Chapter Four
Structural Analysis and Design

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4 -1 Introduction:-

Concrete is the only major building material that can be delivered to the job site in a plastic state. This unique quality makes concrete desirable as a building material because it can be molded to virtually any form or shape.

Concrete used in most construction work is reinforced with steel. When concrete structure members must resist extreme tensile stresses, steel supplies the necessary strength. Steel is embedded in the concrete in the form of a mesh, or roughened or twisted bars. A bond forms between the steel and the concrete, and stresses can be transferred between both components.

In this project, all of design calculation for all structural members would be made upon the structural system which was chosen in the previous chapter.

So, in this project, there are Three types of slabs: one way ribbed slab, two way ribbed slab, and two way flat slab. They would be analyzed and designed by using finite element method of design, with aid of a computer program called "ATIR- Soft ware " to find the internal forces, deflections and moments for ribbed slabs and by using the previous program and "STAAD PRO 2006", Etabs, and Safe programs to find the internal forces, deflections and moments for One way solid slab, and then handle calculation would be made to find the required steel for all members.

The design strength provided by a member, it is connections to other members, and its cross – sections in terms of flexure, and load, shear, and torsion is taken as the nominal strength calculated in accordance with the requirements and assumptions of ACI- code.

NOTE:

*B300…. $fc' = 30\,N/mm^2\,(MPa)$ For circular section

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

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

4 -2 Factored Loads :-

$q_u = 1.2DL + 1.6L$ *ACI – 318 - 02 (9.2.1)*

4 -3 Slabs Thickness calculation:-

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

For rib (R1), as shown in fig.

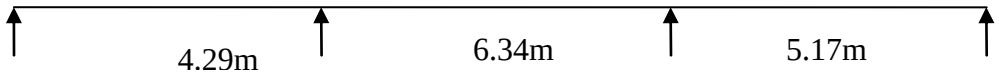


Fig.(4-1): Spans Length of Rib (R1).

The spans are one end continuous ribbed slab

➔ from *ACI-318-02 table (9.5a)*

Min h ≥ :

$\frac{L}{18.5} = \frac{3.64}{18.5} = 0.2\ m$ =20cm

$\frac{L}{21} = \frac{5.54}{21} = 0.26\ m$ =26cm

$\frac{L}{18.5} = \frac{4.52}{18.5} = 0.24\ m$ =24cm

For Rib1 ,will use thickness of slab **32cm**

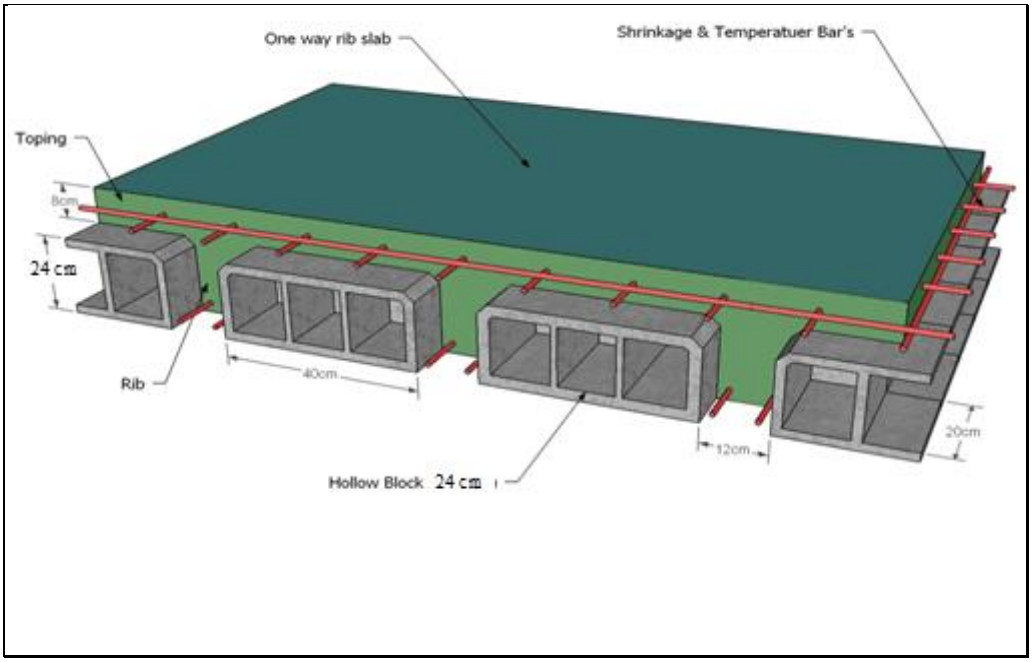


Fig. (4-2) One way ribbed slab [Rib 1]

4 -4 Load Calculation:-

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

- 4.1.1 **Calculation of Dead load :-**

Table (4-1) calculation of the total load for (R1)

Material	Unit weight (KN/m^3)	Thickness (cm)
Tile	**23**	**3**
Mortar	**22**	**2**
Sand	**16**	**7**
Topping slab	**24.5**	**8**
Hollow block	**9**	**24**
Rib	**24.5**	**24**
Plastering	**22**	**2**

Tile: 23*.03*0.52*1=0.36 KN/m
Mortar: 22*0.02*0.52*1=0.23 KN/m
Sand: 16*0.07*0.52*1=0.58 KN/m
Topping: 24.5*.08*0.52*1=1.02 KN/m
Block: 9*0.24*0.4*1=0.86 KN/m
Rib: 24.5*0.12*0.24* 1=0.71 KN/m
Plastering:22*0.02*0.52*1=0.23 KN/m
Partitions:1*0.52*1=0.52 KN/m
➔ **Total dead load = 4.51 KN/m/rib**

❖ **4.4.2 Calculation of Live load:-**

From Jordanian live loads table live load for Hotels is **5 KN/m²**
➔ **Total live load** = 5*0.52 = **2.6 KN/m/rib**

4 -5 Design of Topping:-

❖ **4.4.1Calculation of Dead load**

Tile: 0.03*23*1=0.69 KN/m
Mortar: 0.02*22*1=0.44 KN/m
Sand: 0.07*16*1=1.12 KN/m
Topping: 0.08*24.5*1=1.96 KN/m
Partitions:1*1*1=1.0 KN/m

D.L $_{total}$ = 5.21 KN/m

❖ **4.4.2 Calculation of live load**

L.L $_{total}$ = 5KN/m

➔ Wu = 1.2D.L + 1.6L.L = 1.2*5.21 + 1.6*5 = 14.25K/m

Check ΦMn > Mu

$$Mu = \frac{w_u * l^2}{12} = \frac{14.25 * 0.4^2}{12} = 0.190\,kN.m$$

ΦMn = Φ*fr * s

$$s = \frac{bh^2}{6}$$

$$fr = 0.42 \times \sqrt{fc'}(MPa) \qquad ACI\text{-}318\text{-}02 \quad (22\text{-}5.1)$$

$$Mn = 0.42\sqrt{fc'} * \frac{bh^2}{6}$$

$$= 0.42\sqrt{24} * \frac{1*0.08^2}{6} * 10^3 = 2.194\,kN.m$$

$$\phi * Mn = 0.55 * 2.194 = 1.207\,kN.m.$$

$$\phi * Mn = 1.207 > Mu = 0.190\,KN.m.$$

❖ **4.4.3 Shrinkage and temperature reinforcement must be provided.**

For the shrinkage and temperature reinforcement:

$\rho = 0.0018$ *ACI-318-02 (7.12.2)*

$$As = \rho * b * h = 0.0018 * 1000 * 80 = 144 mm^2 / 1m.$$

A_s (ϕ 8) =50mm^2

So number of bars =144/50 = 2.88 ,so use 3 bars.

Check spacing: 1/(number of bars) =1/2.88 =0.35 >3*h

0.35>3*.08=0.24

Then use Φ 8 @ 20cm for practical purposes in both directions.

4 -6 Design of Rib (1):-

❖ **Materials :-**

Concrete B300 , Fc' = 0.8*30 = 24 N/mm^2 =24Mpa

Reinforcement Steel , fy = 420 N/mm^2 =420 Mpa

❖ **Design constant :-**

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

b_E =Ln/4= 4.29 / 4 =1.07 m = 107 cm

b_E = bw + 16 tf = 12 + 16 (8) = 140 cm

b_E = c/c spacing between beams = 52 cm

Control ……. 52cm

*Requirements For Slab Floor According to *ACI- (318-02)* .

bw ≥ 10cm……ACI(8.11.2)

Select bw=12cm

h ≤ 3.5*bw …….. ACI(8.11.2)

Select h=32cm<3.5*12=42cm

tf ≥ Ln/12≥50mm ……ACI(8.11.6)

Select tf=8cm

❖ **System :-**

One -way ribbed slab :-

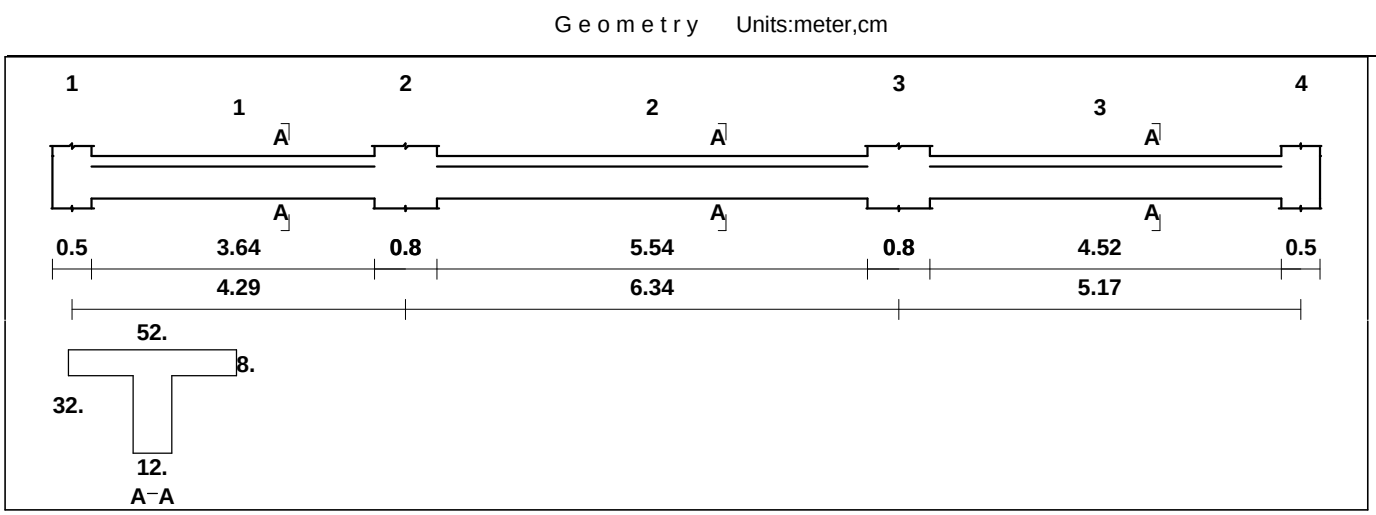
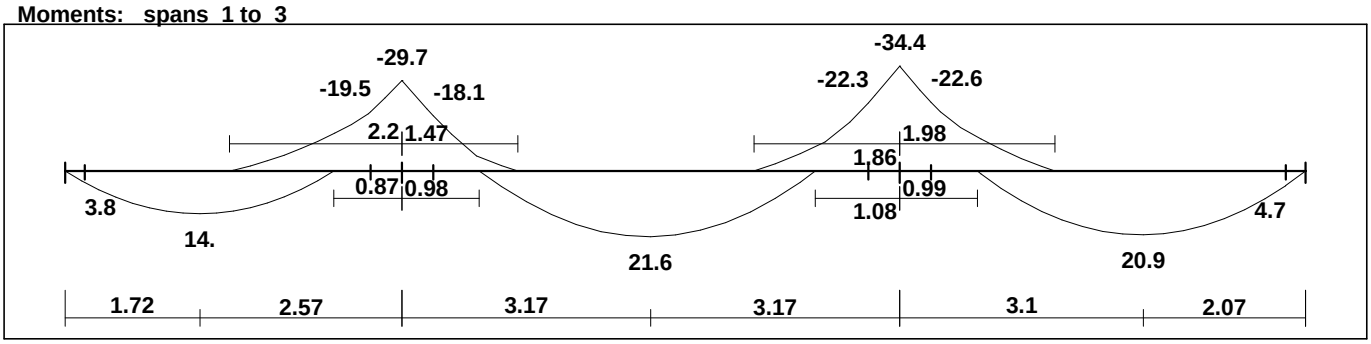


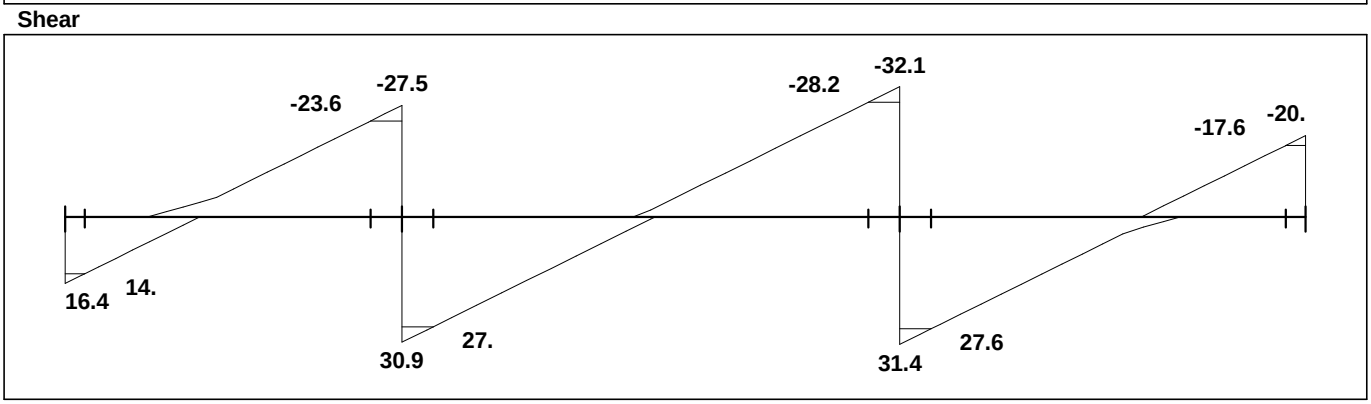
Fig.(4.3): Spans Length of Rib (R1).

❖ **Loading :-**

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

D.L $_{total}$ = 4.51 KN/m/rib **L.L $_{total}$ = 2.6 KN/m/rib**





Flexural Design : -

❖ **Design for positive Moment for Rib (R1):-**

Use Mu max. Positive for span ➔ **Mu= 21.6 KN.m** .
Determine whether the rib will act as **rectangular** or **T-section**:
For a = t_f = 8cm
d = h – cover–dia. of stirrups – db/2 = 320 – 20–8 – 12/2 = 286 mm.
Φ.Mnf = 0.9*0.85 f_c * t_f * b_E *(d- t_f/2)
=0.9* 0.85 (24) (0.08) (0.52)(0.286-0.08/2)*10^3
➔ **Φ.Mnf = 187.9KN.m**

ΦMnf =187.9 KN.m > Mu =21.1 kN.m
➔.'. Rectanglular section

Design as a rectangular with b_E = 52cm
As = ρ.bE .d

$$m = \frac{fy}{0.85 * fc'} = \frac{420}{0.85*24} = 20.58$$

$$Kn = \frac{Mu/\phi}{b*d^2} = \frac{21.6*10^{-3}/0.9}{0.52*(0.286)^2} = 0.564$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2mKn}{fy}}\right) = \frac{1}{20.58}\left(1-\sqrt{1-\frac{2*20.58*0.564}{420}}\right) = 0.00136$$

A_S = ρ.bE .d = (0.00136)* (520)* (286) = 202.3 mm^2 .
Then use 2Φ 12 ,As=226 mm^2

***Check Minimum Reinforcement A_S min ……...*(ACI- 318 - 10.5.1)*

$$A_{s\,min} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) = \frac{\sqrt{24}}{4(420)}(120)(286) = 100mm^2$$

$$A_{s\ min} = \frac{1.4}{(fy)}(bw)(d) = \frac{1.4}{420}(120)(286) = 114.4mm^2 \quad \text{(control)}$$

For 2Φ12 ,As=226 mm^2>114.4 , OK

*****Check for Tension steel yielding:-**

Tension = compression

As * fy = 0.85 * f_c * b * a

$2*113*420 = 0.85*24*520*a$

$a = 8.9mm$

$c = \frac{a}{\beta_1} = \frac{8.9}{0.85} = 10.5mm$

β=0.85….. $f_c < 28MPa$ ……. ACI (10.2.7.3)

$\varepsilon_s = (d-c)/c \times 0.003 = \frac{286-10.5}{10.5} \times 0.003 = 0.079$

$\varepsilon_s = 0.079 > 0.005$

❖ **Design for Negative Moment for Rib (R1):**

use Mu =22.6 KN.m

Design as a rectangular with b = 12 cm

$m = \frac{fy}{0.85 * fc'} = \frac{420}{0.85*24} = 20.58$

Kn = $\frac{Mu/\phi}{b*d^2} = \frac{14.0*10^{-3}/0.9}{0.52*(0.286)^2} = 0.59$

ρ = $\frac{1}{m}(1 - \sqrt{1-\frac{2mRn}{fy}})$

ρ = $\frac{1}{20.58}(1 - \sqrt{1-\frac{2(20.58)(0.59}{420}}) = 0.001$

As = 0.001 (520) (286) = 149 mm^2 .

*****Check Minimum Reinforcement $A_{s\ min}$ ……..(ACI- 318 - 10.5.1)**

$A_{s\ min} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) = \frac{\sqrt{24}}{4(420)}(120)(286) = 100mm^2$

$A_{s\ min} = \frac{1.4}{(fy)}(bw)(d) = \frac{1.4}{420}(120)(286) = 114.4mm^2$ (control)

$A_{s\ min}$ = 134 mm² > A_s =114.4 mm²

➔ **$A_{s\ req}$= 149 mm^2**

# of bars = As/ As_{bar} = 149/79 = 1.88 * Note $A_{Φ10}$ = 79 mm²

Select Bottom bars 2 Φ 10 mm. Total As= 158 mm^2 .

*****Check for Tension steel yielding:-**

Tension = compression

As * fy = 0.85 * f_c * b * a

$2*79*420 = 0.85*24*520*a$

$a = 6.25mm$

$c = \frac{a}{\beta_1} = \frac{6.25}{0.85} = 7.35mm$

β=0.85….. ACI (10.2.7.3)

$\varepsilon_s = (d-c)/c \times 0.003 = 7.35 \times 0.003 = 0.11$

$\varepsilon_s = 0.11 > 0.005 \Rightarrow \phi = 0.9$

❖ **Design for Negative Moment for Rib (R1):**

Use Mu max. negative for support ➔ Mu= 22.6 KN.m

Design as a rectangular with b = 12 cm

$m = \frac{fy}{0.85 * fc'} = \frac{420}{0.85*24} = 20.58$

$Kn = \frac{Mu/\phi}{b*d^2} = \frac{22.6*10^{-3}/0.9}{0.12*(0.286)^2} = 2.56$

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

$\rho = \frac{1}{20.58}(1 - \sqrt{1 - \frac{2(20.58)(2.56)}{420}}) = 0.0065$

As = 0.0065 (120) (286) = 233.08 mm^2 .

