Chapter 5

Structural Analysis And Design

5

5.1 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/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.

5.2 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…. $Fcu = 30N/mm^2 (MPa)$ 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:

$W_u = 1.2\ D_L + 1.6\ L_L$ ACI-code-318-08(9.2.1).

5.3 Check of Minimum Thickness of Structural Member:

Table (5. 1) MINIMUM THICKNESS OF NONPRESTRESSED BEAMSOR ONE-WAY SLABS UNLESS DEFLECTIONS ARE CALCULATED. (ACI 318M-11)

Minimum thickness (h)				
Member	Simply supported	One end continuous	Both end continuous	**Cantileve**
solid one way slabs	L/20	L/24	L/28	**L/10**
Beams or ribbed one way	**L/16**	**L/18.5**	**L/21**	**L/8**

- For Rib :

  $h_{min}$for (one end continuous)=L/18.5=512/18.5=27.86cm

  $h_{min}$for (both end continuous)=L/21=519/21=24.71m

- ForBeam :

  h_{min} for(one end continuous)=L/18.5=625/18.5=33.78cm

  $h_{min}$for(both end continuous)=L/21=699/21=33.28cm

The minimum thickness will be h_{min}=35cm

select 35cm for rib slab with hidden beam

h=35cm (27 cm Hollow Block+8 cm Topping)

5.4 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.

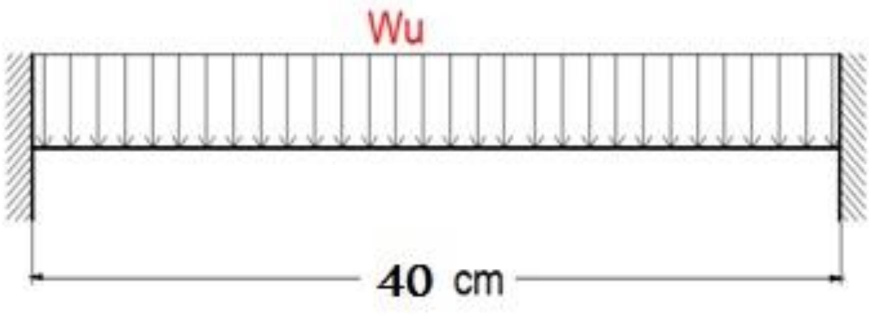


Figure 5. 1 topping load.

✓ Load calculations:

Dead load calculations:

Table (5. 2) Dead load calculation Topping

Dead load from:	δ×γ×1	KN/m
Tiles	0.03×23×1	0.69
Mortar	0.02×22×1	0.44
Coarse sand	0.07×17×1	1.19
Topping	0.08×25×1	2
Interior partitions	2.3×1	2.3
	∑	6.62KN/m

- Live Load :

  L_L =2 KN/m^2

  L_L =2 KN/m^2×1m=2KN/m

- Factored Load :

  W_U = 1.2 ×6.62 + 1.6×2 =11.144 KN/m

  Check the strength condition for plain concrete, $øM_{n} \geq M_{u}$, where ø = 0.55

  $M_n = 0.42\, \lambda \sqrt{f_c'}\, S_m$ (ACI 22.5.1, equation 22-2)

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

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

  $M_u = \frac{W_u L^2}{12} = 0.1358\ \text{KN. m}$ (negative moment)

  $M_u = \frac{W_u L^2}{24} = 0.0679\ \text{KN. m}$ (positive moment)

  $øM_n >> M_u = 0.1358\ \text{KN. m}$

No reinforcement is required by analysis. According to ACI 10.5.4, provide $A_{s,min}$for slabs as shrinkage and temperature reinforcement.

$\rho_{shrinkage}$= 0.0018 ACI 7.12.2.1

$A_s = \rho \times b \times h_{topping}$ =0.0018 ×1000×80 = 144 mm^2/m

Step (s) is the smallest of:

1. 3h = 3×80 =240 mm control by ACI 10.5.4
2. 450mm.
3. S =$380\left(\frac{280}{f_s}\right) - \mathbf{2.5C_c = 380}\left(\frac{280}{\frac{2}{3}420}\right) - \mathbf{2.5 . 20 = 330mm}$ ACI 10.6.4 OR

   S $\leq 300\left(\frac{280}{f_s}\right)$=300 mm

Take ø 8 @ 200 mm in both direction , S = 200 mm <S_{max} = 240 mm … OK

5.5 Design of One-Way Ribbed Slab(RS01, RS02, RS03, RS04) :

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

bw ≥ 10cm……………………………………………ACI(8.13.2)

Select bw=12cm

h ≤ 3.5*bw ……………………………….…….. ACI(8.13.2)

Select h=35cm<3.5*12=42 cm

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

Select tf=8cm

- ✓ Statically system and Dimensions

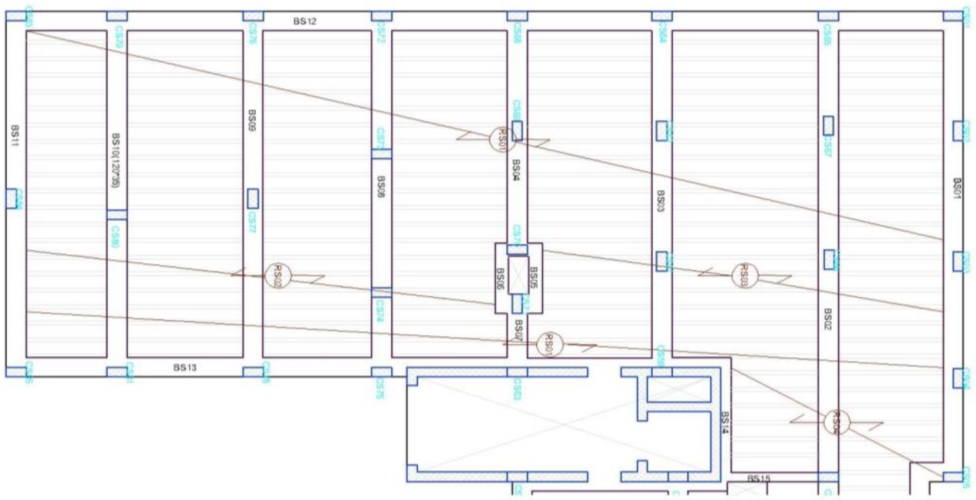


Figure 5. 2 One Way Rib slab (RS01, RS03, RS03)

Load calculations :

- Dead load:

Table (5. 3) Dead load calculation Topping of ribS

Dead load from:	**h×γ×b**	**KN/m**
Tiles	0.03×23×0.52	0.359
Mortar	0.03×22×0.52	0.343
Coarse sand	0.07×17×0.52	0.619
Topping	0.08×25×0.52	1.04
R.c rib	0.27×25×0.12	0.81
Hollow block	0.27×10×0.4	1.08
Plaster	0.03×22×0.52	0.343
Interior partitions	2.3×0.52	1.196
	$\sum$	5.79 KN/m

Dead load /rib = 5.79KN/m

- Live load =2KN/M^2

Live load /rib = 2KN/m^2 × 0.52m = 1.04 KN/m.

- <u>The effective flange (be) :</u>

1) be $\leq \frac{L}{4} = \frac{3000}{4} = 750\text{mm}$

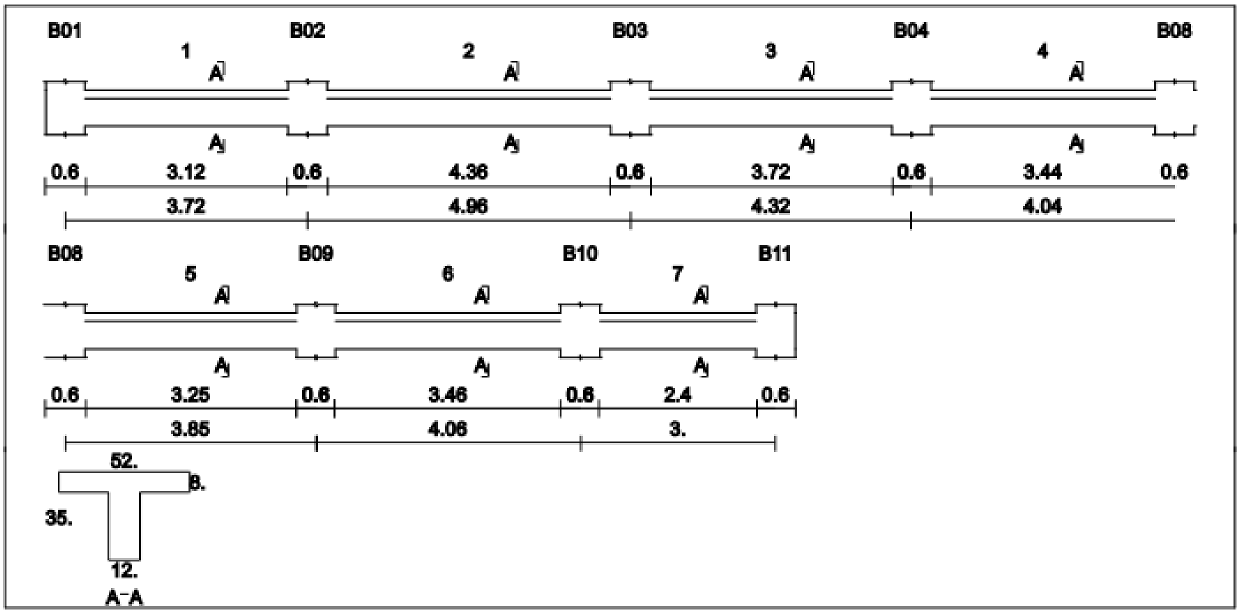
2) be≤ bw +16hf=120+16*80=1400mm

3) be≤ center to center spacing between adjacent beam = $\frac{400}{2} + \frac{400}{2} + 120 = 520mm$

