الأكاديمية العربية الدولية
Arab International Academy

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Redesign of a Residential building

This Graduation Project is submitted in partial fulfillment of the requirements for the

Degree of Bachelor of Science

In Civil Engineering

Prepared by:

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Dedication:

After a complete success in the project I have conducted, I was asked to make a dedication for my graduation project. I dedicate all my work to two people that have affected my life the most, my mother and father, for making the man I ‘am today as I would have never done this without your support and encouragement. Both of you played an important role in my life, to my father who has taught me everything about life’s wonders and to the one that has taught me that the best kind of knowledge to have is that which is learned for its own sake and to the caring person I call my mother thank you for everything you do, you taught me that all of life’s accomplishments along the way could be achieved just one step at a time and that its okay to make mistakes. I would also like to dedicate this to my loving family who has supported me throughout this process. I will always appreciate what they all have done for me, especially Dr. Rawan Guni mat who have helped a lot with. connecting the dots when I faced any difficulties throughout the process of my project. Last but not least I dedicate this work and special thanks to my loyal and trustworthy friends for always being there for me when I need them the most. I’m really honored to have all of you by my side from family to friends.

Table of Contents

الأكاديمية العربية الدولية
Arab International Academy

List of Tables

List of figures

Chapter (1)

Introduction

Ch1: Introduction

1.1 Description:

- Site Information
  The structure is a residential building located **Abu Alanda – Amman**
  It consists of **five floors.**

- Our project consists of the analysis and design of residential structure Project.

- The project is consisting of:

1- Ground floor area = 407m^2
2- First floor area = 403m^2
3- Second floor area = 403m^2
4- Third floor area = 403m^2
5- Basement floor = 407m^2

Total area = 2023 m^2

1.2 Project Objectives:

The aims are:

Redesigning this project by applying theories and studies that we have been learnt in University "**Arab international academy**".

1- Achieve the acceptable probability to perform satisfactorily during their intended life, with a high degree of safety, they should sustain all the loads (Dead load, Live Load) and deformations of normal construction loads and have adequate durability and stiffness.

3- Every structure is designed to serve a particular function.

4- From the structure plans that we have in hands we must insure of the Structure safety for the users in all situations.
We need to make it serviceable, attractive and economically cost efficient.

1.3 Project Plans:

Ground Floor

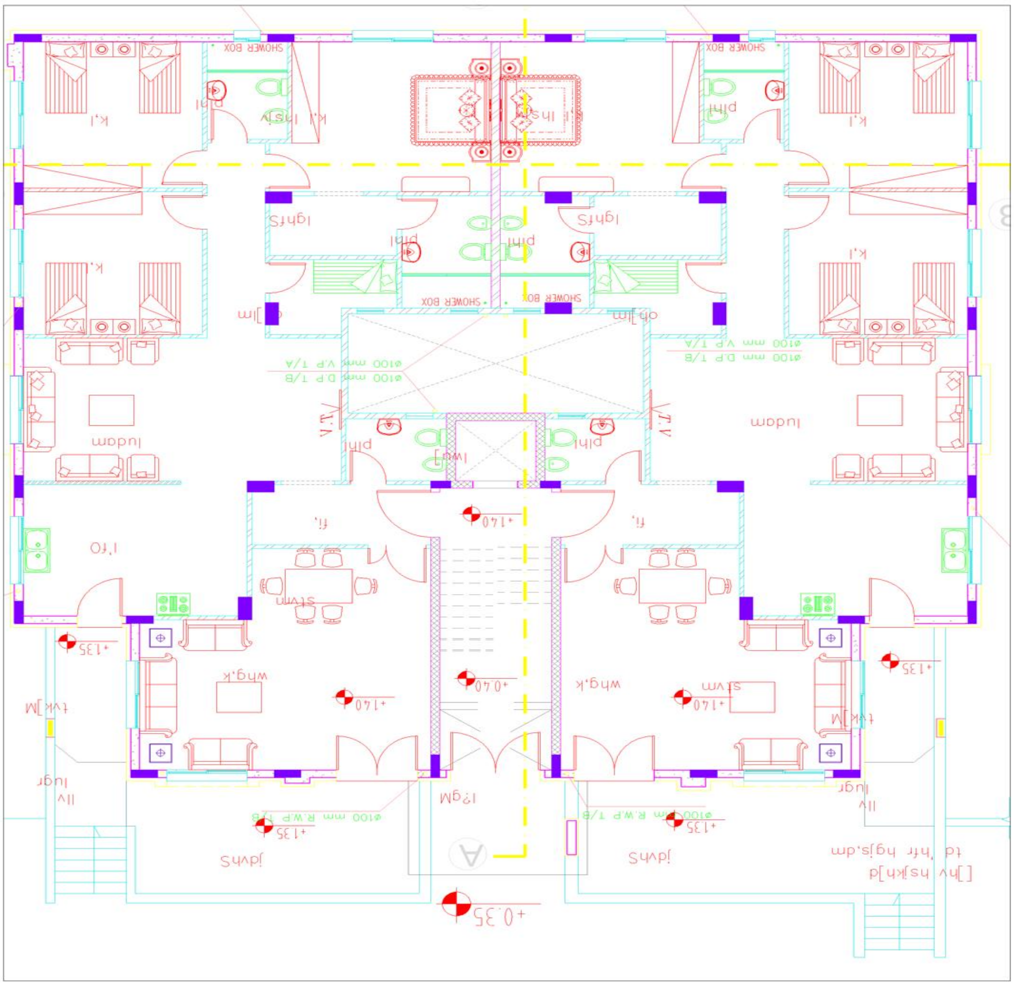


Figure 1.1: Ground Floor plan

Repeated Floor

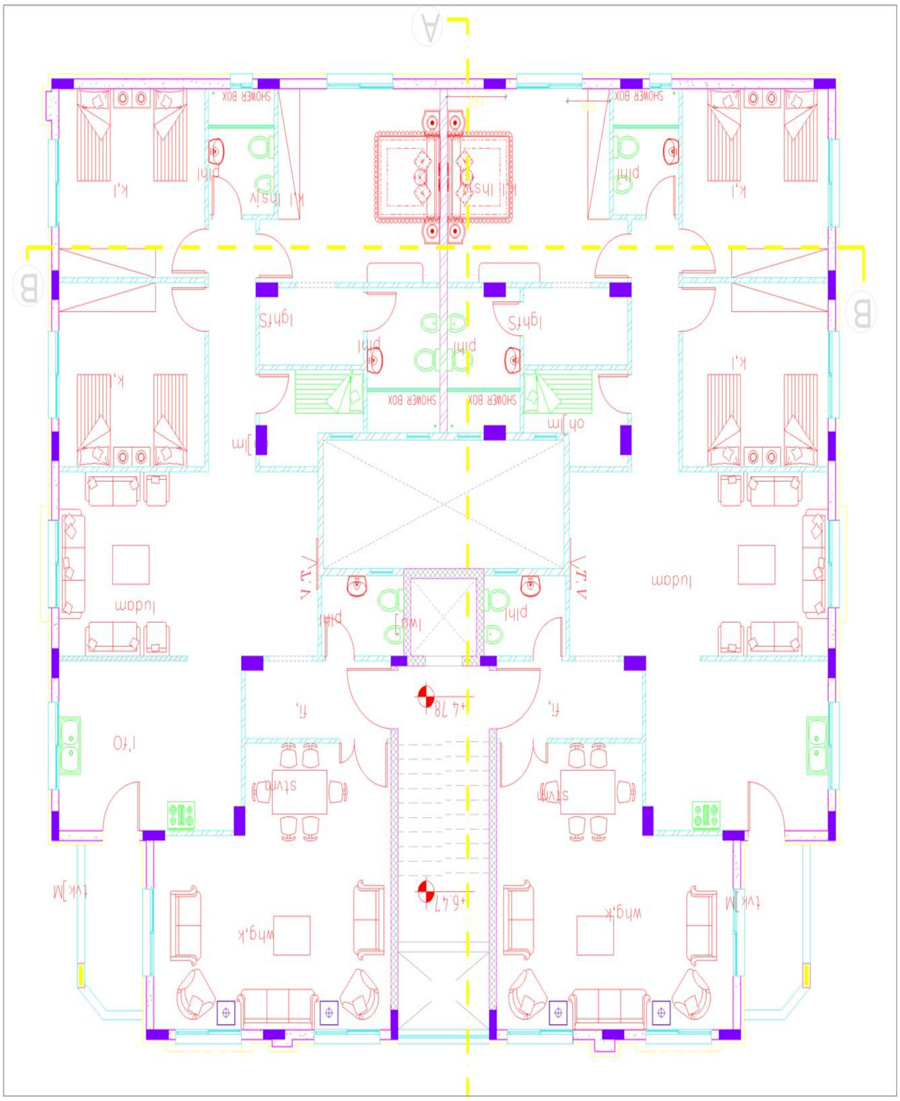


Figure 1.2 : Repeated Floor plan

Front View

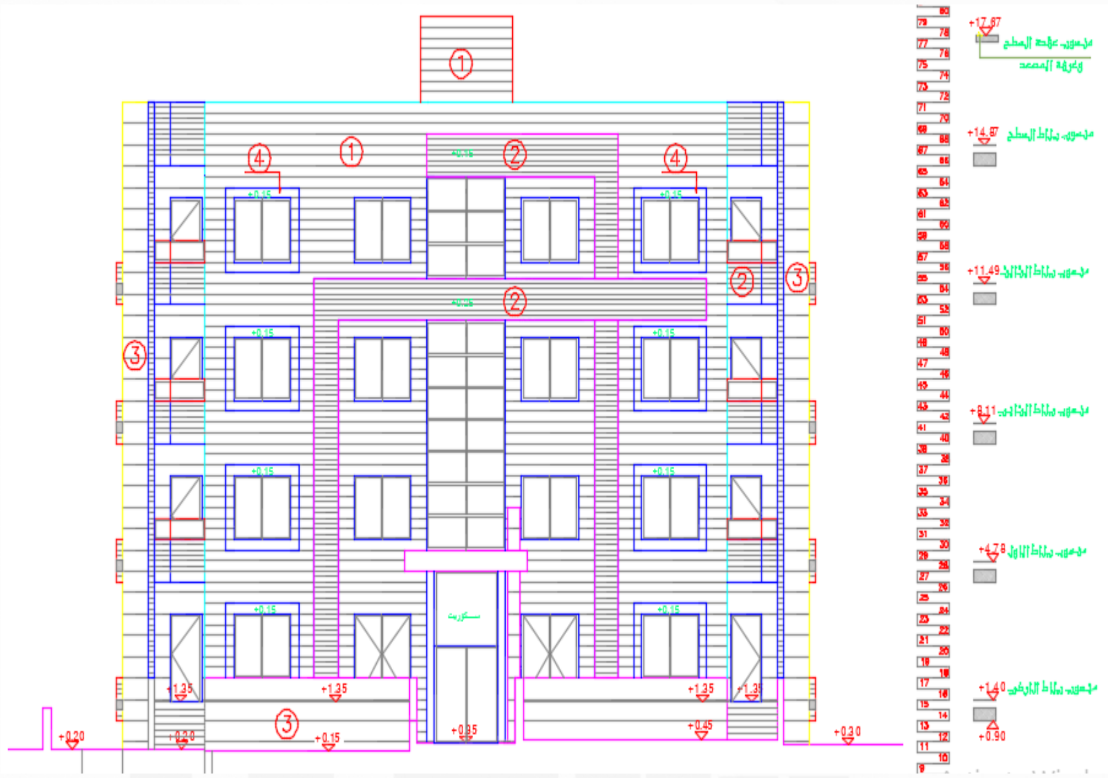


Figure 1.3 : Front View plan

Side View

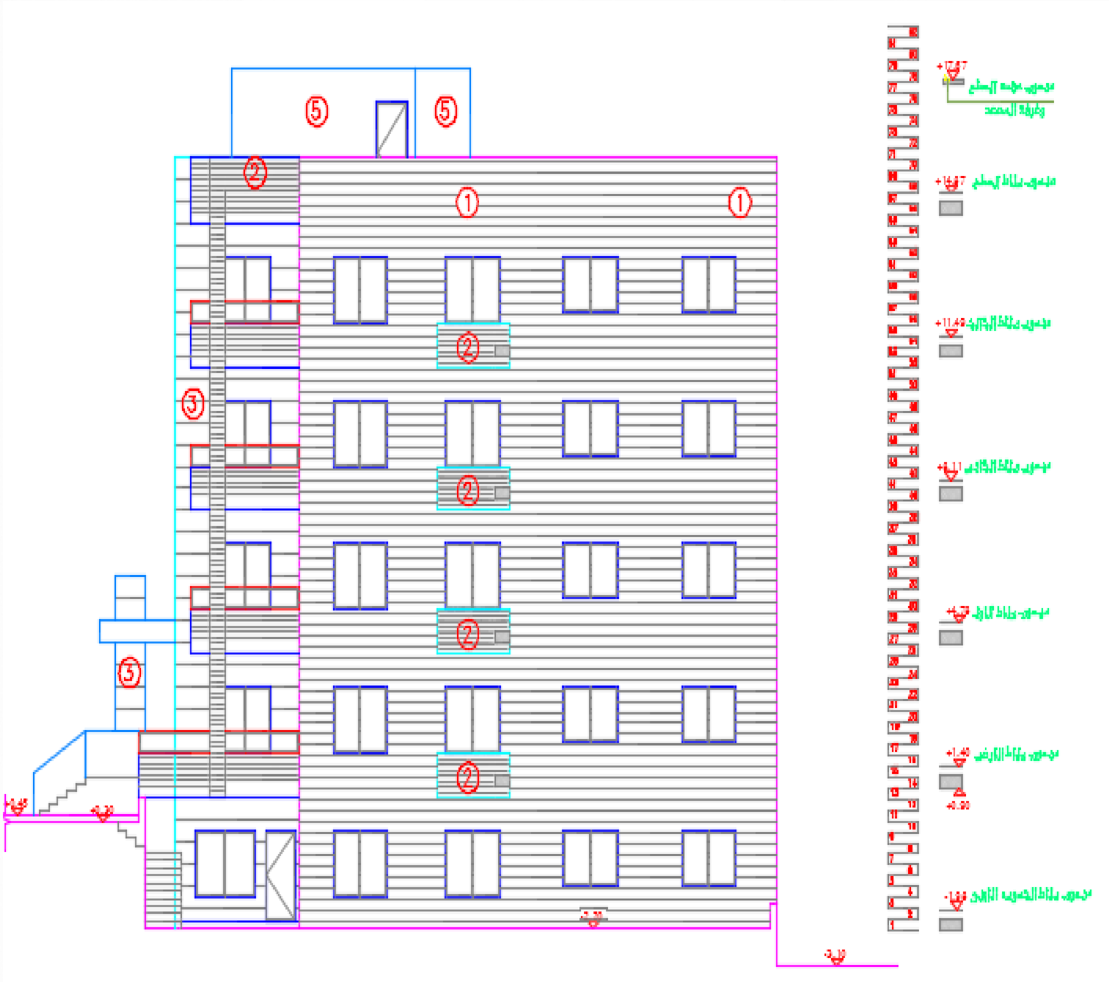


Figure 1.4 : Side View plan

1.4 Design Criteria:

There are four general design criteria for the safety of the project that must be satisfied:

1. Structure systems and member must be designed with sufficient margin of safety against failure.
2. Aesthetics: include such consideration as shape, geometrical proportions, symmetry, surface, texture, and articulation. These are especially important for structure of high visibility such signature building and bridges. The structure engineer must work with planners, architects, other design professional, and the affected community guiding them on the structure and considerations.
3. Functional requirements. A structure must always be designed to serve its intended function as specified by the project requirements.
4. Economy Structure must be designed and built within the target budget of the project.

1.5 Project details:

- Slabs:

One-way ribbed slab and one-way solid slab were used, this type was chosen due either dead and live loads magnitudes or requirements, length of spans and the most important factor is the cost.

- Beams:

Designed according to American Concrete Institute's Building Code (ACI), which is based on limit state of stress.

- Columns:
rectangular short columns were used.

- Foundations:

Single footings were designed.

1.6 Material:

1.6.1 Concrete:

Mixture of Sulfate resistance cement or any other hydraulic cement, fine aggregate, coarse aggregate, and water, with or without admixtures.

*The ultimate compressive strength of concrete, which is used, is **f'c = 28Mpa**

Figure 1.5: Concrete material

1.6.2 Steel Reinforcement:

- Grade **60** steel with a yield stress **420**Mpa was used for reinforcing **steel**.

- Grade **40** steel with a yield stress of **280**Mpa was used for reinforcing **stirrups**.

Figure 1.6: Steel material

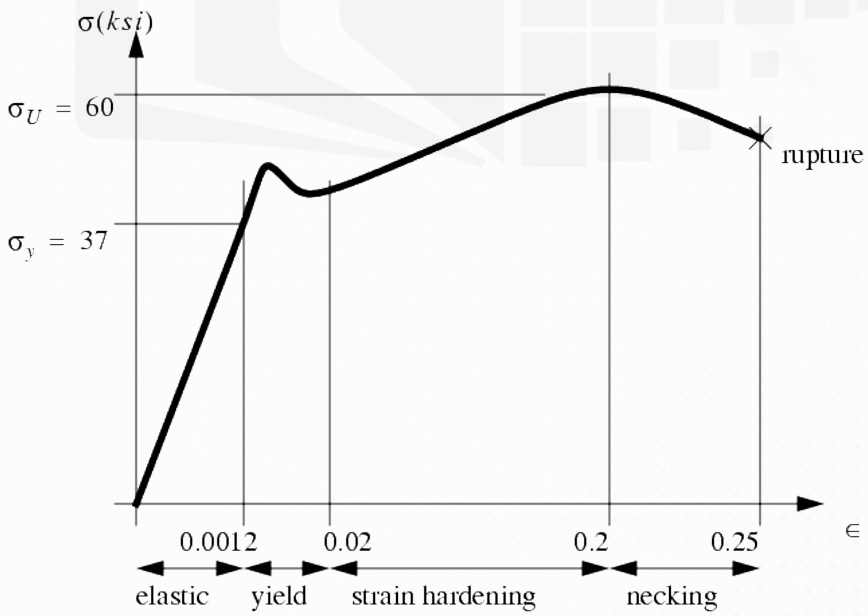


Figure 1.7: Stress strain diagram for steel

1.6.3 Density:

According to the Jordanian buildings code for Loads and Forces (JBC), the following densities were used:

Type	Density (ɣ)
Water	10 kN/m^3
Concrete	24 kN/m^3
Fill	18 kN/m^3
Plaster	22 kN/m^3
Hollow block	0.18 kN/block
Mortar	22 kN/m^3
Tiles	22 kN/m^3

Table 1.1: Density of materials from JBC

1.6.4 Loads:

- Dead Load

Own weight for each material was used to construct this building including tiles, filling, mortar, hollow block, concrete and plaster. There densities were exported from JBC to the Table:

Tiles	**0.025*1*1*22 = 0.55 kN/m^2**
Mortar	**0.025*1*1*22 = 0.55 kN/m^2**
Fill	**0.1*1*1*18 = 1.8 kN/m^2**
Concrete hollow block	**(0.18*5)/0.52 = 1.73kN/m^2**
Partitions	**0.33*3*2.38 = 2.35kN/m^2**
Topping slab	**0.07*1*1*24 = 1.68 kN/m^2**
R.c Rib	**(0.24*24*(0.12+0.16))/(2*0.52) = 1.55 kN/m^2**
Plaster	**0.025*1*1*22 = 0.5 kN/m^2**
Total Dead load	**10.71 kN/m^2**
Dead load for Rib	**10.71 * 0.52 = 5.57 kN/m^2**

Table 1.2: Dead Load calculations

- live load

According to the JBC the Live Load for residential buildings is **$2kN/m^2$**

Occupancy classification	Uniformly distributed load (kN/m2)	Concentrated load (kN)
Office buildings		
Offices and staff rooms	2.5	2.7
Class rooms	3.0	2.7
Corridors, store rooms and Reading rooms	4.0	4.7
Residential buildings: Apartments	2.0	1.8
Public places such as Restaurants	4.0	2.7
Corridors	3.0	4.5

Table 1.3: Live Load from (JBC) Code

Live Load for Rib = 2*0.52 = 1.04 KN/m^2

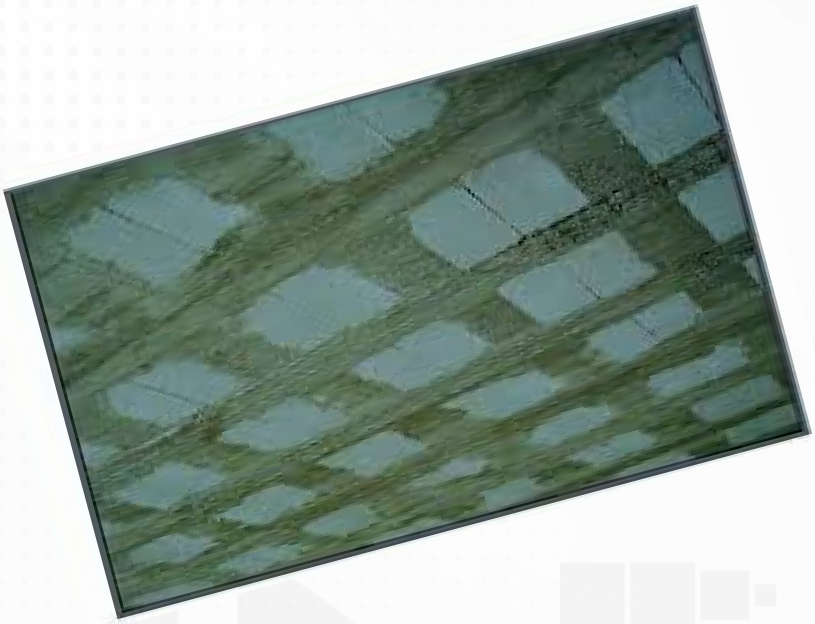
1.7 Codes:

- Design Standard and Codes:

- American Concrete Institute (ACI-318), For the design of structural elements.
- The Jordanian buildings Code (JBC), For live and Dead loads.

Chapter (2)

Design of Slabs



Ch2: Slabs

2.1 Introduction:

A slab is structural element whose thickness is small compared to its own length and width.
Slabs are usually used in floor and roof construction. According to the use of the building, slabs are classified into two types:
1-One-way

2- Two-way.