*****Check Minimum Reinforcement A_s min..(ACI- 318 - 10.5.1)**

❖ $A_{s\,min} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) = \frac{\sqrt{24}}{4(420)}(120)(286) = 100mm^2$

$A_{s\,min} = \frac{1.4}{(fy)}(bw)(d) = \frac{1.4}{420}(120)(286) = 114.4mm^2$ (control)

A_s = 233.08 mm² > A_s =114.4 mm²

➔ **$A_{s\,req}$= 233.08 mm^2**

Select Top bars 2 Φ 12 mm. Total As= 226 mm^2 .

*****Check for Tension steel yielding:-**

Tension = compression

As * fy = 0.85 * f_c * b * a

$2*113**420 = 0.85*24*120*a$

$a = 38.8mm$

$c = \frac{a}{\beta_1} = \frac{38.8}{0.85} = 45.65mm$

β=0.85….. ACI (10.2.7.3)

$\varepsilon_s = (d-c)/c \times 0.003 = \frac{286-45.65}{45.65} \times 0.003 = 0.016$ Ok……

$\varepsilon_s = 0.016 > 0.005 \Rightarrow \phi = 0.9$

❖ Design shear for Rib (R1):-

Factored shear forces at d=0.286 m from support

Vu_{max} =28.2KN (From Shear Envelop)

Determine shear strength provided by concrete (ϕ Vc).

Item #2:

ΦVc/2 < V_u <ΦV_c

1.1**Φ** Vc = 1.1***Φ** * $\frac{\sqrt{fc'}}{6}$ bw * d

= 1.1* 0.75 * $\frac{\sqrt{24}}{6}$ 0.12 * 0.286*10^3 = 23.11 kN

ΦVc =23.11 kN < Vu = 27.8 kN

Not item #2

Item #3:

Φ Vc< V_u <Φ V_c +Φ $V_{s\ min}$

$$\phi Vs\ \min \geq \frac{\phi}{16} \times \frac{\sqrt{fc'}}{} * bw * d = \frac{0.75}{16} \times \sqrt{24} * 0.12 * 0.286 = 7.88KN$$

$$\phi Vs\ \min \geq \frac{\phi}{3} \times bw * d = \frac{0.75}{3} \times 0.12 * 0.286 = 8.58KN \rightarrow control$$

$$23.11 < 28.2 < 31.69$$

$$\therefore \text{item\#3} \rightarrow \text{min. shear reinforcement is required}$$

$$\left(\frac{AV}{s}\right)\min \geq \frac{1}{16} \times \frac{\sqrt{fc'}}{Fy} bw = \frac{1}{16} \times \frac{\sqrt{24}}{420} * 0.12 = 8.7 * 10^{-5}$$

$$\left(\frac{AV}{s}\right)\min \geq \frac{1}{3} \times \frac{bw}{Fy} = \frac{1}{16} \times \frac{0.12}{420} = 9.5 * 10^{-5} \rightarrow control$$

$$try \phi 8(2leg)$$

$$\frac{2 * 50 * 10^{-6}}{s} = 9.5 * 10^{-5}$$

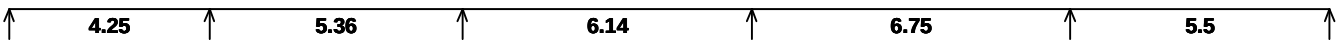
$$s = 1.05m$$

$$but: s \leq \frac{d}{2} \leq 600mm = \frac{0.286}{2} = 0.143m \leq 600$$

$\therefore$ Use Φ8@14cm(2 leg)

4 -7 Design of Beam (B 3):-

- **Material :-**
  - $\Rightarrow$ concrete B300 Fc' = 24 N/mm^2
  - $\Rightarrow$ Reinforcement Steel fy = 420 N/mm^2
- **System : -**



- **Section :-**

  B =80 cm

  h =50 cm

  h=L/18.5 …….,for exterior span ACI-318-02 (9.5.a)

  h= L/21 …….,for interior span ACI-318-02 (9.5.a)

  h= L/18.5 = 5.15/18.5 =0.28 m

  h= L/21 = 6.35/18.5 =0.30 m…… (Control)

  $\Rightarrow$ **Select h = 50cm.**

- **Loading :-**

*****Reaction from rib(R1) ,**

D.L =26.59/0.52 =**51.1 KN/m**

L.L =16.5/0.52 =**31.7 KN/m**

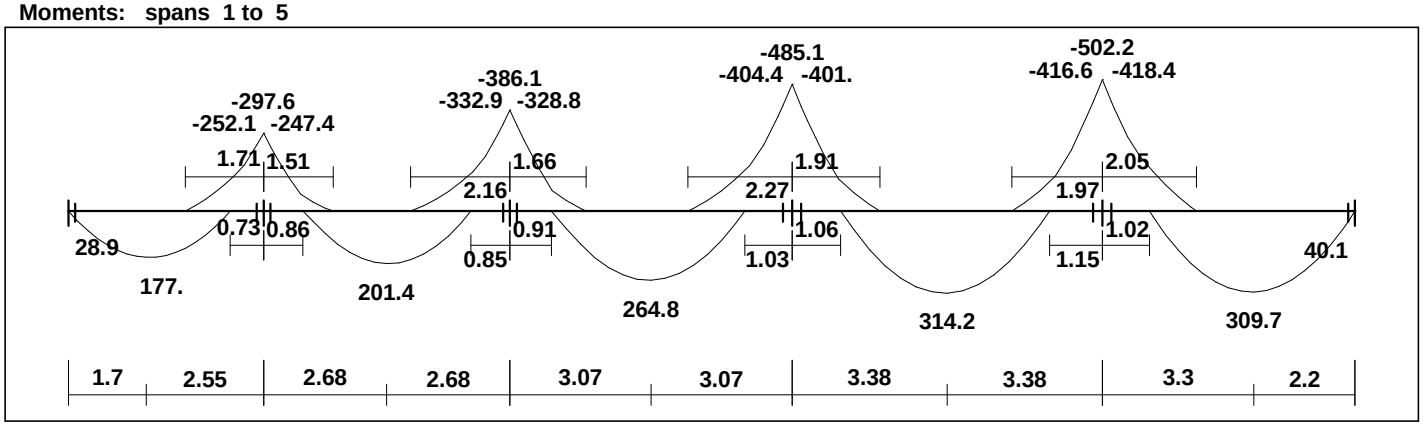
*****Reaction from rib(R2) ,**

D.L =25.12/0.52 =**48.3 KN/m**

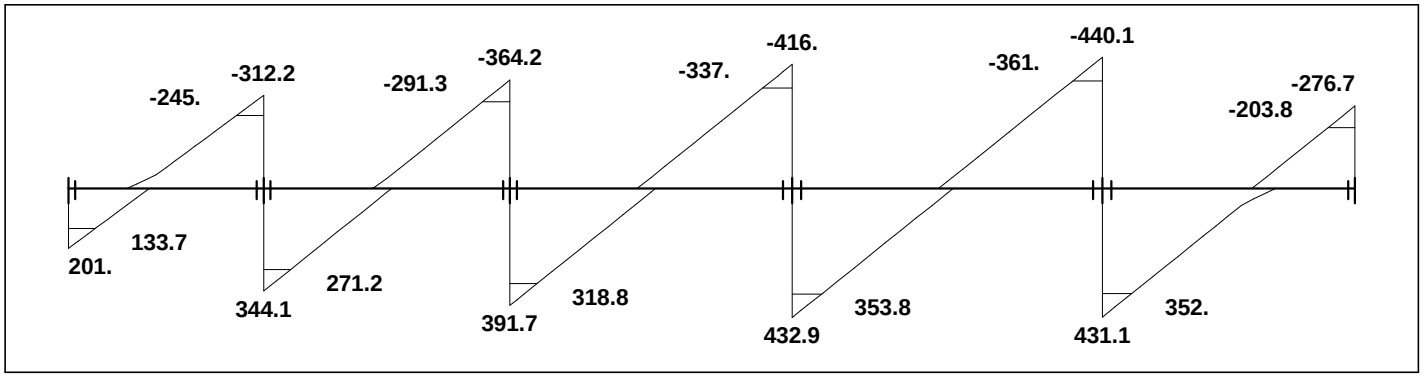
L.L =14.48/0.52 =**27.8 KN/m**

Using "Atir" software for the following values of moment and shear:-

Moments: spans 1 to 5



Shear



R1

Factored

DeadR	7.96	31.91	35.28	10.35
LiveR	8.42	26.4	28.19	9.66
Max R	16.38	58.31	63.47	20.02
Min R	5.67	40.8	47.67	8.65
Service				
DeadR	6.64	26.59	29.4	8.63
LiveR	5.26	16.5	17.62	6.04
Max R	11.9	43.09	47.02	14.67
Min R	5.2	32.15	37.14	7.56

R2

Factored			
DeadR	9.13	30.14	-11.94
LiveR	7.03	23.17	-10.7
Max R	16.16	53.3	-10.42
Min R	9.12	31.79	-22.63
Service			
DeadR	7.61	25.12	-9.95
LiveR	4.39	14.48	-6.69
Max R	12.	39.59	-9.
Min R	7.6	26.15	-16.63

Design of beam :

❖ **Design of Negative Moment for Beam:**

Mu = 418.4 KN .m …… from Envelope shear diagram

$b = 80cm.....h = 50cm$

$d = 500 - 40 - 10 - 10 = 440mm$

$$C_{max} = \frac{3}{7} d$$

$$C_{max} = \frac{3}{7} * 440$$

$$c = 188.6mm$$

$$a = \beta_1 \times c$$

$$a_{max} = 0.85 \times 188.6 = 160.3mm$$

$\Phi = 0.65 + \frac{250}{3} * (0.004 - 0.002) = 0.82$

Mnc = 0.85 f_c * b * a_{max} *(d- a_{max} /2)

= 0.85 (24) (0.8) (0.1603){(0.440-(0.1603/2)}*10^3

= 941.4 KN.m

➔ ΦMnc =0.82*941.4 =772 KN.m > Mu=418.4 KN.m

⇒ The section must be design as Singly Reinforced Concrete section:

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

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

$Kn = \frac{418.4 * 10^{-3} / 0.9}{0.8 * (0.44)^2} = 3.0$ N/mm^2 (Mpa)

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mKn}{fy}}\right)$$

$$\rho = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2(20.6)(3)}{420}}\right) = 0.0078$$

$As_{req} = \rho * b * d = 0.0078 * 800 * 440 = 2746\ mm^2$

""Check Minimum Reinforcement $A_{s\ min}$ ……...(ACI- 318 - 10.5.1)

$$A_{s\,min} = \frac{\sqrt{fc'}}{4(fy)}(b)(d) = \frac{\sqrt{24}}{4(420)}(800)(440) = 1027mm^2$$

$$A_{s\ min} = \frac{1.4}{(fy)}(b)(d) = \frac{1.4}{420}(800)(440) = 1174mm^2 \qquad \text{(control)}$$

$A_s > A_{s\,min}$

$$\#\ \text{of top bars} = \frac{2746}{314} = 8.7$$

$\Rightarrow$ Select 9 Φ20 As=2826 mm^2 .

❖ **Design of Negative Moment for Beam:-**

Mu = 404.4 KN.m …… from Envelope shear diagram

$$C_{max} = \frac{3}{7}d$$

$$C_{max} = \frac{3}{7} * 440$$

$$c = 188.6mm$$

$$a = \beta_1 \times c$$

$$a_{max} = 0.85 \times 188.6 = 160.3mm$$

$\Phi = 0.65 + \frac{250}{3} * (0.004 - 0.002) = 0.82$

$Mnc = 0.85\ f_c * b * a_{max} * (d - a_{max}/2)$

$= 0.85\ (24)\ (0.8)\ (0.1603)\{(0.440 - (0.1603/2)\} * 10^3$

$= 941.4$ KN.m

➔ ΦMnc =0.82*941.4 =772 KN.m > Mu=404.4 KN.m

$\Rightarrow$ The section must be design as Singly Reinforced Concrete section:

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

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

$$Kn = \frac{404.4*10^{-3}/0.9}{0.8*(0.44)^2} = 2.9 \text{ N/mm}^2 \text{ (Mpa)}$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mKn}{fy}}\right)$$

$$\rho = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2(20.6)(2.9)}{420}}\right) = 0.0075$$

$As_{req} = \rho * b * d = 0.0075 * 800 * 440 = 2640 mm^2$

""Check Minimum Reinforcement A_s min ……..(ACI- 318 - 10.5.1)

$$A_{s\,min} = \frac{\sqrt{fc'}}{4(fy)}(b)(d) = \frac{\sqrt{24}}{4(420)}(800)(440) = 1027 mm^2$$

$$A_{s\ min} = \frac{1.4}{(fy)}(b)(d) = \frac{1.4}{420}(800)(440) = 1174 mm^2 \quad \text{(control)}$$

$A_s > A_{s\,min}$

$$\#\text{ of top bars} = \frac{2640}{314} = 8.4$$

$\Rightarrow$ Select 9 Φ20 As=2826 mm^2 .

❖ **Design of Negative Moment for Beam :-**

Mu = 332.9 KN.m …… from Envelope shear diagram

$$C_{max} = \frac{3}{7}d$$

$$C_{max} = \frac{3}{7} * 440$$

$$c = 188.6mm$$

$$a = \beta_1 \times c$$

$$a_{max} = 0.85 \times 188.6 = 160.3mm$$

$$\Phi = 0.65 + \frac{250}{3} * (0.004 - 0.002) = 0.82$$

$Mnc = 0.85\ f_c * b * a_{max} * (d - a_{max}/2)$

= 0.85 (24) (0.8) (0.1603){(0.440-(0.1603/2)}*10^3

= 941.4 KN.m

➔ ΦMnc =0.82*941.4 =772 KN.m > Mu=332.9 KN.m

⇒ The section must be design as Singly Reinforced Concrete section:

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

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

$$Kn = \frac{332.9*10^{-3}/0.9}{0.8*(0.44)^2} = 2.39\ N/mm^2\ (Mpa)$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mKn}{fy}}\right)$$

$$\rho = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2(20.6)(2.39)}{420}}\right) = 0.0061$$

$As_{req}=\rho * b * d = 0.0061 * 800 * 440 = 2147 mm^2$

""Check Minimum Reinforcement $A_{s\ min}$ ……..(ACI- 318 - 10.5.1)

$$A_{s\,min} = \frac{\sqrt{fc'}}{4(fy)}(b)(d) = \frac{\sqrt{24}}{4(420)}(800)(440) = 1027 mm^2$$

$$A_{s\ min} = \frac{1.4}{(fy)}(b)(d) = \frac{1.4}{420}(800)(440) = 1174 mm^2 \quad \text{(control)}$$

$A_s > A_{s\,min}$

$$\#\ \text{of top bars} = \frac{2147}{314} = 6.8$$

⇒ Select 7 Φ20 As=2198 mm^2 .

❖ **Design of Negative Moment for Beam:-**

Mu = 252.1 KN.m …… from Envelope shear diagram

$$C_{max} = \frac{3}{7} d$$

$$C_{max} = \frac{3}{7} * 440$$

$$c = 188.6mm$$

$$a = \beta_1 \times c$$

$$a_{max} = 0.85 \times 188.6 = 160.3mm$$

$\Phi = 0.65 + \frac{250}{3} * (0.004 - 0.002) = 0.82$

Mnc = 0.85 f_c * b * a_{max} *(d- a_{max} /2)

= 0.85 (24) (0.8) (0.1603){(0.440-(0.1603/2)}*10^3

= 941.4 KN.m

➔ ΦMnc =0.82*941.4 =772 KN.m > Mu=252.1 KN.m

⇒ The section must be design as Singly Reinforced Concrete section:

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

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

$$Kn = \frac{252.1 * 10^{-3} / 0.9}{0.8 * (0.44)^2} = 1.8 N/mm^2 \text{ (Mpa)}$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mKn}{fy}}\right)$$

$$\rho = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2(20.6)(1.8)}{420}}\right) = 0.0045$$

$As_{req} = \rho * b * d = 0.0045 * 800 * 440 = 1584 mm^2$

""**Check Minimum Reinforcement** $A_{S\ min}$ **……..(ACI- 318 - 10.5.1)**

$$A_{s\,min} = \frac{\sqrt{fc'}}{4(fy)}(b)(d) = \frac{\sqrt{24}}{4(420)}(800)(440) = 1027mm^2$$

$$A_{s\ min} = \frac{1.4}{(fy)}(b)(d) = \frac{1.4}{420}(800)(440) = 1174mm^2 \quad \text{(control)}$$

$A_S > A_{S\,min}$

# of top bars $= \frac{1584}{314} = 5.04$

$\Rightarrow$ Select 6 Φ20 As=1884 mm^2 .

❖ **Design of Positive Moment for Beam:-**

Mu = 314.2KN .m …… from Envelope shear diagram

$$C_{max} = \frac{3}{7}d$$

$$C_{max} = \frac{3}{7} * 440$$

$$c = 188.6mm$$

$$a = \beta_1 \times c$$

$$a_{max} = 0.85 \times 188.6 = 160.3mm$$

Φ=0.65+ $\frac{250}{3}$ *(0.004-0.002) = 0.82

Mnc = 0.85 f_c * b * a_{max} *(d- a_{max} /2)

= 0.85 (24) (0.8) (0.1603){(0.440-(0.1603/2)}*10^3

= 941.4 KN.m

➔ ΦMnc =0.82*941.4 =772 KN.m > Mu=314.2 KN.m

$\Rightarrow$ The section must be design as Singly Reinforced Concrete section:

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

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

Kn = $\frac{314.2*10^{-3}/0.9}{0.8*(0.44)^2}$ = 2.25 N/mm^2 (Mpa)

ρ = $\frac{1}{m}(1 - \sqrt{1 - \frac{2mKn}{fy}})$

ρ = $\frac{1}{20.6}(1 - \sqrt{1 - \frac{2(20.6)(2.25)}{420}}) = 0.0057$

A_{Sreq}=ρ * b * d = 0.0057 * 800 * 440 = 2006mm^2

""**Check Minimum Reinforcement A_S min ……..(ACI- 318 - 10.5.1)**

$$A_{s\,min}=\frac{\sqrt{fc'}}{4(fy)}(b)(d) \quad =\frac{\sqrt{24}}{4(420)}(800)(440)=1027mm^2$$

$$A_{s\ min}=\frac{1.4}{(fy)}(b)(d) \quad =\frac{1.4}{420}(800)(440)=1174mm^2 \qquad \text{(control)}$$

$A_s > A_{s\,min}$

$$\text{of bottom bars}=\frac{2006}{314}=6.4$$

$\Rightarrow$ Select 7Φ20 As=2198mm^2 .

$\Rightarrow$ Use 4 Φ12 in top ,for practical purposes.

❖ **Design of Positive Moment for Beam:-**

Mu = 264.8KN .m …… from Envelope shear diagram

$$C_{max}=\frac{3}{7}d$$

$$C_{max}=\frac{3}{7}*440$$

$$c=188.6mm$$

$$a=\beta_1\times c$$

$$a_{max}=0.85\times 188.6=160.3mm$$

Φ=0.65+ $\frac{250}{3}$ *(0.004-0.002) = 0.82

Mnc = 0.85 f_c * b * a_{max} *(d- a_{max} /2)

= 0.85 (24) (0.8) (0.1603){(0.440-(0.1603/2)}*10^3

= 941.4 KN.m

➔ ΦMnc =0.82*941.4 =772 KN.m > Mu=264.8 KN.m

$\Rightarrow$ The section must be design as Singly Reinforced Concrete section:

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

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

$$Kn=\frac{264.8*10^{-3}/0.9}{0.8*(0.44)^2}\ 1.9N/mm^2\ (Mpa)$$

$$\rho=\frac{1}{m}\left(1-\sqrt{1-\frac{2mKn}{fy}}\right)$$

$$\rho = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2(20.6)(1.9)}{420}}\right) = 0.0048$$

$As_{req} = \rho * b * d = 0.0048 * 800 * 440 = 1690 mm^2$

""Check Minimum Reinforcement $A_{s\ min}$ ……...(ACI- 318 - 10.5.1)

$$A_{s\ min} = \frac{\sqrt{fc'}}{4(fy)}(b)(d) = \frac{\sqrt{24}}{4(420)}(800)(440) = 1027 mm^2$$

$$A_{s\ min} = \frac{1.4}{(fy)}(b)(d) = \frac{1.4}{420}(800)(440) = 1174 mm^2 \quad \text{(control)}$$

$A_s > A_{s\ min}$

$$\#\ \text{of bottom bars} = \frac{1690}{314} = 5.4$$

$\Rightarrow$ Select6 Φ20 As=1884 mm^2 .

