4

Chapter Four

Structural Analysis and Design

4.1 Introduction.

4.2 Design method and requirements.

4.3 Check of minimum thicknesses of structural members.

4.4 Design of topping.

4.5 (Rib 1 or 3 , BF) Calculations.

4.6 (Beam 2, BF) Design of Beam.

4.1 Introduction:

Many structures are built of reinforced concrete: bridges, buildings, retaining walls, tunnels, and others.

Reinforced concrete is logical union of two materials: plain concrete, which possesses highcompressive strength but little tensile strength, and steel bars embedded in the concrete, whichcan provide the needed strength in tension.

Plain concrete is made by mixing cement, fine aggregate, coarse aggregate, water, and frequentlyadmixtures.

Understanding of reinforced concrete behavior is still far from complete, building codes andspecifications that give design procedures are continually changing to reflect latest knowledge.

Structural concrete can be classified into:

- Lightweight concrete with unit weight from about 1350 to 1850 kg/m^3.
- Normal weight concrete with unit weight from about 1800 to 2400 kg/m^3.
- Heavyweight concrete with unit weight from about 3200 to 5600 kg/m^3.

4.2 Design method and requirements:

The design strength provided by a member is calculated in accordance with the requirements andassumptions of **ACI_code (318_14)**.

✓ **Strength design method:**

In ultimate strength design method, the service loads are increased by factors to obtain the loadat which failure is considered to be occurring.

This load called factored load or factored service load. The structure or structural element is thenproportioned such that the strength is reached when factored load is acting.

The computation ofthis strength takes into account the nonlinear stress-strain behavior of concrete.

The strength design method is expressed by the following,
<u>Strength provided ≥ strength required to carry factored loads.</u>

NOTE:
The statically calculation and the key plans dependent on the architectural plans.

- ✓ Code : ACI 2014
  UBC
- ✓ Material :

Concrete: B350…. ($fc' = 35*0.8 = 28MPa$) .

Reinforcement steel : The specified yield strength of the reinforcement
{fy = 420 N/mm²(MPa)}
Mild steel : A-36
Connection Type : Weld , Bolts

- ✓ **Factored loads:**

The factored loads for members in our project are determined by:
$W_u = 1.2\ D_L + 1.6\ S_L$ ACI-code-318-14(9.2.1).

4.3 Check of minimum thickness of structural member :

TABLE 9.5(a) — MINIMUM THICKNESS OF NONPRESTRESSED BEAMS OR ONE-WAY SLABS UNLESS DEFLECTIONS ARE CALCULATED. (ACI 318M-14)

	Minimum thickness , h			
	Simply supported	One end continuous	Both end continuous	Cantilever
Member	Members not supporting or attached to partitions or other construction likely to be damaged by large deflection			
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 (4.1): Check of minimum thickness of structural members

For rib :

h_{min}=L/18.5=6.13/18.5=33.14 cm
select : 35cm thickness with 27 cm block and 8 toping .

For beam :

$h_{min}$for(one end continuous)=L/18.5=6.93/18.5=37.46 cm
select h=(27+8)=35 cm for rib slab with drop beam h=50 cm (deflection control).

4.4 Design of topping:

✓ **Statically system for topping** :

Consider the topping as strip of (1m) width, and span of mold length with both end fixed in the ribs

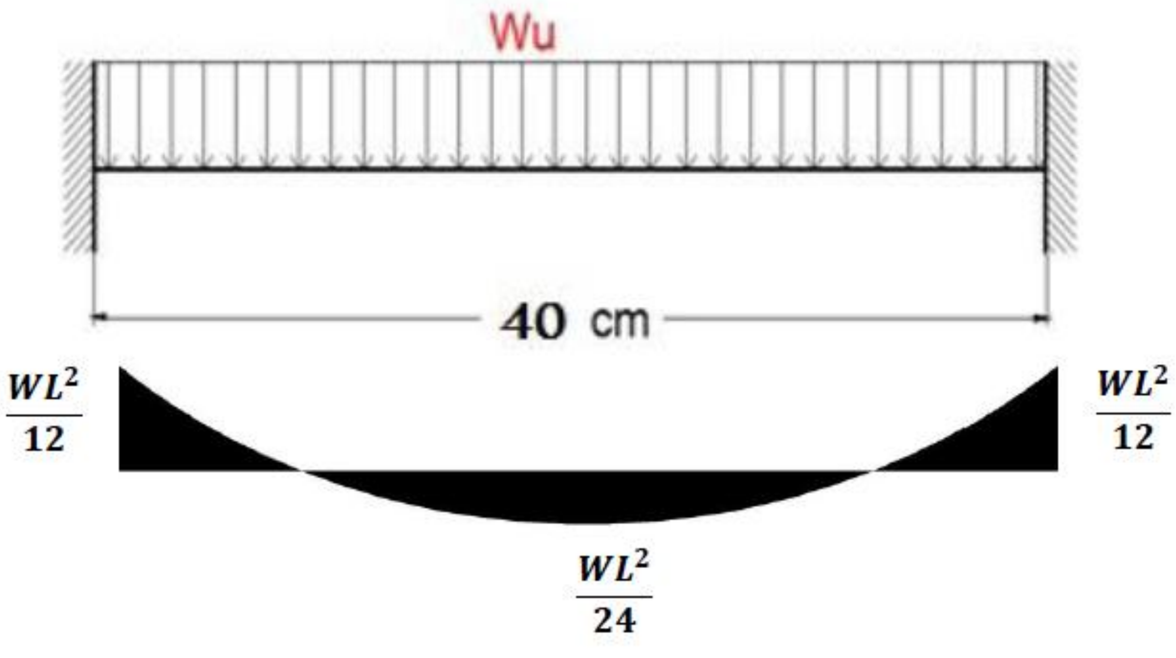


Fig 4.1: topping load and moment diagram.

For the topping , the total dead load to be used in the analysis and design is calculated as follows:

Table (4 – 2) Dead load calculation for topping

No.	Partsof Rib	Quality Density KN/m^3	Calculation
1	**Reinforced Concrete Topping**	25	0.08×25×1
2	**Sand**	16	0.07×16×1
3	**Mortar**	22	0.02×22×1
4	**Tile**	23	0.03×23×1
5	**Partition**	0	0×1
		∑ =	4.25 KN/m

Nominal total dead load = 4.25 KN/m^2.
Nominal total live load = 5KN/m^2.

Design of topping for ribbed slab as a plain concrete section :-

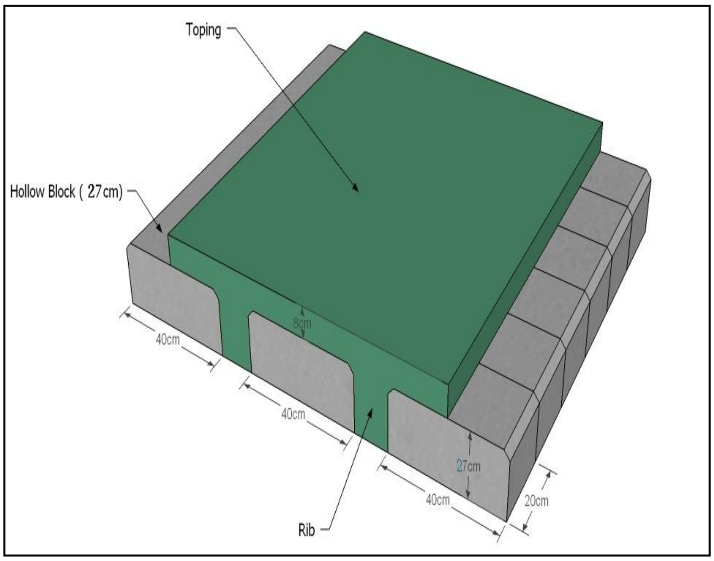


Fig. (4-2) : Topping of one way rib slab

$q_u = 1.2 \times D + 1.6 \times L$

13.1KN/m. (total factored load)

$$\text{Max } M_u = \frac{W_u * l^2}{12} = 0.175 \text{ KN.m}$$

$$\text{Max } V_u = \frac{W_u * l}{2} = 2.62 \text{ KN.m}$$

Design of shear:

Used fy = 420 MPa& $fc' = 24$MPa

$$\Phi * V_c = 0.75 \times \sqrt{28} \times \frac{1}{6} \times 1000 \times 80 = 52.92KN >> 2.62kN$$

No shear reinforcement is required.

Design of Moment:

$$\emptyset M_n = 0.55 * 0.42 * \sqrt{28} * 1000 * 80^2/6 = 1.3 \text{ KN.m}$$

$$\emptyset M_n = 1.3 \text{ KN.m} > M_u = 0.175 \text{ KN.m}$$

No structural reinforcement is required.

The strength of plain concrete section > loaded section.

The plain concrete section is safe; however, minimum reinforcement for shrinkage and temperature to control the cracks should be used.

$\rho = 0.0018$,*ACI-318-11*

$$A_s = \rho * b * h = 0.0018 * 1000 * 80 = 144 \text{ mm}^2.$$

∴Use Φ 8 @15 cm

$As = 335.1mm^2/m > As_{\min} = 144mm^2/m$ *Ok*

4.5) Design Rib :

Design of Rib R12

Fig. (4-3) : Ground floor Ribs

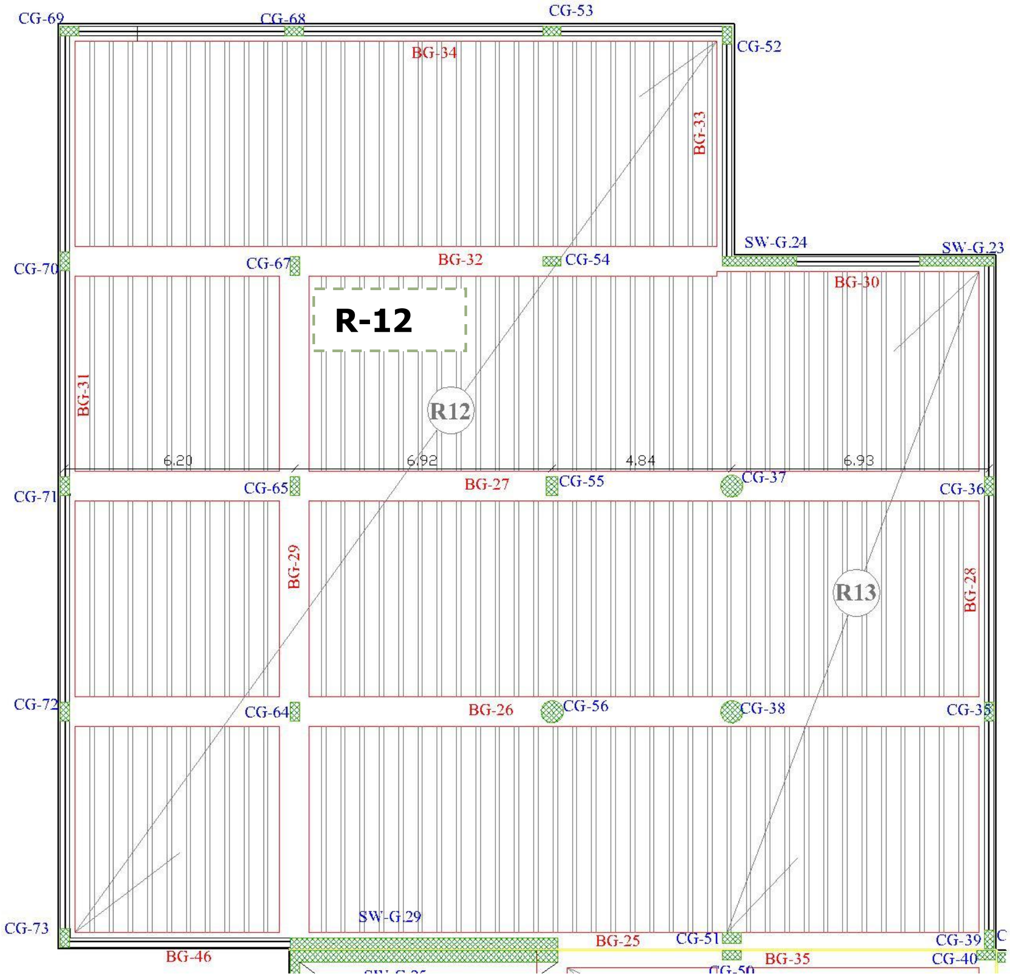


Fig. (4-4) : Rib (12) location in ground floor

For the one-way ribbed slabs, the total dead load to be used in the analysis and design is calculated as follows:

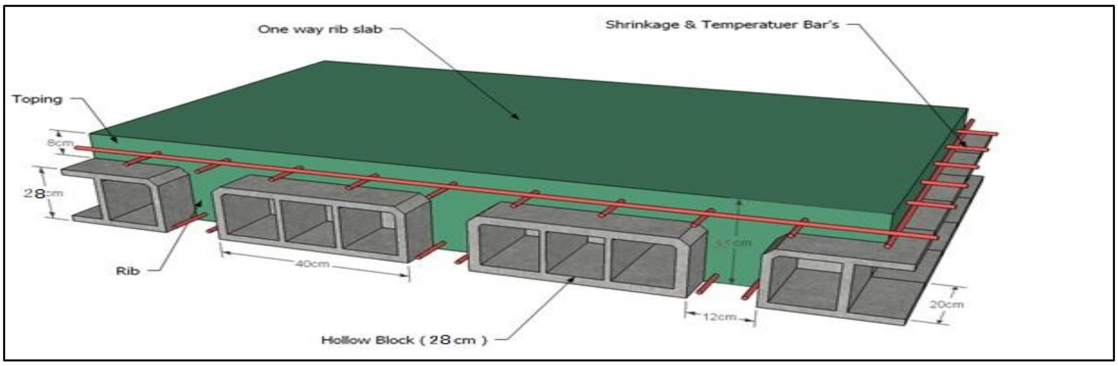


Fig. (4-5) : One way rib slab

Calculation of the total dead load for one way rib slab is shown in the following table:

Table (4 – 3) Calculation of the total dead load for one way rib slab.

No.	Material	Quality Density KN/m^3	Calculation
1	**Topping**	25	0.52×0.08×25 = 1.04
2	**Rib**	25	0.27×0.12×25 = 0.81
3	**Sand**	16	0.52×0.07×16 = 0.5824
4	**Mortar**	22	0.52×0.02×22 =0.2288
5	**Tile**	23	0.52×0.03×23 =0.3588
6	**Plaster**	22	0.52×0.02×22 =0.2288
7	**Block**	15	0.4×0.27×15 = 1.62
8	**Partitions**	0.0	0.0

∑ =	4.87	KN/m

By using **ATIR** program we get the envelope moment and shear force diagram as the follows:-

G e o m e t r y   Units:meter,cm

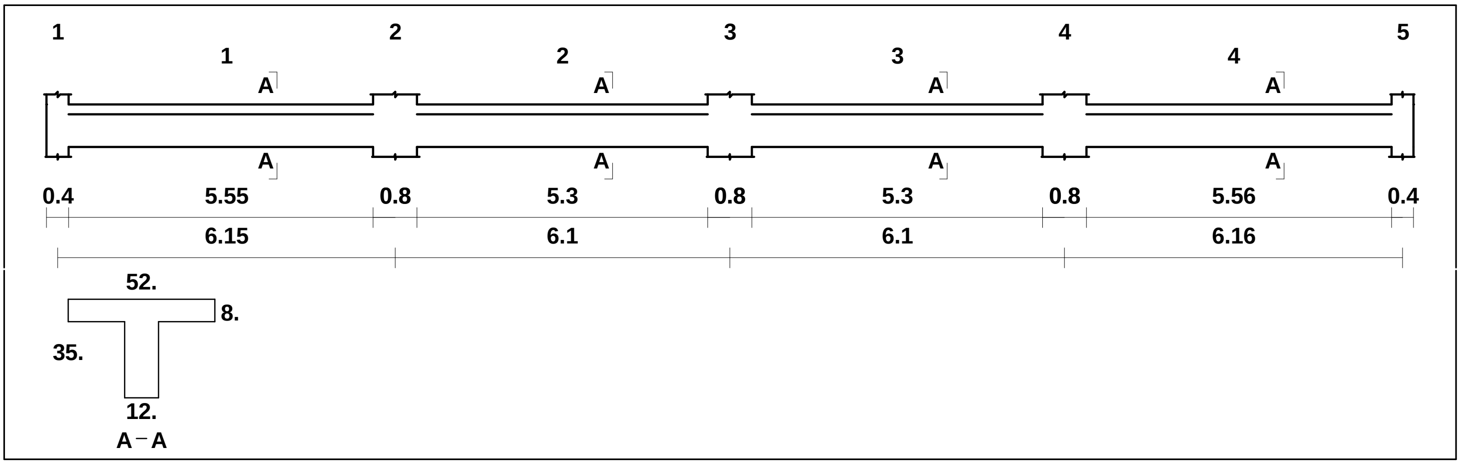
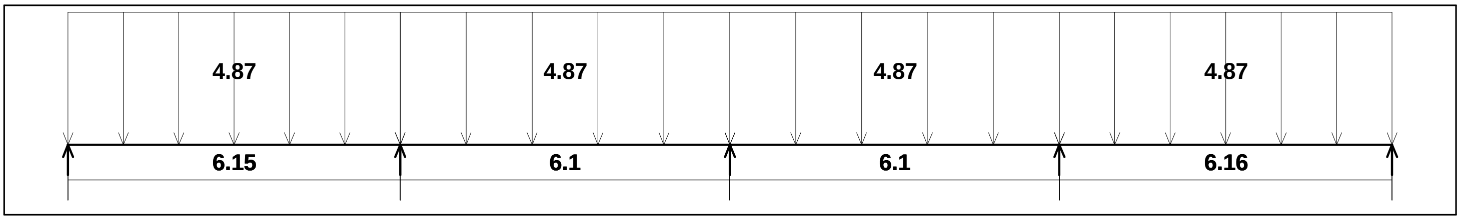


Figure (4.6) Geometry of rib R12

L o a d i n

load group no. 1
Dead load - Service

Units:kN,meter



Live load - Service

Load factors: 1.20,1.20/1.60,0.00

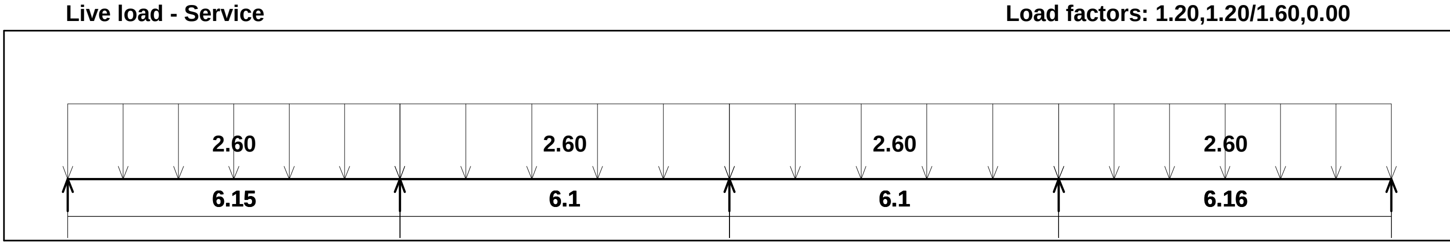
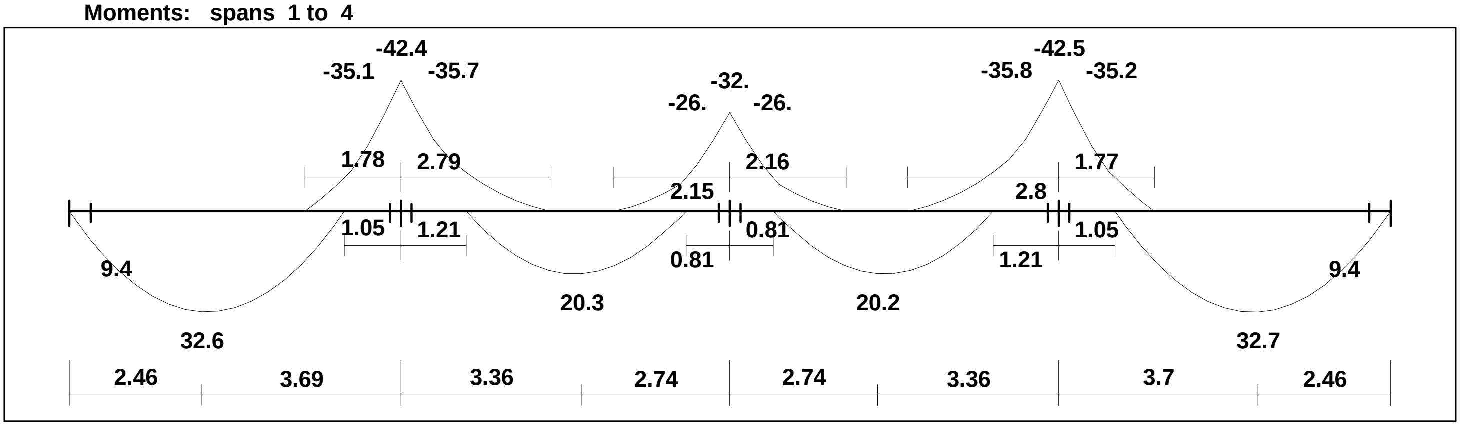


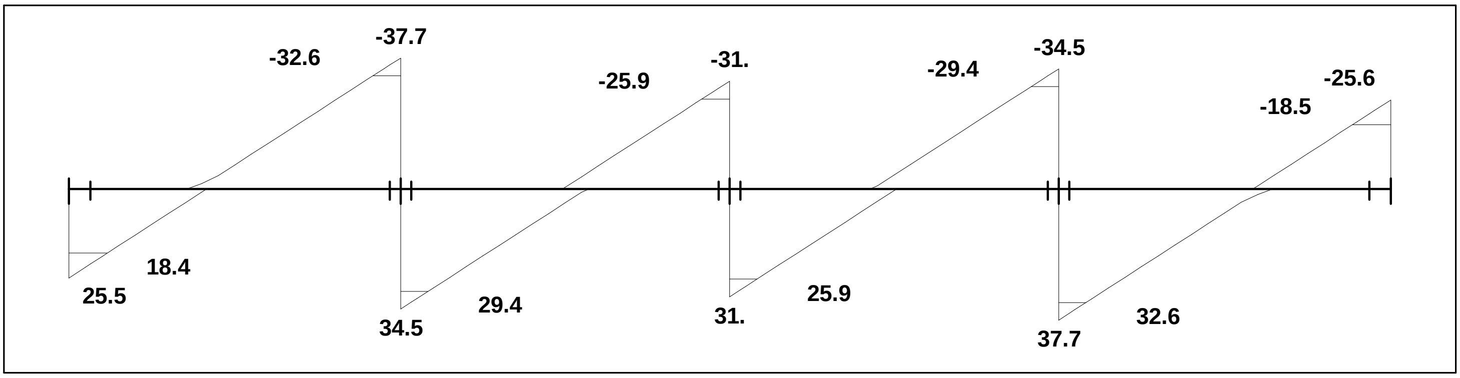
Figure (4.7) Loading of rib R12(KN/m).