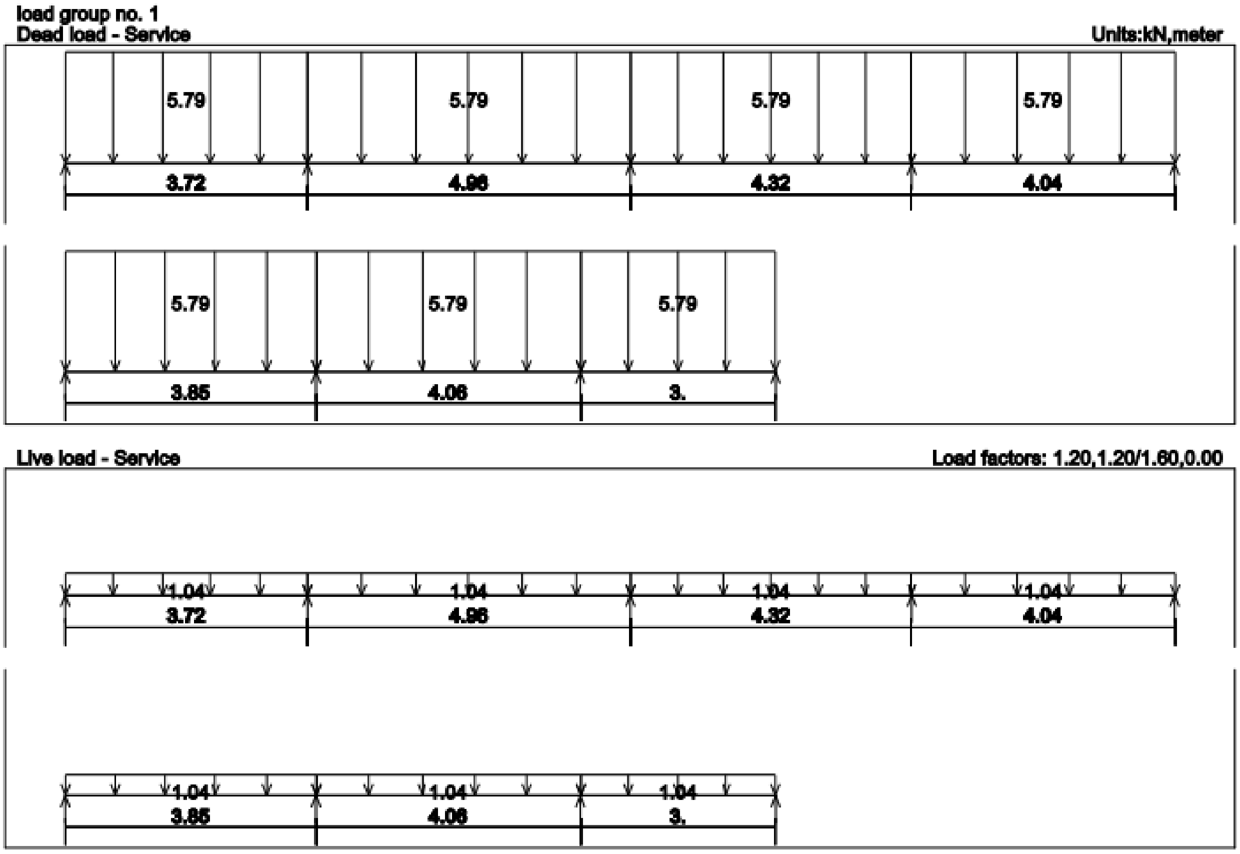
Take be=520 mm

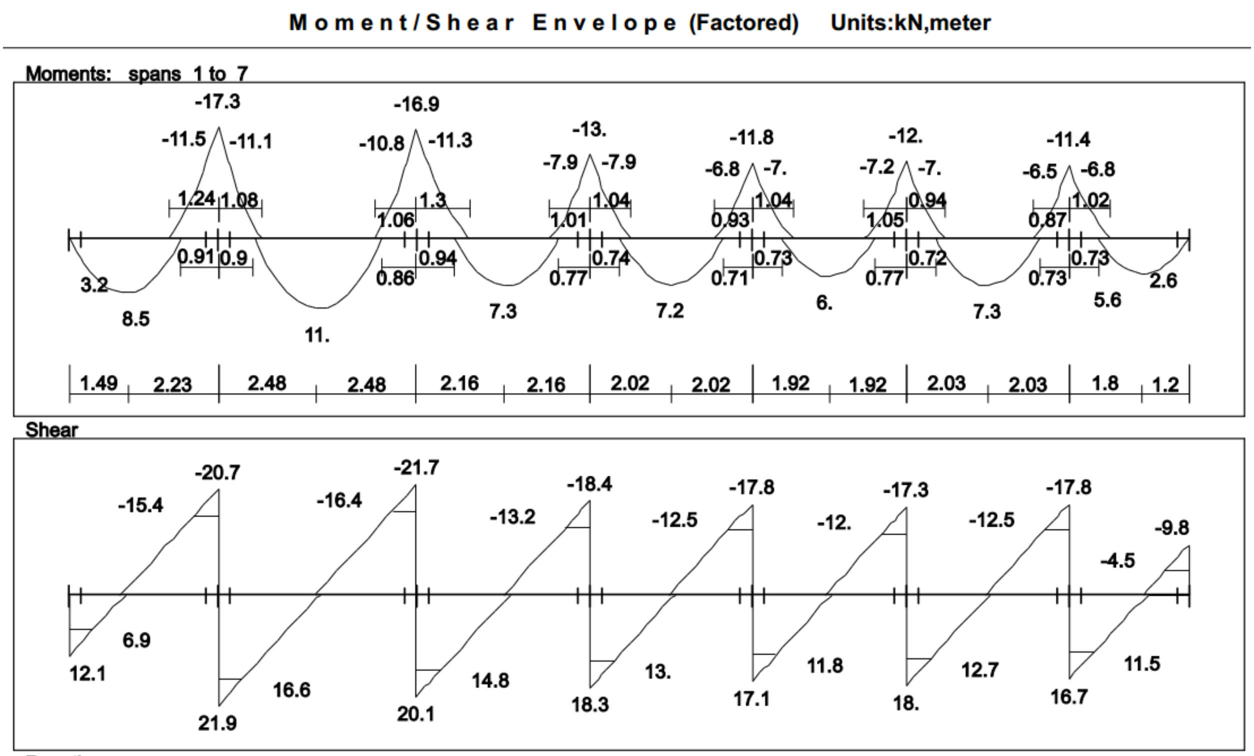
1- Rib (RS01)

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



L o a d i n g





Reactions

Factored								
DeadR	9.26	33.93	32.9	28.5	27.09	27.64	27.43	7.45
LiveR	2.86	8.63	8.82	8.19	7.78	7.57	7.08	2.34
Max R	12.12	42.56	41.72	36.68	34.87	35.21	34.51	9.79
Min R	8.62	37.2	35.86	30.99	29.65	30.24	29.9	6.89
Service								
DeadR	7.72	28.28	27.41	23.75	22.57	23.03	22.86	6.21
LiveR	1.79	5.39	5.51	5.12	4.86	4.73	4.43	1.46
Max R	9.5	33.67	32.93	28.86	27.44	27.76	27.29	7.67
Min R	7.32	30.32	29.27	25.3	24.17	24.66	24.4	5.86

Figure 5. 3 Shear & Moment Envelope Diagram (RS01)

- Design of positive moment:

M_u = 11 KN.m.

Assume bar diameter ø 12 for main positive reinforcement.

d =h- cover - $d_{stirrups} - \frac{d_b}{2} = 350 - 20 - 8 - \frac{12}{2} = 316\ mm.$

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

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

$= 0.85 \times 24 \times 520 \times 80 \times \left(316 - \frac{80}{2}\right) \times 10^{-6} = 234.22KN.m$

$M_{nf} \gg \frac{M_u}{\varphi} = \frac{11}{0.9} =$ 12.22 KN.m

the section will be designed as rectangular section with b_e =520 mm.

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{11\times10^6}{0.9\times520\times316^2} = 0.235 Mpa.$

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

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times0.235}{420}}\right) = 0.000563$

$A_{s,req}$ = ρ.b.d = 0.000563×520×316= 92.512mm^2

- Check for $A_{s,min}$.

$A_{s,min}$is the maximum of :-

$A_{s,min} = 0.25\frac{\sqrt{f'_c}}{f_y} b_w.d \geq \frac{1.4}{f_y} b_w.d$

1. $A_{s,min} = 0.25\frac{\sqrt{24}}{420}\ 120\times316 = 110.6mm^2$
2. $A_{s,min} = \frac{1.4}{420}120\times316 = 126.4mm^2\ \ Control$

A_s = 92.512 mm^2 ≤ $A_{s,min}$ = $126.4mm^2$

Use 2ø10 , $A_{s,provided}$= 157 mm^2>$A_{s,required}$= 126.4 mm^2. Ok

Check for strain:

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

$c = \frac{a}{\mathcal{B}_1} = \frac{6.22}{0.85} = 7.32\ mm$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{316-7.32}{7.32}\right) = 0.127 > 0.005 \quad 0k$$

- Design of negative moment:

M_u = -11.5 KN.m.

Assume bar diameter ø 12 for main positive reinforcement.

d =h- cover - $d_{stirrups} - \frac{d_b}{2} = 350-20-8-\frac{12}{2} = 316mm.$

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{11.5\times10^6}{0.9\times120\times316^2} = 1.066\ Mpa.$

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

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times1.066}{420}}\right) = 0.00261$

$A_{s,req}$ = ρ.b.d = 0.00261 ×120×316 = 98.971 mm^2

- Check for $A_{s,min}$.

$A_{s,min}$is the maximum of :-

$A_{s,min} = 0.25 \frac{\sqrt{f'_c}}{f_y} b_w . d \geq \frac{1.4}{f_y} b_w . d$

1. $A_{s,min} = 0.25 \frac{\sqrt{24}}{420}\ 120 \times 316 = 110.6\ mm^2$
2. $A_{s,min} = \frac{1.4}{420} 120 \times 316 = 126.4\ mm^2\ Control$

$A_s = 98.971\ mm^2 \leq A_{s,min} = 126.4 mm^2$

Use 2ø10 , $A_{s,provided} = 157\ mm^2 > A_{s,required} = 126.4\ mm^2$. Ok

Check for strain:

$a = \frac{A_{s}.f_y}{0.85 b\ f_c'} = \frac{157 \times 420}{0.85 \times 120 \times 24} = 26.94\ mm$

$c = \frac{a}{\mathcal{B}_1} = \frac{26.94}{0.85} = 31.69\ mm$

$$\varepsilon_s = 0.003 \left(\frac{d - c}{c}\right) = 0.003 \left(\frac{316 - 31.69}{31.69}\right) = 0.0269 > 0.005 \qquad 0k$$

- ✓ Shear Design for (RS01):

V_u at distance d from support= 16.6 KN

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

$V_c = \frac{1.1}{6}\ \lambda \sqrt{f_c'} b_w d = \frac{1.1}{6} \sqrt{24}\ \times 120 \times 316 \times 10^{-3} = 34.05 KN$

ø V_c =0.75×34.05 =25.55 KN.