❖ **Design of Positive Moment for Beam:**

Mu = 201.4 KN .m …… from Envelope shear diagram

$$C_{max} = \frac{3}{7}d$$

$$C_{max} = \frac{3}{7} * 440$$

$$c = 188.6 mm$$

$$a = \beta_1 \times c$$

$$a_{max} = 0.85 \times 188.6 = 160.3 mm$$

$\Phi = 0.65 + \frac{250}{3} * (0.004 - 0.002) = 0.82$

$Mnc = 0.85\ f_c * b * a_{max} * (d - a_{max}/2)$

$= 0.85\ (24)\ (0.8)\ (0.1603)\{(0.440 - (0.1603/2)\} * 10^3$

$= 941.4$ KN.m

➔ $\Phi Mnc = 0.82 * 941.4 = 772$ KN.m > Mu=201.4 KN.m

$\Rightarrow$ The section must be design as Singly Reinforced Concrete section:

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

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

$$Kn = \frac{201.4*10^{-3}/0.9}{0.8*(0.44)^2} = 1.44\ N/mm^2\ (Mpa)$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1-\frac{2mKn}{fy}}\right)$$

$$\rho = \frac{1}{20.6}\left(1 - \sqrt{1-\frac{2(20.6)(1.44)}{420}}\right) = 0.0036$$

$As_{req} = \rho * b * d = 0.0036 * 800 * 440 = 1267 mm^2$

""**Check Minimum Reinforcement** $A_{s\ min}$ **……..(ACI- 318 - 10.5.1)**

$$A_{s\,min} = \frac{\sqrt{fc'}}{4(fy)}(b)(d) = \frac{\sqrt{24}}{4(420)}(800)(440) = 1027 mm^2$$

$$A_{s\ min} = \frac{1.4}{(fy)}(b)(d) = \frac{1.4}{420}(800)(440) = 1174 mm^2 \quad \text{(control)}$$

$A_s > A_{s\,min}$

$$\#\ \text{of bottom bars} = \frac{1267}{314} = 4.03$$

$\Rightarrow$ Select 5 Φ20 As=1570 mm^2 .

❖ **Design of Positive Moment for Beam:-**

Mu = 177 KN .m …… from Envelope shear diagram

$$C_{max} = \frac{3}{7}d$$

$$C_{max} = \frac{3}{7}*440$$

$$c = 188.6mm$$

$$a = \beta_1 \times c$$

$$a_{max} = 0.85 \times 188.6 = 160.3mm$$

$\Phi = 0.65 + \frac{250}{3}*(0.004-0.002) = 0.82$

$Mnc = 0.85\ f_c * b * a_{max} * (d - a_{max}/2)$

$= 0.85\ (24)\ (0.8)\ (0.1603)\{(0.440-(0.1603/2)\}*10^3$

$= 941.4$ KN.m

➔ ΦMnc =0.82*941.4 =772 KN.m > Mu=177 KN.m

⇒ The section must be design as Singly Reinforced Concrete section:

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

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

$Kn = \frac{177.9*10^{-3}/0.9}{0.8*(0.54)^2} = 1.28\ N/mm^2\ (Mpa)$

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

$\rho = \frac{1}{20.6}(1 - \sqrt{1 - \frac{2(20.6)(1.28)}{420}}) = 0.0031$

$As_{req} = \rho * b * d = 0.0031 * 800 * 440 = 1091 mm^2$

""Check Minimum Reinforcement $A_{s\ min}$ ……...(ACI- 318 - 10.5.1)

$A_{s\ min} = \frac{\sqrt{fc'}}{4(fy)}(b)(d) = \frac{\sqrt{24}}{4(420)}(800)(440) = 1027 mm^2$

$A_{s\ min} = \frac{1.4}{(fy)}(b)(d) = \frac{1.4}{420}(800)(440) = 1174 mm^2$ (control)

$A_s > A_{s\ min}$

$\#\ of\ bottom\ bars = \frac{1091}{314} = 3.47$

⇒ Select 5 Φ20 As=1570 mm^2 .

Design for shear:

$\Phi Vc = \frac{1}{6} * \Phi * \sqrt{fc'} * b * d = \frac{1}{6} * 0.75\sqrt{24} * 0.8 * 0.44 * 1000 = 215.6 KN$

Check if the dimensions are big enough:

$= \frac{1}{6} * \Phi * \sqrt{fc'} * b * d + \frac{2}{3}\Phi\sqrt{fc'} * b * d$

$= (\frac{1}{6} * 0.75\sqrt{24} * 0.8 * 0.44 + \frac{2}{3} * 0.75 * \sqrt{24} * 0.8 * 0.44) * 1000$

=1077.8 KN >354.9 ,dimensions are big enough.

For Vu=203.8 KN

Item 1:

$Vu < \frac{\Phi Vc}{2}$

ΦVc/2=215.6/2=107.8 KN , not item 1.

Item 2:

$\frac{\Phi Vc}{2} < Vu < \Phi Vc \rightarrow$Item 2

Minimum shear reinforcement is required.

$$\left(\frac{Av}{s}\right)_{min} \geq \frac{1}{3}\frac{bw}{Fyt} \geq \frac{1}{16}\frac{\sqrt{fc'}}{Fyt} * bw$$

$$\left(\frac{Av}{s}\right)_{min} \geq \frac{1}{3}\frac{0.8}{420} \geq \frac{1}{16}\frac{\sqrt{24}}{420} * 0.8$$

$$\left(\frac{Av}{s}\right)_{min} \geq 6.34 * 10^{-4} \geq 5.8 * 10^{-4}$$

$$\left(\frac{Av}{s}\right)_{min} = 6.34 * 10^{-4}$$

$try 4@10$

$$\frac{4 * 79 * 10^{-6}}{s} = 6.34 * 10^{-4}$$

$$s = 0.50 \rightarrow but \rightarrow s \leq \frac{d}{2} = \frac{440}{2} = 220mm \leq 600mm$$

∴ Use Φ10 ,4 legs @200mm c/c

For Vu=361.0 KN

Item 1:

$Vu < \frac{\Phi Vc}{2}$

ΦVc/2=215.6/2=107.8 KN , not item 1.

Item 2:

$\frac{\Phi Vc}{2} < Vu < \Phi Vc$

Not item 2.

Item 3:

$\Phi Vc < Vu < \Phi Vc + \Phi Vs_{\min}$

$\Phi Vc \geq \frac{\Phi}{16}\sqrt{fc'} * bw * d = \frac{0.75}{16}\sqrt{24} * 0.8 * 0.44 * 1000 = 80.8KN$

$\Phi Vs_{\min} \geq \frac{\Phi}{3} bw * d = \frac{0.75}{3} * 0.8 * 0.44 * 1000 = 88KN(control)$

$\therefore \Phi Vc + \Phi Vs_{\min} = 215.6 + 88 = 303.6KN$

$\therefore$ not item 3

Item 4:

$\Phi Vc + \Phi Vs_{\min} < Vu < \Phi Vc + \frac{\Phi}{3}\sqrt{fc'} * bw * d$

$\Phi Vc + \frac{\Phi}{3}\sqrt{fc'} * bw * d = 215.6 + \frac{0.75}{3}\sqrt{24} * 0.8 * 0.44 * 1000 = 646.7KN$

$303.6 < 361 < 646.7$

$\therefore Item\#4$

$\frac{Av}{s} = \frac{Vs}{Fy * d}$

$Vs = \frac{Vu}{\Phi} - Vc = \frac{361}{0.75} - \frac{215.6}{0.75} = 193.9KN$

$\frac{Av}{s} = \frac{193.9 * 10^{-3}}{420 * 0.44} = 1.05 * 10^{-.3}$

$Try \Phi 10, 4leg$

$\frac{4 * 79 * 10^{-6}}{s} = 1.05 * 10^{-.3} \rightarrow s = 300mm, but$

$s \leq \frac{d}{2} \leq 600$

$s \leq 600 \leq \frac{440}{2} = 220$

$\therefore$ Use Φ10 ,4 legs @200mm c/c

For Vu=337KN

Vu< $\frac{\Phi Vc}{2}$

ΦVc/2=215.6/2=107.8 KN , not item 1.

Item 2:

$\frac{\Phi Vc}{2} < Vu < \Phi Vc$

Not item 2.

Item 3:

$$\Phi Vc < Vu < \Phi Vc + \Phi Vs_{\min}$$

$$\Phi Vc \geq \frac{\Phi}{16}\sqrt{fc'} * bw * d = \frac{0.75}{16}\sqrt{24} * 0.8 * 0.44 * 1000 = 80.8KN$$

$$\Phi Vs_{\min} \geq \frac{\Phi}{3} bw * d = \frac{0.75}{3} * 0.8 * 0.44 * 1000 = 88KN(control)$$

$$\therefore \Phi Vc + \Phi Vs_{\min} = 215.6 + 88 = 303.6KN$$

$\therefore$ not item 3

Item 4:

$$\Phi Vc + \Phi Vs_{\min} < Vu < \Phi Vc + \frac{\Phi}{3}\sqrt{fc'} * bw * d$$

$$\Phi Vc + \frac{\Phi}{3}\sqrt{fc'} * bw * d = 215.6 + \frac{0.75}{3}\sqrt{24} * 0.8 * 0.44 * 1000 = 646.7KN$$

$$303.6 < 337 < 646.7 \rightarrow \therefore Item4$$

$$\frac{Av}{s} = \frac{Vs}{Fy * d}$$

$$Vs = \frac{Vu}{\Phi} - Vc = \frac{337}{0.75} - \frac{215.6}{0.75} = 162KN$$

$$\frac{Av}{s} = \frac{162 * 10^{-3}}{420 * 0.44} = 8.8 * 10^{-.4}$$

$$Try\Phi 10, 4leg$$

$$\frac{4 * 79 * 10^{-6}}{s} = 8.8 * 10^{-.4} \rightarrow s = 359mm, but$$

$$s \leq \frac{d}{2} \leq 600$$

$$s \leq 600 \leq \frac{440}{2} = 220$$

$\therefore$ Use Φ10 ,4 legs @200mm c/c

For Vu=291.3 KN

tem 1:

Vu< $\frac{\Phi Vc}{2}$

ϕVc/2=215.6/2=107.8 KN , not item 1.

Item 2:

$\frac{\Phi Vc}{2} < Vu < \Phi Vc$

Not item 2.

Item 3:

$\Phi Vc < Vu < \Phi Vc + \Phi Vs_{\min}$

$\Phi Vc \geq \frac{\Phi}{16}\sqrt{fc'} * bw * d = \frac{0.75}{16}\sqrt{24} * 0.8 * 0.44 * 1000 = 80.8KN$

$\Phi Vs_{\min} \geq \frac{\Phi}{3} bw * d = \frac{0.75}{3} * 0.8 * 0.44 * 1000 = 88KN(control)$

$\therefore \Phi Vc + \Phi Vs_{\min} = 215.6 + 88 = 303.6KN$

$\therefore Item3$

$215.6 < 293.1 < 303.6$

Minimum shear reinforcement is required.

$\left(\frac{Av}{s}\right)_{\min} \geq \frac{1}{3}\frac{bw}{Fyt} \geq \frac{1}{16}\frac{\sqrt{fc'}}{Fyt} * bw$

$\left(\frac{Av}{s}\right)_{\min} \geq \frac{1}{3} * \frac{0.8}{420} \geq \frac{1}{16}\frac{\sqrt{24}}{420} * 0.8$

$\left(\frac{Av}{s}\right)_{\min} \geq 6.34 * 10^{-4} \geq 5.8 * 10^{-4}$

$\left(\frac{Av}{s}\right)_{\min} = 6.34 * 10^{-4}$

$try 4@10$

$\frac{4 * 79 * 10^{-6}}{s} = 6.34 * 10^{-4}$

$s = 0.50 \rightarrow but \rightarrow s \leq \frac{440}{2} = 220mm \leq 600mm$

∴ Use Φ10 ,4 legs @200mm c/c

For Vu=245 KN

Item 1:

Vu< $\frac{\Phi Vc}{2}$

ΦVc/2=215.6/2=107.8 KN , not item 1.

Item 2:

$\frac{\Phi Vc}{2} < Vu < \Phi Vc$

Not item 2.

Item 3:

$\Phi Vc < Vu < \Phi Vc + \Phi Vs_{\min}$

$\Phi Vc \geq \frac{\Phi}{16}\sqrt{fc'} * bw * d = \frac{0.75}{16}\sqrt{24} * 0.8 * 0.44 * 1000 = 80.8KN$

$\Phi Vs_{\min} \geq \frac{\Phi}{3} bw * d = \frac{0.75}{3} * 0.8 * 0.44 * 1000 = 88KN(control)$

$\therefore \Phi Vc + \Phi Vs_{\min} = 215.6 + 88 = 303.6KN$

$\therefore Item3$

$215.6 < 245 < 303.6$

Minimum shear reinforcement is required.

$\left(\frac{Av}{s}\right)_{\min} \geq \frac{1}{3}\frac{bw}{Fyt} \geq \frac{1}{16}\frac{\sqrt{fc'}}{Fyt} * bw$

$\left(\frac{Av}{s}\right)_{\min} \geq \frac{1}{3} * \frac{0.8}{420} \geq \frac{1}{16}\frac{\sqrt{24}}{420} * 0.8$

$\left(\frac{Av}{s}\right)_{\min} \geq 6.34 * 10^{-4} \geq 5.8 * 10^{-4}$

$\left(\frac{Av}{s}\right)_{\min} = 6.34 * 10^{-4}$

$try 4@10$

$\frac{4 * 79 * 10^{-6}}{s} = 6.34 * 10^{-4}$

$s = 0.50 \rightarrow but \rightarrow s \leq \frac{440}{2} = 220mm \leq 600mm$

∴ Use Φ10 ,4 legs @200mm c/c

4-8 Design of Two Way Ribbed Slab :-

❖ **Material :-**

⇒ concrete B300 Fc' = 24 N/mm^2

⇒ Reinforcement Steel fy = 420 N/mm^2

❖ **System : -**

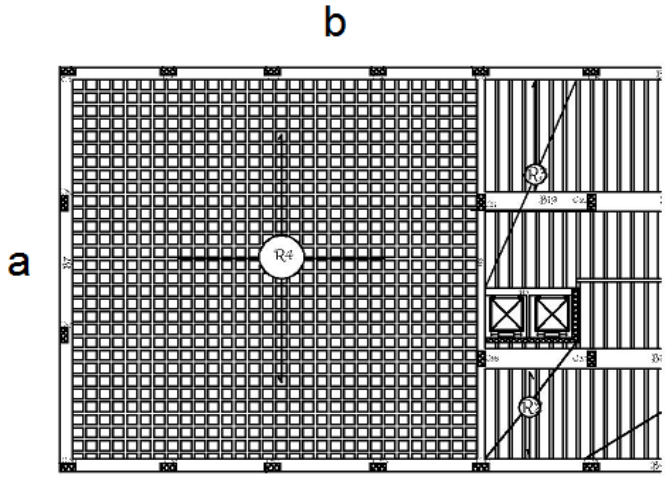


Fig.(4-8-1)

Plan of two way slab.

❖ **Section :-**

a =15.3 m

b=15.5 m

Table 10-6 Minimum Thickness of Slabs without Interior Beams (Table 9.5(c))

Yield strength, f_y MPa	Without drop panels†			With drop panels†		
	Exterior panels		Interior panels	Exterior panels		Interior panels
	Without edge beams	With edge beams††		Without edge beams	With edge beams††	
280	$\frac{\ell_n}{33}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{40}$	$\frac{\ell_n}{40}$
420	$\frac{\ell_n}{30}$	$\frac{\ell_n}{33}$	$\frac{\ell_n}{33}$	$\frac{\ell_n}{33}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{36}$
520	$\frac{\ell_n}{28}$	$\frac{\ell_n}{31}$	$\frac{\ell_n}{31}$	$\frac{\ell_n}{31}$	$\frac{\ell_n}{34}$	$\frac{\ell_n}{34}$

According to table(9.5c):-

h=Ln/33 …….,for exterior span ACI-318-02 (9.5.c)

h= 15.5/33=0.47 m

Select h = 50cm.

bw=50/3.5=15cm

❖ **Loading :-**

Tile: 23*0.03*0.65*0.65=0.29
Mortar: 22*0.02*0.65*0.65=0.19
Sand: 16*0.07*0.65*0.65=0.47
Topping: 25*0.08*0.65*0.65=0.85
Block: 0.2*0.4*0.5*0.42=0.017
Rib: (0.65+0.5)*0.15*0.42*25=1.8
Plastering:22*0.02*0.65*0.65=0.19
Partitions:1*0.65*0.65=0.43
Total Dead Load =4.33 KN
Total Dead Load =4.33/(0.65*0.65)=10 KN/m^2
Live Load =5 KN/m^2

$$qu = 1.2*10 + 1.6*5 = 20 KN/m^2$$

Design of Shear:

$$d = 500 - 20 - 8 - 14 = 458mm$$

$$1.1\phi Vc = \frac{1}{6} * \Phi * \sqrt{fc'} * bw * d * 1.1 = 1.1 * \frac{0.75}{6}\sqrt{24} * 0.15 * 0.458 * 1000 = 42.1K$$

$$W_a = 0.5$$

$$W_b = 0.5$$

$$Vu_a = \frac{0.5}{2} * 15.5 * 15.3 * 20 * \frac{0.65}{15.5} = 49.7KN$$

$$Vu_b = \frac{0.5}{2} * 15.5 * 15.3 * 20 * \frac{0.65}{15.3} = 50.4KN$$

Item 3:

$$\Phi Vc < Vu < \Phi Vc + \Phi Vs_{\min}$$

$$\Phi Vs_{\min} \geq \frac{0.75}{16} * \sqrt{24} * 0.15 * 0.458 * 1000 = 15.7KN$$

$$\Phi Vs_{\min} \geq \frac{0.75}{3} * 0.15 * 0.458 * 1000 = 17.2KN$$

$$42.1 < 50.4 < 42.6 + 17.2$$

$\therefore$ Minimum Shear is Required

$$\left(\frac{Av}{s}\right)_{\min} \geq \frac{\sqrt{fc'} * bw}{16 * Fy} = \frac{\sqrt{24} * 0.15}{16 * 420} = 1.1 * 10^{-4}$$

$$\left(\frac{Av}{s}\right)_{\min} \geq \frac{bw}{3 * Fy} = \frac{0.15}{3 * 420} = 1.2 * 10^{-4} \quad \text{control}$$

$$\frac{2 * 50 * 10^{-6}}{S} = 1.2 * 10^{-4}$$

$$S = 0.83m$$

but

$$s \leq \frac{d}{2} \leq 600$$

$$s \leq 600 \leq \frac{458}{2} = 229$$

$\therefore$ Use Φ8 ,2 legs @200mm c/c

❖ **Design of Moment for Two way Ribbed Slab(Simply Supported):**

$$m = \frac{la}{lb} = \frac{15.3}{15.5} \approx 1$$

$$Ca_{neg} = 0$$

$$Cb_{neg} = 0$$

$$Ca_{D,.L} = 0.036$$

$$Cb_{D,.L} = 0.036$$

$$Ca_{L,.L} = 0.036$$

$$Cb_{L,.L} = 0.036$$

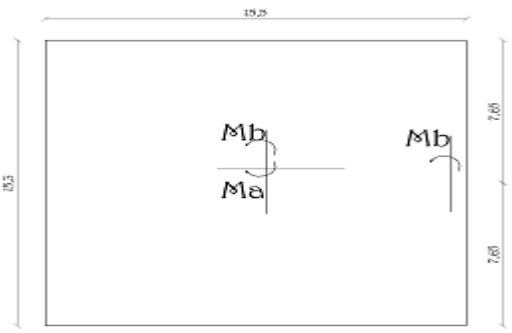


Fig.(4-8-2)

$$Ma_{Pos} = 0.036*12*15.3^2*0.65 + 0.036*8*0.65*15.3^2* = 110KN.m$$
$$Mb_{Pos} = 0.036*12*15.5^2*0.65 + 0.036*8*0.65*15.5^2* = 113KN.m$$

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

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

$$Kn = \frac{113*10^{-3}/0.9}{0.650*0.458^2} = 0.92 \text{ (Mpa)}$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mKn}{fy}}\right)$$

$$\rho = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2(20.6)(0.92)}{420}}\right) = 0.0022$$

$As_{req} = \rho * b * d = 0.0022 * 650 * 458 = 669mm^2$

""Check Minimum Reinforcement A_s min ……..(ACI- 318 - 10.5.1)

$$A_{s\,min} = \frac{\sqrt{fc'}}{4(fy)}(b)(d) = \frac{\sqrt{24}}{4(420)}(150)(458) = 201mm^2$$

$$A_{s\,min} = \frac{1.4}{(fy)}(b)(d) = \frac{1.4}{420}(150)(458) = 229mm^2 \quad \text{(control)}$$

$A_s > A_{s\,min}$

$$\#\text{ of bottom bars} = \frac{669}{314} = 2.2$$

$\Rightarrow$ Select 2 Φ20 $As = 628mm^2$.