M o m e n t / S h e a r   E n v e l o p e (Factored)   Units:kN,meter

Moments: spans 1 to 4



Shear



Reactions

Factored

DeadR	14.14	40.97	32.95	41.01	14.16
LiveR	11.41	31.2	29.02	31.23	11.42
Max R	25.55	72.17	61.97	72.24	25.59
Min R	12.79	52.74	41.9	52.77	12.82
Service					
DeadR	11.78	34.14	27.46	34.18	11.8
LiveR	7.13	19.5	18.13	19.52	7.14
Max R	18.91	53.64	45.6	53.69	18.94
Min R	10.94	41.5	33.05	41.53	10.97

Figure (4.8) Moment and Shear Envelop for rib R12

- **<u>Design of shear for rib R12</u>**

Categories for shear design:

Vu = 32.6KN

Use Φ8 with two legs

d=350-20-8-7=315

1. ***Region I :***

$1.1\Phi Vc \geq Vu$

$$1.1\Phi\ \mathrm{Vc} = 1.1\Phi \frac{\sqrt{fc'}}{6} \times \mathrm{b_w} \times \mathrm{d}$$

$$1.1\Phi\ \mathrm{Vc} = 1.1 \times 0.75 \times \frac{\sqrt{28}}{6} \times 120 \times 315 = 27.5\mathrm{KN} < \mathrm{Vu} = 32.6\mathrm{KN}$$

$$\Phi\ \mathrm{Vs\ min} = \Phi \times \frac{\sqrt{fc'}}{16} \times bw \times d$$

$$\Phi\ \mathrm{Vs\ min} = 0.75 \times \frac{\sqrt{28}}{16} \times 120 \times 315 = 9.38KN$$

$$\Phi Vs_{\min} = \Phi(\frac{1}{3}) \times bw \times \mathrm{d}$$

$$\Phi Vs_{\min} = 0.75(\frac{1}{3}) \times 120 \times 315 = 9.45KN$$

$$\Phi Vc + \Phi Vs_{\min} = 25 + 9.45 = 34.45KN$$

$$27.5KN < 32.6KN \leq 34.45KN$$

Case III minimum Shear reinforcement required .So,

Use Φ 8,2 leg

A_v = 100.53 mm^2.

$$\mathrm{Vs} = \frac{v_u}{\emptyset} - v_c = \frac{32.6}{0.75} - 33.33 = 10.14\ KN$$

$$Sreq = \frac{A_v f_{yt}\, d}{v_s} = \frac{100.53 * 420 * 315}{10.14 * 1000} = 1311.65\, mm$$

$$s_{req} \leq s_{max}$$

$$s_{max} \leq \frac{d}{2}\ or\ s_{max} \leq 600\, mm$$

$$\frac{d}{2} = \frac{315}{2} = 158\, mm\ (control)$$

Use Φ8, @ 15 cm (2Legs).

- **Design of Positive Moment:**

Effective Flange width (b_E) , *ACI-318-14*

b_E For T- section is the smallest of the following:

b_E = (2200) / 4 = 550 mm

b_E = 120 + 16 (80) = 1400mm

b_E = 520 mm.................. control

»Use M_u max positive for span 1 = 32.6 kN.m

» Determine whether the rib will act as rectangular or T – section:

For hf = 0.08 m

~ Assume bar diameter Φ14 for main positive reinforcement.

d = 350 – 20 – 8–7= 315mm

Φ*Mn =0.9*0.85* fc'*b*hf* (d- hf/2)

= 0.9*0.85*28*0.52*0.08*(0.315-0.08/2)=245 KN.m

Φ*Mn= 245 KN.m>> M_u = 32.6 KN.m

The section will be designed as a rectangular section with b_E = 520mm

$$As\ \min = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) >= As\ \min = \frac{1.4}{(fy)}(bw)(d)\ \textit{ACI-318}\ \text{-14}$$

$$As\ \min = \frac{\sqrt{28}}{4(420)}(120)(315) = 119.1 mm^2$$

$$As\ \min = \frac{1.4}{420}(120)(315) = 126mm^2 \sim \underline{\text{control}}$$

$$m = \frac{fy}{0.85\,fc'} = \frac{420}{0.85(28)} = 17.65$$

$$Kn = \frac{Mu}{\Phi bd^2} = \frac{32.6 * 10^6}{(0.9)(520)(315)^2} = 0.70\,Mpa$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2*m*kn}{fy}}\right) = \frac{1}{17.65}\left(1 - \sqrt{1 - \frac{2*17.65*0.7}{420}}\right) = 0.00169$$

A_s = 0.00169(520) (315) = 276.82 mm^2> A_s min = 126 mm^2

# of bars = A_s / $A_{s\ bar}$ = 276.82/154= 1.798 * Note $A_{\Phi 14}$= 154mm^2

Select bottom bars 2Φ14

Total $As_{(provide)}$ =308 mm^2>276.82mm^2

* Check Strain for the magnitude of under strength factor Φ:

Tension = Compression

$$A_s \times fy = 0.85 \times f_c' \times b \times a$$

$$308 \times 420 = 0.85 \times 28 \times 520 \times a$$

$$a = 10.45mm$$

$$x = \frac{a}{0.85} = \frac{10.45}{0.85} = 12.29mm$$

$$\varepsilon_s = \frac{315 - 12.29}{12.29} \times 0.003 = 0.0739$$

$$\varepsilon_s = 0.0739 > 0.005$$

Ok…….

»Use M_u max positive for span 2 = 20.3kN.m

» Determine whether the rib will act as rectangular or T – section:

For hf = 0.08 m

~ Assume bar diameter Φ14 for main positive reinforcement.

d = 350 – 20 – 8–7= 315mm

Φ*Mn =0.9*0.85* fc'*b*hf* (d- hf/2)

= 0.9*0.85*28*0.52*0.08*(0.315-0.08/2)=245 KN.m

Φ*Mn= 245 KN.m>> M_u = 20.3 KN.m

The section will be designed as a rectangular section with b_E = 520mm

$$As\ \min = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) >= As\ \min = \frac{1.4}{(fy)}(bw)(d)\ ACI\text{-}318\text{ -14}$$

$$As\ \min = \frac{\sqrt{28}}{4(420)}(120)(315) = 119.1mm^2$$

$$As\ \min = \frac{1.4}{420}(120)(315) = 126mm^2 \sim \underline{\text{control}}$$

$$m = \frac{fy}{0.85\, fc'} = \frac{420}{0.85(28)} = 17.65$$

$$Kn = \frac{Mu}{\Phi bd^2} = \frac{20.3*10^6}{(0.9)(520)(315)^2} = 0.44Mpa$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2*m*kn}{fy}}\right) = \frac{1}{17.65}\left(1-\sqrt{1-\frac{2*17.65*0.44}{420}}\right) = 0.00106$$

A_S = 0.00106(520) (315) = 173.63mm^2> $As\ \min$ = 126 mm^2

# of bars = A_s / $A_{s\ bar}$ = 173.63/113= 1.53 * Note $A_{\Phi 12}$= 113mm^2

Select bottom bars 2Φ12

Total $As_{(provide)}$ =226.19 mm^2>173.63mm^2

* Check Strain for the magnitude of under strength factor Φ:

Tension = Compression

$$A_s \times fy = 0.85 \times f_c' \times b \times a$$

$$226.19 \times 420 = 0.85 \times 28 \times 520 \times a$$

$$a = 7.68mm$$

$$x = \frac{a}{0.85} = \frac{7.68}{0.85} = 9mm$$

$$\varepsilon_s = \frac{315-9}{9} \times 0.003 = 0.102$$

$\varepsilon_s = 0.102 > 0.005$

Ok…….

»Use M_u max positive for span 3 = 20.2kN.m =”span 2 Mu max=20.3KN.m”

» Determine whether the rib will act as rectangular or T – section:

For hf = 0.08 m

~ Assume bar diameter Φ14 for main positive reinforcement.

d = 350 – 20 – 8–7= 315mm

Φ*Mn =0.9*0.85* ƒc'*b*hf* (d- hf/2)

= 0.9*0.85*28*0.52*0.08*(0.315-0.08/2)=245 KN.m

Φ*Mn= 245 KN.m>> M_u = 20.3 KN.m

The section will be designed as a rectangular section with b_E = 520mm

$$A_s \ \min = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) >= A_s \ \min = \frac{1.4}{(fy)}(bw)(d) \ \textit{ACI-318 -14}$$

$$A_s \ \min = \frac{\sqrt{28}}{4(420)}(120)(315) = 119.1 mm^2$$

$$A_s \ \min = \frac{1.4}{420}(120)(315) = 126 mm^2 \sim \underline{\text{control}}$$

$$m = \frac{fy}{0.85\,fc'} = \frac{420}{0.85(28)} = 17.65$$

$$Kn = \frac{Mu}{\Phi bd^2} = \frac{20.3*10^6}{(0.9)(520)(315)^2} = 0.44\,Mpa$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2*m*kn}{fy}}\right) = \frac{1}{17.65}\left(1-\sqrt{1-\frac{2*17.65*0.44}{420}}\right) = 0.00106$$

A_s = 0.00106(520) (315) = 173.63 mm^2> A_s min = 126 mm^2

# of bars = A_s / $A_{s\ bar}$ = 173.63/113= 1.53 * Note $A_{\Phi 12}$= 113mm^2

Select bottom bars 2Φ12

Total $As_{(provide)}$ =226.19 mm^2>173.63mm^2

* Check Strain for the magnitude of under strength factor Φ:

Tension = Compression

$A_s \times fy = 0.85 \times f_c' \times b \times a$

$226.19 \times 420 = 0.85 \times 28 \times 520 \times a$

$a = 7.68mm$

$$x = \frac{a}{0.85} = \frac{7.68}{0.85} = 9mm$$

$$\varepsilon_s = \frac{315 - 9}{9} \times 0.003 = 0.102$$

$\varepsilon_s = 0.102 > 0.005$

Ok…….