Figure 2.1: One way ribbed slab

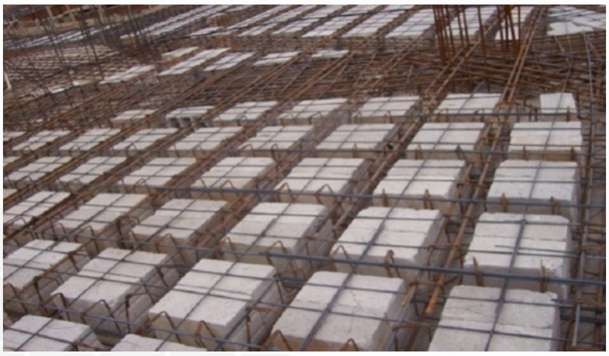


Figure 2.2 : Two way ribbed slab]

2.2 Comparison between One-way and Two-way slabs:

Two-way slabs are surface members acting in a both directions reverse One-way slabs that act in one direction. Two-way slabs are supported on all four sides and one-way slabs are supported in just two opposite directions.

The ratio of length to width of two-way slab should be less than 2 otherwise One-way action is obtained, even though supports are provided on all sides. In many cases the slabs are of such proportions and are supported in such way that Two-way action results. An optimal plan geometry with all same supports is a square, based on load distribution into both directions.

2.3 Types of slabs :

1-Ribbed slab :

Ribbed slabs are widely used in many countries such as Jordan. This type of slabs or flooring system consists of series of small closed spaced reinforced concrete T-beams. These floors are suitable for building with light live loads. attributed to the rapid shattering, high fire resistance , ease and fast construction, good sound and thermal insulation and it's ability to hide beams and service pipes.

Figure 2.3 : Section of Ribbed Slab

2-Waffle slab :

This type of slab, also called two way ribbed slab,but without hollow block. It's supported by reinforced concrete ribs or joists .The ribs are usually tapered and uniformly spaced and supported on girders that rest on columns.

Figure 2.4 :Waffle Slab

3- Solid Slab :

These slabs are supported on one or two opposite sides, on one opposite side its called one way slab and on two opposit sides ; its called Two-way slab. A slab supported on four sides with length to width ratio greater than two, should be designed as Two-way slab.

Figure 2.5 :Solid Slab

2.4 Design procedure for one way ribbed slab :

1. Determine the **direction** of ribs which was chosen.

2. The overall slab thickness **h** is determined based on deflection control requirement. Also, thickness of topping slab t, rib width b, and hollow block size, if any, are to be determined based on code requirements.

3. The **factored load** on each of the ribs is computed.

4. The **Shear force** and **bending moment diagrams** using the load evaluated in step 3 (From Prokon software).

5.Check if the section behaves as **T-section** or **rectangular section**.

6. Design **positive** and **negative moment** reinforcement.
Clear distance between bars is to be checked to guarantee a free flow of concrete.

7. The strength of web in **shear** is checked

Layout of slabs

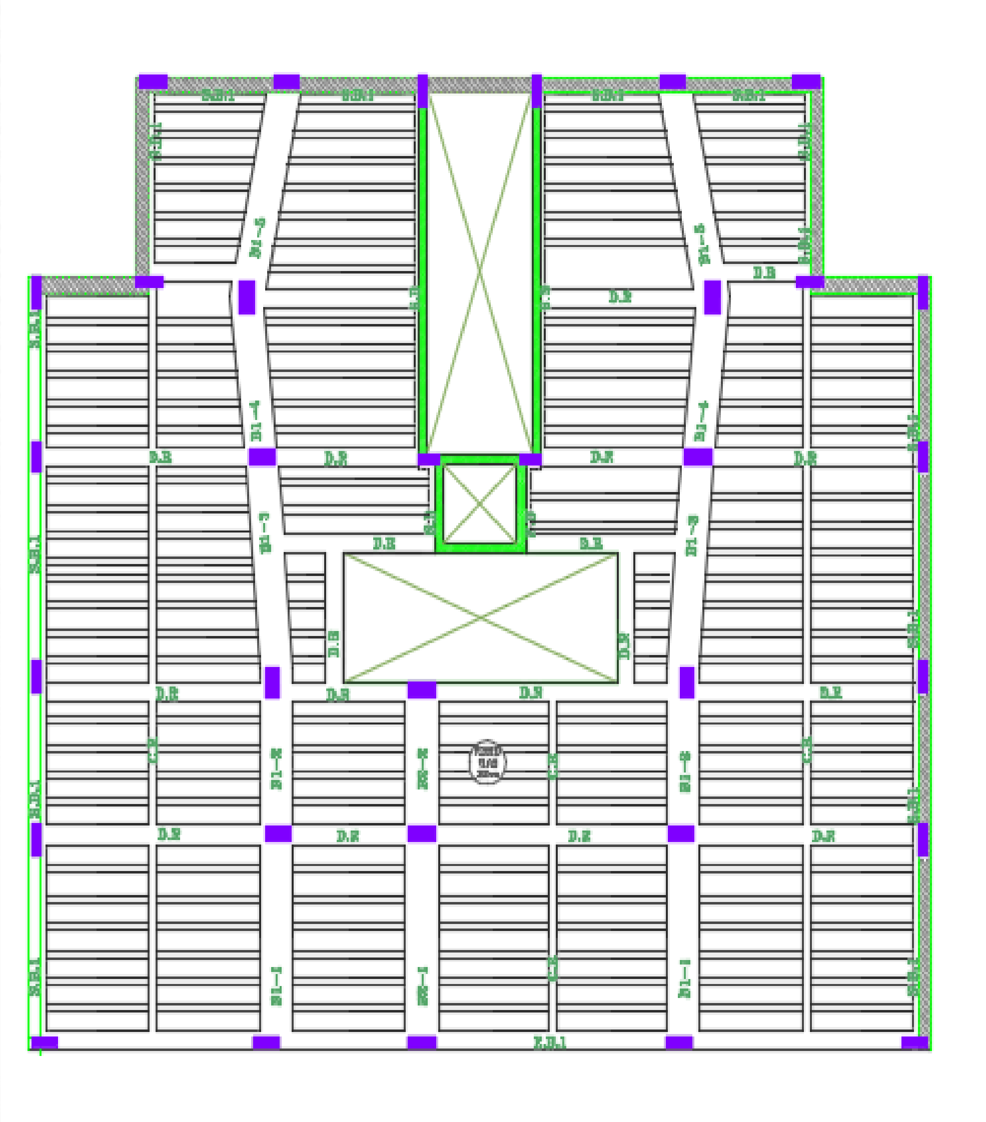


Figure 2.6: Layout of Slabs.

2.5 Design of Ribbed Slab:

- **Check the use one way or two-way slab:**

$$\frac{\text{Long direction}}{\text{Short direction}} > \mathbf{2}\ \boldsymbol{or} < \mathbf{2} \quad = \quad \frac{6.2}{4.25} < \mathbf{2} \quad = \quad 1.46 < \mathbf{2}$$

Use one way reinforcement because the maximum length between 7 – 5 m.

Use One Way Ribbed Slab

- **Thickness of slab:**

- Minimum thickness of slab from (ACI-318) code:

Both ends for rib	= L/21	6.25/21 = 0.30 m
One end for rib	= L/18.5	5.8/18.5 = 0.31 m
Contilever	= L/8	2.1/8 = 0.26 m

Slab thickness = 310 mm

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

Table 2.1: Minimum thickness from (ACI-318) code

1) <u>Design of section (Rib.1)</u>:

D.L =10.71kN/m2
L.L = 2 KN/m2
h = 310 mm
Cover = 35 mm
d = h−cover= 275 mm
bf = 520 mm
bw = 120 mm
F'c = 28 Mpa
Fy = 420 Mpa

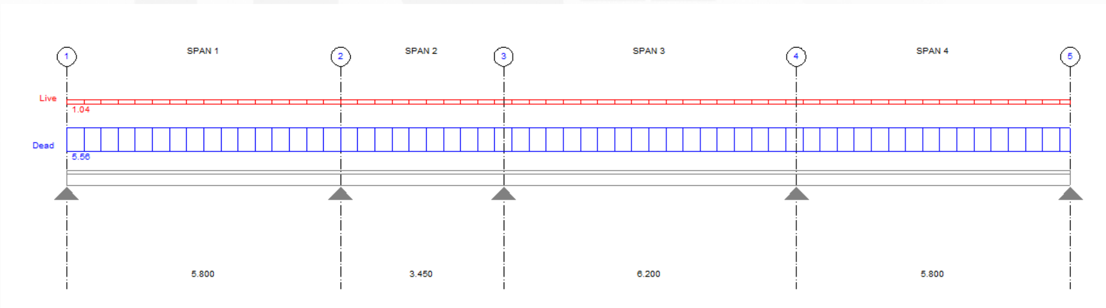


Figure 2.7: Load distribution for Rib (1)

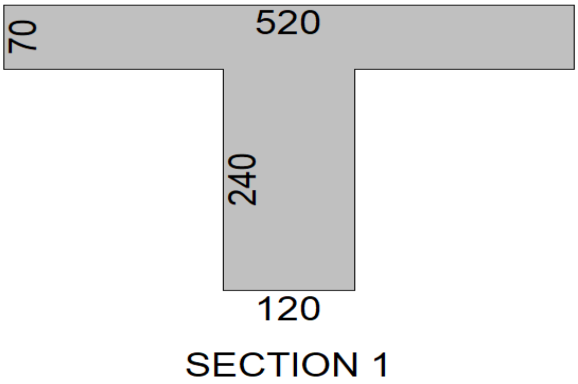


Figure 2.8: Section for Rib (1)

Shear Force

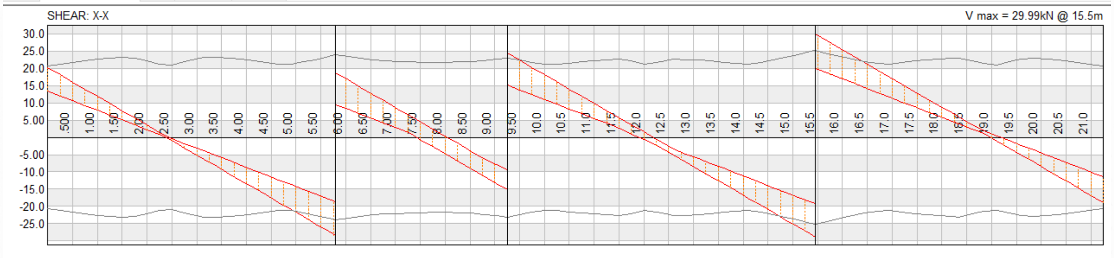


Figure 2.9: Shear force diagram for Rib (1)

Bending Moment

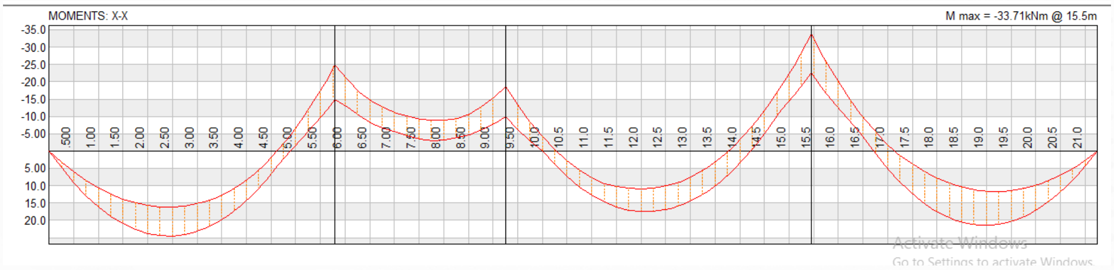


Figure 2.10: Bending moment diagram for Rib (1)

- Equations used for design:

. $\rho = \frac{0.85Fc'}{Fy}\left(1 - \sqrt{1 - \frac{2.354Mu}{0.9Fc'bd^2}}\right)$ **. As-ve = ρ*bw*d**

. Asmin = 2 ϕ 12 = 226 mm^2 **. As+ve = ρ*bf*d**

1) Moment for 0.25 m = **-** 4.8 kN.m

- $\rho = \frac{0.85*28}{420}\left(1 - \sqrt{1 - \frac{2.354*4.8*10^6}{0.9*28*120*275^2}}\right) = 0.0014$
- As-ve = 0.0014*120*275 = 46.2 mm^2
- $\rho_{max} = \frac{0.85*0.85*28}{420}\left(\frac{0.003}{0.003+0.004}\right) = 0.0206$
- use Asmin = 226 mm^2

2) Moment for 2.50 m = 24.56 kN.m

ρ = 0.0016 **As =** 228.8 mm^2 **use As =** 228.8 mm^2

3) Moment for 5.8 m = **-** 24.75 KN.m

ρ = 0.0077 **As =** 254.1 mm^2 **use As =** 254.1 mm^2

4) Moment for 9.25 m = **-** 18.61 KN.m

ρ = 0.0057 **As =** 188.1 mm^2 **use Asmin =** 226 mm^2

5) Moment for 12 m = 17.51 KN.m

ρ = 0.0019 **As =** 271.7 mm^2 **use As =** 271.7 mm^2

6) Moment for 15.45 m = **-** 33.71 KN.m

ρ = 0.01 **As =** 330 mm^2 **use As =** 330 mm^2

7) Moment for 19.1 m = 21.4 KN.m

ρ = 0.0014 **As =** 200.2 mm^2 **use Asmin =** 226 mm^2

8) Moment for 21.25 m = **-** 2.74 kN.m

ρ = 0.0008 **As =** 26.4 mm^2 **use Asmin =** 226 mm^2

2) <u>Design of section (Rib.2)</u>:

D.L =10.71kN/m2
L.L = 2 kN/m2
h = 310 mm
Cover = 35 mm
d = h−cover= 275 mm
bf = 520 mm
bw = 120 mm
F'c = 28 Mpa
Fy = 420 Mpa

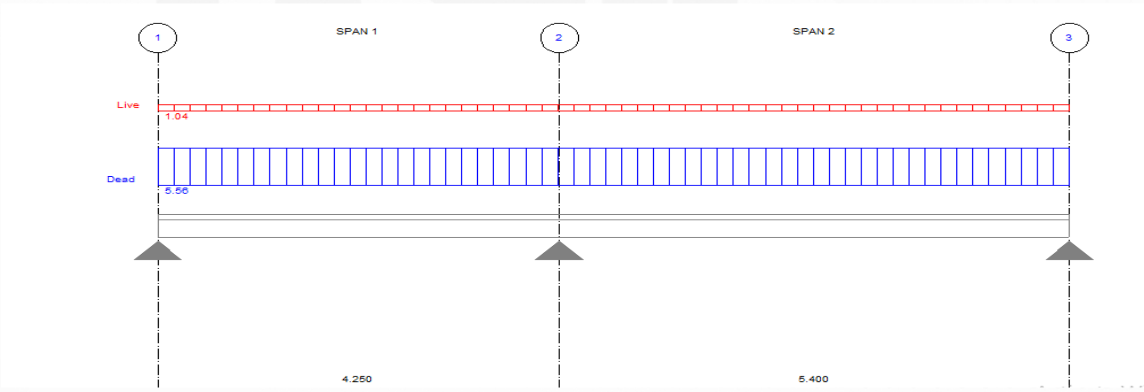


Figure 2.11 : Load distribution for Rib (2)

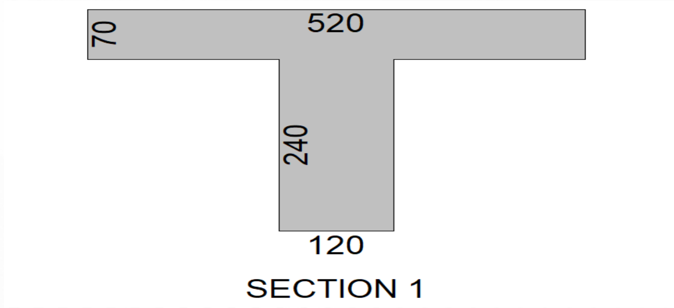


Figure 2.12 : section for rib (2)

Shear Force

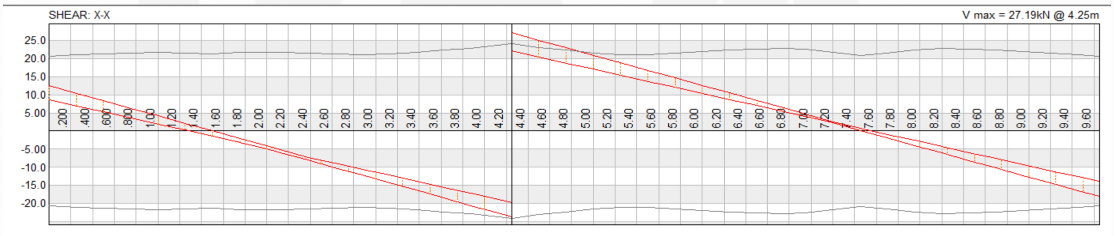


Figure 2.13 : Shear force diagram for Rib (2)

Bending Moment

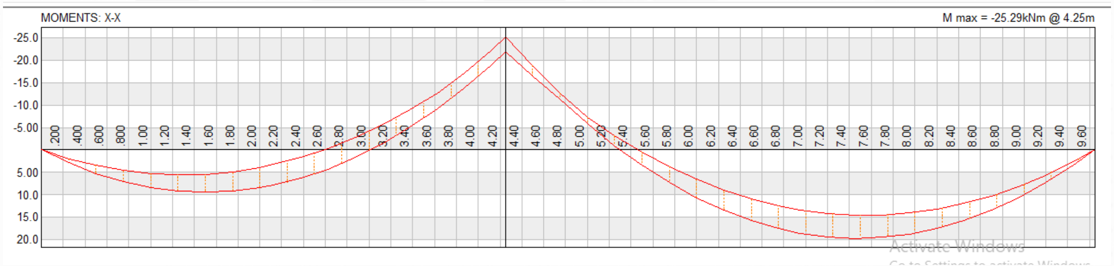


Figure 2.14 : Bending moment diagram for Rib (2)

1) Moment for 0.25 m = **-** 2.88 kN.m

ρ = 0.00084 **As =** 27.72 mm^2 **use As**$_{\mathbf{min}}$ **=** 226 mm^2

2) Moment for 1.50 m = 9.46 kN.m

ρ = 0.0006 **As =** 85.8 mm^2 **use As**$_{min}$ **=** 226 mm^2

3) Moment for 4.25 m = **-** 25.29 kN.m

ρ = 0.0079 **As =** 260.7 mm^2 **use As =** 260.7 mm^2

4) Moment for 7.45 m = 19.71 kN.m

ρ = 0.0013 **As =** 185.9 mm^2 **use As**$_{min}$ **=** 226 mm^2

5) Moment for 9.65 m = - 3.46 kN.m

ρ = 0.001 **As =** 33 mm^2 **use As**$_{min}$ **=** 226 mm^2

3) <u>Design of section (Rib.3)</u>:

D.L =10.71kN/m2
L.L = 2 kN/m2
h = 310 mm
Cover = 35 mm
d = h—cover= 275 mm
bf = 520 mm
bw = 120 mm
F'c = 28 Mpa
Fy = 420 Mpa

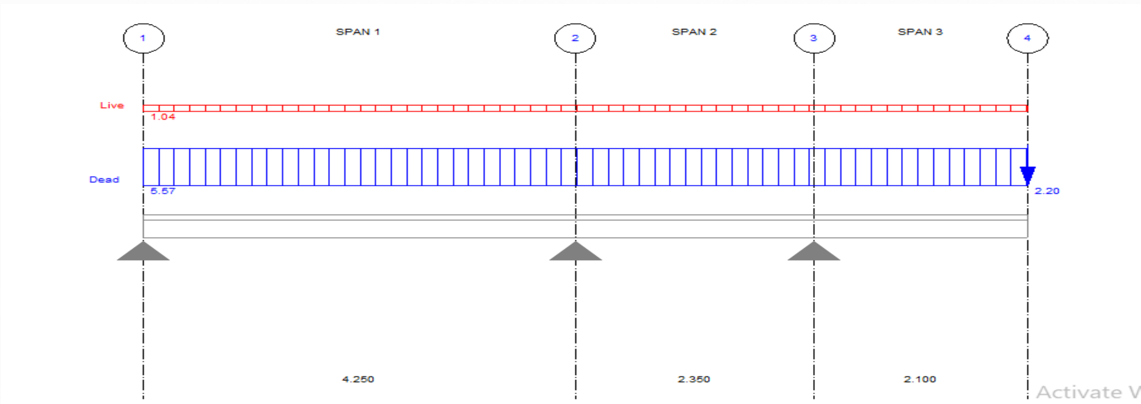


Figure 2.15 : Load distribution for Rib (3)

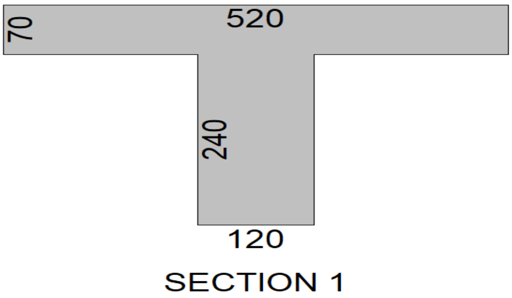


Figure 2.16 : section for rib (3)

Shear Force

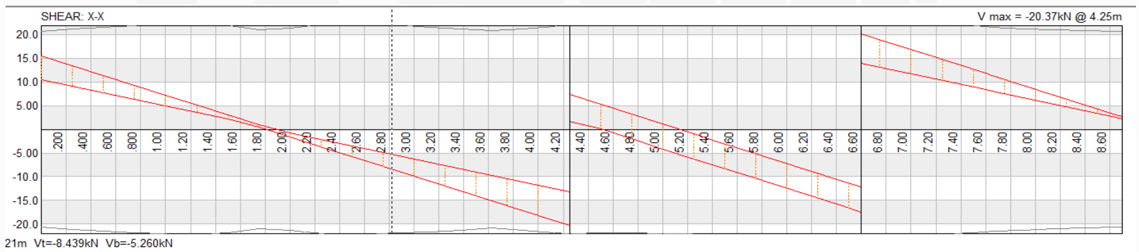


Figure 2.17 : Shear force diagram for Rib (3)

Bending Moment

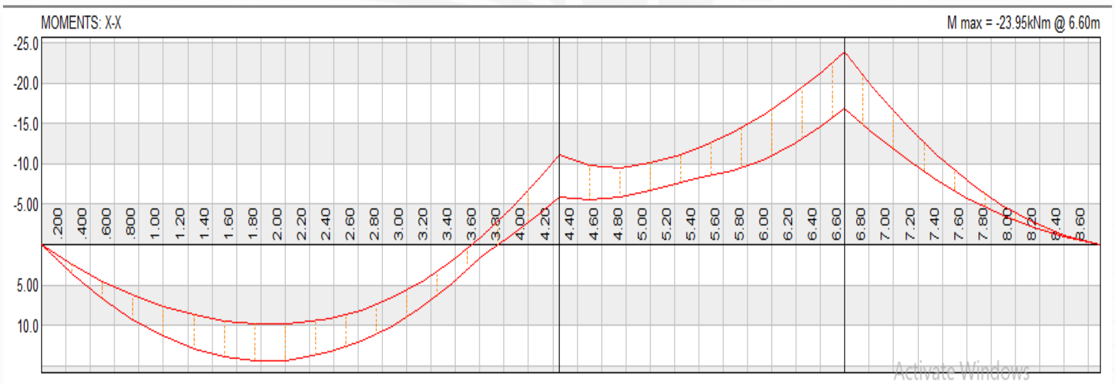


Figure 2.18 : Bending moment diagram for Rib (3)