*****Check for Tension steel yielding:-**

Tension = compression

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

$2*314*420 = 0.85*24*650*a$

$a = 20mm$

$$c = \frac{a}{\beta_1} = \frac{20}{0.85} = 23.4mm$$

β=0.85….. $f_c < 28$MPa ……. ACI (10.2.7.3)

$$\varepsilon_s = (d-c)/c \times 0.003 = \frac{458 - 23.4}{152.1} \times 0.003 = .056$$

$$\varepsilon_s = 0.056 > 0.005$$

Load on Beam 1:

D.L=7.9*1.2*10=95KN/m
L.L=7.9*1.6*5=63.2KN/m
Self Weight of Beam=0.5*0.5*25=6.25KN/m
Weight of Wall=0.3*25*3.55=26.6KN/m

Load on Beam 9:

D.L=7.9*1.2*10=95KN/m
L.L=7.9*1.6*5=63.2KN/m
Self Weight of Beam=0.5*0.5*25=6.25KN/m
Weight of Wall=0.3*25*3.55=26.6KN/m

4-9 Stair Design:-

4-9-1 Design of Stair #1 :-

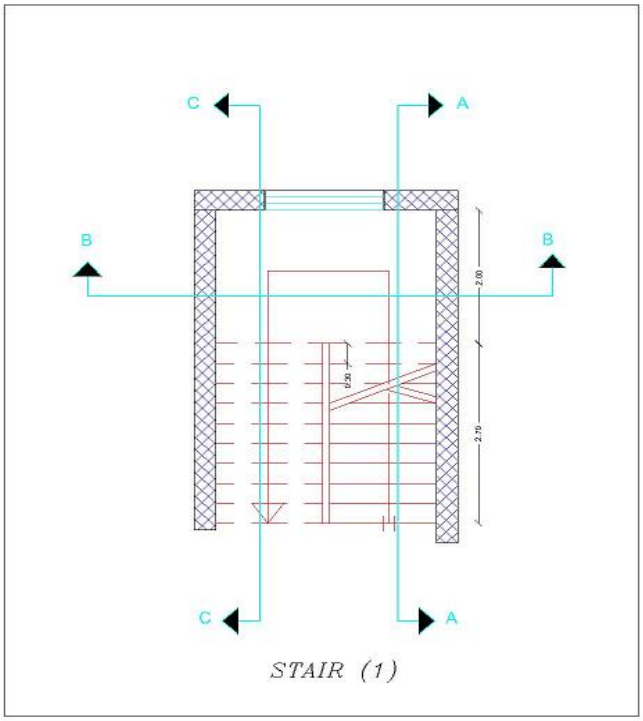


Fig.(4-9-1)

L=2.7+1.0=3.7m

h=3.7/20=0.185m

***Use h= 20 cm

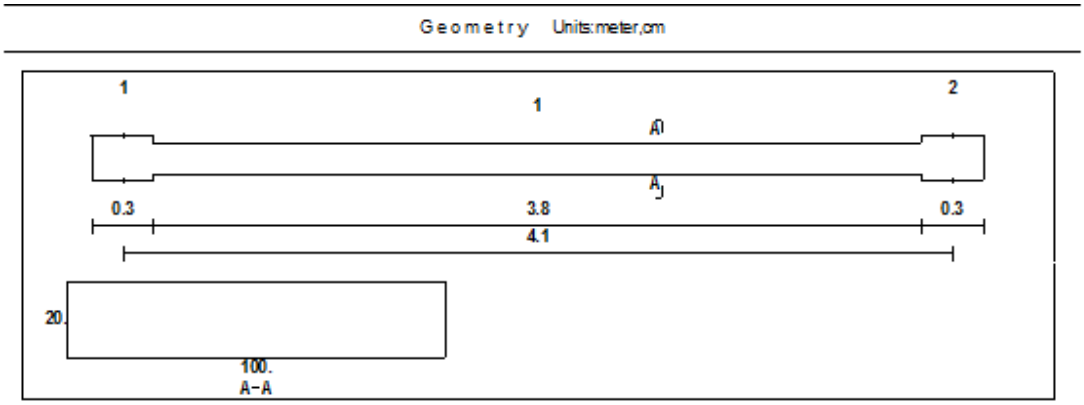
Flight Load calculation:-

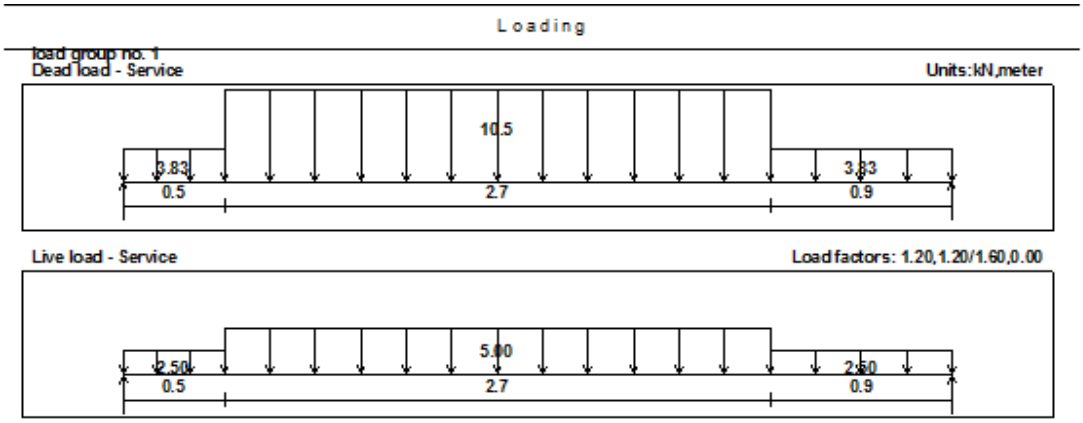
- **Dead Load:-**
  - Tiles = 0.03*(0.33+0.17)*(24/0.30) = 1.2 KN/m^2.
  - Mortar =(0.17+0.3)* 0.02*(22/0.3) = 0.68 KN/ m^2.
  - Stairs = ((0.5*b*h)*25=(0.5*0.3*0.17*25)\0.3= 2.125 KN/ m^2.
  - Slab = 0.20 *25/ Cos 33=6 KN/ m^2.
  - Plaster = (0.02*22)/ (Cos33) 0.25 KN/ m^2.

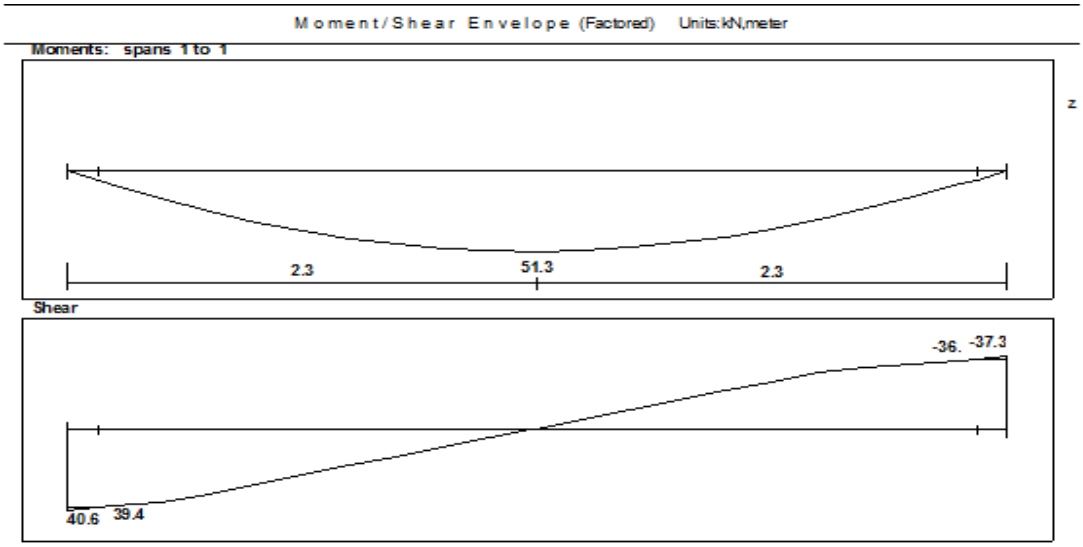
Total dead load = 10.5 KN/m

- **Live load:-**

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







Design of Bending:-

Calculate the magnitude of the maximum moment by using the shear diagram.

d = 200-20 -7 = 173 mm..

Mu = 51.3 kN.m/m

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

$$R_n = \frac{51.3*10^{-3}/0.9}{1*0.173^2} = 1.90$$

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

$$\rho = \frac{1}{m}\left(1 - \sqrt{1-\frac{2mRn}{fy}}\right)$$

$$\rho = \frac{1}{20.6}\left(1 - \sqrt{1-\frac{2(20.6)(1.95)}{420}}\right) = 0.0049$$

$A_{s\ req} = \rho bd = 0.0049*\ 1000*173 = 845\ mm^2/m$

Use $As_{req} = 854\ mm^2$.

Use Φ 14 @ 17.5cm for main reinforcement

$A_{s\ min} = 0.0018*1000*200=360\ mm^2$

Use Φ 10 @ 20cm for transverse reinforcement

4.9.1.6. Design of shear:-

Vu = 36 KN .

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

$$\phi Vc = \frac{0.75*\sqrt{24}*1*0.173}{6} = 106\ KN$$

Vu = 36 KN < Ø.Vc = 106 KN .

No shear Reinforcement is required.

Landing Load calculation:-

- **Dead Load:-**
  - Tiles = 0.03*22 = 0.66 KN/m^2.
  - Mortar = 0.02*22= 0.44 KN/ m^2.
  - Sand = 0.07*16= = 1.12 KN/ m^2.
  - Slab = 0.2*25= 5 KN/ m^2.
  - Plastering = 0.02 *22=0.44 KN/ m^2.

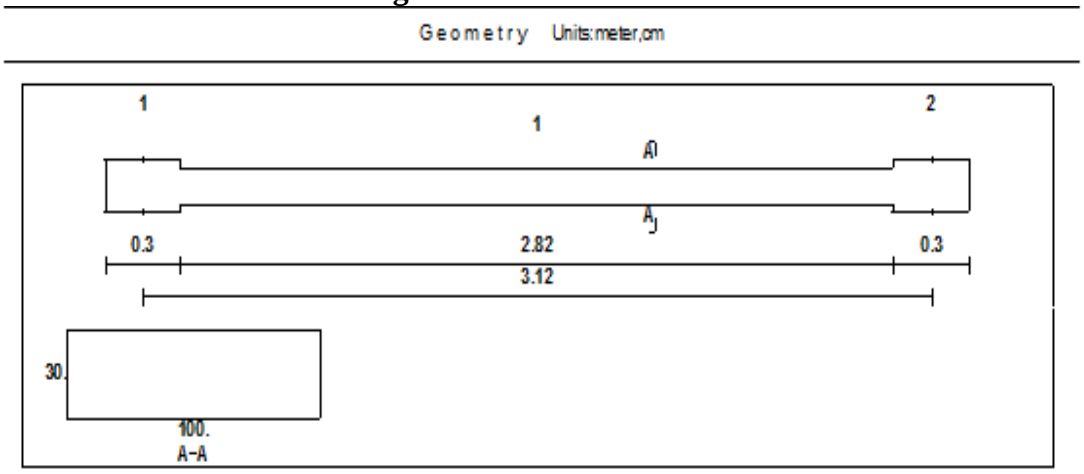
dead load = 7.66 KN/m

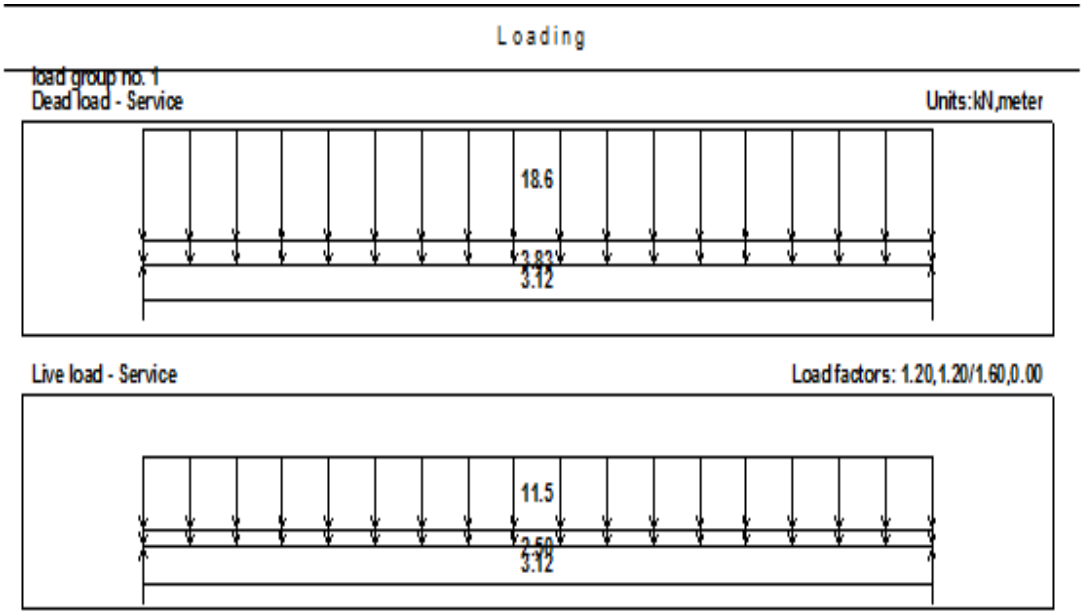
Additional dead load from flight=18.6KN/m

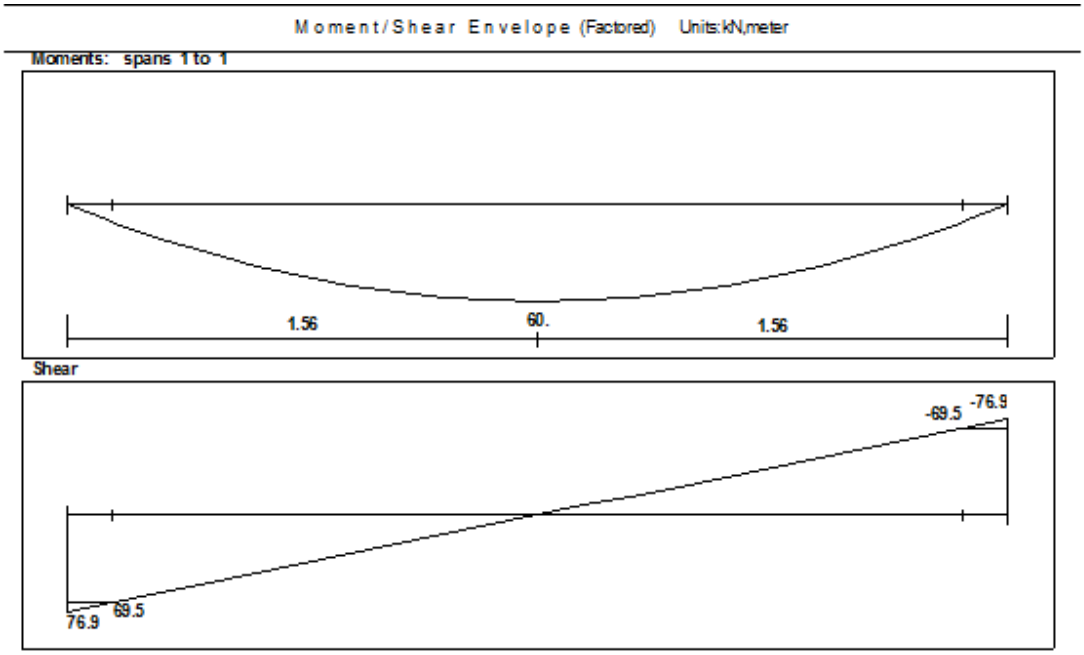
- **Live load:-**

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

Additional live load from flight=11.5KN/m







Design of Bending:-

Calculate the magnitude of the maximum moment by using the shear diagram.

d = 200-20 -7 =170 mm..

Mu =60 kN.m/m

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

$$R_n = \frac{60 * 10^{-3} / 0.9}{1 * 0.17^2} = 2.2$$

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

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mRn}{fy}}\right)$$

$$\rho = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2(20.6)(2.2)}{420}}\right) = 0.0056$$

$A_{s\ req} = \rho bd = 0.0056 * 1000 * 170 = 957 mm^2/m$

Use $As_{req} = 957 mm^2$.

Use Φ 12 @ 10cm for main reinforcement

$A_{s\ min} = 0.0018 * 1000 * 200 = 360\ mm^2$

Use Φ 10 @ 20 cm for transverse

Design of shear:-

Vu = 69.5 KN .

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

$$\phi Vc = \frac{0.75 * \sqrt{24} * 1 * 0.173}{6} = 106\,KN$$

Vu = 69.5 KN < Ø.Vc = 106 KN .

No shear is required.