»Use M_u max positive for span 4 = 32.7 kN.m =”span 1 Mu max=32.6 KN.m ”

» Determine whether the rib will act as rectangular or T – section:

For hf = 0.08 m

~ Assume bar diameter Φ14 for main positive reinforcement.

d = 350 – 20 – 8–7= 315mm

Φ*Mn =0.9*0.85* ƒc'*b*hf* (d- hf/2)

= 0.9*0.85*28*0.52*0.08*(0.315-0.08/2)=245 KN.m

Φ*Mn= 245 KN.m>> M_u = 32.6 KN.m

The section will be designed as a rectangular section with b_E = 520mm

$As\ \min = \frac{\sqrt{fc'}}{4(fy)}(bw)(d)$ >=$As\ \min = \frac{1.4}{(fy)}(bw)(d)$ *ACI-318* -14

$$As\ \min = \frac{\sqrt{28}}{4(420)}(120)(315) = 119.1mm^2$$

$As\ \min = \frac{1.4}{420}(120)(315) = 126mm^2$ ~ <u>control</u>

$$m = \frac{fy}{0.85\,fc'} = \frac{420}{0.85(28)} = 17.65$$

$$Kn = \frac{Mu}{\Phi bd^2} = \frac{32.6*10^6}{(0.9)(520)(315)^2} = 0.70\,Mpa$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2*m*kn}{fy}}\right) = \frac{1}{17.65}\left(1 - \sqrt{1 - \frac{2*17.65*0.7}{420}}\right) = 0.00169$$

A_s = 0.00169(520) (315) = 276.82 mm^2> $A_{s\ min}$ = 126 mm^2

# of bars = A_s / $A_{s\ bar}$ = 276.82/154= 1.798 * Note $A_{\Phi 14}$= 154mm^2

Select bottom bars 2Φ14

Total As $_{(provide)}$ =308 mm^2>276.82mm^2

* Check Strain for the magnitude of under strength factor Φ:

Tension = Compression

$A_s \times fy = 0.85 \times f_c' \times b \times a$

$308 \times 420 = 0.85 \times 28 \times 520 \times a$

$a = 10.45mm$

$$x = \frac{a}{0.85} = \frac{10.45}{0.85} = 12.29mm$$

$$\varepsilon_s = \frac{315 - 12.29}{12.29} \times 0.003 = 0.0739$$

$\varepsilon_s = 0.0739 > 0.005$

Ok…….

- **<u>Design of Max Negative Moment for (Rib):</u>**

»The maximum negative moment from spans with support (2) =”support(4)” is

Mu = - 35.8 kN.m

Mn = 35.8/ 0.9 = 39.78 kN.m

~ Assume bar diameter Φ16for main negative reinforcement.

d = 350 – 20 – 8–8= 314mm

Φ*Mn =0.9*0.85* fc'*b*hf* (d- hf/2)

= 0.9*0.85*28*0.52*0.27*(0.314-0.27/2)=538.32KN.m

Φ*Mn= 538.32KN.m>> M_u = 35.8 KN.m

The section will be designed as a rectangular section with b_E = 520mm

$$As\ \min = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) >= As\ \min = \frac{1.4}{(fy)}(bw)(d)\ ACI\text{-}318\text{-}14$$

$$As\ \min = \frac{\sqrt{28}}{4(420)}(120)(314) = 118.7mm^2$$

$$As\ \min = \frac{1.4}{420}(120)(314) = 125.6mm^2 \sim \underline{\text{control}}$$

$$m = \frac{fy}{0.85\,fc'} = \frac{420}{0.85(28)} = 17.65$$

$$Kn = \frac{Mu}{\Phi bd^2} = \frac{35.8*10^6}{(0.9)(520)(314)^2} = 0.78Mpa$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2*m*kn}{fy}}\right) = \frac{1}{17.65}\left(1-\sqrt{1-\frac{2*17.65*0.78}{420}}\right) = 0.00189$$

A_s = 0.00189(520) (314) = 308.6 mm^2> $As\ \min$ = 126 mm^2

# of bars = A_s / $A_{s\ bar}$ = 308.6/201.1= 1.53 * Note $A_{\Phi16}$= 201.1mm^2

Select top bars 2Φ16

Total $As_{(provide)}$ =402.12 mm^2>308.6mm^2

* Check Strain for the magnitude of under strength factor Φ:

Tension = Compression

$$A_s \times fy = 0.85 \times f_c' \times b \times a$$

$$402.12 \times 420 = 0.85 \times 28 \times 520 \times a$$

$$a = 13.65mm$$

$$x = \frac{a}{0.85} = \frac{13.65}{0.85} = 16.06mm$$

$$\varepsilon_s = \frac{314 - 16.06}{16.06} \times 0.003 = 0.0557$$

$\varepsilon_s = 0.0557 > 0.005$

Ok…….

»The maximum negative moment from spans with support (3) is

Mu = 26 kN.m

Mn = 26/ 0.9 = 28.89kN.m

~ Assume bar diameter Φ14for main negative reinforcement.

d = 350 – 20 – 8–7= 315mm

Φ*Mn =0.9*0.85* ƒc'*b*hf* (d- hf/2)

= 0.9*0.85*28*0.52*0.27*(0.315-0.27/2)=541.33 KN.m

Φ*Mn= 541.33KN.m>> M_u =26 KN.m

The section will be designed as a rectangular section with b_E = 520mm

$$As\ \min = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) >= As\ \min = \frac{1.4}{(fy)}(bw)(d)\ ACI\text{-}318\text{-}14$$

$$As\ \min = \frac{\sqrt{28}}{4(420)}(120)(315) = 119.1mm^2$$

$$As\ \min = \frac{1.4}{420}(120)(315) = 126mm^2$$ ~ control

$$m = \frac{fy}{0.85fc'} = \frac{420}{0.85(28)} = 17.65$$

$$Kn = \frac{Mu}{\Phi bd^2} = \frac{26 * 10^6}{(0.9)(520)(315)^2} = 0.56Mpa$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2*m*kn}{fy}}\right) = \frac{1}{17.65}\left(1 - \sqrt{1 - \frac{2*17.65*0.56}{420}}\right) = 0.00135$$

A_s = 0.00135(520) (315) = 221.13mm^2> As min = 126 mm^2

# of bars = A_s / $A_{s\ bar}$ = 221.13/154= 1.44 * Note $A_{\Phi 14}$= 154mm^2

Select top bars 2Φ14

Total As $_{(provide)}$ =308 mm^2>126mm^2

* Check Strain for the magnitude of under strength factor Φ:

Tension = Compression

$$A_s \times fy = 0.85 \times f_c' \times b \times a$$

$$308 \times 420 = 0.85 \times 28 \times 520 \times a$$

$$a = 10.45mm$$

$$x = \frac{a}{0.85} = \frac{10.45}{0.85} = 12.29mm$$

$$\varepsilon_s = \frac{315 - 12.29}{12.29} \times 0.003 = 0.0739$$

$$\varepsilon_s = 0.0739 > 0.005$$

Ok…….

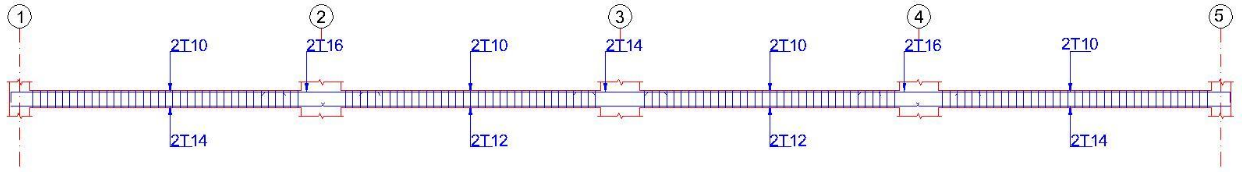
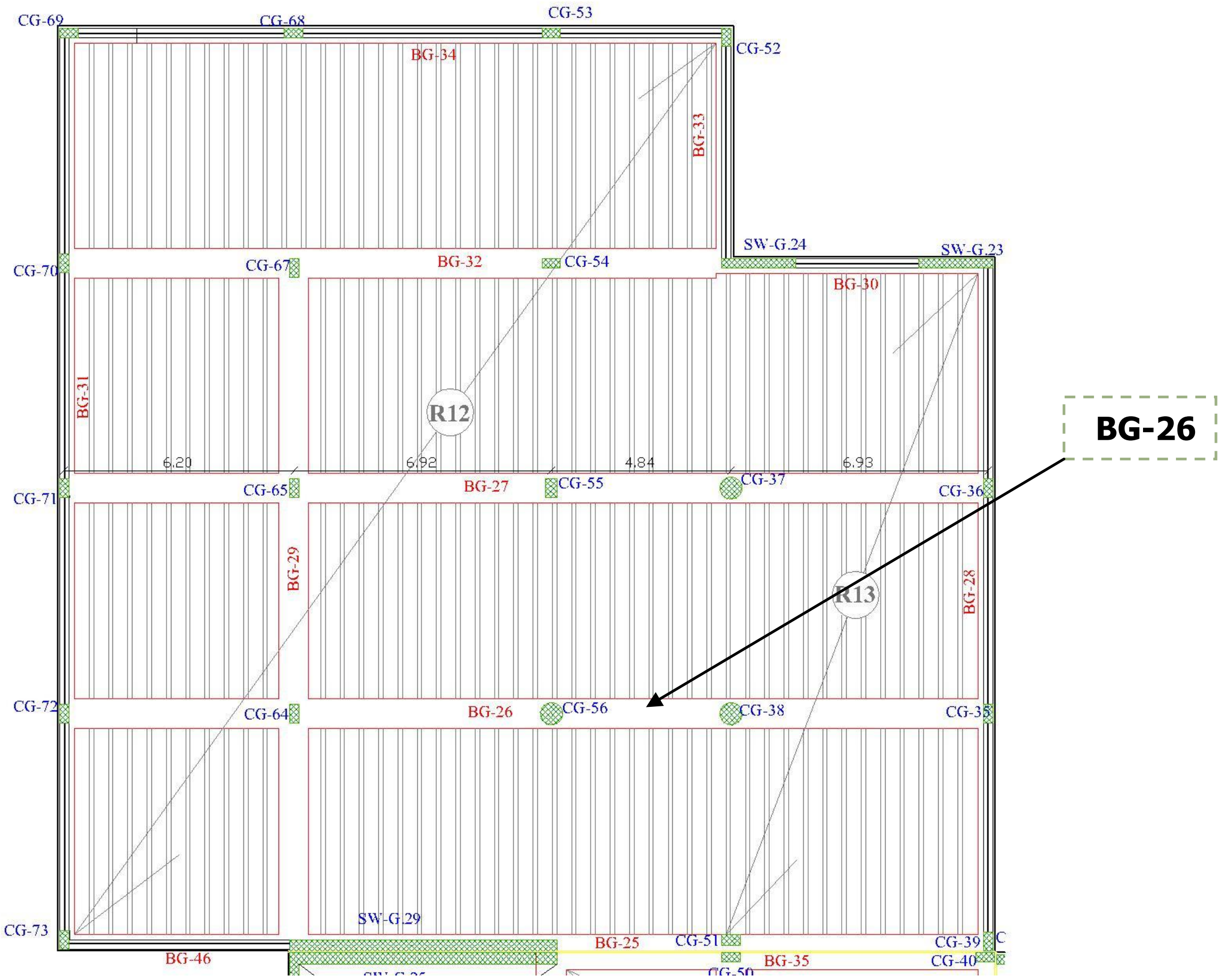


