4

Chapter 4

Structural Analysis & Design

City Center "A"-

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City Center "B"-

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4.1 Introduction:

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

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

4.1.1 Design method and requirements:

The design strength provided by a member is calculated in accordance with the requirements and assumptions of ACI_code (318M_14).

4.1.2 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 occur.

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.

4.2 Factored loads: -

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

W_u = 1.4 D_LACI-code-318-14(9.2.1).

W_u = 1.2 D_L + 1.6 L_L ACI-code-318-14(9.2.2).

Materials:-

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

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

$f_{yt} = 420\ Mpa$, will be used in design and calculations.

4.3Slabs Thickness calculation:-

According to ACI-Code-318-14 table 9.5(a), the minimum thickness of non- prestressed beams or one way, slabs unless deflections are computed for one end continuous for one-way rib slabb given as following:

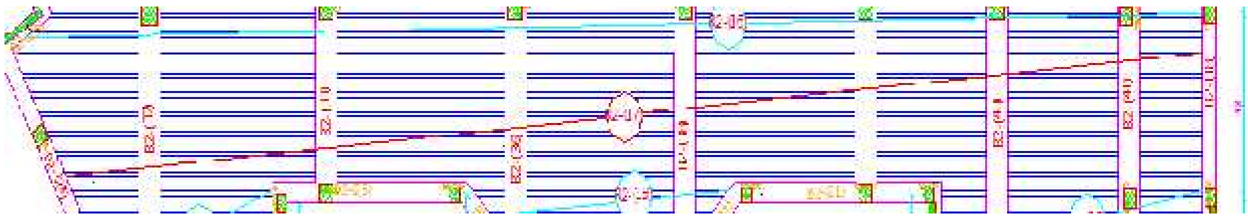


Fig (4-1):Rib(R2-(17)) at the First floor

Fig (4-2): spans of rib (R2-(17))

Hminfor two end continuous beam

Hmin = L/21 longest two end continuous supported is 6.78m

Hmin =6780/21 = 322.85 mm

For First floor slab, use thickness of slab 35cm.

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:

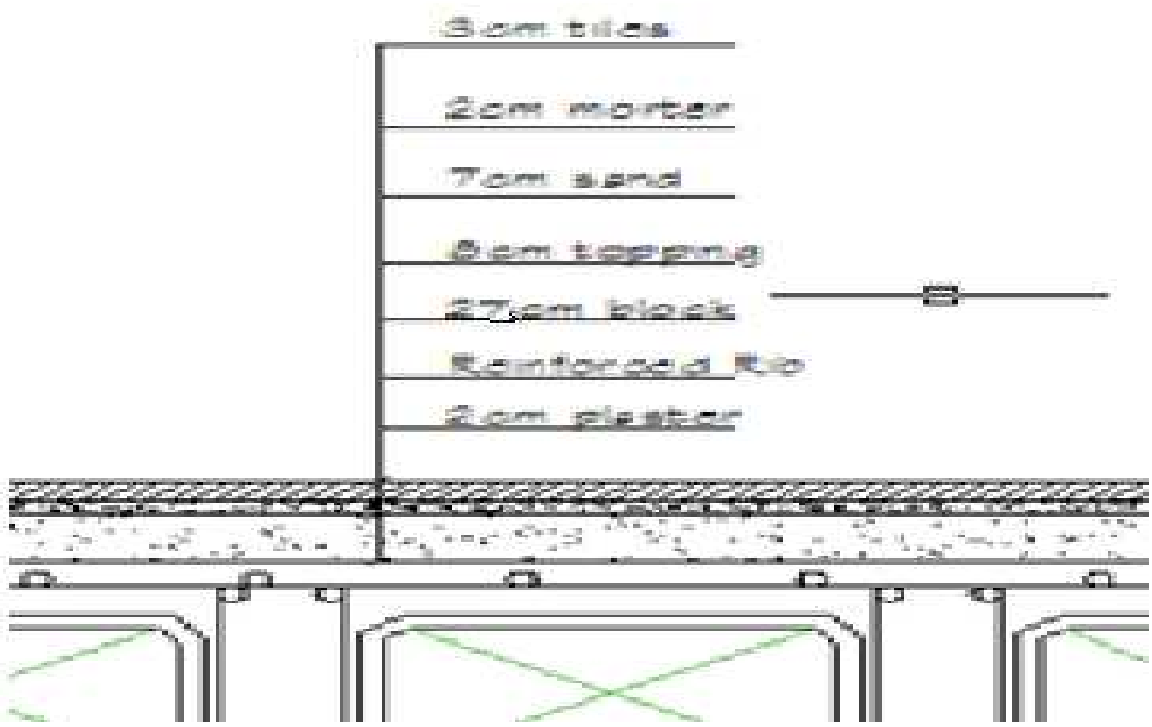


Fig (4-3) Typical section in ribbed slab

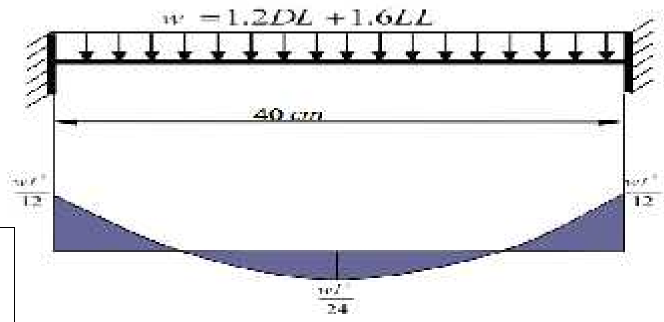


Fig (4-4) Typical section in topping

Material	Unitweight (KN/m³)	Thickness (cm)	load
Tile	**23**	3	0.69
Mortar	**22**	2	0.44
Sand	**16**	7	1.12
Topping slab	**25**	8	2
partition	$2.3 KN/m^2$		2.3
$D.L_{tot}$			6.55

table (4-1) calculation of total load ofR1-09

4.5 Design of Topping:-

4.5.1 Calculation of Dead load For 1m strip

Material	Unit weight(kN/m3)	الرقم المتسلسل
tile	23	**1**
mortar	22	**2**
sand	16	**3**
toppting	25	**4**
block	10	**5**
rib	25	**6**
plaster	22	**7**
partion	2.3(KN/m2)	**8**

table (4-2) calculation of dead load for topping.

4.5.2 Calculation of live load

From Jordan's Code

$L.L_{total}$ = 4KN/m

Wu = 1.2D.L + 1.6L.L

= 1.2*6.55 + 1.6*4 = 14.3 KN/m

Design of shear :-

Used fy = 420 MPa& fc' = 24MPa

Φ* Vc = $0.75 \times \sqrt{24} \times \frac{1}{6} \times 1000 \times 80 \times 0.001 = 49KN > 2.86kN * ¤$

No shear reinforcement is required.

CheckΦMn> Mu

$$Mu = \frac{w_u * l^2}{12} = \frac{14.3 * 0.4^2}{12} = 0.19kN.m$$

$$Mn = 0.42\sqrt{fc'} * s$$

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

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

$$Mn = 0.42 \times \sqrt{24} \times \frac{1000 * 80^2}{6} \times 10^{-6} = 2.19kN.m$$

Ø=0.55 for plain concrete

$¤ \times Mn = 0.55 * 2.19 = 1.205kN.m.$

$¤ \times Mn = 1.204kN.m > Mu = 0.195kN.m.$

No reinforcement is required according to ACI-Code -318M-14, so As min for slabs as Shrinkage and temperature reinforcement .

<u>Shrinkage and temperature reinforcement must be provided.</u>

For the shrinkage and temperature reinforcement:

° = 0.0018

ACI-318-14 (7.12.2)

*As = ° * b * h = 0.0018 * 1000 * 80 = 144 mm2/ m*

As (8) =50.27mm^2

So number of bars =144/50.27 = 2.86

1/N =350 mm

The step is the smallest of :-

1_ S=3*h = 240mm. control

2_ S =380($\frac{280}{fs}$)2.5Cc= 380($\frac{280}{(2/3)\times 420}$)-2.5*20

= 330

select mesh 8**/**20cm, *As.*prov = 2.51 cm^2/m >***As***min=1.44 cm^2/m

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

From practical consideration, the secondary reinforcement parallel to the rib shall be placed in the slab and spaced at distance not more than half of the spacings between ribs (usually two bars upon each 40 cm width block).

