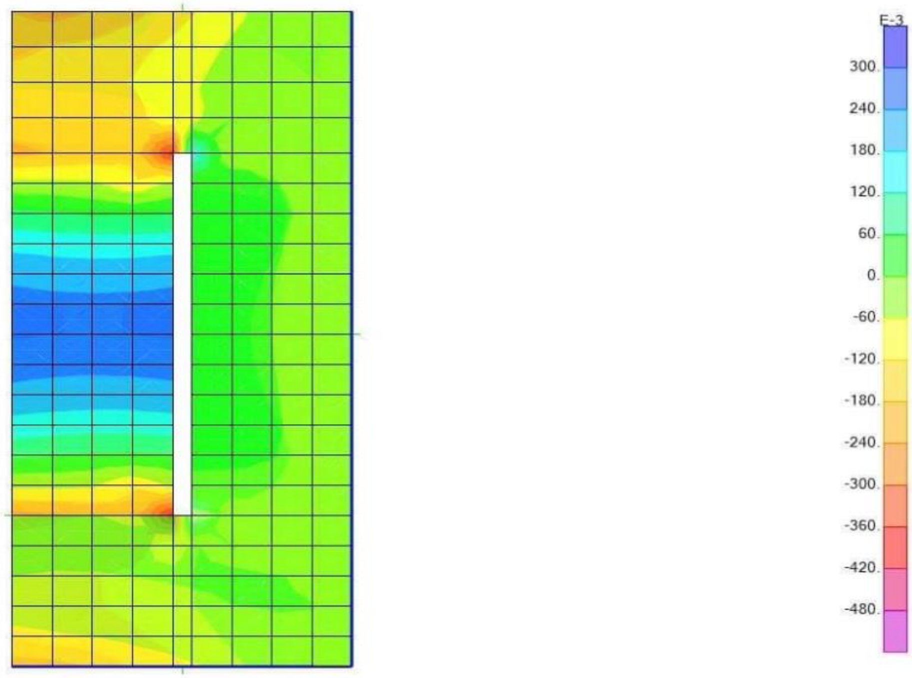


Figure 2.23 Bending Moment In X-Direction

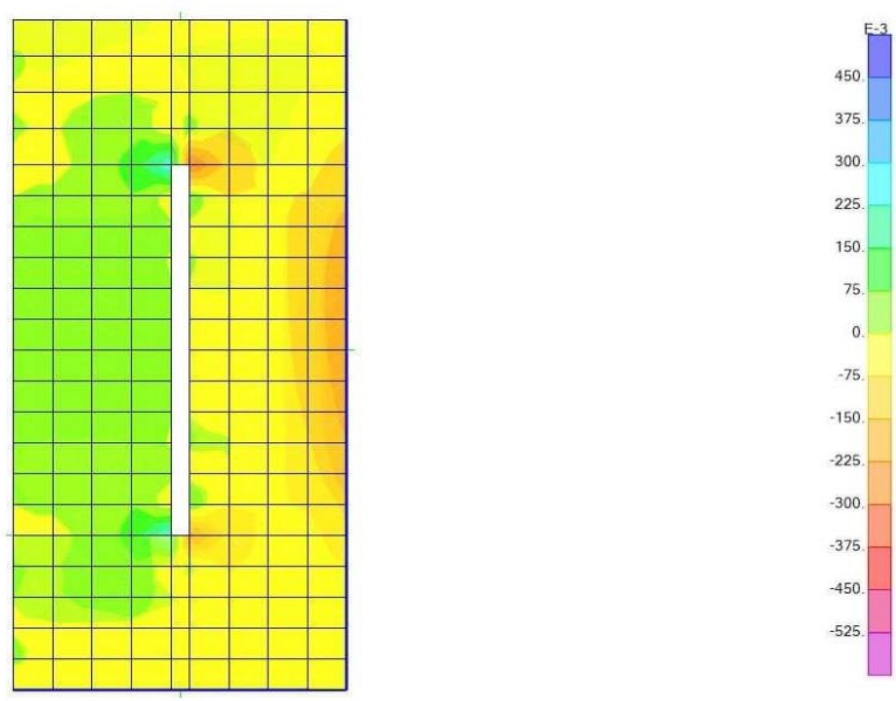


Figure 2.24 Bending Moment In Y-Direction

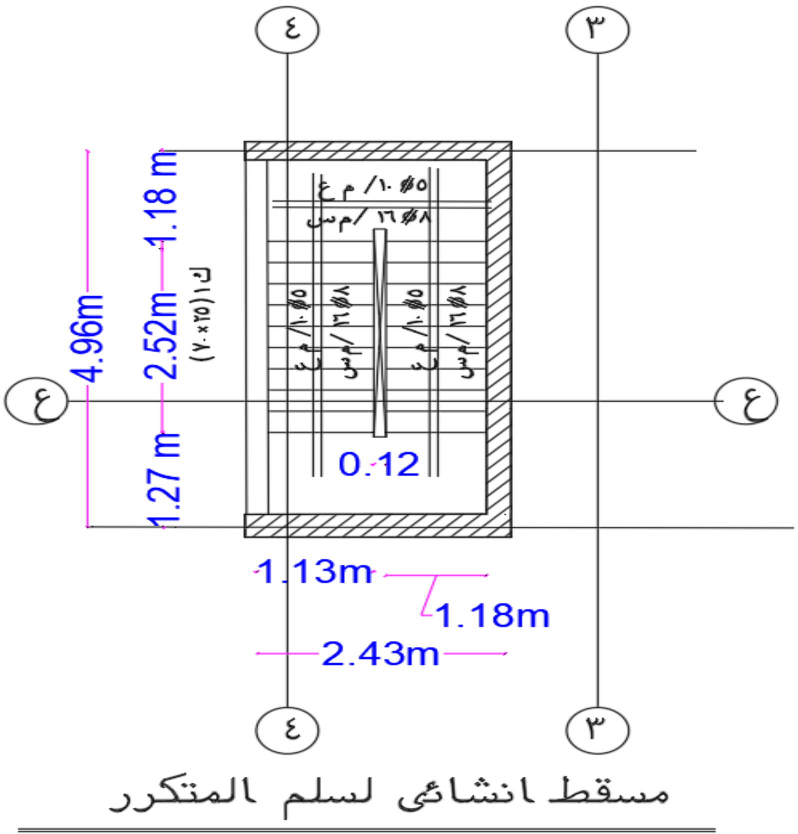


Figure 2.25 Stair Reinforcement in plan

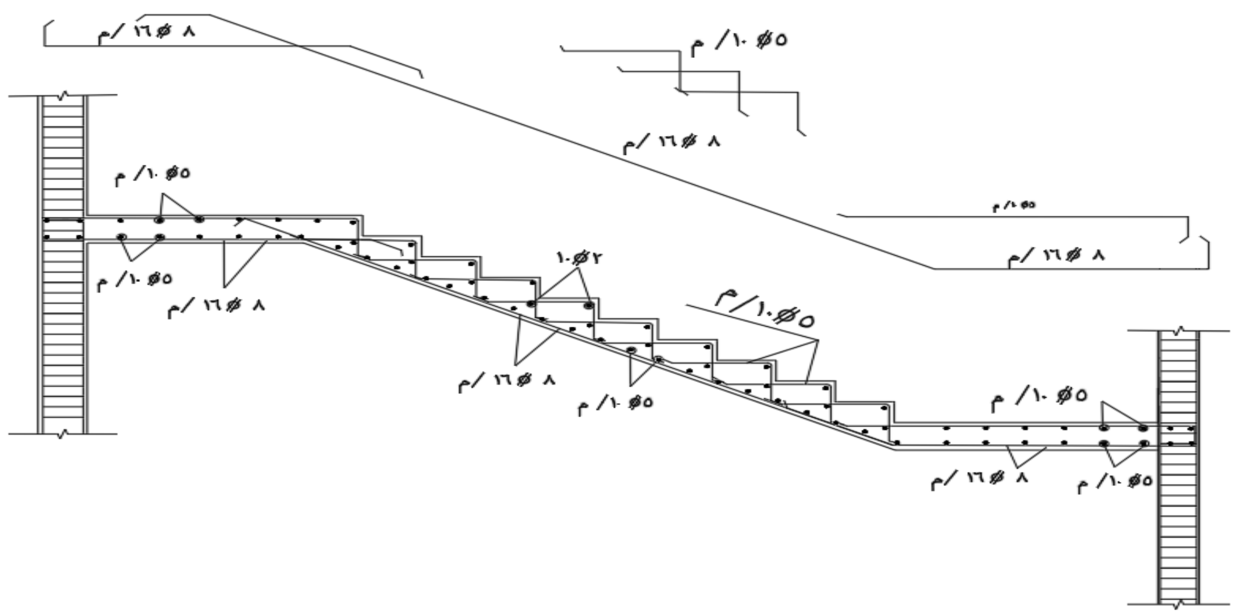


Figure 2.26 Reinforcement details

2.5 Design of Beams

Concrete dimensions

- Assume b = 25 cm
- t =$\frac{L}{12}$=80 cm
- **Loads on beams**
  - Own weight
  - Load from slab
  - Load from wall
- As=$\frac{Mu}{Fy*j*d}$ USE 2 ∯ 16 / m (1 ك) and 4 ∯ 16 / m (2ك)

2.5.1 Basement Slab Beams :

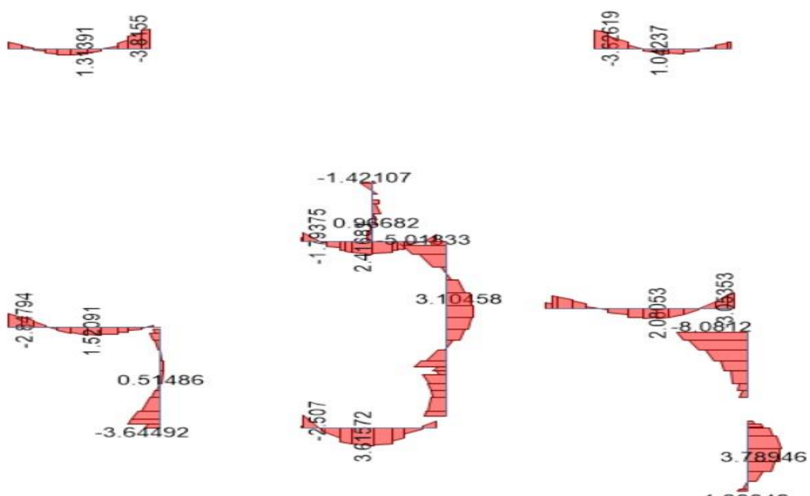


Figure 2.27 Moment Of Basement Slab Beams

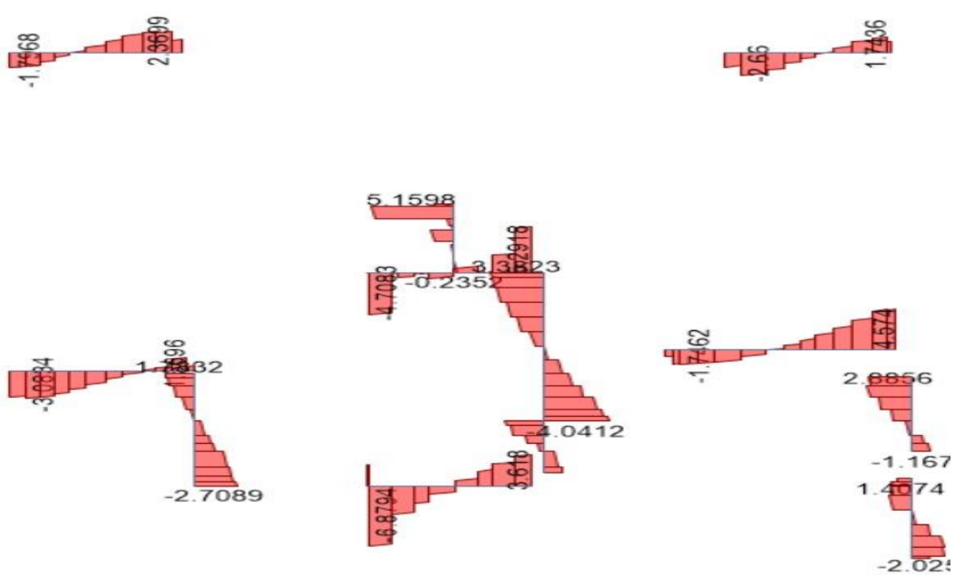


Figure 2.28 Shear Of Basement Slab Beam

2.6 Design of Columns

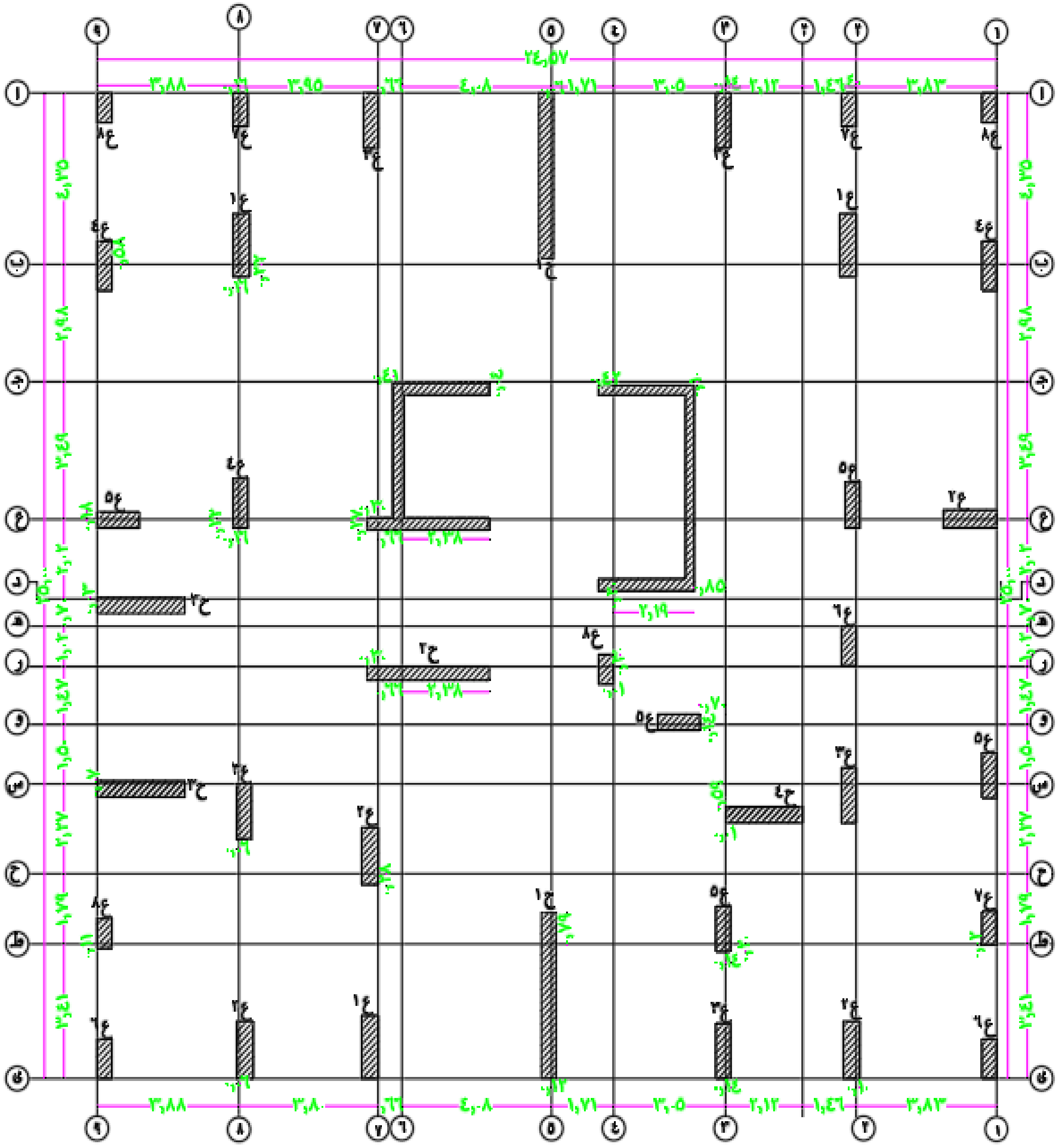


Figure 2.29 Columns Axis

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

- **For ع1 on axis (5-5) , (ي – ي)**
  - **Concrete Dimension**
    - P_U = 725 ton (From Etabs Program)
    - $P_{u\ actual}$ = 725 *1.1 = 797.5 ton
    - $P_{u\ actual} = 0.35(A_c - A_s)F_{cu} + 0.67\ A_s\ F_y$ → assume $A_s = 0.01\ A_c$
    - $797.5 * 10^3 = 0.35 * (0.99\ A_c) * 300 + 0.67 * 3600*0.01\ A_c$
    - $A_c = 6227.06\ cm^2$
    - Assume b= 45 cm
    - $t = \frac{Ac}{b} = 160$ cm
    - A_{cact}= 45 * 160 = 72 cm^2

- Check of Buckling (braced column)
  - in short direction
  - k = 0.80 (1-2)

$\lambda = \frac{k*Ho}{b} = \frac{0.8*2.90}{0.45} = 5.15 < 15$ ⇛ Short Column

  - in Long direction
  - k = 0.80 (1-2)

$\lambda = \frac{k*Ho}{t} = \frac{0.80*2.90}{1.60} = 1.45 < 15$ ⇛ Short Column

Neglected buckling in short and long direction

- $A_s = 0.01* 450 * 1600 = 72\ cm^2$

Use 36 ∯ 18

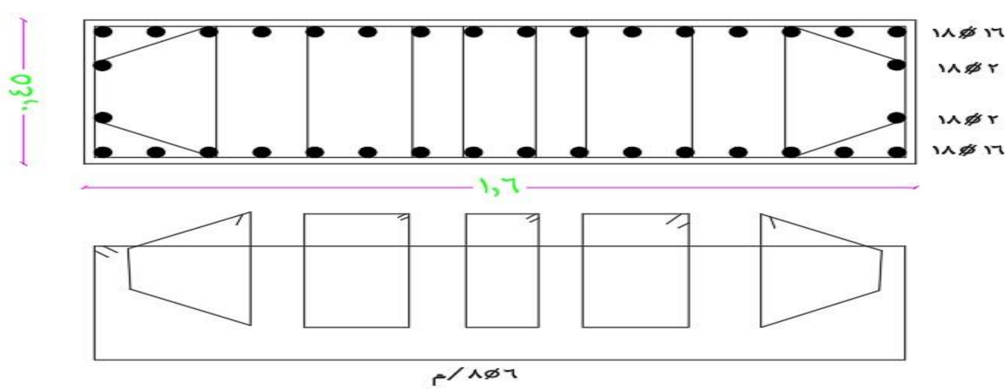


Figure 2.30 Column Cross Section

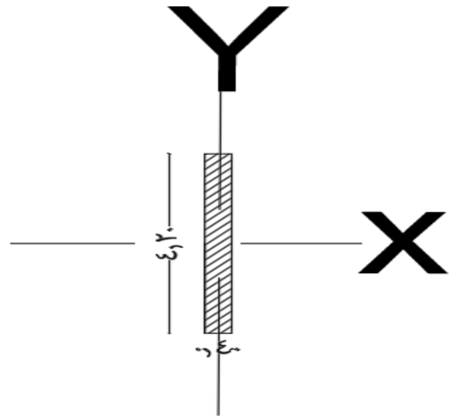
2.6.2 Table of Columns

Joint	P_u etabs (T)	$P_{u\ act}$ (T)	$\mu = A_s/A_c$	b (cm)	t (cm)	Ø	Sample
1	725.0078	725	1.00%	45	160	36Ø16	C1
2	716.2165	716	1.00%	45	160	36Ø16	C1
3	690.8475	691	1.00%	45	160	36Ø16	C1
4	640.3195	640	1.00%	45	145	26Ø18	C2
5	625.9972	626	1.00%	45	145	26Ø18	C2
6	625.3069	625	1.00%	45	145	26Ø18	C2
7	622.3681	622	1.00%	45	145	26Ø18	C2
8	565.0497	565	1.00%	40	140	26Ø18	C3
9	550.3094	550	1.00%	40	140	26Ø18	C3
10	537.0696	537	1.00%	40	140	26Ø18	C3
11	533.5909	534	1.00%	40	140	26Ø18	C3
12	532.9364	533	1.00%	40	140	26Ø18	C3
13	500.1122	500	1.00%	40	125	20Ø18	C4
14	495.8886	498	1.00%	40	125	20Ø18	C4
15	487.1236	487	1.00%	40	125	20Ø18	C4
16	468.4492	469	1.00%	40	115	20Ø18	C5
17	460.033	460	1.00%	40	115	20Ø18	C5
18	459.304	459	1.00%	40	115	20Ø18	C5
19	440.7449	441	1.00%	40	115	20Ø18	C5
20	435.8601	436	1.00%	40	115	20Ø18	C5
21	393.9528	394	1.00%	40	100	20Ø18	C6
22	393.5869	394	1.00%	40	100	20Ø18	C6
23	393.5869	394	1.00%	40	100	20Ø18	C6
24	393.5869	394	1.00%	40	85	14Ø18	C7
25	393.5869	394	1.00%	40	85	14Ø18	C7
26	393.5869	394	1.00%	40	85	14Ø18	C7
27	393.5869	394	1.00%	40	75	12Ø18	C8
28	393.5869	394	1.00%	40	75	12Ø18	C8
29	393.5869	394	1.00%	40	75	12Ø18	C8
30	393.5869	394	1.00%	40	75	12Ø18	C8

Figure 2.31 Table Columns Load And Section (Ultimat)

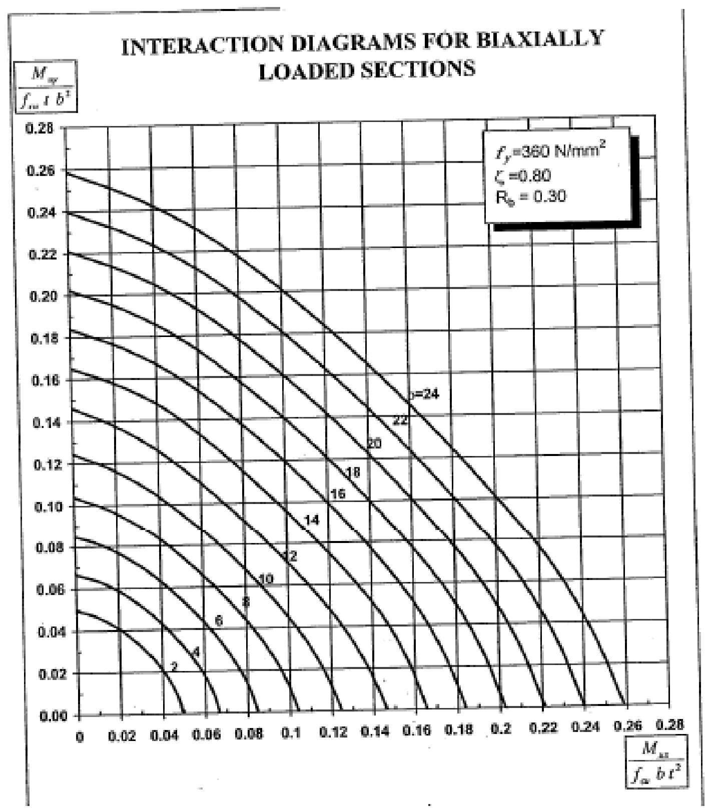
2.7 Design of ShearWall (W_1=0.45m*4.20m) on asix (7-7)