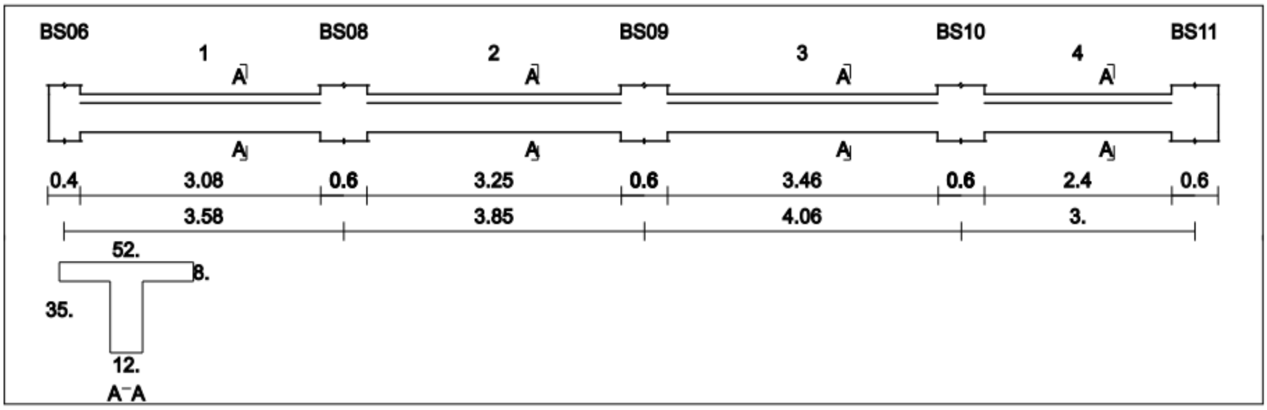
0.5 ø V_c =0.5×25.55 =12.78 KN

0.5 ø V_c < V_u < ø V_c ……………. NO

Minimum shear reinforcement is required except for concrete joist construction. So, No shear reinforcement is provided

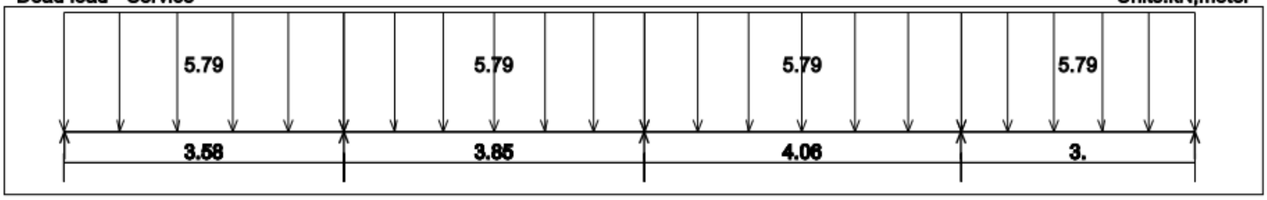
2- Rib (RS02)

Geometry Units:meter,cm

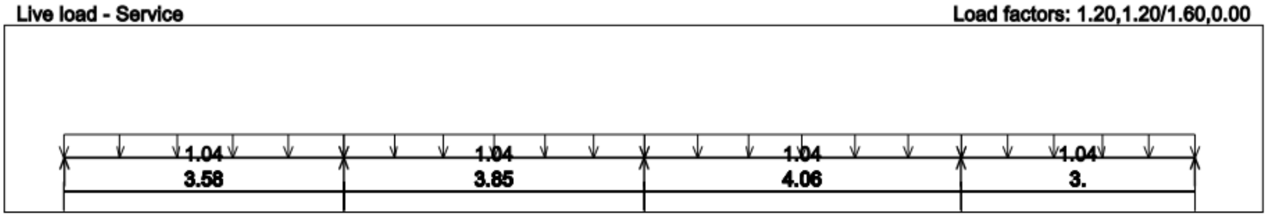


Loading

load group no. 1
Dead load - Service
Units:kN,meter

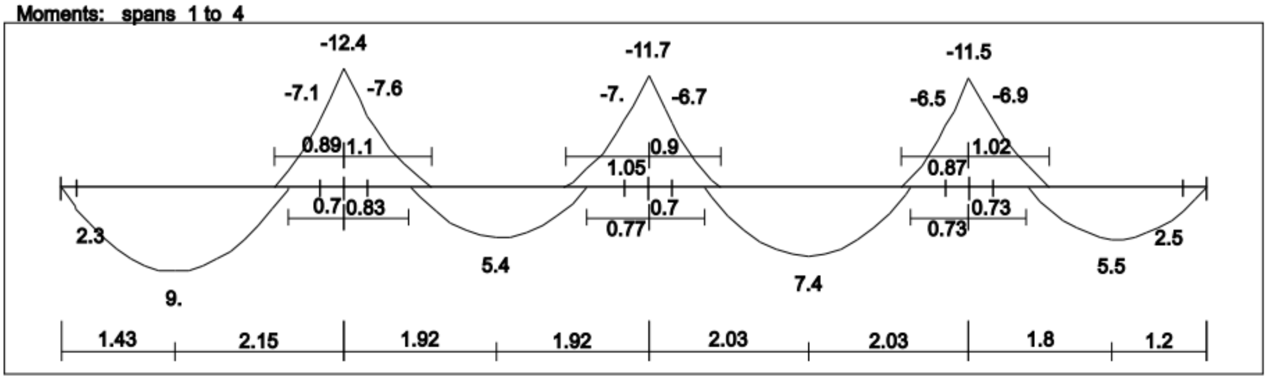


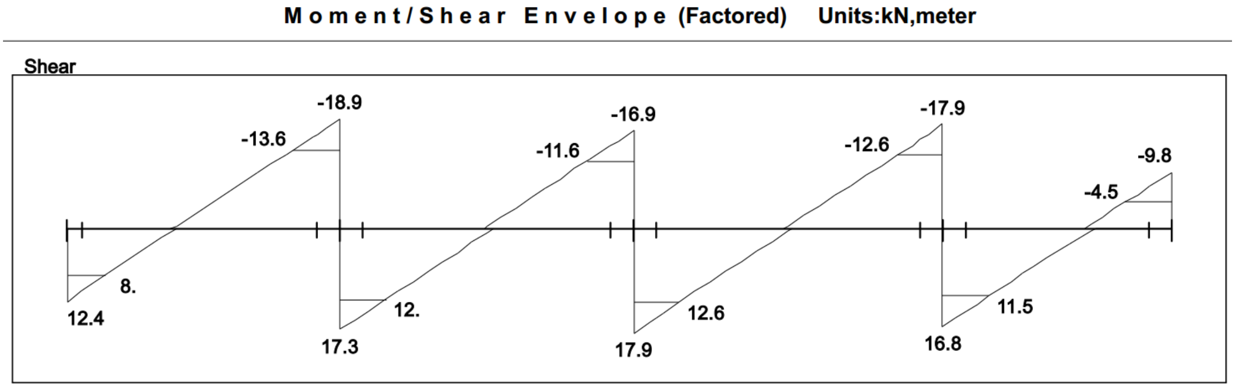
Live load - Service
Load factors: 1.20,1.20/1.60,0.00



Moment/Shear Envelope (Factored) Units:kN,meter

Moments: spans 1 to 4





Reactions

Factored					
DeadR	9.72	28.73	27.27	27.53	7.43
LiveR	2.7	7.46	7.45	7.13	2.33
Max R	12.42	36.19	34.72	34.66	9.76
Min R	9.35	31.7	29.97	30.05	6.88
Service					
DeadR	8.1	23.94	22.72	22.94	6.19
LiveR	1.69	4.66	4.65	4.45	1.46
Max R	9.79	28.6	27.38	27.4	7.65
Min R	7.87	25.79	24.41	24.52	5.84

Figure 5. 4 Shear & Moment Envelope Diagram (RS02)

- Design of positive moment:

M_u = 9 KN.m.

Assume bar diameter ø 12 for main positive reinforcement.

d =h- cover - $d_{stirrups} - \frac{d_b}{2} = 350 - 20 - 8 - \frac{12}{2} = 316\ mm.$

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

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

$= 0.85 \times 24 \times 520 \times 80 \times \left(316 - \frac{80}{2}\right) \times 10^{-6} = 234.22 KN.m$

$M_{nf} \gg \frac{M_u}{\varphi} = \frac{9}{0.9} =$ 10 KN.m

the section will be designed as rectangular section with b_e =520 mm.

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{9\times10^6}{0.9\times520\times316^2} = 0.193 Mpa.$

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

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2\times20.6\times0.193}{420}}\right) = 0.000462$

$A_{s,req}$ = ρ.b.d = 0.000462×520×316= 75.92mm^2

- Check for $A_{s,min}$.

$A_{s,min}$is the maximum of :-

$A_{s,min} = 0.25\frac{\sqrt{f'_c}}{f_y}b_w.d \geq \frac{1.4}{f_y}b_w.d$

3. $A_{s,min} = 0.25\frac{\sqrt{24}}{420}\ 120 \times 316 = 110.6mm^2$
4. $A_{s,min} = \frac{1.4}{420}120 \times 316 = 126.4mm^2\ \ Control$

$A_s = 75.92\ mm^2 \leq A_{s,min} = 126.4mm^2$

Use 2ø10 , $A_{s,provided} = 157\ mm^2 > A_{s,required} = 126.4\ mm^2$. Ok

Check for strain:

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

$c = \frac{a}{\mathcal{B}_1} = \frac{6.22}{0.85} = 7.32\ mm$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{316-7.32}{7.32}\right) = 0.127 > 0.005 \quad 0k$$

- Design of negative moment:

M_u = -7.6 KN.m.

Assume bar diameter ø 12 for main positive reinforcement.

$d = h\text{- cover - } d_{stirrups} - \frac{d_b}{2} = 350 - 20 - 8 - \frac{12}{2} = 316mm.$

$R_n = \frac{M_u}{\emptyset bd^2} = \frac{7.6\times10^6}{0.9\times120\times316^2} = 0.705\ Mpa.$

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

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times0.705}{420}}\right) = 0.00171$

$A_{s,req} = \rho.b.d = 0.00171 \times120\times316 = 64.84\ mm^2$

- Check for $A_{s,min}$.

$A_{s,min}$is the maximum of :-

$A_{s,min} = 0.25\frac{\sqrt{f'_c}}{f_y}b_w.d \geq \frac{1.4}{f_y}b_w.d$

3. $A_{s,min} = 0.25\frac{\sqrt{24}}{420}\ 120 \times 316 = 110.6\ mm^2$
4. $A_{s,min} = \frac{1.4}{420}120 \times 316 = 126.4\ mm^2\ Control$

$A_s = 64.84\ mm^2 \leq A_{s,min} = 126.4mm^2$

Use 2ø10 , $A_{s,provided} = 157\ mm^2 > A_{s,required} = 126.4\ mm^2$. Ok

Check for strain:

$a = \frac{A_{s.} f_y}{0.85 b\, f_c'} = \frac{157 \times 420}{0.85 \times 120 \times 24} = 26.94\, mm$

$c = \frac{a}{\mathcal{B}_1} = \frac{26.94}{0.85} = 31.69\, mm$

$$\varepsilon_s = 0.003 \left(\frac{d-c}{c}\right) = 0.003 \left(\frac{316 - 31.69}{31.69}\right) = 0.0269 > 0.005 \qquad 0k$$

- ✓ Shear Design for (RS02):

V_u at distance d from support= 13.6 KN

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

$V_c = \frac{1.1}{6} \lambda \sqrt{f_c'} b_w d = \frac{1.1}{6} \sqrt{24} \times 120 \times 316 \times 10^{-3} = 34.05 KN$

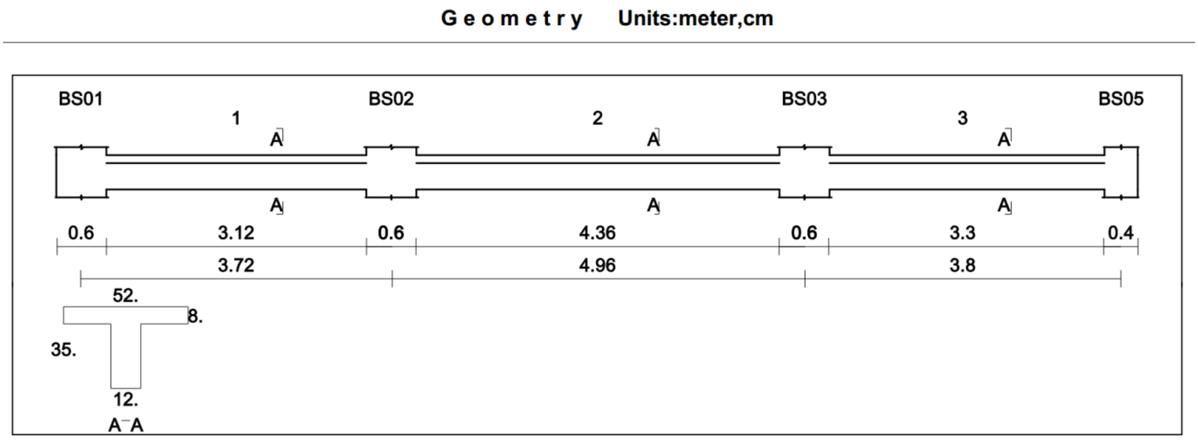
ø V_c =0.75×34.05 =25.55 KN.