1) Moment for 0.25 m = 4.25 kN.m

ρ = 0.001 As = 33 mm^2 use As_{min} = 226 mm^2

2) Moment for 1.75 m = 19.34 kN.m

ρ = 0.0009 As = 128.7 mm^2 use As_{min} = 226 mm^2

3) Moment for 4.25 m = - 11.18 kN.m

ρ = 0.0033 As = 108.9 mm^2 use As = 260.7 mm^2

4) Moment for 6.6 m = - 23.95 kN.m

ρ = 0.0074 As = 244.2 mm^2 use As = 244.2 mm^2

4) <u>Design of solid slab (S)</u>:

D.L = 6kN/m2
L.L = 1 kN/m2
h = 200 mm
Cover = 30 mm
d = h—cover= 170 mm
b = 1000 mm
F'c = 28 Mpa
Fy = 420 Mpa

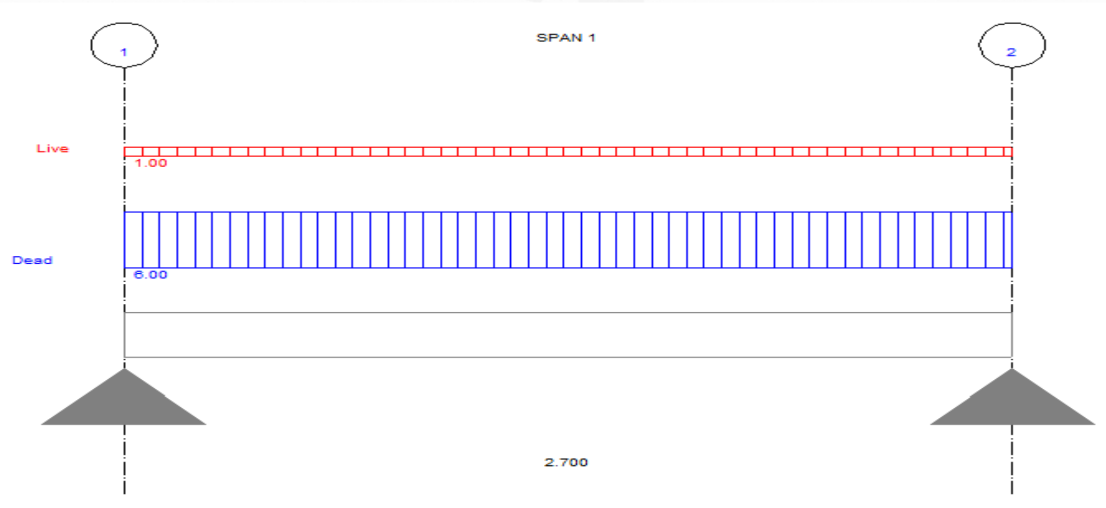


Figure 2.19 : Load distribution for Solid slab

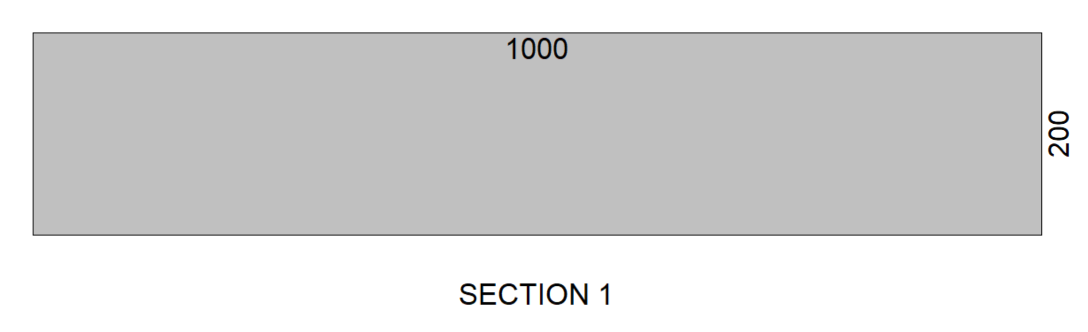


Figure 2.20 : section for solid slab

Shear Force

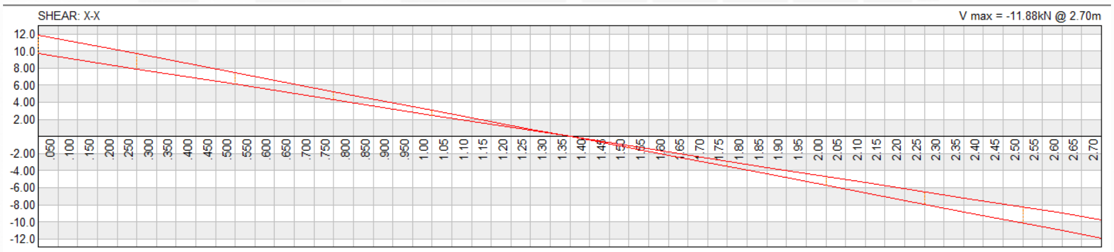


Figure 2.21 : Shear force diagram for Solid slab

Bending Moment

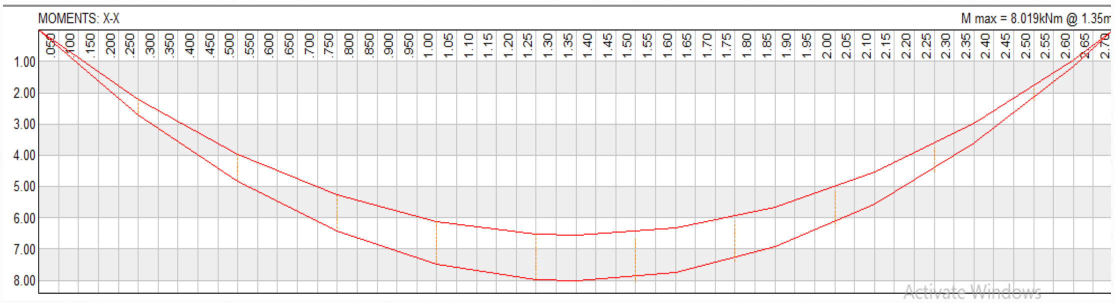


Figure 2.22 : Bending moment diagram for Solid slab

Moment for 1.35 m = 8.02 kN.m

ρ = 0.0007 **As =** 119 mm^2 **use As$_{min}$ =** 561 mm^2

<u>Use: 1ϕ12/200mm</u>

As 5 ϕ12 = 565.5 mm^2

- <u>Shrinkage</u>

As = ρbd

= (0.0018 * 1000 * 170) = 306 mm^2

<u>Use: 1ϕ10/200 mm</u>

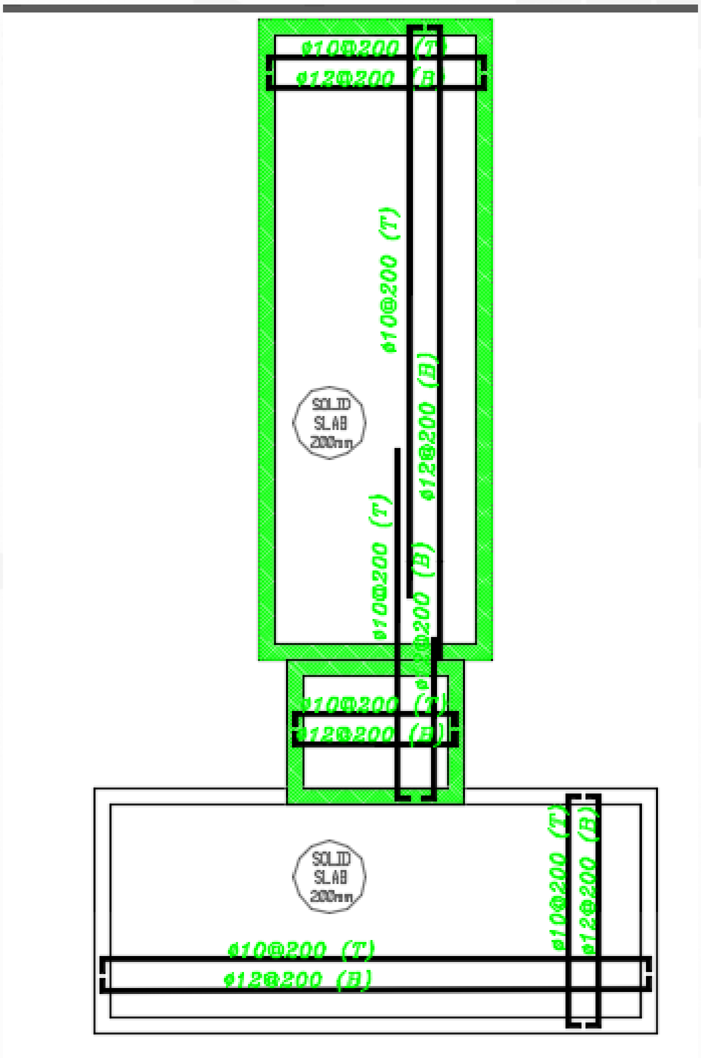


Figure 2.23 : Reinforcement detailing for solid slab

2.6 Shear Design For Rib :

support : (1000/800/350/200)

- **Wu = 1.2*DL + 1.6*LL**
  Wu = 1.2*5.56 + 1.6*1.04 = 8.336 kN/m
- $\boldsymbol{\varphi Vc = 0.75 \times 0.17 \times \sqrt{fc'} \times b \times d \times 10^{-3}}$
  $\boldsymbol{\varphi Vc = 0.75 \times 0.17 \times \sqrt{28} \times 120 \times 275 \times 10^{-3}}$ **= 22.26 kN**
- $\boldsymbol{½\ \varphi Vc = 11.13\ kN}$

1) Shear Force (RiB.1)

Span AB (1)

- X=0.175+0.275 = 0.45 m
- Vus=20.26 kN
- Vud = Vus – (W*X) = 20.26 – (8.336 * 0.45) = 16.5 kN
- $½\varphi Vc < Vud < \varphi Vc$ 11.13 < 16.5 < 22.26

S_1 = d/2 = 0.275/2 = 0.1375 m
S_2 = Av*Fy / 0.35*bw = 100.5*280 / 0.35*120 = 0.67 m
S_3 = 0.600 m

Use: 1ϕ8/140mm

Span AB (2)

- X=0.5+0.275 = 0.775 m
- Vus=28.43 kN
- Vud = Vus – (W*X) = 28.43 – (8.336 * 0.775) = 21.97 kN
- ½φVc < Vud < φVc 11.13 < 21.97 < 22.26

S1 = d/2 = 0.275/2 = 0.1375 m
S2 = Av*Fy / 0.35*bw = 100.5*280 / 0.35*120 = 0.67 m
S3 = 0.600 m

Use: 1ϕ8/140mm

Span BC (3)

- X=0.5+0.275 = 0.775 m
- Vus=18.58 kN
- Vud = Vus – (W*X) = 18.58 – (8.336 * 0.775) = 12.12 kN
- ½φVc < Vud < φVc 11.13 < 12.12 < 22.26

S1 = d/2 = 0.275/2 = 0.1375 m
S2 = Av*Fy / 0.35*bw = 100.5*280 / 0.35*120 = 0.67 m
S3 = 0.600 m

Use: 1ϕ8/140mm

Span BC (4)

- X=0.4+0.275 = 0.675 m
- Vus=15.32 kN
- Vud = Vus – (W*X) = 15.32 – (8.336 * 0.675) = 9.69 kN
- Vud < ½φVc < φVc 9.69 < 11.13 < 22.26

Use the minimum reinforcement : 1ϕ8/200mm

Span CD (5)

- $X = 0.4 + 0.275 = 0.675$ m
- $Vus = 24.70$ kN
- $Vud = Vus - (W*X) = 24.70 - (8.336 * 0.675) = 19.07$ kN
- $\frac{1}{2}\varphi Vc < Vud < \varphi Vc$ $\quad 11.13 < 19.07 < 22.26$

$S_1 = d/2 = 0.275/2 = 0.1375$ m
$S_2 = Av*Fy / 0.35*bw = 100.5*280 / 0.35*120 = 0.67$ m
$S_3 = 0.600$ m

Use: 1ϕ8/140mm

Span CD (6)

- $X = 0.5 + 0.275 = 0.775$ m
- $Vus = 29.01$ kN
- $Vud = Vus - (W*X) = 29.01 - (8.336 * 0.775) = 22.55$ kN
- $\varphi Vc < Vud$ $\quad 22.26 < 22.55$
- $Vs = (Vud - \varphi Vc) / \varphi = (22.55 - 22.26) / 0.75 = 0.386$ kN
- $Vc1 = 2\,\varphi Vc / \varphi = 2*22.26 / 0.75 = 59.36$ KN
- $Vs < Vc1$ $\quad 0.386 < 59.36$

$S1 = Av*Fy*d / Vs = 100.5*280*275 / 0.386*10^6 = 0.2036$ m
$S_2 = d/2 = 0.275/2 = 0.1375$ m
$S_3 = Av*Fy / 0.35*bw = 100.5*280 / 0.35*120 = 0.67$ m
$S_4 = 0.600$ m

Use: 1ϕ8/140mm

Span DE (7)

- $X = 0.5 + 0.275 = 0.775$ m
- $Vus = 30.03$ kN
- $Vud = Vus - (W*X) = 30.03 - (8.336 * 0.775) = 23.569$ kN

- $\varphi Vc < Vud$ 22.26 < 23.569

- $Vs = (Vud - \varphi Vc) / \varphi = (23.569 – 22.26) / 0.75 = 1.74$ kN
- $Vc1 = 2\ \varphi Vc / \varphi = 2*22.26 / 0.75 = 59.36$ KN

- $Vs < Vc1$ 1.74 < 59.36

S1 = Av*Fy*d / Vs = 100.5*280*275 / 1.74*10^6= 0.44 m
S2 = d/2 = 0.275/2 = 0.1375 m
S3 = Av*Fy / 0.35*bw = 100.5*280 / 0.35*120 = 0.67 m
S4 = 0.600 m

Use: 1ϕ8/140mm

Span DE (8)

- X=0.175+0.275 = 0.45 m
- Vus=18.89 kN

- Vud = Vus – (W*X) = 18.89 – (8.336 * 0.45) = 15.13 kN

- $\frac{1}{2}\varphi Vc < Vud < \varphi Vc$ 11.13 < 15.13 < 22.26

S1 = d/2 = 0.275/2 = 0.1375 m
S2 = Av*Fy / 0.35*bw = 100.5*280 / 0.35*120 = 0.67 m
S3 = 0.600 m

Use: 1ϕ8/140mm

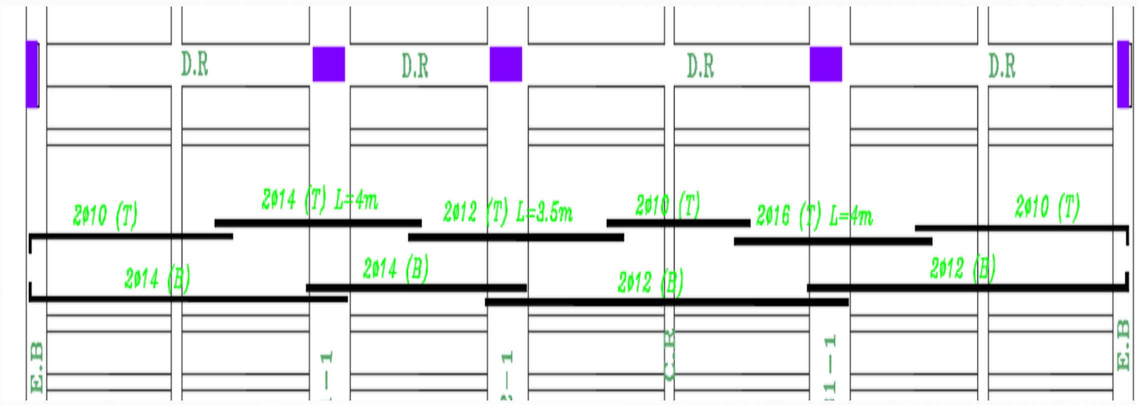


Figure 2.24 : Reinforcement detailing for Rib (1)

2) Shear Force (R.B.2)

Span AB (1)

- X=0.1+0.275 = 0.375 m
- Vus=11.55 kN
- Vud = Vus – (W*X) = 11.55 – (8.336 * 0.375) = 8.42 kN
- Vud < ½φVc < φVc   8.42 < 11.13 < 22.26

Use the minimum reinforcement : 1ϕ8/200mm

Span AB (2)

- X=0.5+0.275 = 0.775 m
- Vus=22.91 kN
- Vud = Vus – (W*X) = 22.91 – (8.336 * 0.775) = 16.44 kN
- ½φVc < Vud < φVc   11.13 < 16.44 < 22.26

S_1 = d/2 = 0.275/2 = 0.1375 m
S_2 = Av*Fy / 0.35*bw = 100.5*280 / 0.35*120 = 0.67 m
S_3 = 0.600 m

Use: 1ϕ8/140mm

Span BC (3)

- X=0.5+0.275 = 0.775 m
- Vus=27.07 kN
- Vud = Vus – (W*X) = 27.07 – (8.336 * 0.775) = 20.61 kN
- ½φVc < Vud < φVc 11.13 < 20.61 < 22.26

S_1 = d/2 = 0.275/2 = 0.1375 m
S_2 = Av*Fy / 0.35*bw = 100.5*280 / 0.35*120 = 0.67 m
S_3 = 0.600 m

Use: 1ϕ8/140mm

Span BC (4)

- X=0.175+0.275 = 0.45 m
- Vus=18.21 kN
- Vud = Vus – (W*X) = 18.21 – (8.336 * 0.45) = 14.45 kN
- ½φVc < Vud < φVc 11.13 < 14.45 < 22.26

S_1 = d/2 = 0.275/2 = 0.1375 m
S_2 = Av*Fy / 0.35*bw = 100.5*280 / 0.35*120 = 0.67 m
S_3 = 0.600 m

Use: 1ϕ8/140mm

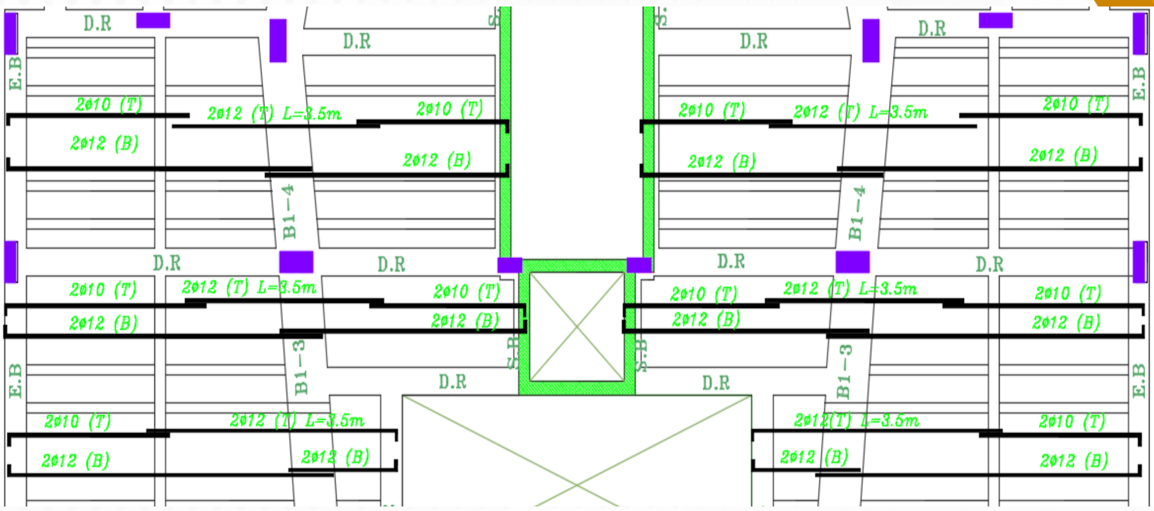


Figure 2.25 : Reinforcement detailing for Rib (2)

3) Shear Force (R.B.3)

Span AB (1)

- X=0.1+0.275 = 0.375 m
- Vus=15.48 kN

- Vud = Vus – (W*X) = 15.48 – (8.336 * 0.375) = 12.35 kN

- ½φVc < Vud < φVc 11.13 < 12.35 < 22.26

S_1 = d/2 = 0.275/2 = 0.1375 m
S_2 = Av*Fy / 0.35*bw = 100.5*280 / 0.35*120 = 0.67 m
S_3 = 0.600 m

Use: 1ϕ8/140mm

Span AB (2)

- $X = 0.5 + 0.275 = 0.775$ m
- $V_{us} = 20.34$ kN
- $V_{ud} = V_{us} - (W*X) = 20.34 - (8.336 * 0.775) = 13.88$ kN
- $\frac{1}{2}\varphi V_c < V_{ud} < \varphi V_c$ $\quad 11.13 < 13.88 < 22.26$

S1 = d/2 = 0.275/2 = 0.1375 m
S2 = Av*Fy / 0.35*bw = 100.5*280 / 0.35*120 = 0.67 m
S3 = 0.600 m

<u>Use:</u> 1ϕ8/140mm

Span BC (3)

- $X = 0.5 + 0.275 = 0.775$ m
- $V_{us} = 7.36$ kN
- $V_{ud} = V_{us} - (W*X) = 7.36 - (8.336 * 0.775) = 0.89$ kN
- $V_{ud} < \frac{1}{2}\varphi V_c < \varphi V_c$ $\quad 0.89 < 11.13 < 22.26$

<u>Use the minimum reinforcement :</u> 1ϕ8/200mm

Span BC (4)

- $X = 0.175 + 0.275 = 0.45$ m
- $V_{us} = 17.48$ kN
- $V_{ud} = V_{us} - (W*X) = 17.48 - (8.336 * 0.45) = 13.72$ kN
- $\frac{1}{2}\varphi V_c < V_{ud} < \varphi V_c$ $\quad 11.13 < 13.72 < 22.26$

S1 = d/2 = 0.275/2 = 0.1375 m
S2 = Av*Fy / 0.35*bw = 100.5*280 / 0.35*120 = 0.67 m
S3 = 0.600 m

<u>Use:</u> 1ϕ8/140mm

Span C (5)

- X=0.175+0.275 = 0.45 m
- Vus=20.15 kN
- Vud = Vus – (W*X) = 20.15 – (8.336 * 0.45) = 16.39 kN

- ½φVc < Vud < φVc   11.13 < 16.39 < 22.26

S1 = d/2 = 0.275/2 = 0.1375 m
S2 = Av*Fy / 0.35*bw = 100.5*280 / 0.35*120 = 0.67 m
S3 = 0.600 m

<u>Use:</u> **1ϕ8/140mm**

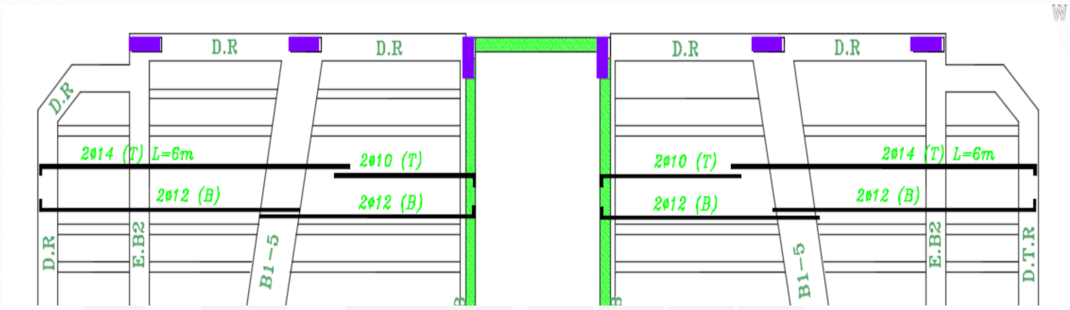


Figure 2.26 : Reinforcement detailing for Rib (3)

- **<u>Shrinkage for R.B</u>**
- As = ρ * Ag

  = (0.0018 * 70 * 1000) = 126 mm^2

<u>Use: 1ϕ10/Block</u>

Chapter (3)

Design of Beams

Ch3 : Beams

3.1 Introduction:

• In many instances in structural and machine design, members must resist forces applied laterally or transversely to their axes. Such members are called beams.