Case (1):



- P_u = 3827.3 KN
- M_{ux} = 12.362 kN.m
- M_{uy} = 1560 kN.m

1. $R_b = \frac{Pu}{Fcu*b*t} = \frac{3827.3 * 10^3}{30 *450 *4200} = 0.067 \approx 0.30$
2. $\frac{Mux}{Fcu*b*t^2} = \frac{12.362 *10^6}{30*450*4200^2} = 0.0001$
3. $\frac{Muy}{Fcu*t*b^2} = \frac{1560 * 10^6}{30*4200*450^2} = 0.06$



4. p = 3.8
5. $\mu = 3.8 * 30 * 10^{-4} = 0.0114$
6. As = 0.0114*450*4200 = 21546 mm^2

case (2):

- P_u = 150.263 Kn
- M_{ux} = 38.504 kn.m
- M_{uy} = 1522.1 kn.m

7. $R_b = \frac{Pu}{Fcu*b*t} = \frac{150.263 * 10^3}{30*450*4200} = 0.3$
8. $\frac{Mux}{Fcu*b*t^2} = \frac{38.504 *10^6}{30*450*4200^2} = 0.00016$
9. $\frac{Muy}{Fcu*t*b^2} = \frac{1522.1 * 10^6}{30*4200*4500^2} = 0.06$
10. p = 3.8
11. $\mu = 3.8 *30*10^{-4} = 0.0114$
12. As = 0.0114 *450*4200 = 21546 mm^2

Figure 2.32 Interaction Diagram for Biaxial Loaded Wall

USE 88 ∯18

Minimum RFT

As = 0.01 A_s = 215.46 mm^2

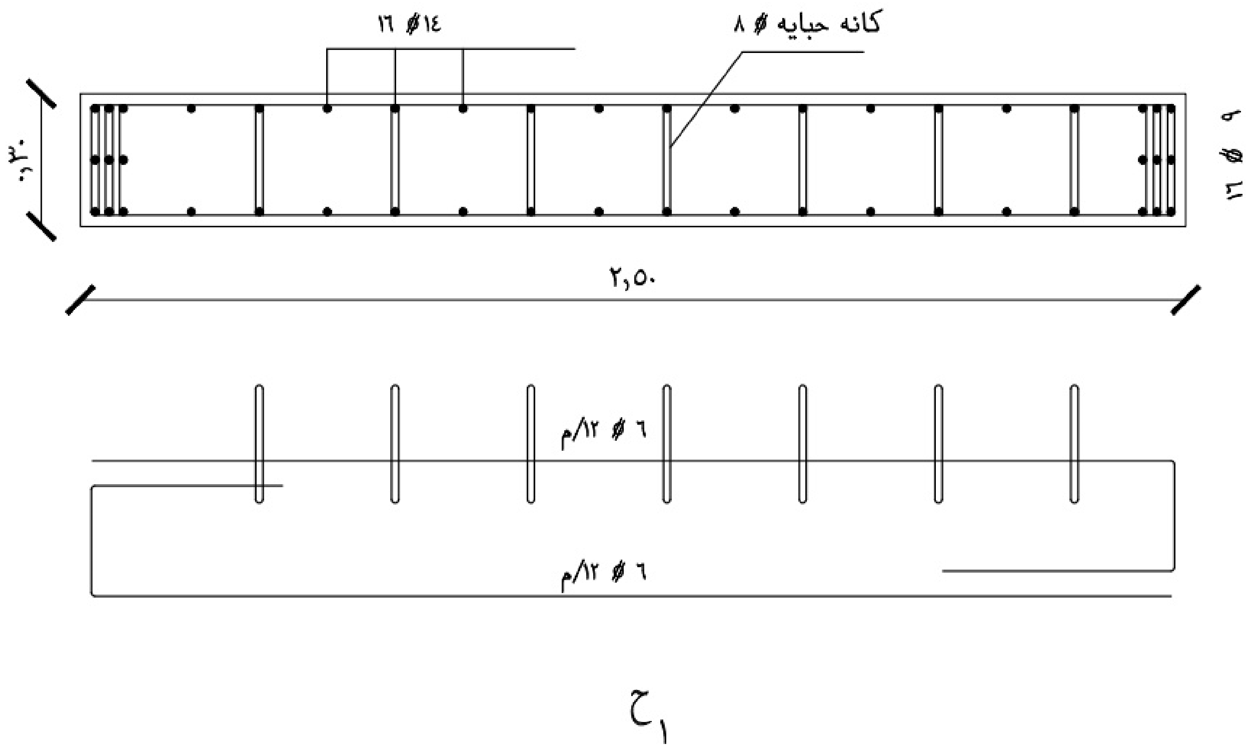


Figure 2.33 W1 Reinforcement

2.7.1 Table of Walls:

Wall Of Section			Case Of Loading			As	Additional steel at each corner
			P_u(ton)	M_{ux} (ton.m)	M_{uy} (ton.m)		
1ح	w1	0.45m*4.20m	15.0263	3.8504	152.21	88 D18	12∯18
			382.73	1.2362	1586		
			1353.8	1.47	1625.88		
2ح	w2	0.40m*3.40m	151.783	1.8397	472.56	76 D18	12∯18
			682.09	0.2301	7.2415		
			493.92	1.7306	468.94		
3ح	w3	0.40m*2.40m	90.6534	0.7153	344.46	58D18	12∯18
			309.32	1.0469	350.194		
			395.79	0.4618	340.09		
4ح	w4	0.40m*2.10m	969.94	1.886	14.8041	44D18	12∯18
			183.731	0.9124	1046.67		
			671.89	1.844	1039.6		

Figure 2.34 Table Wall Load And RFT (Ultimate)

2.8 Design of Core 1-2 (using ETABS program)

Case (1):

- P_u = 1101.4891 t
- M_{ux} = 362.9719 t.m
- M_{uy} = 2894.1329 t.m

Case (2):

- P_u = 339.2703 t
- M_{ux} = 1786.8875 t.m
- M_{uy} = 1883.3574 t.m

Case (3):

- P_u = 782.5734 t
- M_{ux} = 332.4167 t.m
- M_{uy} = 2853.7354 t.m

Use As_{total} =227 ϕ 18

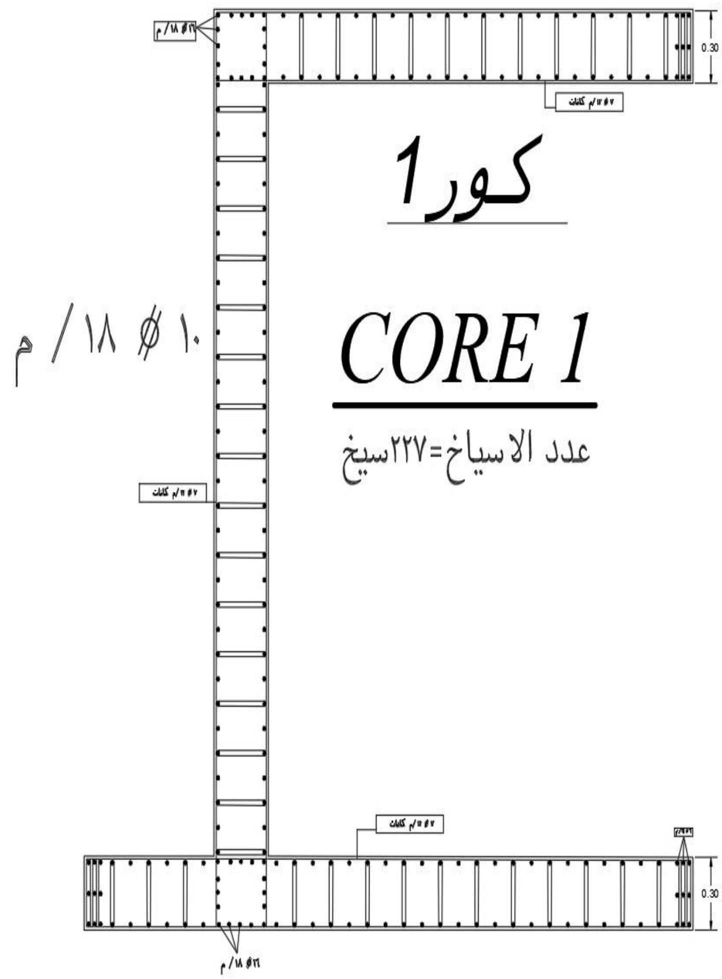


Figure 2.35 Core Cross Section

2.9 Design of Core 1-2 (using ETABS Column program)

Case (1):

- Pu = 993.2064 t
- Mux = 92.4005 t.m
- Muy = 4464.5441 t.m

Case (2):

- Pu = 1283.4098 t
- Mux = 69.0354 t.m
- Muy = 4716.7734 t.m

Case (3):

- Pu = 1869.7428 t
- Mux = 22.2974 t.m
- Muy = 221.5333 t.m

Use Astotal =220 Φ 18

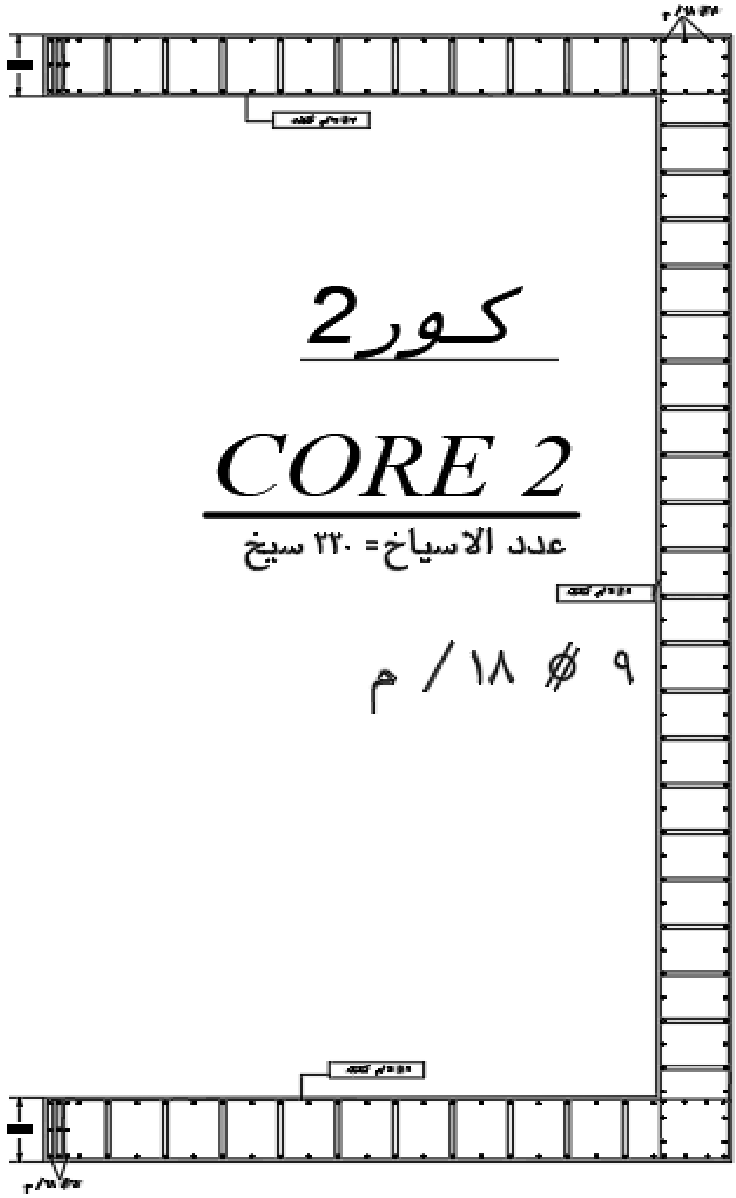


Figure 2.36 Core Cross Section

2.10 Effect of Earthquake on tower

2.10.1 Manual solution

- Zone (2) ⇛ ag = 0.125g = 1.226 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
- Response Modification factor(R) = 5

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

1. $T_1 = C_t\, H^{3/4}$ = $0.05 * 40.60^{3/4}$ = 0.8042 sec
2. $T_C < T_I < T_D$ ⇛ $Sd(T_1) = ag * \Upsilon_I * \frac{2.5\,\eta}{R} * [\frac{TC}{T_1}]$
3. = $1.2226*1*1.5*\frac{2.5}{5}*\frac{.25}{0.8042}$ = .286 > 0.2 ag * Υ_I = .2*1.25*1 = .25
4. **λ = 1** ⇛ as $T_I > 2\ T_C$
5. W_i = D.L +Ψ L.L
   a. D.L $_{\text{(Basement Slab)}}$ = O.w + cover + W_{wall} = 2.50 * 0.25 + 0.15 + 0.27 = 1.045 t/m^2
   b. W $_{\text{(Basement Slab}}$) = 1.045 + 0.25 * 0.4 = 1.145 t/m^2
   c. D.L $_{\text{(Ground Or Repatd Slab)}}$ = O.w + cover + W_{wall} = 2.50 * 0.25 + 0.15 + 0.27 = 1.045 t/m^2
   d. W $_{\text{(Ground Or Repatd Slab)}}$ = 1.045 + 0.25 * 0.3 = 1.12 t/m^2
6. $\mathbf{W_{total}}$ = 15520 ton ⇒ FROM ETABS BROGRAM

   Note (1.1 For Col. And Shear wall And Beams)
7. $F_b = 0..286 * 1 * \frac{15520}{9.81}$ = 452.46 ton

FLOOR	W floor	H	W*H	FORCE	ton(ULT)	Shear ton	Base moment	TOTAL M
base		0				558.1531	0.0	11997.7
1	480	3	1440	F1	7.903612	550.2495	23.7	10432.8
2	480	6	2880	F2	15.80722	534.4423	94.8	8891.6
3	480	9	4320	F3	23.71084	510.7314	213.4	7397.8
4	480	12	5760	F4	31.61445	479.1170	379.4	5975.1
5	480	15	7200	F5	39.51806	439.5989	592.8	4647.3
6	480	18	8640	F6	47.42167	392.1772	853.6	3438.1
7	480	21	10080	F7	55.32528	336.8519	1161.8	2371.1
8	480	24	11520	F8	63.2289	273.6231	1517.5	1470.1
9	480	27	12960	F9	71.13251	202.4905	1920.6	758.7
10	480	30	14400	F10	79.03612	123.4544	2371.1	260.8
11	480	33	15840	F11	86.93973	36.5147	2869.0	260.8
		TOTAL	95040		521.6384			
						moment at base	11997.7	

Figure 2.37 Table Over Turning And Torsional Moment

$M_{overturning}$ = 11997.7 t.m
$M_{torsional} = V_i * e_{min}$ = 252 * 0.05 * 41.5 = 521.63 t.m

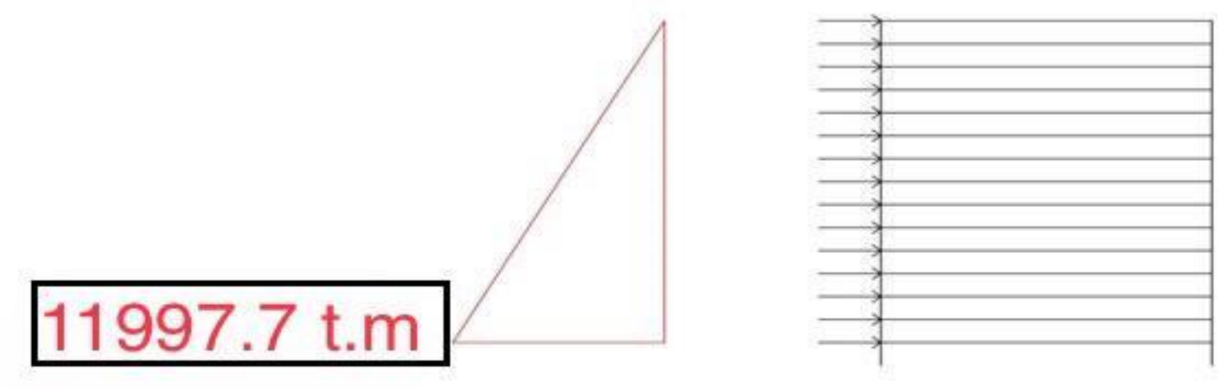


Figure 2.38 Over Turning Moment

Check of stresses between (Shear wall , and cores) and Foundation

Inertia of Walls

- **W1 (0.45*4.20)**

A = 0.45 * 4.20 = 1.89 m^2

$I_X = \frac{0.45*4.20^3}{12}$ = 2.78 m^4

$I_Y = \frac{4.20*0.45^3}{12}$ = 0.032 m^4

- **W2 (0.40*3.40)**

A = 0.40 * 3.40 = 1.36 m^2

$I_X = \frac{3.40*0.40^3}{12}$ = 0.0181 m^4

$I_Y = \frac{0.40 * 3.40^3}{12}$ = 1.310 m^4

- **W3 (0.40*2.40)**

A = 0.40 * 2.40 = 0.96 m^2

$I_X = \frac{2.40*0.40^3}{12}$ = 0.0128 m^4

$I_Y = \frac{0.3*2.3^3}{12}$ = 0.0575 m^4

- **W4 (0.40*2.10)**

A = 0.40* 2.10 = 0.84 m^2

$I_X = = \frac{2.10 *0.40^3}{12}$ = 0.0112 m^4

$I_Y = \frac{0.40 * 2.10^3}{12} = 0.30\ m^4$

- **Core 1**

$A = 2.72\ m^2$

$I_X = \frac{2.63 * 0.30^3}{12} + \frac{0.30 * 3.12^3}{12} + \frac{3.34 * 0.30^3}{12} = 0.77\ m^4$

$I_Y = \frac{0.30 * 2.63^3}{12} + \frac{3.12 * 0.30^3}{12} + \frac{0.30 * 3.34^3}{12} = 1.39\ m^4$

- **Core 2**

$A = 2.54\ m^2$

$I_X = 2 * \frac{2.61 * 0.30^3}{12} + \frac{0.30 * 4.66^3}{12} = 2.54\ m^4$

$I_Y = 2 * \frac{0.30 * 2.61^3}{12} + \frac{4.66 * 0.30^3}{12}\ 0.049\ m^4$

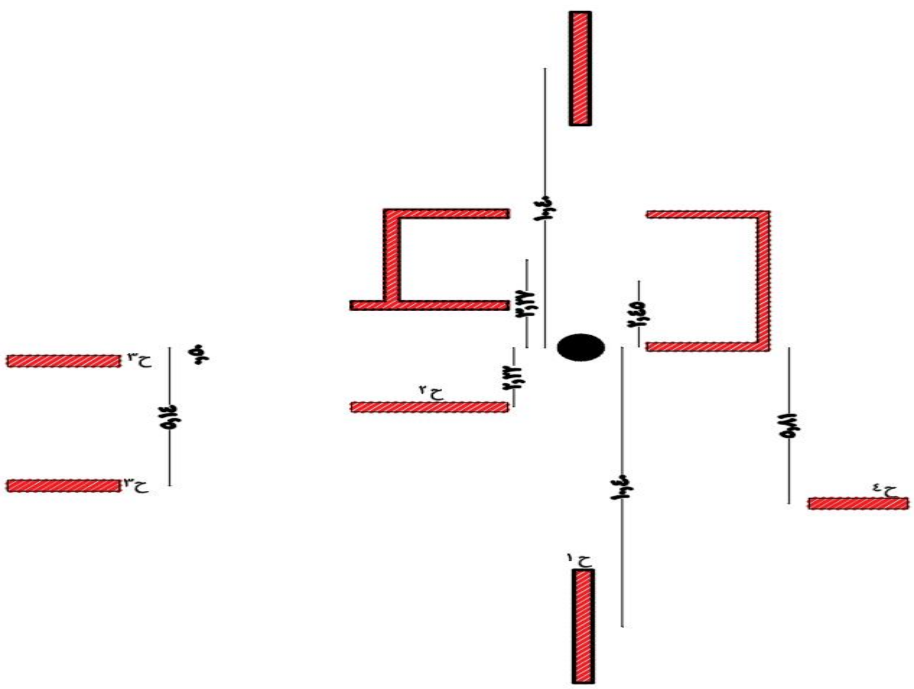


Figure 2.39 Moment of inertia (Ix)

$I_{x\ group} = (1.89 + 2.78 * (10.40)^2) * 2 + (0.0181 + 1.36 * (2.22)^2) + (0.0128 + 0.96 * (5.14)^2) + (0.0128 + 0.96 * (0.50)^2) + (0.0112 + 0.84 * (5.81)^2) + (0.77 + 2.72 * (3.27)^2) + (2.54 + 2.54 * (2.45)^2) = 522.74\ m^4$

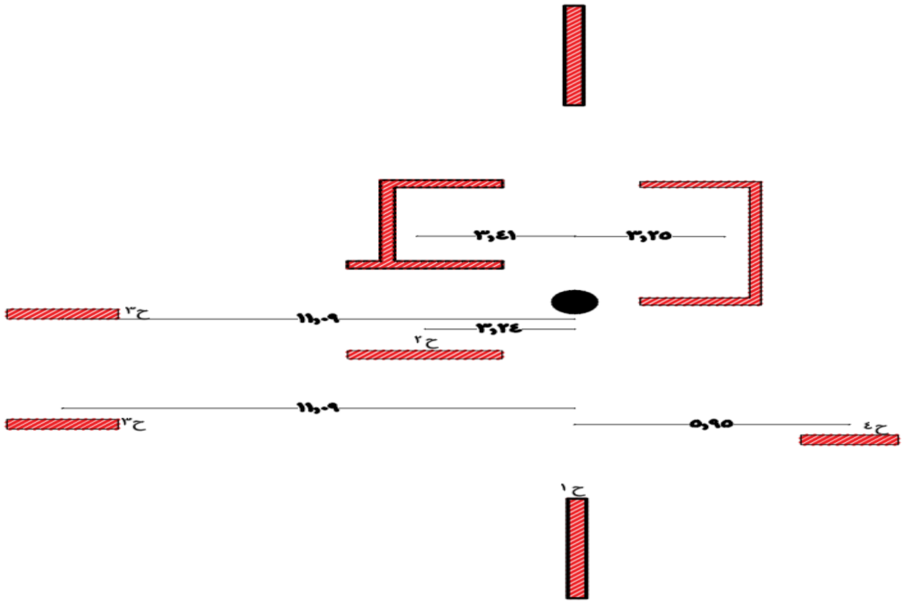


Figure 2.40 Moment of inertia (I_Y)