4.6 Design of Rib (R1-(09)):-

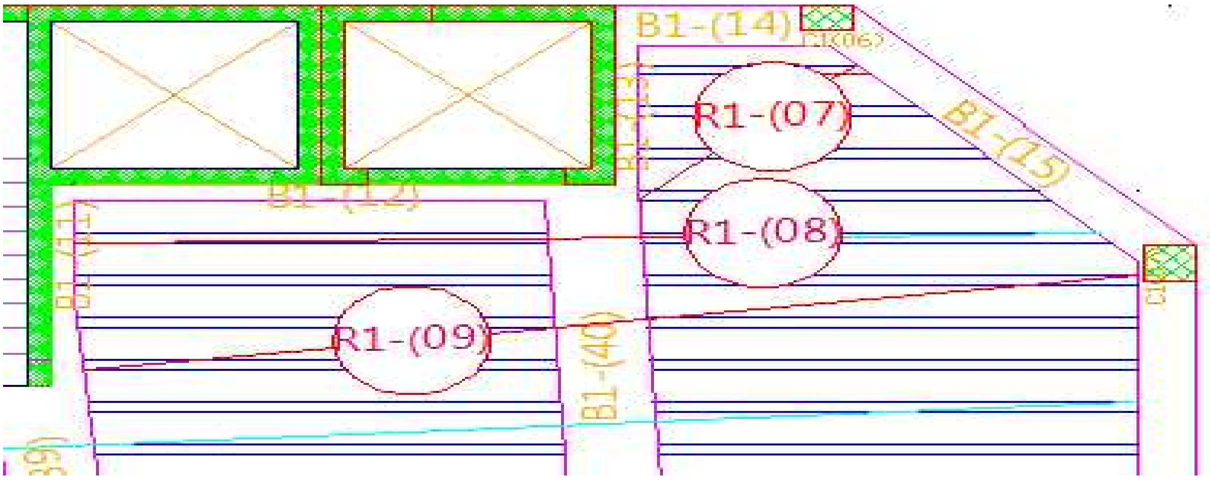


Fig (4-5):Rib(R1-(09)) at the First floor

4.6.1 Design constant:-

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

b_E =Ln/4= 4.83/ 4 =1.21m

b_E = bw + 16 tf = 12 + 16 (8) = 1.4 m

b_E = c/c spacing between adjacent ribs =0.52 m

Control … 52cm

<u>- Requirements for Slab Floor According to *ACI- (318M-14).*</u>

bw ≥ 10cm……………………………………………ACI(8.13.2)

Select bw=12cm

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

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

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

Select tf=8cm

4.6.2 Calculation of Dead load:-

Dead load Calculation		
Tiles	**23*0.03*0.52**	**= 0.3588 KN/m**
Mortar	**22*0.02*0.52**	**= 0.2288 KN/m**
Sand	**16*0.07*0.52**	**= 0.5824 KN/m**
Topping	**25*0.08*0.52**	**= 1.04 KN/m**
Block	**10*0.27*0.4**	**= 1.08 KN/m**
Rib	**25*0.27*0.12**	**= 0.81KN/m**
Plastering	**22*0.02*0.52**	**=0.2288 KN/m**
Partition	**2.3*0.52**	**=1.196 KN/m**

Table (4-3) calculation of the total load for (R1-(09)).