Figure 9 beam detaling

Design of Beam BG-26



Figure(4.10)Beam location in ground floor slab

Load calculations for Beam:

The distributed Dead and Live loads acting upon the Beam **BG-26** can be defined from the support reactions of the rib **R12 AND R13.**

Reactions

Factored					
DeadR	14.14	40.97	32.95	41.01	14.16
LiveR	11.41	31.2	29.02	31.23	11.42
Max R	25.55	72.17	61.97	72.24	25.59
Min R	12.79	52.74	41.9	52.77	12.82
Service					
DeadR	11.78	34.14	27.46	34.18	11.8
LiveR	7.13	19.5	18.13	19.52	7.14
Max R	18.91	53.64	45.6	53.69	18.94
Min R	10.94	41.5	33.05	41.53	10.97

Figure (4.11) Reaction of rib R12

- The support reaction (service) from Dead loads of Rib (R12) upon beam (BG-26) is **(34.14KN)**. The distributed Dead load from Rib (R12) on Beam(BG-26):

$$DL_{from\ Rib}=\frac{34.14}{0.52}=65.65\ KN/m$$

- The support reaction (service) from Live loads of Rib (R12) upon beam (BG-26) is **(19.52KN)**. The distributed Live load from Rib (RG12) on Beam (BG-26):

$$LL_{from\ Rib} = \frac{19.52}{0.52} = 37.5\ KN/m$$

By using **ATIR** program we get the envelope moment and shear diagram as the follows:-

G e o m e t r y Units:meter,cm

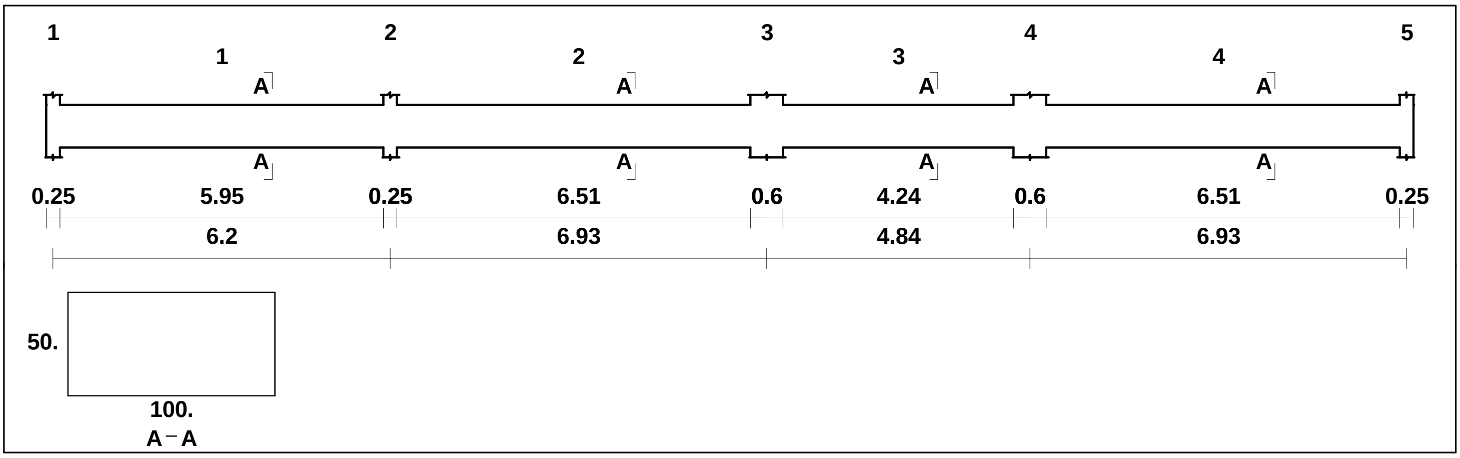


Figure (4.12) Geometry of Beam BG-26

L o a d i n g

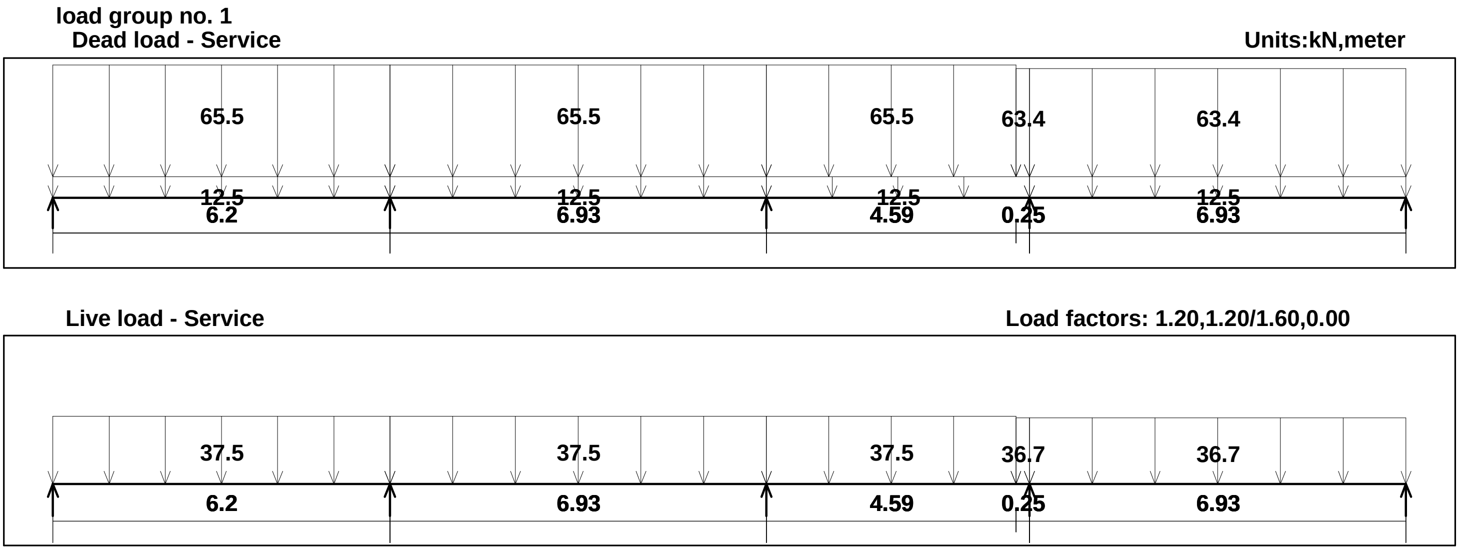


Figure (4.13) Loading of Beam BG-26 (KN/m).

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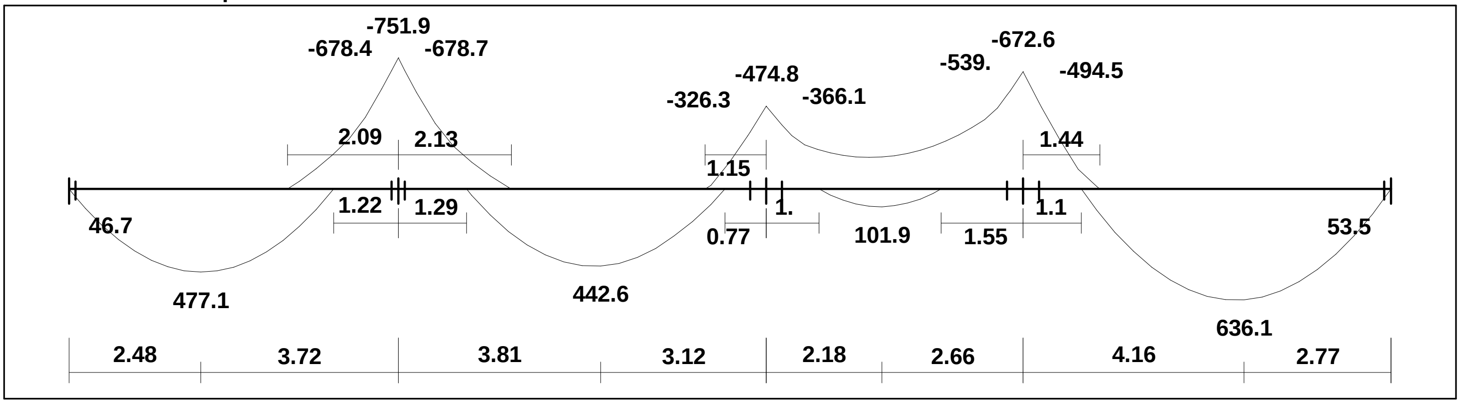
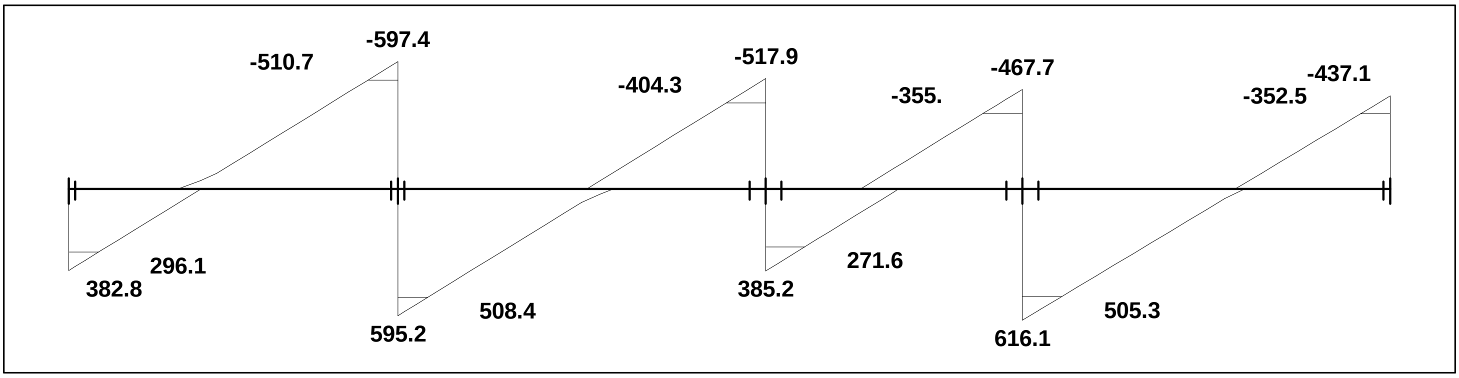
M o m e n t / S h e a r E n v e l o p e (Factored) Units:kN,meter

Moments: spans 1 to 4



Shear



Reactions

Factored

DeadR	217.96	717.78	487.77	629.27	259.77
LiveR	164.88	474.85	415.41	454.57	177.34
Max R	382.84	1192.63	903.18	1083.84	437.11
Min R	192.78	929.2	543.08	731.88	249.93
Service					
DeadR	181.63	598.15	406.48	524.39	216.47
LiveR	103.05	296.78	259.63	284.1	110.84
Max R	284.68	894.93	666.11	808.5	327.31
Min R	165.9	730.29	441.04	588.52	210.32

Figure (4.14) Moment and Shear envelop for Beam BG-26

Assume bar diameter Φ25 for main reinforcement.