$I_{Y\ group} =$ = (0.032 +2.78 *(0)^2)*2 + (1.310 +1.36 * (3.24)^2) +(0.0575 + 0.96 *(11.09)^2)* 2 + (0.30 +0.84 *(5.59)^2) + (1.39 + 2.72 *(3.41)^2) + (0.049 + 2.54 *(3.25)^2) = 338.215 m^4

$I_{X\ group} > I_{Y\ group}$

X Is Critical Direction

$$\mathbf{F_{1,2}} = \frac{-N}{A} \pm \frac{M_x * X}{I_Y} \leq \mathbf{1.25\ Fc}$$

$A_{wall,Core}$ = 13.3 m^2 (From CAD Program)

N_{wall} =5700.67 (From ETAP Program)

M_x = 11997.7 t.m

Y = 10.10 m

$I_{Y\ group}$ = 338.215 m^4

$$F_{1,2} = \frac{-5700.67}{13.3} \pm \frac{11997.7 * 10.10}{338.215}$$

$$F_1 = \frac{-5700.67}{13.3} - \frac{11997.7 * 10.10}{338.215} = -78.7\ Kg/Cm^2 < 1.25 * 105 = 131.25\ Kg/Cm^2$$

$$F_2 = \frac{-5700.67}{13.3} + \frac{11997.7 * 10.10}{338.215} = -7.03\ Kg/Cm^2 < 1.25 * 105 = 131.25\ Kg/Cm^2$$

There is no tension OK Safe

2.10.2 Check of Displacement

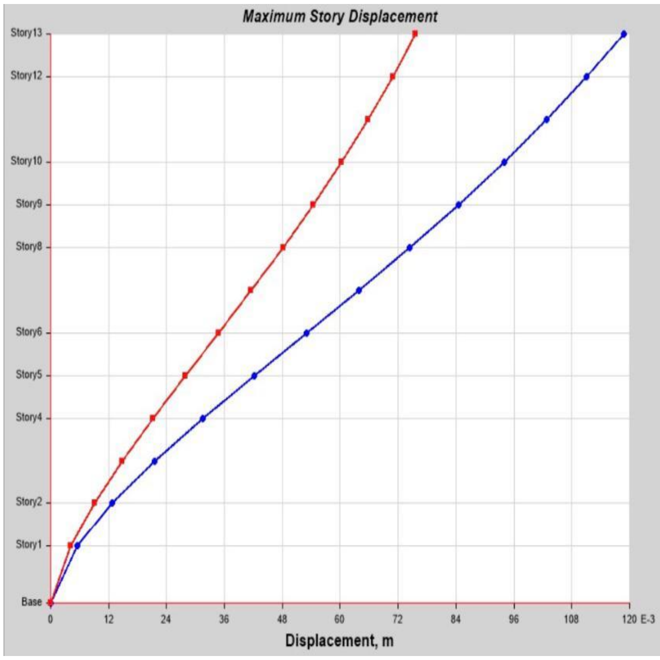


Figure 2.41 Maximum Story Displacement in X direction

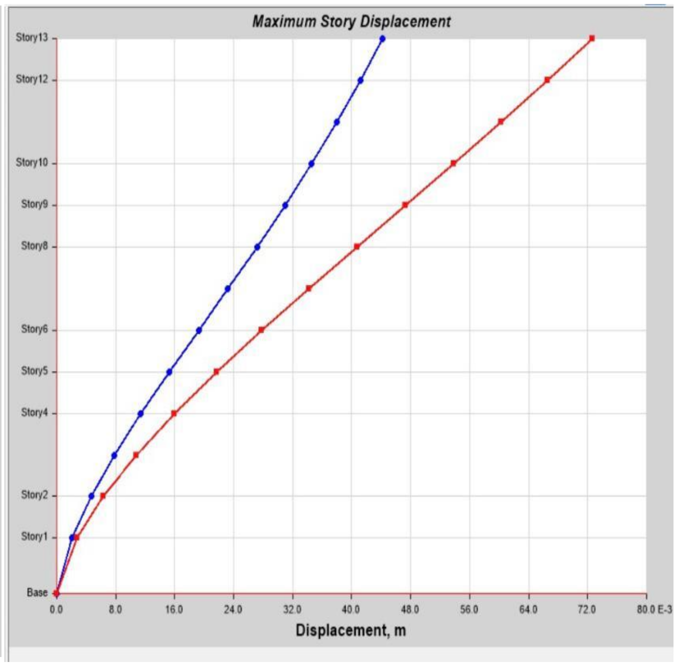


Figure 2.42 Maximum Story Displacement in Y direction

Story	H(m)	$d_{s\,x}$ (m)	$d_{s\,y}$ (m)
Story12	40.6	0.058263	0.036124
Story11	37.7	0.055066	0.033662
Story10	34.8	0.051606	0.031054
Story9	31.9	0.047794	0.028302
Story8	29	0.043594	0.025409
Story7	26.1	0.039017	0.022394
Story6	23.2	0.034107	0.019288
Story5	20.3	0.028939	0.016139
Story4	17.4	0.023621	0.013007
Story3	14.5	0.018293	0.009967
Story2	11.6	0.013135	0.007107
Story1	8.7	0.008377	0.004531
GEROUND	5.8	0.004313	0.002364
BASMENT	2.9	0.001319	0.000759

Figure 2.43 Table Story Response Data

Max Displacement at 12 th Floor

- X-Direction = 0.058263
- Y-Direction = 0.036124

Height of Building = 40.6 m

Allowable Displacement according to ECP = $\frac{H}{500}$ =$\frac{40.6}{500}$ =0.0812 m Working Method

Ultimate Method = 0.0812*1.5 = 0.1218 m

OK , Safe

2.10.3 Check of Drift

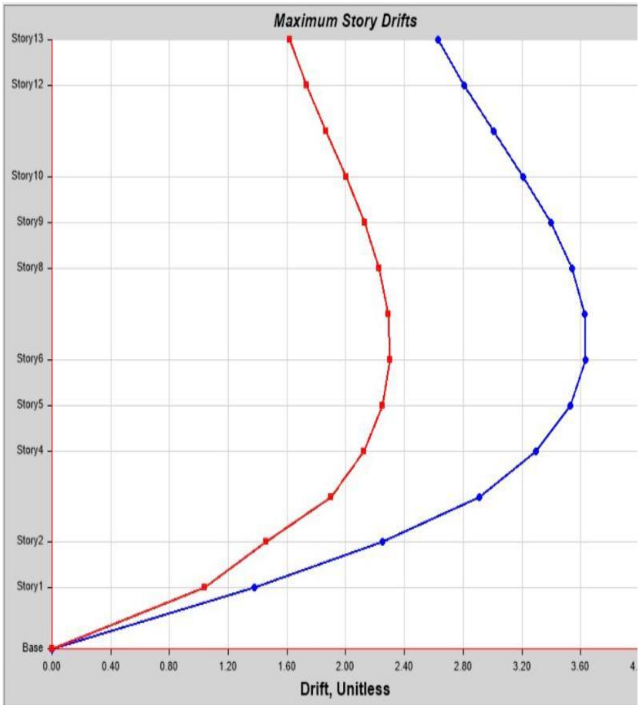


Figure 2.44 Maximum Story Drifts In X diriction

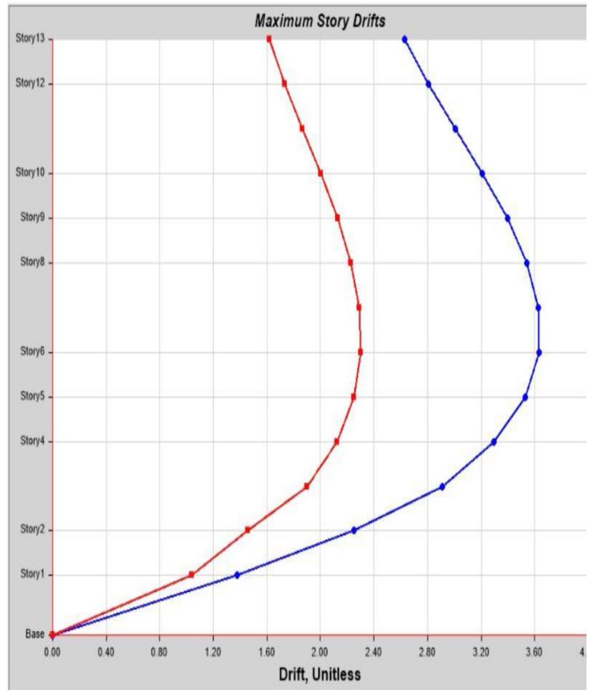


Figure 2.45 Maximum Story Drifts In y diriction

Story	H(m)	d_{ry} (m)	d_{rx} (m)	d_{rvx} (m)	d_{rvy} (m)
Story12	3	0.058263	0.03612	0.0032	0.0025
Story11	3	0.055066	0.03366	0.0035	0.0026
Story10	3	0.051606	0.03105	0.0038	0.0028
Story9	3	0.047794	0.0283	0.0042	0.0029
Story8	3	0.043594	0.02541	0.0046	0.003
Story7	3	0.039017	0.02239	0.0049	0.0031
Story6	3	0.034107	0.01929	0.0052	0.0031
Story5	3	0.028939	0.01614	0.0053	0.0031
Story4	3	0.023621	0.01301	0.0053	0.003
Story3	3	0.018293	0.00997	0.0052	0.0029
Story2	3	0.013135	0.00711	0.0048	0.0026
Story1	3	0.008377	0.00453	0.0041	0.0022
GEROUND	3	0.004313	0.00236	0.003	0.0016
BASMENT	2.65	0.001319	0.00076	0.0013	0.0008

Figure 2.46 Table Story Response Values

Max drift at X-direction = 0.0053m

Max drift at Y-direction = 0.0031m

Allowable Drift according to ECP $=\frac{H}{500}=\frac{2.90}{500}$ =0.0058 m Working Method

Ultimate Method = 0.006 *1.5 = 0.009 m

OK , Safe

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

Story	Diaphragm	Mass X tonf-s²/m	Mass Y tonf-s²/m	XCM m	YCM m	Cum Mass X tonf-s²/m	Cum Mass Y tonf-s²/m	XCCM m	YCCM m
Story1	D1	94.41336	94.41336	12.3997	12.4329	94.41336	94.41336	12.3997	12.4329
Story2	D2	97.23639	97.23639	12.4163	11.8639	97.23639	97.23639	12.4163	11.8639
Story3	D3	97.23651	97.23651	12.4163	11.8639	97.23651	97.23651	12.4163	11.8639
Story4	D4	97.23639	97.23639	12.4163	11.8639	97.23639	97.23639	12.4163	11.8639
Story5	D5	97.23639	97.23639	12.4163	11.8639	97.23639	97.23639	12.4163	11.8639
Story6	D6	97.23639	97.23639	12.4163	11.8639	97.23639	97.23639	12.4163	11.8639
Story7	D7	97.23639	97.23639	12.4163	11.8639	97.23639	97.23639	12.4163	11.8639
Story8	D8	97.23651	97.23651	12.4163	11.8639	97.23651	97.23651	12.4163	11.8639
Story9	D9	97.23639	97.23639	12.4163	11.8639	97.23639	97.23639	12.4163	11.8639
Story10	D10	97.23639	97.23639	12.4163	11.8639	97.23639	97.23639	12.4163	11.8639
Story11	D11	97.23639	97.23639	12.4163	11.8639	97.23639	97.23639	12.4163	11.8639
Story12	D12	97.23639	97.23639	12.4163	11.8639	97.23639	97.23639	12.4163	11.8639
Story13	D13	90.81317	90.81317	12.3938	11.8701	90.81317	90.81317	12.3938	11.8701

Figure 2.47 Table Story Response Values

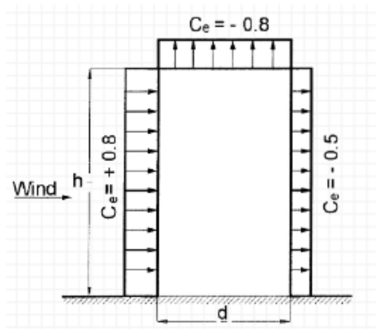
Max Eccentricity
e_x = 0.22
e_y = 0.5

Allowable Difference ≮ 0.15 b = 0.15*26.57 = 3.985 m ⇛ OK Safe
≮ 0.15 t = 0.15*28.2269 = 4.234 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 ⇛ كثافة 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

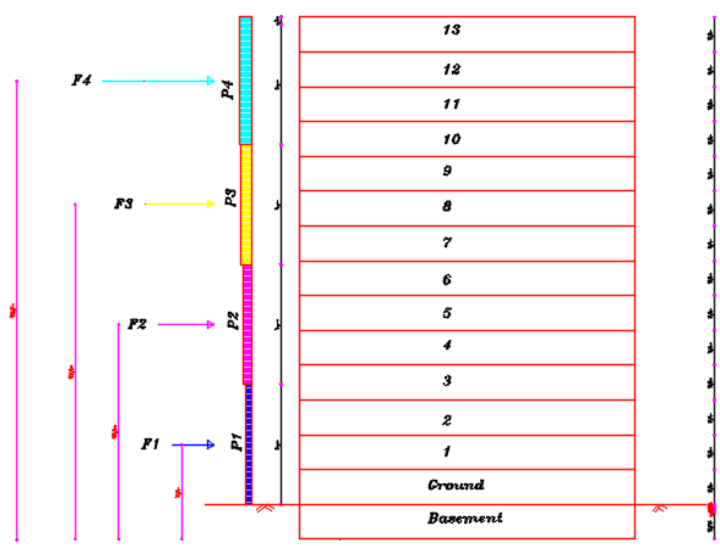


Figure 2.48 Wind Load

Check Of Over Turning Moment

Wt = 13130.9 ton

Resisting Moment = 13130.9 * $\frac{26.5744}{2}$ =174444 ton.m

Over Turning Moment = 11997.7 ton.m

Factor Of Safety = $\frac{\text{Resisting Moment}}{\text{Over Turning Moment}}$ = $\frac{174444}{11997.7}$ = 14.54 > 1.50

Ok, Safe

2.12 Design of Deep Foundation (Raft on Piles)

Use Thickness of Raft = 120 cm
Pile diameter = 60 cm
Pile capacity = 120 ton

Total building weight (working method) = reactions of column , shear wall and core + O.w R.w + O.w raft

- Reactions of columns (Etabs program) =15054 ton
- O.W raft = (o.w slab+ cover + car weight) * area

= (2.5*1.20+ 0.15 + 0.5) * 691 = 2522.15 ton

Total building weight = 15054 + 2522.15 = 17576.15 ton

No. of piles = total building / pile capacity

= 17576.15/120 = 146.46 = 147 piles

Increase no. of piles by 20% t1o carry lateral loads generated by earthquakes.

No.of piles = 147 *1.25= 183.75 ~ 186 piles

Piles spacing (s) = (2→3) D

S= 1.80 m , 2 m on the side and the middle

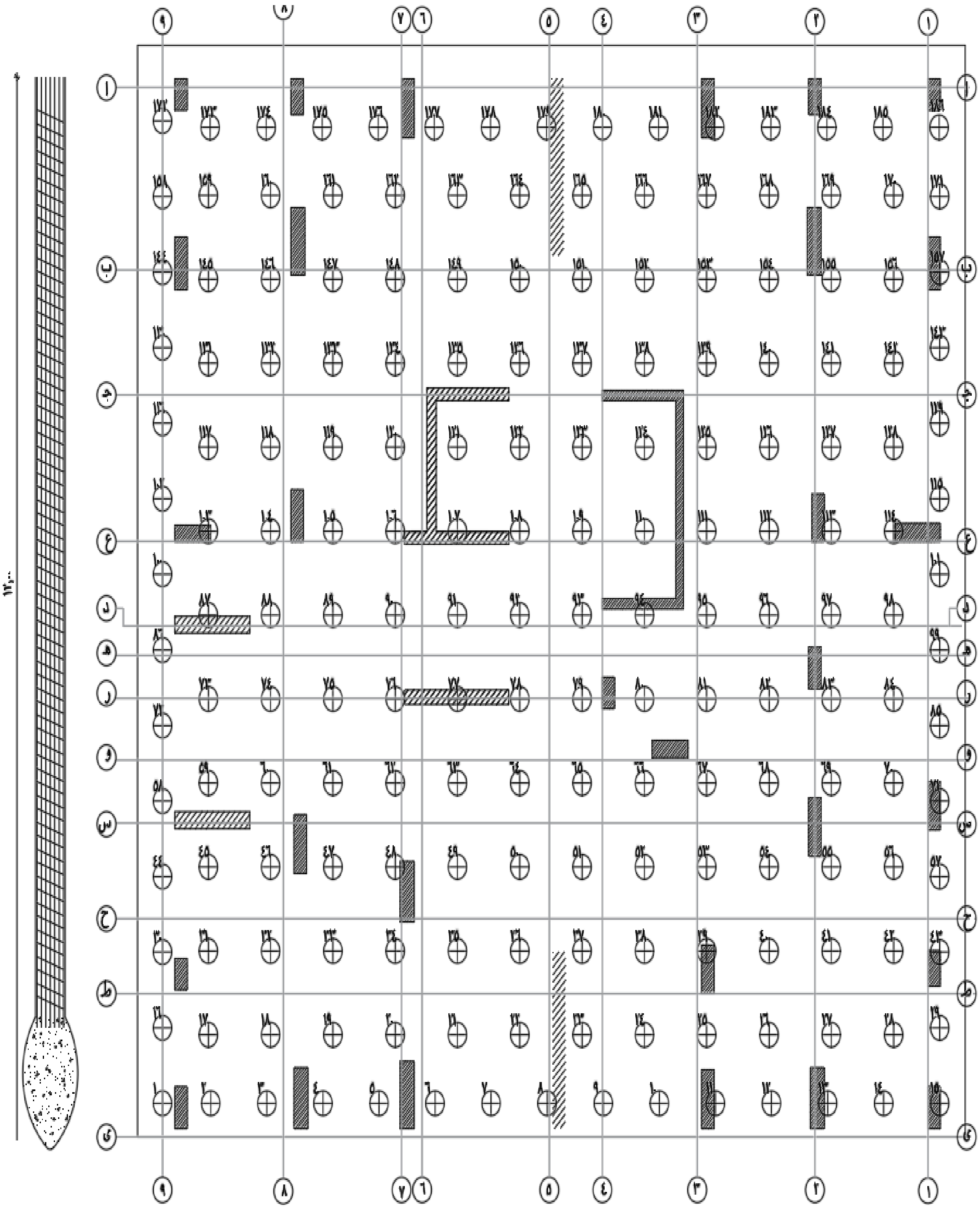


Figure 2.49 Piles arrangement

2.12.1 Design of Raft on Piles (Safe program)

$t_{raft} = 120$ cm

$d_{pile} = 60$ cm

$P = K\,\Delta \rightarrow P = 100$ ton , $\Delta = 1$cm

$K_{stiffness} = p/\Delta = 100/0.01 = 10000$ t/m

Use 85% $K_{stiffness} = 0.85 * 10000 = 8500$ t/m

- For raft

  $A_s = (Mu)/ f_y\, J\, d$

$M_u = As * F_y * J * d = 12 * (\pi/4*1.8^2) * 3600 * 133 * 0.826*10^{-5} = 120.76$ t.m

- Use 8 ∯ 18 Upper and Lower Reinforcement

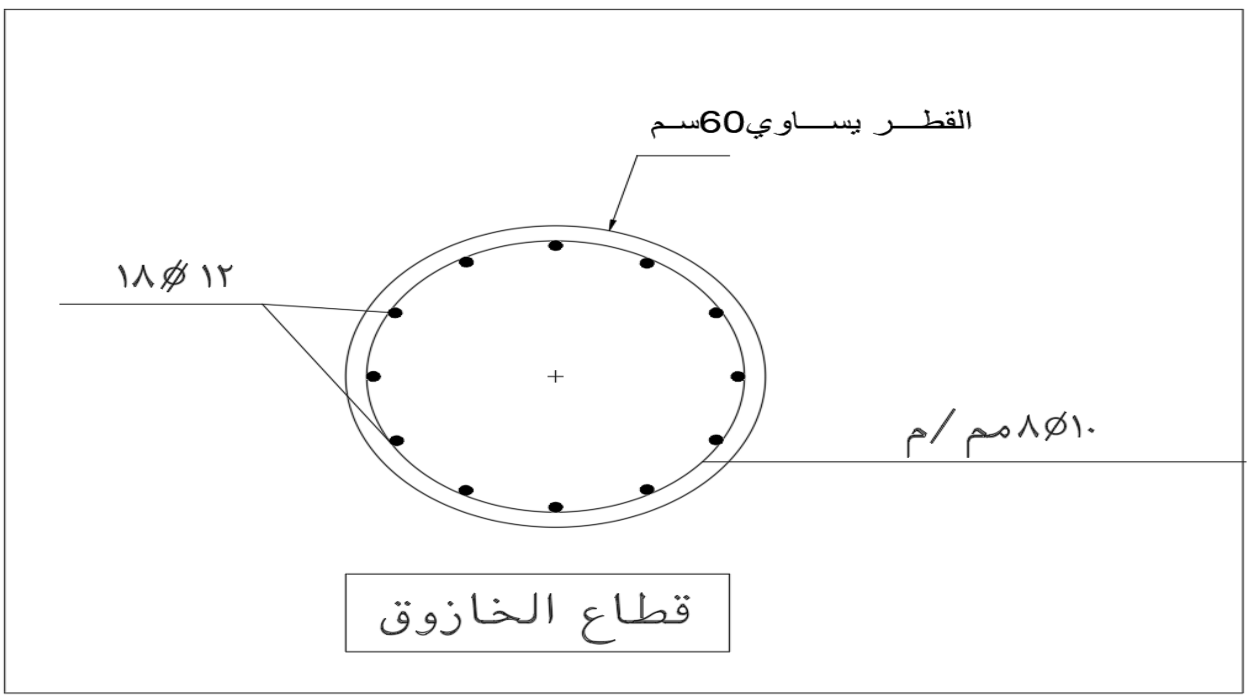


Figure 2.50 Section of Pile

In X–Direction (Upper and Lower)

