Chapter IV

Structural analysis and design

Contents:

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 high compressive strength but little tensile strength, and steel bars embedded in the concrete, which can provide the needed strength in tension.

Plain concrete is made by mixing cement, fine aggregate, coarse aggregate, water, and frequently admixtures. Understanding of reinforced concrete behavior is still far from complete, building codes and specifications 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/m3.
- Normal weight concrete with unit weight from about 1800 to 2400 kg/m3.
- Heavyweight concrete with unit weight from about 3200 to 5600 kg/m3.

4-2 Design Method and Requirements

The design strength provided by a member is calculated in accordance with the requirements and assumptions of ACI_code (318_08).

Strength design method:

In ultimate strength design method, the service loads are increased by factors to obtain the load at which failure is considered to be occurring, this load called factored load or factored service load, the structure or structural element is then proportioned such that the strength is reached when factored load is acting, the computation of this strength takes into account the nonlinear stress-strain behavior of concrete.

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

NOTE:

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

Code: ACI 2008 UBC

Material: Concrete B300

$Fcu = 30 N / mm^2 (MPa)$ For circular section.

But, for rectangular section ($fc' = 30 * 0.8 = 24MPa$).

Reinforcement steel:

The specified yield strength of the reinforcement {fy = 420 N/mm² (MPa)}.

Factored loads:

The factored loads for members in our project are determined by:

Wu = 1.2 DL + 1.6 LL ACI-code-318-08(9.2.1)

4-3 Check of Minimum Thickness of Structural Member

Minimum thickness (h)				
Member	Simply supported	One end continuous	Both end continuous	Cantilever
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 member.

For Rib :

hmin for(one end continuous)=L/18.5=6.16/18.5=33.3cm.

hmin for(both end continuous)=L/21=5.9/21=28.1cm.

hmin for(simply supported)=L/16=4.97/16=31cm.

Take h = 35 cm.

27 cm block + 8 cm topping = 35cm.

For Beam:

hmin for(one end continuous)=L/18.5=5.85/18.5=31.6cm.

hmin for(both end continuous)=L/21=5.95/21=28.3cm.

hmin for(cantilever)=L/8=2/8=25cm.

Take h = 35 cm, but in some regions we have a drop beam.

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.

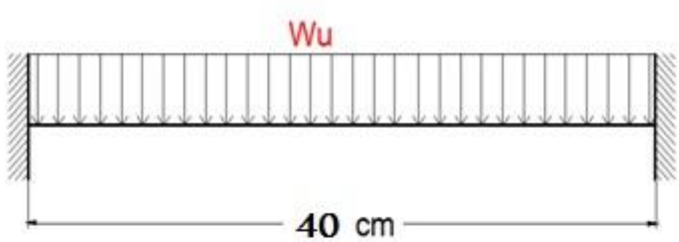

Figure 4-1: Topping load.

Load Calculations:

Dead Load:

No.	Parts of Rib	Calculation	
1	Tiles	0.03*23*1 = 0.69 KN/m	
2	Mortar	0.02*22*1 = 0.44 KN/m	
3	Coarse Sand	0.07*17*1 = 1.19 KN/m	
4	Topping	0.08*25*1 = 2.0 KN/m	
5	partions	KN/m1= 11*	
		Sum =	5.32KN/m

Table 4-2: Dead load calculation of topping.

Live Load:

LL =5 KN/m2.

LL =5 KN/m2×1m=5KN/m.

Factored Load :

WU = 1.2 ×5.32 + 1.6×5 =14.4 KN/m.

Check the strength condition for plain concrete, øMn ≥ Mu, where ø = 0.55.

Mn = 0.42 λ $\sqrt{f_c'}$ Sm (ACI 22.5.1, equation 22-2).

$S_m = \frac{b.h^2}{6} = \frac{1000.80^2}{6} = 1066666.67\ mm^2$.

øMn =0.55×0.42×1×$\sqrt{24}$ ×1066666.67 ×10^{-6} =1.21 KN.m

Mu = $\frac{W_u L^2}{12} = 0.192$ KN. m (negative moment)

Mu = $\frac{W_u L^2}{24} = 0.96\ \text{KN.m}$ (positive moment)

øMn>> Mu = $0.192\ \text{KN.m}$

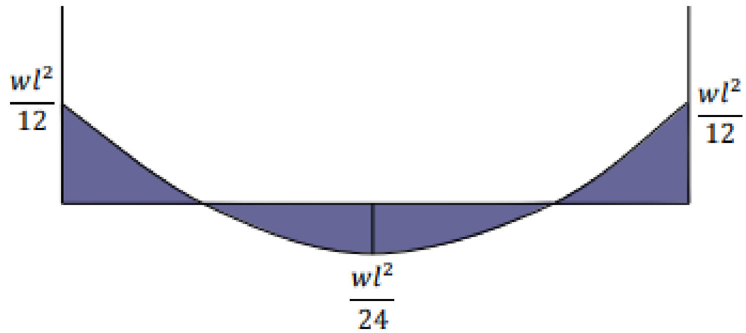

Figure 4-2: Moment diagram.

No reinforcement is required by analysis. According to ACI 10.5.4, provide As,min for slabs as shrinkage and temperature reinforcement.

ρshrinkage= 0.0018. ACI 7.12.2.1

As = ρ×b×htopping =0.0018 ×1000×80 = 144 mm2/m

Step (s) is the smallest of:

3h = 3×80 =240 mm control ACI 10.5.4 450mm.

S =$380\left(\frac{280}{f_s}\right) - 2.5C_c = 380\left(\frac{280}{\frac{2}{3}420}\right) - 2.5\ .20 = 330\text{mm}$ACI 10.6.4.

Take ø 8 @ 200 mm in both direction , S = 200 mm < Smax = 240 mm … OK

4-5 Design of One Way Rib Slab

Requirements For Ribbed Slab Floor According to ACI- (318-08) .

bw ≥ 10cm……………………………………………ACI(8.13.2)

Select bw=12 cm

h ≤ 3.5*bw ……………………………….…ACI(8.13.2)

Select h=35cm<3.5*12= 49 cm

tf ≥ Ln/12=600/12 ≥50mm …………………………………..ACI(8.13.6.1)

Select tf=8cm

Material :concrete B300 Fc' = 24 N/mm2

Reinforcement Steel fy = 420 N/mm2

Section :

B = 520 mm ,Bw= 120 mm ,h= 350 mm ,t= 80 mm , d=350-20-10-12/2= 314 m.

Statically System and Dimensions:

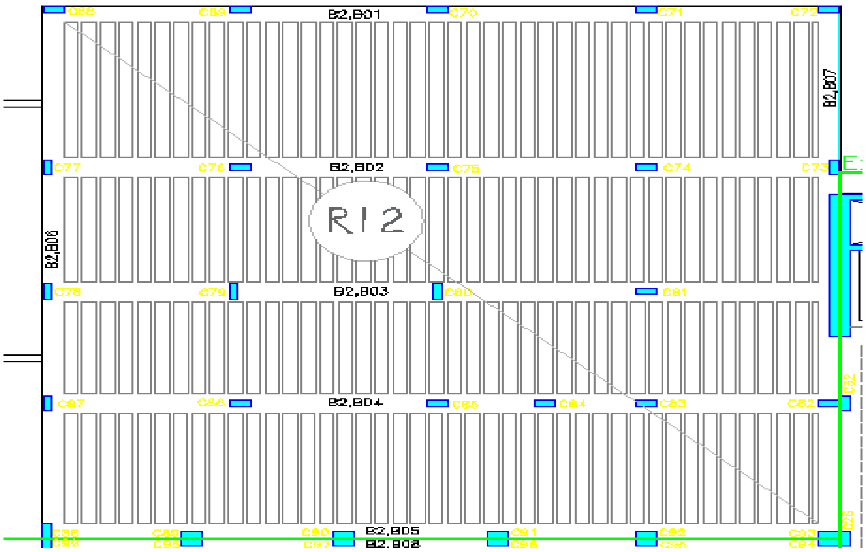

Figure 4-3: One way rib slab (R12).

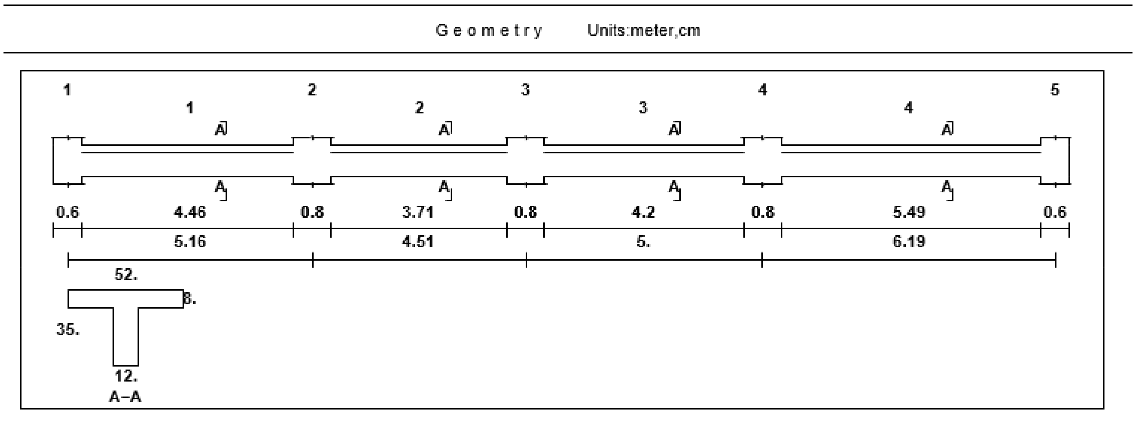

Figure 4-4: Geometry of rib slab (R12).

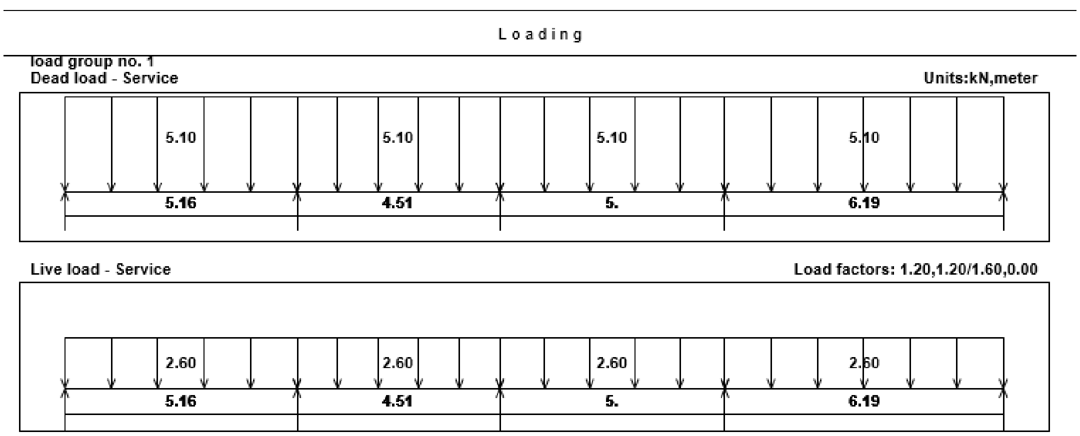

Figure 4-5: Statically system and loads distribution of rib (R12).

Load Calculation:

Dead Load:

No.	Parts of Rib	Calculation
1	Tiles	0.03*23*0.52 = 0.359 KN/m/rib
2	Mortar	0.03*22*0.52 = 0.229 KN/m/rib

3	Coarse Sand	0.07*17*0.52 = 0.620 KN/m/rib
4	Topping	0.08*25*0.52 = 1.04 KN/m/rib
5	RC. Rib	0.27*25*0.12 = 0.81 KN/m/rib
6	Hollow Block	0.27*10*0.4 = 1.08 KN/m/rib
7	plaster	0.02*22*.52= 0.229 KN/m/rib
8	partions	1*0.52= 0.52 KN/m/rib
		Sum = 5.1 KN/m/rib

Table 4-3: Dead Load Calculation of Rib(R12).

Dead Load /rib = 5.1 KN/m.

Live Load:

Live load = 5 KN/M2.

Live load /rib = 5 KN/m2 × 0.52m = 2.6 KN/m.

Effective Flange Width (b_E):-ACI-318-11 (8.10.2)

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

b_E = L / 4 = 619/ 4 =154.75cm

b_E = 12 + 16 t = 12 + 16 (8) = 140 cm

b_E = be ≤ center to center spacing between adjacent beams = 52 cm. Control

b_E For T-section = 52cm .

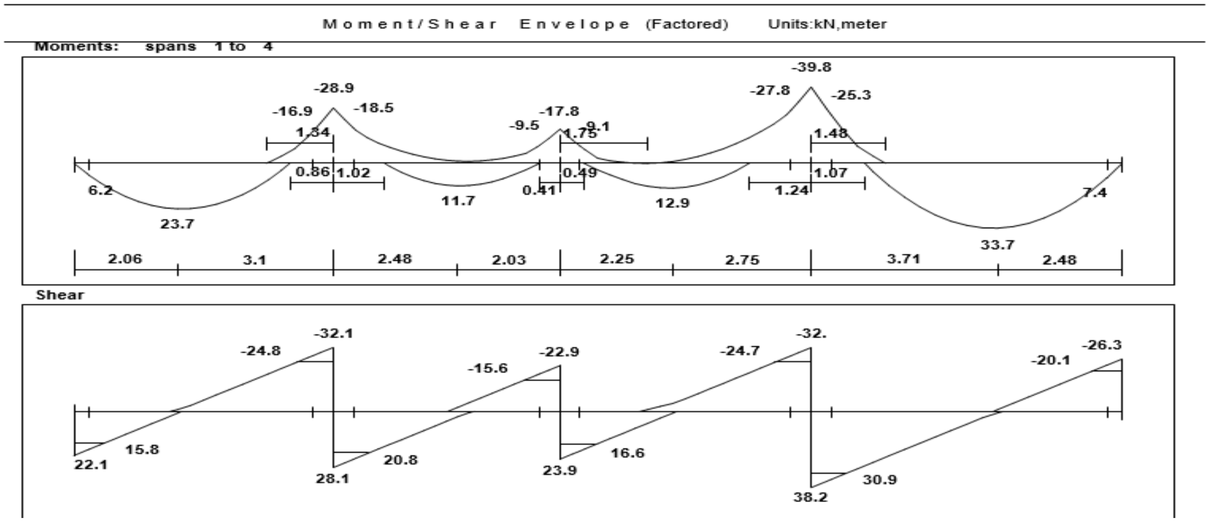

Figure 4-6: Shear and moment envelope diagram of rib (R12).