Selected drop beam

$b_w = 80cm, h = 50cm$

$$d = 500 - 40 - 12 - \frac{25}{2} = 435.5mm$$

- **Design of shear for Beam :**

ACI – 318 – Categories for shear design:

$Vu_{critical}$ = 510 KN

$V_c = \frac{1}{6}\sqrt{fc'}b_w\, d$

$V_c = \frac{1}{6}\sqrt{28} * 1000 * 435.5$

V_c= 384.1KN.

Φ V_c= 0.75*384.1=288.1 KN

$v_{s,min} = \frac{1}{16}\sqrt{fc'}b_w\, d$

$v_{s,min} = \frac{1}{16}\sqrt{28} * 1000 * 435.5$

$v_{s,min} = 144\ kn$

$v_{s,min} = \frac{1}{3}b_w\, d$

$v_{s,min} = \frac{1}{3} * 1000 * 435.5$

$v_{s,min} = 145.2KN$

Case III minimum Shear reinforcement required .So,

$\emptyset(v_c + v_{s,\mathrm{min})} < v_u \le 3\ \emptyset * v_c$

0.75(384.1+145)=396.83<510<3* 0.75*384.1=864.23

So, shear reinforcement are required.

Use 4 leg Φ 12.

Av =452.4 mm^2.

$V_s = V_n – V_C = \frac{510}{0.75}$ – 384.1= 295.9KN

$S = \frac{A_v f_{yt}\, d}{v_s} = \frac{452.4 * 420 * 435.5}{384.1 * 1000} = 215.43\ mm$

$s_{max} \le \frac{d}{2}\ or\ s_{max} \le 600\ mm$

$S\ max = \frac{d}{2} = \frac{435.5}{2} = 217.75\ mm$

Select 4 leg Φ12, @ 150 mm (2 Legs)

- <u>Design of Beam of negative moment :</u>

➢ **Mu =-678.7 KN.m at support (2).**

Mn= Mu/0.9

= 678.7/0.9 =754.11 KN.m

~ Assume bar diameter Φ25for main negative reinforcement.

d = 500-40-12-12.5=435.5 mm

$$m = \frac{fy}{0.85 * fc'} = \frac{420}{0.85 * 28} = 17.65$$

$$Kn = \frac{Mn}{bd^2} = \frac{678.7 * 10^6}{(0.9)(1000)(435.5)^2} = 3.98 Mpa$$

$$A_s \min = \frac{\sqrt{28}}{4(420)}(1000)(435.5) = 1371.7 mm^2$$

$$A_s \min = \frac{1.4}{420}(1000)(435.5) = 1451.67 mm^2 \text{ ~ control}$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mkn}{fy}}\right) = \frac{1}{17.65}\left(1 - \sqrt{1 - \frac{2 * 17.65 * 3.98}{420}}\right) = 0.0104$$

A_s = 0.0104 (1000) (435.5) =4529.2mm^2> A_s min = 1451.67mm^2

# of bars = A_s / $A_{s\ bar}$ = 4529.2/491 = 9.22 * Note $A_{\Phi 25}$ = 491 mm^2

Select bar 10 Φ 25

Total $A_{s\ (provide)}$ = 4910mm^2>4529.2 mm^2

* Check strain for the magnitude of under strength factor Φ:

Tension = Compression

$$A_s \times fy = 0.85 \times f_c' \times b \times a$$

$$4910 \times 420 = 0.85 \times 28 \times 1000 \times a$$

$$a = 86.65 mm$$

$$X = \frac{a}{0.85} = \frac{86.65}{0.85} = 101.94 mm$$

$$\varepsilon_s = \frac{435.5 - 101.94}{101.94} \times 0.003 = 0.0098$$

$$\varepsilon_s = 0.0098 > 0.005$$

Ok…….

***Check for bar distance:**

$$S = \frac{1000 - 2\times 40 - 2\times 12 - 10\times 25}{9} = 71.78mm > 25mm....ok$$

- **Mu =-366.1 KN.m at support (3).**

Mn= Mu/0.9
= 366.1/0.9 = 406.78 KN.m

~ Assume bar diameter Φ25for main negative reinforcement.

d = 500-40-12-12.5=435.5 mm

$$m = \frac{fy}{0.85 * fc'} = \frac{420}{0.85*28} = 17.65$$

$$Kn = \frac{Mn}{bd^2} = \frac{366.1*10^6}{(0.9)(1000)(435.5)^2} = 2.14Mpa$$

$$As\ \min = \frac{\sqrt{28}}{4(420)}(1000)(435.5) = 1371.7mm^2$$

$$As\ \min = \frac{1.4}{420}(1000)(435.5) = 1451.67mm^2 \text{ ~ control}$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mkn}{fy}}\right) = \frac{1}{17.65}\left(1 - \sqrt{1 - \frac{2*17.65*2.14}{420}}\right) = 0.00535$$

A_s = 0.00535 (1000) (435.5) =2330mm^2> $As\ \min$ = 1451mm^2

# of bars = A_s / $A_{s\ bar}$ = 2330/491 = 4.75 * Note $A_{\Phi 25}$ = 491 mm^2

Select bar 5 Φ 25

Total $As_{(provide)}$ = 2455mm^2>2330 mm^2

* Check strain for the magnitude of under strength factor Φ:

Tension = Compression

$$A_s \times fy = 0.85 \times f_c' \times b \times a$$

$2455 \times 420 = 0.85 \times 28 \times 1000 \times a$

$a = 43.32mm$

$$X = \frac{a}{0.85} = \frac{43.32}{0.85} = 50.96mm$$

$$\varepsilon_s = \frac{435.5 - 50.96}{50.96} \times 0.003 = 0.0.22$$

$\varepsilon_s = 0.022 > 0.005$

Ok…….

***Check for bar distance:**

$$S = \frac{1000 - 2 \times 40 - 2 \times 12 - 5 \times 25}{4} = 192.75mm > 25mm....ok$$

- **Mu =-539 KN.m at support (4).**

Mn= Mu/0.9
= 539/0.9 =598.89 KN.m

~Assume bar diameter Φ25for main negative reinforcement.

d = 500-40-12-12.5=435.5 mm

$$m = \frac{fy}{0.85 * fc'} = \frac{420}{0.85 * 28} = 17.65$$

$$Kn = \frac{Mn}{bd^2} = \frac{539 * 10^6}{(0.9)(1000)(435.5)^2} = 3.16 Mpa$$

$$As\ \min = \frac{\sqrt{28}}{4(420)}(1000)(435.5) = 1371.7mm^2$$

$$As\ \min = \frac{1.4}{420}(1000)(435.5) = 1451.67mm^2 \sim \text{control}$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mkn}{fy}}\right) = \frac{1}{17.65}\left(1 - \sqrt{1 - \frac{2 * 17.65 * 3.16}{420}}\right) = 0.0081$$

A_s = 0.0081 (1000) (435.5) =3527.6 mm^2> A_s min = 1451mm^2

\# of bars = A_s / $A_{s\ bar}$ = 3527.6/491 = 7.18 * Note $A_{\Phi 25}$ = 491 mm^2

Select bar 8 Φ 25

Total $As_{(provide)}$ = 3928mm^2>3527.6 mm^2

* Check strain for the magnitude of under strength factor Φ:

Tension = Compression

$A_s \times fy = 0.85 \times f_c' \times b \times a$

$3928 \times 420 = 0.85 \times 28 \times 1000 \times a$

$a = 69.32mm$

$X = \frac{a}{0.85} = \frac{69.32}{0.85} = 81.55mm$

$\varepsilon_s = \frac{435.5 - 81.55}{81.55} \times 0.003 = 0.013$

$\varepsilon_s = 0.013 > 0.005$

Ok…….

***Check for bar distance:**

$S = \frac{1000 - 2\times 40 - 2\times 12 - 8\times 25}{7} = 99.43mm > 25mm....ok$

- <u>Design of positive moment :</u>

***Take Mu = 477.1 KN.m at span (1&2).**

~ Assume bar diameter Φ25for main negative reinforcement.

d = 500-40-12-12.5=435.5 mm

m = $\frac{fy}{0.85 * fc'} = \frac{420}{0.85 * 28} = 17.65$

$$Kn = \frac{Mn}{bd^2} = \frac{477.1 * 10^6}{(0.9)(1000)(435.5)^2} = 2.8Mpa$$

$A_s \min = \frac{\sqrt{28}}{4(420)}(1000)(435.5) = 1371.7mm^2$

$A_s \min = \frac{1.4}{420}(1000)(435.5) = 1451.67mm^2$ ~ control

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mkn}{fy}}\right) = \frac{1}{17.65}\left(1 - \sqrt{1 - \frac{2 * 17.65 * 2.8}{420}}\right) = 0.0071$$

A_s = 0.0071 (1000) (435.5) =3092.1mm^2> A_s min = 1451.67mm^2

# of bars = A_s / $A_{s\,bar}$ = 3092.1/491 = 6.3 * Note $A_{\Phi 25}$ = 491 mm^2

Select bar 7 Φ 25

Total As $_{(provide)}$ = 3437mm^2>3092.1 mm^2

* Check strain for the magnitude of under strength factor Φ:

Tension = Compression

$$A_s \times fy = 0.85 \times f_c' \times b \times a$$

$$3437 \times 420 = 0.85 \times 28 \times 1000 \times a$$

$$a = 60.65mm$$

$$X = \frac{a}{0.85} = \frac{60.65}{0.85} = 71.35mm$$

$$\varepsilon_s = \frac{435.5 - 71.35}{71.35} \times 0.003 = 0.015$$

$$\varepsilon_s = 0.015 > 0.005$$

Ok…….

***Check for bar distance:**

$$S = \frac{1000 - 2 \times 40 - 2 \times 12 - 7 \times 25}{6} = 120.16mm > 25mm....ok$$

***Take Mu = 102 KN.m at span (3).**

~ Assume bar diameter Φ25for main negative reinforcement.

d = 500-40-12-12.5=435.5 mm

$$m = \frac{fy}{0.85 * fc'} = \frac{420}{0.85 * 28} = 17.65$$

$$Kn = \frac{Mn}{bd^2} = \frac{102 * 10^6}{(0.9)(1000)(435.5)^2} = 0.6Mpa$$

$$As\ \min = \frac{\sqrt{28}}{4(420)}(1000)(435.5) = 1371.7mm^2$$

$$As\ \min = \frac{1.4}{420}(1000)(435.5) = 1451.67mm^2 \text{ ~ control}$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mkn}{fy}}\right) = \frac{1}{17.65}\left(1 - \sqrt{1 - \frac{2 * 17.65 * 0.6}{420}}\right) = 0.00145$$

As = 0.00145 (1000) (435.5) =631.5mm^2< As min = 1451.67mm^2

# of bars = A_s / $A_{s\ bar}$ = 1451/314 = 2.96 * Note $A_{\Phi 25}$ = 314 mm^2

Select bar 5 Φ 20

Total As $_{(provide)}$ = 1570mm^2>1451.67 mm^2

* Check strain for the magnitude of under strength factor Φ:

Tension = Compression

$$A_s \times fy = 0.85 \times f_c' \times b \times a$$

$$1570 \times 420 = 0.85 \times 28 \times 1000 \times a$$

$$a = 27.71mm$$

$$X = \frac{a}{0.85} = \frac{27.71}{0.85} = 32.6mm$$

$$\varepsilon_s = \frac{435.5 - 32.6}{32.6} \times 0.003 = 0.037$$

$$\varepsilon_s = 0.037 > 0.005$$

Ok…….