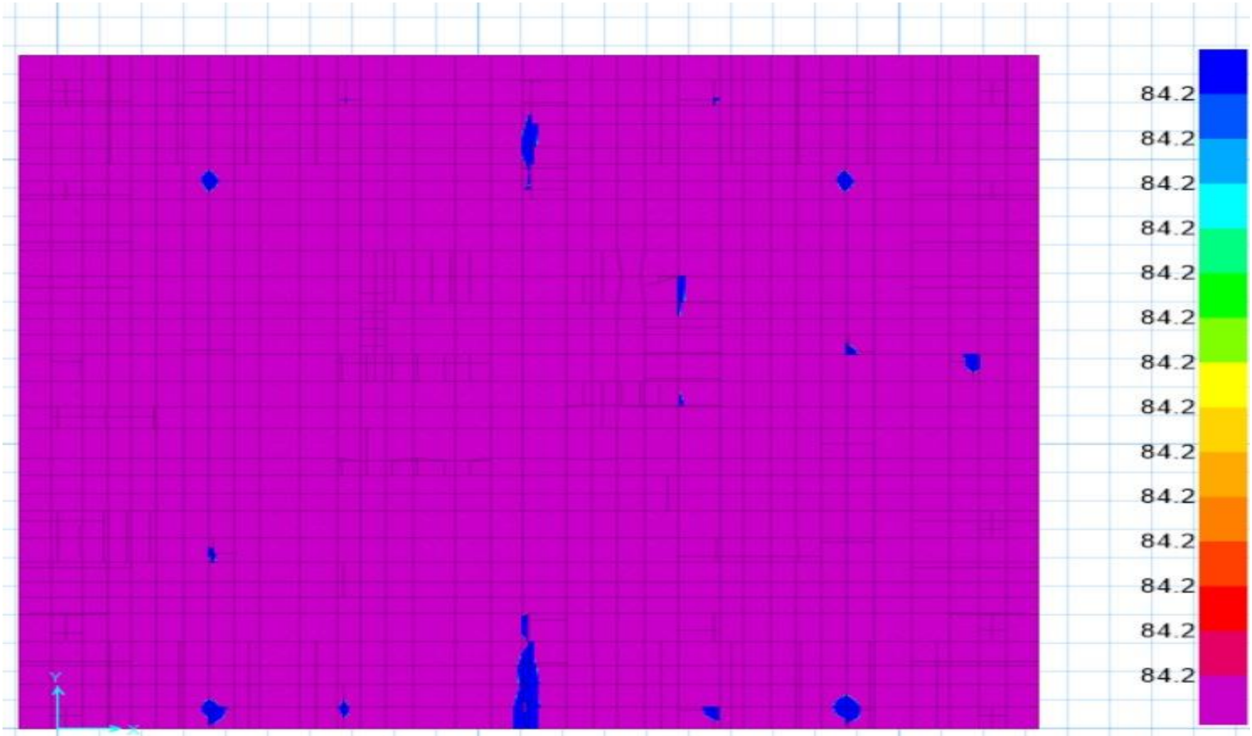


Figure 2.51 Reinforcement in X-Direction (Upper and Lower)

In Y–Direction (Lower and lower)

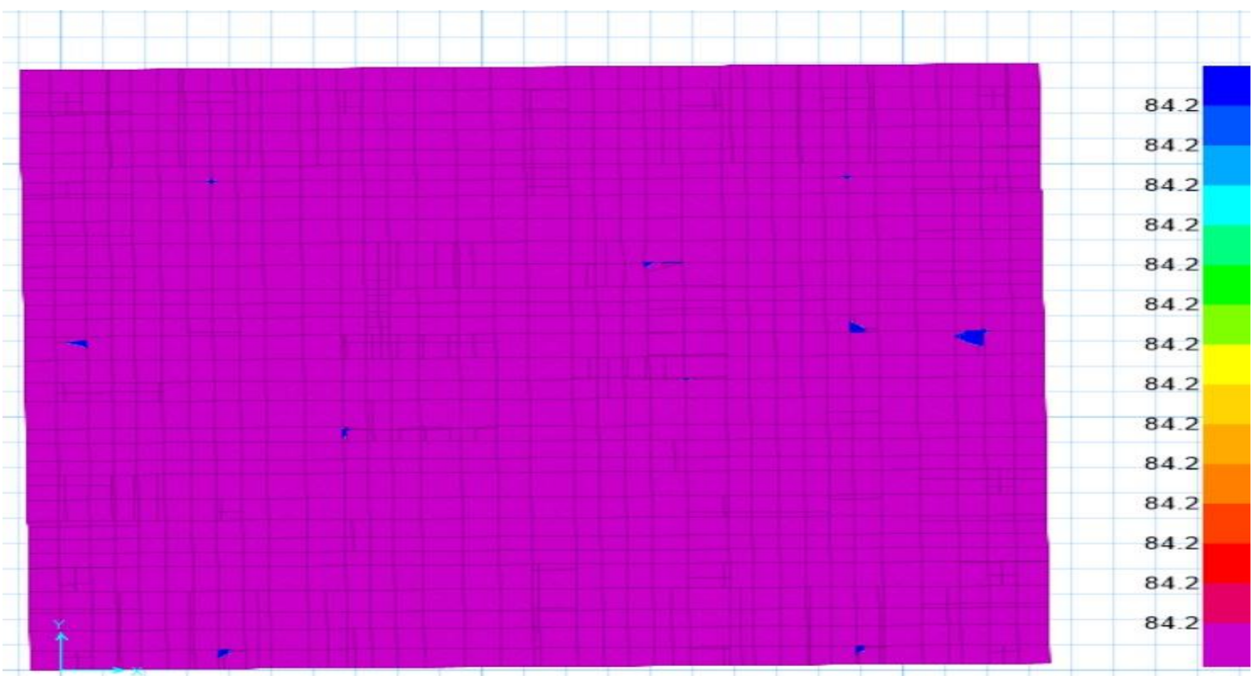


Figure 2.54 Reinforcement in Y-Direction (Upper and Lower)

2.12.1.1Table of piles forces :-

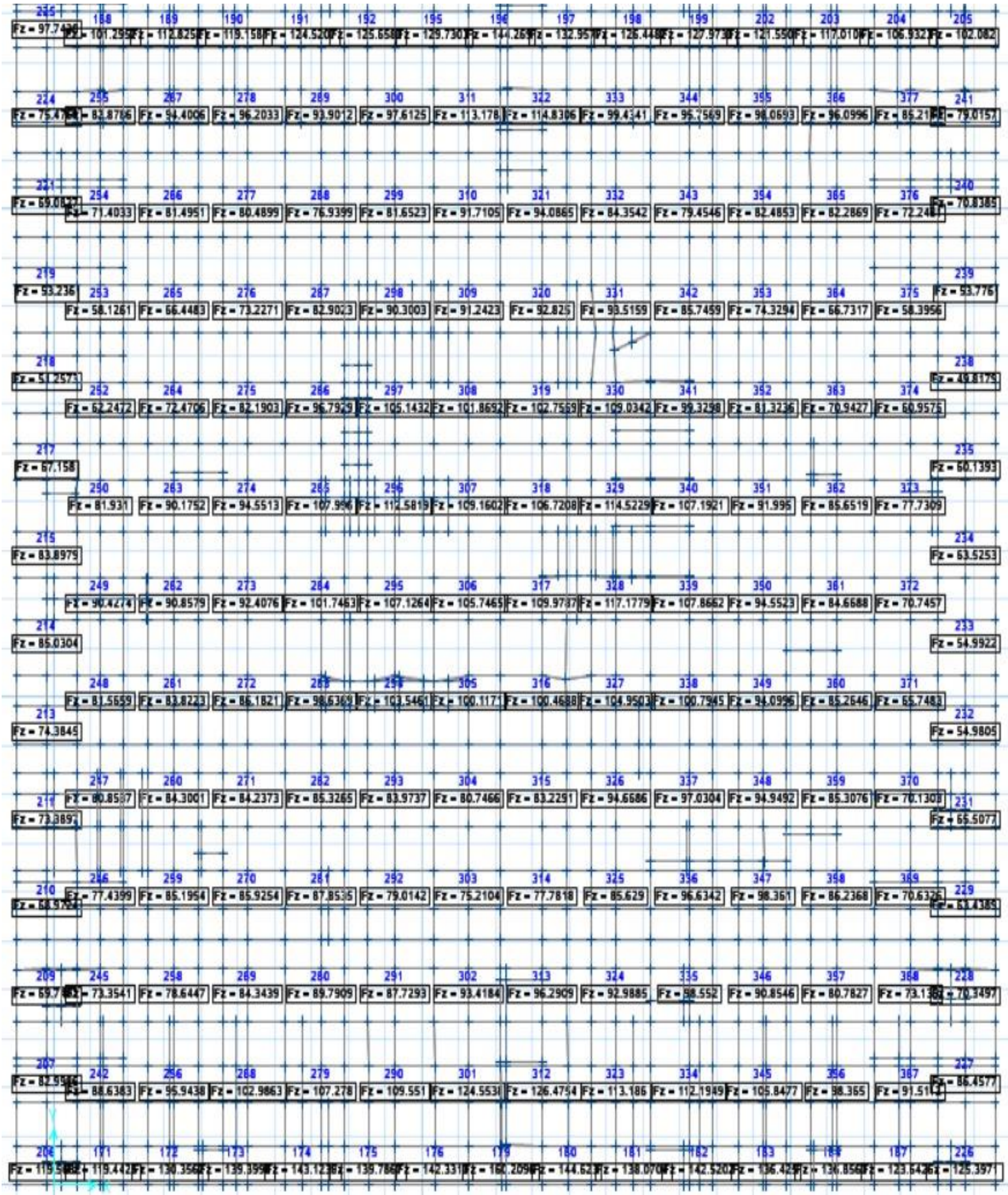


Table 2.55 Piles Forces

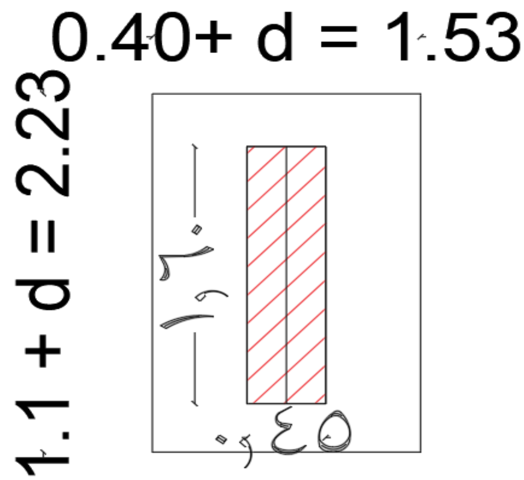
2.12.1.2 Check of punching shear on raft

- for 1 ع (45 * 160)

Col load = 725 ton

t_{raft} = 120 cm

d = t_{raft} – cover = 120 -7 = 113 cm



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

$$\frac{725 * 10^4}{2 * (2230 + 1530) * 1130} = 0.85$$

q_{cup} = *the least of*:-

- 1.70 N/mm^2
- $0.316\sqrt{\frac{f_{cu}}{\gamma_c}} == 0.316\sqrt{\frac{30}{1.5}} = 1.41 N/mm^2$
- $0.316\,(\frac{a}{b} + 0.5)\sqrt{\frac{f_{cu}}{\gamma_c}} = 0.316\,(\frac{400}{1150} + 0.5)\sqrt{\frac{30}{1.5}} = 1.198\ N/mm^2$
- $0.8\,(\frac{\alpha * d}{b0} + 0.2)\sqrt{\frac{f_{cu}}{\gamma_c}} = 0.8\,(\frac{4*1330}{8420} + 0.2)\sqrt{\frac{30}{1.5}} = 2.97\ N/mm^2$

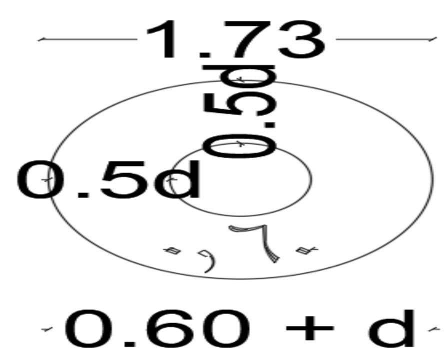
- $q_{up} = 0.85\ N/mm^2 \le q_{cup} = 1.198\ N/mm^2$

OK . Safe

- **For pile no. (16)**

Pile load = 120 ton

d_{pile} = 60 cm



$q_{up} = Q_{up}/\pi D d$ = (120 *10^3) / (π * 173 *113)

= 1.95 kg/cm^2 < 10 kg/cm^2 OK . Safe

2.13 Design of Retaining wall:

1 Loads

- Thickness of wall = 25 cm
- Use Fill sand $\emptyset_{30}$
- γ_{soil} = 1.5 t/m^3
- $K_a = \frac{1-\sin\emptyset}{1+\sin\emptyset}$ = 0.33
- $q_{(\,sur\ charge\,)}$ = 1 t/m^2
- e1 = (Σγ*h+q)K_a -2*C*$\sqrt{K_a}$
  =(1.5*2.9+1)*.33 = 1.78
- F_1 = 0.5 * 1.53 * 2.9 = 2.58t/m^2

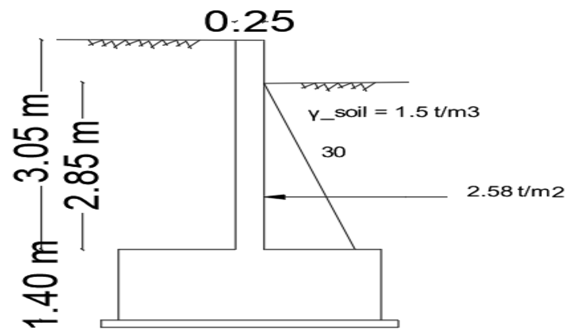


Figure 2.56 Section of Retaining Wall (Critical Section)

2.13.1 Design of critical section (as uncracked section)

- M_{over} = 2.58*$\frac{1}{3}$ *2.9= 2.494 t.m/$m^\backslash$
- t = 250 mm , d = 220 mm
- d = $C_1\sqrt{\frac{M}{b*F_{CU}}}$ = 220 = $C_1\sqrt{\frac{2.494*10^7*1.5}{1000*30}}$ $\rightarrow C_1 = 6.275$, J= .826
- $A_s = \frac{M*1.5}{Fy*d*J} = \frac{2.494*10^7*1.5}{.826*350*220}$ = 571.85 mm^2
- Use $A_{s\,min}$ 6 ∯ 12 / $m^\backslash$ (Vertical) in both side
- Use $A_{s\,min}$ 6 ∯ 12 / $m^\backslash$ (horizontal) in both side

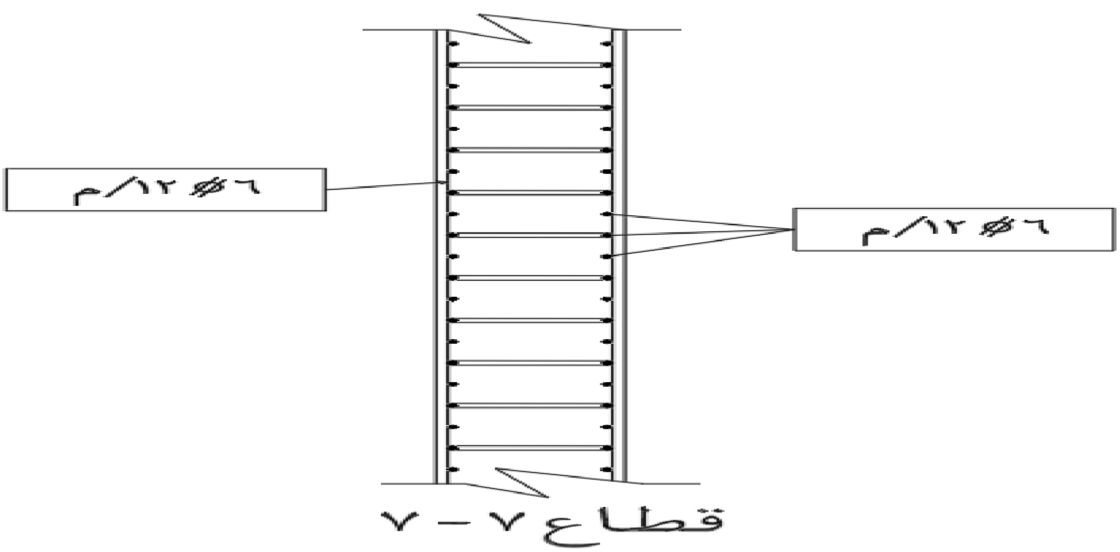
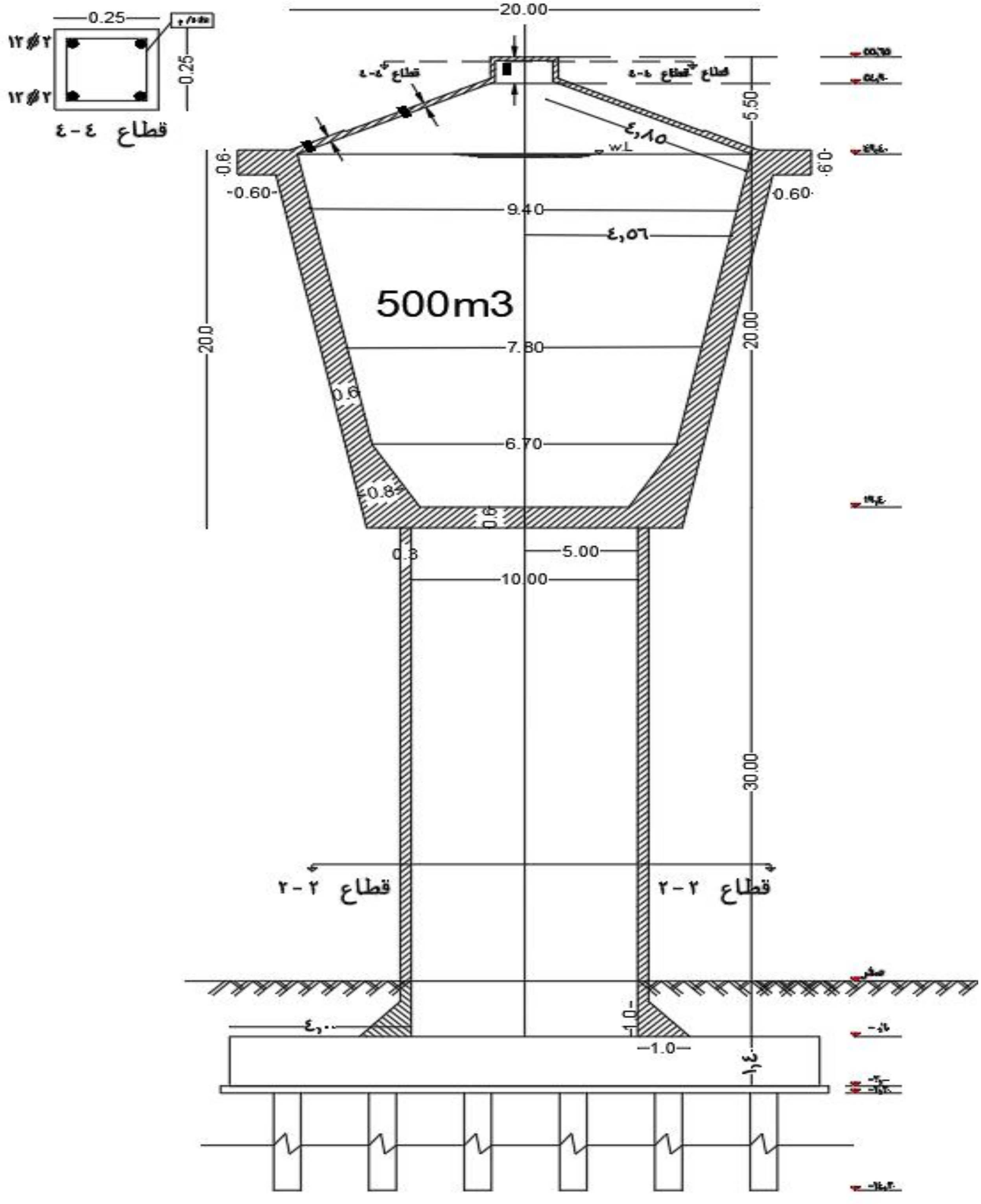


Figure 2.56 RFt of Retaining wall

Uinit (3)Elevated Tank



3.1 INTRODUCTION

3.1.1 Elevated tank consists of:

- Covering Cone
- Horizontal R-Ring Beam
- Vertical Ring Beam
- posts
- Covering Cone of Tank
- Circular Wall
- Conical Wall
- Shaft
- Tank Supporting Tower
- Foundation

3.1.2 Material Properties Used:

- Fcu=30 N/mm^2
- Fy(main steel) = 360 N/mm^2
- Fy(stirrups) = 240 N/mm^2
- Bearing Capacity of Soil = 10 t/m^2