Moment Design for (R 12):

Design of Positive Moment for (Rib12):-(Mu=23.7 KN.m)

Assume bar diameter ø 12 for main positive reinforcement

d =h- cover - dstirrups$-\frac{d_b}{2} = 350 - 20 - 10 - \frac{12}{2} = 314$ mm

Check if a>hf to determine whether the section will act as rectangular or T- section.

Mnf $=0.85.f_c'.b_e.h_f.(d-\frac{h_f}{2})$

$= 0.85 \times 24 \times 520 \times 80 \times \left(314 - \frac{80}{2}\right) \times 10^{-6} = 232.5\ \text{KN.m}$

Mn≫$\frac{M_u}{\varphi} = \frac{23.7}{0.9}$= 26.33KN.m , the section will be designed as rectangular section withbe =520 mm.

Rn= $\frac{M_u}{\emptyset bd^2} = \frac{23.7\times10^6}{0.9\times520\times314^2} = 0.514$ Mpa

m= $\frac{f_y}{0.85f_c'} = \frac{420}{0.85\times24} = 20.6$

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times0.514}{420}}\right) = 0.00124$

As,req = ρ.b.d = 0.00124 ×520×314 = 202.5 mm2

Check for As min:

$A_s \min = \frac{\sqrt{fc'}}{4(fy)}(bw)(d)$ ACI-318 (10.5.1)

$A_s \min = \frac{\sqrt{24}}{4(420)}(120)(314) = 110mm^2$

$A_s \min = \frac{1.4}{(fy)}(bw)(d)$

$A_s \min = \frac{1.4}{420}(120)(314) = 125.6mm^2$ controls

Asreq= 202.5mm2 >Asmin= 125.6 mm2 OK

Use 2 ø 12 ,As,provided= 226 mm2>As,required= 202.5mm2 …. Ok

$S = \frac{120-40-20-(2\times12)}{1} = 36 \text{ mm} > d_b = 12 > 25 \text{ mm}$ OK

Check for strain:

$a = \frac{A_{s.}f_y}{0.85b\,f'_c} = \frac{226\times420}{0.85\times520\times24} = 8.94 \text{ mm}$

$x = \frac{a}{\mathcal{B}_1} = \frac{8.94}{0.85} = 10.53 \text{ mm}$

$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{314-10.53}{10.53}\right) = 0.0864 > 0.005$ 0k

Design of Positive Moment for(Rib12):- (Mu=11.7 KN.m) d =h- cover - dstirrups$-\frac{d_b}{2} = 350 - 20 - 10 - \frac{12}{2} = 314$ mm

Rn= $\frac{M_u}{\emptyset bd^2} = \frac{11.7\times10^6}{0.9\times520\times314^2} = 0.25$ Mpa

m= $\frac{f_y}{0.85f_c'} = \frac{420}{0.85\times24} = 20.6$

ρ= $\frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times0.25}{420}}\right) = 0.000599$

As,req = ρ.b.d = 0.000599×520×314 = 97.8 mm2

Check for As min:-

$A_s \min = \frac{\sqrt{fc'}}{4(fy)}(bw)(d)$ ACI-318 (10.5.1)

$A_s \min = \frac{\sqrt{24}}{4(420)}(120)(314) = 110mm^2$

$A_s \min = \frac{1.4}{(fy)}(bw)(d)$

$A_s \min = \frac{1.4}{420}(120)(314) = 125.6mm^2$ controls

As,required= 125.6 mm2.

Use 2 ø 10 ,As,provided= 157.08 mm2>As,required= 125.6 mm2 … Ok

$S = \frac{120-40-20-(2\times10)}{1} = 40 \text{ mm} > d_b = 10 > 25$ mm OK

Check for strain:

a =$\frac{A_{s.}f_y}{0.85b\, f_c'} = \frac{157\times420}{0.85\times520\times24} = 6.22$ mm

x= $\frac{a}{\mathcal{B}_1} = \frac{6.22}{0.85} = 7.31$mm

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{314-7.31}{7.31}\right) = 0.125 > 0.005 \qquad 0k$$

Design of Positive Moment for (Rib12):- (Mu=12.9 KN.m)

Assume bar diameter ø 12 for main positive reinforcement

d =h- cover - dstirrups$- \frac{d_b}{2} = 350 - 20 - 10 - \frac{12}{2} = 314$ mm

Rn= $\frac{M_u}{\emptyset bd^2} = \frac{12.9\times10^6}{0.9\times520\times314^2} = 0.28$ Mpa

m= $\frac{f_y}{0.85f'_c} = \frac{420}{0.85\times24} = 20.6$

ρ= $\frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times0.28}{420}}\right) = 0.000671$

As,req = ρ.b.d = 0.000671×520×314 = 109.56 mm2

Check for As min:

$A_s \min = \frac{\sqrt{fc'}}{4(fy)}(bw)(d)$ ACI-318 (10.5.1)

$A_s \min = \frac{\sqrt{24}}{4(420)}(120)(314) = 110 mm^2$

$A_s \min = \frac{1.4}{(fy)}(bw)(d)$

$A_s \min = \frac{1.4}{420}(120)(314) = 125.6 mm^2$ controls

Asreq = 125.6 mm2

Use 2 ø10 ,As,provided=157.08 mm2>As,required= 125.6 mm2… Ok

S = $\frac{120-40-20-(2\times10)}{1} = 40 \text{ mm} > d_b = 10 > 25$ mm OK

Check for strain:

$a = \frac{A_{s}.f_y}{0.85b\ f'_c} = \frac{157\times420}{0.85\times520\times24} = 6.22$ mm

$x = \frac{a}{\mathcal{B}_1} = \frac{6.22}{0.85} = 7.31$ mm

$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{314-7.31}{7.31}\right) = 0.125 > 0.005$ 0k

Design of Positive Moment for (Rib12):- (Mu=33.7 KN.m)

Assume bar diameter ø 12 for main positive reinforcement

d =h- cover - dstirrups$-\frac{d_b}{2} = 350-20-10-\frac{12}{2} = 314$ mm

$Rn = \frac{M_u}{\emptyset bd^2} = \frac{33.7\times10^6}{0.9\times520\times314^2} = 0.73$ Mpa

$m = \frac{f_y}{0.85f'_c} = \frac{420}{0.85\times24} = 20.6$

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times0.73}{420}}\right) = 0.00177$

As,req = ρ.b.d = 0.00177×520×314 = 289 mm2

Check for As min:

$A_s\ \min = \frac{\sqrt{fc'}}{4(fy)}(bw)(d)$ ACI-318 (10.5.1)

$A_s\ \min = \frac{\sqrt{24}}{4(420)}(120)(314) = 110mm^2$

$A_s\ \min = \frac{1.4}{(fy)}(bw)(d)$

$A_s\ \min = \frac{1.4}{420}(120)(314) = 125.6mm^2$ controls

Asreq= 289 mm2 >Asmin= 125.6 mm2 OK

Use 2 ø14 ,As,provided=308 mm2>As,required= 289 mm2… Ok

$S = \frac{120-40-20-(2\times14)}{1} = 32 \text{ mm} > d_b = 14 > 25 \text{ mm}$ OK

Check for strain:

$a = \frac{A_{s.}f_y}{0.85b\,f'_c} = \frac{308\times420}{0.85\times520\times24} = 12.2 \text{ mm}$

$x = \frac{a}{\mathcal{B}_1} = \frac{12.2}{0.85} = 14.35 \text{ mm}$

$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{314-14.35}{14.35}\right) = 0.0626 > 0.005$ 0k

Design of Negative Moment for (Rib12):- (Mu= -16.9 KN.m)

Assume bar diameter ø 12 for main positive reinforcement

$d = h\text{- cover - dstirrups} - \frac{d_b}{2} = 350 - 20 - 10 - \frac{12}{2} = 314 \text{ mm}$

$Rn = \frac{M_u}{\emptyset bd^2} = \frac{16.9\times10^6}{0.9\times120\times314^2} = 1.59 \text{ Mpa}$

$m = \frac{f_y}{0.85f'_c} = \frac{420}{0.85\times24} = 20.6$

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times1.59}{420}}\right) = 0.00395$

As,req = ρ.b.d = 0.00395×120×314 = 148.8 mm2

Check for As min:

$A_s \text{ min} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d)$ ACI-318 (10.5.1)

$A_s \text{ min} = \frac{\sqrt{24}}{4(420)}(120)(314) = 110mm^2$

$A_s \min = \frac{1.4}{(fy)}(bw)(d)$

$A_s \min = \frac{1.4}{420}(120)(314) = 125.6mm^2$ controls

Asreq = 148.8mm2 >Asmin= 125.6 mm2OK

Use 2 ø 10 ,As,provided= 157.1 mm2>As,required= 148.8 mm2… Ok

$S = \frac{120-40-20-(2\times10)}{1} = 40 \text{ mm} > d_b = 10 > 25 \text{ mm}$ OK

Check for strain:-

$a = \frac{A_{s.}f_y}{0.85b\, f_c'} = \frac{157.1\times420}{0.85\times120\times24} = 26.95 \text{ mm}$

$x = \frac{a}{\mathcal{B}_1} = \frac{26.95}{0.85} = 31.7 \text{ mm}$

$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{314-31.7}{31.7}\right) = 0.0267 > 0.005$ 0k

Design of Negative Moment for (Rib12):- (Mu= -18.5 KN.m)

Assume bar diameter ø 12 for main positive reinforcement

d =h- cover - dstirrups$-\frac{d_b}{2} = 350 - 20 - 10 - \frac{12}{2} = 314 \text{ mm}$

$Rn = \frac{M_u}{\emptyset bd^2} = \frac{18.5\times10^6}{0.9\times120\times314^2} = 1.74 \text{ Mpa}$

$m = \frac{f_y}{0.85f_c'} = \frac{420}{0.85\times24} = 20.6$

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times1.74}{420}}\right) = 0.00434$

As,req = ρ.b.d = 0.00434×120×314 = 163.5 mm2

Check for As min:-

$A_s \min = \frac{\sqrt{fc'}}{4(fy)}(bw)(d)$ ACI-318 (10.5.1)

$A_s \min = \frac{\sqrt{24}}{4(420)}(120)(314) = 110mm^2$

$A_s \min = \frac{1.4}{(fy)}(bw)(d)$

$A_s \min = \frac{1.4}{420}(120)(314) = 125.6mm^2$ controls

Asreq = 163.5mm2 >Asmin= 125.6 mm2OK

Use 2 ø 12 ,As,provided= 226 mm2>As,required= 163.5 mm2… Ok

$S = \frac{120-40-20-(2\times12)}{1} = 36 \text{ mm} > d_b = 12 > 25 \text{ mm}$ OK

Check for strain:-

$a = \frac{A_{s.}f_y}{0.85b\, f_c'} = \frac{226\times420}{0.85\times120\times24} = 38.77 \text{ mm}$

$x = \frac{a}{\mathcal{B}_1} = \frac{38.77}{0.85} = 45.62 \text{ mm}$

$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{314-45.62}{45.62}\right) = 0.0176 > 0.005$ 0k

Design of Negative Moment for (Rib12):- (Mu=-9.5 KN.m)

Assume bar diameter ø 12 for main positive reinforcement

d =h- cover - dstirrups$-\frac{d_b}{2} = 350 - 20 - 10 - \frac{12}{2} = 314 \text{ mm}$

$Rn = \frac{M_u}{\emptyset bd^2} = \frac{9.5\times10^6}{0.9\times120\times314^2} = 0.892 \text{ Mpa}$

$m = \frac{f_y}{0.85f_c'} = \frac{420}{0.85\times24} = 20.6$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times0.892}{420}}\right) = 0.00217$$

As,req = ρ.b.d = 0.00217 ×120×314 = 81.76 mm2

Check for As min:

$$A_s \min = \frac{\sqrt{fc'}}{4(fy)}(bw)(d)$$ ACI-318 (10.5.1)

$$A_s \min = \frac{\sqrt{24}}{4(420)}(120)(314) = 110m^2$$

$$A_s \min = \frac{1.4}{(fy)}(bw)(d)$$

$$A_s \min = \frac{1.4}{420}(120)(314) = 125.6mm^2$$ controls

Asreq = 125.6 mm2

Use 2 ø10 ,As,provided= 157.08 mm2>As,required= 125.6 mm2… Ok

$$S = \frac{120-40-20-(2\times10)}{1} = 40 \text{ mm} > d_b = 10 > 25 \text{ mm}$$ OK

Check for strain:

$$a = \frac{A_{s.fy}}{0.85b\, f_c'} = \frac{157\times420}{0.85\times120\times24} = 6.22 \text{ mm}$$

$$x = \frac{a}{\mathcal{B}_1} = \frac{6.22}{0.85} = 7.31\text{mm}$$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{284-7.31}{7.31}\right) = 0.125 > 0.005$$ 0k

Design of Negative Moment for (Rib12): (Mu=-9.1 KN.m)

Assume bar diameter ø 12 for main positive reinforcement

d =h- cover - dstirrups$-\frac{d_b}{2} = 350 - 20 - 10 - \frac{12}{2} = 314$ mm

Rn$= \frac{M_u}{\emptyset b d^2} = \frac{9.1\times10^6}{0.9\times120\times314^2} = 0.855$ Mpa

m$= \frac{f_y}{0.85 f_c'} = \frac{420}{0.85\times24} = 20.6$

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times0.855}{420}}\right) = 0.00208$

As,req = ρ.b.d = 0.00208×120×314 = 78.37 mm2

Check for As min:

$A_s \min = \frac{\sqrt{fc'}}{4(fy)}(bw)(d)$ ACI-318 (10.5.1)

$A_s \min = \frac{\sqrt{24}}{4(420)}(120)(314) = 110m^2$

$A_s \min = \frac{1.4}{(fy)}(bw)(d)$

$A_s \min = \frac{1.4}{420}(120)(314) = 125.6mm^2$ controls

Asreq = 125.6 mm2

Use 2 ø10 ,As,provided= 157.08 mm2>As,required= 125.6 mm2… Ok .