• When the loads are not at a right angle to the beam, they also produce axial forces in the beam.

• The transverse loading of a beam may consist of distrubution loads W_1, W_2 , … expressed in Newton's per meter, pounds per feet, or their multiples, kilo Newton's per meter and kip per feet.

Types of loads in beams :

1) Concentrated loads
2) Distributed loads

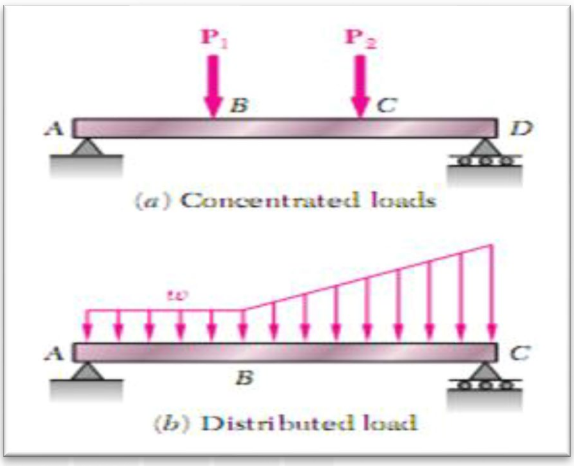


Figure 3.1 : Types of loads in beams

3.2 Definition:

• Beam is a structural element generally carries vertical gravitational forces but can also be used to carry horizontal loads (i.e. loads due to an earthquake or wind)

• The loads carried by a beam are transferred to columns, walls, or girders, which then transfer the force to adjacent structural compression members.

• In Light frame construction the joists rest on the beam.
Beams are characterized by their profile (the shape of their cross-section), their length, and their material. In contemporary construction, beams are typically made of steel, reinforced concrete or wood.

3.3 Type of beams :

1. Hidden beam.
2. Drop beam.
3. Inverted beam.

3.4 Design of Beams :

1) Design of section Beam (B.1):

- DL = $10.71*(\frac{6.25}{2} + \frac{5.8}{2}) + o.31*24 = 71.90$ kN

- LL = 2*$(\frac{6.25}{2}+\frac{5.8}{2})$ = 12.05 kN
- Wu = 1.2*71.90 + 1.6*12.05 = 105.56 kN/m
- Mu = $\frac{WuL^2}{8}$ = $\frac{105.56*4.6^2}{8}$ = 279.2 kN.m
- Mn = $\frac{Mu}{\varphi}$ = $\frac{279.2}{0.9}$ = 310.22 kN.m
- ρmin = $\frac{\sqrt{Fc'}}{4Fy}$ = $\frac{\sqrt{28}}{4*420}$ = 0.00315
- ρmin = $\frac{1.4}{Fy}$ = $\frac{1.4}{420}$ = 0.00333

Take 0.00333

- ρb = $\left(\frac{0.85*0.85Fc'}{Fy}\right)\left(\frac{600}{600+Fy}\right)$ = 0.0283
- ρmax = $\left(\frac{0.003+\frac{Fy}{Es}}{0.008}\right)$ρb = $\left(\frac{0.003+\frac{420}{200000}}{0.008}\right)$0.0283 = 0.018

Use : ρ = 0.011

- R = ρFy $\left(1-0.59\frac{\rho Fy}{Fc'}\right)$ = 0.011*420 $\left(1-0.59\frac{0.011*420}{28}\right)$ = 4.17
- b = $\frac{Mn}{Rd^2}$ = $\frac{310.22*10^6}{4.17*270^2}$ = 1020.5 mm

<u>Use</u> : b = 1000 mm

h = 310 mm

2) Design of section Beam (B.2):

- DL = 10.71*($\frac{3.45}{2}$ + $\frac{6.25}{2}$) + o.31*24 = 57.85 kN
- LL = 2*($\frac{3.45}{2}$ + $\frac{6.25}{2}$) = 9.7 kN
- Wu = 1.2*57.85 + 1.6*9.7 = 84.94 kN/m
- Mu = $\frac{WuL^2}{8}$ = $\frac{84.94*4.25^2}{8}$ = 191.77 kN.m
- Mn = $\frac{Mu}{\varphi}$ = $\frac{191.77}{0.9}$ = 213.08 kN.m
- ρmin = $\frac{\sqrt{Fc\prime}}{4Fy}$ = $\frac{\sqrt{28}}{4*420}$ = 0.00315
- ρmin = $\frac{1.4}{Fy}$ = $\frac{1.4}{420}$ = 0.00333
- ρb = $\left(\frac{0.85*0.85Fc'}{Fy}\right)$ ($\frac{600}{600+Fy}$) = 0.0283
- ρmax = ($\frac{0.003+\frac{Fy}{Es}}{0.008}$)ρb = ($\frac{0.003+\frac{420}{200000}}{0.008}$)0.0283 = 0.018

  Use : ρ = 0.01

- R = ρFy $\left(1 - 0.59\frac{\rho Fy}{Fc\prime}\right)$ = 0.01*420 $\left(1 - 0.59\frac{0.01*420}{28}\right)$ = 3.8283
- b = $\frac{Mn}{Rd^2}$ = $\frac{213.08*10^6}{3.8283*270^2}$ = 763.5 mm

Use : b = 800 mm

h = 310 mm

1) Design of (B.1):

h = 310 mm
Cover = 40 mm
d = h−cover= 270 mm
b = 1000 mm
F'c = 28 Mpa
Fy = 420 Mpa

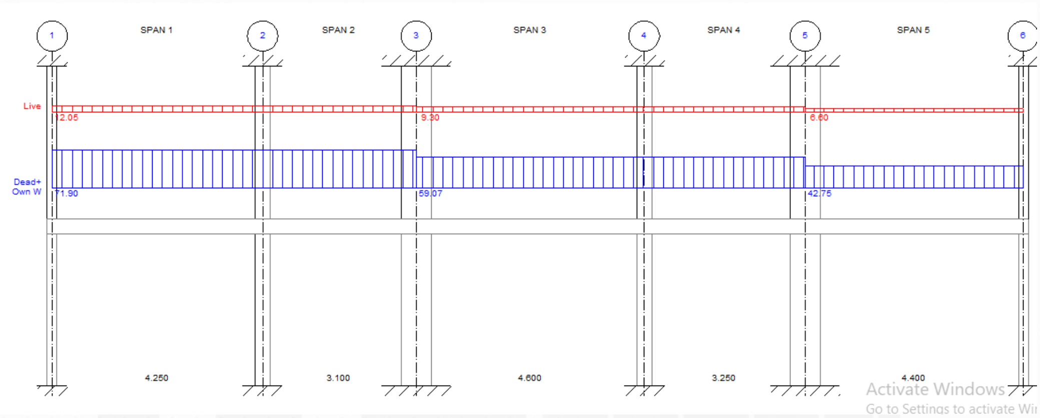


Figure 3.2 : Load distribution for Beam (1)

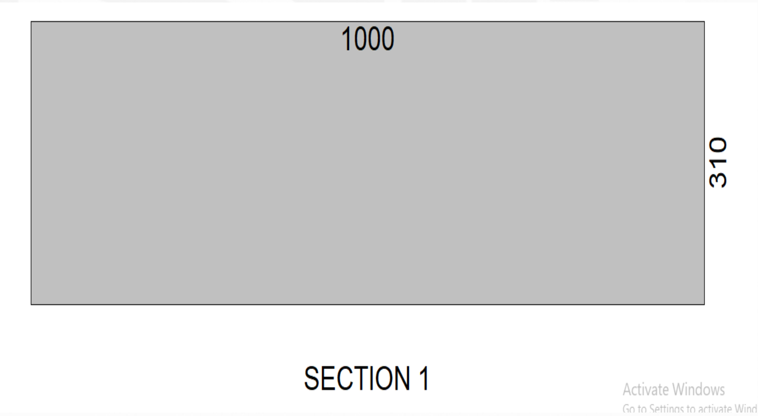


Figure 3.3 : section for Beam (1)

Shear Force

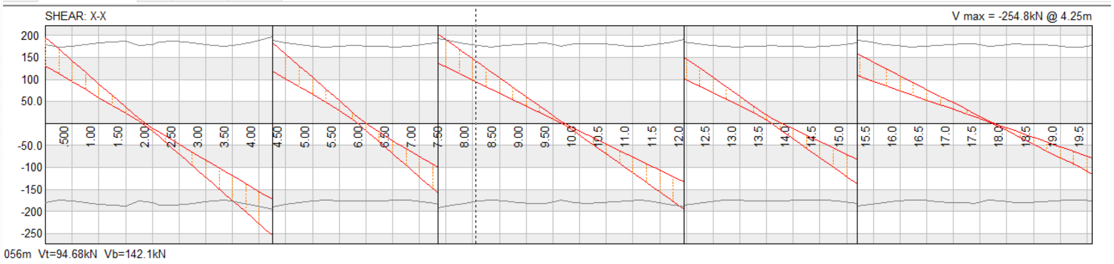


Figure 3.4 : Shear force diagram for Beam (1)

Bending Moment

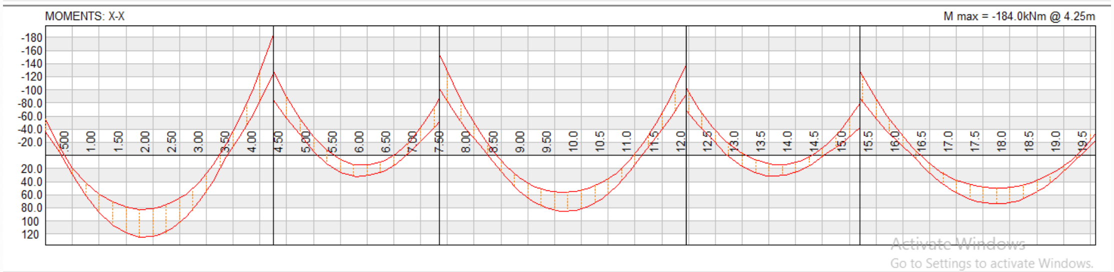


Figure 3.5 : Bending moment diagram for Beam (1)

- Equations used for design :

. ρ = $\mathbf{\frac{0.85Fc'}{Fy}\left(1-\sqrt{1-\frac{2.354Mu}{0.9Fc'bd^2}}\right)}$ **. As = ρ*b*d**

. Asmin = 0.0033*1000*270 = 891 $\mathbf{mm^2}$

1) Moment for 0 m = **-** 55.35 kN.m

- ρ = $\frac{0.85*28}{420}\left(1-\sqrt{1-\frac{2.354*55.35*10^6}{0.9*28*1000*270^2}}\right) = 0.002$
- As = 0.002*1000*270 = 540 mm^2

- ρmax = $\frac{0.85*0.85*28}{420}\left(\frac{0.003}{0.003+0.004}\right) = 0.0206$
- use Asmin = 891 mm^2

2) Moment for 1.75 m = 124.55 kN.m

ρ = 0.0047 **As =** 1274 mm^2 **use As =** 1274 mm^2

3) Moment for 4.25 m = **-** 184 kN.m

ρ = 0.007 **As =** 1924.7 mm^2 **use As =** 1924.7 mm^2

4) Moment for 5.8 m = 32.05 kN.m

ρ = 0.001 **As =** 270 mm^2 **use** $\mathbf{As_{min}}$ **=** 891 mm^2

5) Moment for 7.35 m = **-** 154.4 kN.m

ρ = 0.0059 **As =** 1596.8 mm^2 **use As =** 1596.8 mm^2

6) Moment for 9.35 m = 85.43 kN.m

ρ = 0.003 **As =** 825 mm^2 **use** $\mathbf{As_{min}}$ **=** 891 mm^2

7) Moment for 11.95 m = **-** 137 kN.m

ρ = 0.005 **As =** 1375 mm^2 **use As =** 1375 mm^2

8) Moment for 13.58 m = 31.82 kN.m

ρ = 0.001 **As =** 270 mm^2 **use** $\mathbf{As_{min}}$ **=** 891 mm^2

9) Moment for 15.2 m = **-** 127.55 kN.m

ρ = 0.0047 **As =** 1292.5 mm^2 **use As =** 1292.5 mm^2

10) Moment for 17.65 m = 73.86 kN.m

ρ = 0.0026 **As =** 715 mm^2 **use** $\mathbf{As_{min}}$ **=** 891 mm^2

11) Moment for 19.6 m = **-** 33.14 kN.m

ρ = 0.0012 **As =** 330 mm^2 **use** $\mathbf{As_{min}}$ **=** 891 mm^2

2) <u>Design of (B.2)</u>:

h = 310 mm
Cover = 40 mm
d = h−cover= 270 mm
b = 800 mm
F'c = 28 Mpa
Fy = 420 Mpa

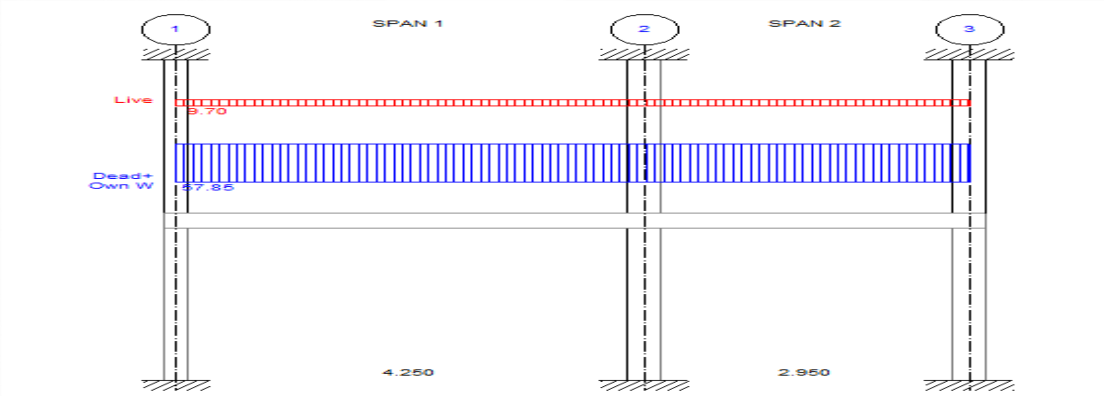


Figure 3.6 : Load distribution for Beam (2)

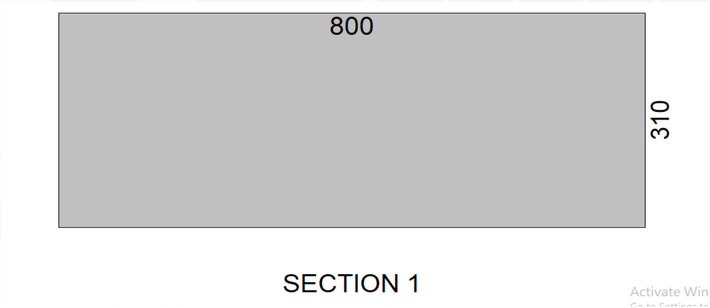


Figure 3.7 : section for Beam (2)

Shear Force

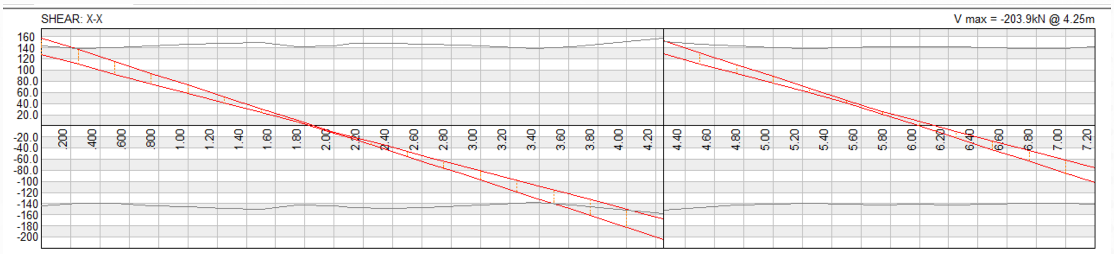


Figure 3.8 : Shear force diagram for Beam (2)

Bending Moment

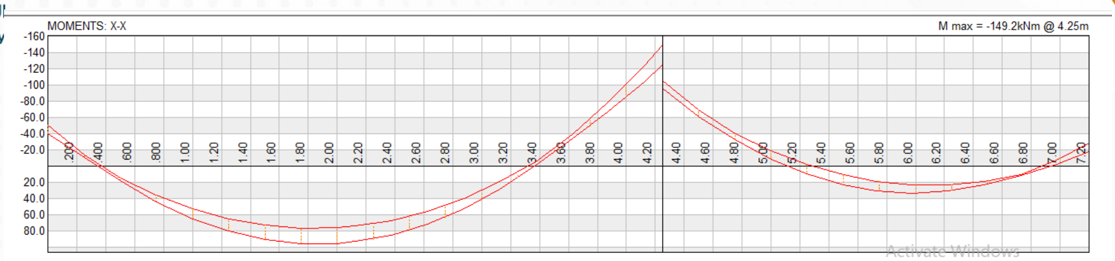


Figure 3.9 : Bending moment diagram for Beam (2)

- Equations used for design :

. $\rho = \frac{0.85Fc'}{Fy}\left(1 - \sqrt{1 - \frac{2.354Mu}{0.9Fc'bd^2}}\right)$ **. As = ρ*b*d**

. Asmin = 0.0033*270*800 = 712.8 mm^2

1) Moment for 0 m = **-** 50.49 kN.m

ρ = 0.0023   **As =** 506 mm^2 **use Asmin =** 712.8 mm^2

2) Moment for 1.75 m = 95.85 kN.m

ρ = 0.0044   **As =** 968 mm^2 **use As =** 968 mm^2

3) Moment for 4.25 m = **-** 149.23 kN.m

ρ = 0.007   **As =** 1540 mm^2 **use As =** 1540 mm^2

4) Moment for 5.975 m = 33.58 kN.m

ρ = 0.0015   **As =** 330 mm^2 **use Asmin =** 712.8 mm^2

5) Moment for 7.2 m = - 28.41 kN.m

ρ = 0.0013   **As =** 286 mm^2 **use Asmin =** 712.8 mm^2

3) Design of (B.3):

h = 410 mm
Cover = 40 mm
d = h−cover= 370 mm
bf = 350 mm
bw = 250 mm
F'c = 28 Mpa
Fy = 420 Mpa

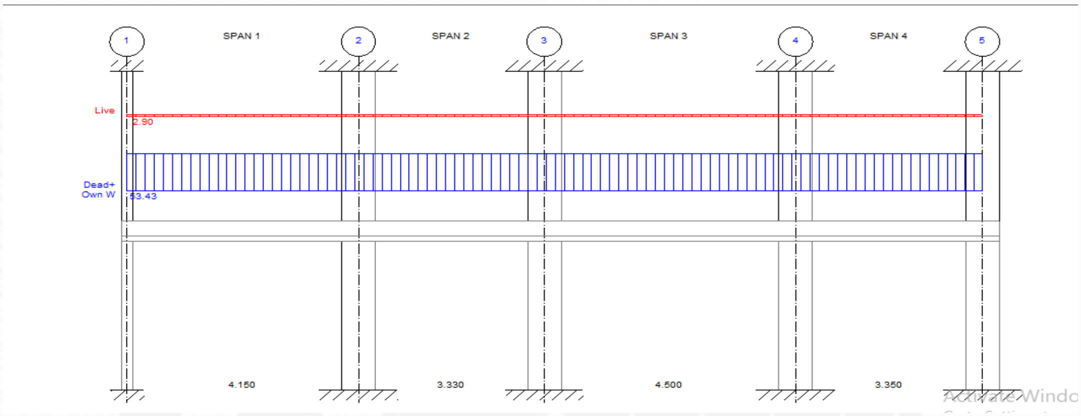


Figure 3.10 : Load distribution for Beam (3)

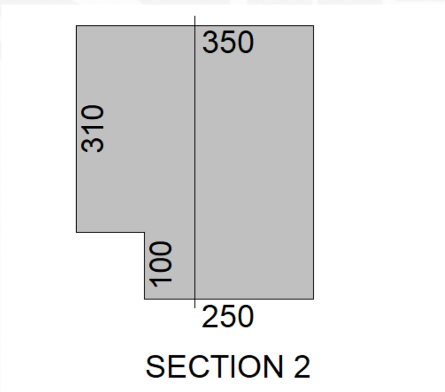


Figure 3.11 : section for Beam (3)

Shear Force

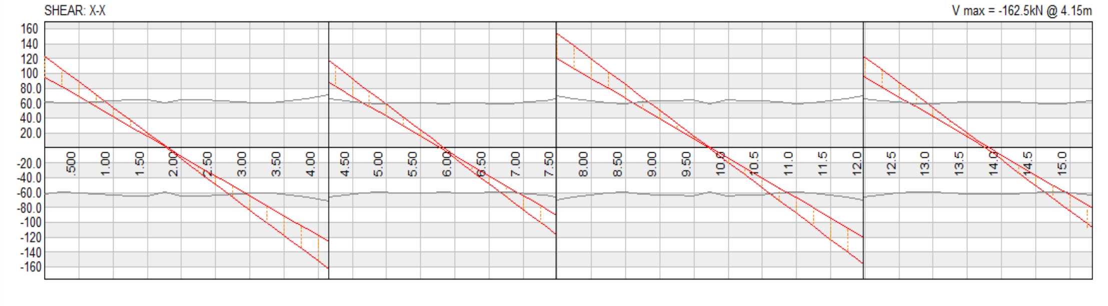


Figure 3.12 : Shear force diagram for Beam (3)