3.1.3 Cover Thickness $\nless$ 5 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

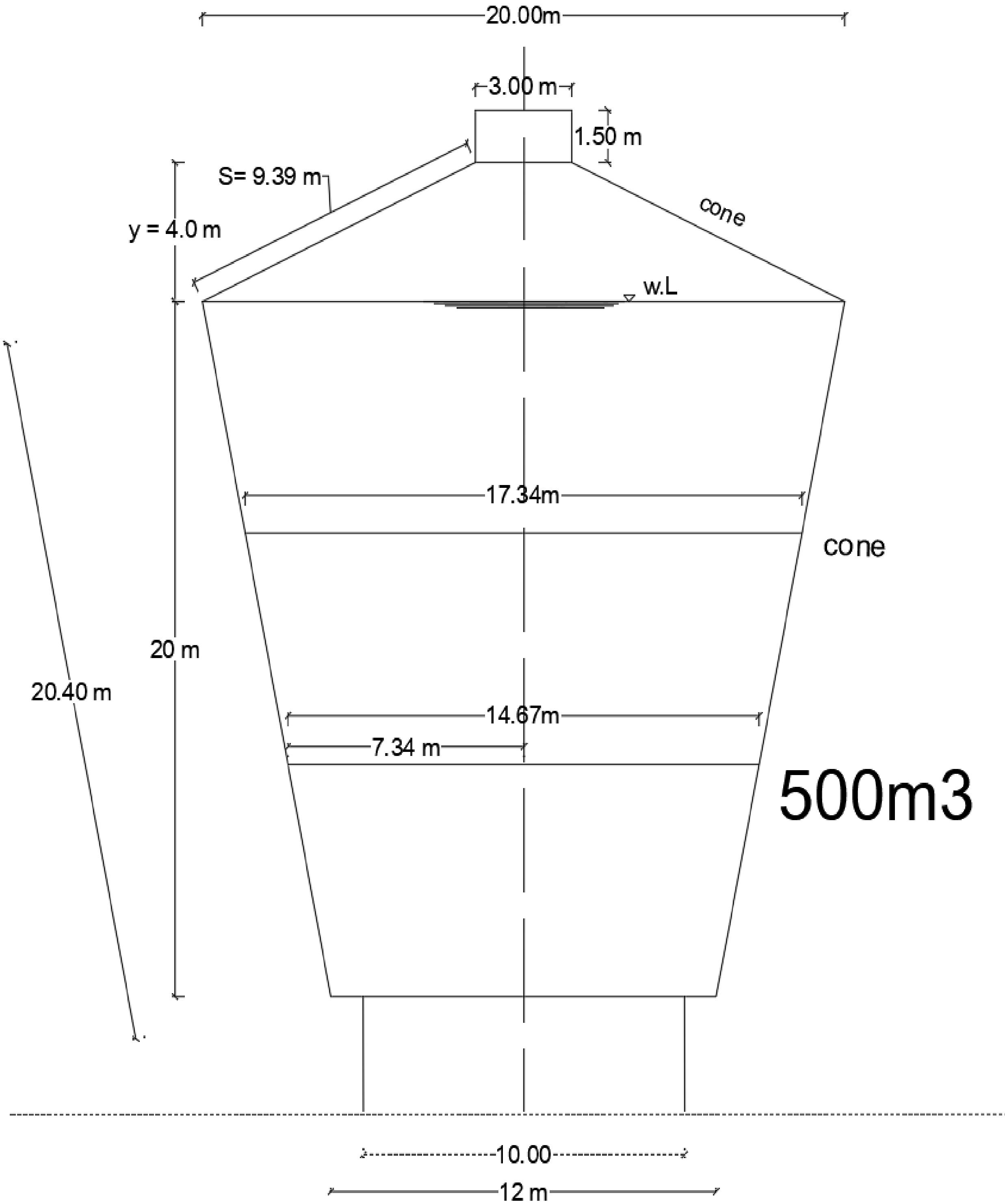
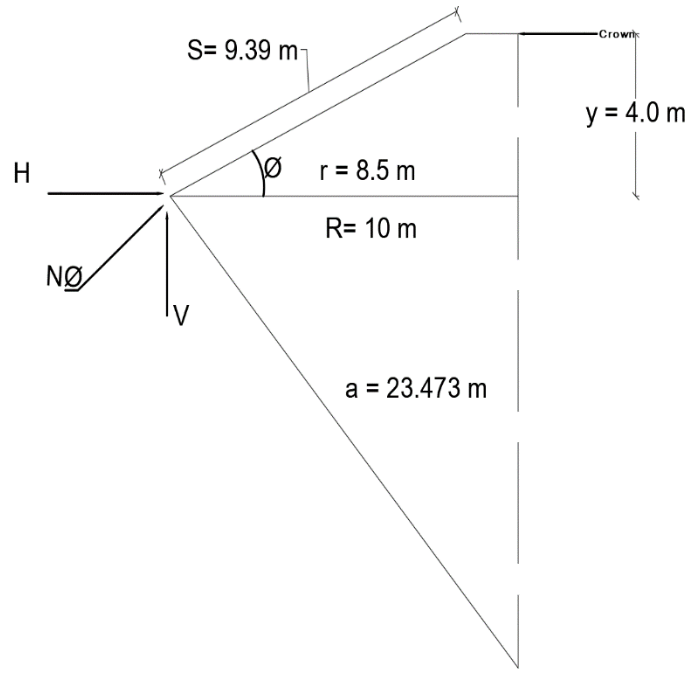


Figure 3.1 Elevated Tank Dimensions

3.3 Design

3.3.1 Covering cone

(1) Geometric Design



- D = 20.00 m
- r = $\frac{D}{2} = \frac{20}{2}$ = 10 m
- y = $\frac{r}{2.4} = \frac{10}{2.4}$ = 4 m
- S = $\sqrt{r^2 + y^2} = \sqrt{8.5^2 + 4^2}$ = 9.39 m
- a = $\frac{r.s}{y} = \frac{10*9.39}{4}$ = 23.475 m
- Tan$\varnothing = \frac{4}{8.5}$
- $\varnothing$ = 25.20 °
- Asume That:
  - T_s = 0.12 m
  - Covering = 0.05 t/m^2
  - L.L = 0.1 t/m^2

(2) Loads

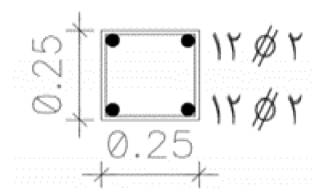
- W = D.L + L.L = own weight + Covering + L.L

  = (2.5 * 0.12) + 0.05 + 0.1 = 0.45 t/m^2

Posts

Height = 1.5 m
assume b_p = 25 cm
tp = 25 cm
No, of posts = 15

V (vertical reaction)) = $\frac{W \, . \, A}{\pi \, . \, D} = \frac{0.45 * \frac{\pi (3)^2}{4}}{\pi (3)}$ = 0.34 t/m



Load for Every post = $\frac{0.34 * \pi * 3}{15}$ + (2.5 * 0.25 * 0.25 * 1.5) = 0.44 ton

Figure3.2 Post reinforcement

As = $\frac{1.4 * 0.44 * 1000}{\frac{3600}{1.15}}$ = $0.2 \ cm^2$ USE 4 ∯ 12

(3) Internal Forces (Merdian Force & Ring Force)

At Crown

$N\theta = NØ = 0.0$ Safe

At Footing

- W Ø = W * $A_{surface}$
- W Ø = 0.45 * $\frac{2\pi rs}{2} = 0.45 * \frac{2\pi * 10 * 9.39}{2} = 132.747\ ton$

V1(vertical reaction) = $\frac{W\,Ø}{2\pi r} = \frac{132.747}{2\pi * 10}$ = 2.112 t/m\

N Ø = $\frac{V1}{SinØ} = \frac{2.112}{\sin(25.20)}$ = 4.95 t/m\ (Compression)

$N\theta$ = Pr * a = W Ø * Cos Ø * a = 0.45 * Cos (25.20) * 23.475 = 9.55 t/m\ (Compression)

(4) Check Of Stress

Fc = $\frac{N\,\theta}{bt} = \frac{9.55 * 10^3}{100 * 15}$ = 6.37 Kg/cm^2 $<$ Fcall = 60 Kg /cm^2 ⟶ ok safe

(5) Moment At Edge

X = 0.6 $\sqrt{a * t}$ = $0.6\sqrt{23.474 * 0.15}$ = 1.120 m

M = $\frac{Wx^2}{2} = \frac{0.45 * 1.12^2}{2}$ = 0.28 t.m Very small (neglected)

Use 6 ∯ 10 / m For ring and Merdian direction

3.3.2 Horizontal R-Ring Beam

$T = (H1 + H2) * r = (\frac{V}{\tan Ø} + \frac{V}{\tan Ø})$

$= (\frac{2.11}{\frac{4}{10}} + \frac{2.11}{\frac{20}{4}}) * 10 = 56.97$ ton

Assume b = 0.5 t

$A_c = b * T = 60\ T$

$0.5\ t^2 = 60 * 56.97$

$t = \sqrt{\frac{60*41.69}{0.5}} = 82.6 = 85$ cm

Use b = 60 cm , t = 120 cm

$A_s = \frac{T}{fs} = \frac{56.97*1000}{2000} = 28.48\ cm^2$

$A_{s\ (modified)} = \frac{As}{\beta cr} = \frac{28.48}{0.65} = 43.8\ cm^2$

Use 10 ∯ 25 $A_{s\ act} = 49.08$ cm

Use 5 ∯ 25 / one side

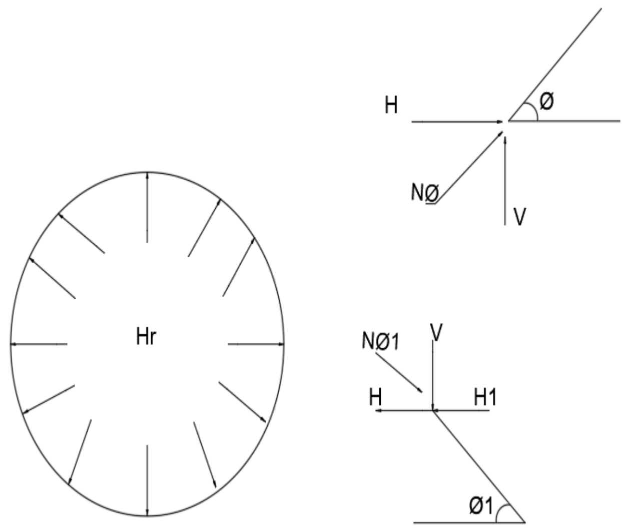


Figure 3.3 Ring Beam

(6) Check Of Ft

$Ft = \frac{T}{Ac + n\ As} = \frac{56.97*1000}{(60*120)+10(49.08)} = 7.41$ Kg / cm^2 < Fall = 18 Kg/cm^2 , ok safe

3.3.3 Desing of conical wall of the tank:-

ASSume t = 60 cm

W = 0.60 * 2.5 = 1.5 t/m

a1 = $\frac{r1.s}{y}$ = $\frac{10*6.8}{6.667}$ = 10.2 m

a2 = $\frac{r2.s}{y}$ = $\frac{8.67*6.8}{6.667}$ = 8.84 m

a3 = $\frac{r3.s}{y}$ = $\frac{7.34*6.8}{6.667}$ = 7.48 m

a4 = $\frac{r4.s}{y}$ = $\frac{6*6.8}{6.667}$ = 6.11 m

Ø1 = 78.69 °

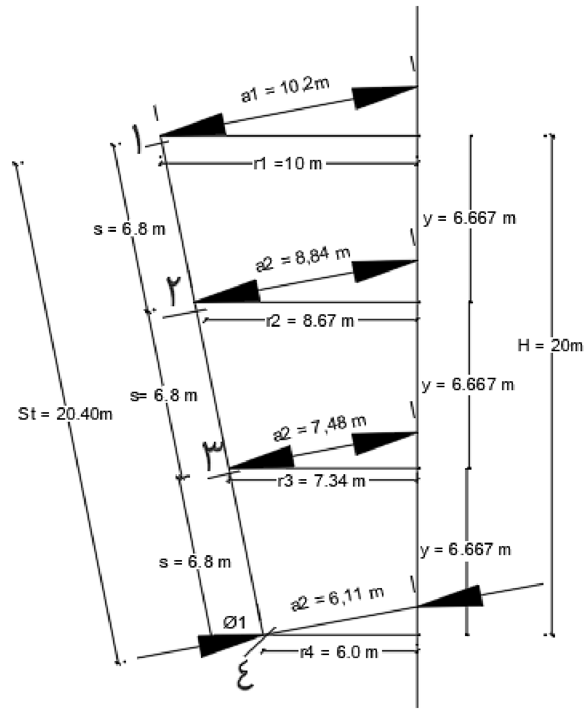


Figure 3.4 Conical wall

At sec (1) :- V1 = 2.11 t/m = Vertical reaction of the covering cone

$N\theta 1$ = (W * Cos Ø1 + γw h1) * a 1

= (1.50 * Cos (78.69)) * 10.2 = 3 t/ m (tension)

N Ø1 = $\frac{V1}{\sin Ø1}$ = $\frac{2.11}{\sin(78.69)}$ = 2.15 t/m (comp)

At sec (2) :-

$N\theta 2$ = (W * Cos Ø1 + γw h2) * a 2

= (1.50 * Cos (78.69) + 1 * 6.667) * 8.84 = 61.53 t/ m (tension)

W Ø2 = weight of cone above sec + weight of covering cone + weight of water above sec

Wcone = W * A_{sur} = Wπ (r1 + r2) * s = 1.5 * π (10 + 8.67) * 6.8 = 598.26 ton

Wcover = V1 * π D = 2.11 * π * 20 = 132.57 ton

Wwater = γw * Vw = $\gamma w * A1 *$ 2 π r_{cg} = 1* 0.5 * 1.13* 6.667 (2 π) (8.67 + $\frac{1.13}{3}$) = 214.12ton

W Ø2 = 598.26 + 132.57 + 214.12 = 944.94 ton

V2 = $\frac{W Ø2}{\pi r2}$ = $\frac{944.94}{\pi * 8.67}$ = 34.7 ton/m

N Ø2 = $\frac{V2}{\sin Ø1}$ = $\frac{34.7}{\sin(78.69)}$ = 35.38 t/m (comp)

At sec (3) :-

$N\theta 3$ = (W * Cos Ø1 + γw h3) * a 3
= (1.50 * Cos (78.69) + 1 * 13.334) * 7.48 = 101.93 t/ m (tension)

W Ø3 = weight of cone above sec + weight of covering cone + weight of water above sec

Wcone = W * A_{sur} = Wπ (r2 + r3) * s = 1.5 * π (8.67 + 7.34) * 6.8 = 513.02 ton
Wcover = V1 * π D = 2.11 * π * 20 = 132.57 ton
Wwater = γw * Vw = $\gamma w * A1 * 2\pi r_{cg}$ = 1* 0.5 * 2.66*13.334 (2 π) (7.34 + $\frac{2.66}{3}$) = 916.67 ton

W Ø3 = 513.02 + 132.57 + 916.67 = 1562.26 ton

$V3 = \frac{W\,Ø3}{\pi r3} = \frac{1562.26}{\pi * 7.34} = 67.74$ ton/m
$N\,Ø3 = \frac{V3}{\sin Ø1} = \frac{67.74}{\sin(78.69)} = 69.08$ t/m (comp)

At sec (4) :-

$N\theta 4$ = (W * Cos Ø1 + γw h4) * a 4
= (1.50 * Cos (78.69) + 1 * 20) * 6.11 = 123.99 t/ m (tension)

W Ø3 = weight of cone above sec + weight of covering cone + weight of water above sec

Wcone = W * A_{sur} = Wπ (r3 + r4) * s = 1.5 * π (7.34 + 6) * 6.8 = 427.47 ton
Wcover = V1 * π D = 2.11 * π * 20 = 132.57 ton
Wwater = γw * Vw = $\gamma w * A1 * 2\pi r_{cg}$ = 1* 0.5 *4 * 20 (2 π) (6 + $\frac{4}{3}$) = 1843.06 ton

W Ø4 = 427.47 + 132.57 + 1843.06 = 2400.1 ton

$V4 = \frac{W\,Ø4}{\pi r4} = \frac{2400.1}{\pi * 6} = 127.32$ ton/m
$N\,Ø4 = \frac{V4}{\sin Ø1} = \frac{127.32}{\sin(78.69)} = 129.841$ t/m (comp)

Calculate As

At sec (2)

$A_s = \frac{N\theta}{fs} = \frac{61.53 * 1000}{0.9 * 2000} = 34.18\ cm^2$

Use 16∯ 18 $A_{s\ act} = 40.7\ cm^2$

Use 8 ∯ 18 / one side

At sec (3)

$A_s = \frac{N\theta}{fs} = \frac{101.93*1000}{0.8*2000} = 63.70\ cm^2$

Use 18 ∯ 22 $A_{s\ act} = 69.11\ cm^2$

Use 9 ∯ 22 / one side

At sec (4)

As min ≥ 0.15%*b*d or 0.6*b*d/fy

As min ≥ 0.15%*1000*750) or 0.6*1000*750/360 =1125 mm^2 or 1250 mm^2

As min=1250 mm^2

As modify = 1250/0.85 = 1470.58 mm^2

Use 8 ∯ 18 / m A_sact = 20.35 cm2

3.3.4 Desing of Floor of the tank:-

(1) Loads

Assume t_f = 60 cm

$W = D.L + \Upsilon_w * h = \Upsilon_c * t_f + \Upsilon_w * h$

W= (2.5 * 0.6) + 1*20 = 21.5 t/m^2

Nf =NØ4 * CosØ1 = 129.841 / Cos(78.69) = 662 t

(2) Rdial moment of floor slab M_r at point 5&6

$M_{r5} = - 0.125\ WR_4^2 = -0.125 * 21.5 * 6^2 = - 96.75$ t.m (tension at inner face)

$M_{r6} = 0.075\ WR_4^2 = 0.075 * 21.5 * 6^2 = 58.05$ t.m (tension at outer face)

(3) Tangention moment of floor slab Mr at point 5&6

$M_{t5} = - 0.025\ WR_4^2 = 0.025 * 21.5 * 6^2 = -19.35$ t.m (tension at inner face)

$M_{t6} = 0.075\ WR_4^2 = 0.075 * 21.5 * 6^2 = 58.05$ t.m (tension at outer face)

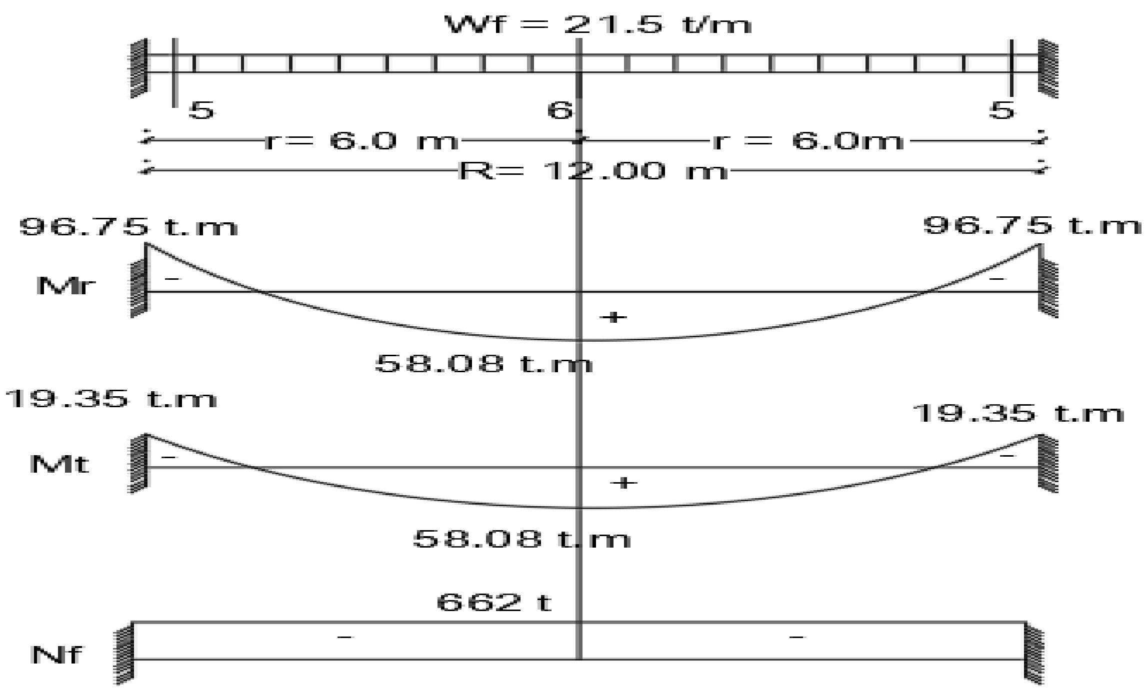


Figure 3.5 Moment in circular floor of Tank

(4) Design

Sec.5: Radial

M= 96.75 t.m ‘ N= -662 t

$t = \sqrt{\frac{M}{3b}} - 3cm = \sqrt{\frac{96.75*10^5}{3*100}} - 3cm = 179.58$ cm Use t = 180 cm ⇨ use hunch

∴ Tf = 80 cm

check of tensile strength (ft)

$F(t) = F_{ct}(N) + F_{ct}(M) < \frac{F_{ct}}{\eta_1 * \eta_1}$

$Ft = \frac{6M}{bt^2} - \frac{N}{bt} = \frac{6*96.75*10^5}{100*80^2} - \frac{662*10^3}{100*80} = 7.95$ Kg/cm^2 < Fcto = $\frac{Fctr}{\xi}$ = 17.6 Kg/cm^2

Calculate As

- $e = \frac{M}{N} = \frac{96.75}{662} = 0.146$ < $\frac{t}{2} - cover = \frac{0.80}{2} - 0.05 = 0.35$

- $es = e + \frac{t}{2} - cover$ = 0.146 + 0.4 – 0.03 = 0.516 m ⟹ Small eccentricity
- $K = \frac{P}{fc*b*t} = \frac{662*10^3}{73*100*80} = 1.133$ ⟹ this point under curve use minimum reinforcement for steel $360/520$
- As min ≥ 0.15%*b*d or 0.6*b*d/fy
- As min ≥ 0.15%*1000*750) or 0.6*1000*750/360 =1125 mm^2 or 1250 mm^2

  As min=1250 mm^2
- As modify = $\frac{1250}{0.85}$ = 1470.58 mm^2
- Use 8 ∯ 18 / m ⟹ A_{sact} = 20.35 cm^2

Sec.6: Radial

M= 58.08 t.m ‘ N= - 662 t

Calculate As

- $e = \frac{M}{N} = \frac{58.08}{662} = 0.087 < \frac{t}{2} - cover = \frac{0.80}{2} - 0.05 = 0.35$ ⟹ Small eccentricity
- $K = \frac{P}{fc*b*t} = \frac{662*10^3}{73*100*80} = 1.13$ ⟹ this point under curve use minimum reinforcement for steel $360/520$
- As min ≥ 0.15%*b*d or 0.6*b*d/fy
- As min ≥ 0.15%*1000*750) or 0.6*1000*750/360 =1125 mm^2 or 1250 mm^2

  As min=1250 mm^2
- As modify = $\frac{1250}{0.85}$ = 1470.58 mm^2
- Use 8 ∯ 16 /m ⟹ A_{sact} = 20.35 cm^2

Sec.5: Tangention

Calculate As

- d =C1 $\sqrt{\frac{M}{Fcu*b}}$ ⇨ 550 =C1 $\sqrt{\frac{19.35*1.5*10^7}{25*1000}}$ C1 =5.10 ⇨ j = .826
- As = $\frac{M}{Fy*j*d}$ = $\frac{19.35*1.5*10^7}{360*0.826*550}$ = 1774.70 mm
- As modify = $\frac{1774.70}{0.85}$ = 2087.88 mm^2
- Use 9 ∯ 18/m ⇨ A_{sact} = 2290.2 mm^2

Sec.6: Tangention

M= 58.08 t.m

- d =C1 $\sqrt{\frac{M}{Fcu*b}}$ ⇨ 550 =C1 $\sqrt{\frac{58.08*1.5*10^7}{25*1000}}$ C1 = 2.94 ⇨ j = 0.75
- As = $\frac{M}{Fy*j*d}$ = $\frac{58*1.5*10^7}{550*0.740*360}$ = 5858 mm^2
- Use 10 ∯ 20 /m ⇨ A_{sact} = 3141.5 mm^2

Section	M(t.m)	N(ton)	Stage	T (cm)	A_s
1	-------	-2.15	1	60	8 ∯ 16 /m
2	-------	-35.38	1	60	8 ∯ 18 /m
3	-------	- 69.08	1	60	8 ∯ 18 /m
4	96.75	-129.841	1	80	8 ∯ 18 /m
5(radial)	96.75	-662	1	80	8 ∯ 18 /m
6(radial)	58.08	-662	2	80	8 ∯ 16 /m
5(tang)	19.35	-------	1	60	9 ∯ 18 /m
6(tang)	58.08	-------	2	60	10 ∯ 20 /m