$S = \frac{120-40-20-(2\times10)}{1} = 40 \text{ mm} > d_b = 10 > 25 \text{ mm}$ OK

Check for strain:

a $=\frac{A_{s.}f_y}{0.85 b\, f_c'} = \frac{157\times420}{0.85\times120\times24} = 6.22$ mm x= $\frac{a}{\mathcal{B}_1} = \frac{6.22}{0.85} = 7.31$mm

$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{284-7.31}{7.31}\right) = 0.125 > 0.005$ 0k

Design of Negative Moment for (Rib12): (Mu=-27.8 KN.m)

Assume bar diameter ø 12 for main positive reinforcement

d =h- cover - dstirrups$-\frac{d_b}{2} = 350 - 20 - 10 - \frac{12}{2} = 314$ mm

Rn= $\frac{M_u}{\emptyset b d^2} = \frac{27.8\times10^6}{0.9\times120\times314^2} = 2.61$ Mpa

m= $\frac{f_y}{0.85 f'_c} = \frac{420}{0.85\times24} = 20.6$

ρ= $\frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times2.61}{420}}\right) = 0.00667$

As,req = ρ.b.d = 0.00667 ×120×314 = 251.3 mm2

Check for As min:

$A_s \min = \frac{\sqrt{fc'}}{4(fy)}(bw)(d)$ ACI-318 (10.5.1)

$A_s \min = \frac{\sqrt{24}}{4(420)}(120)(314) = 110m^2$

$A_s \min = \frac{1.4}{(fy)}(bw)(d)$

$A_s \min = \frac{1.4}{420}(120)(314) = 125.6mm^2$ controls

Asreq = 251.3 mm2 >Asmin= 125.6 mm2OK

Use 2 ø14 ,As,provided= 308 mm2>As,required= 251.3mm2… Ok

$S = \frac{120-40-20-(2\times14)}{1} = 36 \text{ mm} > d_b = 14 > 25 \text{ mm}$ OK

Check for strain:

$a = \frac{A_{s.}f_y}{0.85b\, f'_c} = \frac{308\times420}{0.85\times120\times24} = 12.2 \text{ mm}$

$x = \frac{a}{\mathcal{B}_1} = \frac{12.2}{0.85} = 14.35 \text{ mm}$

$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{284-14.35}{14.35}\right) = 0.0626 > 0.005$ 0k

Design of Negative Moment for (Rib12): (Mu=-25.3KN.m)

Assume bar diameter ø 12 for main positive reinforcement

d =h- cover - dstirrups$- \frac{d_b}{2} = 350 - 20 - 10 - \frac{12}{2} = 314 \text{ mm}$

$Rn = \frac{M_u}{\emptyset bd^2} = \frac{25.3\times10^6}{0.9\times120\times314^2} = 2.38 \text{ Mpa}$

$m = \frac{f_y}{0.85f'_c} = \frac{420}{0.85\times24} = 20.6$

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times2.38}{420}}\right) = 0.00604$

As,req = ρ.b.d = 0.00604 ×120×314 = 227.59 mm2

Check for As min:-

$A_s \min = \frac{\sqrt{fc'}}{4(fy)}(bw)(d)$ ACI-318 (10.5.1)

$A_s \min = \frac{\sqrt{24}}{4(420)}(120)(314) = 110m^2$

$A_s \min = \frac{1.4}{(fy)}(bw)(d)$

A_s min $= \frac{1.4}{420}(120)(314) = 125.6 mm^2$ controls

Asreq = 227.59 mm2 >Asmin= 125.6 mm2OK

Use 2 ø14 ,As,provided= 308 mm2>As,required= 227.59 mm2… Ok

$S = \frac{120-40-20-(2\times14)}{1} = 36 \text{ mm} > d_b = 14 > 25 \text{ mm}$ OK

Check for strain:-

$a = \frac{A_{s.}f_y}{0.85 b\, f'_c} = \frac{308\times420}{0.85\times120\times24} = 12.2 \text{ mm}$

$x = \frac{a}{\mathcal{B}_1} = \frac{12.2}{0.85} = 14.35 \text{ mm}$

$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{284-14.35}{14.35}\right) = 0.0626 > 0.005$ 0k

Shear Design for (R 12):

Vu at distance d from support= 30.9 KN

Shear strength Vc, provided by concrete for the joists may be taken 10% greater than for beams. This is mainly due to the interaction between the slab and closely spaced ribs.(ACI, 8.13.8).

$Vc = \frac{1.1}{6}\sqrt{f'_c} b_w d = \frac{1.1}{6}\sqrt{24} \times 120 \times 314 \times 10^{-3} = 33.84 \text{ KN}$

øVc =0.75×33.84 =25.38 KN

0.5 ø Vc =0.5×25.38 =12.69 KN

0.5 ø Vc< Vu< ø Vc

Vu> ø Vc

For shear design, shear reinforcement is required ($A_{v,}$), Vsmin =$\frac{1}{16}\sqrt{f_c'}$bw d $\geq \frac{1}{3}$ bw d

Vs min=$\frac{1}{16}\sqrt{24} * 120 * 314 = 11.54$kn

Vsmin =$\frac{1}{3}$ bw d=$\frac{1}{3} * 120 * 314 = 12.56$kn

ø(Vc+Vsmin)= 0.75(33.84+12.56)=34.8kn

øVc<Vu <ø (Vc+Vsmin)

25.38 < 30.9 <34.8

For shear design, minimum shear reinforcement is required ($A_{v,min}$), Reinforcement.

Use stirrups (2 leg stirrups) ø 8@150 mm , Av = 2 × 50.24 = 100.5 mm2

Avmin =$\frac{1}{16}\sqrt{f_c'}\frac{b_w s}{fyt} \geq \frac{1}{3}\frac{b_w s}{fyt}$

Avmin=100.5 =$\frac{1}{16}\sqrt{24}\frac{120s}{420} \rightarrow s = 1.145$m

100.5 =$\frac{1}{3}\frac{120s}{420} \rightarrow s = 1.055$m

S max→$\frac{d}{2} = 157$mm

S max →≤600mm

Take (2 leg stirrups) ø 8 @ 150 mm

$A_v = \frac{2*50.3}{0.15} = 670.67$ mm2/mstrip

Vu at distance d from support= 24.7 KN

Shear strength Vc, provided by concrete for the joists may be taken 10% greater than for beams. This is mainly due to the interaction between the slab and closely spaced ribs.(ACI, 8.13.8).

Vc =$\frac{1.1}{6}\sqrt{f_c'}b_w d = \frac{1.1}{6}\sqrt{24} \times 120 \times 314 \times 10^{-3} = 33.84$ KN øVc =0.75×33.84 =25.38 KN

0.5 ø Vc =0.5×25.38 =12.69 KN

Vu=24.7< 0.5 ø Vc =12.69 no

0.5 ø Vc< Vu< ø Vc

12.69< 24.7 < 33.84 ok.

Minimum shear reinforcement is required except for concrete joist construction.

So, no shear required reinforcement is provided.

Vu at distance d from support= 20.8 KN

Shear strength Vc, provided by concrete for the joists may be taken 10% greater than for beams. This is mainly due to the interaction between the slab and closely spaced ribs.(ACI, 8.13.8).

Vc $=\frac{1.1}{6}\sqrt{f_c'}b_w d = \frac{1.1}{6}\sqrt{24} \times 120 \times 314 \times 10^{-3} = 33.84$ KN

øVc =0.75×33.84 =25.38 KN

0.5 ø Vc =0.5×25.38 =12.69 KN

Vu=20.8< 0.5 ø Vc =12.69 no

0.5 ø Vc< Vu< ø Vc

12.69< 20.8 < 33.84 ok.

Minimum shear reinforcement is required except for concrete joist construction.

So, no shear required reinforcement is provided.

Vu at distance d from support= 24.8KN

Shear strength Vc, provided by concrete for the joists may be taken 10% greater than for beams. This is mainly due to the interaction between the slab and closely spaced ribs.(ACI, 8.13.8).

Vc $=\frac{1.1}{6}\sqrt{f_c'}b_w d = \frac{1.1}{6}\sqrt{24} \times 120 \times 314 \times 10^{-3} = 33.84$ KN

øVc =0.75×33.84 =25.38 KN

0.5 ø Vc =0.5×25.38 =12.69 KN

Vu=24.8< 0.5 ø Vc =12.69 no

0.5 ø Vc< Vu< ø Vc

12.69< 24.8 < 33.84 ok.

Minimum shear reinforcement is required except for concrete joist construction.

So, no shear required reinforcement is provided.

So we take for all rib the maximum case:

Take (2 leg stirrups) ø 8 @ 150 mm

$A_v = \frac{2*50.3}{0.15} = 670.67$ mm2/mstrip

4-6 Design of Beam

Material : concrete B300 Fc' = 24 N/mm2

Reinforcement Steel fy = 420 N/mm2

Section :

B = 60 cm

h= 35cm

d=350-40-10-18/2= 291 mm

Statically System and Dimensions:

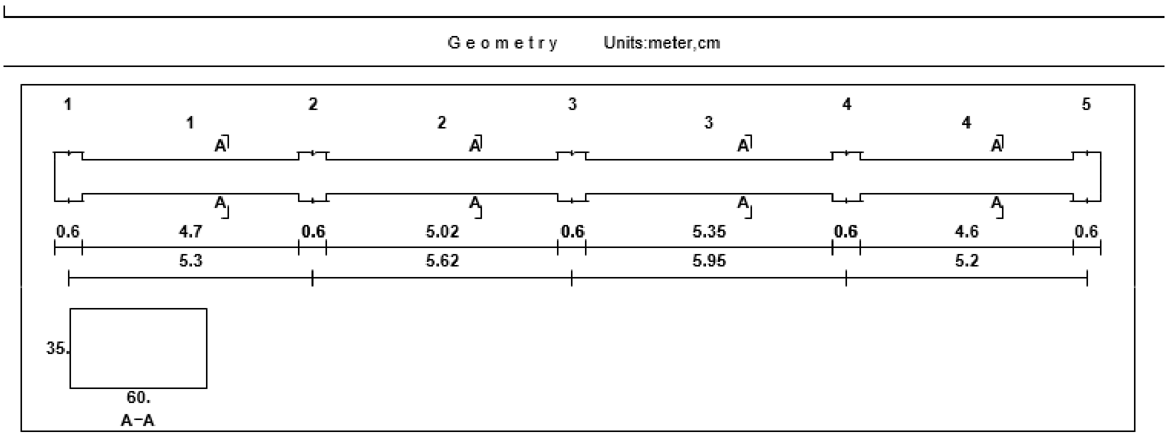

Figure 4-7: Geometry of beam (B1).

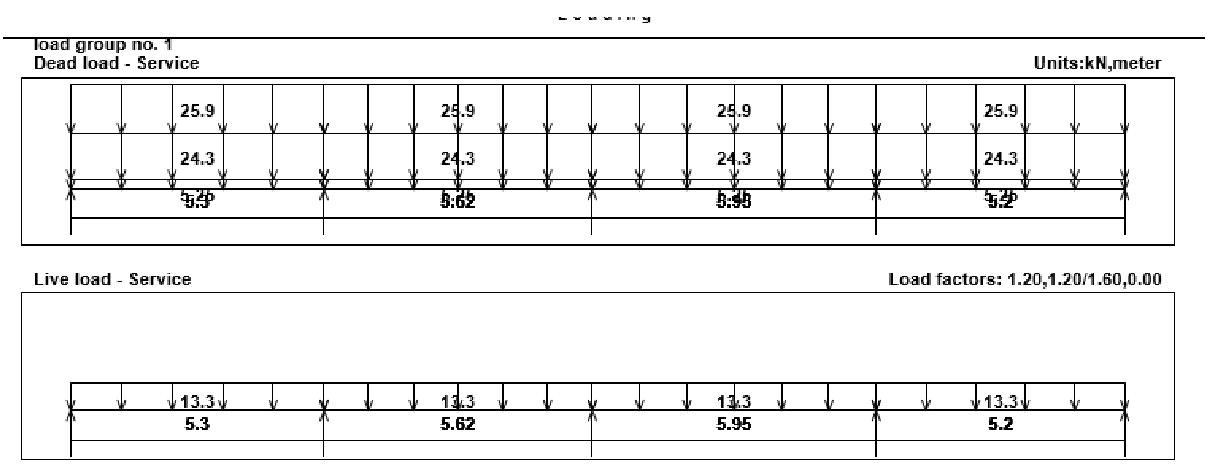

Figure 4-8: Statically system and loads distribution of beam (B 1).

Load Calculations:

Dead Load Calculations for Beam (B1):

The distributed Dead and Live loads acting upon B1 can be defined from the support reactions of the R12.

From Rib12

The maximum support reaction from Dead Loads for R12 upon B1 is7.54 KN, The distributed Dead Load from the R12 on B1.

DL =(12.68/ 0.52) = 24.38 KN / m

Self-weight of beam = 0.35*0.6*25 = 5.25 KN / m

DL =24.38+5.25 = 18.83 KN / m

Dead Load from External wall

D = 3.45*0.3 *25 = 25.9 Kn/m

Live Load calculations for Beam (B1):

From Rib12

The maximum support reaction from Live Loads for R12upon B 1 is 6.69 KN The distributed Live Load from the Rib 12 on B1.

LL =6.69/ 0.52= 13.38 KN/m.

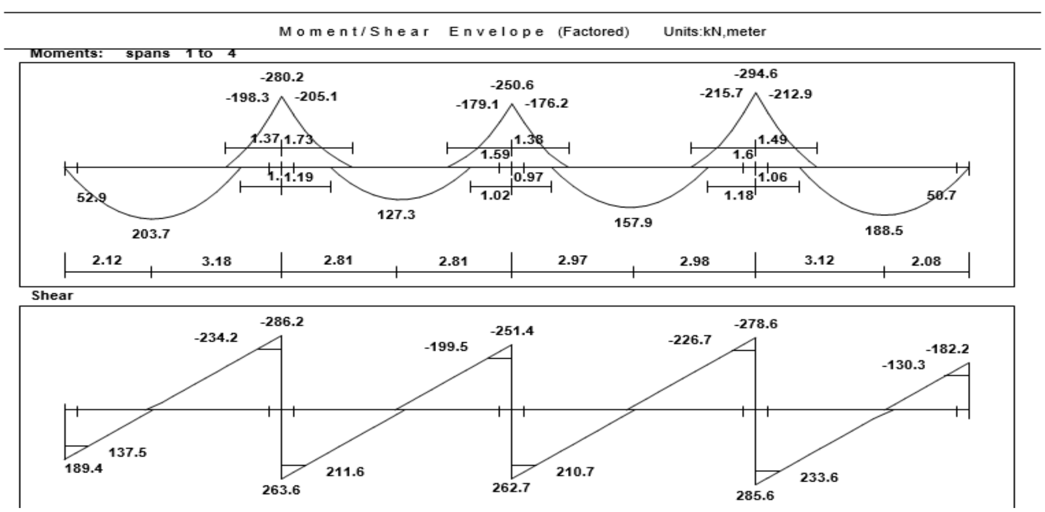

Figure 4-9: Shear and moment envelope diagram of beam (B1).