Bending Moment

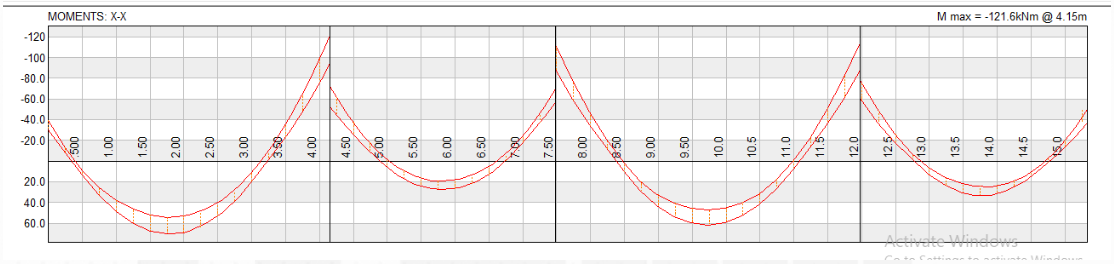


Figure 3.13 : Bending moment diagram for Beam (3)

- Equations used for design :

. $\rho = \frac{0.85Fc'}{Fy}\left(1 - \sqrt{1 - \frac{2.354Mu}{0.9Fc'bd^2}}\right)$

. Asmin-ve = 0.0033*250*370 = 305.25 $\mathbf{mm^2}$ **. As-ve = ρ*bw*d**

. Asmin+ve = 0.0033*350*370 = 427.35 $\mathbf{mm^2}$ **. As+ve = ρ*bf*d**

1) Moment for 0 m = **-** 39.68 kN.m

ρ = 0.003 **As =** 281.25 mm^2 **use As**min = 305.25 mm^2

2) Moment for 1.75 m = 70.59 kN.m

ρ = 0.0039 **As =** 511.87 mm^2 **use As** = 511.87 mm^2

3) Moment for 4.15 m = **-** 121.62 kN.m

ρ = 0.001 **As =** 93.75 mm^2 **use As**min **=** 305.25 mm^2

4) Moment for 5.82 m = 27.67 kN.m

ρ = 0.001 **As =** 131.25 mm^2 **use As**min **=** 427.35 mm^2

5) Moment for 7.48 m = **-** 112.82 kN.m

ρ = 0.0092 **As =** 862.5 mm^2 **use As =** 862.5 mm^2

6) Moment for 9.73 m = 61.76 kN.m

ρ = 0.0034 **As =** 446.25 mm^2 **use As =** 446.25 mm^2

7) Moment for 11.98 m = **-** 114.4 kN.m

ρ = 0.0094 **As =** 881.25 mm^2 **use As =** 881.25 mm^2

8) Moment for 13.65m = 33.27 kN.m

ρ = 0.0018 **As =** 236.25 mm^2 **use As**min **=** 427.35 mm^2

9) Moment for 15.33 m = **-** 49.88 kN.m

ρ = 0.0039 **As =** 365.63 mm^2 **use As =** 365.63 mm^2

3.5 Shear Design For Beams :

support : (600/300/200)

1) Shear Force (B.1)

- $\varphi Vc = 0.75 \times 0.17 \times \sqrt{fc'} \times b \times d \times 10^{-3}$
  $\varphi Vc = 0.75 \times 0.17 \times \sqrt{28} \times 1000 \times 270 \times 10^{-3}$ = **182.15 kN**
- $½\ \varphi Vc = 91.07\ kN$

Span AB (1)

- **W = 1.2*DL + 1.6*LL**
  W = 1.2*71.9 + 1.6*12.05 = 105.56 kN/m

  - X=0.1+0.270 = 0.37 m
  - Vus=195.17 kN

  - Vud = Vus – (W*X) = 195.17 – (105.56 * 0.37) = 156.11 kN

- $½\varphi Vc < Vud < \varphi Vc$ 91.07 < 156.11 < 182.15

S_1 = d/2 = 0.270/2 = 0.135 m
S_2 = Av*Fy / 0.35*bw = 157.07*420 / 0.35*1000 = 0.1884 m
S_3 = 0.600 m

Use: 1ɸ10/140mm

Span AB (2)

- **W = 1.2*DL + 1.6*LL**
  W = 1.2*71.9 + 1.6*12.05 = 105.56 kN/m

  - X=0.15+0.27 = 0.42 m
  - Vus=254.8 kN
  - Vud = Vus – (W*X) = 254.8 – (105.56 * 0.42) = 210.46 kN
  - φVc < Vud 182.15 < 210.46
  - Vs = (Vud - φVc) /φ = (209.93 – 182.15) / 0.75 = 37.04 kN
  - Vc1 = 2 φVc / φ = 2*182.15/ 0.75 = 485.73 kN
  - Vs < Vc1 37.04 < 485.73

S1 = Av*Fy*d / Vs = 157.07*420*270 / 37.04*10^6= 0.48 m
S2 = d/2 = 0.270/2 = 0.135 m
S3 = Av*Fy / 0.35*bw = 157.07*420 / 0.35*100 = 0.1884 m
S4 = 0.600 m

Use: 1ϕ10/140mm

Span BC (3)

- **W = 1.2*DL + 1.6*LL**
  W = 1.2*71.9 + 1.6*12.05 = 105.56 kN/m

  - X=0.1+0.270 = 0.37 m
  - Vus=183.45 kN
  - Vud = Vus – (W*X) = 183.45 – (105.56 * 0.37) = 144.39 kN

- ½φVc < Vud < φVc 91.07 < 144.39 < 182.15

S1 = d/2 = 0.270/2 = 0.135 m
S2 = Av*Fy / 0.35*bw = 157.07*420 / 0.35*1000 = 0.1884 m
S3 = 0.600 m

Use: 1ϕ10/140mm

Span BC (4)

- **W = 1.2*DL + 1.6*LL**
  W = 1.2*71.9 + 1.6*12.05 = 105.56 kN/m

  - X=0.3+0.270 = 0.57 m
  - Vus=158.71 kN

  - Vud = Vus – (W*X) = 158.71 – (105.56 * 0.57) = 98.5 kN

- ½φVc < Vud < φVc 91.07 < 98.5 < 182.15

S1 = d/2 = 0.270/2 = 0.135 m
S2 = Av*Fy / 0.35*bw = 157.07*420 / 0.35*1000 = 0.1884 m
S3 = 0.600 m

Use: 1ϕ10/140mm

Span CD (5)

- **W = 1.2*DL + 1.6*LL**
  W = 1.2*59.07 + 1.6*9.3 = 85.76 kN/m

  - X=0.3+0.27 = 0.57 m
  - Vus=202.7 kN

  - Vud = Vus – (W*X) = 202.70 – (85.76 * 0.570) = 153.8 kN

- ½φVc < Vud < φVc 91.07 < 153.8 < 182.15

S1 = d/2 = 0.27/2 = 0.135 m
S2 = Av*Fy / 0.35*bw = 157.07*420 / 0.35*1000 = 0.1884 m
S3 = 0.600 m

Use: 1ϕ10/140mm

Span CD (6)

- **W = 1.2*DL + 1.6*LL**
  W = 1.2*59.07 + 1.6*9.3 = 85.76 kN/m

  - X=0.15+0.27 = 0.42 m
  - Vus=194.83 kN
  - Vud = Vus – (W*X) = 194.83 – (85.76 * 0.42) = 158.8 kN

- $\frac{1}{2}\varphi Vc < Vud < \varphi Vc$ 91.07 < 158.8 < 182.15

S1 = d/2 = 0.27/2 = 0.135 m
S2 = Av*Fy / 0.35*bw = 157.07*420 / 0.35*1000 = 0.1884 m
S3 = 0.600 m

Use: 1ϕ10/140mm

Span DE (7)

- **W = 1.2*DL + 1.6*LL**
  W = 1.2*59.07 + 1.6*9.3 = 85.76 kN/m

  - X=0.15+0.27 = 0.42 m
  - Vus=150.75 kN
  - Vud = Vus – (W*X) = 150.75 – (85.76 * 0.42) = 114.7 kN

- $\frac{1}{2}\varphi Vc < Vud < \varphi Vc$ 91.07 < 114.7 < 182.15

S1 = d/2 = 0.27/2 = 0.135 m
S2 = Av*Fy / 0.35*bw = 157.07*420 / 0.35*1000 = 0.1884 m
S3 = 0.600 m

Use: 1ϕ10/140mm

Span DE (8)

- **W = 1.2*DL + 1.6*LL**
  W = 1.2*59.07 + 1.6*9.3 = 85.76 kN/m

  - X=0.3+0.27 = 0.57 m
  - Vus=138.11 kN
  - Vud = Vus – (W*X) = 138.11 – (85.76 * 0.57) = 89.22 kN

- Vud < ½φVc < φVc   89.22 < 91.07 < 182.15

Use the minimum reinforcement : 1ϕ10/200mm

Span EF (9)

- **W = 1.2*DL + 1.6*LL**
  W = 1.2*42.75 + 1.6*6.6 = 61.86 kN/m

  - X=0.3+0.27 = 0.57 m
  - Vus=157.65 kN
  - Vud = Vus – (W*X) = 157.65 – (61.86 * 0.57) = 122.38 kN

- ½φVc < Vud < φVc   91.07 < 122.38 < 182.15

S1 = d/2 = 0.27/2 = 0.135 m
S2 = Av*Fy / 0.35*bw = 157.07*420 / 0.35*1000 = 0.1884 m
S3 = 0.600 m

Use: 1ϕ10/140mm

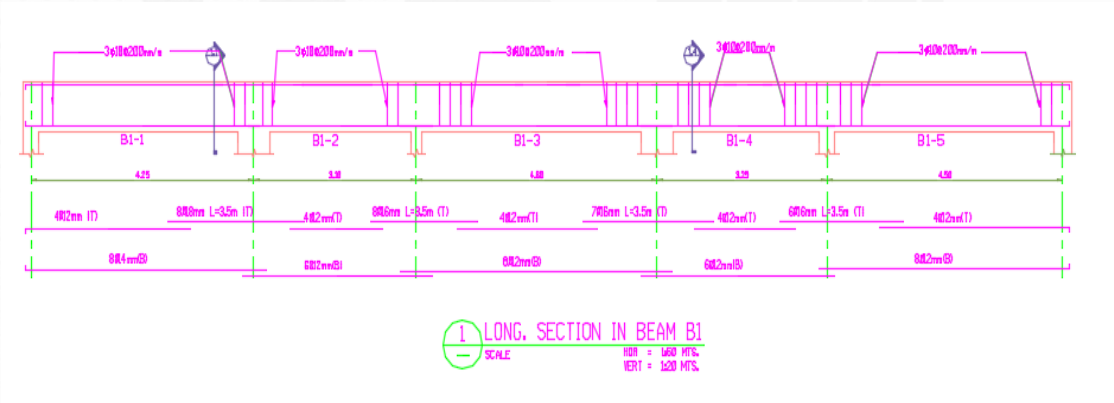
Span EF (10)

- **W = 1.2*DL + 1.6*LL**

  W = 1.2*42.75 + 1.6*6.6 = 61.86 kN/m

- X=0.1+0.27 = 0.37 m
- Vus=115.19 kN
- Vud = Vus – (W*X) = 115.19 – (61.86 * 0.37) = 92.3 kN
- Vud < ½φVc < φVc   92.3 < 91.07 < 182.15

<u>Use the minimum reinforcement :</u> 1ф10/200mm



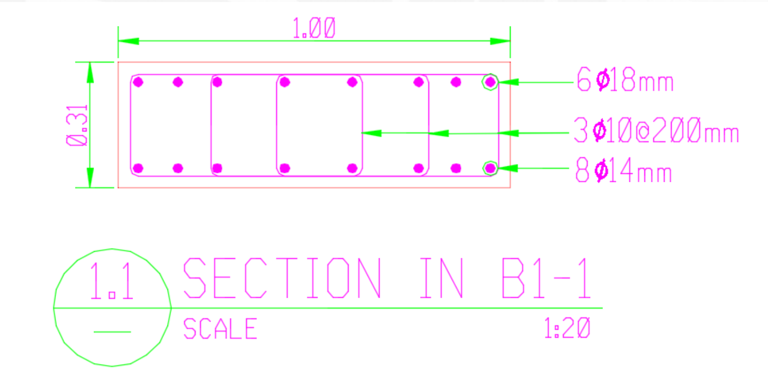


Figure 3.14 : Reinforcement detailing for Beam (1)

2) Shear Force (B.2)

- $\varphi Vc = 0.75 \times 0.17 \times \sqrt{fc'} \times b \times d \times 10^{-3}$
  $\varphi Vc = 0.75 \times 0.17 \times \sqrt{28} \times 800 \times 270 \times 10^{-3}$ = 145.7 kN
- $½\ \varphi Vc = 72.85$ kN
- **W = 1.2*DL + 1.6*LL**
  W = 1.2*57.85 + 1.6*9.7 = 84.94 kN/m

Span AB (1)

- X=0.1+0.275 = 0.375 m
- Vus=157.95 kN
- Vud = Vus – (W*X) = 157.95 – (84.94 * 0.375) = 126.1 kN
- $½\varphi Vc < Vud < \varphi Vc$ 72.85 < 126.1 < 145.7

S1 = d/2 = 0.27/2 = 0.135 m
S2 = Av*Fy / 0.35*bw = 157.07*420 / 0.35*800 = 0.235 m
S3 = 0.600 m

Use: 1ϕ10/140mm

Span AB (2)

- X=0.15+0.27 = 0.42 m
- Vus=203.88 kN
- Vud = Vus – (W*X) = 203.88 – (84.94 * 0.42) = 168 kN
- $\varphi Vc < Vud$ 145.7 < 168
- Vs = (Vud - φVc) /φ = (168 – 145.7) / 0.75 = 29.7 kN
- Vc1 = 2 φVc / φ = 2*145.7/ 0.75 = 388.5 kN
- Vs < Vc1 29.7 < 388.5

S1 = Av*Fy*d / Vs = 157.07*420*270 / 29.7*10^6= 0.6 m
S2 = d/2 = 0.270/2 = 0.135 m
S3 = Av*Fy / 0.35*bw = 157.07*420 / 0.35*800 = 0. 235 m
S4 = 0.600 m

Use: 1ϕ10/140mm

Span BC (3)

- X=0.15+0.27 = 0.42 m
- Vus=152.37 kN
- Vud = Vus – (W*X) = 152.37 – (84.94 * 0.42) = 116.7 kN

- ½φVc < Vud < φVc   72.85 < 116.7 < 145.7

S_1 = d/2 = 0.27/2 = 0.135 m
S_2 = Av*Fy / 0.35*bw = 157.07*420 / 0.35*800 = 0.235 m
S_3 = 0.600 m

Use: 1ϕ10/140mm

Span BC (4)

- X=0.15+0.27 = 0.42 m
- Vus=102.64 kN
- Vud = Vus – (W*X) = 102.64 – (84.94 * 0.42) = 66.96 kN

- Vud < ½φVc < φVc   66.96 < 72.85 < 145.7

Use the minimum reinforcement : 1ϕ10/200mm

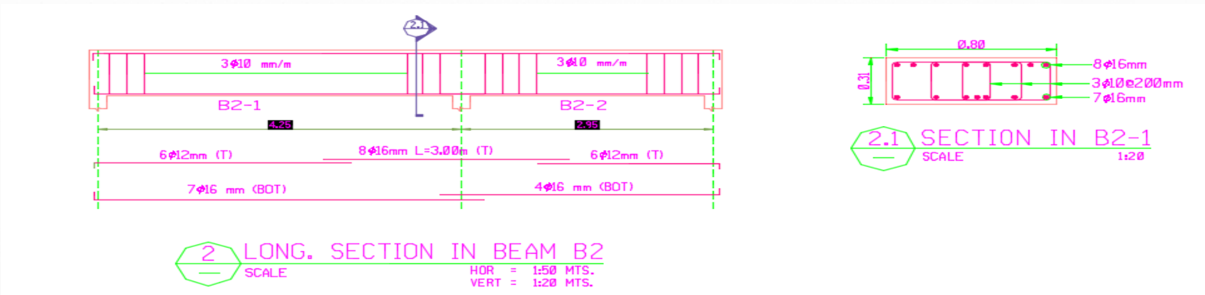


Figure 3.15 : Reinforcement detailing for Beam (2)

3) Shear Force (B.3)

- **$\varphi Vc = 0.75 \times 0.17 \times \sqrt{fc'} \times b \times d \times 10^{-3}$**

$\varphi Vc = 0.75 \times 0.17 \times \sqrt{28} \times 350 \times 270 \times 10^{-3}$ = 63.75 kN

- **½ $\varphi Vc = 31.87kN$**
- **W = 1.2*DL + 1.6*LL**

  W = 1.2*53.43 + 1.6*2.9 = 68.75 kN/m

Span AB (1)

- X=0.1+0.27 = 0.37 m
- Vus=123.09 kN
- Vud = Vus – (W*X) = 123.09 – (68.75 * 0.37) = 97.65 kN
- φVc < Vud   63.75 < 97.65
- Vs = (Vud - φVc) /φ = (97.65 – 63.75) / 0.75 = 45.2 KN
- Vc1 = 2 φVc / φ = 2*63.75/ 0.75 = 170 kN
- Vs < Vc1   45.2 < 170

S1 = Av*Fy*d / Vs = 157.07*420*270 / 45.2*10^6= 0.39 m
S2 = d/2 = 0.27/2 = 0.135 m
S3 = Av*Fy / 0.35*bw = 157.07*420 / 0.35*350 = 0.538 m
S4 = 0.600 m

Use: 1ϕ10/140mm

Span AB (2)

- $X = 0.3 + 0.27 = 0.57$ m
- $V_{us} = 162.49$ kN
- $V_{ud} = V_{us} - (W*X) = 162.49 - (68.75 * 0.57) = 123.3$ kN

- $\varphi V_c < V_{ud}$ $\quad 63.75 < 123.3$

- $V_s = (V_{ud} - \varphi V_c) / \varphi = (123.3 - 63.75) / 0.75 = 79.4$ kN
- $V_{c1} = 2\,\varphi V_c / \varphi = 2*63.75 / 0.75 = 170$ kN

- $V_s < V_{c1}$ $\quad 79.4 < 170$

$S_1 = A_v*F_y*d / V_s = 157.07*420*270 / 79.4*10^6 = 0.224$ m
$S_2 = d/2 = 0.270/2 = 0.135$ m
$S_3 = A_v*F_y / 0.35*b_w = 157.07*420 / 0.35*350 = 0.538$ m
$S_4 = 0.600$ m

Use: 1ϕ10/140mm

Span BC (3)

- $X = 0.3 + 0.27 = 0.57$ m
- $V_{us} = 117$ kN
- $V_{ud} = V_{us} - (W*X) = 117 - (68.75 * 0.57) = 77.8$ kN

- $\varphi V_c < V_{ud}$ $\quad 63.75 < 77.8$

- $V_s = (V_{ud} - \varphi V_c) / \varphi = (77.8 - 63.75) / 0.75 = 18.73$ kN
- $V_{c1} = 2\,\varphi V_c / \varphi = 2*63.75 / 0.75 = 170$ kN

- $V_s < V_{c1}$ $\quad 18.73 < 170$

$S_1 = A_v*F_y*d / V_s = 157.07*420*270 / 18.73*10^6 = 0.95$ m
$S_2 = d/2 = 0.270/2 = 0.135$ m
$S_3 = A_v*F_y / 0.35*b_w = 157.07*420 / 0.35*350 = 0.538$ m
$S_4 = 0.600$ m

Use: 1ϕ10/140mm

Span BC (4)

- X=0.3+0.27 = 0.57 m
- Vus=115.77 kN
- Vud = Vus – (W*X) = 115.77 – (68.75 * 0.57) = 76.58 kN

- $\varphi Vc < Vud$ 63.75 < 76.58

- Vs = (Vud - φVc) /φ = (76.58 – 63.75) / 0.75 = 17.11 kN
- Vc1 = 2 φVc / φ = 2*63.75/ 0.75 = 170 kN

- Vs < Vc1 17.11 < 170

S1 = Av*Fy*d / Vs = 157.07*420*270 / 17.11*10^6= 1.04 m
S2 = d/2 = 0.270/2 = 0.135 m
S3 = Av*Fy / 0.35*bw = 157.07*420 / 0.35*350 = 0.538 m
S4 = 0.600 m

Use: 1ϕ10/140mm

Span CD (5)

- X=0.3+0.27 = 0.57 m
- Vus=154.94 kN
- Vud = Vus – (W*X) = 154.94 – (68.75 * 0.57) = 115.75 kN

- $\varphi Vc < Vud$ 63.75 < 115.75

- Vs = (Vud - φVc) /φ = (115.75 – 63.75) / 0.75 =69.3 kN
- Vc1 = 2 φVc / φ = 2*63.75/ 0.75 = 170 kN

- Vs < Vc1 69.3 < 170

S1 = Av*Fy*d / Vs = 157.07*420*270 / 69.3*10^6= 0.257 m
S2 = d/2 = 0.27/2 = 0.135 m
S3 = Av*Fy / 0.35*bw = 157.07*420 / 0.35*350 = 0.538 m
S4 = 0.600 m

Use: 1ϕ10/140mm

Span CD (6)

- X=0.3+0.27 = 0.57 m
- Vus=155.65 kN
- Vud = Vus – (W*X) = 155.65 – (68.75 * 0.57) = 116.46 kN
- φVc < Vud 63.75 < 116.46
- Vs = (Vud - φVc) /φ = (116.46 – 63.75) / 0.75 = 67.28 kN
- Vc1 = 2 φVc / φ = 2*63.75/ 0.75 = 170 kN
- Vs < Vc1 67.28 < 170

S1 = Av*Fy*d / Vs = 157.07*420*270 / 67.28 *10^6= 0.264 m
S2 = d/2 = 0.27/2 = 0.135 m
S3 = Av*Fy / 0.35*bw = 157.07*420 / 0.35*350 = 0.538 m
S4 = 0.600 m

<u>Use:</u> 1ф10/140mm

Span DE (7)

- X=0.3+0.27 = 0.57 m
- Vus=123.73 kN
- Vud = Vus – (W*X) = 123.73 – (68.75 * 0.57) = 84.54 kN
- φVc < 63.75 < 84.54

Vud

- Vs = (Vud - φVc) /φ = (84.54 – 63.75) / 0.75 = 27.7 kN
- Vc1 = 2 φVc / φ = 2*63.75/ 0.75 = 170 kN
- Vs < Vc1 27.7 < 170