4-9-2 Design of Stair#2 :-

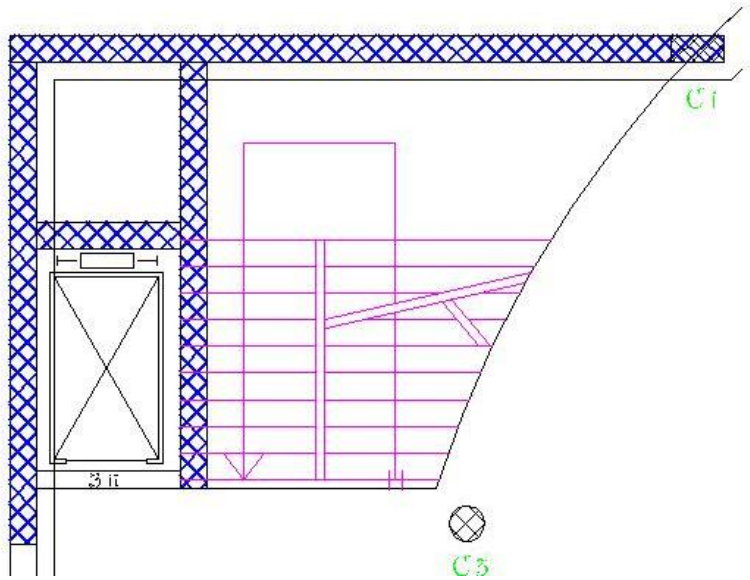


Fig.(4-9-2).
Stair (2)

L=2.7+1.0=3.7m

h=3.7/20=0.185m

***Use h= 20 cm

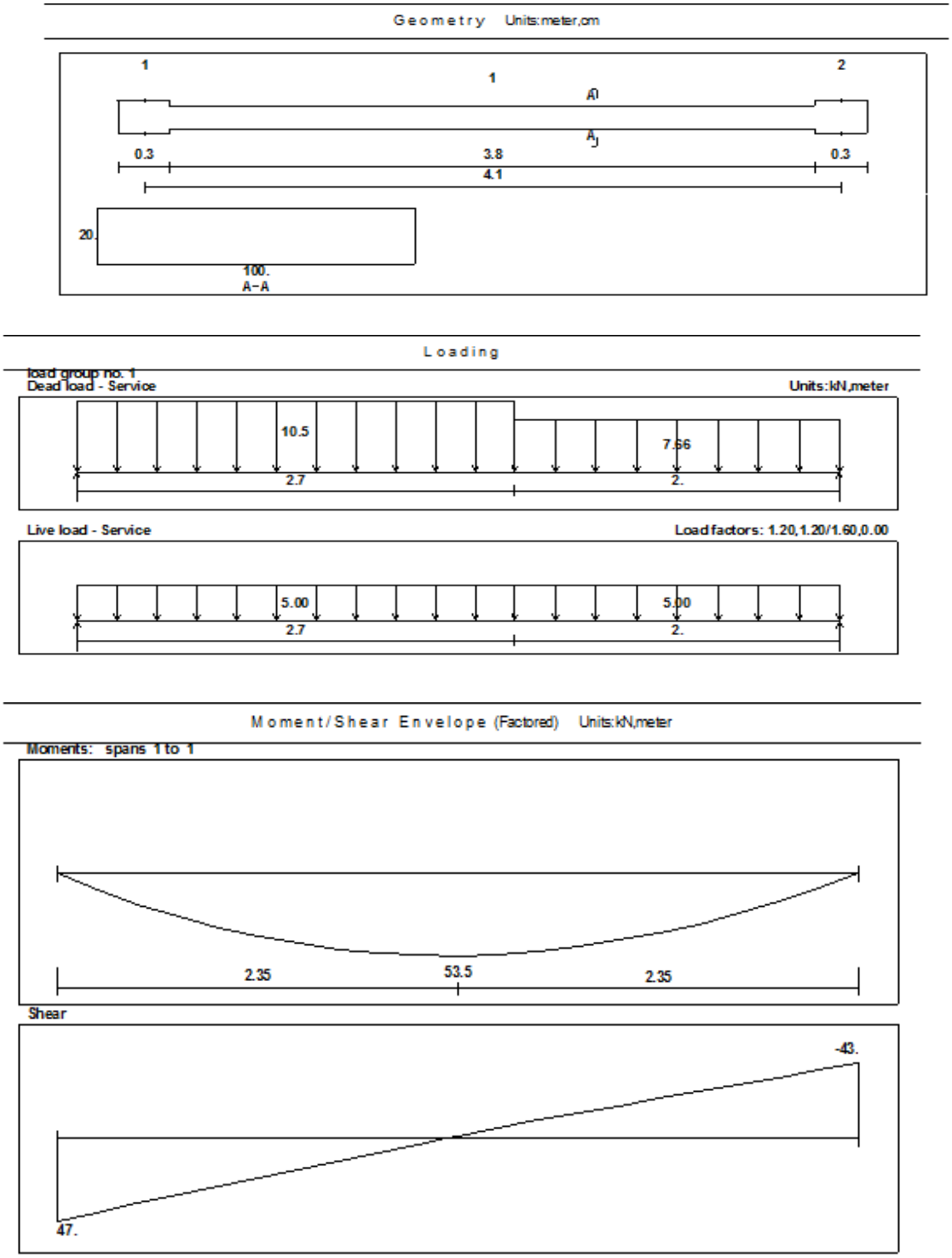
Flight Load calculation:-

- **Dead Load:-**
  - Tiles = 0.03*(0.33+0.17)*(24/0.30) = 1.2 KN/m^2.
  - Mortar =(0.17+0.3)* 0.02*(22/0.3) = 0.68 KN/ m^2.
  - Stairs = ((0.5*b*h)*25=(0.5*0.3*0.17*25)\0.3= 2.125 KN/ m^2.
  - Slab = 0.20 *25/ Cos 33=6 KN/ m^2.
  - Plaster = (0.02*22)/ (Cos 33) = 0.25 KN/ m^2.

Total dead load = 10.5 KN/m

❖ **Live load:-**

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



Design of Bending:-

Calculate the magnitude of the maximum moment by using the shear diagram.

d = 200-20 -7 = 173 mm..

Mu = 53.5 kN.m/m

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

$$R_n = \frac{53.5 * 10^{-3} / 0.9}{1 * 0.173^2} = 2.0$$

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

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mRn}{fy}}\right)$$

$$\rho = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2(20.6)(2)}{420}}\right) = 0.0049$$

$A_{s\ req} = \rho bd = 0.0049 * 1000 * 173 = 845\ mm^2/m$

Use $As_{req} = 854\ mm^2$.

Use Φ 14 @ 17.5cm for main reinforcement

$A_{s\ min} = 0.0018 * 1000 * 200 = 360\ mm^2$

Use Φ 10 @ 20cm for transverse reinforcement

Design of shear:-

Vu = 36 KN .

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

$$\phi Vc = \frac{0.75 * \sqrt{24} * 1 * 0.173}{6} = 106\ KN$$

Vu = 36 KN < Ø.Vc = 106 KN .

No shear Reinforcement is required.

Landing Load calculation:-

- **Dead Load:-**
  - Tiles = 0.03*22 = 0.66 KN/m^2.
  - Mortar = 0.02*22= 0.44 KN/ m^2.
  - Sand = 0.07*16= = 1.12 KN/ m^2.
  - Slab = 0.2*25= 5 KN/ m^2.
  - Plastering = 0.02 *22=0.44 KN/ m^2.

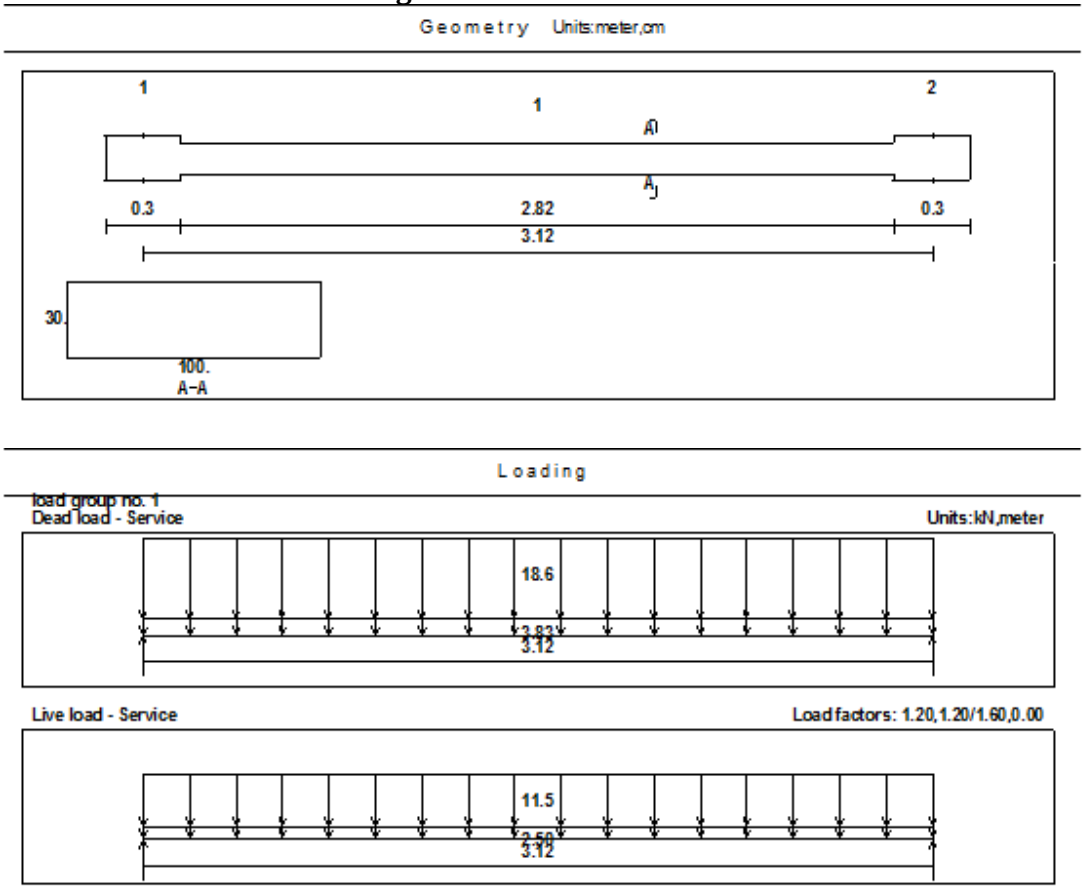
dead load = 7.66 KN/m

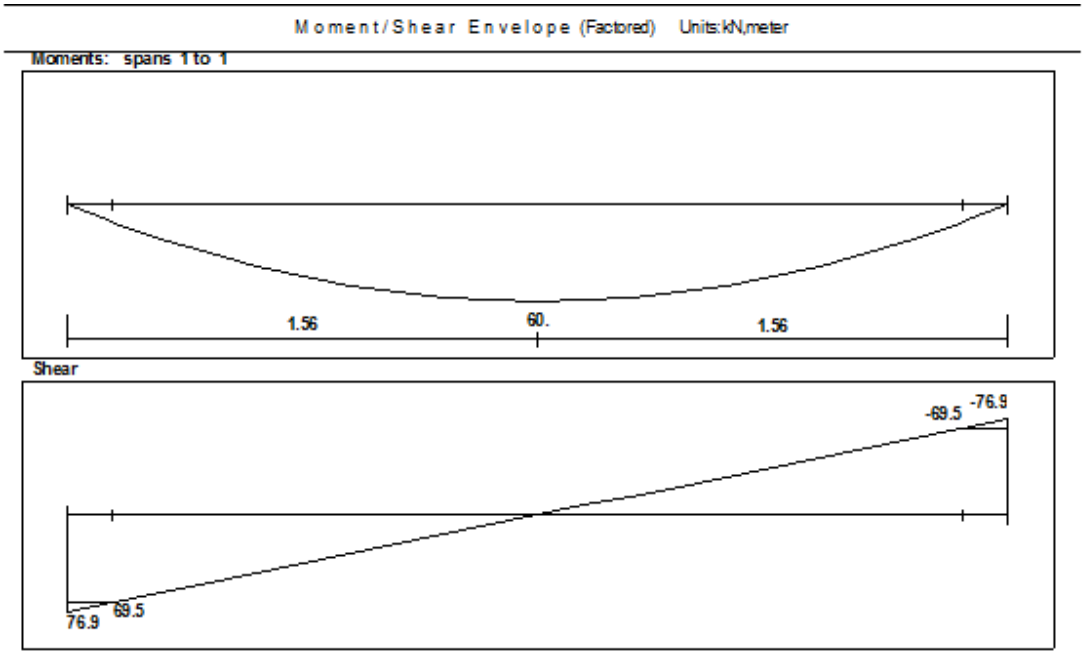
Additional dead load from flight=18.6KN/m

- **Live load:-**

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

Additional live load from flight=11.5KN/m





Design of Bending:-

Calculate the magnitude of the maximum moment by using the shear diagram.

d = 200-20 -7 = 170 mm..

Mu = 60 kN.m/m

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

$$R_n = \frac{60 * 10^{-3} / 0.9}{1 * 0.17^2} = 2.2$$

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

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mRn}{fy}}\right)$$

$$\rho = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2(20.6)(2.2)}{420}}\right) = 0.0056$$

$A_{s\ req} = \rho bd = 0.0056 * 1000 * 170 = 957 mm^2/m$

Use $As_{req} = 957 mm^2$.

Use Φ 12 @ 10cm for main reinforcement

$A_{s\ min} = 0.0018 * 1000 * 200 = 360\ mm^2$

Use Φ 10 @ 20 cm for transverse

Design of shear:-

Vu = 69.5 KN .

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

$$\phi Vc = \frac{0.75 * \sqrt{24} * 1 * 0.173}{6} = 106\,KN$$

Vu = 69.5 KN < Ø.Vc = 106 KN .

No shear Reinforcement is required.

4-9-3 Design of Stair #3 :-

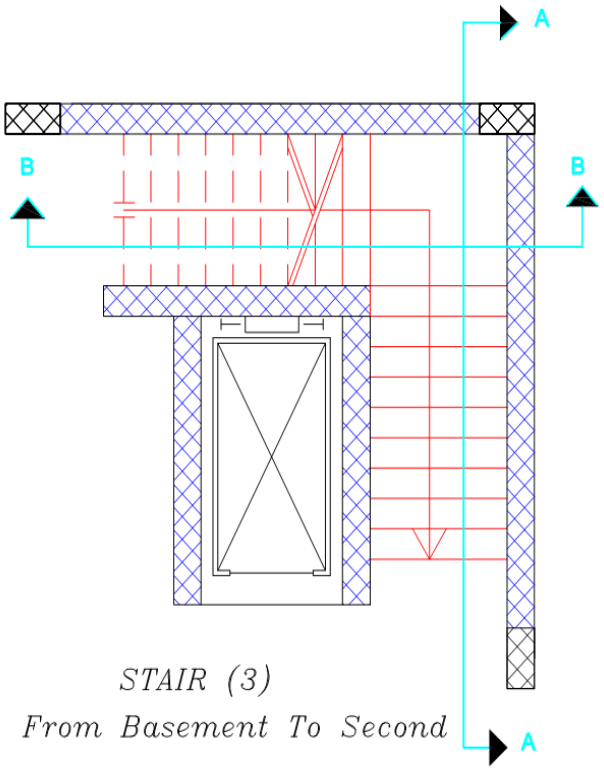


Fig.(4-9-3)

L=2.7+1.0=3.7m

h=3.7/20=0.185m

***Use h= 20 cm

Flight Load calculation:-

- **Dead Load:-**
  - Tiles = 0.03*(0.33+0.17)*(24/0.30) = 1.2 KN/m^2.
  - Mortar =(0.17+0.3)* 0.02*(22/0.3) = 0.68 KN/ m^2.
  - Stairs = ((0.5*b*h)*25=(0.5*0.3*0.17*25)\0.3= 2.125 KN/ m^2.
  - Slab = 0.20 *25/ Cos 33=6 KN/ m^2.
  - Plaster = (0.02*22)/ (Cos 33) = 0.52 KN/ m^2.

Total dead load = 10.5 KN/m

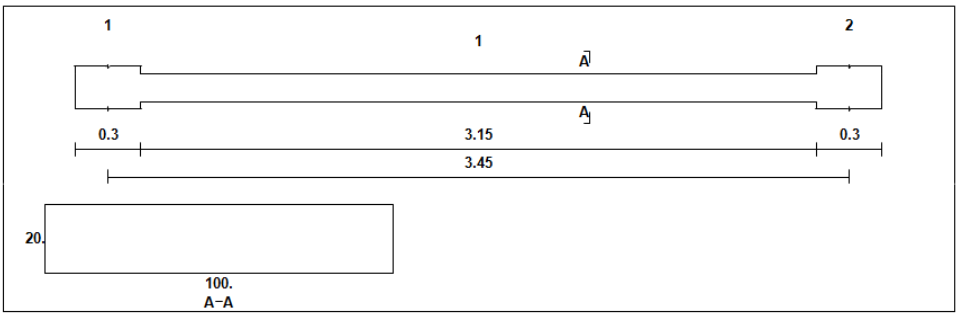
- **Live load:-**

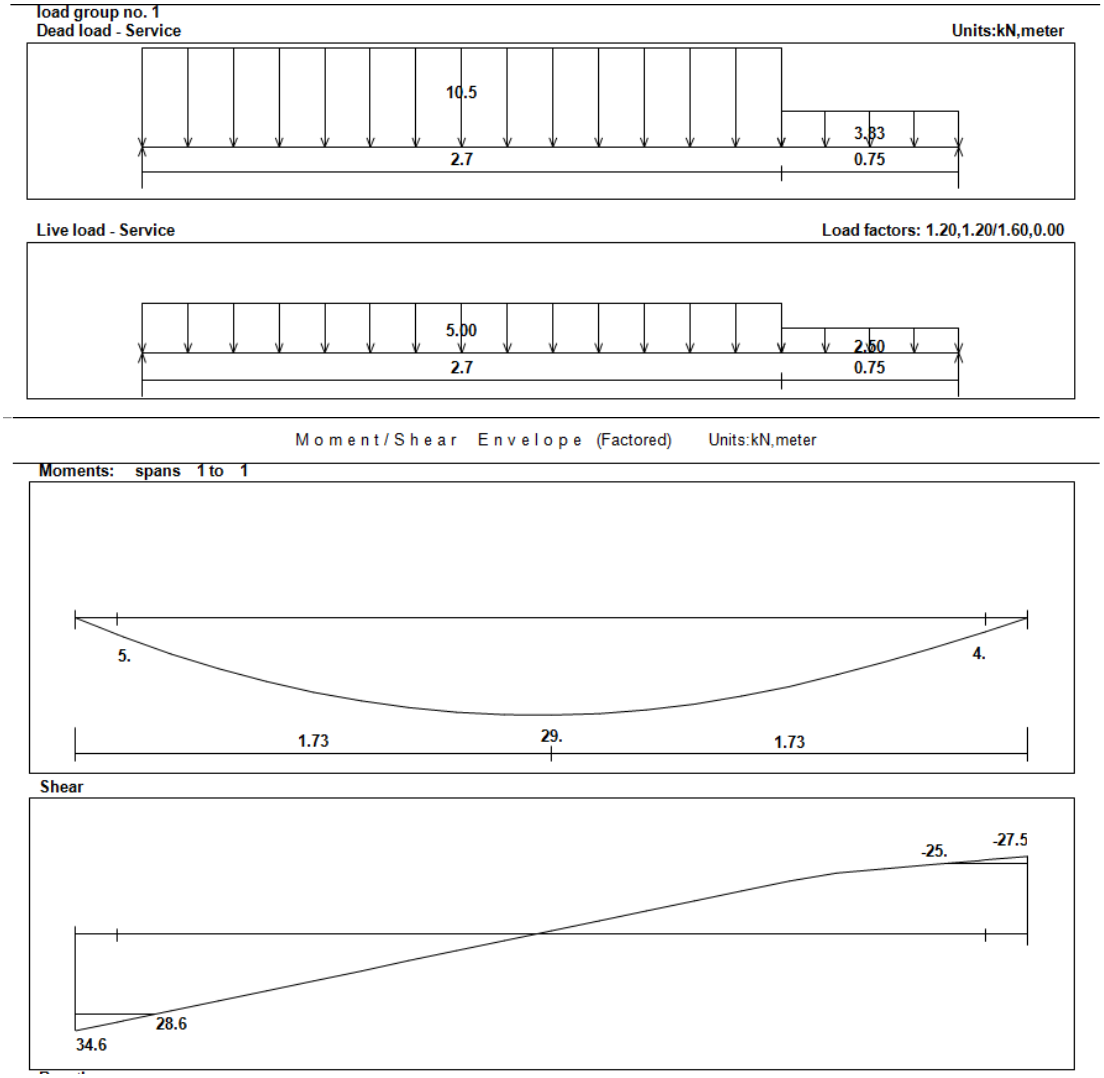
Live load for stairs =5 KN/ m^2.