0.5 ø V_c =0.5×25.55 =12.78 KN

0.5 ø $V_c < V_u <$ ø V_c ……………. NO

Minimum shear reinforcement is required except for concrete joist construction. So, No shear reinforcement is provided

Rib (RS03)



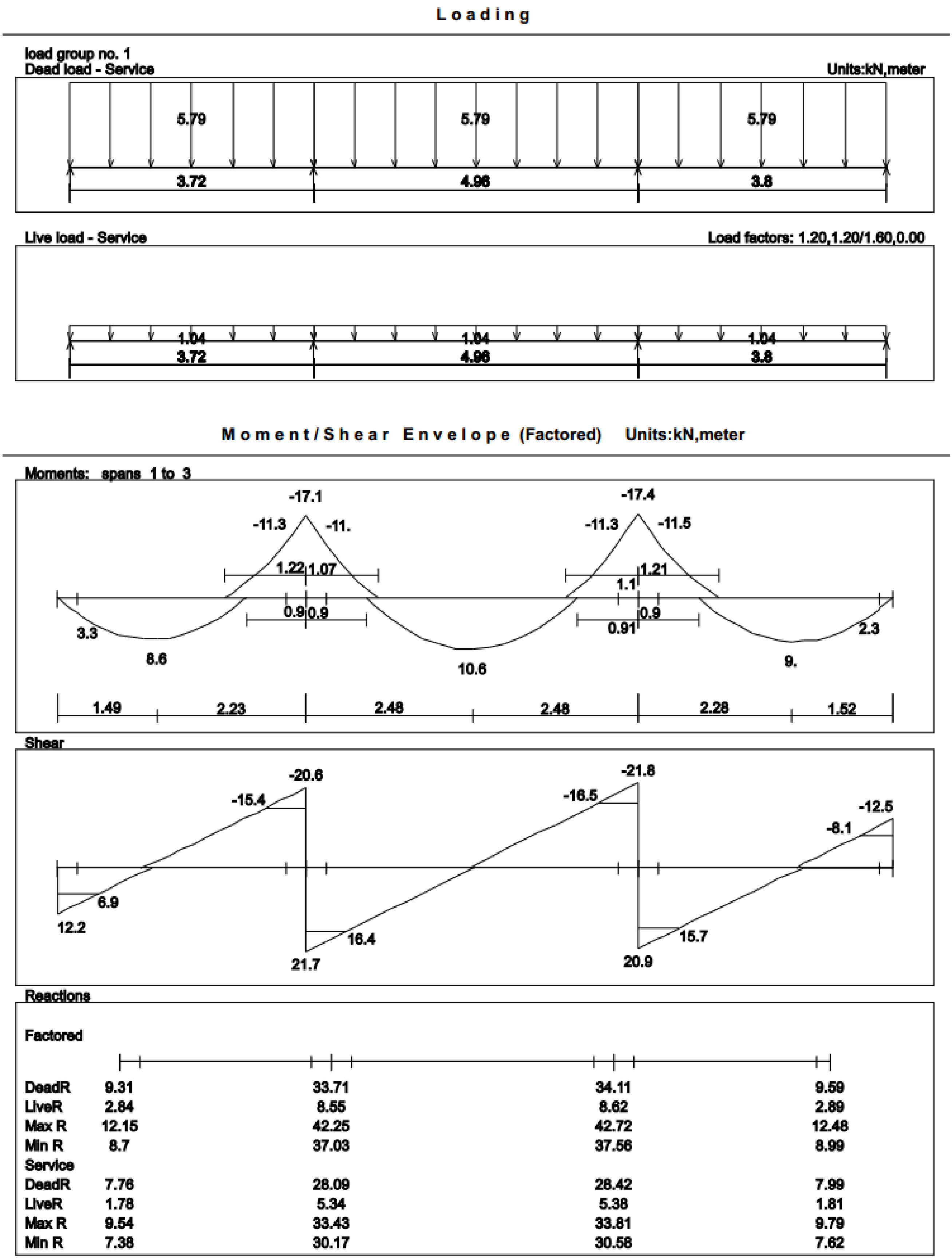


Figure 5. 5 Shear & Moment Envelope Diagram (RS03)

- Design of positive moment:

M_u = 10.6 KN.m.

Assume bar diameter ø 12 for main positive reinforcement.

d =h- cover - $d_{stirrups} - \frac{d_b}{2} = 350 - 20 - 8 - \frac{12}{2} = 316\ mm.$

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

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

$= 0.85 \times 24 \times 520 \times 80 \times \left(316\ - \frac{80}{2}\right) \times 10^{-6} = 234.22 KN.m$

$M_{nf} \gg \frac{M_u}{\varphi} = \frac{10.6}{0.9} =$ 11.78 KN.m

the section will be designed as rectangular section with b_e =520 mm.

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{10.6\times10^6}{0.9\times520\times316^2} = 0.227 Mpa.$

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

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2\times20.6\times0.227}{420}}\right) = 0.000544$

$A_{s,req}$ = ρ.b.d = 0.000544×520×316= 89.39mm^2

- Check for $A_{s,min}$.

$A_{s,min}$ is the maximum of :-

$A_{s,min} = 0.25 \frac{\sqrt{f'_c}}{f_y} b_w.d \geq \frac{1.4}{f_y} b_w.d$

5. $A_{s,min} = 0.25 \frac{\sqrt{24}}{420}\ 120 \times 316 = 110.6 mm^2$

6. $A_{s,min} = \frac{1.4}{420} 120 \times 316 = 126.4 mm^2\ \ Control$

A_s = 89.39 mm^2 ≤$A_{s,min}$ = $126.4 mm^2$

Use 2ø10 , $A_{s,provided}$= 157 mm^2>$A_{s,required}$= 126.4 mm^2. Ok

Check for strain:

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

$c = \frac{a}{\mathcal{B}_1} = \frac{6.22}{0.85} = 7.32\ mm$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{316 - 7.32}{7.32}\right) = 0.127 > 0.005 \quad 0k$$

- Design of negative moment:

M_u = -11.5 KN.m.

Assume bar diameter ø 12 for main positive reinforcement.

$d = h$- cover - $d_{stirrups} - \frac{d_b}{2} = 350 - 20 - 8 - \frac{12}{2} = 316mm.$

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{11.5\times10^6}{0.9\times120\times316^2} = 1.066\ Mpa.$

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

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times1.066}{420}}\right) = 0.00261$

$A_{s,req} = \rho.b.d = 0.00261 \times120\times316 = 98.97\ mm^2$

- Check for $A_{s,min}$.

$A_{s,min}$ is the maximum of :-

$A_{s,min} = 0.25\frac{\sqrt{f'_c}}{f_y} b_w.d \geq \frac{1.4}{f_y} b_w.d$

5. $A_{s,min} = 0.25\frac{\sqrt{24}}{420}\ 120\times316 = 110.6\ mm^2$
6. $A_{s,min} = \frac{1.4}{420} 120\times316 = 126.4\ mm^2\ Control$

$A_s = 98.97\ mm^2 \leq A_{s,min} = 126.4 mm^2$

Use 2ø10 , $A_{s,provided} = 157\ mm^2 > A_{s,required} = 126.4\ mm^2$.   Ok

Check for strain:

$a = \frac{A_s.f_y}{0.85 b\ f_c'} = \frac{157\times420}{0.85\times120\times24} = 26.94\ mm$

$c = \frac{a}{\beta_1} = \frac{26.94}{0.85} = 31.69\ mm$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{316-31.69}{31.69}\right) = 0.0269 > 0.005 \qquad 0k$$

✓ Shear Design for (RS03):

V_u at distance d from support= 16.5 KN

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

$V_c = \frac{1.1}{6}\ \lambda\sqrt{f_c'} b_w d = \frac{1.1}{6}\sqrt{24}\ \times 120\times316\times10^{-3} = 34.05KN$

ø V_c =0.75×34.05 =25.55 KN.

0.5 ø V_c =0.5×25.55 =12.78 KN

0.5 ø V_c < V_u < ø V_c ……………. NO

Minimum shear reinforcement is required except for concrete joist construction. So, No shear reinforcement is provided

5.6 Design of Beam(BS10) :

✓ Load calculations:

Load calculations for BS10:

Dead Load Calculations for Beam(BS10):-

Table (5. 4)Dead Load Calculations for Beam(BS10)

Dead load from:	h×γ×1	KN/m
Tiles	0.03×23×1	0.69
Mortar	0.03×22×1	0.66
Coarse sand	0.07×17×1	1.19
Reinforced concrete	0.35×25×1	8.75
Plaster	00.2×22×1	0.44
	∑	11.7 KN/m

The distributed Dead and Live loads acting upon BS10 can be defined from the support reactions of the RS01, RS02

From RS01 & RS02

The maximum support reaction (Service) from Dead Loads for RS01 upon BS10 is 22.86 KN . The distributed Dead Load from the RS01 on BS10:

DL =22.86/ 0.52= 43.96 KN/m

The maximum support reaction (Service) from Dead Loads for RS02 upon BS10 is 22.94 KN . The distributed Dead Load from the RS02 on BS10:

DL =22.94/ 0.52= 44.12 KN/m

Live Load calculations: The maximum support reaction (Service) from Live Loads for RS01 upon BS10 is 4.43 KN .

The distributed Live Load from the RS01 on BS10:

LL =4.43/ 0.52= 8.52 KN/m

Live Load calculations: The maximum support reaction (Service) from Live Loads for RS02 upon BS10 is 4.45 KN .