S1 = Av*Fy*d / Vs = 157.07*420*270 / 27.7 *10^6= 0.64 m
S2 = d/2 = 0.275/2 = 0.1375 m
S3 = Av*Fy / 0.35*bw = 157.07*420 / 0.35*350 = 0.538 m
S4 = 0.600 m

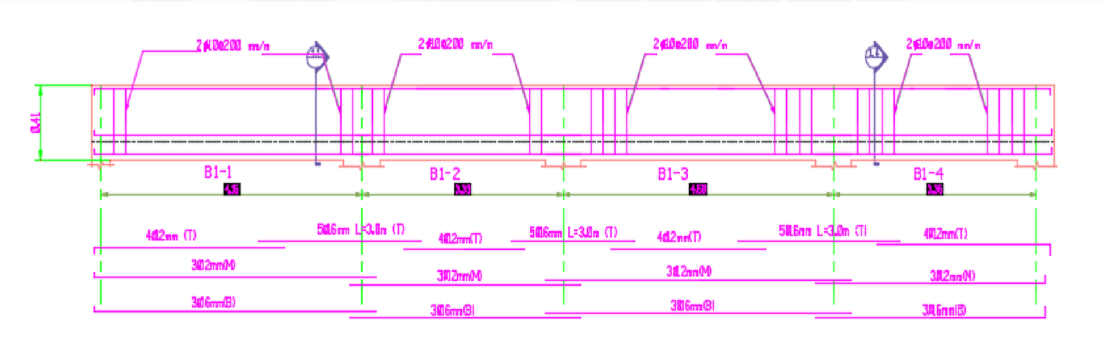
<u>Use:</u> 1ф10/140mm

Span DE (8)

- X=0.3+0.27 = 0.57 m
- Vus=107.23 kN
- Vud = Vus – (W*X) = 107.23 – (68.75 * 0.57) = 68 kN
- φVc < Vud 63.75 < 68
- Vs = (Vud - φVc) /φ = (68 – 63.75) / 0.75 = 4.68 kN
- Vc1 = 2 φVc / φ = 2*63.75/ 0.75 = 170 kN
- Vs < Vc1 4.68 < 170

S1 = d/2 = 0.275/2 = 0.1375 m
S2 = Av*Fy / 0.35*bw = 157.07*420 / 0.35*350 = 0.538 m
S3 = 0.600 m

<u>Use:</u> 1ϕ10/140mm



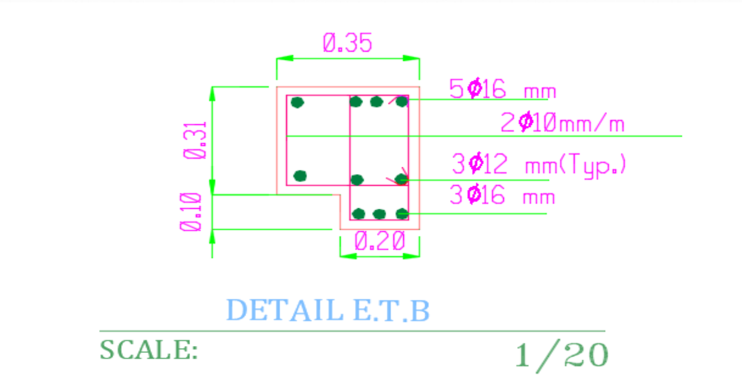


Figure 3.16 : Reinforcement detailing for Beam (3)

Chapter (4)

Design of Columns

Ch4 : Columns

4.1 Introduction:

• Columns are defined as vertical structural members subjected not only to compression force but also to bending moment about one or both axes of the section, and with a height at least three times its least lateral dimension.

• They collect loads from the different floors and then transfer it to the soil through the foundation.

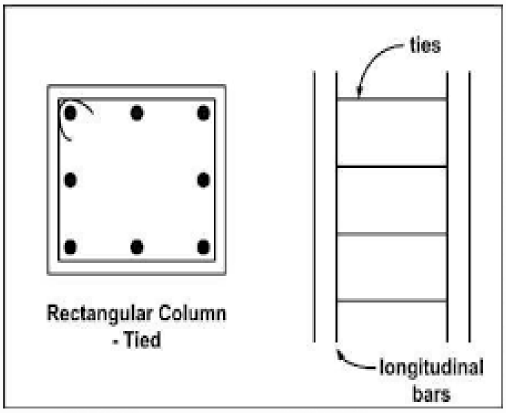
• The axial load column is subjected to compression that should achieve a minimum eccentricity of the axial load to be designed. Columns are generally named as to compression member, because the compression forces dominate their behavior.

4.2 Short and slender column:

• Before say that column is short or slender you need to know if the column braced or un-braced.

• Structural frames whose joints are restrained against lateral displacement by attachment to rigid elements or by bracing are called braced or non sway frame.

• The rigid elements are (shear walls, elevator shaft, or reinforced masonry walls). If there is no any rigid elements the columns are considered un-braced.



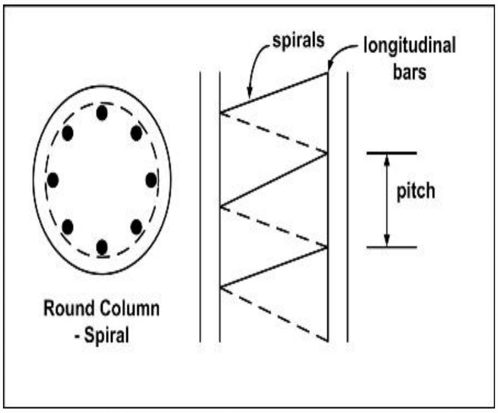


Figure 4.1: Rectangular Column

Figure 4.2: Slender Column

4.3 Design of Columns :

- Check if column slender or short :

$$\left(\frac{K * L}{r} \leq 40\right)$$

- L = 3300mm
- $I = \frac{b*h^3}{12} = \frac{300*600^3}{12} = 5.4*10^9 mm^4$
- A = 300 * 600 = 180000 mm^2
- $r = \sqrt{\frac{I}{A}} = \sqrt{\frac{5.4*10^9}{180000}} = 173.2$
- k for Fixed – partially Fixed = 1

$$\frac{1*3300}{173.2} = 19.1 \leq 40$$

The column is short.

1) Design of section Column (C.1):

- Pu = 1.2*295.95 + 1.6*49.68 = 438.24 kN
- Pu = 438.24 + (1.2(0.6*0.3*3*24)) = 453.79 kN
- Pu = 453.79*5 = 2269 kN
- Pu = φPn

  Pu = 0.8φ (0.85 Fc' Ac + As Fy)

  2269*1000 = 0.8*0.65 (0.85*28Ac + 0.15Ac*420)

  Ac = 173338.72 mm^2
- $h = \dfrac{Ac}{300} = \dfrac{173338.72}{300} = 577$

Use : h = 600 mm

b = 300 mm

- ey = 0.015 + 0.03*0.3 = 0.024 m
- ex = 0.015 + 0.03*0.6 = 0.033 m

- My = Pu*ey = 2269*0.024 = 55 KN.m
- Mx = Pu*ex = 2269*0.033 = 75 KN.m
- $Ry = \dfrac{My}{\varphi\, Fc'b\, d^2} = \dfrac{55*10^6}{0.65*28*600*300^2} = 0.055$
- $Rx = \dfrac{Mx}{\varphi\, Fc'b\, d^2} = \dfrac{75*10^6}{0.65*28*300*600^2} = 0.038$
- $Kn = \dfrac{Pu}{\varphi\, Fc'b\, d} = \dfrac{2269*10^3}{0.65*28*300*600} = 0.69$
- $\gamma = \dfrac{600-120}{600} = 0.8$
- $\rho_{req} < \rho_{min} = 0.001$ **Use:** $\rho_{min} = 0.001$
- As = ρbh = 0.001*300*600 = 1800 mm^2

- As from table **10 ϕ 16** = 2011mm^2

Use: 10 ϕ 16

Minimum clear distance between bars :

1) 1.5 db = 1.5*16 = 24 **m**m

2) 40 **m**m

Maximum clear distance between bars :

- 600 – (2*30) – 20 –(16*4) = 456 mm
- 456/3 = 152 mm

  Max = 150 mm < 152mm (use douple set of ties)

- 600 – (2*30) – 40 – (16*4) = 436 mm
- 463/3 = 145 mm

  145 mm < Max = 150 mm (OK)

The spacing of ties :

- S = 16*db = 16*16 = 256 mm
- S = 48*dt = 48*10 = 480 mm
- S = width = 300 mm

Use: 2 ϕ 10 @ 200mm

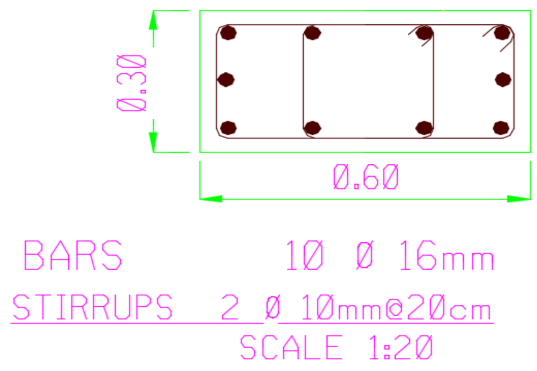


Figure 4.3 : Reinforcement detailing for column (1)

2) Design of section Column (C.2):

- Pu = 1.2*215.75 + 1.6*11.71 = 279.5 kN
- Pu = 279.5 + (1.2(0.6*0.2*3*24)) = 289.86 kN
- Pu = 289.86*5 = 1450 kN
- Pu = φPn
  Pu = 0.8φ (0.85 Fc' Ac + As Fy)
  1450*1000 = 0.8*0.65 (0.85*28Ac + 0.15Ac*420)
  Ac = 117162.25 mm^2
- $h = \frac{Ac}{200} = \frac{117162.25}{200} = 585\ \mathbf{mm}$

Use : h = 600 mm
b = 200 mm

- ey = 0.015 + 0.03*0.2 = 0.021 m
- ex = 0.015 + 0.03*0.6 = 0.033 m
- My = Pu*ey = 1450*0.021 = 31 KN.m
- Mx = Pu*ex = 1450*0.033 = 48 KN.m
- $Ry = \frac{My}{\varphi\ Fc'b\ d^2} = \frac{31*10^6}{0.65*28*600*200^2} = 0.069$
- $Rx = \frac{Mx}{\varphi\ Fc'b\ d^2} = \frac{48*10^6}{0.65*28*200*600^2} = 0.036$
- $Kn = \frac{Pu}{\varphi\ Fc'b\ d} = \frac{1450*10^3}{0.65*28*200*600} = 0.44$
- $\gamma = \frac{600-120}{600} = 0.8$
- $\rho_{req} < \rho_{min} = 0.001$ **Use:** $\rho_{min} = 0.001$
- As = ρbh = 0.001*200*600 = 1200 mm^2
- As from table **8 ϕ 14** = 1232mm^2

Use: 8 ϕ 14

Minimum clear distance between bars :

1) 1.5 db = 1.5*14 = 21 mm

2) 40 mm

Maximum clear distance between bars :

- 600 – (2*30) – 20 –(14*4) = 464mm
- 464/3 = 154.66 mm

  Max = 150 < 154.66 (use douple set of ties)

- 600 – (2*30) – 40 –(16*4) = 444mm
- 444/3 = 148 mm **148 < Max = 150 (OK)**

The spacing of ties :

- S = 16*db = 16*14 = 224 mm
- S = 48*dt = 48*10 = 480 mm
- S = 200 mm

Use: 2 ϕ 10 @ 200mm

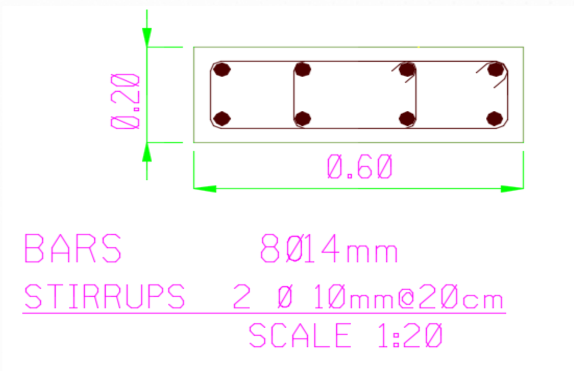


Figure 4.4 : Reinforcement detailing for column (2)

3) Design of section Column (C.3):

- Pu = 1.2*242.63 + 1.6*40.68 = 356.25 kN
- Pu = 356.25 + (1.2(0.6*0.3*3*24)) = 371.8 kN
- Pu = 371.8*5 = 1859 kN
- Pu = φPn
  Pu = 0.8φ (0.85 Fc' Ac + As Fy)
  1859*1000 = 0.8*0.65 (0.85*28Ac + 0.15Ac*420)
  Ac = 41186.63 mm^2
- $h = \frac{Ac}{300} = \frac{41186.63}{300} = 137.3\ \mathbf{mm}$

Use : h = 600 mm
b = 300 mm

- ey = 0.015 + 0.03*0.3 = 0.024 m
- ex = 0.015 + 0.03*0.6 = 0.033 m
- My = Pu*ey = 1859*0.024 = 44.61 KN.m
- Mx = Pu*ex = 1859*0.033 = 61.34 KN.m
- $Ry = \frac{My}{\varphi\ Fc' b\ d^2} = \frac{44.61*10^6}{0.65*28*600*300^2} = 0.045$
- $Rx = \frac{Mx}{\varphi\ Fc' b\ d^2} = \frac{61.34*10^6}{0.65*28*300*600^2} = 0.031$
- $Kn = \frac{Pu}{\varphi\ Fc' b\ d} = \frac{1859*10^3}{0.65*28*300*600} = 0.568$
- $\gamma = \frac{600-120}{600} = 0.8$
- $\rho_{req} < \rho_{min} = 0.001$ **Use:** $\rho_{min} = 0.001$
- As = ρbh = 0.001*300*600 = 1800 mm^2

- As from table **10 ϕ 16** = 2011 mm^2

<u>Use</u>: 10 ϕ 16

<u>Minimum clear distance between bars :</u>

1) 1.5 db = 1.5*14 = 21 mm

2) 40 mm

<u>Maximum clear distance between bars :</u>

- 600 – (2*30) – 20 –(16*4) = 456
- 456/3 = 152 mm

  Max = 150 < 152 (use douple set of ties)

- 600 – (2*30) – 40 –(16*4) = 436
- 463/3 = 145 mm **145 < Max = 150 (OK)**

<u>The spacing of ties :</u>

- S = 16*db = 16*16 = 256 mm
- S = 48*dt = 48*10 = 480 mm
- S = 300 mm

<u>Use</u>: 2 ϕ 10 @ 200mm

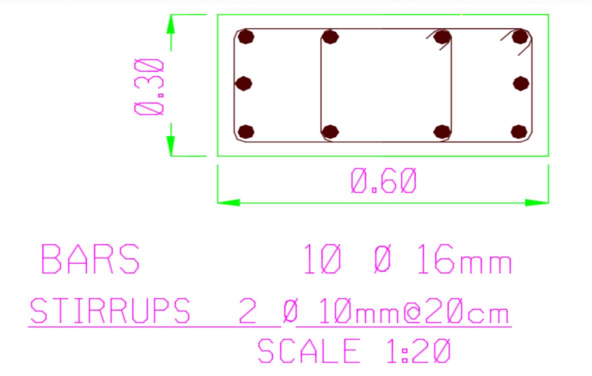


Figure 4.5 : Reinforcement detailing for column (3)

Chapter (5)

Design of Stairs

Ch5 : Stairs

5.1 Introduction :

- Staircase is an important component of abuilding providing access to different floors and roof of the building .
- It consists of a flight of steps (stairs) and one or more intermediate landing slabs between the floor levels .
- Stairs can be defined as series of steps suitably arranged for the purpose of connecting different floors of a building .
- It may also be defined as an arrangement of treads , stringers , newel post , hand rails , and baluster , so designed and constructed as to provide an easy and quick access to the different floors .

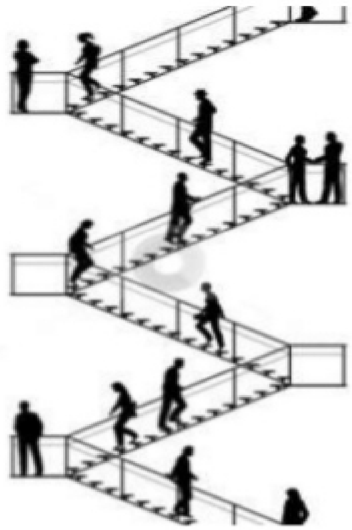


Figure 5.1 : Stairs

5.2 Types of stairs:

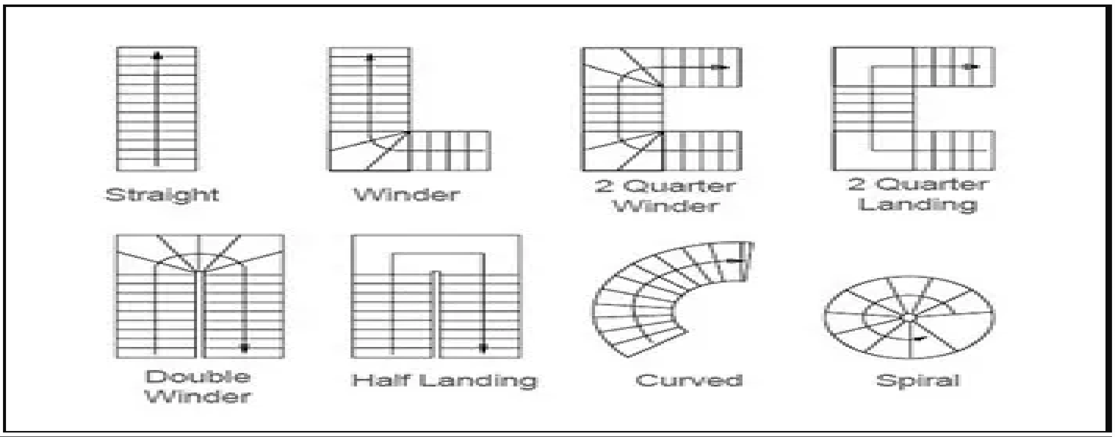


Figure 5.2 : Types of stairs

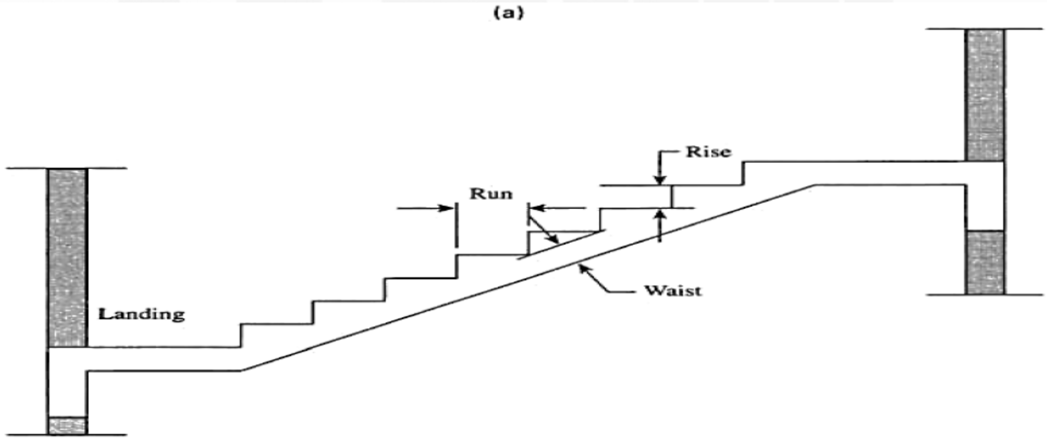


Figure 5.3 : Parts of stairs

5.3 Design of stairs :

- Length of riser = $\dfrac{\text{height}}{\text{number of riser}} = \dfrac{1690}{10}$ = 169 mm ~~ Take 170 mm

- Length of Run = $\dfrac{\text{horizontal length of the stairs}}{\text{number of run}} = \dfrac{2700}{9}$ = 300 mm

- Design of stairs :

Simply Supported :Thickness of slab from American Concrete Institute (ACI-318)

- h = $\frac{l}{20}$ = $\frac{3450}{20}$ = 172.5 mm ~~ Take 200 mm

- h avg = $\frac{200+(200+170)}{2}$ = 285 mm ~~ Take 290 mm

- Dead Load :

Type	Thickness (m)	Density (kN/m^3)	DL (kN/m^2)
Slap	0.29	24	6.96
Tile	0.025	24	0.6
Morter	0.025	22	0.55
Sand	0.1	18	1.8
Bleaching	0.025	22	0.55

Table 5.1 : Calculation Dead Load for stairs

Total Dead Load = 10.46 kN/m^2

- Live Load :

Live Load = $\frac{3}{\cos 30}$ = 3.5 kN $/m^2$

- Design :

- b = 1000 mm
- d = h – c.c - $\frac{ø}{2}$ = 200 mm – 20 mm - $\frac{16}{2}$ = 172 mm
- from prokon : **Vumax** = 31.31 kn , **Mumax** = 27.01 kN.m
- $Mu = \emptyset\ As*Fy(d - \frac{a}{2})$

  $27.01 * 10^6 = 0.9*As*420*(172 - \frac{0.0176As}{2})$

  $As = 424.6\ mm^2$

- $a = \frac{Fy*AS}{0.85*b*f'c} = \frac{420*As}{0.85*1000*28} = 0.0176 * As$
- $As\ min = 0.0033*1000*172 = 567.6\ mm^2$
- As max = 0.0206*1000*172 = 3543.2 mm^2
- As min > As Take As min = 567.6
- **<u>Use</u> : 1 ø 16 / 200 mm**

- As 4 ø 16 = 804.24 mm^2

Check for shear :

$Vc=\varphi*0.17*bw*d*\sqrt{f'c} = 0.75\times0.17\times1000\times172\times\sqrt{28}\times10^{-3}$ =116.04 kNφ

- $V_{umax} < ½\varphi Vc <$ 31.31 < 58.02 < 116.04

φVc

5.4 Design of slab:

- $h = \frac{l}{20} = \frac{2700}{20}$ = 135 mm ~~ Take 150 mm
- b = 1000 mm

- Dead Load :

Type	Thickness (m)	Density (kN/m^3)	DL (kN/m^2)
Slab	0.15	24	3.6
Tile	0.025	24	0.6
Morter	0.025	22	0.55
Sand	0.1	18	1.8
Bleaching	0.025	22	0.55

Table 5.2 : Calculation Dead Load for slab stairs

Total Dead Load = 7.1 kN/m^2

- Live Load :

Live Load = 3 kN/m^2

- Design :

- b = 1000 mm
- $d = h - c.c - \frac{ø}{2}$ = 150 mm – 20 mm - $\frac{14}{2}$ = 123 mm
- from prokon : **V_{umax}** = 34.93 kN , **M_{umax}** = 23.58 kN.m
- $Mu = \emptyset\ As*Fy(d - \frac{a}{2})$