Landing Dead Load calculation:-

Tiles =0.3*22*1
Mortar =0.02*22*1
Sand =0.07*16
Concrete =0.2*25*1
Plastering =0.02*22*1

=7.66 KN/m





Design of Bending:-

Calculate the magnitude of the maximum moment by using the shear diagram.

d = 200-20 -7 = 173 mm..

Mu = 29 kN.m/m

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

$$R_n = \frac{29*10^{-3}/0.9}{1*0.173^2} = 1.08$$

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

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mRn}{fy}}\right)$$

$$\rho = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2(20.6)(1.08)}{420}}\right) = 0.0026$$

$A_{s\ req} = \rho bd = 0.0026 * 1000 * 173 = 450\ mm^2/m$

Use Φ 10 @ 15cm for main reinforcement

$A_{s\ min} = 0.0018 * 1000 * 200 = 360\ mm^2$

Use Φ 10 @ 20cm for transverse reinforcement

4.9.1.6. Design of shear:-

Vu = 28.6 KN .

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

$$\phi Vc = \frac{0.75 * \sqrt{24} * 1 * 0.173}{6} = 106\ KN$$

Vu = 28.6 KN < Ø.Vc = 106 KN .

No shear Reinforcement is required.

9-10 Design of Column:-

9-10-1 Design of Col #6(In Basement Floor):

Dead load from (*Beam 2*)(basement Slab&3rd Slab)=247.8*2=495.6 KN

Live load from (*Beam 2*) (basement Slab&3rd Slab)=60.9*2=121.8 KN

Dead load from (*Beam 2*) (ground,1st,2nd Slab)=137*3=411 KN

Live load from (*Beam 2*) (ground,1st,2nd Slab)=37*3=111 KN

Self Weight of Columns=25*0.6*0.3*(3.73+3.73+3.46+2.5+2.5)=72 KN

Total Dead Load=495.6+411+72=979 KN

Total Live Load=121.8+111=233 KN

$Pu = 1.2 * 979 + 1.6 * 233 = 1548KN$

$Lu = 3.73$

$*** Assume\ \rho g = 0.015$

$*** Braced column$

$Pn = 0.8Ag(0.85 fc'(1 - \rho g) + \rho g * fy)$

$Pn = \frac{1548}{0.65} = 2382KN$

$2.382 = 0.8Ag(0.85 * 24(1 - 0.015) + 0.015 * 420)$

$Ag = 0.113cm^2$

$Use\ 60 * 30 = 0.18\ m^2$

Check Slenderness Effect:

$$\left(\frac{k.L_u}{r}\right) \le (34 - 12\left(\frac{M_1}{M_2}\right)) \le 40 \ldots\ldots\ldots\ldots ACI.10-12-2$$

L_u : $Actual\ unsupported\ (unbraced)\ length$

K : $effective\ length\ factor$

R : $radius\ of\ gyration = 0.3h = \sqrt{\frac{I}{A}}$

$$\frac{K * Lu}{rx} = \frac{1 * 3.73}{0.3 * 0.3} = 41.4$$

$$\frac{K * Lu}{ry} = \frac{1 * 3.73}{0.3 * 0.6} = 20.7$$

$\Rightarrow$ **Long Column in Y Direction.**

$\Rightarrow$ **Short Column in X Direction.**

*****The Long Column Case Used To Design.**

$$EI = 0.4\frac{E_c I_g}{1+\beta_d} \quad \text{……..…..[}ACI318-2002\ (Eq.\ 10-15)\text{]}$$

$$E_c = 4750\sqrt{fc'} = 4750*\sqrt{24} = 23270Mpa$$

$$\beta_d = \frac{1.2*979}{Pu} = \frac{(1175)}{1548} = 0.76$$

$$I_g = \frac{b*h^3}{12} = \frac{0.3*0.6^3}{12} = 0.0054m^4$$

$$EI = \frac{0.4*23270*10^6*0.0054}{1+0.76} = 28.6MN.m^2$$

$$P_{cr} = \frac{\pi^2 EI}{(KLu)^2} \quad \text{……..…..}ACI318-2002(Eq.\ 10-13)$$

$$P_c = \frac{3.14^2*28.6}{(1.0*3.73)^2} = 20.2MN.$$

$$Cm = 0.6+0.4\left(\frac{M1}{M2}\right) \quad \text{……..…..}ACI318-2002(Eq.10-16)$$

$$Cm = 1 \quad \text{……}According\ to\ ACI318-2002(10.10.6.4)$$

$$\delta_{ns} = \frac{Cm}{1-(Pu/0.75P_{c)}} \geq 1.0 \quad \text{……..…..}ACI318-2002(Eq.\ 10-12)$$

$$\delta_{ns} = \frac{1}{1-(1548/0.75*20.2*10^3)} = 1.1 > 1$$

$$e_{\min} = 15+0.03*h = 15+0.03*600 = 33mm = 0.033m$$

$$e = e_{\min} \times \delta_{ns} = 0.033*1.1 = 0.0363$$

$$\frac{e}{h} = \frac{0.0363}{0.6} = 0.0605$$

$From\ Interaction\ Diagram$

$$\frac{\phi P_n}{A_g} = \frac{1548}{0.6\times 0.3} \times \frac{145}{1000} = 1.25\ \ Ksi$$

$$\rho_g = 0.01$$

$A_s = \rho_g \times A_g = 0.01 \times 600 \times 300 = 1800\ mm^2$`

$Use \phi 25, A_s = 314 mm^2$

$\therefore Use 6\phi 25$

$A_s = 6 * 314 = 1884 mm^2$

9-10-2 Two Way Shear Action(Punching):

Interior Column(In Third Floor):

Design of Column(C 28) :

❖ **Material :-**

⇒ concrete B300 Fc' = 24 N/mm^2

⇒ Reinforcement Steel fy = 420 N/mm^2

Circular Column D=50cm

Slab Thickness =32cm

$d = 320 - 20 - 20 = 280 mm$

$Ln = 2.75$

$*** Dead\ Load = 4.2 KN/m^2$

$*** Live Load = 5 KN/m^2$

$*** \text{Support Reaction} = 372 KN$

$Wu = 1.2 * 4.2 + 1.6 * 5 = 13.04 KN/m^2$

$r = 250 + 140 = 390 mm$

$b_\circ = 2\pi r = 2 * \pi * 0.39 = 2.45 m$

$A = \pi * 0.390^2 = 0.478 m^2$

$Vu = 372 - (0.478 * 13.04) 365$

$$Vc \le \frac{1}{6}(1 + \frac{4}{\beta c})\sqrt{fc'} b_\circ * d = \frac{1}{6}(1+2)\sqrt{fc'} b_\circ * d = 0.5\sqrt{fc'} b_\circ * d$$

$$Vc \le \frac{1}{12}(\frac{\alpha s * d}{b_\circ} + 2)\sqrt{fc'} b_\circ * d = \frac{1}{12}(\frac{40 * 0.28}{2.45} + 2)\sqrt{fc'} b_\circ * d = 0.54\sqrt{fc'} b_\circ * d$$

$$Vc \le \frac{1}{3}\sqrt{fc'} b_\circ * d = 0.333\sqrt{fc'} b_\circ * d \rightarrow control$$

$\phi Vc = 0.75 * 0.333\sqrt{24} * 2.45 * 0.28 * 1000 = 839.3 KN > 365$

4-11Design of Footing:-

4-11-1Design of strip Footing:-

❖ **Materials :-**

Concrete B350 , Fc' = 0.8*35 = 28N/mm^2 =28Mpa
Reinforcement Steel , fy = 420 N/mm^2 =420 Mpa

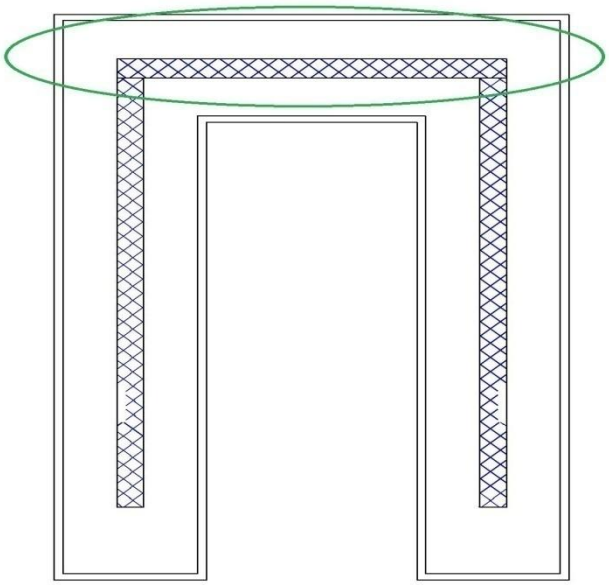


Fig (4-11-1)

Plan of strip footing.

Determination of load:-

From slab and Wall Weight :
Dead:
Weight from "Etabs" =270KN/m
Live:
Weight from "Etabs" =30KN/m
Soil density = 18 KN/m^3.

Allowable soil Pressure = 400 KN/m2.

Assume depth of strip =70cm.

Q $_{allow}$ =400 - (0.7*25+0.15*25) – (0.6*18) -5= 363 KN/m^2

⇒ For one meter strip

$$b = \frac{300}{363} = 0.8\ m$$

b = 1.0 m ,

assume h= 70 cm

d= 700 – 75 – 20 = 605 mm

$$q_{ultimate} = \frac{372}{1.0*1.0} = 372KN/m^2 > 363KN/m^2$$

Check of One Way Shear:-

$$Vu = \left(\frac{1.0-0.3}{2} - 0.605\right) \times 372*1 = 94KN$$

$$\phi Vc = \frac{0.75}{6}\sqrt{28} \times 1 \times 0.605 \times 10^3 = 400KN$$

$$\phi Vc > Vu \quad OK$$

Design of Bending Moment:-

(for 1.0m Strip)

$$Mu = \frac{372 * 0.6^2}{2} = 186\ KN.m$$

$$\text{Kn} = \frac{\phi Mn}{bd^2} = \frac{186 * 10^{-3} / 0.9}{1 * 0.605^2} = 0.6Mpa$$

$$m = \frac{fy}{0.85 * fc'} = \frac{420}{0.85 * 28} = 17.6$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 \times m \times Kn}{f_y}}\right)$$

$$\rho = \frac{1}{17.6}\left(1 - \sqrt{1 - \frac{2 \times 17.6 \times 0.6}{420}}\right) = 0.00145$$

$$As_{\text{Req.}} = \rho * b * d = 0.00145 * 605 * 1000 = 880\ \ mm^2$$

$$As_{\min} = 0.0018 * b * h = 0.0018 * 300 * 1000 = 540mm^2$$

$$As_{\text{Req.}} = 880 > As_{Shrinkage} = 540mm^2$$

Use Φ 12@12.5cm **in both directions**

Check of strain:-

As * fy = 0.85 * f_c * b * a

$$880*420=0.85*28*1000*a$$

$$a=15.5mm$$

$$c=\frac{a}{\beta_1}=\frac{15.5}{0.85}=18.2mm$$

$$\varepsilon_s=\frac{605-18.2}{18.2}\text{X}0.003$$

$$\varepsilon_s=0.096>0.005$$

$$\Rightarrow \text{OK}$$

4-11-2 Design of Isolated footing(F1):-

❖ **Materials :-**

Concrete B350 , Fc' = 0.8*35 = 28N/mm^2 =28Mpa

Reinforcement Steel , fy = 420 N/mm^2 =420 Mpa

Load Calculation(For C5 in ribbed slab region):-

P(Dead)=1669 KN

P(Live)=393 KN

Pu = 2632 KN

Soil weight = 18KN/m^2

Column geometry 60 *30 cm

Allowable soil pressure = 400 KN/m^2

Design of Footing Area:-

$q_{all\ net}$ = 400-(0.6*18)-(0.6*25+0.15*25)-5= 370 KN/m^2

Area (A) = Total load / Allowable Soil Pressure

$$=\frac{1669+393}{370}=5.6m^2$$

Try 2.8 * 2.5 Area = 7.0 m^2

Pu = 1.2*1669+1.6*393=2632 KN

$$q_{ultimate}=\frac{1.2*1669+1.6393}{2.80*2.5}=376KN/m^2>370KN/m^2$$

Thickness Determination:-

Assume h = 60 cm ….. d = 600-75-20 = 505 mm

❖ **Check for One Way Shear Action**

$$Vu = \left(\frac{2.80-.6}{2} - 0.505\right) \times 376 * 2.80 = 624.4KN$$

$$\phi Vc = \frac{0.75}{6}\sqrt{28} \times 2.80 \times 0.505 \times 10^3 = 935.3KN$$

$$\phi Vc > Vu \quad OK$$

❖ **Check for Two Way shear Action (Punching).**

$h = 600mm$

$d = 505mm$

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:

$$\beta_C = \frac{Column\ Length\ (a)}{Column\ Width\ (b)} = \frac{60}{30} = 2$$

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

$b_\circ = (0.505+0.6)*2+(0.505+0.3)*2 = 3.82m$

$\alpha_s = 40$ for interior column

$$\phi.V_C = \phi.\frac{1}{6}\left(1+\frac{2}{\beta_c}\right)\sqrt{f_c'}\, b_o d = \frac{0.75}{6}*\left(1+\frac{2}{2.0}\right)*\sqrt{28}*3.82*0.505*10^3 = 2552kN$$

$$\phi.V_C = \phi.\frac{1}{12}\left(\frac{\alpha_s}{b_o/d}+2\right)\sqrt{f_c'}\, b_o d = \frac{0.75}{12}*\left(\frac{40*0.505}{3.82}+2\right)*\sqrt{28}*3.82*0.555*10^3 = 4712kN$$

$$\phi.V_C = \phi.\frac{1}{3}\sqrt{f_c'}\, b_o d = \frac{0.75}{3}*\sqrt{28}*3.82*0.505*10^3 = 2552\ kN\ control$$

$$Vu = \left[(2.80*2.5)-(0.3+.505)(0.6+0.505)\right]*376 = 2298kN$$

$\phi VC > Vu$ ok

Design for Bending Moment.

***In 2.8 m direction

$$Mu = 376 * \frac{(2.8-.6)}{2} * \frac{(2.8-.6)}{4} * 2.5 = 570 KN.m$$

$$m = \frac{fy}{0.85 * fc'} = \frac{420}{0.85*28} = 17.6$$

$$\text{Kn} = \frac{Mu/\phi}{b*d^2} = \frac{570*10^{-3}/0.9}{2.80*(0.505)^2} = 0.9 \text{ MPa}$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1-\frac{2mRn}{fy}}\right)$$

$$\rho = \frac{1}{17.6}\left(1 - \sqrt{1-\frac{2(17.6)(0.9)}{420}}\right) = 0.00219$$

$$As_{req} = 0.00219\ (2800)\ (505) = 3097\ mm^2$$

$$\rho_{min} = 0.0018$$

$$A_{s\ min} = 0.0018\ (2800)\ (600) = 3024\ mm^2$$

# of bars = As/ As $_{bar}$ = 2595/154 = 21.1

$\Rightarrow$ Note $A_{\Phi 14}$ = 154 mm²

Use 22Φ 14 As provided = 3234mm²

***In 2.5m direction

$$Mu = 376 * \frac{(2.5-.6)}{2} * \frac{(2.5-.6)}{4} * 2.8 = 475 KN.m$$

$$m = \frac{fy}{0.85 * fc'} = \frac{420}{0.85*28} = 17.6$$

$$\text{Kn} = \frac{Mu/\phi}{b*d^2} = \frac{475*10^{-3}/0.9}{2.50*(0.505)^2} = 0.8 \text{ MPa}$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1-\frac{2mRn}{fy}}\right)$$

$$\rho = \frac{1}{17.6}\left(1 - \sqrt{1-\frac{2(17.6)(0.8)}{420}}\right) = 0.00194$$

As_{req} = 0.00203(2500) (505) = 2450 mm^2

ρ_{min} = 0.0018

$A_{s\ min}$ = 0.0018 (2500) (600) = 2700 mm^2

# of bars = As/ As_{bar} = 2700/154 = 18.1

$\Rightarrow$ Note $A_{\Phi 14}$ = 154 mm²

Use 20Φ 14 As provided = 3080mm²

❖ **Check for yielding:-**

Tension = Compression

$As * fy = 0.85 * fc' * b * a$

$3234 * 10^{-6} * 420 = 0.85 * 28 * 2.80 * a$

$a = 20.3mm$

$c = \frac{a}{\beta_1} = \frac{20.3}{0.85} = 24mm$

$\varepsilon_s = \frac{505 - 24}{24} * 0.003 = 0.06$

$\varepsilon_s = 0.06 > 0.005$ .....OK

❖ **Check transfer of load at base of column:**

$\Phi Pn = \Phi(0.85 fc' Ag)$

$\Phi Pn = 0.65(0.85)(28)(0.6 * 0.3) * 10^3 = 2785\,kN >$ Pu =2632 KN.

No dowels are required, but use minimum dowels.

$As_{min} = 0.005 * Ag = 0.005 * 60 * 30 = 9cm^2$

Use the column bars as a dowels

Select 20Φ18

$As_{Provided} = 45cm^2 > As_{Req.} = 9cm^2$

Use 20 Φ 18 dowels As Provided = 45 cm2

❖ **Development Length (L_d):-**

$Ld = \frac{9}{10} \frac{fy}{\gamma\sqrt{fc}} \frac{\omega t * \omega e * \omega s}{\frac{ktr + cb}{db}} * db$

Cb=75+14/2=82mm

Ld=$\frac{9}{10}\frac{420}{\sqrt{28}}\frac{1.0*1.0*0.8}{\frac{0+82}{14}} * 14 = 137\ mm$

Available Ld =(2800-600)/2 – 75 = 1025>137 mm

OK.