The distributed Live Load from the RS02 on BS10:

LL =4.45/ 0.52= 8.56 KN/m

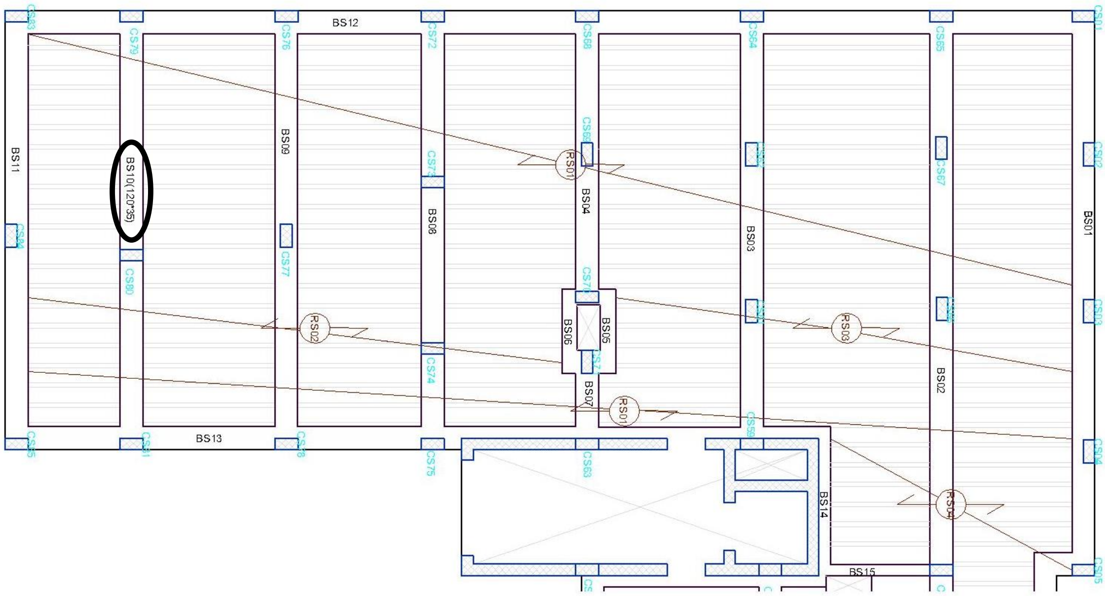
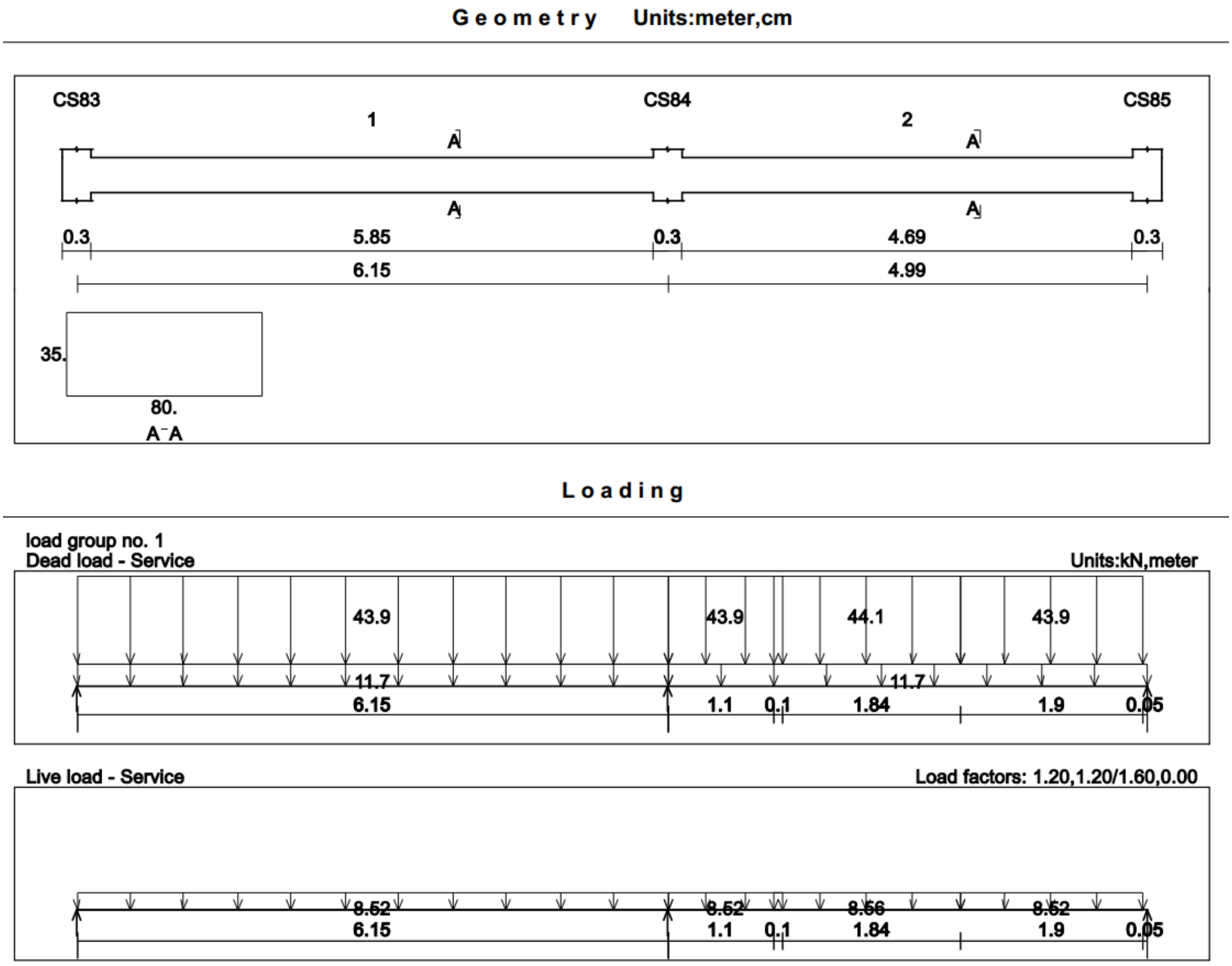


Figure 5. 6 BS10



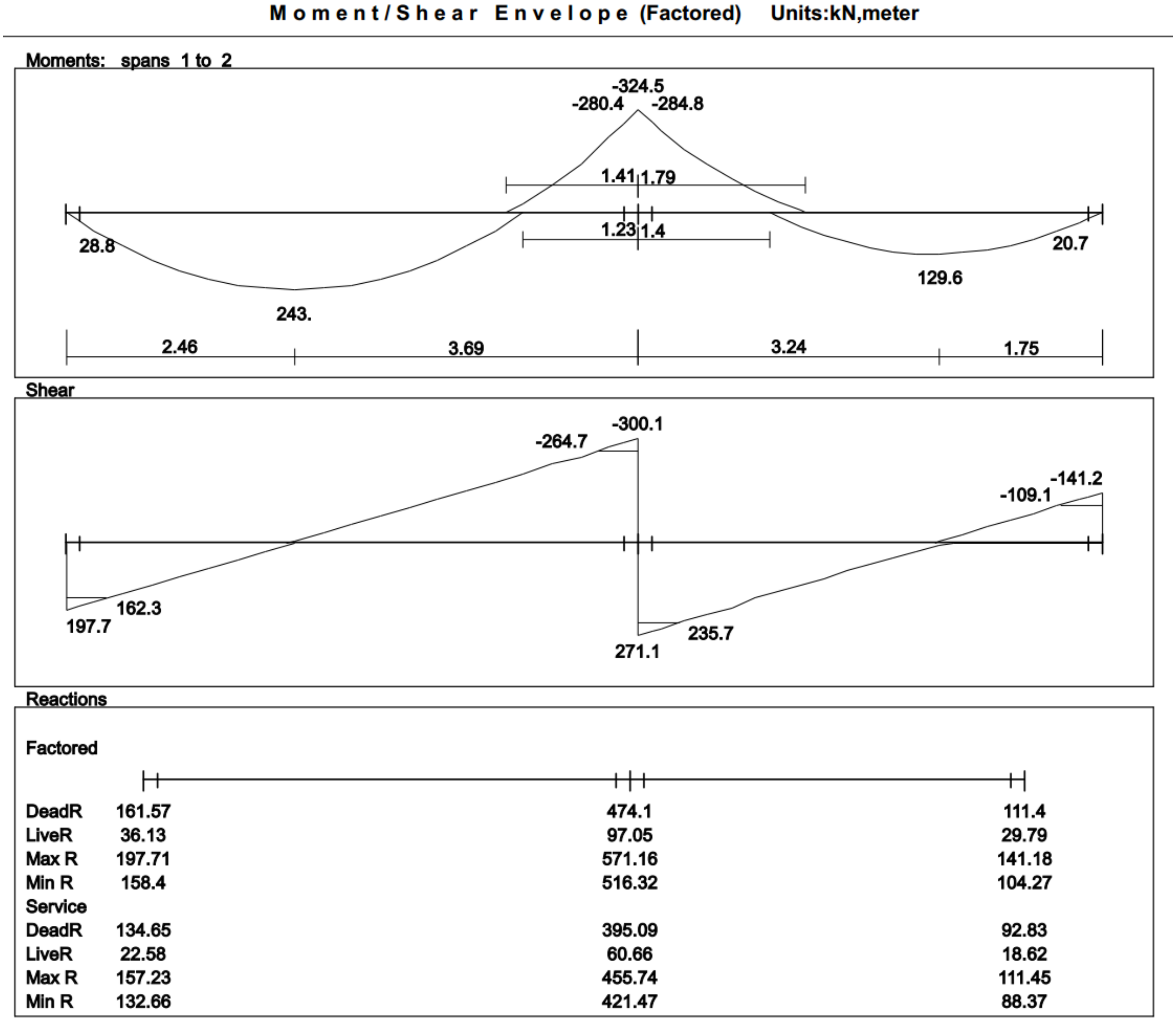


Figure 5. 7Loading and Moment /Shear Envelope.

- ✓ Flexural Design for (BS10) :

Determine of $M_{n,max}$:

$$d = 350 - 40 - 8 - \frac{18}{2} = 293\ mm$$

$$c = \frac{3}{7}d = \frac{3}{7} \times 293 = 125.75\ \ mm$$

$$a = \mathcal{B}.c = 125.75\ \times 0.85 = 106.736mm$$

Mn_{max} = $0.85f_c'ab(d - \frac{a}{2})$ = 0.85×24×106.736×1000×(293-106.736/2) ×10^{-6}= 521.778 KN.m

∅ Mn_{max} = 0.82× 313.07 = 427.858 KN.m> -284.8

Design as singly reinforcement

Design for positive moment :

1) $\boldsymbol{Mu\ =\ 243}$

$$Rn = \frac{M_u}{\emptyset b d^2} = \frac{243 \times 10^6}{0.9 \times 1200 \times 293^2} = 2.62\ Mpa.$$

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

ρ= $\frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2\times20.6\times2.62}{420}}\right) = 0.00670$

A_s = ρ.b.d = 0.00670×1200×293 =2355.72mm^2

Check for $A_{s,min}$.