  $23.58 * 10^6 = 0.9*As*420*(172 - \frac{0.0176As}{2})$

  As = 527.03 mm^2
- $a = \frac{Fy*AS}{0.85*b*f'c} = \frac{420*As}{0.85*1000*28} = 0.0176 * As$
-
- As min = 0.0033*1000*123 = 405.9 mm^2
- As max = 0.0206*1000*123 = 2533.8 mm^2

- As max > As > As min Take As = 527.03 mm^2
- **Use:** 1 **ø 16 / 200 mm** As 4 ø 16 = 804.24 mm^2

Check for shear :

$\phi Vc=\phi*0.17*bw*d*\sqrt{f'c} = 0.75\times0.17\times1000\times123\times\sqrt{28}\times10^{-3}$ =82.98 kN

- Vumax < ½φVc < 34.93 < 41.49 < 82.98

φVc

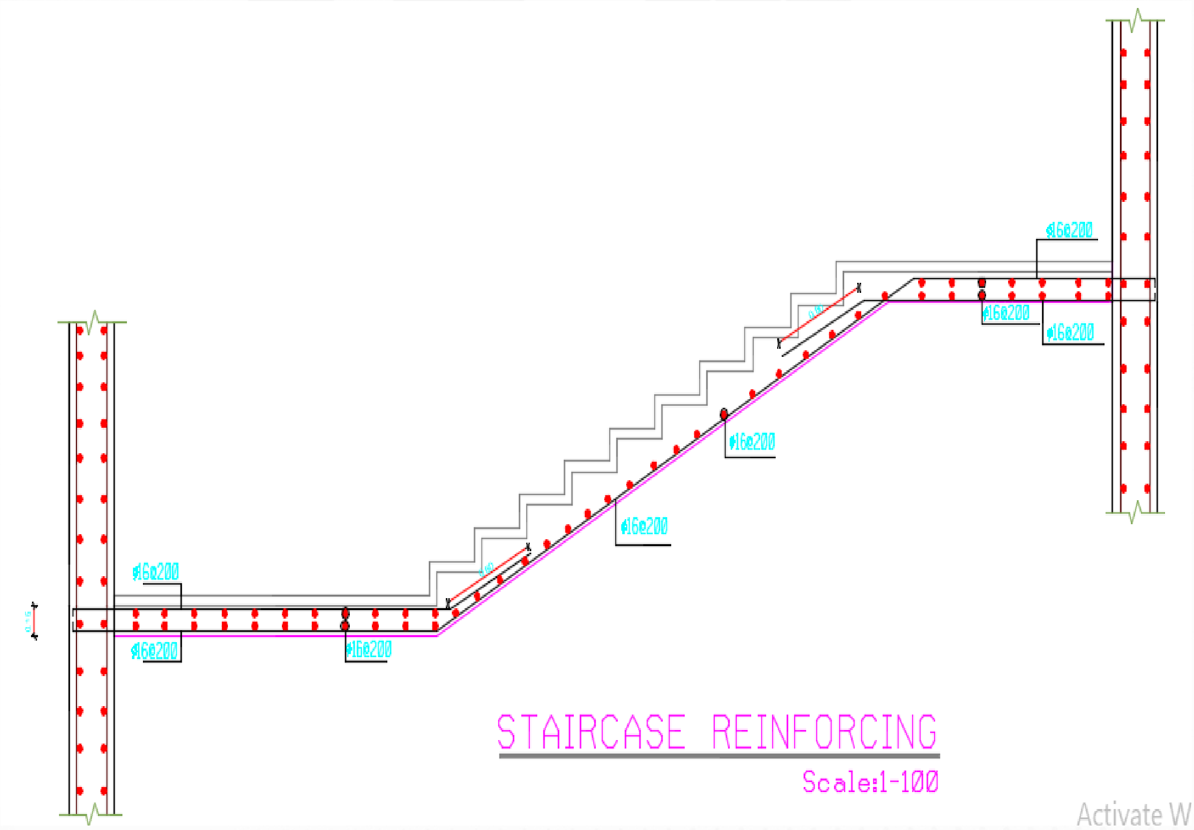


Figure 5.4 : Reinforcement detailing for stairs and slab

Chapter (6)

Design of Walls

Ch6 : Wall

6.1 Introduction:

A wall is a structure that defines an area. It is used for architecture or civil purposes. There are the following types of walls:

1- Bearing walls:-walls resist compression force only.

2- Shear walls:-walls resist compression force and lateral force.

3- Retaining walls: - walls are used to hold back masses of earth or other loose material.

4- Basement walls: - walls are used to hold back masses of earth or other loose material and compression force from the building.

6.2 Design the shear Wall :

Wall (1) :

- Find the dimension (B) :

- Service Load Pu = 225 kN/m Bearing Capacity = 280 kN

- qu = bearing capacity – weight of soil – weight of concrete
  = 280 – 0.5*18 – 0.5*24 = 259 kN

- qu = $\frac{Pu}{Af}$ 259 = $\frac{225}{B*L}$ B = 1100 mm

- d = h – dc = 0.5 – 0.075 = 0.425 m

- One way shear analysis :

- $Vc = \varphi*0.17*\sqrt{f'c} = 0.75*0.17*\sqrt{28} = 674.6$ kN/m²
- $Vc*d = qu\,(\frac{L}{2} - \frac{C}{2} - d)$

$674.6*d = 259\,(\frac{1}{2} - \frac{1}{2} - d)$ d = 0.38 m **0.425 > 0.38 OK**

- Cheak for capacity :

- Capacity = 259 + 18*0.5 + 24*0.5 = 280 kN/m² **280 = 280 OK**
- qu = 259*1.2 = 310.8 kN/ m²

- Check for one way shear :

- $\varphi Vc = Vu$
- $Vu = qu*B\,(\frac{L}{2} - \frac{C}{2} - d) = 310.8*1\,(\frac{1}{2} - \frac{1}{2} - 0.425) = 132$ kN
- $dreq = \frac{Vu}{0.17*0.75\sqrt{f'c}\,B} = \frac{132*10^3}{0.17*0.75\sqrt{28}*1000} = 195.6mm$ **425 > 195.6 OK**

- For **main** reinforcement :

- $M = (qu*B\,(\frac{L}{2} - \frac{C}{2})^2)/2 = (310.8*1\,(\frac{1.1}{2} - \frac{0.3}{2})^2)/2 = 24.86$ kN.m
- $\rho = \frac{0.85Fc'}{Fy}\left(1 - \sqrt{1 - \frac{2.354Mu}{0.9Fc'bd^2}}\right)$

$= \frac{0.85*28}{420}\left(1 - \sqrt{1 - \frac{2.354*24.86*10^6}{0.9*28*1000*425^2}}\right) = 0.0004$

ρmin > ρ **0.002 > 0.0004** **use ρmin =0.002**

- As = ρbd = 0.002*1000*425 = 850 mm²
- As = 850*1.1 = 935 mm²
- Spacing (S)= $\frac{B}{n} < 3h < 450 = \frac{1000}{5} < 3*500 < 450$

Use 5 ø 16 / 200 mm

As 5 ø 16 = 1005 mm² > 935 OK

- For **secondary** reinforcement :

- use ρmin =0.0012
- As = ρbd = 0.0012*1000*425 = 510 mm²

Use 5 ø 14 / 200 mm

As 5 ø 14 = 769.7 mm² > 765 OK

Table 11.6.1—Minimum reinforcement for walls with in-plane $V_u \le 0.5\phi V_c$

Wall type	Type of nonprestressed reinforcement	Bar/wire size	f_y, MPa	Minimum longitudinal[1], ρ_ℓ	Minimum transverse, ρ_t
Cast-in-place	Deformed bars	≤ No. 16	≥ 420	0.0012	0.0020
			< 420	0.0015	0.0025
		> No. 16	Any	0.0015	0.0025
	Welded-wire reinforcement	≤ MW200 or MD200	Any	0.0012	0.0020
Precast[2]	Deformed bars or welded-wire reinforcement	Any	Any	0.0010	0.0010

[1]Prestressed walls with an average effective compressive stress of at least 1.6 MPa need not meet the requirement for minimum longitudinal reinforcement ρ_ℓ.

[2]In one-way precast, prestressed walls not wider than 3.7 m and not mechanically connected to cause restraint in the transverse direction, the minimum reinforcement requirement in the direction normal to the flexural reinforcement need not be satisfied.

Table 6.1 : Minimum reinforcement for Walls

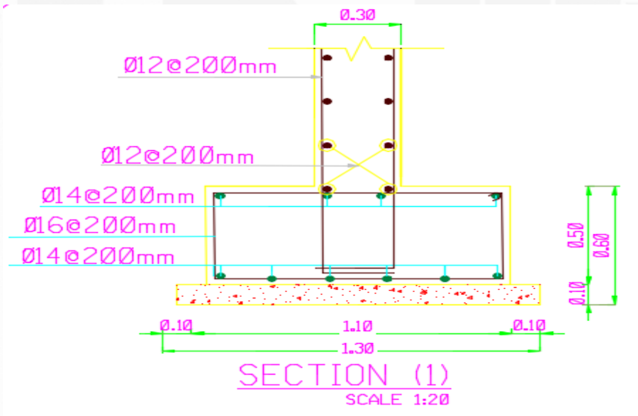


Figure 6.1 : Reinforcement detailing for shear wall (1)

Wall (2) :

- Find the dimension (B) :

- Service Load Pu = 225 kN/m
- qu = bearing capacity – weight of soil – weight of concrete
  = 280 – 9 – 12 = 259 kN
- qu = $\frac{\text{Pu}}{\text{Af}}$ 259 = $\frac{225}{B*L}$ B = 1100mm
- d = h – dc = 0.5 – 0.075 = 0.425 m

- One way shear analysis :

- $Vc = \varphi*0.17*\sqrt{f'c} = 0.75*0.17*\sqrt{28} = 674.6$ kN/m²
- $Vc*d = qu\ (\frac{L}{2} - \frac{C}{2} - d\ )$

$674.6*d = 259\ (\frac{1}{2} - \frac{1}{2} - d\ )$ d = 0.38 m **0.425 > 0.38 OK**

- Cheak for capacity :

- Capacity = 259 + 18*0.5 + 24*0.5 = 280 kN/m² **280 = 280 OK**
- qu = 259*1.2 = 310.8 kN/m²

- Check for one way shear :

- $\varphi Vc = Vu$
- $Vu = qu*B\ (\frac{L}{2} - \frac{C}{2} - d\ ) = 310.8*1\ (\frac{1}{2} - \frac{1}{2} - 0.425\ ) = 132$ kN
- $d_{req} = \frac{Vu}{0.17*0.75\sqrt{f'c}\ B} = \frac{132*10^3}{0.17*0.75\sqrt{28}*1000} = 195.6mm$ **425 > 195.6 OK**

- For **main** reinforcement :

- $M = (qu*B\ (\frac{L}{2} - \frac{C}{2})^2) / 2 = (310.8*1\ (\frac{1}{2} - \frac{0.2}{2})^2) / 2 = 24.86$ kN.m
- $\rho = \frac{0.85Fc'}{Fy}\left(1 - \sqrt{1 - \frac{2.354Mu}{0.9Fc'bd^2}}\right)$

$= \frac{0.85*28}{420}\left(1 - \sqrt{1 - \frac{2.354*24.86*10^6}{0.9*28*1000*425^2}}\right) = 0.0004$

ρmin > ρ **0.002 > 0.0004** **Take ρmin =0.002**

- As = ρbd = 0.002*1000*425 = 850 mm²
- As = 850*1.1 = 935 mm²
- Spacing (S)= $\frac{B}{n} < 3h < 450 = \frac{1000}{5} < 3*500 < 450$

Use 1 ø 16 / 200 mm **As 1 ø 16 = 1005 mm² > 841.5 OK**

- For **secondary** reinforcement :

- use ρmin =0.0012
- As = ρbd = 0.0012*1000*425 = 510 mm²

Use 1ø 14 / 200 mm
As 5ø 14 = 769.7 mm² > 765 OK

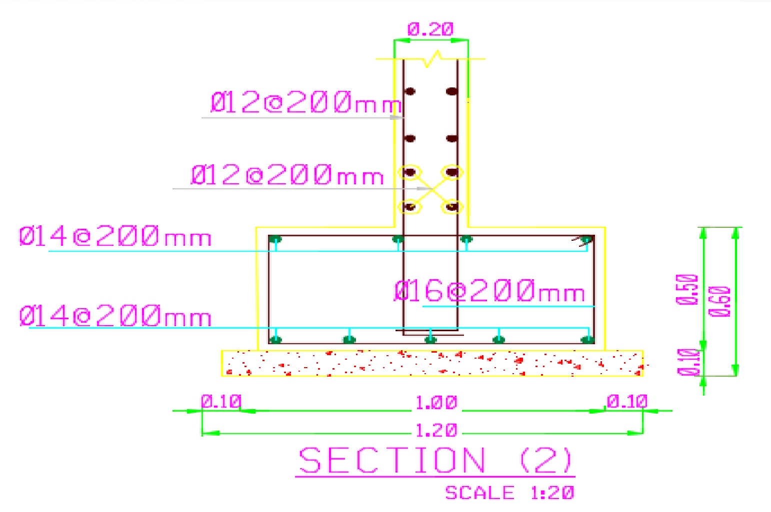


Figure 6.2 : Reinforcement detailing for shear wall (2)

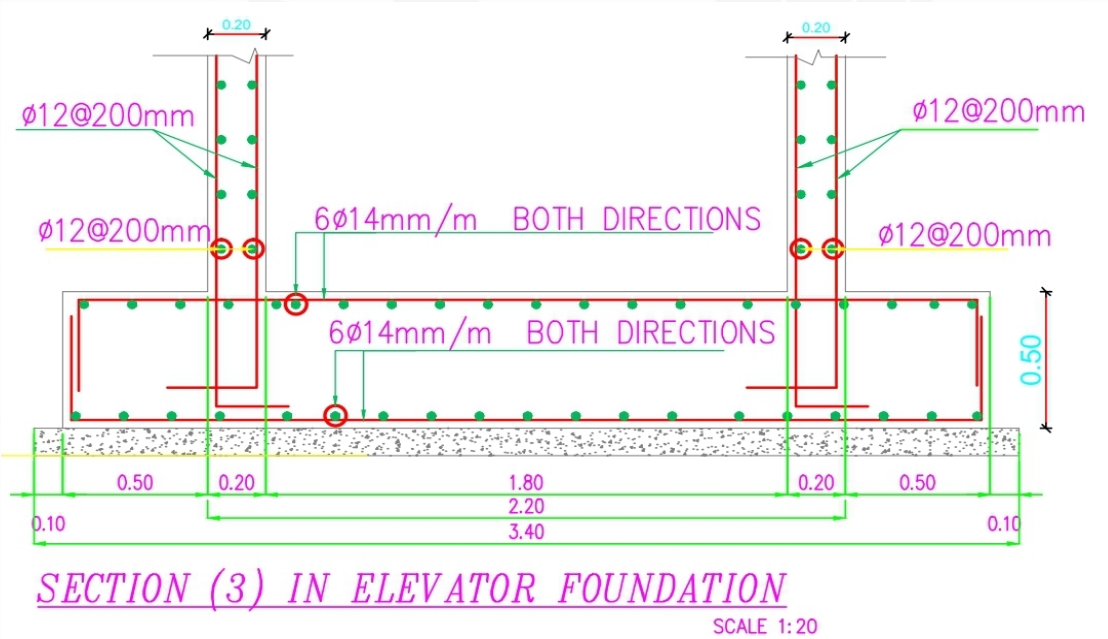


Figure 6.3 : Reinforcement detailing for Elevator Foundation

Chapter (7)

Design of Foundation

Ch7 : Foundation

7.1 Introduction:

• The main purpose of any structural portion is to transmit applied loads Safely from part of the structure to another, and we know that the load from buildings and other structures must be transmitted into the ground through foundations, then into the underlying supporting material .

• Foundations usually support the main load-bearing members, i.e., columns and walls , of the structure .

• Since the bearing pressures that a soil can sustain are much smaller than the compressive stresses in columns or walls,
foundations must be used to reduce the pressures applied directly
to the soil by spreading the supported loads over an area large enough to prevent rupture or excessive deformation of the soil .

• The foundation must be stable and safe in the first place, and its Safety is ensured

- By keeping a factor of safety.
- By avoiding structural failure of foundation itself.
- By avoiding the excessive settlements .

-Soil pressure under footings:

• The distribution of soil under a footing is a function of the following :
- The type of the soil.
- The relative rigidity of the soil.
- The foundation pad.

-Bearing capacity of soil = 280 kN

7.2 Types of foundations:

• In general, foundation of buildings and bridges may be divided into two major categories :

1) Shallow foundations : The types of it's Foundation are :

- Spread footing
- Wall footing
- Eccentric footing
- Combined footing
- Strap footing

2) Deep foundations : The types of it's Foundation are :

- Caissons
- Basement Walls
- Piles

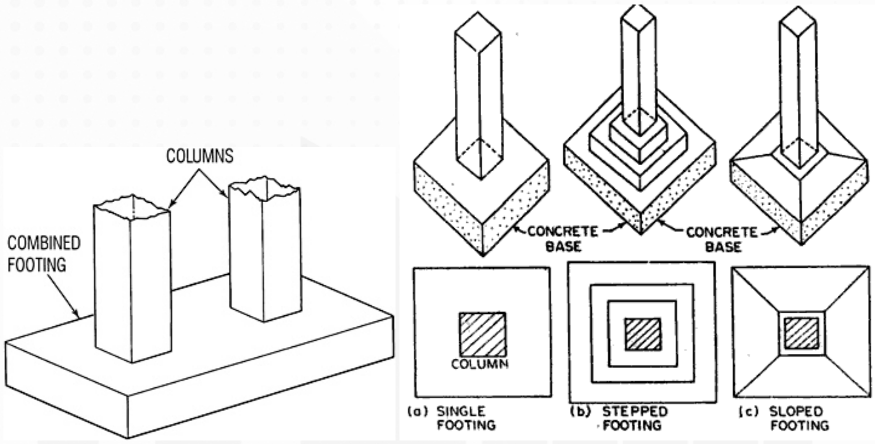


Figure 7.1 : Single and combined footing

• We can know if the foundation is shallow or deep foundation according to :

- If $D_f \leq B$ the foundation is the classified as **shallow foundations**.

- If $D_f > B$ the foundation is the classified as **Deep foundations**.

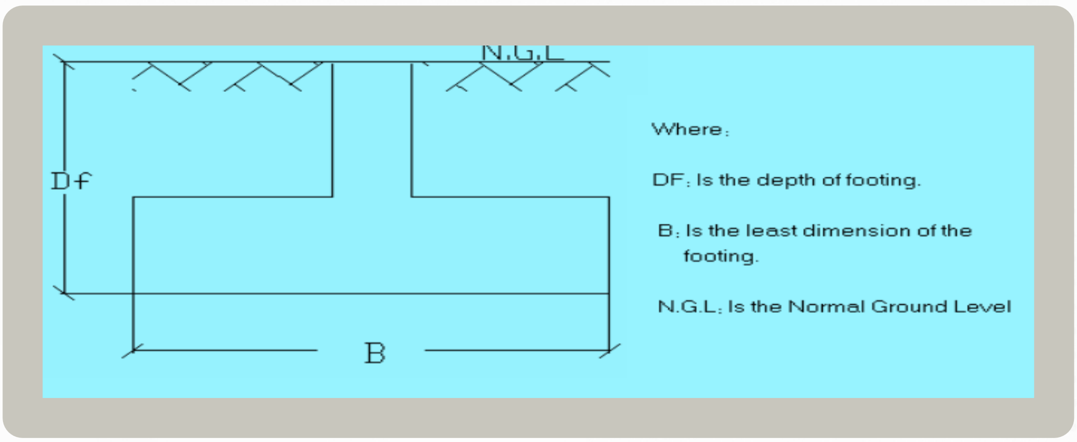


Figure 7.2 : Foundation

7.3 Design the Base :

Base (1) :

- Find the dimension (B & L) :

- Service Load Pu = 295.95 + 49.68 = 345.63 kN
- Pu = 345.63 + (0.6*0.3*3*24) = 358.59 kN
- Pu = 358.59*5 = 1792.95 kN
- qu = 280 – 9 – 12 = 259 kN
- qu = $\frac{Pu}{Af}$ 259 = $\frac{1792.95}{B+2B}$
- qu = $\frac{Pu}{Af}$ 259 = $\frac{1792.95}{B*L}$
- d = h – dc = 0.5 – 0.075 = 0.425 m

B = 2300 mm

L = 3000 mm

- One way shear analysis :

- $Vc = \phi*0.17*\sqrt{f'c} = 0.75*0.17*\sqrt{28} = 674.6$ kN/m²
- $Vc*d = qu\ (\frac{L}{2} - \frac{C}{2} - d\ )$

674.6*d = 259 ($\frac{3}{2} - \frac{0.6}{2} - d$) d = 0.33 m

- Two way shear analysis :

- $Vc = \phi*0.17*\sqrt{f'c} = 0.75*0.33*\sqrt{28} = 1309.64$ kN/m²
- $4d^2 + 2d(b + c) = \frac{BLq}{Vc}$

$4d^2 + 2d(0.6 + 0.3) = \frac{2.3*3*259}{1309.64}$ d = 0.40 m

Take d = 0.40 **0.425 > 0.4 OK**

- Cheak for capacity :

- Capacity = 259 + 18*0.5 + 24*0.5 = 280 kN/m² **280 = 280 OK**
- qu = 259*1.2 = 310.8 kN/m²

- Check for one way shear :

- $\phi Vc = Vu$
- $Vu = qu*B\,(\frac{L}{2} - \frac{C}{2} - d) = 310.8*1\,(\frac{3}{2} - \frac{0.6}{2} - 0.425) = 240.85$ kN
- $d_{req} = \frac{Vu}{0.17*0.75\sqrt{f'c}\,B} = \frac{240.85*10^3}{0.17*0.75\sqrt{28}*1000} = 356.9mm$ **425 > 356.9 OK**

- Check for Two way shear :

- $\phi Vc = Vu$
- $Vu = qu\,((A) - ((c + d)(c + d))) =$
  $310.8\,((3*2.3) - ((0.6 + 0.425)(0.3 + 0.425))) = 1913.55$ kN
- $d_{req} = \frac{Vu}{0.33*0.75\sqrt{f'c}\,B} = \frac{1913.55*10^3}{0.33*0.75\sqrt{28}*3500} = 417.4mm$ **425 > 417.4 OK**

- Reinforcement for **secondary** direction :