Total dead load = 5.584 KN/m/rib

4.6.3 Calculation of Live load:-‘

From Jordanian live loads table live load for malls is 4 KN/m^2

Total live load = 4*0.52 = 2.08 KN/m/rib

Material :-

concrete B300 Fc' = 24 N/mm^2

Reinforcement Steel fy = 420 N/mm^2

Section :-

b =12cm bf=52 cm

h =35cmTf=8 cm

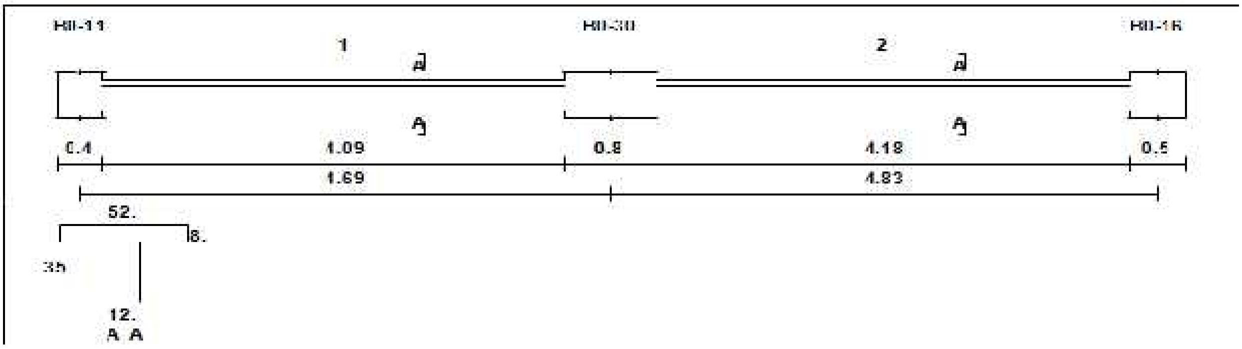


Fig. (4-6) Geometry of Rib (R1-(09)).

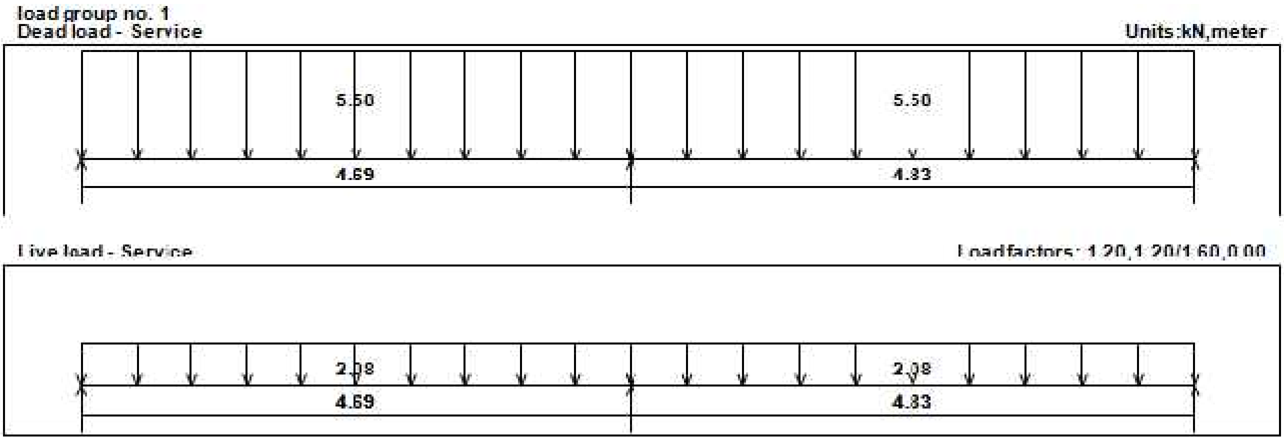


Fig. (4-7) :Service load of Rib (R1-(09))