Moment Design for (B11):

Flexural Design of Positive Moment for(B1):-(Mu=203.7 KN.m)

Determine of Mn,max

d =350 – 40 -10 – 18\2 = 291 mm

$x = \frac{3}{7} d = \frac{3}{7}.291 = 124.7$ mm

$a = \mathcal{B}.x = 124.7 * 0.85 = 106$ mm

Mnmax= $0.85 * f_c' * a * b(d - \frac{a}{2})$ = 0.85*24*106*600*(291-106/2) *10-6= 308.8 KN.m

∅ Mnmax = 0.82* 308.8 = 253.22 KN.m >203.7 KN.m .

Design as singly reinforcement

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{203.7 \times 10^6}{0.9 \times 600 \times 291^2} = 4.45 Mpa$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 \times 24} = 20.6$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 4.45}{420}}\right) = 0.0121$$

As = ρ.b.d = 0.0121×600×291 = 2112.66 mm2

Check for As,min:

$$Asmin = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) = \frac{\sqrt{24}}{4 * 420} * 600 * 291 = 509.14 \text{ mm2}$$

$$Asmin = \frac{1.4}{(fy)}(bw)(d) = \frac{1.4}{420} * 600 * 291 = 582 \text{ mm2}$$ Controls

As= 2112.66 mm2

Use 7ø 20 Bottom, As,provided= 2200 mm2>As,required= 2112.66 mm2… Ok

Check spacing :

$$S = \frac{600-40*2-20-(7\times20)}{6} = 60 \text{ mm} > d_b = 20 > 25 \text{ mm} \quad \text{OK}$$

Check for strain:-

$$a = \frac{A_{s.}f_y}{0.85b\, f'_c} = \frac{2200\times420}{0.85\times600\times24} = 75.5 \text{ mm}$$

$$x = \frac{a}{\mathcal{B}_1} = \frac{75.5}{0.85} = 88.8 \text{ mm}$$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{291-88.8}{88.8}\right) = 0.00683 > 0.005 \qquad 0k$$

Flexural Design of Positive Moment for (B1):-(Mu=127.3 KN.m)

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{127.3\times10^6}{0.9\times600\times291^2} = 2.78\text{Mpa.}$$

$$m = \frac{f_y}{0.85f'_c} = \frac{420}{0.85\times24} = 20.6$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times2.78}{420}}\right) = 0.00714$$

As = ρ.b.d = 0.00714×600×291 = 1246.6 mm2.

Check for As,min:

$$\text{Asmin} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) = \frac{\sqrt{24}}{4*420}*600*291 = 509.14 \text{ mm2}$$

$$\text{Asmin} = \frac{1.4}{(fy)}(bw)(d) = \frac{1.4}{420}*600*291 = 582 \text{ mm2Controls}$$

As= 1246.6 mm2.

Use 5ø18Bottom, As,provided= 1272.35 mm2>As,required= 1246.6 mm2 … Ok

Check spacing :

$$S = \frac{600-40*2-20-(5*18)}{4} = 102.5\text{mm} > d_b = 18 > 25 \qquad \text{OK}$$

Check for strain:-

$$a = \frac{A_{s.f_y}}{0.85b\, f_c'} = \frac{1272.35 \times 420}{0.85 \times 600 \times 24} = 43.66 \text{ mm}$$

$$x = \frac{a}{\mathcal{B}_1} = \frac{43.66}{0.85} = 51.36 \text{ mm}$$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{291-51.36}{51.36}\right) = 0.01399 > 0.005 \qquad 0k$$

Flexural Design of Positive Moment for(B1):-(Mu=157.9 KN.m)

$$Rn = \frac{M_u}{\emptyset b d^2} = \frac{157.9 \times 10^6}{0.9 \times 600 \times 291^2} = 3.45 \text{ Mpa}$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 \times 24} = 20.6$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times 20.6\times 3.45}{420}}\right) = 0.00906$$

As = ρ.b.d = 0.00906 ×600×291 = 1581.9 mm2

Check for As,min:

$$Asmin = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) = \frac{\sqrt{24}}{4*420}*600*291 = 509.14 \text{ mm2}$$

$$Asmin = \frac{1.4}{(fy)}(bw)(d) = \frac{1.4}{420}*600*291 = 582 \text{ mm2}$$

As = 1581.9 mm2Controls

Use 7ø 18, As,provided= 1781.3 mm2>As,required= 1581.9 mm2… Ok

Check spacing:

$$S = \frac{600-40*2-20-(7*18)}{6} = 62.33 \text{ mm} > d_b = 18 > 25 \qquad \text{OK}$$

Check for strain:

$$a = \frac{A_{s.f_y}}{0.85b\, f'_c} = \frac{1781.3 \times 420}{0.85 \times 600 \times 24} = 61.12 \text{ mm}$$

$$x = \frac{a}{\mathcal{B}_1} = \frac{61.12}{0.85} = 71.9\text{mm}$$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{291-71.9}{71.9}\right) = 0.00914 > 0.005 \qquad 0k$$

Flexural Design of Positive Moment for(B1):-(Mu=188.5 KN.m)

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{188.5 \times 10^6}{0.9 \times 600 \times 291^2} = 4.12 \text{ Mpa}$$

$$m = \frac{f_y}{0.85f'_c} = \frac{420}{0.85 \times 24} = 20.6$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2 \times 20.6 \times 4.12}{420}}\right) = 0.0111$$

As = ρ.b.d = 0.0111 ×600×291 = 1938.1 mm2

Check for As,min:

$$\text{Asmin} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) = \frac{\sqrt{24}}{4*420}*600*291 = 509.14 \text{ mm2}$$

$$\text{Asmin} = \frac{1.4}{(fy)}(bw)(d) = \frac{1.4}{420}*600*291 = 582 \text{ mm2}$$

As = 1938.1mm2Controls

Use 8 ø 18 ,As,provided= 2035.75 mm2>As,required= 1938.1 mm2… Ok

Check spacing :

$$S = \frac{600-40*2-20-(8*18)}{7} = 50.85 \text{ mm} > d_b = 18 > 25 \qquad \text{OK}$$

Check for strain:-

$$a = \frac{A_{s}.f_y}{0.85b\ f'_c} = \frac{2035.75 \times 420}{0.85\times600\times24} = 70 \text{ mm}$$

$$x = \frac{a}{\mathcal{B}_1} = \frac{70}{0.85} = 82.35 \text{ mm}$$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{291-82.35}{82.35}\right) = 0.00758 > 0.005 \qquad \text{0k}$$

Flexural Design of Negative Moment for(B1):-(Mu=-198.3 KN.m)

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{198.3\times10^6}{0.9\times600\times291^2} = 4.34 \text{ Mpa}$$

$$m = \frac{f_y}{0.85f'_c} = \frac{420}{0.85\times24} = 20.6$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times4.34}{420}}\right) = 0.0118$$

As = ρ.b.d = 0.0118 ×600×291 = 2060.28 mm2

Check for As,min:

$$Asmin = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) = \frac{\sqrt{24}}{4*420}*600*291 = 509.14 \text{ mm2}$$

$$Asmin = \frac{1.4}{(fy)}(bw)(d) = \frac{1.4}{420}*600*291 = 582 \text{ mm2}$$

As =2060.28 mm2Controls

Use7 ø 20 ,As,provided= 2200 mm2>As,required= 2060.28mm2… Ok

Check spacing :

$$S = \frac{600-40*2-20-(7*20)}{6} = 60 \text{ mm} > d_b = 20 > 25 \qquad \text{OK}$$

Check for strain:

$$a = \frac{A_{s.f_y}}{0.85b\, f_c'} = \frac{2200\times420}{0.85\times600\times24} = 75.5 \text{ mm}$$

$$x = \frac{a}{\mathcal{B}_1} = \frac{75.5}{0.85} = 88.8\text{mm}$$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{291-88.8}{88.8}\right) = 0.00683 > 0.005 \qquad 0k$$

Flexural Design of Negative Moment for(B1):-(Mu=-205.1 KN.m)

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{205.1\times10^6}{0.9\times600\times291^2} = 4.49 \text{ Mpa}$$

$$m = \frac{f_y}{0.85f_c'} = \frac{420}{0.85\times24} = 20.6$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times4.49[}{420}}\right) = 0.0122$$

As = ρ.b.d = 0.0122 ×600×291 = 2130.12 mm2

Check for As,min:

$$Asmin = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) = \frac{\sqrt{24}}{4*420}*600*291 = 509.14 \text{ mm2}$$

$$Asmin = \frac{1.4}{(fy)}(bw)(d) = \frac{1.4}{420}*600*291 = 582 \text{ mm2}$$

As =2130.12 mm2Controls

Use7 ø 20 ,As,provided= 2200 mm2>As,required= 2130.12 mm2… Ok

Check spacing :

$S = \frac{600-40*2-20-(7*20)}{6} = 60 \text{ mm} > d_b = 20 > 25$ OK

Check for strain:

$a = \frac{A_{s.f_y}}{0.85b\, f_c'} = \frac{2200\times420}{0.85\times600\times24} = 75.5 \text{ mm}$

$x = \frac{a}{\mathcal{B}_1} = \frac{75.5}{0.85} = 88.8\text{mm}$

$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{291-88.8}{88.8}\right) = 0.00683 > 0.005$ 0k

Flexural Design of Negative Moment for(B1):-(Mu=-179.1 m)

$Rn = \frac{M_u}{\emptyset bd^2} = \frac{179.1\times10^6}{0.9\times600\times291^2} = 3.9 \text{ Mpa}$

$m = \frac{f_y}{0.85f_c'} = \frac{420}{0.85\times24} = 20.6$

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times3.9}{420}}\right) = 0.0104$

As = ρ.b.d = 0.0104 ×600×291 = 1815.84 mm2

Check for As,min:

Asmin = $\frac{\sqrt{fc'}}{4(fy)}(bw)(d)$ = $\frac{\sqrt{24}}{4*420}*600*291$ = 509.14 mm2

Asmin = $\frac{1.4}{(fy)}(bw)(d)$ = $\frac{1.4}{420}*600*291$ = 582 mm2

As = 1815.84 mm2Controls

Use 6 ø 20 ,As,provided= 1885 mm2>As,required= 1815.84 mm2… Ok

Check spacing :

$S = \frac{600-40*2-20-(6*20)}{5} = 76 \text{ mm} > d_b = 20 > 25$ OK

Check for strain:

$a = \frac{A_{s.f_y}}{0.85b\, f'_c} = \frac{1885\times420}{0.85\times600\times24} = 64.7 \text{ mm}$

$x = \frac{a}{\mathcal{B}_1} = \frac{64.7}{0.85} = 76.12\text{mm}$

$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{291-76.12}{76.12}\right) = 0.00847 > 0.005$ 0k

Flexural Design of Negative Moment for(B1):-(Mu=-176.2m)

$Rn = \frac{M_u}{\emptyset bd^2} = \frac{176.2\times10^6}{0.9\times600\times291^2} = 3.85 \text{ Mpa}$

$m = \frac{f_y}{0.85f'_c} = \frac{420}{0.85\times24} = 20.6$

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times3.85}{420}}\right) = 0.0106$

As = ρ.b.d = 0.0106 ×600×291 = 1850.76 mm2

Check for As,min:

Asmin = $\frac{\sqrt{fc'}}{4(fy)}(bw)(d)$ = $\frac{\sqrt{24}}{4*420}*600*291$ = 509.14 mm2

Asmin = $\frac{1.4}{(fy)}(bw)(d)$ = $\frac{1.4}{420}*600*291$ = 582 mm2

As = 1850.76 mm2Controls

Use 6 ø 20 ,As,provided= 1885 mm2>As,required= 1850.76 mm2… Ok

Check spacing :

$$S = \frac{600-40*2-20-(6*20)}{5} = 76 \text{ mm} > d_b = 20 > 25 \qquad \text{OK}$$

Check for strain:

$$a = \frac{A_{s.f_y}}{0.85b\, f'_c} = \frac{1885\times420}{0.85\times600\times24} = 64.7 \text{ mm}$$

$$x = \frac{a}{\mathcal{B}_1} = \frac{64.7}{0.85} = 76.12\text{mm}$$

$$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{291-76.12}{76.12}\right) = 0.00847 > 0.005 \qquad \text{0k}$$

Flexural Design of Negative Moment for(B1):-(Mu=-215.7 kn.m)

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{215.7\times10^6}{0.9\times600\times291^2} = 4.72 \text{ Mpa}$$

$$m = \frac{f_y}{0.85f'_c} = \frac{420}{0.85\times24} = 20.6$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times4.72}{420}}\right) = 0.013$$

As = ρ.b.d = 0.013 ×600×291 = 2269.8 mm2

Check for As,min:

$$\text{Asmin} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) = \frac{\sqrt{24}}{4*420}*600*291 = 509.14 \text{ mm2}$$

$$\text{Asmin} = \frac{1.4}{(fy)}(bw)(d) = \frac{1.4}{420}*600*291 = 582 \text{ mm2}$$

As =2269.8 mm2Controls

Use 8ø 20,As,provided= 2513.3 mm2>As,required= 2269.8 mm2… Ok

Check spacing :

$S = \frac{600-40*2-20-(8*20)}{7} = 48.6 \text{ mm} > d_b = 20 > 25$ OK

Check for strain:-

$a = \frac{A_{s.f_y}}{0.85b\, f_c'} = \frac{2513.3\times420}{0.85\times600\times24} = 86.24 \text{ mm}$

$x = \frac{a}{\mathcal{B}_1} = \frac{86.24}{0.85} = 101.46 \text{ mm}$

$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{291-101.46}{101.46}\right) = 0.0056 > 0.005$ 0k

Flexural Design of Negative Moment for(B1):-(Mu=-212.9 kn.m)

$Rn = \frac{M_u}{\emptyset bd^2} = \frac{212.9\times10^6}{0.9\times600\times291^2} = 4.66 \text{ Mpa}$

$m = \frac{f_y}{0.85f_c'} = \frac{420}{0.85\times24} = 20.6$

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times4.66}{420}}\right) = 0.0128$

As = ρ.b.d = 0.0128 ×600×291 = 2234.88 mm2

Check for As,min:

Asmin = $\frac{\sqrt{fc'}}{4(fy)}(bw)(d)$ = $\frac{\sqrt{24}}{4*420}*600*291$ = 509.14 mm2

Asmin = $\frac{1.4}{(fy)}(bw)(d)$ = $\frac{1.4}{420}*600*291$ = 582 mm2

As =2234.88 mm2Controls

Use 8ø 20,As,provided= 2513.3 mm2>As,required= 2234.88 mm2… Ok

Check spacing :

$S = \frac{600-40*2-20-(8*20)}{7} = 48.6 \text{ mm} > d_b = 20 > 25$ OK

Check for strain:-

$a = \frac{A_{s.}f_y}{0.85b\ f_c'} = \frac{2513.3\times420}{0.85\times600\times24} = 86.24 \text{ mm}$

$x = \frac{a}{\mathcal{B}_1} = \frac{86.24}{0.85} = 101.46 \text{ mm}$

$\varepsilon_s = 0.003\left(\frac{d-x}{x}\right) = 0.003\left(\frac{291-101.46}{101.46}\right) = 0.0056 > 0.005$ 0k

Shear Design for (B 1):

Case 3 :

for shear design, minimum shear reinforcement is required ($A_{v,min}$), Reinforcement.