Ldb= $\frac{fy}{4\sqrt{fc'}} \times d_b$

Ldb= $\frac{420}{4\sqrt{28}} \times 18 = 1270\ mm$ control

But not less than:

Ldb = 0.04(fy) d_b =0.04(420)*16 =268.8 mm

Available Ld =h-cover-db=60-7.5-2*1.6=74.3 cm

74.3 > 12.7 ……. OK

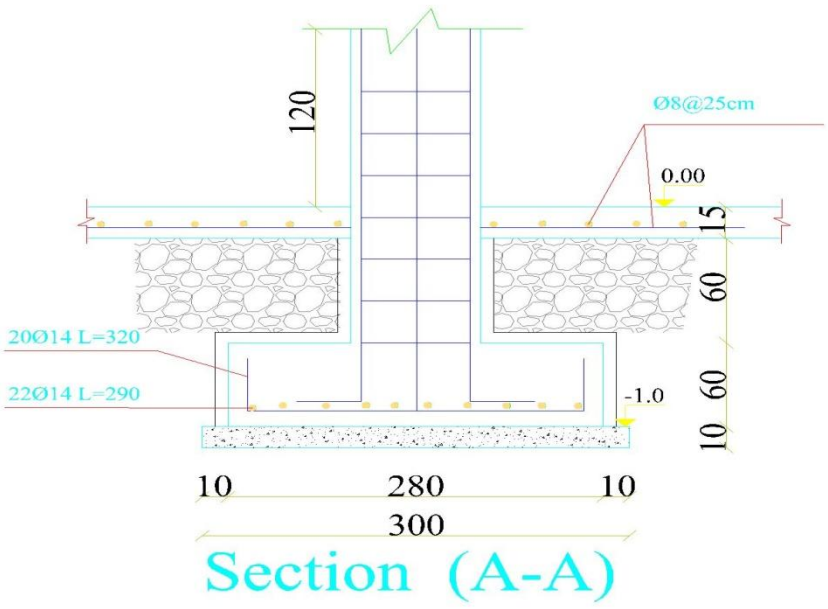


Fig.(4-11-2)

Section (A-A) in Isolated Footing.

4-12Design of Basement Wall:-

- **Position :-**
  Basement Wall (BW) .
- **Material :-**
  Concrete B300 , Fc' = 0.8*30= 24 N/mm^2 =24Mpa
  Reinforcement Steel , fy = 420 N/mm^2 =420 Mpa
- **System :-**

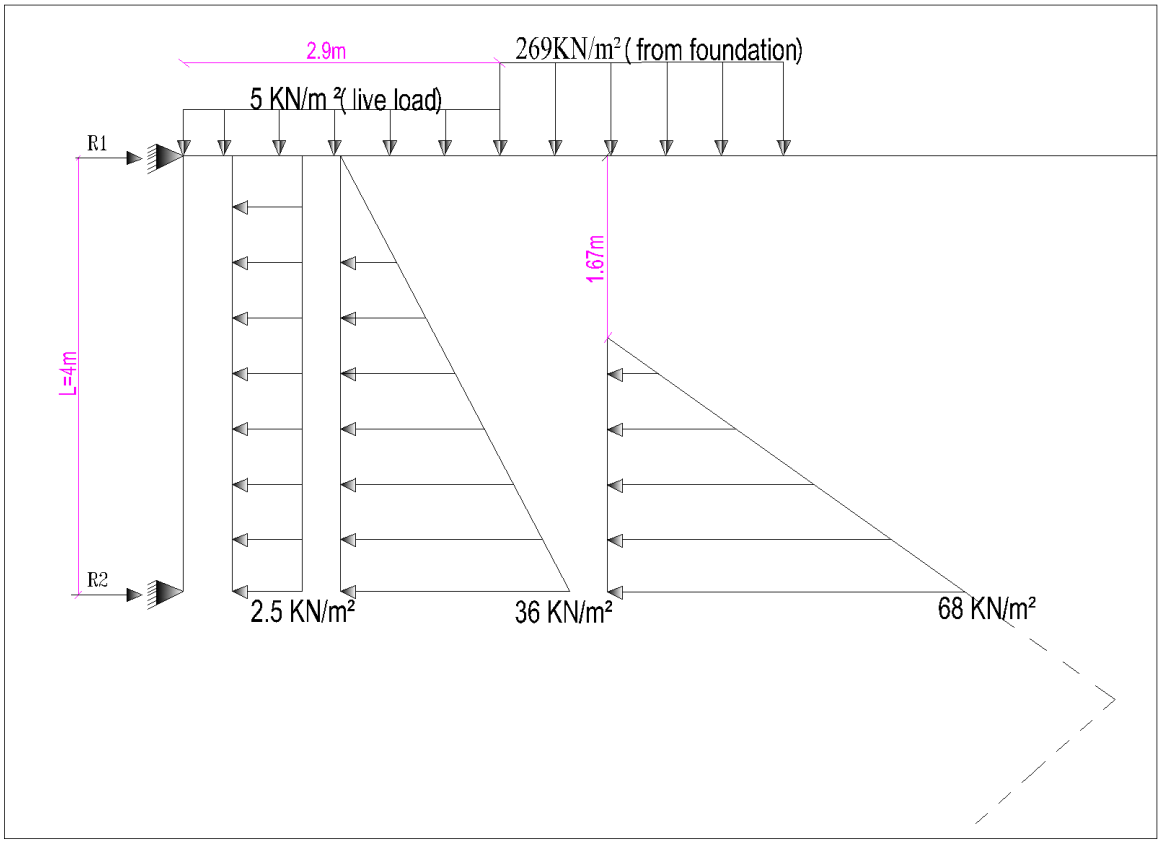


Fig.(4-12-1).

Load on Basement wall

❖ **Loading :-**

Load calculation

$\phi = 30°$

$K_0 = 1 - \sin 30 = 0.5$

$\gamma = 18KN/m^3$

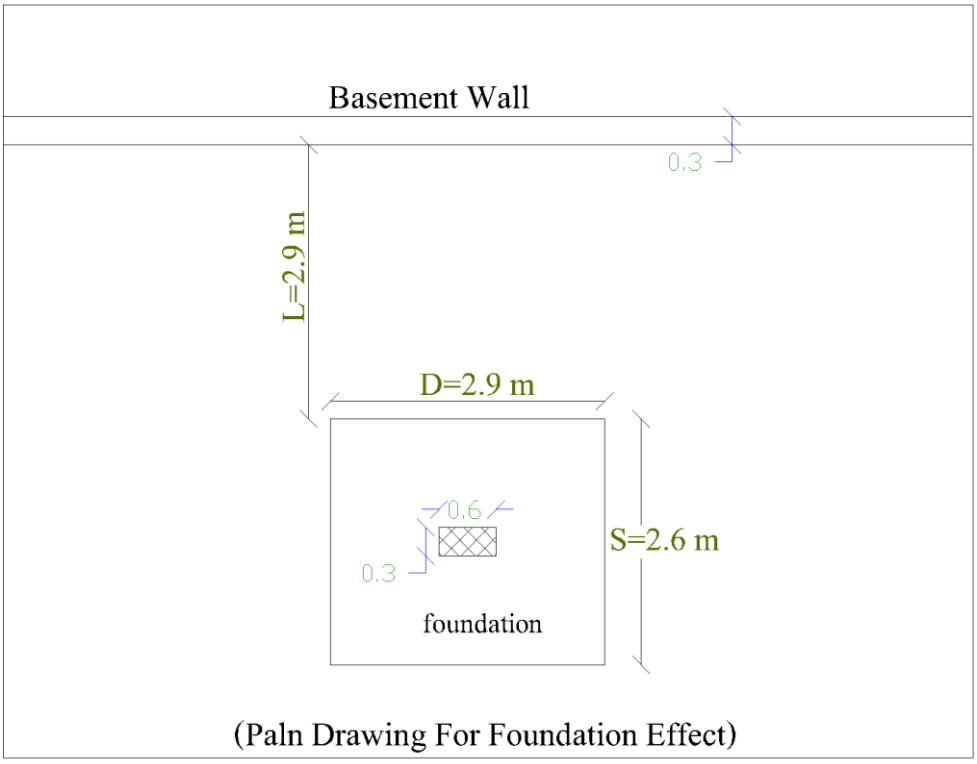
Self weight of earth

$e_o = \gamma h k_o = 18 * 4 * 0.5 = 36KN/m^2$

Load from live load [L.L=5 KN/m^2]

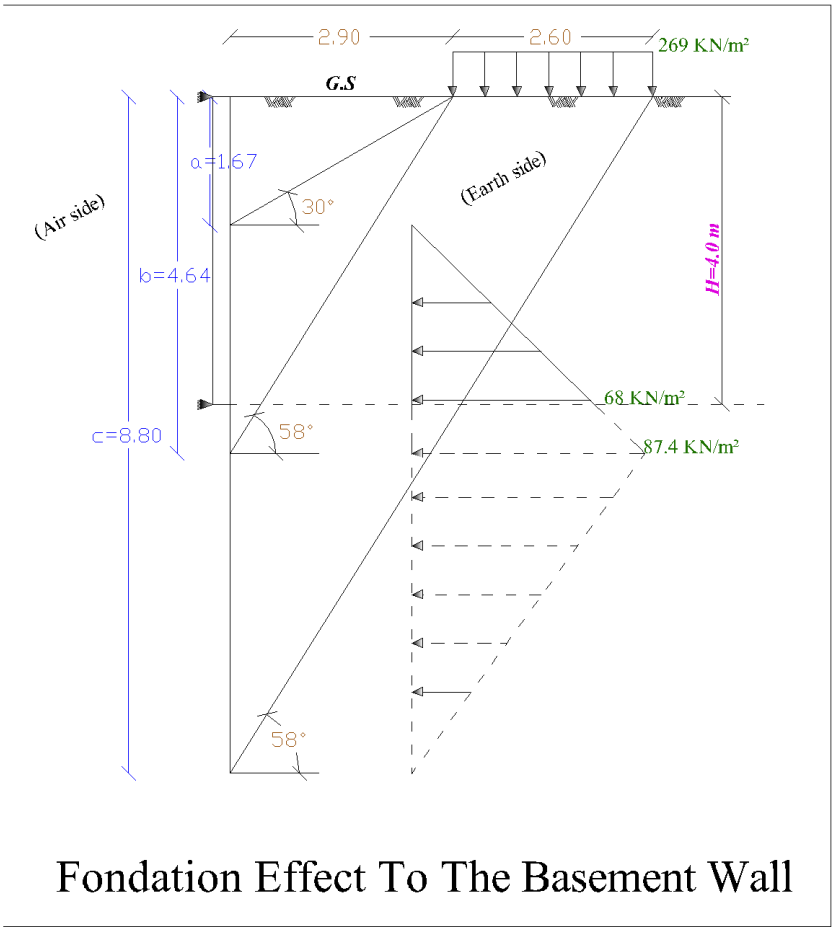
$W = P * K_0 = 5 * 0.5 = 2.5KN/m$

Load from foundation (F= 2028 KN)



(Paln Drawing For Foundation Effect)

Fig. (4-12-2).



Fondation Effect To The Basement Wall

Fig.(4-12-3).

D = 2.9
S = 2.6
L = 2.9

$$F.^{*} = \frac{F}{A} = \frac{2028}{2.6 * 2.9} = 269\ KN/m^2$$

$$\delta_{a.} = \frac{2}{3}\emptyset = \frac{2}{3} * 30 = 20°$$

(wall friction angle)

$\propto = \beta = 0$

$$f. = \frac{\cos(\propto + \emptyset)}{\sin(\emptyset + \delta_a) + \sqrt{\frac{\sin(\emptyset + \delta_a) * \cos(\alpha + \beta) * \cos(\delta_a - \alpha)}{\sin(\emptyset - \beta)}}} = 0.44$$

U_a: ground sliding angle

$\tan U_a. = \frac{\sin(\emptyset) + f * \cos(\emptyset + \delta_a - \propto)}{\cos(\emptyset) + f * \sin(\emptyset + \delta_a - \propto)} = 1.48$

$U_a = 56°$

From atir program:

$V_{u.} = 147.9\ KN$

$M_{u.} = 123.2\ KN.m$

$d. = 300 - 20 - \frac{14}{2} = 273\ mm$

$\emptyset V_{c.} = 0.75 * \frac{1}{6} * \sqrt{fc'} * b * d = 0.75 * \frac{1}{6} * \sqrt{24} * 1 * 0.273 * 1000 = 167.2\ KN$

$\emptyset V_c \geq V_u$ ……OK.

For vertical reinforcement

$$k_n = \frac{M_u/\phi}{b * d^2} = \frac{123.2/0.9}{1 * (0.273)^2} * 10^{-3} = 1.84$$

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

$$\rho. = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2m * k_n}{f_y}}\right) = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2 * 20.59 * 1.84}{420}}\right) = 0.0046$$

$A_S = \rho * b * d = 0.0046 * 273 * 1000 = 1256\ mm^2$ ……..(control)

$A_{S\,min} = 0.0012 * b * h = 0.0012 * 1000 * 300 = 360\ mm^2$

Use Φ 14@10 cm (in vertical direction) in air side

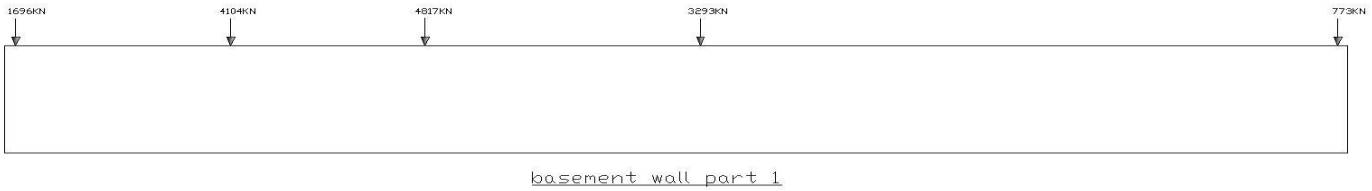
Use Φ 14@20 cm (in vertical direction) in earth side

For horizontal reinforcement

$$A_{S\,min} = 0.002 * b * h = 0.002 * 1000 * 300 = 600\ mm^2$$

Use Φ 10@25 cm (in horizontal direction) in both sides

Design of the basement wall (1) under concrete column :-



We will design the columns in the basement wall by considering new dimensions of the columns (1m * 0.3m)

Design the transport load in tension position: (along o.4h = 0.4*4=1.6 m vertically)…….. according to German Code

$$T = \frac{Pu}{4}\left(1 - \frac{X}{h}\right)$$

$$T_1 = \frac{773}{4}\left(1 - \frac{0.6}{4}\right) = 164.26\ \text{KN}$$

$$\text{As} = \frac{T}{0.9 * fy}$$

$$\text{As} = \frac{164.26}{0.9 * 42} = 4.345\ \ cm^2$$

But we have from the horizontally reinforcement along 1.6 m As=4.74 cm^2
So no need for extra reinforcement.

$$T_2 = \frac{3293}{4}\left(1 - \frac{0.6}{4}\right) = 699.76 \text{ KN}$$

$$\text{As} = \frac{T}{0.9 * fy}$$

$$\text{As} = \frac{699.76}{0.9 * 42} = 18.51 \quad cm^2$$

As from **Φ 10@25 cm = 4.74** cm^2
18.51-4.74=13.77 cm^2
Use 5 Φ 14 in each side with As = **15.4** cm^2 > **13.77** cm^2

And the same calculation to the rest.

$$T_3 = \frac{4187}{4}\left(1 - \frac{0.6}{4}\right) = 1023.6 \text{ KN}$$

$$\text{As} = \frac{T}{0.9 * fy}$$

$$\text{As} = \frac{1023.6}{0.9 * 42} = 27.1 \quad cm^2$$

As from **Φ 10@25 cm = 4.74** cm^2
27.1-4.74=22.36 cm^2
Use 8Φ 14 in each side with As = **24.64** cm^2 > **22.36** cm^2

$$T_4 = \frac{4104}{4}\left(1 - \frac{0.6}{4}\right) = 872.1 \text{ KN}$$

$$\text{As} = \frac{T}{0.9 * fy}$$

$$\text{As} = \frac{872.1}{0.9 * 42} = 23.1 \quad cm^2$$

As from **Φ 10@25 cm = 4.74** cm^2

23.1-4.74=18.36 cm^2

Use 6 Φ 14 in each side with As = **18.48** cm^2 > **18.36** cm^2

$$T_5 = \frac{1696}{4}\left(1 - \frac{0.6}{4}\right) = 360.4 \text{ KN}$$

$$\text{As} = \frac{T}{0.9*fy}$$

$$\text{As} = \frac{360.4}{0.9*42} = 9.5 \quad cm^2$$

As from **Φ 10@25 cm = 4.74** cm^2

9.5-4.74=4.76 cm^2

Use 3 Φ 10 in each side with As = **4.74** cm^2

$$T_6 = \frac{1365}{4}\left(1 - \frac{0.6}{4}\right) = 288.15 \text{ KN}$$

$$\text{As} = \frac{T}{0.9*fy}$$

$$\text{As} = \frac{288.15}{0.9*42} = 7.6 \quad cm^2$$

As from **Φ 10@25 cm = 4.74** cm^2

7.6-4.74=2.86 cm^2

Use 2 Φ 10 in each side with As = **3.16** cm^2 > **2.86** cm^2

$$T_7 = \frac{3122}{4}\left(1 - \frac{0.6}{4}\right) = 663.43 \text{ KN}$$

$$\text{As} = \frac{T}{0.9*fy}$$

$$\text{As} = \frac{663.43}{0.9*42} = 17.55 \quad cm^2$$

As from **Φ 10@25 cm = 4.74** cm^2

17.55-4.74=12.81 cm^2

Use 5 Φ 14 in each side with As = **15.4** cm^2 > **12.81** cm^2

$T_8 = \frac{1775}{4}\left(1 - \frac{0.6}{4}\right) = 377.19$ KN

As = $\frac{T}{0.9*fy}$

As = $\frac{377.19}{0.9*42} = 9.98 \quad cm^2$

As from **Φ 10@25 cm = 4.74** cm^2

9.98-4.74=5.24 cm^2

Use 2 Φ 14 in each side with As = **6.16** cm^2 > **5.24** cm^2

$T_9 = \frac{2372}{4}\left(1 - \frac{0.6}{4}\right) = 504$ KN

As = $\frac{T}{0.9*fy}$

As = $\frac{504}{0.9*42} = 13.3 \quad cm^2$

As from **Φ 10@25 cm = 4.74** cm^2

13.3-4.74=8.56 cm^2

Use 3 Φ 14 in each side with As = **9.24** cm^2 > **8.56** cm^2

$T_{10} = \frac{2142}{4}\left(1 - \frac{0.6}{4}\right) = 455.18$ KN

As = $\frac{T}{0.9*fy}$

As = $\frac{455.18}{0.9*42} = 12 \quad cm^2$

As from **Φ 10@25 cm = 4.74** cm^2

12-4.74=7.26 cm^2

Use 3 Φ 14 in each side with As = **9.24** cm^2 > **7.26** cm^2

$T_{11} = \frac{2147}{4}\left(1 - \frac{0.6}{4}\right) = 456.24$ KN

As = $\frac{T}{0.9*fy}$

As = $\frac{456.24}{0.9*42} = 12.1 \quad cm^2$

As from **Φ 10@25 cm = 4.74** cm^2

12.1-4.74=7.36 cm^2

Use 3 Φ 14 in each side with As = **9.24** cm^2 > **7.36** cm^2

$T_{12} = \frac{2320}{4}\left(1 - \frac{0.6}{4}\right) = 493$ KN

As = $\frac{T}{0.9*fy}$

As = $\frac{493}{0.9*42} = 13 \quad cm^2$

As from **Φ 10@25 cm = 4.74** cm^2

13-4.74=8.26 cm^2

Use 3 Φ 14 in each side with As = **9.24** cm^2 > **8.26** cm^2

$T_{13} = \frac{1368}{4}\left(1 - \frac{0.6}{4}\right) = 290.7$ KN

As = $\frac{T}{0.9*fy}$

As = $\frac{290.7}{0.9*42} = 7.7 \quad cm^2$

As from **Φ 10@25 cm = 4.74** cm^2

7.7-4.74=2.96 cm^2

Use 2 Φ 10 in each side with As = **3.16** cm^2 > **2.96** cm^2

4-13Design of well walls:

Live load=5 KN/m^2

- **Water Load :**

$q_1 = \gamma \times h$

$\gamma_{\text{waterl}} = 10\ Kn/m^3$

$qwater = 10 \times 3.4 = 34\text{KN/m}^2$

- **Soil Load :**

$q_2 = \gamma * h * K_a$

$K_a = 0.5$

$\gamma_{soil} = 18\ Kn/m^3$

$qsoil = 18 * 5 * 0.5 = 45\text{KN/m}^2$

$\Rightarrow$ **Design** :

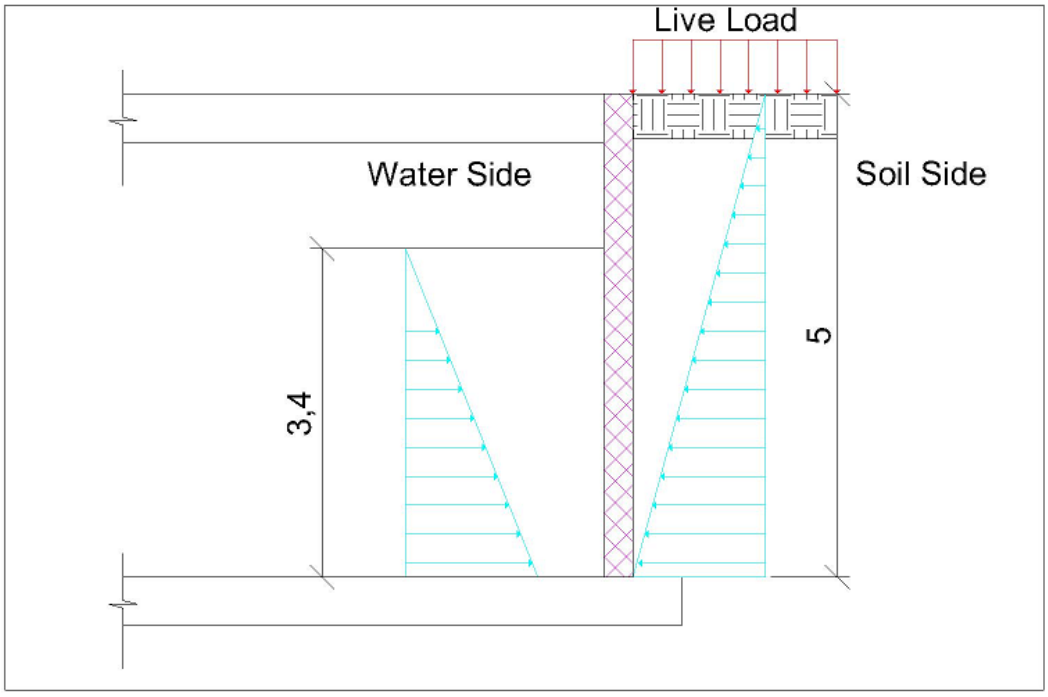
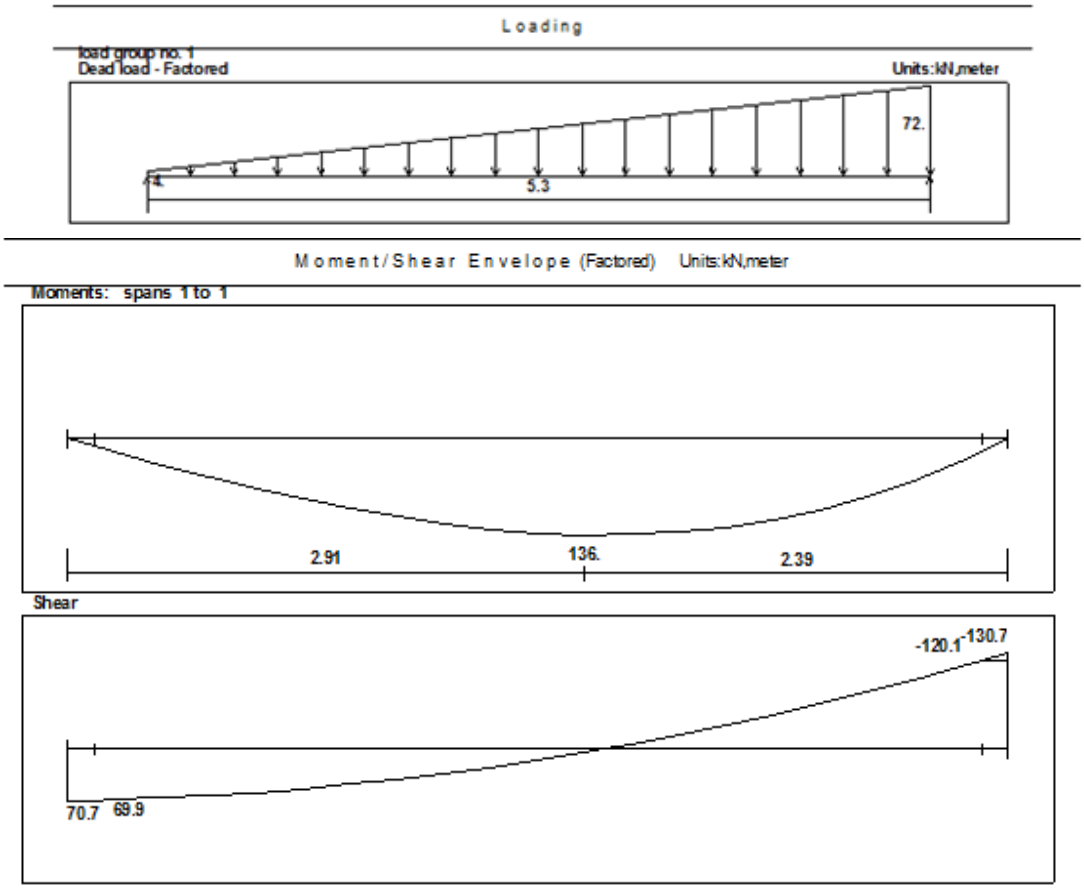


Fig .(4-13-1)
Load of well wall.



m.136 kN =Mu

Vud =120.1 kn

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

$$d = 300 - 20 - 10 = 270mm$$

$$K_n = \frac{Mn}{bd^2} \Rightarrow K_n = \frac{136*10^{-3}/0.9}{1.0*0.270^2} = 2.01$$

Design of the Vertical reinforcement:

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

$$\rho = \frac{1}{20.6}(1-\sqrt{1-\frac{2\times 20.6\times 2.7}{420}}) = 0.0052$$

$$As_{req} = 0.0052\times 1000\times 270 = 1404mm^2$$

$$As_{\min} = 0.0012*1000*300 = 360\,mm^2/m$$

$$As_{\min} = 360mm^2/m < As_{req} = 1634mm^2/m$$

$$\#of\ bar\ in\ on\ meter = \frac{1404}{201} = 7$$

Use Φ 16 @ 14cm

Design of the Horizontal reinforcement:

$$As_{horezantal} = 0.002*1000*300 = 600mm^2/m$$

$$\#of\,bar = \frac{600}{113} = 5.3$$

Use Φ 12@15cm for horizontal reinforcement

: Check for Shear

$$\phi\times Vc \geq Vu$$

$$\phi\times Vc = \frac{0.75}{6}\sqrt{fc'}\times b\times d = \frac{0.75}{6}\sqrt{24}\times 1000\times 0.270$$

$$\phi.Vc = 166 > Vu = 120.1kN$$

$\therefore$ *No Shear reiforcement is required*

4-14 Design of shear wall (W6 in Part 1):-

- **Material :-**
  concrete B300 Fc' = 24 N/mm^2
  Reinforcement Steel fy = 420 N/mm^2
- **According to soil profile type "SA" ,and Seismic Zone Factor "Z"=0.3:**
  Ca:" Seismic Coefficient" =0.24
  Cv:"Seismic Coefficient"=0.24
  I:"Importance Factor"=1.0
  R:"Structural System"=5.5
- **System For Shear Walls :-**
  The basement floor is considered as rigid box, due to enough walls .

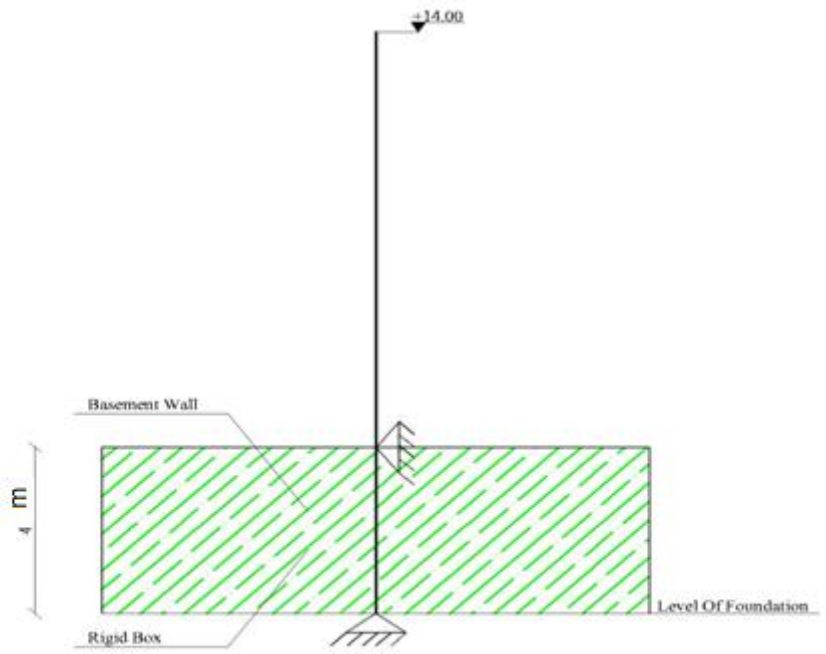


Fig.(4-14-1)
Shear wall W6 in part 1.

- **Section :-**

t=30 cm .shear wall thickness
Lw = 2.6 m .shear wall width
hw=14 m.

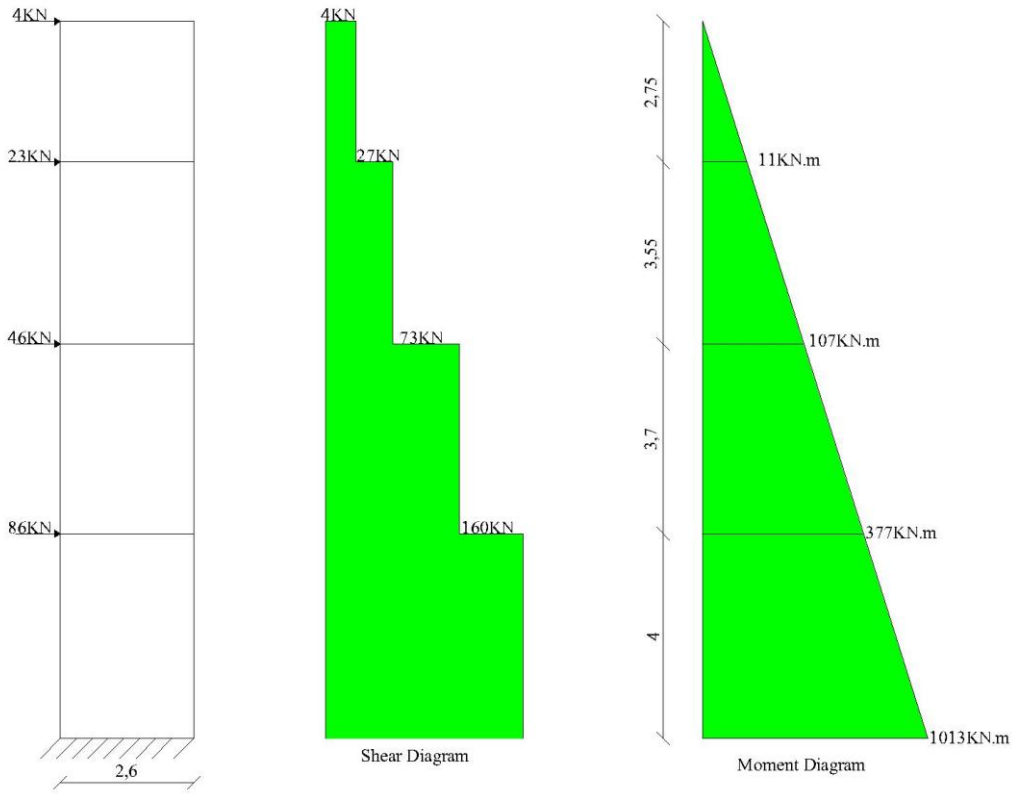


Fig. (4-14-2).
Load from earthquake (X Direction) for Shear wall (W05).

❖ **Design of the Horizontal reinforcement:**

Critical Section:

$\frac{lw}{2} = \frac{2.6}{2} = 1.3m \ldots\ldots \quad control$

$\frac{hw}{2} = \frac{14}{2} = 7.0m$

$d = 0.8 \times lw = 0.8 \times 1.3 = 1.04m$

$V_u = 160KN$

$M_u = 1013KN.m$

$$V_{c1} = \frac{\sqrt{fc'}}{6} \times b \times d$$

$$V_{c2} = \frac{\sqrt{fc'} \times b \times d}{4} + \frac{N_u \times d}{4 \times L_w}$$

$$V_{c3} = \left[\frac{\sqrt{fc'}}{2} + \frac{l_w \left(\sqrt{fc'} + \frac{2 \times N_u}{l_w \times h} \right)}{\left\langle \frac{M_u}{V_u} - \frac{l_w}{2} \right\rangle} \right] \times \frac{h \times d}{10}$$

$$V_{c1} = \frac{\sqrt{24}}{6} \times 0.30 \times 1.04 * 10^3 = 255\ KN$$

$$Assume\ N_u = 0$$

$$V_{c2} = \frac{\sqrt{24} \times 0.30 \times 1.04 * 10^3}{4} + \frac{0 \times 1.04}{4 \times 1.3} = 382\ KN$$

$$V_{c3} = \left[\frac{\sqrt{24}}{2} + \frac{1.3\left(\sqrt{24}\right)}{\left\langle \frac{1013}{160} - \frac{1.3}{2} \right\rangle} \right] \times \frac{0.30 \times 1.3}{10} \times 10^3 = 142KN \quad \text{Control}$$

$$\left\langle \frac{M_u}{V_u} - \frac{l_w}{2} \right\rangle = 5.2 > 0$$

$$\frac{A_{Vh}}{S_2} = \frac{V_s}{F_y \times d}$$

$$V_s = \frac{V_u}{\phi} - V_c = \frac{160}{0.75} - 142 = 71 \quad KN$$

$$\frac{A_{Vh}}{S_2} = \frac{71 \times 10^{-3}}{420 \times 1.04} = 0.00016m$$

$$\frac{A_{Vh\min}}{S_2} = 0.0025 \times h = 0.0025 \times 0.30 = 0.00075 \ldots \text{control}$$

$$S_2 \le \frac{l_w}{5} = \frac{1.3}{5} = 0.26\ m$$

$$S_2 \le 3 \times h = 3 \times 0.30 = 0.9m$$

$$S_2 = \frac{2 \times A_{vh}}{0.00075} = \frac{2 \times 113 \times 10^{-6}}{0.00075} = 0.3\ m$$

$$\therefore Use \phi 12@20cm \quad c/c \quad in\ two\ layers\ (horizontal)$$

❖ **Design of Vertical reinforcement:**

$$A_{Vn} = \left[0.0025 + 0.5\left(2.5 - \frac{h_w}{l_w} \right)\left(\frac{A_{vh}}{S_2 \times h} - 0.0025 \right) \right] \times S_1 \times h$$

$$A_{Vn} = \left[0.0025 + 0.5\left(2.5 - \frac{14}{2.6} \right)\left(\frac{2 \times 113}{200 \times 300} - 0.0025 \right) \right] \times S_1 \times h$$

$$A_{Vn} = 0.0025 \times S_1 \times h$$

$$S_1 = \frac{2 \times 113 \times 10^{-6}}{0.0025 \times 0.30} = 0.30\ m \quad \ldots \quad control$$

$$S_1 \le \frac{l_w}{3} = \frac{2.6}{3} = 0.86\ m$$

$$S_1 \le 3 \times h = 3 \times 0.30 = 0.9\ m$$

$$S1 \le 0.45m$$

$$\therefore Use \phi 12@20cm \quad c/c \quad in\ two\ layers\ (vertical)$$

❖ **Design of Moment:**

The boundary element is required if :

$$C \ge \frac{Lw}{600 * (\delta n / hw)}$$

$$\delta n : Lateral\ Displacement\ \ for\ wall$$

$$assume \quad \delta n / hw \ge 0.007$$

$$C \ge \frac{2.6}{600 * 0.007} = 0.6m$$

$$Cw = C - 0.1 \times Lw$$

$$Cw \ge 0.6 - 0.1 \times 2.6 = 0.35m$$

$$Cw \ge \frac{C}{2} = \frac{0.6}{2} = 0.3\ \ m$$

No boundary is required

4-15 Design of Dome:-

❖ **Materials :-**

Steel , fy = 4000 psi

Glass , 25KN/m^3

❖ **Loads:-**

Dead load=2*.07*25=0.5KN/m^2

Live load "Snow Load" =1.25KN/m^2

Wind Load

After Design We get the profile as Horizontal is HSS 4X2 X0.125

as Vertical HSS 4 X 2 X 0.125

❖ **Design Strength of Weld Metal:-**

$a = 0.25''$

$Fpu''weld'' = 70ksi$

$T = Lw * \phi Rnw$

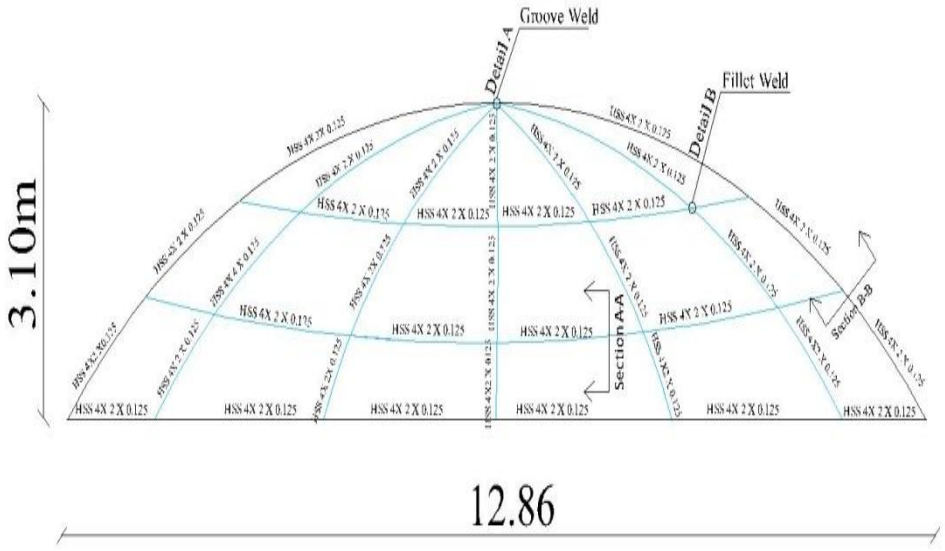
$T = 0.027KN \rightarrow From\ ''Sap''$

$$\frac{0.027}{4.448} = Lw * 0.75(0.707 * \frac{1}{4}) * 0.6 * 70$$

$Lw = 0.003inch$

$Lw = 0.075mm$

$Select\ Lw = 1.0cm$



Elevation of Dome

Fig. (4-15-1)