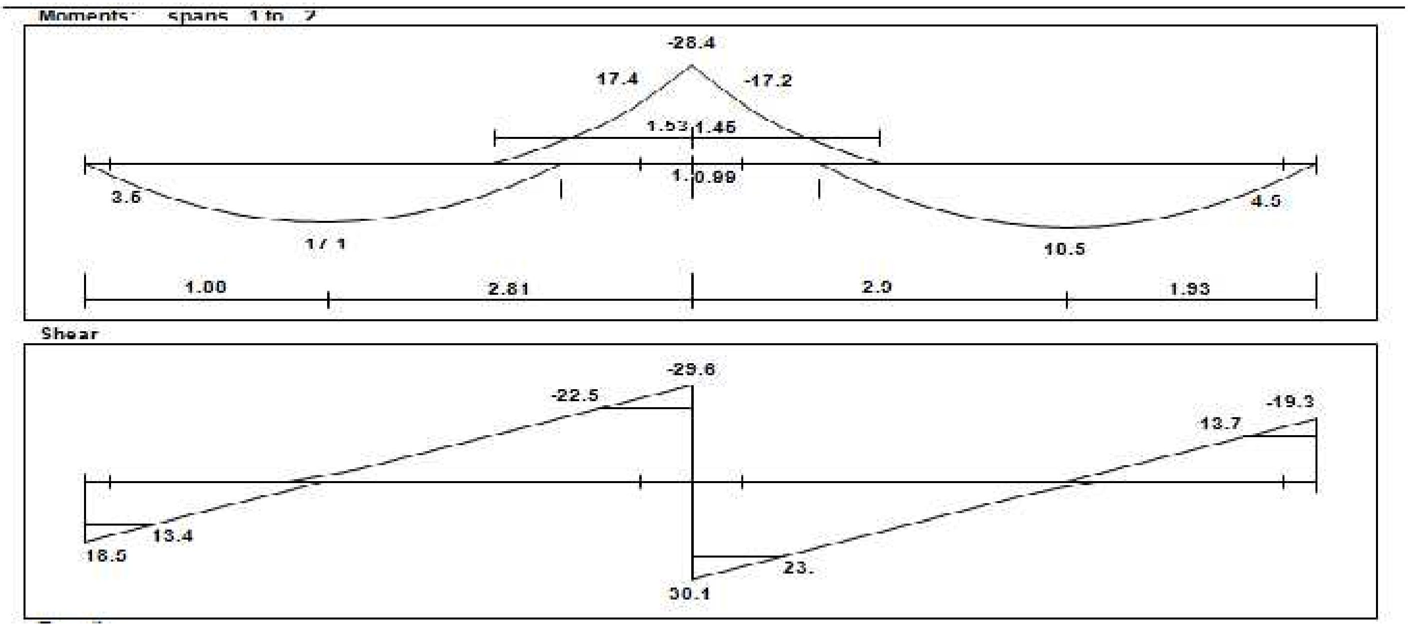


Fig. (4-8) :Rib Envelope(R1-(09))

Reactions

Factored			
DeadR	11.66	39.88	12.25
LiveR	6.84	19.81	7.02
Max R	18.51	59.68	19.27
Min R	10.61	49.58	11.32
Service			
DeadR	9.72	33.23	10.21
LiveR	4.28	12.38	4.39
Max R	14.	45.61	14.59
Min R	9.06	39.29	9.63

Fig. (4-9) :RibReactions(R1-(09))

4.6.4 Design of flexure:-

4.6.4.1 Design of Negative moment of rib (R1-(09)):

1) Maximum negative moment $Mu^{(-)}$ **=17.4KN.m.**

d = depth - cover – diameter of stirrups – (diameter of bar/ 2)

$= 350 - 20 - 10 - \frac{12}{2} = 315$ mm.

Mn = Mu /ϕ =17.4 / 0.9 = 19.33KN.m

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

$K_n = \frac{M_n}{b*d^2} = \frac{19.33*10^{-3}}{0.12*(0.315)^2} = 1.623$MPa

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

$= \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2*1.623*20.6}{420}}\right) = 0.00403$

$\rightarrow A_s = \rho * b * d = 0.00403 * 120 * 315 = 152.334\ mm^2$.

$As_{min} = \frac{\sqrt{f'_c}}{4\,(f_y)} * b_w * d \;\geq\; \frac{1.4}{f_y} * b_w * d$ …………(ACI-10.5.1)

$= \frac{\sqrt{24}}{4*420} * 120 * 315 \;\geq\; \frac{1.4}{420} * 120 * 315$

= 110.23 mm^2<126 mm^2 …………. Larger value is control.

→ As_{min} = 126 mm^2 < As_{req} = 152.334 mm^2.

∴ As = 152.334 mm^2.

2 Φ10 = 157.08 mm^2 > As_{req} = 152.334mm^2 . OK.

∴ Use 2 Φ10

→ **Check for strain:- ($\varepsilon_s \geq 0.005$)**

Tension = Compression

A_s * fy = 0.85 * f'_c * b * a

157.08 * 420 = 0.85 * 24 * 120 * a

a = 26.95 mm.