Use stirrups (2 leg stirrups) ø 8/ 150 mm , Av = 2 × 50.24 = 100.5 mm2

Vu = 234.2 KN

$Vc = \frac{1}{6}\sqrt{fc'}b_w\ d = \frac{1}{6}\sqrt{24} * 600 * 291 = 142.56$ KN

Check for section dimensions:

$Vs = \frac{Vu}{\Phi} - Vc = \frac{234.2}{0.75} - 142.56 = 169.71$

$Vs,max = \frac{2}{3}\sqrt{fc'}b_w\ d = \frac{2}{3}\sqrt{24} * 600 * 291 = 570.24$ KN

Vs= 169.71< Vs,max = 570.24 - the section is large enough .

Find the maximum stirrups spacing:

If $Vs < V_s' = \frac{1}{3}\sqrt{fc'}b_w\ d$ then $Smax \le \frac{d}{2}$ or $Smax \le 600$ mm

$V_s' = \frac{1}{3}\sqrt{fc'}b_w\ d = \frac{1}{3}\sqrt{24} * 600 * 291 * 10\text{-}3 = 285.12$ KN

Smax ≤ 600 mm, Smax ≤ $\frac{d}{2}$ = $\frac{291}{2}$ = 145.5 mm Control

Check for Vs, min:

Av, min = $\frac{1}{16}\sqrt{fc'}\frac{bw\ S}{fyt}$ but not less than

Av, min = $\frac{1}{3}\frac{bw\ S}{fyt}$ Control ($\frac{1}{16}\sqrt{fc'} = \frac{4.9}{16} < \frac{1}{3}$)

Vs,min= $\frac{1}{16}\sqrt{fc'}b_w$ d = $\frac{1}{16}\sqrt{24} * 600 * 291$ * 10-3 = 53.5 KN

Vs,min= $\frac{1}{3}b_w$ d = $\frac{1}{3} * 600 * 291$ * 10-3 = 58.2 KN Control

Φ Vc= 0.75*142.56 =106.92 KN

Φ Vsmin≥ 0.75 ($\frac{1}{3}$) * bw * d =0 .75* ($\frac{1}{3}$)*600*291*10-3 = 43.65 KN Controls

Φ Vsmin ≥ 0.75 ($\frac{\sqrt{fc'}}{16}$) * bw * d = 0.75*($\frac{\sqrt{24}}{16}$) * 600 * 291*10-3 = 40.1 KN

Φ Vc<Vu ≤ Φ Vc + Φ Vsmin

106.92<234.2 ≤ 106.92+43.65= 150.57 …… not satisfied

Cases 1&2&3 is not suitable

Case 4 :

$v_{s'} = \frac{1}{3}\sqrt{fc'}b_w$ d = $\frac{1}{3}\sqrt{24} *\ 600 * 291$ = 285.12KN

$\emptyset(v_c + v_{s,min)} < v_u \leq \emptyset(v_c + v_{s'})$

0.75(142.56+ 43.65) < 234.2< 0.75(142.56 + 285.12)

139.66 < 234.2 <320.76

Use 2 leg Φ 10

As =158 mm2

Vs = Vn – Vc = $\frac{234.2}{0.75}$ – 142.56 = 169.7KN

$$S = \frac{A_v f_{yt}\, d}{v_s} = \frac{158 * 420 * 291}{169.7 * 1000} = 113.95 \text{ mm}$$

$$s_{max} \le \frac{d}{2} = \frac{291}{2} = 145.5 \text{ mm} \qquad \text{control}$$

or $s_{max} \le 600$ mm

Use 2 leg Φ 10 @120

4-7 Design of One Way Solid Slab.

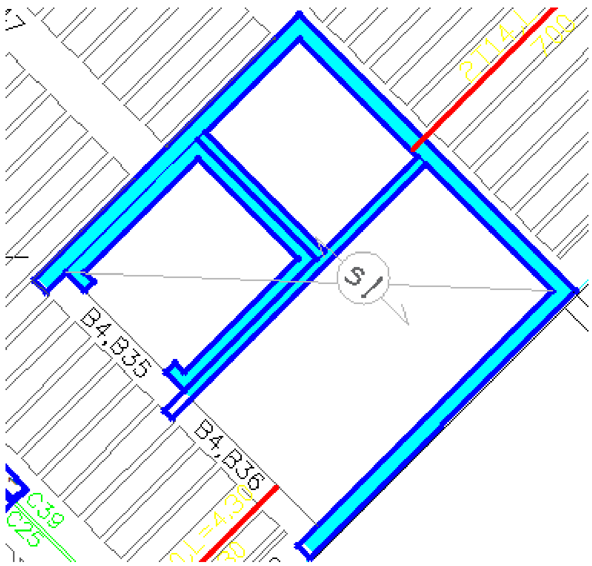

Figure 4-10: One way solid slab(S1).

Material:-

concrete B300 Fc' = 24 N/mm^2

Reinforcement Steel Fy = 420 N/mm^2

Slab Thickness Calculation:-

The overall depth must satisfy ACI Table (9.5.a):

Min H (deflection requirement) : “For one end continuous”

$$\frac{L}{24} = \frac{3.5}{24} = 0.15m$$

For One way solid slab, will use thickness of slab 15 cm.

Load Calculation:-

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

-Load Calculation For the Horizontal Slab:- (For one Meter Strip)

#	material	calculation
1	**Tiles**	**0.03*22=0.66**
2	**mortar**	**0.03*22=0.66**
3	**Coarse sand**	**0.07*16=1.12**
4	**RC concrete**	**0.15*25=3.75**
5	**plaster**	**0.02*22=0.44**
	Sum	**6.63**

Table 4-4: Dead Load Calculation of Solid Slab.

Live load =5 KN/m

- ✓ Design of Positive Moment :

Design of Positive Moment :-(Mu=16.5 KN.m)

Assume bar diameter Φ12 for main reinforcement .

$$m = \frac{fy}{0.85 * fc'} = \frac{420}{0.85 * 24} = 20.59$$

Rn = $\dfrac{Mu/\phi}{b*d^2}$

Rn = $\dfrac{16.5*10^{6}/0.9}{1000*(124)^2}$ = 1.19(Mpa)

ρ = $\dfrac{1}{m}(1 - \sqrt{1-\dfrac{2m*Rn}{fy}})$

ρ = $\dfrac{1}{20.59}(1 - \sqrt{1-\dfrac{2(20.59)(1.19)}{420}})$ = 0.0029

As=ρ * b * d = 0.0029* 100 *12.4= 3.6cm²

Check for As min:

$A_s \text{ min} = \rho_{min} * b * h = 0.0018*100*15 = 2.7cm^2$

As_{req}= 3.6cm^2 >As_{min}= 2.7cm^2 **OK**

Use ø 12/25cm , $A_{s,provided}$= 4.52cm^2>$A_{s,required}$= 3.6cm^2 …. Ok

Design of Positive Moment :-(Mu=12.2KN.m)

m = $\dfrac{fy}{0.85 * fc'}$ = $\dfrac{420}{0.85*24}$ = 20.59

Rn = $\dfrac{Mu/\phi}{b*d^2}$

Rn = $\dfrac{12.2*10^{6}/0.9}{1000*(124)^2}$ = 0.88 (Mpa)

ρ = $\dfrac{1}{m}(1 - \sqrt{1-\dfrac{2m*Rn}{fy}})$

$$\rho = \frac{1}{20.59}(1 - \sqrt{1 - \frac{2(20.59)(0.88)}{420}}) = 0.0021$$

As=ρ * b * d = 0.0021* 100 *124= 2.6cm²

Check for As min:

$$As\ \min = \rho_{\min} * b * h = 0.0018*100*15 = 2.7cm^2$$

As_{req}= 2.6cm^2 <As_{min}= 2.7cm^2 **Not OK**

Use 3ø12/1m strip , $A_{s,provided}$= 3.39cm^2≥$A_{s,required}$= 2.6cm^2 …. Ok

- ✓ Design of Negative Moment:

Design of Negative Moment:- (Mu=15.9 KN.m)

$$m = \frac{fy}{0.85 * fc'} = \frac{420}{0.85*24} = 20.59$$

$$Rn = \frac{Mu/\phi}{b*d^2}$$

$$Rn = \frac{15.9*10^6/0.9}{1000*(124)^2} = 1.15\ (Mpa)$$

$$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{2m*Rn}{fy}})$$

$$\rho = \frac{1}{20.59}(1 - \sqrt{1 - \frac{2(20.59)(1.15)}{420}}) = 0.0028$$

As=ρ * b * d = 0.0028* 100 *124= 3.47 cm²

Check for As min:-

$As\ \min = \rho_{\min} * b * h = 0.0018*100*15 = 2.7cm^2$

$As_{req} = 3.47cm^2 > As_{min} = 2.7cm^2$ **OK**

Use ø12/25cm , $A_{s,provided} = 4.52cm^2 \geq A_{s,required} = 3.47cm^2$ …. Ok

Shrinkage and Temperature:-

$\rightarrow \rho = 0.0018$

$As\ \min = \rho_{\min} * b * h = 0.0018*100*15 = 2.7cm^2$ (control)

Use 3Φ12/1m strip.

Shear Design:-

Check Whether Thickness Is Adequate For Shear:-

$V_{u,\max}$ = 29.3 KN/ 1m strip

d = h – 15 – db = 200 – 15 – (12 /2) = 124 mm

$\Phi Vc = \frac{1}{6} * \Phi * \sqrt{fc'} * bw * d$

$= \frac{1}{6} * 0.75 * \sqrt{24} * 1000 * 124$ = 75.9 KN / 1 m strip

ΦVc/2= 37.95 KN > $V_{u,\max}$ = 29.3 KN/ 1m strip.

The thickness of the slab is adequate enough.

4-8 Design of Column (C28/GF).

Material :

concrete B350 $F_c' = 24\ N/mm^2$

Reinforcement Steel $F_y = 420\ N/mm^2$

Load Calculation:-

Service Load:-

Dead Load =1553.9KN

Live Load =858.65 KN

Factored Load:-

$P_U = 1.2 \times 1553.9 + 1.6 \times 858.65 = 3238.52KN$

Dimensions of Column:-

$Assume\ \rho g = 0.01$

$\phi * Pn = 0.65\ x\ 0.8 \times Ag\{0.85\ \ fc'(1-\rho g)+\rho g * Fy\}$

$3238.52*1000 = 0.65\,x\,0.8 \times Ag\,\{0.85*24\,(1-0.01)+0.01*420\}$

Ag= 255605.37 mm2

Assume Rectangular Section

Try h = 600mm

b = 450mm

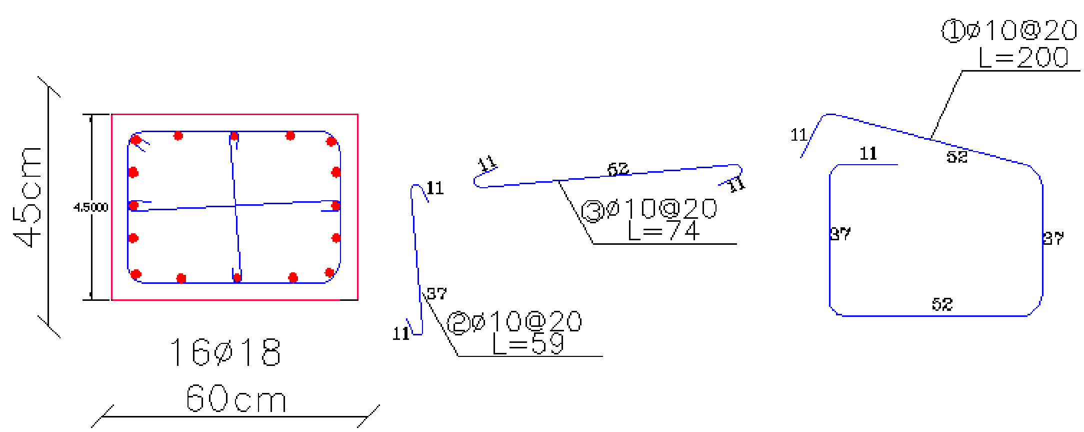

Figure 4-11: Column (C28) section and reinforcement.

Design of the tie reinforcement :

S $\leq$16 db (longitudinal bar diameter)

S$\leq$ 48dt (tie bar diameter).

S $\leq$ Least dimension.

spacing $\leq 16\times d_b = 16\times 2.8 = 44.8$cm ….

spacing $\leq 48\times dt = 48\times 1.0 = 48$ cm

spacing $\leq$ least.dim =45cm control

Useϕ10@40 cm

For Using Column We have using $\boxed{16\nu 18.}$

4-9 Design of Isolated Footing.