***Check for bar distance:**

$$S = \frac{1000 - 2 \times 40 - 2 \times 12 - 5 \times 20}{4} = 199mm > 25mm....ok$$

***Take Mu = 636.1$_{\mathbf{KN.m}}$ at span (4).**

~ Assume bar diameter Φ25for main negative reinforcement.

d = 500-40-12-12.5=435.5 mm

m = $\frac{fy}{0.85 * fc'}$ = $\frac{420}{0.85 * 28}$ = 17.65

$$Kn = \frac{Mn}{bd^2} = \frac{636.1 * 10^6}{(0.9)(1000)(435.5)^2} = 3.73Mpa$$

A s min $= \frac{\sqrt{28}}{4(420)}(1000)(435.5) = 1371.7mm^2$

A s min $= \frac{1.4}{420}(1000)(435.5) = 1451.67mm^2$ ~ control

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2mkn}{fy}}\right) = \frac{1}{17.65}\left(1-\sqrt{1-\frac{2*17.65*3.73}{420}}\right) = 0.00971$$

65

A $_s$ = 0.00971 (1000) (435.5) =4228.71mm^2> A $_{s\ min}$ = 1451.67mm^2

# of bars = A_s / $A_{s\ bar}$ = 4228.71/491 = 8.61 * Note $A_{\Phi 25}$ = 491 mm^2

Select bar 9 Φ 25

Total $As_{(provide)}$ = 4419mm^2>4228.71 mm^2

* Check strain for the magnitude of under strength factor Φ:

Tension = Compression

$$A_s \times fy = 0.85 \times f_c' \times b \times a$$

$$4419 \times 420 = 0.85 \times 28 \times 1000 \times a$$

$$a = 77.98mm$$

$$X = \frac{a}{0.85} = \frac{77.98}{0.85} = 91.7mm$$

$$\varepsilon_s = \frac{435.5 - 91.7}{91.7} \times 0.003 = 0.0112$$

$$\varepsilon_s = 0.0112 > 0.005$$

Ok…….

***Check for bar distance:**

$$S = \frac{1000 - 2 \times 40 - 2 \times 12 - 9 \times 25}{8} = 83.9mm > 25mm\ldots ok$$

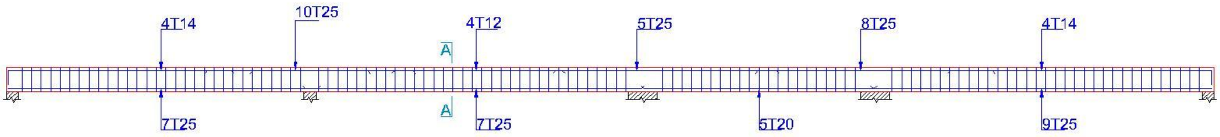


Figure (4.15) reinforcement for beam BG-26

4.3.8 Design of column G2

$f_c' = 28$ MPa

$F_y = 420\ MPa$

The Column is an interior one.

DL = 1200KN
LL = 450KN
Pu = 1.2DL+1.6LL

Pu = 1.2(1200)+1.6(450)

Pu = 2160KN

1- Check for slenderness:-

$$\frac{k\ ln}{r} \leq 34 - 12(\frac{M_1}{M_2}) \leq 40$$

*$\frac{M_1}{M_2} = 1$braced frame with M_{min}

K=1 for column in non-sway frames.

$$\frac{k\ ln}{r} \leq 34 - 12 = 22 \leq 40$$

$\frac{k\ ln}{r_x} = \frac{1*3.7}{0.3*0.6} = 20.55 < 22$ Column is short about x-axis

$\frac{k\ ln}{r_y} = \frac{1*3.7}{0.3*0.4}$ =30.83 $>$ 22 Column is long about y-axis

$Nominal\ axial\ strenght\ of\ column\ Pn = Pnx$

2- Calculate the minimum eccentricity $e_{min}$and the minimum moment M_{min}

ex_{min}= (15+0.03h) = 15+0.03*600 = 33 mm

$Pu_{factored}$ = 2160KN**.**

M_{min}= Pu*ex_{min}= 2160* $\frac{33}{1000}$ =71.28KN.

3- Compute EI

$$I_g = \frac{bh^3}{12} = \frac{400 * 600^3}{12} = 0.72 * 10^{10} mm^4$$

$$\beta_{dns} = \frac{1.2\ D\ (sustaind)}{1.2D + 1.6L} = 0.666$$

EI $= \frac{0.4*4750\sqrt{fc'}\ Ig}{1+\beta_{dns}} = \frac{0.4*4750\sqrt{28'}*0.72*10^{\wedge}10}{1+0.666} = 43.45 MN/m^2$

4- Determine the Euler buckling load, P_c:

$$P_c = \frac{\pi^2 EI}{(kl_n)^2} = \frac{\pi^2 * 43.45}{(1 * 3.7)^2} = 31.32 MN$$

5- Calculate the moment magnifier factor δ_{ns} :

$$C_m = 0.6 + 0.4 \frac{M_1}{M_2} = 0.6 + 0.4 * 1 = 1 > 0.4$$

$$\delta_{ns} = \frac{C_m}{1 - \frac{Pu}{0.75 P_c}} = \frac{1}{1 - \frac{2.160}{0.75*31.32}} = 1.101$$

1.4$> 1.101 > 1$……….ok

The magnified eccentricity and moment:

e $= e_{min} * \delta_{ns} = 33 * 1.101 = 36.333\ mm$

$$M_{ux} = \delta_{ns} * M_{ux} = 1.101 * 71.28 = 78.48\ KN.m$$

6- Select column reinforcement

We will use the tide column interaction diagrams

$$\frac{ey}{h} = \frac{36.333}{600} = 0.0605$$

Compute ratio γ

$$\gamma = \frac{d - d'}{h} = \frac{600 - 2*40 - 2*10 - 18}{600} = 0.803$$

$$\frac{\emptyset P_n}{A_g} = \frac{P_u}{A_g} = \frac{2160 * 10^{-3}}{0.6 * 0.4} = 9Mn/m\hat{}2$$

$\rho_g = 0.012$

∴select reinforcement

$A_{st} = \rho_g A_g = 0.012 * 600 * 400 = 2880mm^2$

$A_s \emptyset 18 = 254.47\ mm^2$

$$\frac{A_s}{A_s \emptyset 20} = 9.43$$

Use $12\emptyset 18 with A_s = 3052mm^2 > 2800mm^2$ ok

***Design of ties**

Use ties $\emptyset 10$ with spacing of ties shall not exceed

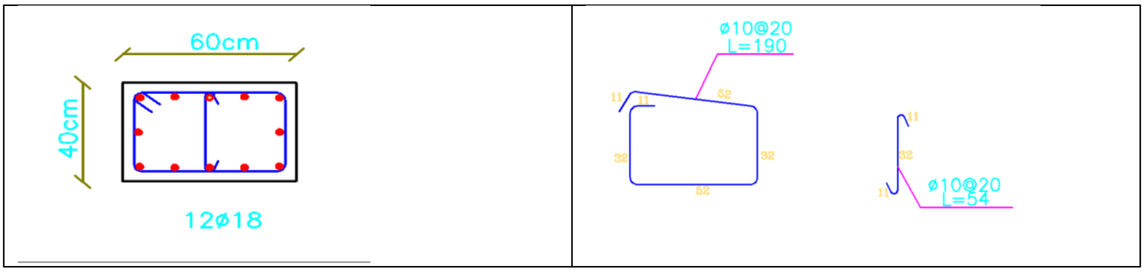
1) 48 times the tie diameter , $48d_s = 48 * 10 = 480mm$
2) 16 times the longitudinal bar diameter $16d_b = 16 * 18 = 288mm$ ……….. control
3) The lest dimension of column =40 mm

Use ties $\emptyset 10$@ 200 mm

1-Check for clear spacing between longitudinal bars

Clear spacing $= \frac{600-40*2-10*2-2*16}{3} = 154.7 > 40\ mm > 1.5 * 18 = 27$

80.8



(4.13) Design of footing:

Pu = 1146 KN .... $f_c' = 28\ Mpa$

Calculate the weight of footing, soil and the surcharge floor load

$h_{footing} = 45\ cm$

$W_{footing} = 0.45 * 25 = 11.25KN/\ m^2$

Weight soil = 0.45*18 = 8.1 $KN/\ m^2$

Total surcharge load on foundation

W=11.25+8.1+5 = 24.35 $KN/\ m^2$

- $\boldsymbol{net\ soil\ pressure, q_{a_{net}}}$:

$$q_{a_{net}} = 350 - 24.35 = 325.65\ KN/\ m^2$$

Pu = 1146 KN

$$q_u = \frac{1146}{2 * 1.8} = 318.33KN/m^2$$

$q_{bu} = 1.4 * q = 1.4 * 318.33 = 445.662 > q_{a_{net}}$ ….. ok

$\boldsymbol{calculate\ d}$:

$$V_u = q_u * b\left(\frac{a}{2} - \frac{l}{2} - d\right) = 445.662 * 1.8(\frac{2}{2} - \frac{0.55}{2} - d)$$

$V_u = 581.59 - 802.19\ d$

$V_c = \frac{1}{6} * \sqrt{f_c'} * bw * d\ = \frac{1}{6} * \sqrt{28} * 1800 * d = 1587.45\ d$

$\emptyset V_c = V_u$ ………………….$\emptyset = 0.75$

0.75*1587.45=1190.60

1190.60d = 518.59 -802.19 d

d =0.260m

assume cover 50mm and steel bar Ø14

h =260+75+14 =349mm

take h =450mm

d = 450-75-14 =361mm

- Punching shear

Let $V_u = \emptyset V_c$

$$V_u = 445.662(2 * 1.8 - 0.911 * 0.711)$$

$$V_u = 1315.72\ KN$$

B = $\frac{550}{350} = 1.57$

$b_\circ = 2 * 91.1 + 2 * 71.1$= 3.244m

$$\alpha_s = 40 - interior$$

V_c=$\frac{1}{6} * \left(1 + \frac{2}{B}\right)\sqrt{f_c'} * b_\circ * d$ $\quad$ $where\,\frac{1}{6} * \left(1 + \frac{2}{B}\right) = \frac{1}{6} * \left(1 + \frac{2}{1.57}\right) = .38$

$V_c = \frac{1}{12}\left(\frac{\alpha_s d}{b_\circ} + 2\right)\sqrt{f_c'} * b_\circ * d$ $\quad$ where $\frac{1}{12}\left(\frac{\alpha_s d}{b_\circ} + 2\right) = \frac{1}{12}\left(\frac{40*0.386}{3.244} + 2\right) = 0.54$

V_c=$\frac{1}{3} * \sqrt{f_c'} * b_\circ * d$ $\quad$ where $\frac{1}{3} = 0.333 \ldots\ldots\ldots\ldots control$

Take V_c=$\frac{1}{3} * \sqrt{f_c'} * b_\circ * d = \frac{1}{3} * \sqrt{28} * 3244 * 361 * 10^{-3} = 2065.6KN$

$\emptyset V_c = 0.75 * 2065.6 = 1549.2\ KN$

$$\emptyset V_c = 1549.2 > V_u = 1315.72KN \ldots\ldots\ldots ok$$

Design flexure in long direction

b =2m $\quad$ h =450mm $\quad$ d=450-75-14/2=368

$$M_u = 445.662 * 2 * 0.541 * 0.541/2 = 130.44 KN.m$$

$m = \frac{f_y}{0.85\, f_c'} = \frac{420}{0.85*28} = 17.65$.