$A_{s,min}$ =$0.25 \frac{\sqrt{f'_c}}{f_y} b_w.d \geq \frac{1.4}{f_y} b_w.d$

$A_{s,min}$ =$0.25 \frac{\sqrt{24}}{420}\ 1200 \times 293 = 1025.28\ mm^2$

$A_{s,min}$ =$\frac{1.4}{420} 1200 \times 293 = 1172\ mm^2\ Control.$

$A_{s,min=}$1172mm^2< A_s =2355.72 mm^2

Use10ø 18 Bottom, $A_{s,provided}$= 2545mm^2>$A_{s,required}$= 2396.74 mm^2. Ok

Check spacing :

S_{max}= $380\left(\frac{280}{f_y}\right) - 2.5C_c = 203.33\ control \quad OR \quad S = 300\left(\frac{280}{f_s}\right) = 200$

$S = \frac{1200-40\times2-10\times2-(18\times10)}{9} = 102.22mm > 25\ \ ...\ \ OK$

$< 203.33\ ...\ OK$

Check for strain:

a =$\frac{A_s.f_y}{0.85b\ f_c'} = \frac{2545\times420}{0.85\times1200\times24} = 43.66\ mm$

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

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{293-51.36}{51.36}\right) = 0.0141 > 0.005 \qquad 0k$$

2) M_u = 129.6KN.m

R_n= $\frac{M_u}{\emptyset b d^2} = \frac{129.6\times10^6}{0.9\times1200\times293^2} = 1.398 Mpa.$

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

ρ= $\frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2\times20.6\times1.677}{420}}\right) = 0.00345$

A_s = ρ.b.d = 0.00345 ×1200×293= 1212.02 mm^2.

Check for $A_{s,min}$.

$A_{s,min} = 0.25\frac{\sqrt{f'_c}}{f_y}b_w.d \geq \frac{1.4}{f_y}b_w.d$

$A_{s,min} = 0.25\frac{\sqrt{24}}{420}\ 1200 \times 293 = 1025.28\ mm^2$

$A_{s,min} = \frac{1.4}{420}1200 \times 293 = 1172\ mm^2\ Control.$

$A_{s,min}$=1212.02 mm^2>A_s = 1172 mm^2

Use 5ø 18 Bottom, $A_{s,provided}$= 1272.5 mm^2>$A_{s,required}$= 1212.02 mm^2. Ok

Check spacing :

$S = \frac{1200-40\times2-2\times10-(5\times18)}{4} = 252.5\ mm > 25$

$> S_{max}$

Use 5ø 18 Bottom, $A_{s,provided}$= 1781.5 mm^2>$A_{s,required}$= 1212.02 mm^2. Ok

$S = \frac{1200-40\times2-2\times10-(7\times18)}{6} = 162.3\ mm > 25 \ldots ok$

Check for strain:

$a = \frac{A_s.f_y}{0.85b\ f_c'} = \frac{1272.5\ \times420}{0.85\times1000\times24} = 26.2\ mm$

$c = \frac{a}{\mathcal{B}_1} = \frac{26.2}{0.85} = 30.82\ mm$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{293-30.82}{30.82}\right) = 0.0255 > 0.005 \qquad 0k$$

<u>Design for Negative moment :</u>

M_u = -284.8KN.m

$R_n = \frac{M_u}{\emptyset bd^2} = \frac{284.8\times10^6}{0.9\times1200\times293^2} = 3.083Mpa.$

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

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times3.083}{420}}\right) = 0.008$

$A_s = \rho.b.d = 0.008\times1200\times293$= 2812. 8 mm^2.

Check for $A_{s,min}$.

$A_{s,min} = 0.25\frac{\sqrt{f'_c}}{f_y}b_w.d \geq \frac{1.4}{f_y}b_w.d$

$A_{s,min} = 0.25\frac{\sqrt{24}}{420}\ 1200 \times 293 = 1025.28\ mm^2$

$A_{s,min} = \frac{1.4}{420}1200 \times 293 = 1172\ mm^2\ Control.$

$A_{s,min}$= 1172 mm^2< A_s = 2812.8 mm^2

Use 12 ø 18Top , $A_{s,provided}$= 3054 mm^2>$A_{s,required}$= 2812. 8mm^2.   Ok

Check spacing :

$S = \frac{1000-40*2-2*10-(18\times12)}{11} = 62.18\ mm > 25\ \ldots.\quad OK$

Check for strain:

$a = \frac{A_{s.}f_y}{0.85b\ f_c'} = \frac{2859.68\times420}{0.85\times1200\times24} = 49.06mm$

$c = \frac{a}{\mathcal{B}_1} = \frac{49.06}{0.85} = 57.72mm$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{293-57.72}{57.72}\right) = 0.0122 > 0.005 \qquad 0k$$

- ✓ Shear Design for (BS10):

1. Vu = 264.7, 235.7 KN

$V_c = \frac{1}{6}\sqrt{fc'}b_w\ d == \frac{1}{6}\sqrt{24}*1200*293*10^{-3} = 257.08$KN

Φ V_c= 0.75*257.08 =215.31 KN

$V_{s,min} = \frac{1}{3}b_w\ d == \frac{1}{3}\ 1200*\ 293*10^{-3} = 117.19\ KN$control

$V\ _{s,min} = \frac{1}{16}\sqrt{fc'}b_w\ d = \frac{1}{16}*\sqrt{24}*1200*293*10^{-3} = 74.75KN$

$V_{s'} = \frac{1}{3}\sqrt{fc'}b_w\ d = \frac{1}{3}\sqrt{24}*1200*293*10^{-3} = 574.15$

$$\emptyset V_c < V_u \leq \emptyset(V_c + V_{s,\mathrm{min}})$$

215.31 < 264.7, 235.7 < 0.75(257.08+117.19)

252.66 < 264.7 < 280.7 …ok

shear reinforcement are required .

Use 4 leg Φ 8 .

Av =201.2 mm^2 .

$V_s = V_n - V_c = \frac{264.7}{0.75} - 257.08 = 95.85$ KN

$$S = \frac{A_v f_{yt}\ d}{v_s} = \frac{201.2*420*293}{75.05*1000} = 210.71\ mm$$

$$s_{max} \leq \frac{d}{2} = \frac{293}{2} = 146.5\ mm \qquad (control) \qquad or\ s_{max} \leq 600\ mm$$

Use 4 leg Φ 8 @150 mm .

2. V_u = 162.3 KN, 109.1

$V_c = \frac{1}{6}\sqrt{fc'}b_w\, d == \frac{1}{6}\sqrt{24} * 1200 * 293 * 10^{-3} = 257.08$KN

Φ V_c= 0.75*257.08 =215.31 KN

$\frac{1}{2}$ Φ V_c= 0.5×179.42 =107.655 KN

$V_{s,min} = \frac{1}{3}b_w\, d == \frac{1}{3}\ 1200 *\ 293 * 10^{-3} =\ 117.19\ KN$control

$V_{\ s,min} = \frac{1}{16}\sqrt{fc'}b_w\, d = \frac{1}{16} * \sqrt{24} * 1200 * 293 * 10^{-3} = 74.75KN$

$V_{s'} = \frac{1}{3}\sqrt{fc'}b_w\, d = \frac{1}{3}\sqrt{24} * 1200 * 293 * 10^{-3} = 574.15$

$$\frac{1}{2}\emptyset V_c < V_u \leq \emptyset V_c$$

107.655< 162.3,109.1 ≤ 215.31

Minimum shear reinforcement is required .

$$\frac{A_s}{s} = \frac{1}{3}\cdot\frac{b}{f_y} \to \frac{201.2}{s} = \frac{1}{3}\cdot\frac{1200}{420} \to s = 211.26\ mm$$

$$\frac{A_s}{s} = \frac{1}{16}\cdot\sqrt{\acute{f_c}}\frac{b}{f_y} \to \frac{100.6}{s} = \frac{1}{16}\cdot\sqrt{24}\cdot\frac{1200}{420} \to s = 229.99\ mm$$

$$s_{max} = \frac{d}{2} = \frac{293}{2} = 146.5mm\quad CONTROL \qquad or\ \ 600$$

Use 4 leg Φ 8 @150 mm .

5.7 Design of Basement wall

4.6.1 Load Calculation:-

$\gamma = soildensity =\ 18KN/m^3.$

$\emptyset = angleofinternalfriction = 30°.$

LL=5 KN/m^2 .

Thickness = 30cm, cover = 4cm .

The design will be for 1m width .