Material :-

concrete B350 Fc' = 24 N/mm^2

Reinforcement Steel Fy = 420 N/mm^2

Load Calculations:-

Dead Load = 2207.78 Kn , Live Load = 1066.15 Kn

Total services load = 2207.78 + 1066.15 = 3273.93 Kn

Total Factored load = 1.2*2207.78 + 1.6*1066.15 = 4355.18Kn

Column Dimensions (a*b) = 60*60 cm

Soil density = 20 Kg/cm3

Allowable Bearing Capacity = 350 Kn/m2

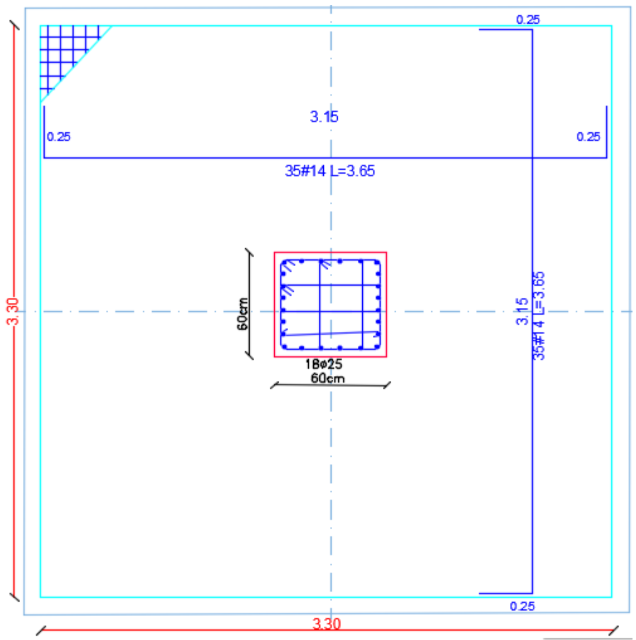

Figure 4-12 :Foot plan.

Assume h = 80cm

$q_{net-allow}$ = 350 – 25*0.8 – 20*0.6 – 0.7*25 = 300.5kn/m2

Area of Footing :-

$$A = \frac{Pt}{q_{net-allow}} = \frac{3273.93}{300.5} = 10.89m^2$$

Assume Square Footing

B required = 3.3 m

Select B = 3.3 m

Bearing Pressure :-

q_u = 4355.18/3.3*3.3= 399.92 Kn/m^2

Design of Footing :-

Design of One Way Shear Strength :-

Critical Section at Distance (d)From The Face of Column

Assume h = 80cm , bar diameter ø 14 for main reinforcement and 7.5 cm Cover

d = 800 – 75 – 14 = 711 mm

$$Vu = q_u * \left(\frac{B-a}{2} - d\right) * L$$

$$Vu = 399.92 * \left(\frac{3.3-0.6}{2} - 0.711\right) * 3.3 = 843.31Kn$$

$$\phi.Vc = \phi.\frac{1}{6} * \sqrt{fc'} * b_w * d$$

$$\phi.Vc = 0.75 * \frac{1}{6} * \sqrt{24} * 3300 * 711 = 1436.8Kn$$

$$\phi.Vc = 1436.8KN > Vu = 843.31Kn$$

$$\therefore \quad Safe$$

Design of Two Way Shear Strength :-

$$Vu = Pu - FR_b$$

$$FR_b = q_u * area \ of \ \ critical \ \ section$$

$$Vu = 4355.18 - 399.92[(0.6 + 0.711) * (0.6 + 0.711)] = 3667.8Kn$$

The punching shear strength is the smallest value of the following equations:

$$\phi.V_c = \phi.\frac{1}{6}\left(1 + \frac{2}{\beta_c}\right)\sqrt{f_c{}'}\,b_o d$$

$$\phi.V_c = \phi.\frac{1}{12}\left(\frac{\alpha_s}{b_o / d} + 2\right)\sqrt{f_c{}'}\,b_o d$$

$$\phi.V_c = \phi.\frac{1}{3}\sqrt{f_c{}'}\,b_o d$$

Where:-

$$\beta_C = \frac{Column\ Length\ (a)}{Column\ Width\ (b)} = \frac{60}{60} = 1$$

b_o = Perimeter of critical section taken at (d/2) from the loaded area

$b_o = (0.6 + 0.711) * 4 = 5244\ mm$

α_s = 40 for interior column

$$\phi.V_C = \phi.\frac{1}{6}\left(1 + \frac{2}{\beta_c}\right)\sqrt{f_c'}\,b_o d = \frac{0.75}{6} * \left(1 + \frac{2}{1}\right) * \sqrt{24} * 5244 * 711 = 6849.7Kn$$

$$\phi.V_C = \phi.\frac{1}{12}\left(\frac{\alpha_s}{b_o / d} + 2\right)\sqrt{f_c'}\,b_o d = \frac{0.75}{12} * \left(\frac{40 * 711}{5244} + 2\right) * \sqrt{24} * 5244 * 711 = 8474.6Kn$$

$$\phi.V_C = \phi.\frac{1}{3}\sqrt{f_c'}\,b_o d = \frac{0.75}{3} * \sqrt{24} * 5244 * 711 = 4566.4Kn$$

ΦVc =4566.4 Kn>Vu=3667.8Kn

Design of Bending Moment :-

Critical Section at the Face of Column

FR = $q_u * \left(\frac{B-a}{2}\right) * L$ = 399.92 * $\left(\frac{3.3-0.6}{2}\right)$ *3.3 =1781.6Kn

Mu = 1781.6*0.675 = 1202.58 Kn.m

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{1202.58\times10^6}{0.9\times3300\times711^2} = 0.8Mpa$

$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85\times24} = 20.59$

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2\times20.59\times0.8}{420}}\right) = 0.00194$

$A_{s,req}$ = ρ.b.d = 0.00194×3300×711 = 4551.82mm^2

$A_{s,min}$ = 0.0018*3300*800 = 4752 mm^2

$A_{s,req} < A_{s,min}$ = 4752 mm^2 ,

As,min is control.

Check for Spacing :-

S = 3h = 3*80 = 240cm

S = 380*$\left(\frac{280}{\frac{2}{3}*420}\right)$ – 2.5*75 = 192.5 cm

S = 45 cm **……… is control**

Use 35ø14 in Both Direction, $A_{s,provided}$= 5033.7mm^2>$A_{s,required}$= 4752 mm^2… Ok

Check for strain:

Tension = Compression

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

$$5033.7 \times 420 = 0.85 \times 24 \times 3300 \times a$$

$$a = 31.4mm$$

$$c = \frac{31.4}{0.85} = 36.9$$

$$\varepsilon_s = \frac{711 - 36.9}{36.9} \times 0.003 = 0.0548 > 0.005 ...ok$$

Design of Dowels :-

$$\Phi Pn.b = \Phi(0.85 fc' A_1 \times \sqrt{\frac{A_2}{A_1}})$$

A_1 = 60* 60 = 0.36 m^2

A_2 = 3.3* 3.3 = 10.89 m^2

$$\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{10.89}{0.36}} = 5.5 > 2 \ldots\ldots\ldots\ldots\ldots \sqrt{\frac{A_2}{A_1}} = 2$$

$$\Phi Pn.b = 0.65 \times (0.85 \times 24 \times 0.36 \times 2) = 9547.2 Kn$$

$$\Phi Pn = 9547.2 > Pu = 4355.18 \ldots\ldots ok$$

No Need For Dowels

As,min = 0.005 * Ac = 0.005 * 600* 600 = 1800 mm2

Use 12ø14, $A_{s,provided}$= 1847.25 mm^2>$A_{s,required}$= 1800mm^2… Ok

Development Length In Footing :-

Tension Development Length In Footing :-

$$Ld_{T\ req} = \frac{9}{10} * \frac{F_y}{\lambda\sqrt{f_c}} * \frac{\psi_e\psi_s\psi_t}{\frac{ktr+cb}{db}} * db > 300mm$$

$$Ktr = 0\ (Nostripes)$$

$$\frac{ktr + cb}{db} = 2.5$$

$$Ld_{T\ req} = \frac{9}{10} * \frac{420}{1*\sqrt{24}} * \frac{1*1*0.8}{2.5} * 14 = 345.7mm > 300mm$$

$$Ld_{T\ available} = \frac{3300-600}{2} - 75 = 1275\ mm$$

$Ld_{T\ available}$ = 1275 mm >ld_{req} = 345.7 mm…….. **OK**

Compression Development Length In Footing :-

$$Ld_{Creq} = \frac{0.24*Fy*dB}{\sqrt{24}} > 0.043*Fy*dB > 200mm$$

$$Ld_{Creq} = \frac{0.24*420*14}{\sqrt{24}} = 288.05 > 0.043*420*14 = 252.84 > 200mm$$

Ld_{Creq}= 288.05mm

$Ldc_{available}$ = 800 – 75 – 14 – 14 = 697mm >Ld_{Creq}= 288.0 mm …….. **Ok**

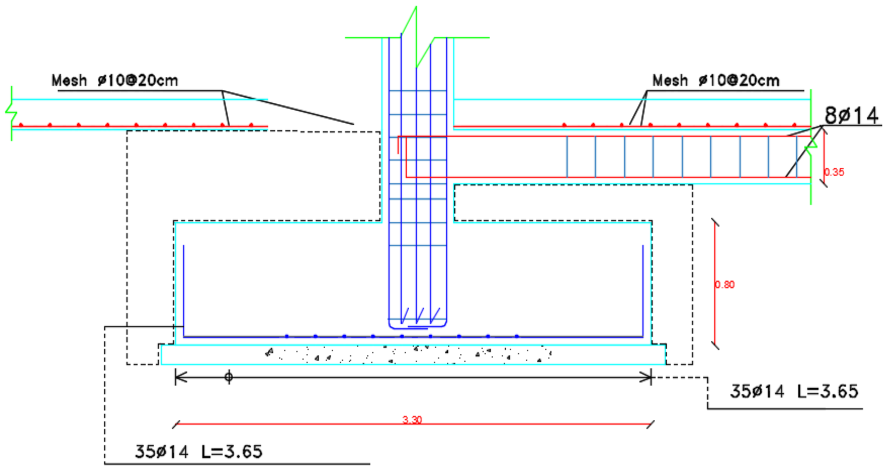

Figure 4-13:Foot reinforcement details.

4-10 Design of Stair.

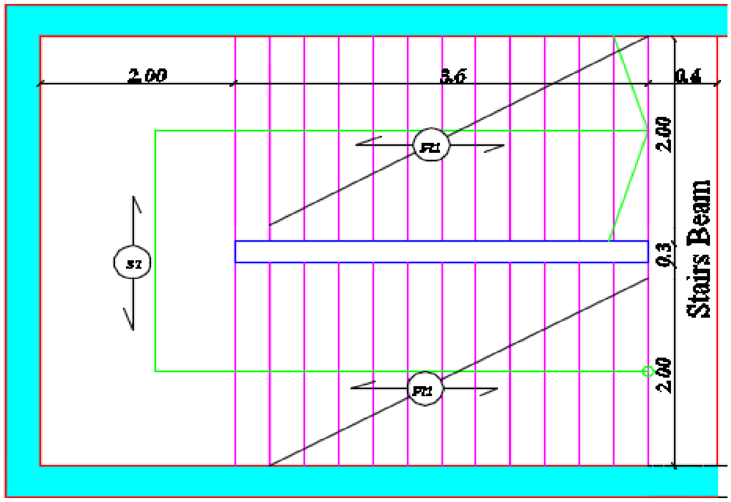

Figure 4-14: Stair plan.

Material:-

Concrete B300 Fc' = 24 N/mm^2

Reinforcement Steel fy = 420 N/mm^2

Determination the Thickness of Slab (flight and landing):

L = 4.4m

h_{req} = 4.4/ 20 =**0.22m**

Take h= 25cm.

$\Rightarrow$ **Use h = 25cm**.

Rise = 15cm, run = 30cm

$\theta = tan^{-1}(\frac{rise}{run}) = tan^{-1}(\frac{15}{30}) = 26.57$

Cos θ = 0.894

Load Calculations at section:

Load on Flight:

Dead Load:

For 1m strip:

Flight = (25*0.20)/(Cos26.57) = 5.59KN/m.

Horizontal Mortar = 0.03*22*1 = 0.66 KN/ m.

vertical Mortar = 0.03*22*1*(15/30) = 0.33 KN/ m.

Plaster = (0.02*22)/ (Cos 26.57) = 0.49 KN/ m.

Horizontal tiles =23*0.04*(33/30) = 1.012KN / m.

Vertical tiles =23*0.03*(15/30) =0.345 KN/m

Triangle = 25*0.15*1*0.5 = 1.875KN/m

Total dead load = 10.564 KN/ m.

Live load:

Live load for stairs =5 KN/ m^2.

Factor Loads:

Q_u=1.2*10.502+1.6*5 = 20.67 KN/m.

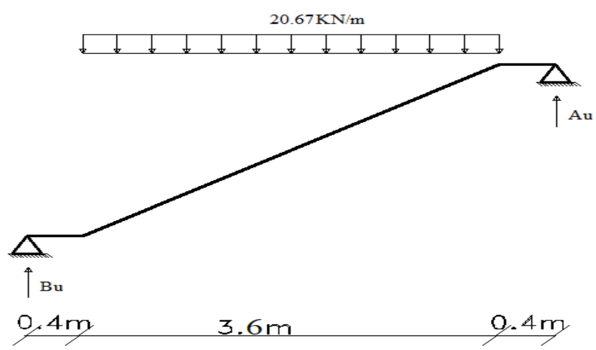

Figure 4-15: structural system of flight.

Au = 20.67*3.6*0.5 = 37.21 KN

Max Vu =37.21

Max Mu =(37.21×(0.4+1.5))-(20.67×(1.5*1.5 /2))

=47.4 KN.m

Design of Shear:

Assume Ø 12 for main reinforcement:-

So, d = 250-20 -12\2 = 224 mm

Max Vu = 37.21 KN.

$$\phi Vc = \frac{\phi\sqrt{f_c'} * b_w * d}{6}$$

$$\phi Vc = \frac{0.75 * \sqrt{24} * 1000 * 224}{6} = 137.2KN$$

Vu = 37.21 KN< ϕVc = 137.2 KN.