$R_n = \frac{M_u}{\emptyset\, b*d^2} = \frac{130.44*10^6}{0.9*2000*(368)^2} = 0.535$MPa.

$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}})$

$= \frac{1}{17.65}\left(1 - \sqrt{1 - \frac{2*17.65*0.535}{420}}\right) = 0.0013$

$A_s = \rho * b * d = 0.0013 * 2000 * 368 = 956.8\ mm^2$.

$$As_{min} = 0.0018 * b * h = 0.0018 * 2000 * 450 = 1620\text{mm2}$$

$As_{min} = 1620\ mm^2 > As_{req} = 956.8 mm^2$.

∴ $As = 1620\ mm^2$.

$n = \frac{As_{req}}{A_{bar}\emptyset 14} = \frac{1620}{154} + 1 = 11.52$

∴ Use 12Φ14 with Aspro= 1848 mm^2.

$$S = \frac{2000 - 75 * 2 - 12 * 14}{11} = 153mm$$

Step S is the smallest of

1- 3h = 3*450=1350mm
2- 450………control

S = 153< $S_{max} = 450 \ldots\ldots\ldots ok$

Check of strain

$$a = \frac{A_s A_f}{0.85 f_c' b} = \frac{1848 * 420}{0.85 * 28 * 2000} = 16.3$$

$$c = \frac{a}{0.85} = \frac{16.3}{0.85} = 19.2$$

$$\epsilon = 0.003\left(\frac{d - c}{c}\right) = 0.003\left(\frac{361 - 19.2}{19.2}\right) = .0534 > 0.005 \ldots\ldots\ldots\ldots ok$$

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- Design flexure for short direction:

Take steel bare of Ø14

b =1800mm   h =450mm

d =450-75-14-$\frac{14}{2} = 354mm$

$$f_c' = 28\ MPa \qquad f_y = 420\ MPa$$

$$M_u = 445.662 * 1.8 * 0.548 * 0.548/2 = 120.45\ KN.m$$

$m = \frac{f_y}{0.85\ f_c'} = \frac{420}{0.85*28} = 17.65$.

$R_n = \frac{M_u}{\emptyset\ b*d^2} = \frac{120.45*10^6}{0.9*1800*(354)^2} = 0.593$MPa.

$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}})$

$= \frac{1}{17.65}\left(1 - \sqrt{1 - \frac{2*17.65*0.593}{420}}\right) = 0.00143$

$A_s = \rho * b * d = 0.00143 * 1800 * 354 = 911.65\ mm^2$.

$$As_{min} = 0.0018 * b * h = 0.0018 * 1800 * 450 = 1458\text{mm2}$$

$As_{min} = 1458\ mm^2 > As_{req} = 911.65\ mm^2$.

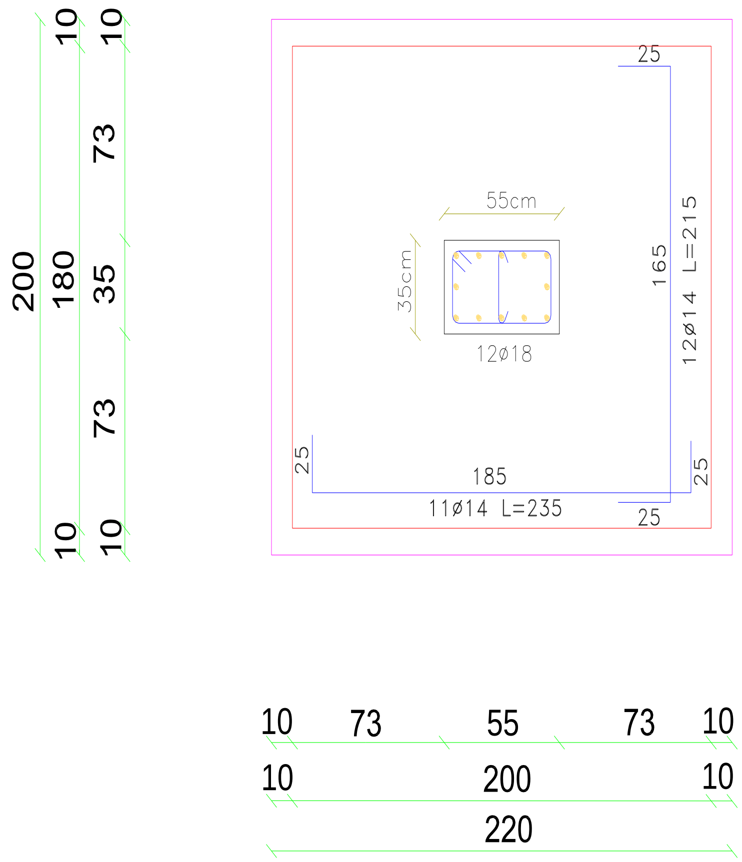
∴ As = 1458 mm^2.

n= $\frac{As_{req}}{A_{bar}\emptyset 14} = \frac{1458}{154}$ +1=10.47

∴ **Use 11Φ14   with As=1694** mm^2

$$S = \frac{1800 - 75 * 2 - 11 * 14}{10} = 149.6mm$$

S = 149.6< $S_{max} = 450 \ldots\ldots\ldots ok$


200
180
10
73
35
73
10
25
55cm
35cm
12ø18
165
12ø14 L=215
25
185
11ø14 L=235
25
10 73 55 73 10
10 200 10
220

Design of stairs :

Design of flight :

Calculation of load :

-	density kn/m2	Calculation kn/m
flight	25	25*.25/cos29.5= 7.81
plaster	22	22*.02/cos29.5= 0.51
horizontal mortar	22	22*.03*1= 0.66
horizontal tiles	23	23*.07*33/30 =0.76
vertical tiles	23	23*.07*17.5/30= 0.4
vertical mortar	22	22*0.03*17.5/30= 0.39
triangle	25	25*0.17/2= 2.1
		∑=12 kn/m

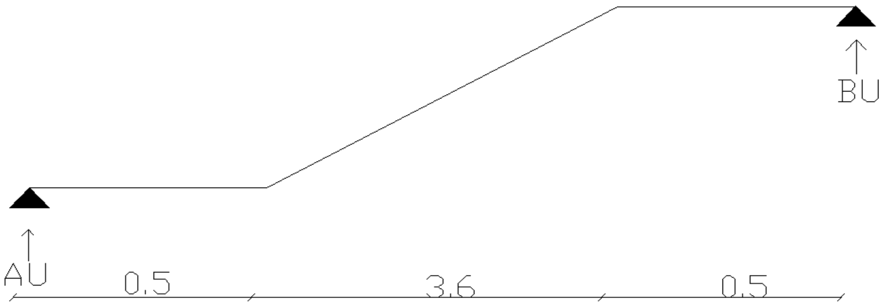
Calculation of h min:

h min: 4.1/20 =20.5 cm
choose h = 25 cm

Design of flight :

dead load =12 kn /m
live load = 5 kn /m

qu=1.2*DD+ 1.6*LL
qu= 1.2*12+ 1.6*5
qu=22.4 kn /m



AU= BU = 22.4*3.6/2 =49.3KN

Max Vu =40.3*cos 29.5 = 35 kn
Max mu =40.3*2.3 – 22.4*1.8*0.9 = 56.4kn.m

Design for shear :
Max vu =35kn

d = 250-20-12/2 = 224mm

$\emptyset V_c = 0.75 * \frac{1}{6} * 28^{.5} * 1000 * 224 = 148\ KN$

$\emptyset V_c \geq$ vu

No shear reinforcement required

Design moment :

Max Mu = 56.4 kn/m

$m = \frac{f_y}{0.85\ f_c'} = \frac{420}{0.85*28} = 17.65$

$R_n = \frac{M_u}{\emptyset\ b*d^2} = \frac{56.4*10^6}{0.9*1000*(224)^2} = 1.25\text{MPa}.$

$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}}) = 0.00305$

$A_s = \rho$ * b *d = 0.0.00305 * 100 * 22.4= 6.85 cm^2.

$$As_{min} = 0.0018 * b * h = 0.0018 * 100 * 25 = 4.5\ cm2$$

∴ As = 6.85cm^2.

Select 1Ø14 / 20cm

As =π/4 1.4^2 *100/2 = 7.6≥ As req

Check strain :

As *fy = 0.85 *fc * a * b

β 1=0.85

7.69*420= .85*28* a *100

a= 13.5mm

x = 13.5/0.85 = 15.96mm

strain

$$\left(\frac{d-x}{x}\right)0.003 = 0.039 > 0.005 \text{ ok.}$$

$\emptyset = 0.9$

Design of landing :

Calculation of dead load :

material	density kn/m2	calculation
concrete	25	25*025 =6.25
sand	16	16*0.07 =1.1
mortar	22	22*0.02 = 0.4
tiles	23	23*0.03 =0.7
plaster	22	22*.02 =0.4
		$\sum$ =8.85 kn/m

Dead load = 8.85 kn/m

Live load = 5 kn / m

Factoral load :

qu=1.2*DD+ 1.6*LL
qu= 1.2*8.85+ 1.6*5
qu=18.62 /m

Au=Bu from flight = 40.3

40.3+18.62 = 58.92 kn/m

Au = Bu= 165 KN

Analysis

MAX Vu =165-58.92*.349= 144.4 kn

MAX MU =(165*5.6/2) – (58.92*3.3*1.65)=223.6 KN. M

Design of shear:

MAX Vu = 144.4 kn

$$\emptyset V_c = 0.75 * \frac{1}{6} * 28^{.5} * 1000 * 224 = 148\ KN$$

No shear reinforcement required .

Design of moment :

MAX MU = 223.6 KN. M

$m = \frac{f_y}{0.85\ f_c'} = \frac{420}{0.85*28} = 17.65$

$R_n = \frac{M_u}{\emptyset\ b*d^2} = \frac{223.6*10^6}{0.9*1000*\ (224)^2} = 4.95$MPa.

$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}}) = 0.01336$

As =30 cm2

As>As min

Use 1∅18/ 10cm

Check strain :

As *fy = 0.85 *fc * a * b

β 1=0.85

300*420= .85*28* a *100

a= 52.9mm

x = 13.5/0.85 = 62.2 mm

strain :

$\left(\frac{d-x}{x}\right)0.003 = 0.0077 > 0.005$ ok.

$\emptyset = 0.9$