Figure 3.6 Table RFT Floor

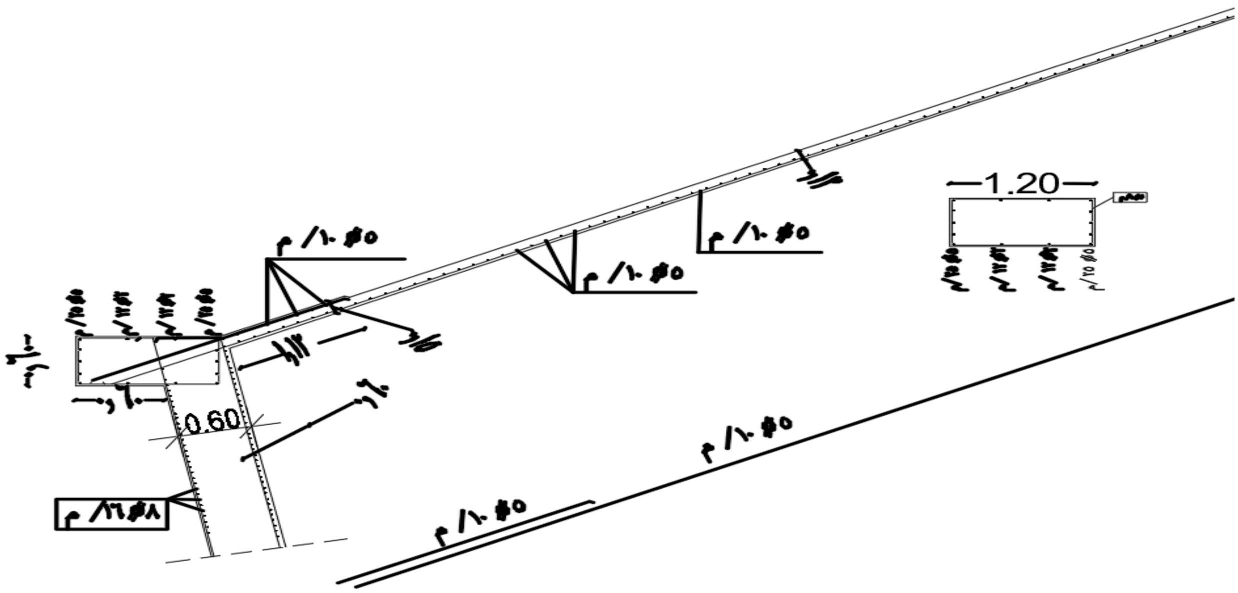


Figure 3.7 RFT details

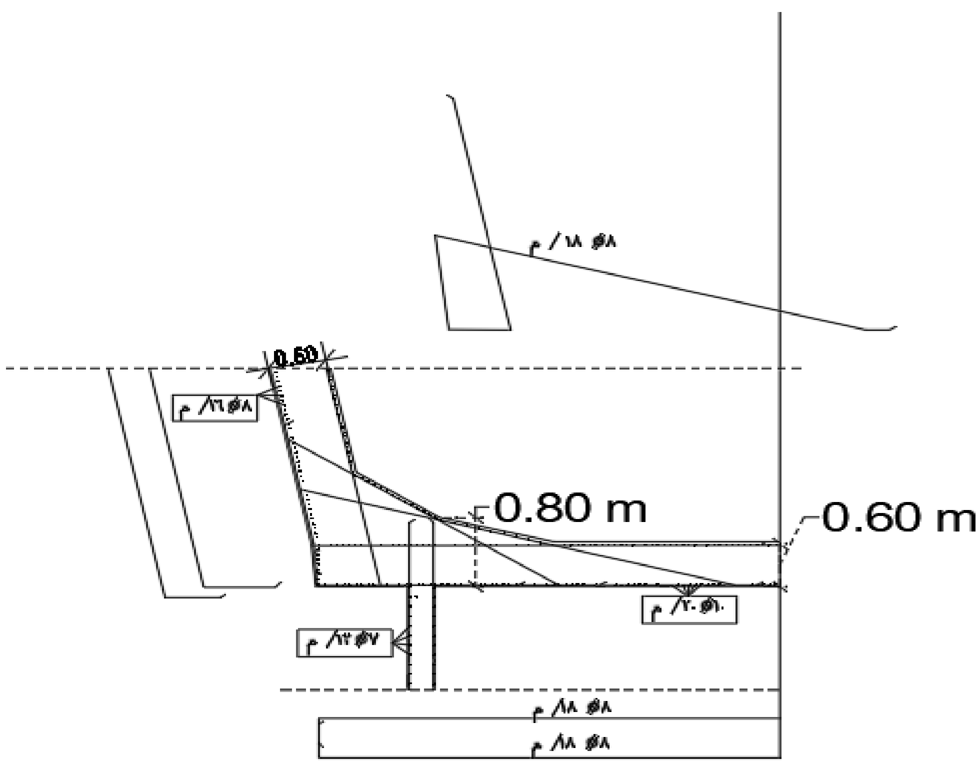


Figure 3.8 Floor RFT details

3.3.5 Design of water tower :-

- Assume thickness of shaft = 30 cm

Check of stability :-

- **Vertical load :-**
  - $W_{cone} = W * A_{surface} = 0.45 * 132.74 = 59.73$ ton
  - $W_{B1} = \Upsilon_C * t_b * B * \Pi\, d = 2.5 * 0.60 * 1.20 * \pi * 10 = 56.54$ ton
  - $W_{conical\ wall} = W * A = 2.5*0.45*\pi*\frac{9.7^2}{4} = 26.46$ ton
  - $W_{water} = \Upsilon_w * \pi_{wer} = 1*500 = 500$ ton
  - $W\ shaft = 2.5 * 30 * \frac{\pi * (10^2 - 9.40^2)}{4} = 685.65$ t

  Case of empty tank = $\sum$ vertical long with Wt of water

  = 828.38 ton ≈ 828.4 ton

  Case of Full tank = $\sum$ vertical long with Wt of water = 828.38 + 500 = 1328.38 ton

- **Horizontal loads: -**

  → earthquake load

$Fb = Sd\,(T1) * \lambda * \frac{W}{g}$ → ultimate base shear force

$T1 = Ct * H^{3/4} = 0.075 * (50)^{3/4} = 1.41$ sec

- Seismic zone: -
  Second zone ($ag = 0.125\ g = 0.125 * 9.81 = 1.23\ m/sec^2$)

- Soil type (c): -
  S = 1.50 , Tb = 0.10 , Tc = 0.25 , Td = 1.20

$$Tc \leq T1 \leq Td$$

$$Sd\,(T1) = ag * \Upsilon_I * s * \frac{2.5}{R} \left(\frac{Tc}{T1}\right) \geq 0.20\ ag * \Upsilon_I$$

- Υ_I (importance factor) = 1.2
- R (reduction factor) =9.40

$$Sd\,(T1) = 1.23 * 1.2 * 1.5 * \frac{2.5}{9.40} \left(\frac{0.25}{1.41}\right) = 0.1043 \geq 0.20 * 1.23 * 1.2 = 0.3$$

Use: -

- Sd(T1) =0.3

- $\lambda = 1 \rightarrow$ as T1 > 2Tc
- Wtank = 1328 ton

$$Fb = 0.3 * 1 * \frac{1328}{9.81} = 40.61 \text{ ton}$$

$Mo_{\text{ver turning}} = 40.61 * 50 = 2030.5$ m.t

$M_{\text{stability}}$= Wt. (without water) * R = 828 * 9.40 = 7783.2 m.t

$M_{\text{stability}}$= Wt. (with water) * R = 1328 * 9.40 = 12483.2 m.t

Check of overturning :

Factor of safety $= \frac{Mstability}{Movertuning}$

- F.O.S (with out water) $= \frac{7783.2}{2030.5} = 3.83 > 1.5$ OK ,safe
- F.O.S (with water) $= \frac{12483.2}{2030.5} = 6.147 > 1.5$ OK ,safe

Check the sliding :

- Resisting force = M* W_{total}
  - With out water = 0.3 * 828 = 248.4 ton
  - With water = 0.3 * 1328 = 398.4 ton

Factor of safety $= \frac{Resisting}{sliding\ force}$

- F.O.S (with out water) $= \frac{248.4}{40.61} = 6.11 > 1.5$ OK ,safe
- F.O.S (with water) $= \frac{398.4}{40.61} = 9.81 > 1.5$ OK ,safe

Wind load (height = 54 m)

$P_e = q * C_e * K$

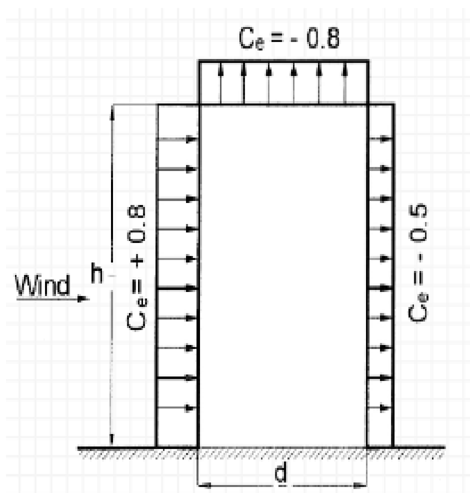
$q = 0.5 * 10^{-3} * P * V^2 * C_e * C_s$

$P = 1.25$ kg/m^3 $\rightarrow$ كثافة الهواء

V = 30 m / sec $\rightarrow$ سرعة الرياح الأساسية

$C_e = 1 \rightarrow$ معامل طبوغرافية الأرض

$C_s = 1 \rightarrow$ معامل المنشأ



$\therefore q = 0.5 * 10^{-3} * 1.25 * (30)^2 * 1 * 1 = 0.5625$ KN/m^2 $\rightarrow$ ضغط الرياح الأساسي

Ce = 0.8 + 0.5 = 1.3 → معامل ضغط الرياح→ Assume Zone (A) as more critical

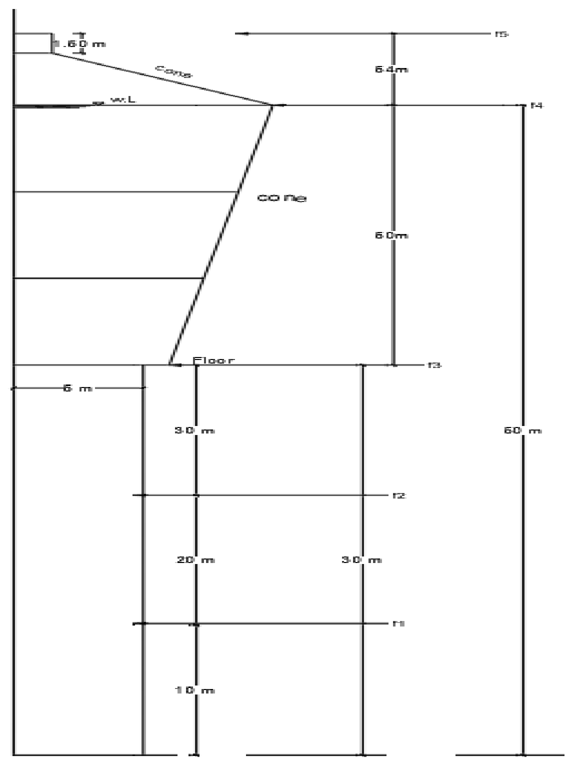
$Pe_1 = q * Ce * K_{0-10} = 0.5625 * 1.3 * 1 = 0.73125$ KN /m^2

$Pe_2 = q * Ce * K_{10-20} = 0.5625*1.3*1.15 = 0.84$ KN /m^2

$Pe_3 = q * Ce * K_{20-30} = 0.5625*1.3*1.4 = 1.024$ KN /m^2

$Pe_4 = q * Ce * K_{30-50} = 0.5625*1.3*1.6 = 1.17$ KN /m^2

$Pe_5 = q * Ce * K_{50-80} = 0.5625*1.3*1.85 = 1.353$ KN /m^2



<u>Fi = Pei * (hs*bs)</u>

F5 = 1.353 * 4 * 1.5 = 8.118 KN

F4 = 1.17 * 20 * 10 = 234 KN

F3 = 1.024 * 10 *6 = 61.44 KN

F2 = 0.84 * 10 * 6 = 50.4 KN

F1 = 0.73125 * 10 * 6 = 43.875 KN

Figure3.9 Wind load

$M_{over\ turning} = \sum f * h$

= (43.875 * 10) + (50.4 * 20) + (61.4 * 30) + (234 * 50) + (8.118 * 54)

= 15427 KN.m = 1542.7 ton.m

$M_{over\ turning\ (wind\ load)} < M_{over\ turning\ (earthquakes)}$ → wind load is safe

3.3.6 Check of stresses between shaft and foundation :-

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

N = 1328 ton

M_x = 2030.5 m.t

A $=\frac{\pi*(10^2-9.40^{\ 2})}{4}$ = 9.14 m^2

Ix = $\frac{\pi*(10^4-9.40^4)}{64}$ = 107.62 m^4

Y = 5 m

F1,2 =$\frac{-1328}{9.14} \pm \frac{2030.5}{107.62} * 5$

F1= - 50.95 t/m^2= - 5.095 kg/cm^2 < 131.25kg/cm^2

F2=- 239.63 t/m^2 = - 23.963 kg/cm^2 < 131.25kg/cm^2

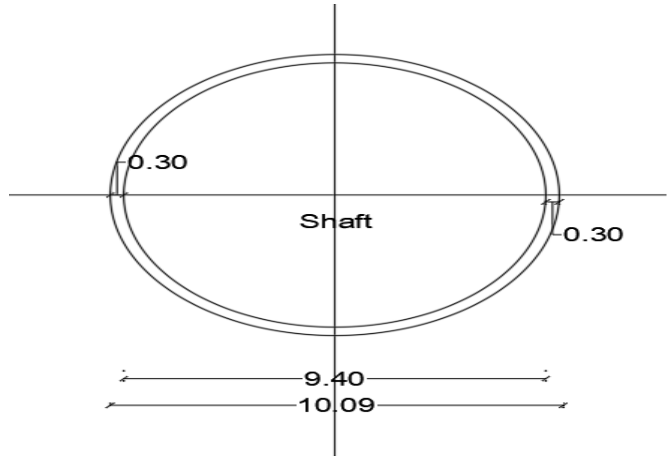


Figure 3.10 Dimensions Shaft

Ok,Safe,no tension

3.3.7 Design of Shaft

P = 1328 t
M = 2030.5 m.t
r = 5 m

rs` = 4.70 m

r` = r – cover = 5 - 0.05 = 4.95 m

Ac = $\pi * (r^2 - rs`^2)$= $\pi * (\ 5^2 - 4.70^2)$= 9.14 m^2

$\frac{P_u}{f_{cu}*A_c} = \frac{1328*10^3}{250*9.14*100^2}$ =0.058

$\frac{M_u}{f_{cu}*A_c*r} = \frac{2030.5\ *10^5}{250*9.14*100^2*500}$ =0.0177

$\eta = \frac{r`}{r} = \frac{4.95}{5}$ = 0.99

$\frac{rs`}{r} = \frac{4.70}{5} = 0.94$

$\mu = ∮ * f_{cu} * 10^{-4} = 2 * 25 * 10^{-4} = 0.005$

$\mu_{min} = 0.008$

As total = μ_{min} * Ac= 0.008* 9.14 * $(100)^2$ = 731.2 cm^2

Use 250 ∯ 20   As = 785.39 Cm^2

As/side = 392.69 Cm^2 → 125 ∯20 → use 7 ∯20 / m` (for vertical bars)

Use 7 ∯ 12 / m` (for horizontal bars) …… stirrups

3.3.8 Design of foundation

Assume t_f = 1.4 m

Wt = Wtank + O.Wraft = 1328+ 2.5 * 1.40 * $\frac{\pi*(10)^2}{4}$ = 1602.88 t

NO.of piles = (1.2:1.4) * $\frac{W_t}{pile\ capacity}$ = 1.2 * $\frac{1602.88}{100}$ = 19.2 = 19 piles

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

N = 1328 t

M = 2030.5 m.t

n = use 25 pile

$\sum r^2$ = $(12 * 5.62^2) + (9 * 4.20^2) + (4 * 1.76^2)$ = 532.52 m^2

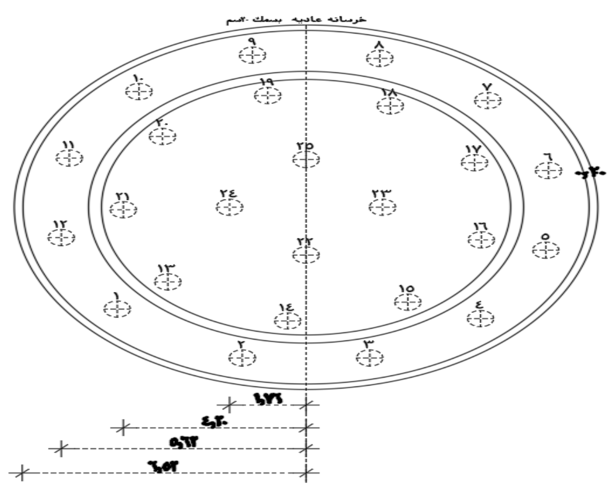


Figure 3.11 PileS

For point (1)

$F1,2 = \frac{-1328}{25} \pm \frac{2030.5}{532.52} * 5.62$

F1= -31.69 t < 100 t

F2= -74 t < 100 t

For point (2)

$$F1,2 = \frac{-1328}{25} \pm \frac{2030.5}{532.52} * 4.20$$

F1= -37.105 t < 100 t

F2= -69.13t < 100 t

For point (3)

$$F1,2 = \frac{-1328}{25} \pm \frac{2030.5}{532.52} * 1.76$$

F1= -46.40 t < 100 t
F2= -59.83 t < 100 t

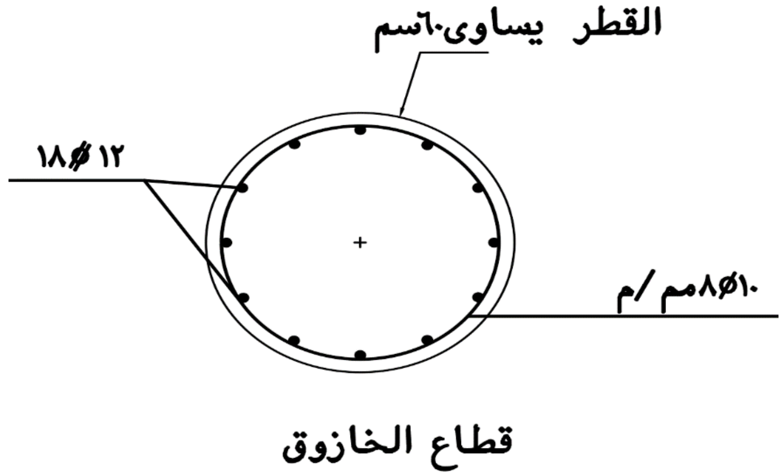


Figure 3.12 sec Pile

For Raft:

$$As = \frac{Mu}{fy * j * d}$$

$$Mu = As * fy * j * d = 8 * \frac{\pi * 2 * 2}{4} * 3600 * 0.826 * 133 * 10^{-5} = 99.39\ t.m$$

$$As_{act} = \frac{Mu}{fy * j * d} = \frac{99.39 * 10000}{360 * 0.826 * 133} = 25.132\ cm^2$$

$$A\grave{s} = \frac{0.15}{100} * b * d = \frac{0.15}{100} * 1000 * 1330 = 1995 = 19.95\ cm^2$$