Neglect the axial load, since its low value

$q1 = soilpressure = Ko * \gamma * h.$

$q2 = surchargepressure =\ Ko * LL.$

$Ko = soilpressurecoefficientatrest =\ \ 1 - \sin\emptyset.$

So ,

$Ko = 1 - \sin\emptyset = 0.5.$

$q1 = 0.5 * 19 * 2.70 = 25.65\frac{KN}{m^2}.$

$$q2 = 0.5 * 5 = \quad 2.5\frac{KN}{m^2}.$$

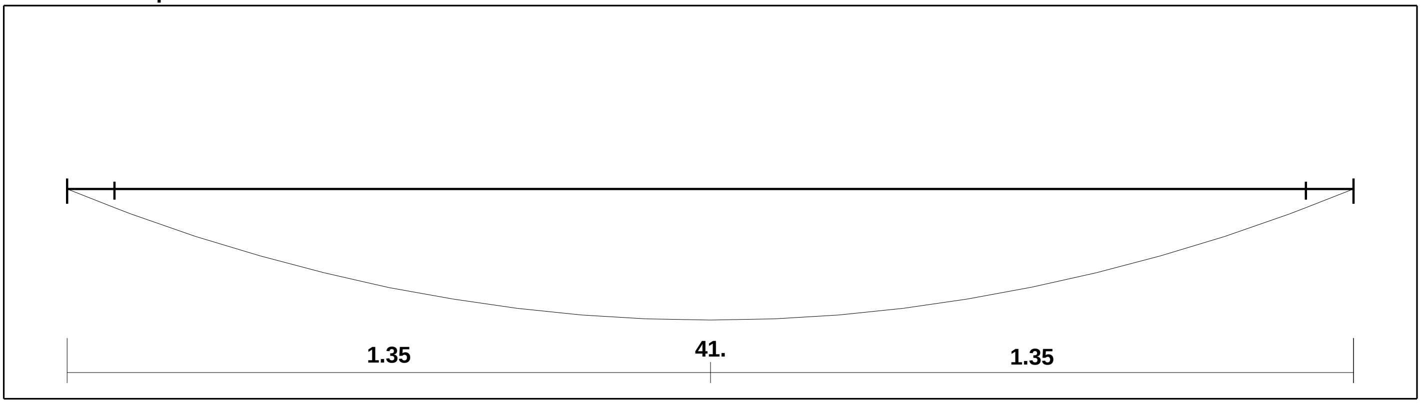
Factored Load :-

$$q1u = 25.65 * 1.6 = 41.04\ KN/m^2$$

$$q2u = 2.5 * 1.6 = 4\ KN/m^2$$

Moment/Shear Envelope (Factored) Units:kN,meter

Moments: spans 1 to 1



Shear

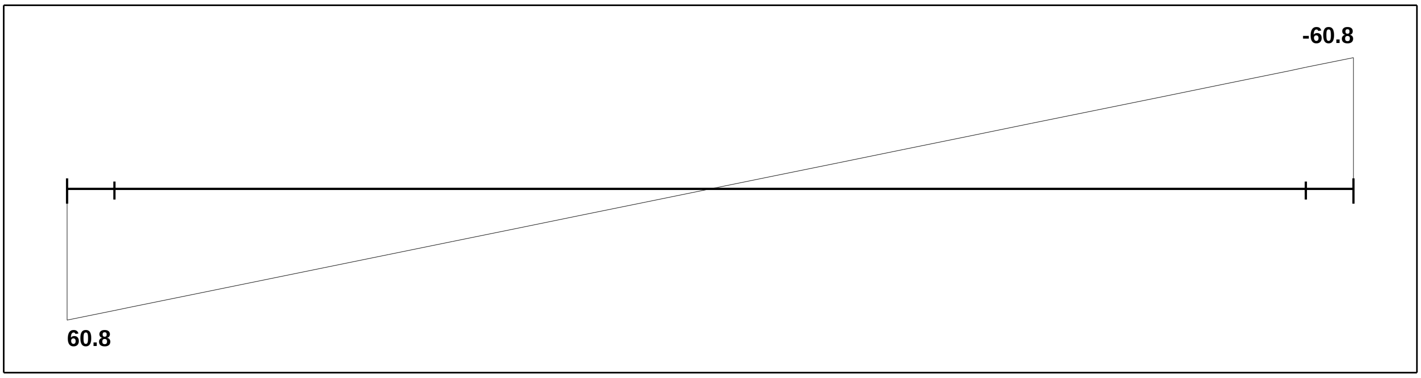


Figure 5. 8 Loading and Moment /Shear Envelope

Design of bending moment of wall :-

Design for positive moment Mu =41 KN.m .

$$d = 300 - 40 - \frac{16}{2} = 252\ mm.$$

$$Mn = \frac{Mu}{0.9} = \frac{41}{0.9} = 45.56\ KN.m$$

$$Rn = \frac{Mn * 10^6}{b * d^2} = \frac{45.56 * 10^6}{1000 * 252^2} = 0.717\ Mpa.$$

$$m = \frac{Fy}{0.85 * fc'} = \frac{420}{0.85 * 25} = 19.76$$

$$\rho = \frac{1}{m} * \left(1 - \sqrt{1 - \frac{2 * Rn * m}{Fy}}\right) = \frac{1}{19.76} * \left(1 - \sqrt{1 - \frac{2 * 1.25 * 19.76}{420}}\right)$$

$$= 1.74 * 10^{-3}$$

$Asreq \ = \rho * b * d = 1.74 \ * 10^{-3} * 1000 * 252 = 437.71 \, mm^2/m. \dots . control.$

$Asminv = 0.0012 * b * h = 0.0012 * 1000 * 300 = 360 \, mm^2/m.$

$$Asminforflexture \ = 0.25 * \frac{\sqrt{fc'}}{fy} * bw * d = 0.25 * \frac{\sqrt{25}}{420} * 1000 * 252$$

$$= 750 \, mm^2/m.$$

$$Asminforflexture \ = \frac{1.4}{fy} * bw * d = \frac{1.4}{420} * 1000 * 252 = 840 mm^2/m \dots . control.$$

For inside wall Select $\emptyset 12@25cm = 452.4 \, mm^2 > 437.71 mm^2$.

For outside wall Select $\emptyset 12@12.5cm = 904 mm^2 > 840 mm^2$.

4.12.3 Design of shear force :-

$d = 300 - 40 - 8 = 252mm$

$$\emptyset Vc = 0.75 * \frac{1}{6} * \sqrt{fc'} * b * d = 0.75 * \frac{1}{6} * \sqrt{25} * 1000 * 252 * 10^{-3} = 157.5 \, KN.$$

$(\emptyset Vc = 157.5) > (Vu = 60.8)$.

No shear Reinforcement is required and thickness of wall is adequate enough.

But horizontal Reinforcement due to Cracking:

$Asreqh = 0.002 * b * h = 0.002 * 1000 * 300 = 600 mm^2/m.$

For one side $As = 300 \, mm^2/m.$

Select for one side horizontal reinforcement $\emptyset 10@25cm = 314.16 \, mm^2 > 300 \, mm^2$

5.8 Design of Stair:

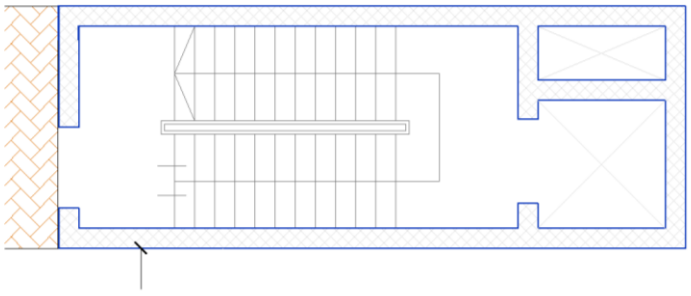


Figure 5.9: Stair Plan.

✓ Material :-

⇒ concrete B300 $Fc' = 24\ N/mm^2$

⇒ Reinforcement Steel $Fy = 420\ N/mm^2$

✓ Design of Flight :-

✓ Determination of Thickness:-

$h_{min} = L/20$

$h_{min} = 5.8/20 = 29$ cm

Take h = 30cm

The Stair Slope by $\theta = \tan^{-1}(15/30) = 26.56^o$

✓ Load Calculation:-

Dead Load For Flight For 1m Strip:-

Table5-5:Dead Load Calculation of Flight.

No.	Parts of Flight	Calculation	
1	Tiles	27*0.03*1*(0.35+0.15/0.3) = 1.35 KN/m	
2	Mortar	22*0.02*1*(0.3+0.15/0.3) = 0.66 KN/m	
3	Stair	25*1*(0.3+0.15/2)/0.3 = 1.875 KN/m	
4	Slab	25*0.25*1 / COS 26.56 = 6.99 KN/m	
5	Plaster	22*0.03*1 / COS 26.56^o = 0.738 KN/m	
		Sum	13 KN/m

Live Load For Landing For 1m Strip = 4*1 = 4 KN/m

Factored Load For Flight :-

$W_U = 1.2 \times 13 + 1.6 \times 4 = 23.6$ KN/m

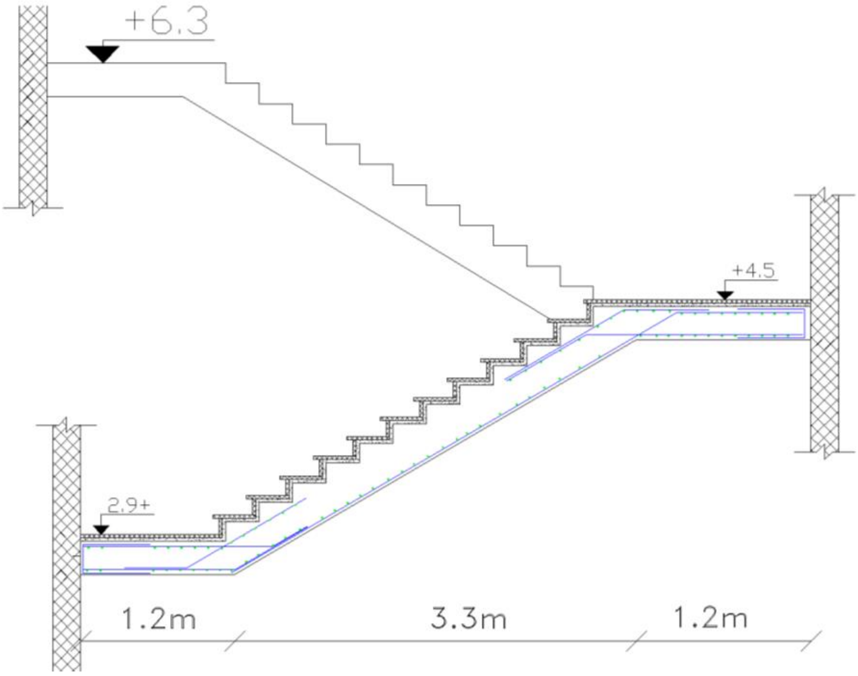


Fig 5.10: Stair Section.

R = (W*L)/2=23.6*3.31/2=42.3 KN

1- Design of Shear for Flight :- (Vu=27.45 KN)

Assume bar diameter ø 14 for main reinforcement

d =h- cover $-\frac{d_b}{2} = 300 - 20 - \frac{14}{2} = 273mm$

Vu = 42.3KN

$V_c = \frac{1}{6}\sqrt{fc'}b_w\, d\ = \frac{1}{6}\sqrt{24} * 1000 * 273\ = 222.9\text{KN}/m$

$\Phi\ V_{c\,=}$ 0.75* 222.9 = 167.2KN /m

Vu = 42.3 <$\Phi\ V_{C\,=}$ 0167.2KN /m

<u>The thickness is enough .</u>

2- Design of Bending Moment for Flight :- (Mu=80.25KN.m)

$$\text{Mu}= \mathbf{80.25} * (\mathbf{1.5} + \mathbf{1.6}) - \frac{\mathbf{23.6*1.35^2}}{\mathbf{2}} = \mathbf{89.1}.\boldsymbol{m\ KN}$$

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{89.1\times10^6}{0.9\times1000\times273^2} = 1.2\ \ Mpa$

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

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2\times20.59\times1.2}{420}}\right) = 0.00294$

$A_{s,req}$ = ρ.b.d = 0.00294 ×1000×273 = 804.2mm^2

$A_{s,min}$= 0.0018*1000*300 = 540 mm^2

As_{req}>$A_{s,min}$=540 mm^2

As_{req}=804.2 mm^2

Check for Spacing :-

1) S = 3h = 3*300 =900 mm
2) S = 380*(280/(2/3 * 420)) – 2.5*20 = 330 ≤ S = 300*(280/(2/3 * 420)) = 250mm
3) S = 450 mm

S = 250 mm ……… is control

Use ø14 @ 250 mm

3- Lateral or Secondary Reinforcement For Flight :-

$A_{s,req}$= $A_{s,min}$ =0.0018*1000*300= 540mm^2

Use ø14@ 300 mm ,$A_{s,provided}$= 461.7 mm^2>$A_{s,required}$= 450mm^2… Ok

- ✓ Design of Landing :
- ✓ Load Calculation:-

Dead Load For Landing For 1m Strip:-

Live Load For Landing For 1m Strip = 4*1 = 4KN/m

Factored Load For Landing :-

W_U = 1.2 ×9.26+ 1.6×4=19.3KN/m

- ✓ System of Landing :-

$$R= \frac{\mathbf{19.3*1.7}}{\mathbf{2}} + \mathbf{21.15*1.9 = 75.45}\ \boldsymbol{KN}$$

1- Design of Shear:- (Vu=75.45 KN)