- $M = (qu*L\,(\frac{B}{2} - \frac{C}{2})^2) / 2 = (310.8*1\,(\frac{2.3}{2} - \frac{0.3}{2})^2) / 2 = 155.4$ kN.m
- $\rho = \frac{0.85Fc'}{Fy}\left(1 - \sqrt{1 - \frac{2.354Mu}{0.9Fc'bd^2}}\right)$

  $= \frac{0.85*28}{420}\left(1 - \sqrt{1 - \frac{2.354*155.4*10^6}{0.9*28*1000*425^2}}\right) = 0.0023$

  ρMax > ρ > ρmin **0.021 > 0.0023 > 0.0018 OK**

- As = ρbd = 0.0023*1000*425 = 987.7 mm²
- As = 987.7*3 = 2963.1 mm²
- Spacing (S)= $\frac{B}{n} < 3h < 450 = \frac{1000}{5} < 3*500 < 450$

  Use 1 ø 16 / 200 mm

  As 15 ø 16 = 3016 mm² > 2963.1 OK

- Reinforcement for **main** direction :

- $M = (qu*B\,(\frac{L}{2} - \frac{C}{2})^2) / 2 = (310.8*1\,(\frac{3}{2} - \frac{0.6}{2})^2) / 2 = 223.77$ kN.m
- $\rho = \frac{0.85Fc'}{Fy}\left(1 - \sqrt{1 - \frac{2.354Mu}{0.9Fc'bd^2}}\right)$

  $= \frac{0.85*28}{420}\left(1 - \sqrt{1 - \frac{2.354*223.77*10^6}{0.9*28*1000*425^2}}\right) = 0.0033$

  ρMax > ρ > ρmin **0.021 > 0.0033 > 0.0018 OK**

- As = ρbd = 0.0033*1000*425 = 1436.1 mm^2
- As = 1436.1*2.3 = 3302.97 mm^2
- Spacing (S)= $\frac{B}{n} < 3h < 450 = \frac{1000}{6} < 3*500 < 450$

Use 1 ø 18 / 160 mm

As 14 ø 18 = 3303 mm^2 > 3302.97 OK

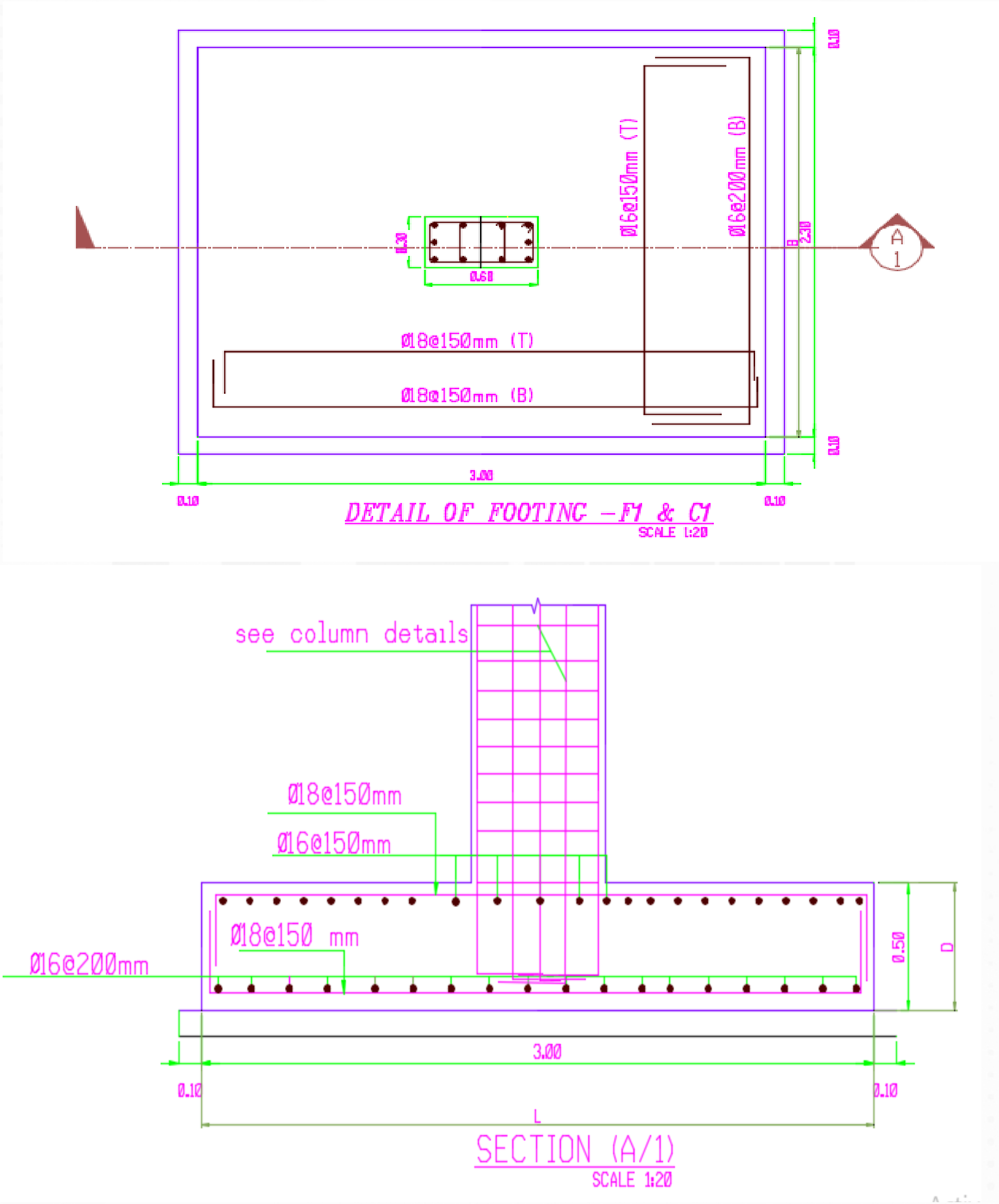


Figure 7.3 : Reinforcement detailing for Base (1)

Base (2) :

- Find the dimension (B & L) :

- Service Load Pu = 242.63 + 40.68 = 283.31 kN
- Pu = 283.31 + (0.6*0.3*3*24) = 296.27 kN
- Pu = 296.27*5 = 1481.35 kN
- qu = 280 – 9 – 12 = 259 kN
- qu = $\frac{Pu}{Af}$ 259 = $\frac{1481.35}{B+2B}$
- qu = $\frac{Pu}{Af}$ 259 = $\frac{1481.35}{B*L}$
- d = h – dc = 0.5 – 0.075 = 0.425 m

B = 2000 mm

L = 3000 mm

- One way shear analysis :

- $Vc = \phi*0.17*\sqrt{f'c} = 0.75*0.17*\sqrt{28} = 674.6$ kN/m²
- $Vc*d = qu\ (\frac{L}{2} - \frac{C}{2} - d\ )$

$674.6*d = 259\ (\frac{3}{2} - \frac{0.6}{2} - d\ )$ d = 0.33 m

- Two way shear analysis :

- $Vc = \phi*0.17*\sqrt{f'c} = 0.75*0.33*\sqrt{28} = 1309.64$ kN/m²
- $4d^2 + 2d(b + c) = \frac{BLq}{Vc}$

$4d^2 + 2d(0.6 + 0.3) = \frac{2*3*259}{1309.64}$ d = 0.36 m

Take d = 0.36 **0.425 > 0.36 OK**

- Cheak for capacity :

- Capacity = 259 + 18*0.5 + 24*0.5 = 280 kN/m² **280 = 280 OK**
- qu = 259*1.2 = 310.8 kN/m²

- Check for one way shear :

- $\phi Vc = Vu$
- $Vu = qu*B\ (\frac{L}{2} - \frac{C}{2} - d\ ) = 310.8*1\ (\frac{3}{2} - \frac{0.6}{2} - 0.425\ ) = 240.85$ kN
- $d_{req} = \frac{Vu}{0.17*0.75\sqrt{f'c}\,B} = \frac{240.85*10^3}{0.17*0.75\sqrt{28}*1000} = 356.9mm$ **425 > 356.9 OK**

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- Check for Two way shear :

- $\phi Vc = Vu$
- $Vu = qu\ ((A) - ((c + d)(c + d))) =$
  $310.8\ ((3*2) - ((0.6 + 0.425)(0.3 + 0.425))) = 1633.8$ kN
- $d_{req} = \frac{Vu}{0.33*0.75\sqrt{f'c}\ B} = \frac{1633.8*10^3}{0.33*0.75\sqrt{28}*3500} = 356.4mm$ **425 > 356.4 OK**

- reinforcement for **secondary** direction :

- $M = (qu*L\ (\frac{B}{2} - \frac{C}{2})^2) / 2 = (310.8*1\ (\frac{2}{2} - \frac{0.3}{2})^2) / 2 = 112.27$ kN.m
- $\rho = \frac{0.85Fc'}{Fy}\ (1 - \sqrt{1 - \frac{2.354Mu}{0.9Fc'bd^2}})$

  $= \frac{0.85*28}{420}\ (1 - \sqrt{1 - \frac{2.354*112.27*10^6}{0.9*28*1000*425^2}}) = 0.0016$

  ρmin > ρ **0.0018 > 0.0016** **Take ρmin =0.0018**

- As = ρbd = 0.0018*1000*425 = 765 mm^2
- As = 765*3 = 2295 mm^2
- Spacing (S)= $\frac{B}{n} < 3h < 450 = \frac{1000}{5} < 3*500 < 450$

  Use 1 ø 16 / 200 mm

  As 15 ø 16 = 3016 mm² > 2295 OK

- reinforcement for **main** direction :

- $M = (qu*B\ (\frac{L}{2} - \frac{C}{2})^2) / 2 = (310.8*1\ (\frac{3}{2} - \frac{0.6}{2})^2) / 2 = 223.77$ kN.m
- $\rho = \frac{0.85Fc'}{Fy}\ (1 - \sqrt{1 - \frac{2.354Mu}{0.9Fc'bd^2}})$

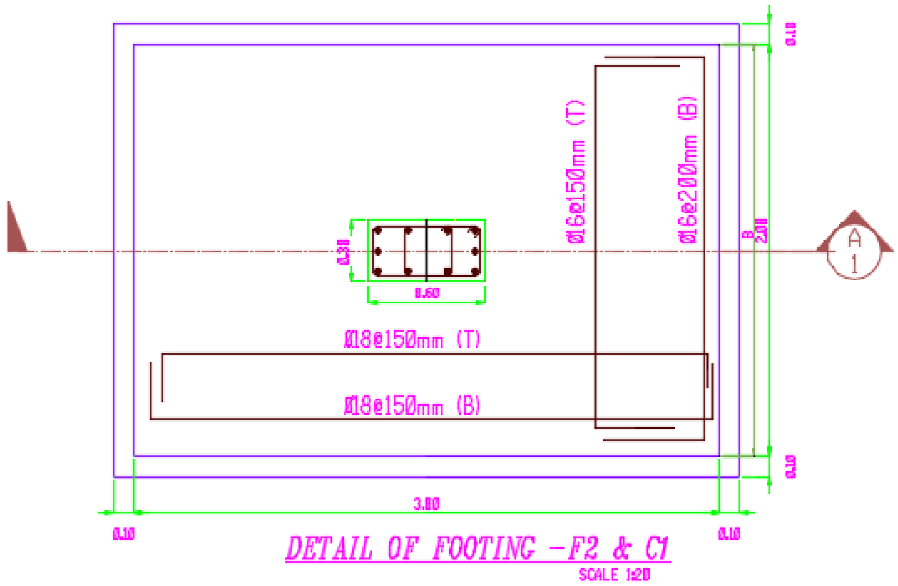
  $= \frac{0.85*28}{420}\ (1 - \sqrt{1 - \frac{2.354*223.77*10^6}{0.9*28*1000*425^2}}) = 0.0033$

  ρMax > ρ > ρmin **0.021 > 0.0033 > 0.0018 OK**

- As = ρbd = 0.0033*1000*425 = 1436.1 mm^2
- As = 1436.1*2 = 2872.15 mm^2
- Spacing (S)= $\frac{B}{n} < 3h < 450 = \frac{1000}{6} < 3*500 < 450$

  Use 1 ø 18 / 160 mm

  As 12 ø 18 = 3054 mm² > 2872.15 OK



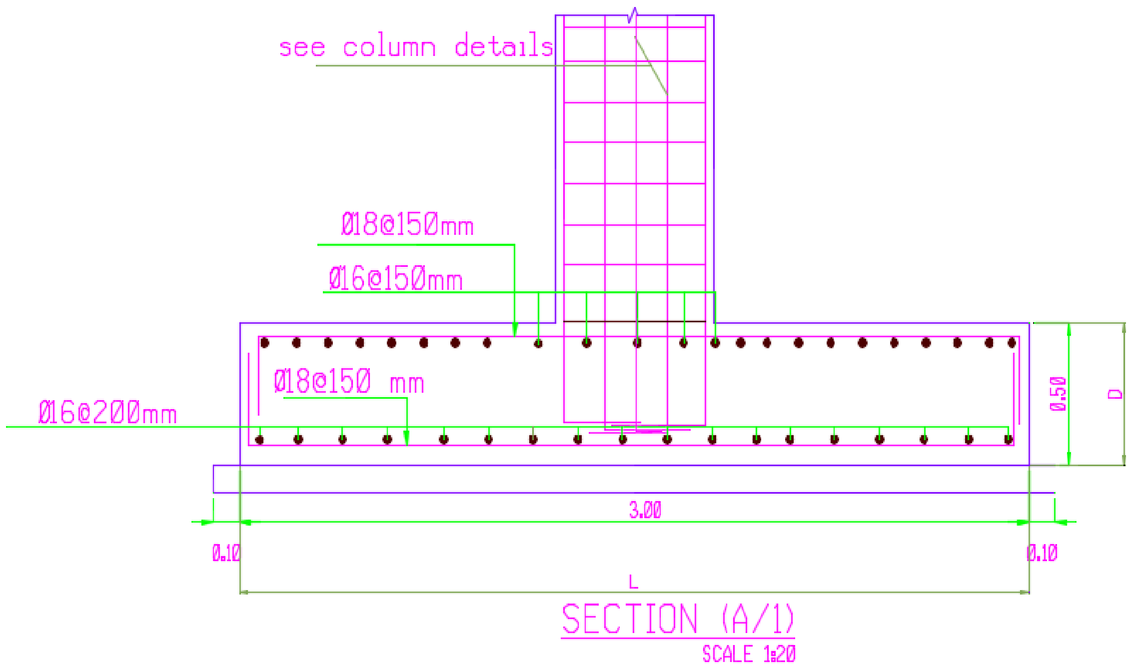


Figure 7.4 : Reinforcement detailing for Base (2)

Chapter (8)

(Reinforcement Tables)

Reinforcement detailing for Rib (1)

Sec#	Mu (kN.m)	ρ	As (mm^2)	Asmin (mm^2)	Flexure reinforcement	Vu (kN)		Stirrups
Sup A	-4.8	0.014	46.2	226	$2\phi10$	–		–
Span AB	24.56	0.0016	228.8	226	$2\phi14$	A	20.3	$1\phi8$ @140mm
						B	28.4	$1\phi8$ @140mm
Sup B	-24.75	0.0077	254.1	226	$2\phi14$	–		–
Span BC	–	–	–	–	–	B	18.6	$1\phi8$ @140mm
						C	15.3	$1\phi8$ @200mm
Sup C	-18.61	0.0057	188.1	226	$2\phi12$	–		–
Span CD	17.51	0.0019	271.7	226	$2\phi12$	C	24.7	$1\phi8$ @140mm
						D	29	$1\phi8$ @140mm
Sup D	-33.71	0.01	330	226	$2\phi16$	–		–
Span DE	21.40	0.0014	200.2	226	$2\phi12$	D	30	$1\phi8$ @140mm
						E	18.9	$1\phi8$ @140mm
Sup E	-2.74	0.0008	26.4	226	$2\phi10$	–		–

Reinforcement detailing for Rib (2)

<table>
<tr><th>Sec#</th><th>Mu (kN.m)</th><th>ρ</th><th>As (mm²)</th><th>Asmin (mm²)</th><th>Flexure reinforcement</th><th colspan="2">Vu (kN)</th><th>Stirrups</th></tr>
<tr><td>Sup A</td><td>-2.88</td><td>0.0008</td><td>27.72</td><td>226</td><td>2ϕ10</td><td colspan="2">–</td><td>–</td></tr>
<tr><td rowspan="2">Span AB</td><td rowspan="2">9.46</td><td rowspan="2">0.0006</td><td rowspan="2">85.8</td><td rowspan="2">226</td><td rowspan="2">2ϕ12</td><td>A</td><td>11.55</td><td>1ϕ8 @200mm</td></tr>
<tr><td>B</td><td>22.91</td><td>1ϕ8 @140mm</td></tr>
<tr><td>Sup B</td><td>-25.29</td><td>0.0079</td><td>260.7</td><td>226</td><td>2ϕ12</td><td colspan="2">–</td><td>–</td></tr>
<tr><td rowspan="2">Span BC</td><td rowspan="2">19.71</td><td rowspan="2">0.0013</td><td rowspan="2">185.9</td><td rowspan="2">226</td><td rowspan="2">2ϕ12</td><td>B</td><td>27.07</td><td>1ϕ8 @140mm</td></tr>
<tr><td>C</td><td>18.21</td><td>1ϕ8 @140mm</td></tr>
<tr><td>Sup C</td><td>-3.46</td><td>0.001</td><td>33</td><td>226</td><td>2ϕ10</td><td colspan="2">–</td><td>–</td></tr>
</table>

Reinforcement detailing for Rib (3)

Sec#	Mu (kN.m)	ρ	As (mm^2)	Asmin (mm^2)	Flexure reinforcement	Vu (kN)		Stirrups
Sup A	-3.61	0.001	33	226	2ϕ10	–		–
Span AB	19.34	0.0009	128.7	226	2ϕ12	A	15.48	1ϕ8 @200mm
						B	20.34	1ϕ8 @140mm
Sup B	-11.18	0.0033	108.9	226	2ϕ14	–		–
Span BC	–	–	–	–	–	B	7.36	1ϕ8 @200mm
						C	17.48	1ϕ8 @140mm
Sup C	-23.95	0.0074	244.2	226	2ϕ14	–		–
Span C	–	–	–	–	–	20.15		1ϕ8 @140mm

Reinforcement detailing for Beam (1)

Sec#	Mu (kN.m)	ρ	As (mm^2)	Asmin (mm^2)	Flexure reinforcement	Vu (kN)		Stirrups
Sup A	-55.35	0.002	540	891	4ϕ12	–		–
Span AB	124.55	0.0047	1274	891	8ϕ14	A	195.17	1ϕ10 @140mm
						B	254.8	1ϕ10 @140mm
Sup B	-184	0.007	1924.7	891	8ϕ18	–		–
Span BC	32.05	0.001	275	891	6ϕ12	B	183.45	1ϕ10 @140mm
						C	158.7	1ϕ10 @200mm
Sup C	-154.4	0.0059	1596.8	891	8ϕ16	–		–
Span CD	85.43	0.003	825	891	8ϕ12	C	202.7	1ϕ10 @140mm
						D	194.83	1ϕ10 @140mm
Sup D	-137	0.005	1375	891	7ϕ16	–		–
Span DE	31.82	0.001	275	891	6ϕ12	D	150.75	1ϕ10 @140mm
						E	138.11	1ϕ10 @200mm
Sup E	-127.55	0.0047	1292.5	891	6ϕ16	–		–
Span EF	73.86	0.0026	715	891	8ϕ12	E	157.65	1ϕ10 @140mm
						F	115.19	1ϕ10 @200mm
Sup F	-33.14	0.0012	330	891	4ϕ12	–		–

Reinforcement detailing for Beam (2)

Sec#	Mu (kN.m)	ρ	As (mm^2)	As_{min} (mm^2)	Flexure reinforcement	Vu (kN)		Stirrups
Sup A	-50.49	0.0023	506	712.8	6ϕ12	–		–
Span AB	95.85	0.0044	968	712.8	7ϕ16	A	157.95	1ϕ10 @140mm
						B	203.88	1ϕ10 @140mm
Sup B	-149.23	0.007	1540	712.8	8ϕ16	–		–
Span BC	33.58	0.0015	330	712.8	4ϕ16	B	152.37	1ϕ10 @140mm
						C	102.64	1ϕ10 @200mm
Sup C	-28.41	0.0013	286	712.8	6ϕ12	–		–

Reinforcement detailing for Beam (3)

Sec#	Mu (kN.m)	ρ	As (mm^2)	Asmin (mm^2)	Flexure reinforcement	Vu (kN)		Stirrups
Sup A	-39.58	0.003	281.25	305.25	4ϕ12	–		–
Span AB	70.59	0.0039	511.87	427.35	8ϕ14	A	123.09	1ϕ10 @140mm
						B	162.49	1ϕ10 @140mm
Sup B	-121.62	0.01	93.75	305.25	8ϕ18	–		–
Span BC	27.67	0.01	131.25	427.35	6ϕ12	B	117	1ϕ10 @140mm
						C	115.77	1ϕ10 @140mm
Sup C	-112.82	0.0092	862.5	305.25	8ϕ16	–		–
Span CD	61.76	0.0034	446.25	427.35	8ϕ12	C	154.94	1ϕ10 @140mm
						D	155.65	1ϕ10 @140mm
Sup D	-77.92	0.0094	881.25	305.25	7ϕ16	–		–
Span DE	33.27	0.0018	236.25	427.35	6ϕ12	D	123.73	1ϕ10 @140mm
						E	107.23	1ϕ10 @140mm
Sup E	-49.88	0.0039	365.63	305.25	6ϕ16	–		–

Reinforcement detailing for Columns

Direction	Pu (kN)	Mu (kN.m)	ρ	As (mm^2)	Asmin (mm^2)	Flexure reinforcement
Main	1792.9	223.7	0.0033	3302.9	1759	1ϕ18 @160mm
secondary	1792.9	155.4	0.0023	2963.1	2295	1ϕ16 @200mm

	2269	0.01	1800	1800	10ϕ16	2ϕ10 @250mm
C (2)	1450	0.01	1800	1800	8ϕ14	2ϕ10 @200mm
C (3)	1859	0.01	1800	1800	10ϕ16	2ϕ10 @250mm

Reinforcement detailing for Foundation (1)

REFERNCES:

DESIGN CODE AND SPECIFICATIONS :

1. The American concrete institute (ACI) :
   (Design guidelines of the ACI 318 – 11)

2. Jordanian Building Code (JBC):
   Live load and Dead Load (material weights)

3. Nelson Book :

Direction	Pu (kN)	Mu (kN.m)	ρ	As (mm^2)	Asmin (mm^2)	Flexure reinforcement
Main	1481.4	223.7	0.0033	2872.2	1530	1ϕ18 @160mm
secondary	1481.4	112.3	0.0018	2295	2295	1ϕ16 @200mm

Design of column