Use

8 ∯ 20 / m` upper reinforcement

8 ∯ 20 / m` lower reinforcement

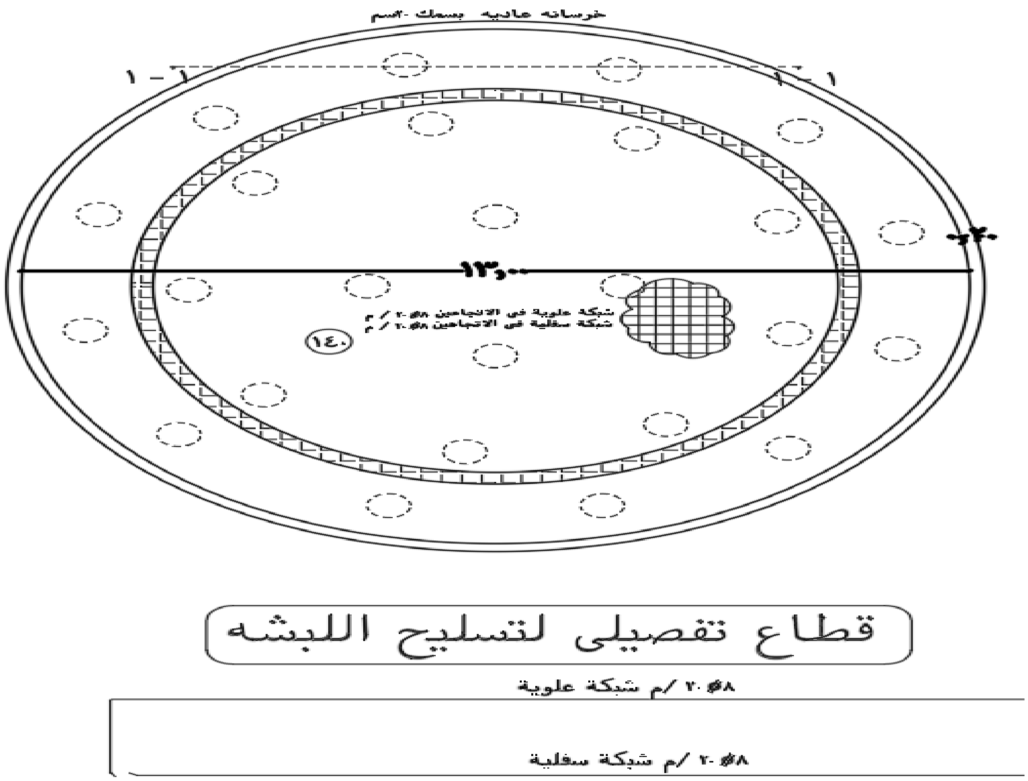


Figure 3.13 Raft RFT details

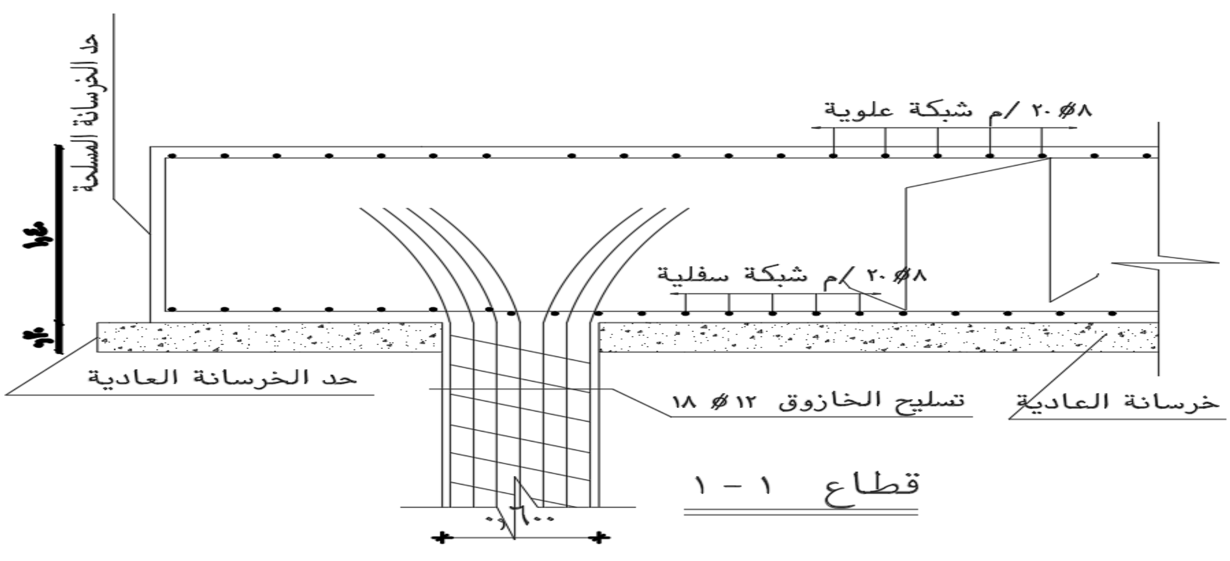
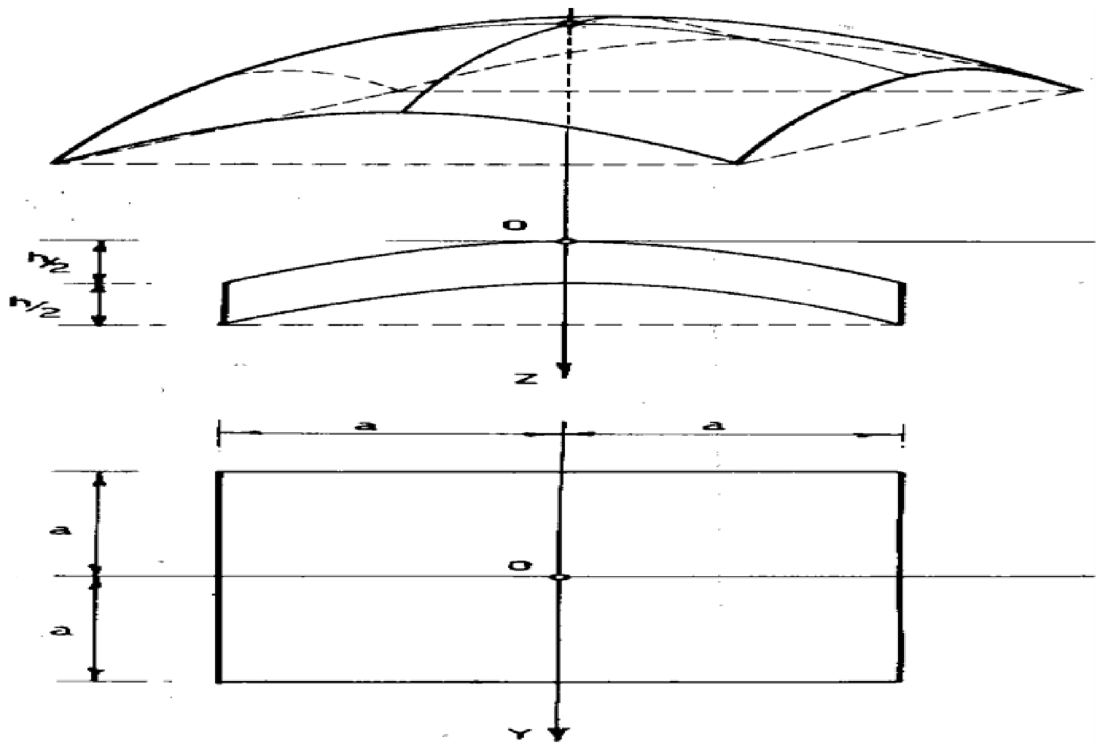
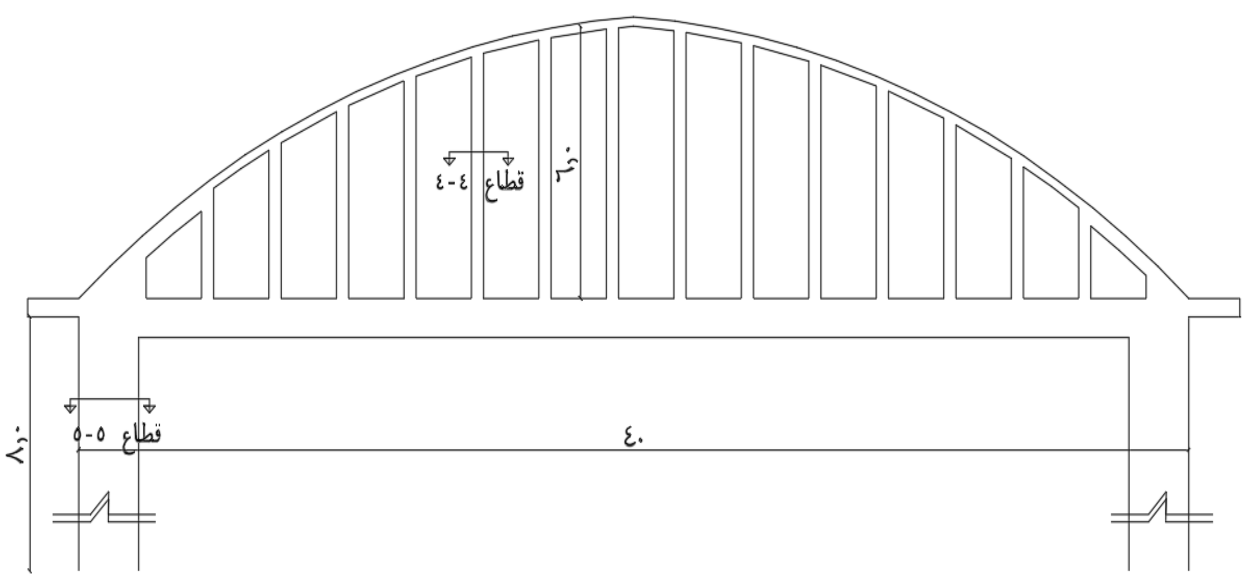


Figure 3.14 Raft RFT details

Uinit (4) Reinforced Concret Hall
parabolic shell





4.1 INTRODUCTION

4.1.1 Material Properties Used:

- F_{cu}=250 kg/cm2 , Fc = 60 kg/cm2
- Fy(main steel)=3600 kg/cm2 , Fs = 2000 kg/cm2
- Fy(stirrups)=2400 kg/cm2
- Weight of used brick =1400 kg/m3
- Bearing Capacity of Soil = 1.0 kg/m2
- passion ratio=0.2
- Ec=22000 mpa

4.1.2 Cover Thickness

- Columns Cover = 2.5 cm
- Foundations Cover = 5 cm
- Posts Cover = 2.5 cm
- Hanger Cover = 2.5 cm

4.1.3 Loads Used:

- L.L= 0.1 t/m2
- Cover = 0.05 t/m2
- D.L = Own weight + Covering Material

4.2 Concrete dimensions: -

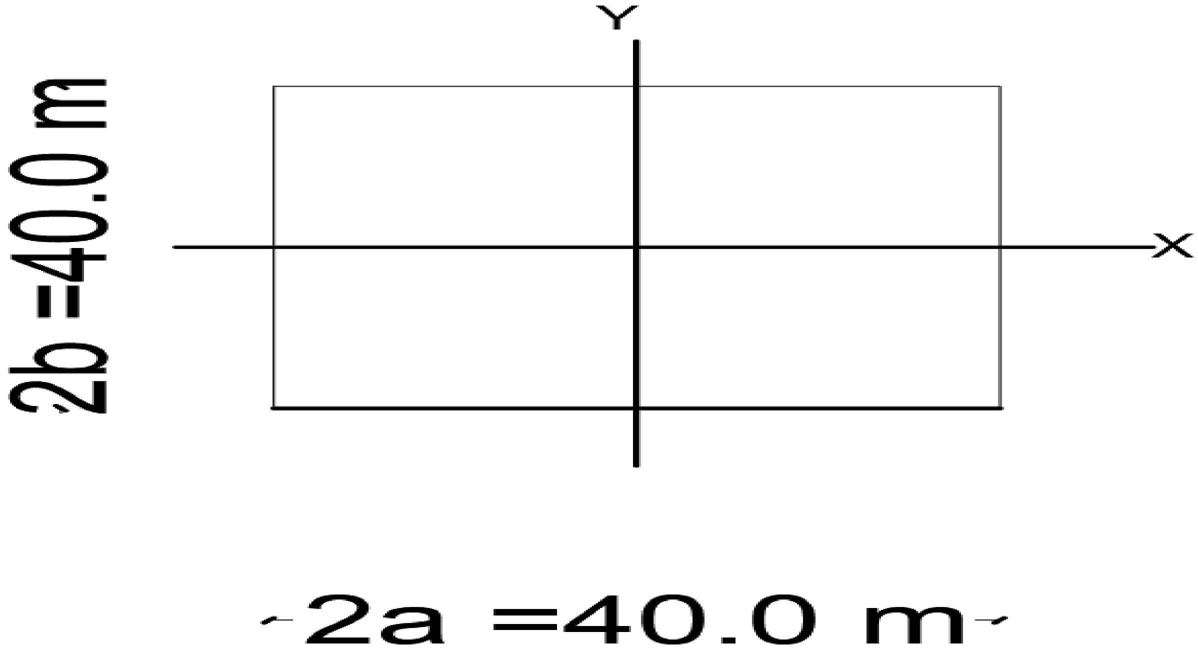


Figure 4.1 Concrete dimensions

a = 20 m
L = 2a = 40 m

b =20 m
L = 2b = 40 m

ts = 10 cm

hx = hy = $\frac{Lsh}{6-8} = \frac{40}{7} = 5.7 \approx 6$ m

Total Load Surface (P) = D.L + L.L
= Own weight + Covering Material + L.L

= (2.5 * 0.1 + 0.05) + 0.1 = 0.4 t / m` = 400 Kg /m`

r = a = $\frac{a2+y2}{2y} = \frac{202+32}{2*3}$ = 68.2 m

The constant factor for detrmining the force is

$\frac{4\,P\,r}{\pi 3} = \frac{4*0.40*68.2}{\pi 3} = 3.52$ t / m`

the maximum compressive Force at the middle of the shell

$N_x = N_y = -\,3.84 * 3.52 = -13.516$ t / m`

the maximum compressive Force at the zone adjacent of the edge

$N_y = -\,7.30 * 3.52 = -25.696$ t / m`

the maximum principal compressive and principal tensile force and the share Force at the corner of the shell

$-\,N_2 = N_1 = N_{xy} = 12.68 * 3.52 = 44.63$ t / m`

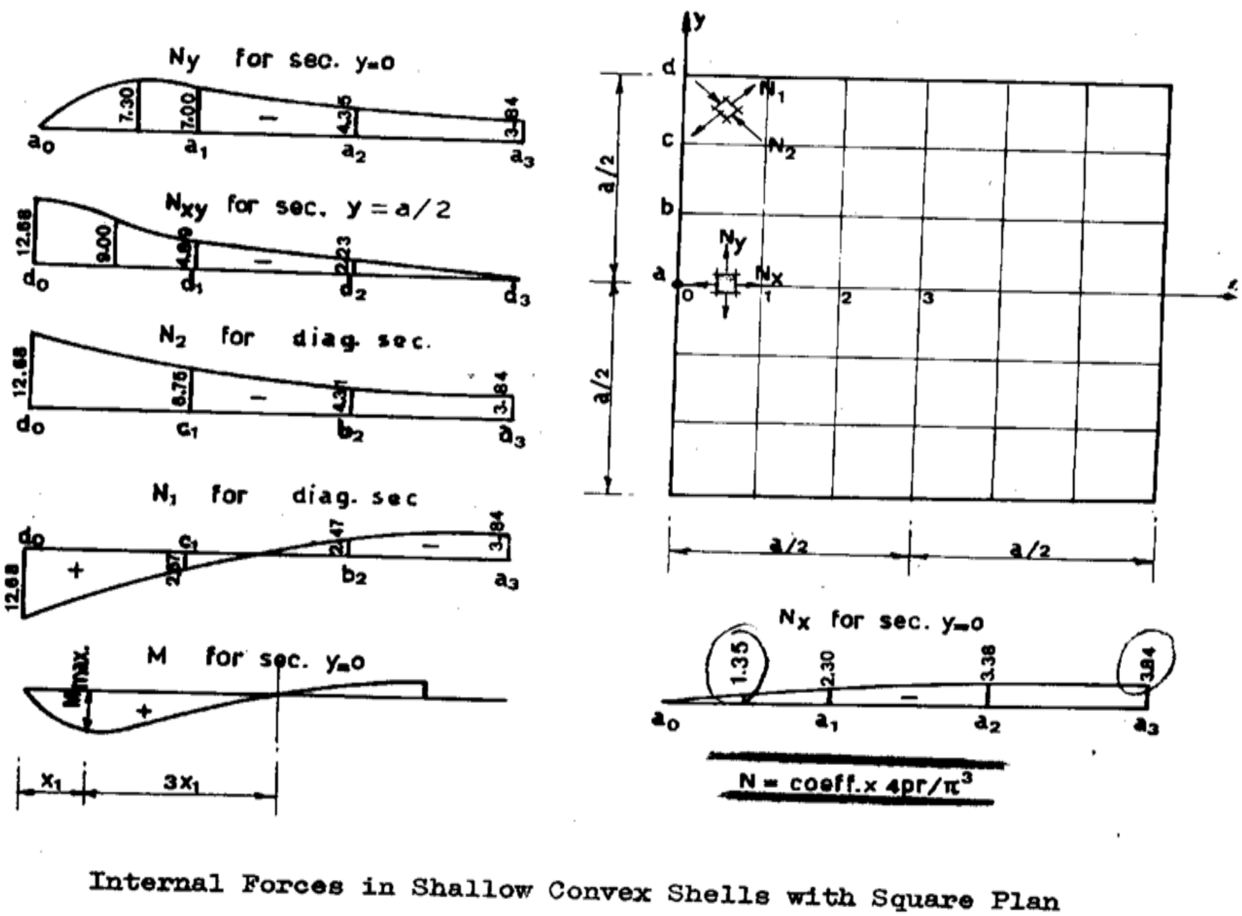


Figure 4.2 Internal forces in shallow convex shalls with square plan

4.3 Check of Stresses

- **Maximum compressive stress at corner**

Fc= $\frac{N_x}{bt}$ = $\frac{13.516*1000}{10*100}$ = 13.516 kg/cm^2 < $F_{c\ all}$ = 60 kg/cm^2 ,, Ok

- **Maximum compressive stress at center**

Fc= $\frac{N_{x0}}{bt}$ = $\frac{25.696 *1000}{10*100}$ = 25.696 kg/cm^2 < $F_{c\ all}$ = 60 kg/cm^2 ,, Ok

- **Principle compressive stress at corner**

Fc= $\frac{N_{xy}}{bt}$ = $\frac{44.63*1000}{10*100}$ = 44.63 kg/cm^2 < $F_{c\ all}$ = 60 kg/cm^2 ,, Ok

4.4 Moment :-

ts = 10 cm

X = 0.60 $\sqrt{rt}$ = 0.60 (68.2 * 0.10)$^{0.50}$ = 1.57 m

∴ M = 0.094 *r * t * p = 0.094 * 68.2 * 0.10 * 0.40 = 0.256 = 0.3 t.m/m

The reinforcement is generally one mesh 6 ∯ 10 in each direction

the compressive Force N_X in this section can be determined by linear interpolation thus

N_X = Coeff * $\frac{4\,P\,r}{\pi 3}$ * X * 12 /40 = -1.35 * 3.52 * 1.57 * 12 /40 = -2.238 t/m

Due to the high principal compressive and tensile stresses at the corners, it is recommended to increase the thickness of the ahell gradually from 10 to 16 cms along a length of about 0.15 measured along the diagonal

The maximum compressive stresses :-

σ_c = -16700 /100*10 = -16.7 Kg /cm^2
σ_c= -31800 /100*10 = - 31.8 Kg /cm^2
σ_c = -55200 / 100 * 16 = - 34.8 Kg /cm^2

The diagonal tension reinforcement on both Sides at the corner can be calculated as follows

$A_{s1} = \frac{55+44}{2\sigma_S} = \frac{55+44}{2*1.4} = 35\ cm^2$ /m

$A_{s2} = \frac{44+34}{2\sigma_S} = \frac{44+34}{2*1.4} = 28\ cm^2$ /m

$A_{s3} = \frac{34+26}{2\sigma_S} = \frac{34+26}{2*1.4} = 22\ cm^2$ /m

$A_{s4} = \frac{26+20}{2\sigma_S} = \frac{26+20}{2*1.4} = 16\ cm^2$ /m

$A_{s5} = \frac{20+11.6}{2\sigma_S} = \frac{20+11.6}{2*1.4} = 11.2\ cm^2$ /m

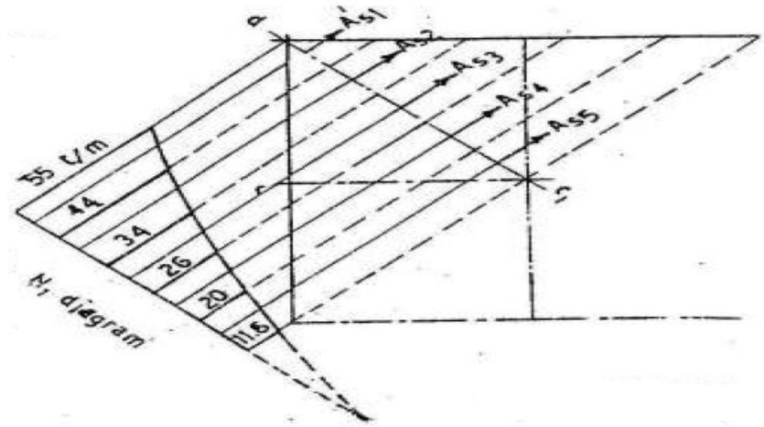


Figure 4.3 The diagonal tension reinforcement

4.5 Concrete Dimensions of System :

Parabolic Shell Thickness (ts) = 10 cm

1) Tie

- assume b = 40 cm
- t = 70 cm

2) Hanger

- assume b = 25 cm
- t = 25 cm

3) Main Girder

- assume b = 40 cm
- t = $\frac{span}{30-35} = \frac{400}{30-35} = 140\ cm$

4.5.1 Design of Girder

Weq = 1.4 o.w Girder +∑v+ W tie + W Hanger

- Own Weight Of Girder = 1.4 * 2.5 * 0.4 * 1.40 = 1.96 t/m
- Load From Hanger = W Hanger = 1.4 (2.5*0.25*0.40*6) = 2.1 t

W tie = 1.4 (2.5 * 0.4 * 0.7 * 3.33) = 3.26 ton
W tie + W Hanger = 3.26 + 2.1 = 5.36 ton

Load From Slab

- S = Nxy * L =44.63 * 20.62 = 373.14 ton
- H = S cosθ = 373.14 * $\frac{20}{20.62}$ = 361.92 ton
- V = S sinθ = 373.14 * $\frac{5}{20.62}$ = 90.5 ton

- $Wu = \frac{5.36*9}{40} + 1.96 + \frac{90.5}{40} = 7.69$ t/m

  Mu = 0.05 * $\frac{wL^2}{8}$ =0.05 * $\frac{7.69*40^2}{8}$ = 76.91 t.m

- Nu = 0.95 * $\frac{wL^2}{8F}$ = 243.516 t (comp)
- e = $\frac{Mu}{Nu} = \frac{76.91}{238.45} = 0.315 < \frac{t}{2}$ small eccentricity
- $\frac{Nu}{Fcu* b*t} = \frac{243.516*10^3}{250*40*140} = 0.17$
- $\frac{Mu}{Fcu*b* t^2} = \frac{76.91* 10^5}{250*40* 140^2} = 0.039$