No shear Reinforcement is required. So the depth of the stair is OK.

Design of Bending Moment:

Max Mu = 61.4 kN.m

$Mn = \frac{mu}{0.9} = \frac{47.4}{0.9}$ = 52.7 KN.m.

$$K_n = \frac{Mn}{b \cdot d^2}$$

$$k_n = \frac{52.7 \times 10^6}{1000 \times 224^2} = 1.05 MPa$$.

$$m = \frac{fy}{0.85 \times fc'}$$

$$m = \frac{420}{0.85 \times 24} = 20.6$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mk_n}{f_y}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2*20.6*1.05}{420}}\right) = 0.0026$$

As_{req} = 0.0026*1000*224 = 682.4mm2.

$As_{min} = 0.0018 * b * h = 0.0018*1000*250 = 450mm$

As_{min} = 450mm≤ As_{req} = 682.4mm2

Use Φ 12@15cm

As provided =753.9 mm2>As req.

Step(s) is the smallest of :

3h = 3*250 = 750mm.

450mm

S= 380($\frac{280}{fs}$)- 2.5cc = 380($\frac{280}{280}$)- 2.5*20 = 330mm .

S= 150mm < Smax

Check Strain:

T=C

As*fy = 0.85* f_c' *b*a

420×753.9 = 0.85×24×1000×a

a=15.5 mm

$x = \frac{a}{\beta_1} = \frac{15.5}{0.85} = 18.3mm$ × Note: $f_c' = 24$ MPa< 28 MPa→ $\beta_1 = 0.85$

$\varepsilon_s = (\frac{d-x}{x})* 0.003$

$= (\frac{224-18.3}{18.3})\times 0.003 = 0.0337 > 0.005$

∴ ∅ = 0.9 .... OK.

5 -Lateral reinforcement:

As min = 4.5 cm2

Use Φ10 @ 20 cm

As = 4.74cm2/m

Design of landing:

Load on landing:

Dead Load:

Slab = 0.25 *25*1 = 6.25 KN/ m.

Tiles = 0.03*23*1 = 0.69 KN/m.

Mortar = 0.02*22*1= 0.44 KN/ m.

Plaster = 0.03*22*1 = 0.66 KN/ m.

Sand = 17*0.08*1= 1.36 KN/m

Total dead load =8.15 KN/ m.

Live load:

Live load for stairs = 5 KN/ m.

Qu=1.2*8.15+1.6*5=17.78KN/m.

Au or Bu from Analysis:

Au = 37.21 KN

$W = \frac{AU}{B} = \frac{37.21}{2.15}$

W= 17.3KN/m

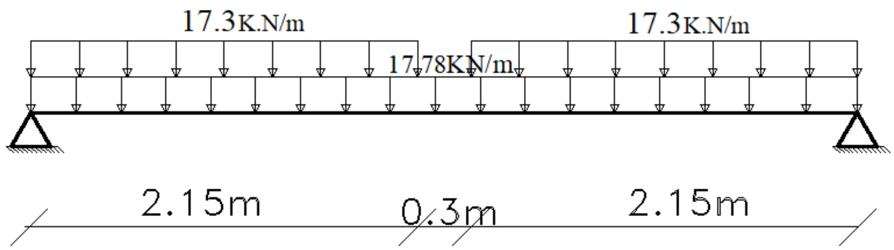

Figure 4-16: Structural system of landing.

Vu = (17.78×4.6/2) + (17.3×2.15)

Vu = 78.1KN.

Mu max =(78.1×2.3)-(17.78×2.3*2.3/2)-(17.3×2.15×1.225)

Mu max= 87.04 KN/m

Design of Shear for landing:

Assume Ø12 for main reinforcement:-

So, d = 250-20 -12\2 = 224 mm

Max Vu As the support reaction = 78.1KN.

$$\phi Vc = \frac{\phi \sqrt{f_c'} * b_w * d}{6}$$

$$\phi Vc = \frac{0.75 * \sqrt{24} * 1000 * 224}{6} = 137.2 KN$$

Vu = 78.1KN< ϕVc = 137.2 KN.

No shear Reinforcement is required. So the depth of the stair is OK.

Design of Bending Moment for landing :

Max Mu = 87.04 kN.m

$Mn = \frac{mu}{0.9} = \frac{87.04}{0.9}$ = 96.71 KN.m.

$$K_n = \frac{Mn}{b \cdot d^2}$$

$$k_n = \frac{96.71 * 10^6}{1000 * 224^2} = 1.93 MPa$$.

$$m = \frac{fy}{0.85 \times fc'}$$

$$m = \frac{420}{0.85 \times 24} = 20.6$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mk_n}{f_y}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 1.93}{420}}\right) = 0.0048$$

As_{req} = 0.0048*1000*224= 1075.2mm2.

$As_{min} = 0.0018 * b * h = 0.0018 * 1000 * 250 = 450mm$

As_{min} = 450mm2≤ As_{req} = 1075.2mm2

Use Φ 12@15cm

As provided =1130 mm2>As req.

Step(s) is thr smallest of :

3h = 3*250 = 750mm .

450mm

S= 380($\frac{280}{fs}$)- 2.5cc = 380($\frac{280}{280}$)- 2.5*20 = 330mm .

S= 150mm < Smax

Check Strain:

T=C

As×fy = 0.85× f_c' ×b×a

420×1130= 0.85×24×1000×a

a=23.26 mm

$x = \frac{a}{\beta_1} = \frac{23.26}{0.85} = 27.37$ mm × Note: $f_c' = 24$ MPa< 28 MPa→ $\beta_1 = 0.85$

$\varepsilon_s = (\frac{d-x}{x}) \times 0.003$

$= (\frac{224-27.37}{27.37}) \times 0.003 = .0215 > 0.005$

∴ $\emptyset = 0.9$.... OK.

Lateral reinforcement:

As min = 4.5 cm2

Use Φ10 @ 20 cm

As = 5.53cm2/m

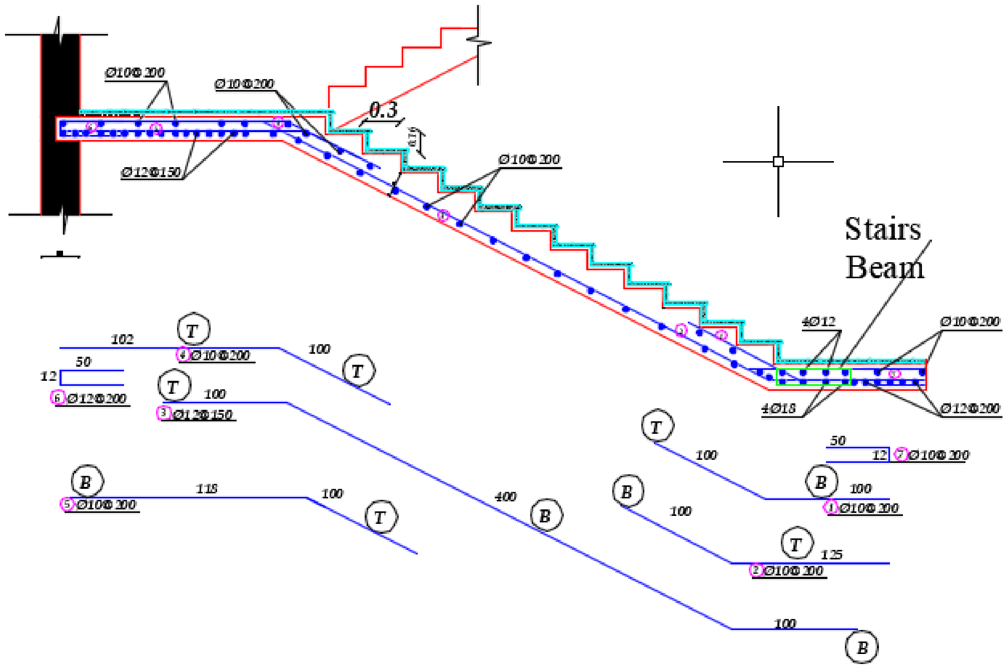

Figure14-17: Reinforcement for stairs.

4-11 Design of Basement Wall .

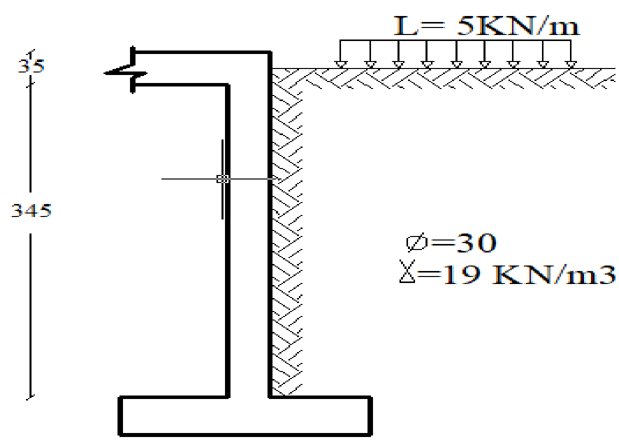

Figure 4-18: Geometry of basement.

Material:-

Concrete B350 Fc' = 28 N/mm^2

Reinforcement Steel fy = 420 Mpa

$\emptyset = 30° \quad \gamma = 19.00 KN/m^3$

- Soil at rest

$Ko = 1 - \sin \emptyset$

$= 1 - \sin 30$

$= 0.50$

Load on basement wall:

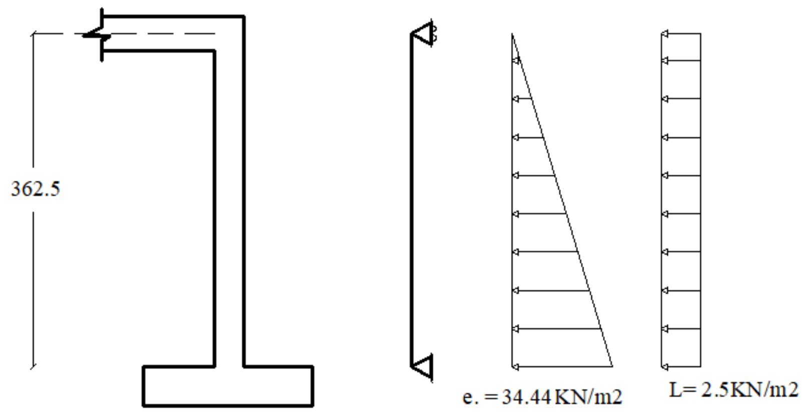

Figure 4-19: system and loads of basement.

For 1m length of wall:

- **Weight of backfill:**

$e = Ko * \gamma * h$

$= 0.50 \times 19.0 \times 3.625$ =34.44KN/m

$q_{1\ (Factored)} = 1.6 \times e$

$q_{1\ (Factored)} = 1.6 \times 34.44 = 55.1 KN/m$

- **Load from live load:**

LL=5 KN/m2

$q2 = Ko \times LL$

= 0.50 * 5 =2.50 KN/m

$q_{2\ (Factored)}$ = 1.6 *2.50 =4.0 KN/m

Design of the shear force:

- Assume Ø14 for main reinforcement.
- Assume h = 300 mm,

$d = 300 - 20 - 14 = 266$ mm

By using **ATIR** program, we get the envelope moment and shear force diagram

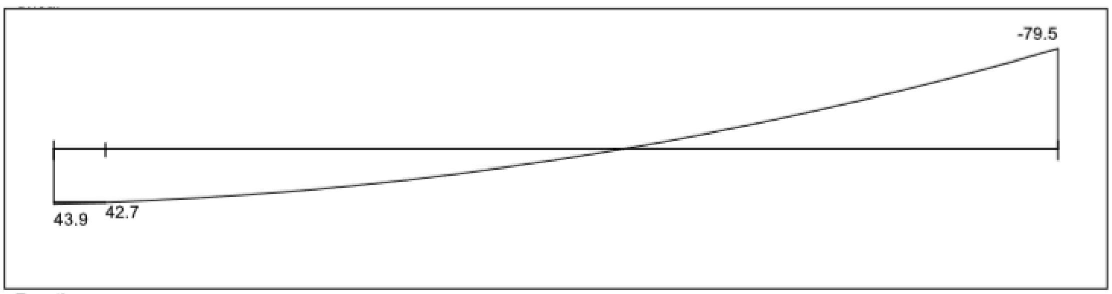

Figure 4-20 shear of basement.

Max Vu = 79.5 KN.

$$\phi Vc = \frac{\phi\sqrt{f_c'} * b_w * d}{6}$$

$$\phi Vc = \frac{0.75 \times \sqrt{24} \times 1000 \times 266}{6} = 162.9KN$$

Vu = 79.5 KN < ϕVc = 162.9 KN.

No shear Reinforcement is required.

Design of bending moment:

By using **ATIR** program, we get the envelope moment and moment force diagram

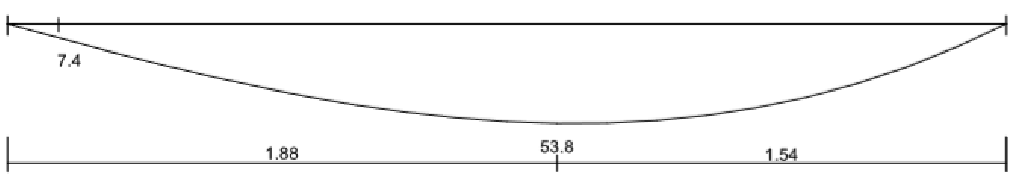

Figure 4-21 moment of basement.

Mu max =53.8 KN.m

$$\mathrm{Mn} = \frac{\mathrm{Mu}}{0.9} = \frac{53.8}{0.9} = 59.8\ \mathrm{KN.m}$$

$$\mathrm{Kn} = \frac{\mathrm{Mn} \times 10^6}{\mathrm{b} \times \mathrm{d}^2} = \frac{59.8 \times 10^6}{1000 \times 266^2} = .845\ \mathrm{Mpa}$$

$$\mathrm{m} = \frac{\mathrm{Fy}}{0.85 \times \mathrm{fc}'} = \frac{420}{0.85 \times 24} = 20.6$$

$$\rho = \frac{1}{\mathrm{m}} \times \left(1 - \sqrt{1 - \frac{2 \times \mathrm{kn} \times \mathrm{m}}{\mathrm{Fy}}}\right)$$

$$= \frac{1}{20.60} \times \left(1 - \sqrt{1 - \frac{2 \times 0.845 \times 20.6}{420}}\right)$$

$$= 2.06 \times 10^{-3}$$

$$\mathrm{Asreq} = \rho \times \mathrm{b} \times \mathrm{d} = 2.06 \times 10^{-3} \times 1000 \times 266 = 5.4674\ \mathrm{cm}^2/\mathrm{m}$$

$$\mathrm{Asmin} = 0.0012 \times \mathrm{b} \times \mathrm{h} = 0.0012 \times 1000 \times 300 = 3.60\ \mathrm{cm}^2/\mathrm{m}$$

As_{min} = 3.60 cm^2/m $\leq As_{req}$ = 5.4674 cm^2/m

Use Φ 14@20cm

As provided =6.16 cm^2/m >As req =5.4674 cm^2/m.