Assume bar diameter ø 14 for main reinforcement

$$d = h - cover - \frac{d_b}{2} = 300 - 20 - \frac{14}{2} = 273mm$$

$$V_c = \frac{1}{6}\sqrt{fc'}b_w\, d == \frac{1}{6}\sqrt{24} * 1000 * 273 = 222.9\text{KN}$$

Φ* V_c = 0.75* 222.9= 1167.2 KN> Vu = 75.45 KN…... Thickness of slab is enough

2- Design of Bending Moment :- (Mu=70.6KN.m)

Assume bar diameter ø 14 for main reinforcement

$$d = h - cover - \frac{d_b}{2} = 250 - 20 - \frac{14}{2} = 273mm$$

$$R_n = \frac{M_u}{\emptyset b d^2} = \frac{70.6\times10^6}{0.9\times1000\times273^2} = 1.052\ Mpa$$

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

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.59}\left(1-\sqrt{1-\frac{2\times 20.59\times 0.382}{420}}\right) = 0.002575$$

$A_{s,req} = \rho.b.d = 0.002575\times1000\times273 = 703\ mm^2$

$A_{s,min} = 0.0018*1000*273 = 540mm^2$

$A_{s,req} = 703\ mm^2$……… is control

Check for Spacing:-

4) S = 3h = 3*300 =900mm

5) S = 380*(280/(2/3 * 420)) – 2.5*20 = 330 ≤ S = 300*(280/(2/3 * 420)) = 200mm

6) S = 450 mm

S = 200 mm ……… is control

Use ø14 @ 200 mm

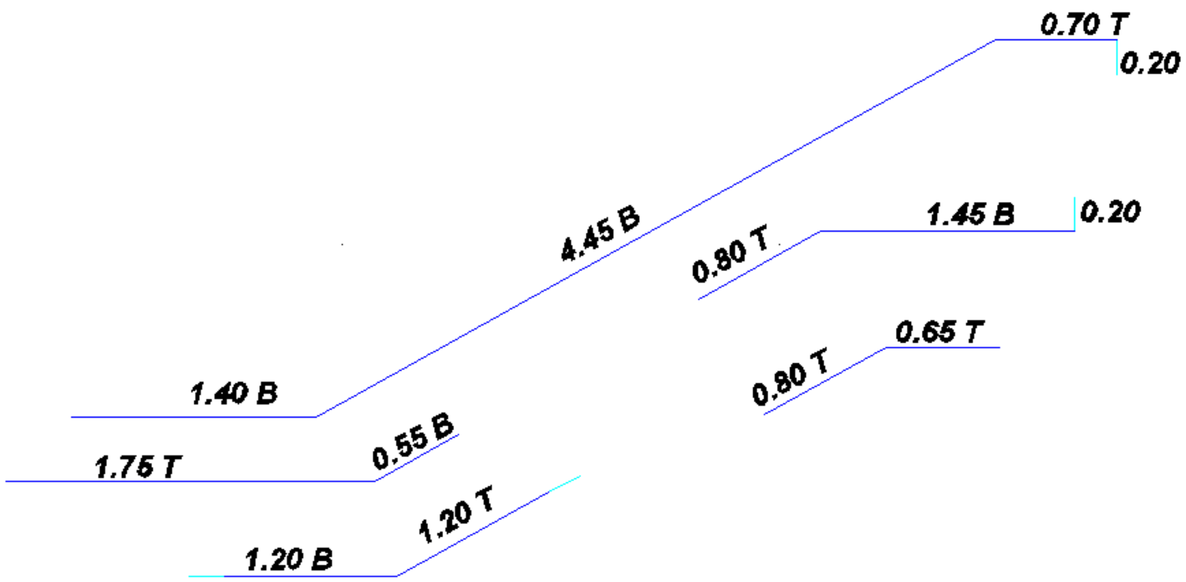


Fig 5.11: Stair Reinforcement.

5.9 Design of Column:

Column : C-9……. (within basement floor)

PD= 537.18KN

PL = 78.28KN

Use Fc' = 24 Mpa

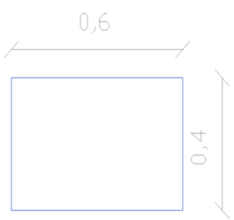


Fig 5.12:section of column -19

$$P_u = 1.2DL + 1.6LL = 1.2\times537.18 + 1.6\times78.28 = 769.864KN$$

$$P_n = \frac{P_u}{\phi} = \frac{769.864}{0.65} = 1184.4\ KN \Rightarrow \phi = 0.65 - for\ tied\ column$$

Assume rectangular section with:

$\rightarrow Use\ \rho\ =\ 2.5\ \%$

$P_n\ = 0.8 \times A_g\left(0.85 \times f_c' + \rho_g\left[f_y - 0.85 \times f_c'\right]\right)$

$\rightarrow Use\ 0.8\ for\ tied\ column$

$1184.4 * 10\text{^}3 = 0.8 \times A_g(0.85 \times 24 + 0.025[420 - 0.85 * 24])$

$A_g\ = 48716.68mm^2$

$\rightarrow Use 0.6 \times 0.4m^2 with A_g = 240000mm^2 > A_{g.required} = 48716.68mm^2$

$1)\ Check\ for\ Slenderness:$

$$\frac{K \times l_u}{r} \leq 34 - 12\left(\frac{M_1}{M_2}\right) \leq 40$$

$$\left(\frac{M_1}{M_2}\right) = 1 - for\ braced\ frame\ with\ M_{min}.$$

l_u: *Actual unsupported (unbraced) length.*

r: *radius of gyration of its crosssection* $=\ 0.3h$

$l_u\ = 2.6m$

$K = 1.0 - for\ columns\ in\ nonsway\ frame.$

$a)\ In\ 40cm\text{–}Dirction:$

$$\frac{K \times l_u}{r} \leq 34 - 12 \times 1.0 = 22 < 40$$

$$\frac{K \times l_u}{r_x} = \frac{1 \times 2.6}{0.3 \times 0.4} = 21.6 < 22$$

$\therefore short\ Column\ for\ bending\ about\ X - axis.$

$b)\ In\ 600cm\text{–}Dirction:$

$$\frac{K \times l_u}{r} \leq 34 - 12 \times 1.0 = 22 < 40$$

$$\frac{K \times l_u}{r_y} = \frac{1 \times 2.6}{0.3 \times 0.6} = 14.4 < 22$$

$\therefore short\ Column\ for\ bending\ about\ Y - axis.$

Selecting Longitudinal Bars:

$4000*1000 = 0.65 \text{x}\ 0.8 \times \text{Ag}\ \{0.85*24(537.18 - Ast) + Ast*420\}$

Ast=2735mm2

Use12ϕ25 ,Ast,prov=5890.4mm^2>Ast=2735mm^2

ρg =Ast/Ag= 0.00114

- ✓ **Design of the tie reinforcement :**

S ≤16 db (longitudinal bar diameter)

S≤ 48dt (tie bar diameter).

S ≤ Least dimension.

spacing ≤16×d_b=16×2.5=40cm ….

spacing ≤48×dt=48×1.0 = 48 cm

spacing ≤least.dim=40 cm control

Useϕ10@25 cm

5.10 Design of Isolated Footing (F11):

P_D = 690KN (service).

P_L = 220KN (service).

P_U =1180 KN (factored).

Column Dimensions = a*b=35*35 cm

Allowable bearing capacity q_{all}= 350 KN/m^2.

Area of Footing:

Soil Density = 18 KN/ m ^{3}

assume h = 40 cm.

$q_{all \cdot net}$ = 350 - 5 - 0.4×25- 0.11×18 = 315.2 KN/m ^{2}

$$\text{Area} \quad A = \frac{PD + PL}{qall\ln et} = \frac{832.96 + 55.73}{379.9} = 2.88$$

Use B,L(min) = 1.7m,take B =L= 1.8 m, A = 3.24m ^{2} for foundation.

Depth of footing:

Assume h = 40 cm.

Check one way shear:

$$= \frac{1180}{3.24} = 364.2 \text{ KN/ m}^2 . q_{ult} = \frac{P_u}{Area}$$

d =400 – 75 – 20 = 305 mm

$$= \frac{0.75}{6} * \sqrt{24} * 2.4 * 0.305 * 1000 = 336.19KN \quad \Phi V_c = \Phi \frac{1}{6}\sqrt{f_c'}\, b_w d$$

$$Vu = q_{ult} \times \left(\frac{B-a}{2} - d\right) \times L$$

$$Vud = 364.2 \times \left(\frac{1.8 - 0.35}{2} - 0.305\right) \times 1.8 = 275.34KN$$

øV_c =336.19 KN >V_{ud} = 275.34KN →→→→ *ok*

- Check two-way shear:

$$\frac{d}{2} = \frac{505}{2} = 252.5mm.$$

To calculate V_u at the critical section which take rectangular shape with dimension equal (0.35 +0.305) equals (0.655)

Inner area =0.655* 0.655=0.429m ^{2}

Outer area = area of the footing = 3.24 m^2

V_u = q_u (outer area – inner area) = 364.2 ×(5.76–0.8090) =1023.77 KN

According to ACI , Vc shall be the smallest of :

$$= 0.5\sqrt{f_c'}\, b_o d \quad V_c = \frac{1}{6}\left(1+\frac{2}{\beta_c}\right)\sqrt{f_c'}\, b_o d$$

$$= 0.447\sqrt{f_c'}\, b_o d \quad V_c = \frac{1}{12}\left(\frac{\alpha_s}{b_o/d}+2\right)\sqrt{f_c'}\, b_o d$$

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

Where:

= a / b = 35 /35 = 1 β_c

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

= 2×(0.655+0.655)= 3. 62 m

= 40 for interior column. α_s

$$\phi V_c = 0.75 \times 0.33\sqrt{24} \times 3.62 \times 0.305 \times 1000 = 1338.7KN$$

$$\varphi Vc = 1338.7\ KN > \ Vu \ = 1023.77\ KN$$

SO h = 40 cm Is OK.

Design of flexural reinforcement both directions :

$$\text{Mu} = \left(q_{ult} \times L \times \left(\frac{C^\wedge 2}{2}\right)\right)$$

$$= (364.2 \times 1.8 \times 1 \times 1) \div 2 \ = 163.89\text{KN.m}$$

Mn = 163.89/0.9 = 182.1KN.m.

$$\text{Rn} = \frac{Mn}{b * d^2}$$

$$\text{Rn} = \frac{252 * 10^6}{2400 * (505)^2} = 1.088\text{Mpa}$$

$= \dfrac{fy}{0.85 * fc'} = \dfrac{420}{0.85 * 24} = 20.6\,m$

$\rho = \dfrac{1}{m}(1 - \sqrt{1 - \dfrac{2mRn}{fy}})$

$\rho = \dfrac{1}{20.59}(1 - \sqrt{1 - \dfrac{2(20.6)(1.088)}{420}}) = .00266$

A req = ρ× b × d = .00266×1800×305=1462.29mm²….. control

$A_{s\ min} = 0.0018 \times 1800 \times 400 = 1296\ mm^2$

So , Use 8Φ 16 with As = 1608.5 mm²> As req = 1462.29mm²…for each direction x and y.