From chart

ρ = 1

- μ = ρ * Fcu * 10^{-4} = 1 * 25 * 10^{-4} = 25 * 10^{-4}
- As = A$s^\backslash$ = μ *b * t = 25 * 10^{-4} * 400 * 1400 = 1400 mm^2
- Astotal = 2800 mm^2
- As min = $\frac{0.8}{100}$ * b * t
- = $\frac{0.8}{100}$ * 400 * 1400 = 4480 mm^2
- As = A$s^\backslash$ =$\frac{4480}{2}$ = 2240 mm^2

  use As min = 2240 mm^2

- use 24 ∯ 12

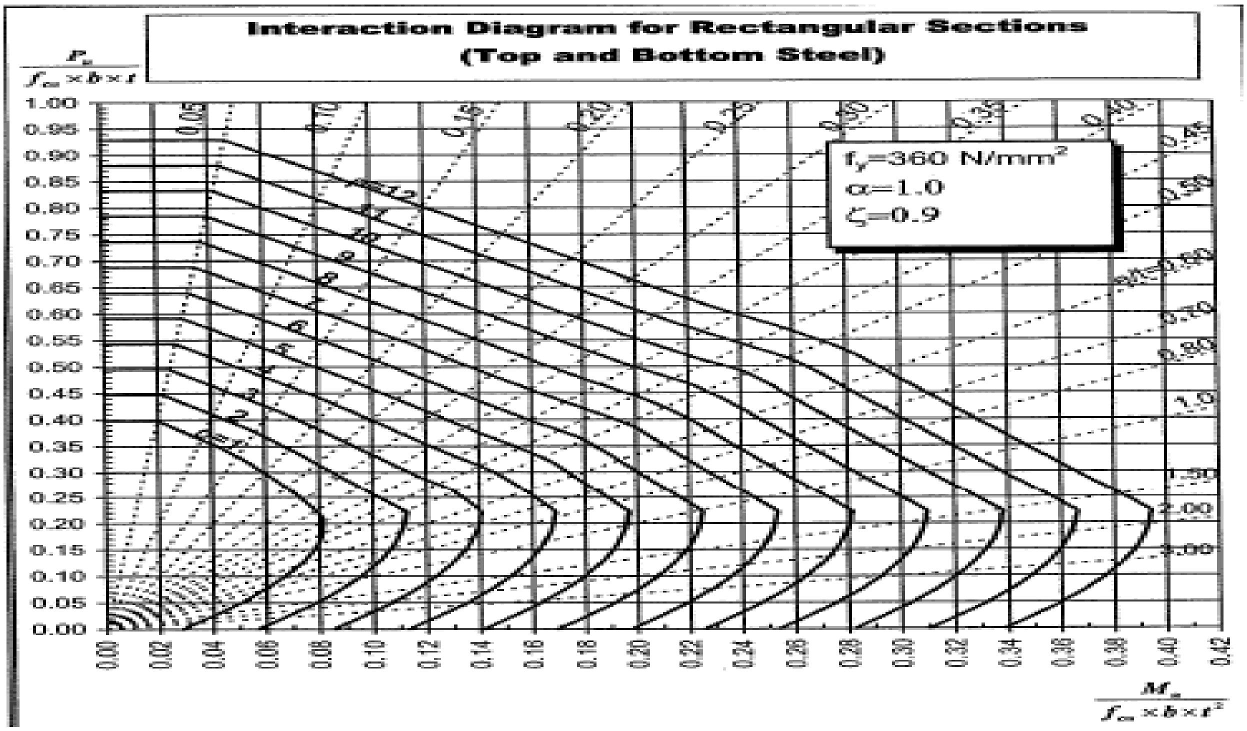


Figure 4.4 Interaction Diagram

4.5.2 Design of Tie

P =Rx = $\frac{0.95\ WL^2}{8h}$ = 243.51 ton

As = $\frac{P}{\frac{Fy}{ys}} = \frac{243.51*10^4}{\frac{3600}{1.15}}$ = 777.9 mm^2 = 7.77 cm^2

use 4 ∯ 18

4.5.3 Design of Hanger : -

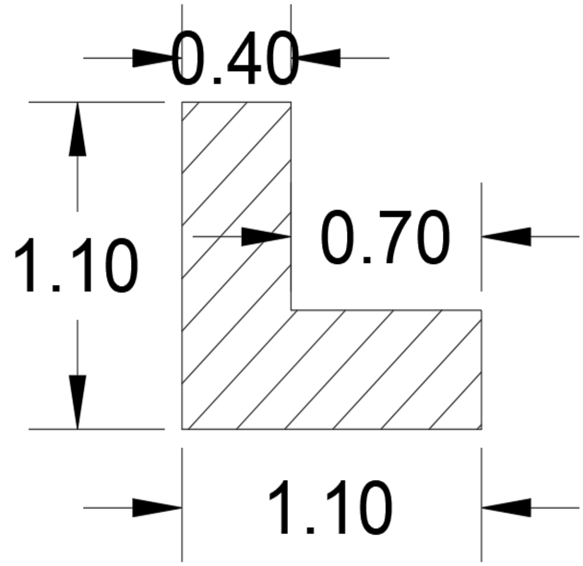
Load From Hanger = 1.4 (O.W Of Hanger + O.W Of Tie)

= 1.4 (2.5 * 0.4 * 0.25 * 5 + 2.5 * 0.4 * 0.8 * 4) = 2.198 ton

As = $\frac{2.198*10^4}{\frac{3600}{1.15}} = 7.021\ mm^2$

use 4 ∯ 16

4.5.4 Design of column (40*110): -



$P = \frac{Weq*L}{2} = \frac{7.69*40}{2} = 153.8$ t

$P_{u\ actual} = 0.35\ (A_c\ )F_{cu} + 0.67\ A_s\ F_y$

assume $A_s = 0.01\ A_c$

$154 * 10^4 = 0.35* 0.72 * 25 + 0.67 * 360 *A_s$

$A_s = 63.82$ cm2

USE 24 ∯ 20

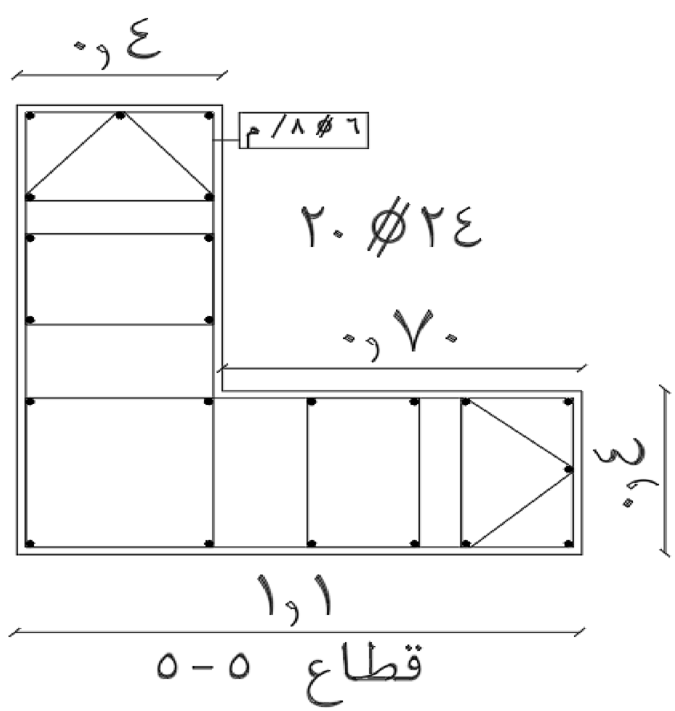


Figure 4.5 RFT details

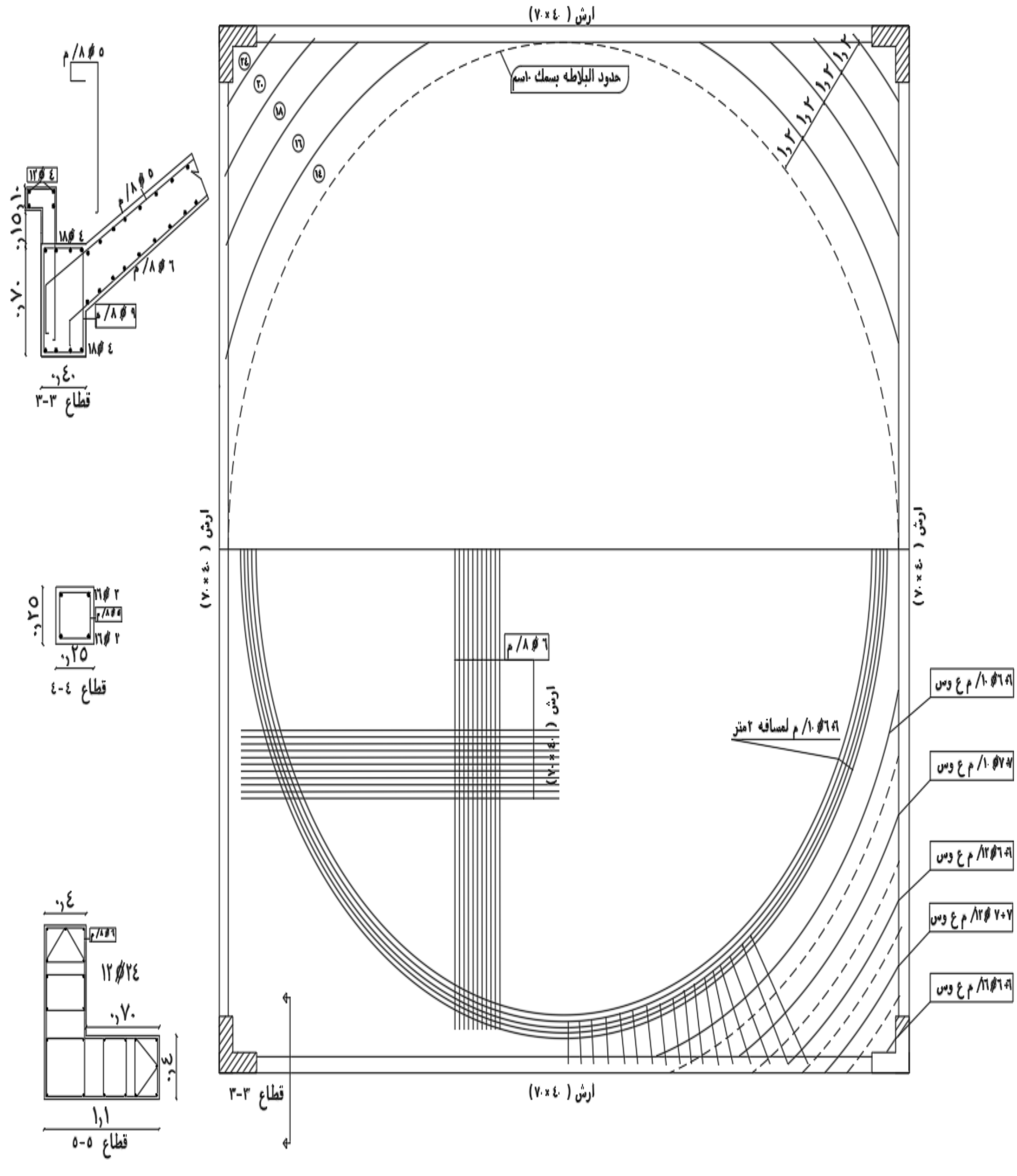


Figure 4.6 RFT details

4.5.5 Design of Foundation : -

Column Working Load = 153.7 ton
Column Dimension a = 40 cm
b = 110 cm

Plain Concrete Depth = 0.4 m

Plain Concrete Extension = 0.40 m

1 Concrete Dimension :

- B.C = $\frac{153.7}{A_{pc}}$
- 10 = $\frac{106.7}{A_{Pc}}$
- A_{Pc}= 15.31 m^2 = Lpc *Bpc
  = (1.10 + 2c) * (0.40 + 2c)

  C = 1.58 m
- Use take Lpc = 4.15 m
- Bpc = 4.15 m

∴ L_{RC} = 3.75 m

B_{RC} = 3.75 m

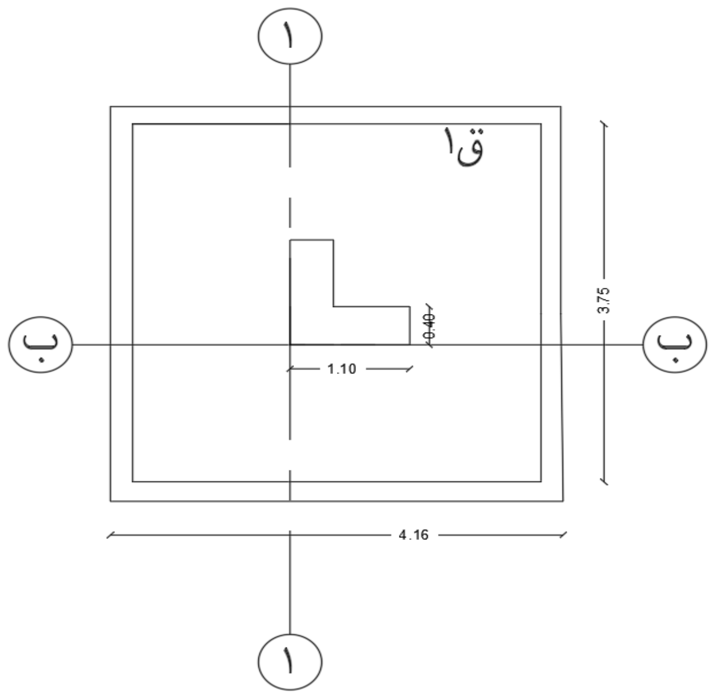


Figure 4.7 Section of footing (elevation)

2 Check of Stress:

- $q_{act} = \frac{153.7}{A_{PC}}$
- $q_{act} = \frac{153.7}{4.15*4.15}$ = 8.92 t/m^2 < B.C = 10 t/m^2, Ok

$q_{act} = \frac{P_U}{A_{R.C}}$ $\frac{153.7}{3.75*3.75}$ = 109.3 KN

Z= $\frac{3.75-1.10}{2} = 0.63$

$M_U = \frac{q_{act} *Z^2}{2} = \frac{109.3 *1^2}{2} = 54.65$ KN.m

d =$c1 \sqrt{\frac{M_U}{FCU*b}} = C1 \sqrt{\frac{54.65*10^6}{35*1000}}$ ➡ d ≈ 500 ➡ t = 600

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

- Qp = 1537 – 109.3 * (0.4 + 0.6)*(1.10 + 0.6) = 1351.19 KN

- $q_p = \frac{Qp}{2((a+d)+(a+d))*d}$
- $q_p = \frac{1351.19 * 10^3}{2((400+600)+(1100+600))*600}$ = 0.417 N/mm^2

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

2- 0.316 *(0.5+$\frac{a}{b}$) * $\sqrt{\frac{F_{CU}}{\gamma_C}}$ = 0.316*(0.5 + $\frac{0.4}{1.10}$) * $\sqrt{\frac{25}{1.5}}$ = 1.11 Nlmm^2

3- 0.316* $\sqrt{\frac{F_{CU}}{\gamma_C}}$ = 0.316* $\sqrt{\frac{25}{1.5}}$ = 1.29 Nlmm^2

4- 0.8*($\frac{\propto * d}{2(a+d+b+d)}$ +.2)* $\sqrt{\frac{F_{CU}}{\gamma_C}}$ =

$= 0.8\ (\frac{4*600}{2(400+600+1100+600)} + 0.2)*\sqrt{\frac{25}{1.5}} = 2.1\ \text{N/mm}^2$

$q_p < q_{p\ (allow)}$ ➨ OK, safe

3 Check of Shear

$Qsh_1 = q_{act}$ * c = 109.3 * 1.58 = 172.69 ton

$qs_h = \frac{Qsh}{B*d} = \frac{172.69 * 10^3}{3750 * 600}$ = 0.1 KN / mm^2

$q_{sh(allow)}$ = 0.16 * $\sqrt{\frac{F_{CU}}{\gamma_C}}$ = 0.16 * $\sqrt{\frac{25}{1.5}}$ = 0.65 KN / mm^2

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

$A_S = \frac{M_U}{F_Y * J * d} = \frac{54.65 * 10^6}{360*0.826*600}$ = 306.30 mm^2

A_S = 9 ∯ 12 / m

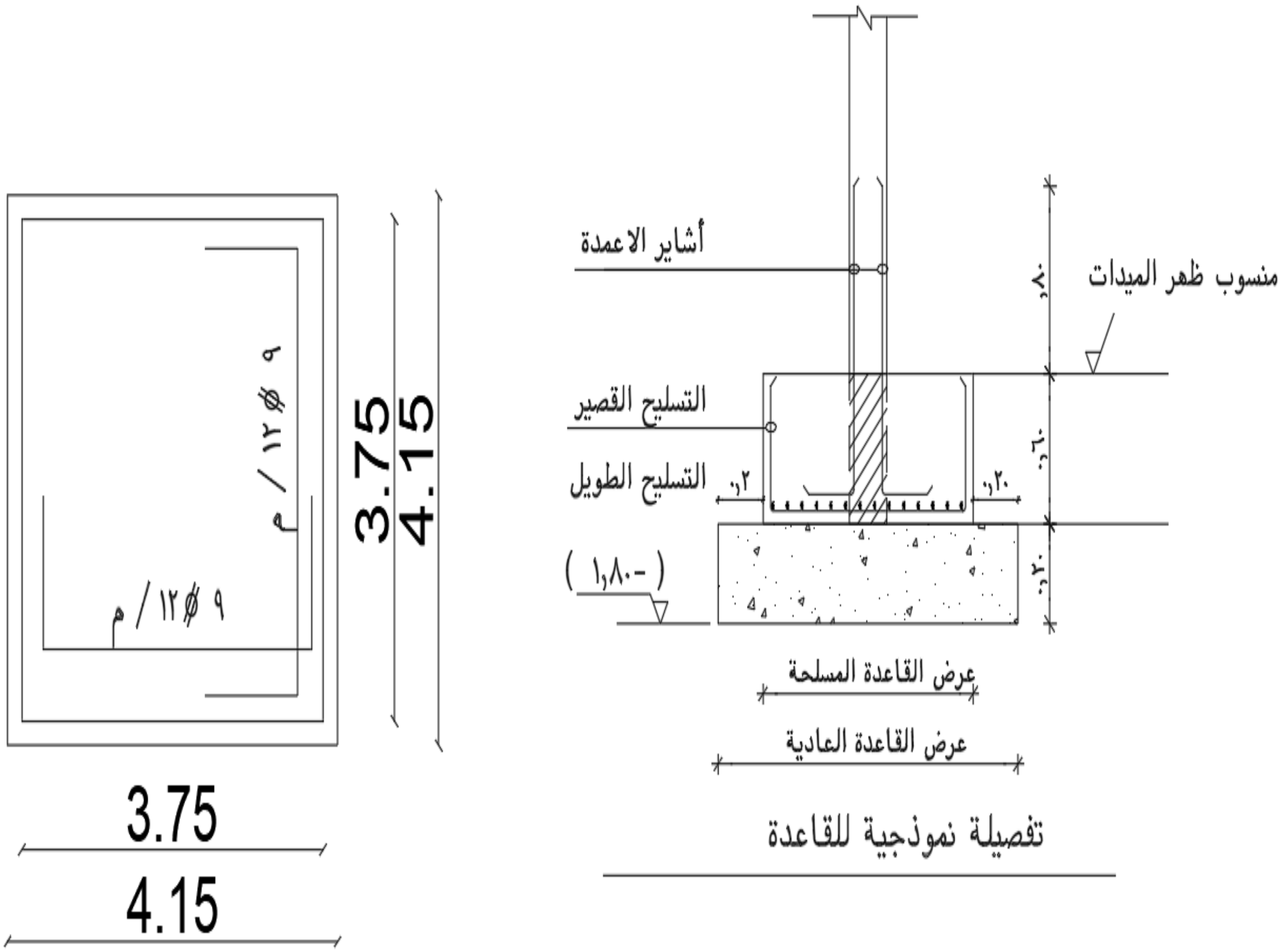
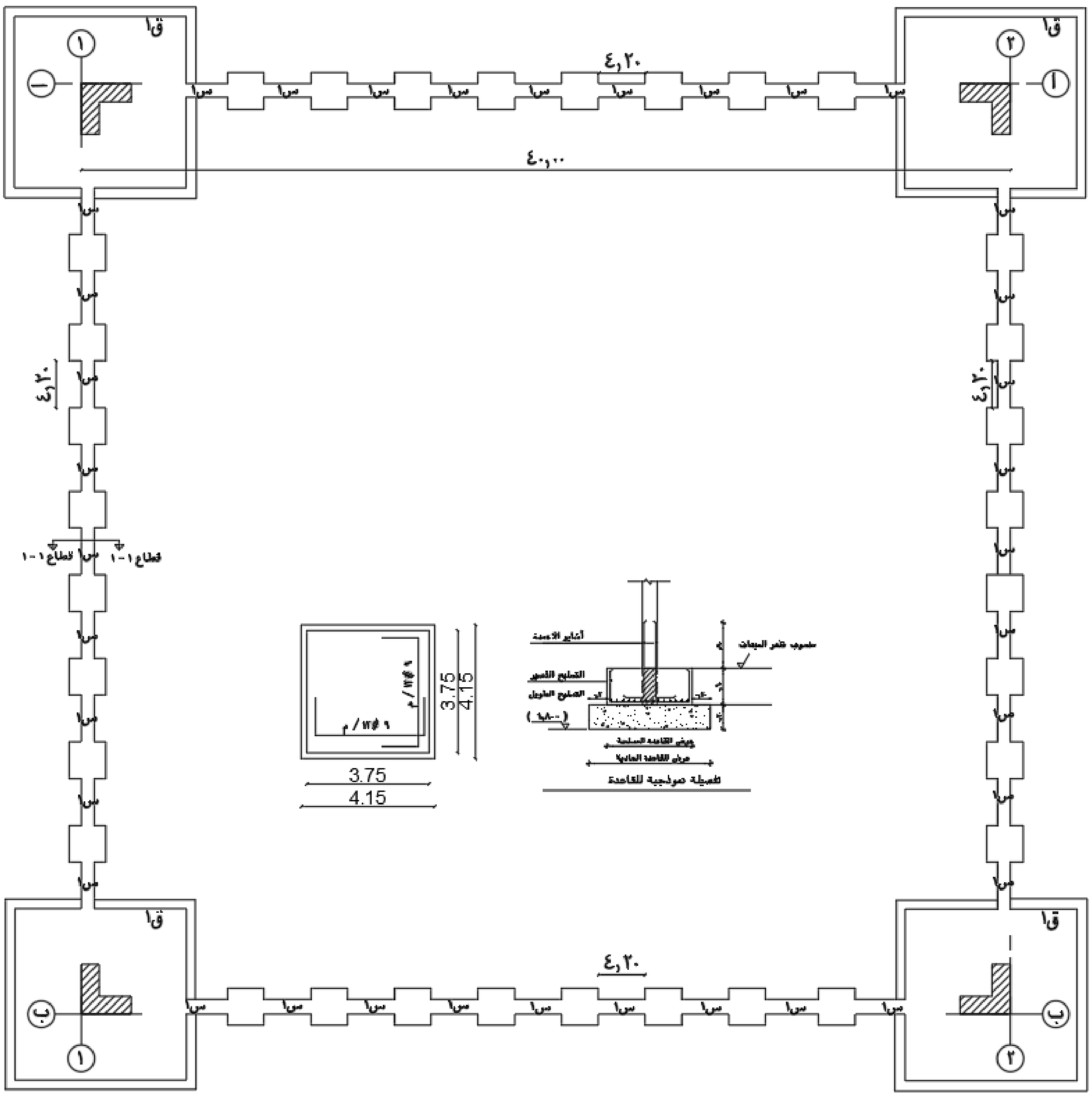
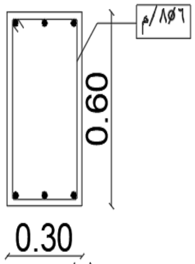


Figure 4.8 RFT details

Figure 4.9 RFT details foundation



جدول القواعد على اجهاد لايقل عن ١,٠٠ كجم/سم٢



قطاع ١ - ١

النموزج	الابعاد	الابعاد	التسليح السفلى		التسليح العلوى	
	عادية	مسلحة	قصير	طويل	قصير	طويل
ق١	٠,٢	٠,٦×٣,٧٥×٣,٧٥	٩ Ø ١٢ / م	٩ Ø ١٢ / م	--------	--------
ق٢	صفر	٠,٤ ×٠,٨×٠,٨	٥ Ø ١٢ / م	٥ Ø ١٢ / م	--------	--------

نموذج	ابعاد (م)		التسليح السفلى		التسليح العلوى		كانات
	عرض	ارتفاع	عدل	مكسح	عدل	مكسح	
س١	٠,٣٠	٠,٦٠	٣Ø١٦	–	٣Ø١٦	--	٦Ø٨/م

Figure 4.10 Table RFT

References:

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