Step(s) is the smallest of :

- 3h = 3*300 = 900mm .
- 450mm
- S= 380($\frac{280}{fs}$)- 2.5c_c = 380($\frac{280}{280}$)- 2.5*20 = 330mm .

S= 200mm < Smax

Select ∅14@20cm/m in one direction.

With as =6.16 cm^2/m

Select ∅10@20cm/m in the other direction.

With as =3.92 cm^2/m

Design of the horizontal reinforcement:

$$\mathrm{Asmin} = 0.0012 * \mathrm{b} * \mathrm{h} = 0.0012 * 1000 * 300 = 360\mathrm{cm}^2/\mathrm{m}$$

Select$\emptyset \mathbf{10@20cm/m}$, in two layer.

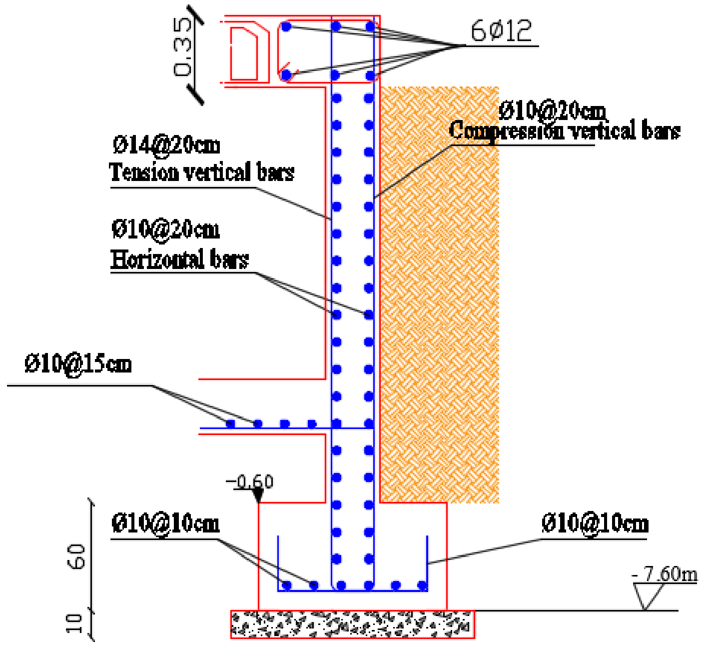

Figure 4-22: Reinforcement for basement wall.

4-12 Design of Shear Wall (SW1, F2).

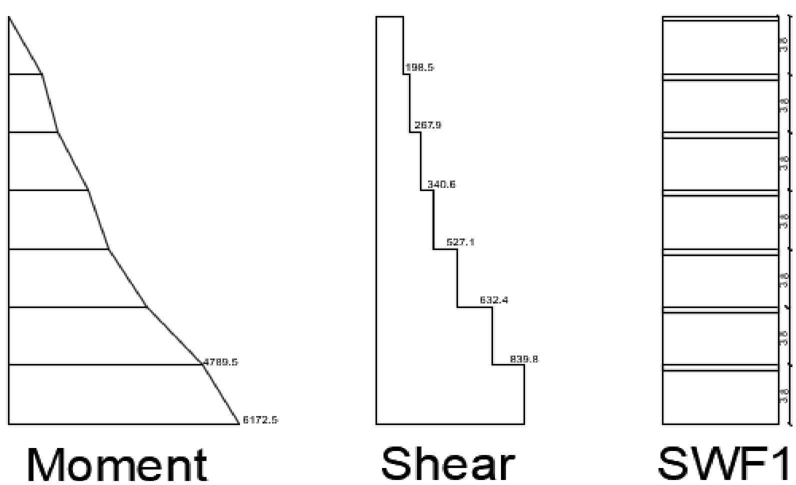

Figure 4-23: Moment and shear diagram for shear wall.

Material:-

Concrete B300 Fc' = 24 N/mm^2

Reinforcement Steel fy = 420 N/mm^2

h=30cm .shear wall thickness

Lw = 6.12m .shear wall width

Hw for one wall = 3.8 m story height

***Design of shear**

$$\sum Fx = Vu = 840KN$$

Design of the Horizontal reinforcement:

The critical Section is the smaller of:

$$\frac{lw}{2} = \frac{6.12}{2} = 3.06m\ldots\ldots \quad control$$

$$\frac{hw}{2} = \frac{26.6}{2} = 13.3m$$

$$storyheigh(Hw) = 3.8m$$

$$d = 0.8 \times lw = 0.8 \times 6.12 = 4.9m$$

Design as rectangular section:

d=4.9m, b=h=30cm

$$\emptyset V_{nmax} = \emptyset \frac{5}{6}\sqrt{f_c'}hd$$

$$= 0.75 \times 0.83 \times \sqrt{24} \times 300 \times 4900 \times 10^{-3} = 4482.93KN > V_u$$

V_c is the smallest of :

$$1 - V_c = \frac{1}{6}\sqrt{f_c'}hd = \frac{1}{6}\sqrt{24} \times 300 \times 4900 \times 10^{-3} = 1200.2KN$$

$$2 - V_c = 0.25\sqrt{f_c'}hd + \frac{N_u d}{4l_w} = 0.25\sqrt{24} \times 300 \times 4900 \times 10^{-3} + 0 = 1800.4KN$$

$$3-\ V_c = \left[0.5\sqrt{f_c} + \frac{l_w\left(0.1\sqrt{f_c'} + 0.2\,\frac{N_u}{l_w h}\right)}{\frac{M_u}{V_u} - \frac{l_w}{2}}\right]\frac{hd}{10}$$

$$= \left[0.5\sqrt{24} + \frac{6.12\left(0.1\sqrt{24}+0\right)}{3}\right]300 \times \frac{4900}{10} = 830.4KN \dots\dots cont$$

$$\frac{6172.5-4789.5}{3.8} = \frac{M_u - 4789.5}{3.8-3.06} \Longrightarrow M_u = 5058.8KN.m$$

$$\frac{M_u}{V_u} - \frac{l_w}{2} = \frac{5058.8}{840} - \frac{6.12}{2} = 3$$

Vu = 840KN>0.75*830.4 = 622.8KN. **'Horizontal reinforcement is required'**

$$\emptyset V_c + \emptyset V_s = V_u$$

$$V_s = \frac{V_u}{\emptyset} - V_c$$

$$V_s = 289.6$$

$$\frac{Avh}{s} = \frac{V_s}{fy \times d}$$

$$\frac{Avh}{s} = \frac{289.6 * 10\text{^}3}{420 \times 4900} = 0.14$$

$$\left(\frac{Avh}{s}\right)\boldsymbol{min} = 0.0025 \times h = 0.0025 \times 300 = 0.75 > 0.14 \dots cont$$

Maximum spacing is the least of:

$\frac{lw}{5} = \frac{6120}{5}$ = 1224mm

3×h = 3×300 = 900mm

450 mm ……. Control

Try ϕ12 (As = 113.09 mm2) for two layers

$$\rho = \frac{Avh}{h*S2} = \frac{2*113.09}{S2} = 0.75$$

S2 = 301.57 mm , select ϕ12@250 mm

→useϕ12@250 mm in two layer

Design of uniform Vertical reinforcement:-

$$\frac{h_w}{L_w} = \frac{26.6}{6.12} = 4.3$$

$$\rho_{vmin} > 0.0025 + 0.5\left(2.5 - \frac{h_w}{l}\right)(\rho_t - 0.0025) > 0.0025$$

For this wall with $\frac{hw}{lw}$ = 4.3 > 2.5

$$\frac{Avv}{sv} = 0.0025 + 0.5\left(2.5 - \frac{h_w}{l}\right)\left(\frac{Avh}{s \times h} - 0.0025\right) \times h$$

$$\frac{Avv}{sv} = 0.82$$

Select Φ 12@200mm. In two layer

-Maximum spacing is the least of :

$$\frac{lw}{5} = \frac{6120}{5} = 1224\text{mm}$$

3×h = 3×300 = 900mm

450 mm ……. Control

Select Φ 12@200mm In tow layer

Design of bending moment (vertical steel in boundary) :

$$A_{sv} = \left(\frac{6120}{200}\right) \times 226.2 = 6921.72mm^2$$

$$w = \left(\frac{A_{st}}{L_w h}\right)\frac{f_y}{f_c{}'} = \left(\frac{6921.72}{6120 * 300}\right)\frac{420}{24} = 0.066$$

$$\alpha = \frac{P_u}{l_w h f_c{}'} = 0$$

$$\frac{C}{l_w} = \frac{w + \alpha}{2w + 0.85\beta_1} = \frac{0.066 + 0}{2 \times 0.066 + 0.85 \times 0.85} = 0.077$$

$$\emptyset M_n = \emptyset\left[0.5A_{sv}f_y l_w\left(1 + \frac{P_u}{A_{st}f_y}\right)\left(1 - \frac{c}{l_w}\right)\right]$$

$$= 0.9[0.5 \times 6921.72 \times 420 \times 6120(1 + 0)(1 - 0.077)] = 7389.7KN.m > Mu.$$

Select Φ 12@200mm for vertical reinforcement.

4-13 Column Coordinates.

Column NO.	X-AXIS	Y-AXIS
C1	33.23	48.48
C2	26.54	48.48
C3	21.2	48.48
C4	51.73	43.95
C5	45.97	43.95
C6	40	43.95
C7	33.23	43.95
C8	25.94	43.95
C9	20.37	43.95
C10	16.7	43.95
C11	51.73	39.67
C12	45.97	39.67
C13	40	39.67
C14	33.38	39.67
C15	26.54	39.67
C16	20.37	40.13
C17	13.4	40.9
C18	51.73	30
C19	45.97	34.2
C20	40	34.2
C21	34.45	34.2
C22	27.15	34.2
C23	20.37	34.13
C24	17.38	36.38
C25	23.5	30.8
C26	51.73	25.72
C27	45.97	27.43

C28	**40**	**27.43**
C29	**34.45**	**27.43**
C30	**27.15**	**27.43**
C31	**30.32**	**24**
C32	**51.86**	**20.39**
C33	**45.97**	**20.39**
C34	**40**	**20.39**
C35	**33.76**	**20.6**
C36	**33.38**	**20.2**
C37	**29.95**	**23.63**
C38	**26.54**	**27**
C39	**23.11**	**30.47**
C40	**19.84**	**33.6**
C41	**17.1**	**36.5**
C42	**12.97**	**40.46**
C43	**9.16**	**36.5**
C44	**6.63**	**33.95**
C45	**2.1**	**29.56**
C46	**12.97**	**32.5**
C47	**10.34**	**30**
C48	**6.03**	**25.4**
C49	**8.62**	**22.55**
C50	**4.5**	**18.52**
C51	**0.44**	**13.98**
C52	**13.4**	**17.97**
C53	**9.16**	**13.74**
C54	**4.5**	**9.22**
C55	**7.72**	**6.46**
C56	**10.34**	**3.82**
C57	**14.64**	**8.27**

C58	**18.88**	**12.5**
C59	**23.5**	**17.11**
C60	**25.94**	**19.6**
C61	**29.59**	**15.94**
C62	**33.23**	**12.1**
C63	**27.15**	**13.43**
C64	**22.57**	**8.81**
C65	**18.33**	**4.58**
C66	**13.95**	**0.46**
C67	**74.4**	**4.58**
C68	**69.1**	**4.58**
C69	**63.5**	**4.58**
C70	**57.5**	**4.58**
C71	**52.33**	**4.58**
C72	**52.28**	**10.87**
C73	**57.5**	**10.87**
C74	**63.5**	**10.87**
C75	**69.1**	**10.87**
C76	**73.96**	**10.87**
C77	**74.4**	**15.97**
C78	**69.27**	**15.97**
C79	**63.5**	**15.97**
C80	**57.5**	**15.97**
C81	**52.33**	**20.39**
C82	**57.5**	**20.39**
C83	**60.45**	**20.39**
C84	**63.5**	**20.39**
C85	**69.1**	**20.39**
C86	**73.96**	**20.39**
C87	**74.4**	**25.72**

C88	**70.51**	**25.72**
C89	**66.2**	**25.72**
C90	**61.8**	**25.72**
C91	**57.5**	**25.72**
C92	**52.33**	**25.72**
C93	**61.95**	**30**
C94	**58.23**	**33.8**
C95	**55.95**	**36**
C96	**52.28**	**39.67**
C97	**52.28**	**30**
C98	**70.51**	**30.23**
C99	**66.55**	**34.32**
C100	**62.6**	**38.16**
C101	**59.69**	**40.9**
C102	**56.7**	**43.95**
C103	**52.18**	**43.95**
C104	**52.18**	**48.48**
C105	**52.28**	**52.24**
C106	**56.5**	**52.24**
C107	**60.96**	**47.96**
C108	**63.64**	**45**
C109	**66.55**	**42.1**
C110	**70.27**	**38.38**
C111	**74.56**	**30.8**
C112	**73.96**	**34.32**
C113	**73.96**	**42.6**
C114	**70.76**	**46.3**
C115	**67.84**	**49.32**
C116	**64.9**	**52.24**
C117	**64.9**	**52.24**

C118	**69.65**	**52.24**
C119	**74.56**	**47.34**
C120	**69.65**	**56.91**
C121	**73.96**	**60.89**
C122	**76.57**	**57.95**
C123	**97.52**	**55.1**
C124	**83.2**	**51.32**
C125	**87.41**	**47.18**
C126	**83.43**	**43.13**
C127	**78.67**	**38.38**
C128	**79.22**	**47.34**
C129	**75.85**	**51**
C130	**51.73**	**34.32**
C131	**52.18**	**34.32**

Table 4-5: Column coordinates.