$c = \frac{a}{\beta_1} = \frac{26.95}{0.85}$ = 31.7 mm. * Note: f'_c = 24MPa< 28 MPa→ β_1 = 0.85

$\varepsilon_s = \frac{d-c}{c}$ * 0.003

$= \frac{315-31.7}{31.7}$ * 0.003 = 0.027> 0.005 ∴ф =0.9 OK

4.6.4.2Design of Positive moment of rib (R1-(09))

d = depth - cover – diameter of stirrups – (diameter of bar/ 2)

= 350– 20 – 10 –$\frac{10}{2}$= 315 mm.

→**$M_{u\,max}$ = 18.5KN.m**

b_E ≤ Distance center to center between ribs = 520 mm……….. Controlled.

≤ Span/4 =4830/4 = 1207.5 mm.

≤ (16* t_f) + b_w =(16* 80) +120 =1400 mm.

→ **b_E= 520 mm.**

→ $M_{nf} = 0.85\, f_c' * b_E * t_f * \ d - \frac{t_f}{2}$

$= 0.85 * 24 * 0.52 * 0.08 * \ 0.315 - \frac{0.08}{2} \ * 10^3 = 233.37\ KN.m$

ϕM_{nf} = 0.9 * 233.37 = 210.0KN.m

→ ϕM_{nf} =210.0KN.m> $M_{u\,max}$= 18.5KN.m.

∴ Design as rectangular section.

1) Maximum positive moment $Mu^{(+)}$ **= 18.5KN.m**

Mn = Mu /ϕ =18.5 / 0.9 = 20.56KN.m

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

$K_n = \frac{M_n}{b*d^2} = \frac{20.56*10^{-3}}{0.52*(0.315)^2}$ = 0.398MPa

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

$= \frac{1}{20.6} \ 1 - \sqrt{1 - \frac{2*0.398*20.6}{420}}$ = 0.00096

→ $A_s = \rho$ * b *d = 0.00096 * 520 *315 = 157.248 mm^2.

$As_{min} = \frac{\sqrt{f_c'}}{4\,(f_y)} * b_w * d \ \geq \frac{1.4}{f_y} * b_w * d$ …………(ACI-10.5.1)

$= \frac{\sqrt{24}}{4*420} * 120 * 315 \ \geq \frac{1.4}{420} * 120 * 315$

= 110.22 mm^2<126 mm^2 …………. Larger value is control.

→ As_{min} = 126 mm^2<As_{req} = 157. mm^2.

∴ As = 157.248 mm^2.

2 Φ10= 157.1 mm^2>As_{req} = 157 mm^2 . OK.

∴ Use 2 Φ10

→ **Check for strain:- ($\varepsilon_s \geq 0.005$)**

Tension = Compression

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

157.1* 420 = 0.85 * 24 * 520 * a

a = 6.22 mm.

$c = \frac{a}{\beta_1} = \frac{6.22}{0.85}$ = 7.3 mm. * Note: f_c' = 24MPa< 28 MPa→ β_1 = 0.85

$\varepsilon_s = \frac{d-c}{c}$ * 0.003

= $\frac{315-7.3}{7.3}$ * 0.003 = 0.126> 0.005 ∴ф =0.9 OK

2) Maximum positive moment ***Mu*** **(+) =17.1KN.m**

Mn = Mu /ф =17.1 / 0.9 =19KN.m

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

$K_n = \frac{M_n}{b*d^2} = \frac{17.1*10^{-3}}{0.52*(0.315)^2}$ = 0.33MPa

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

$= \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2*0.33*20.6}{420}}\right)$ = 0.00079

→A_s = ρ * b *d = 0.00079* 120 *315 = 29.86mm^2.

$As_{min} = \frac{\sqrt{f_c'}}{4\,(fy)} * b_w * d \;\geq\; \frac{1.4}{fy} * b_w * d$ …………(ACI-10.5.1)

$= \frac{\sqrt{24}}{4*420} * 120 * 315 \;\geq\; \frac{1.4}{420} * 120 * 315$

= 110.2mm^{2}126 mm^2 …………. Larger value is control.

→As_{min} = 126mm^2>As_{req}29.86 mm^2.

∴ As = 126mm^2.

2 Φ10 = 157 mm^2>As_{req} = 126 mm^2 . OK.

∴ Use 2 Φ10

→ **Check for strain:- ($\varepsilon_s \geq 0.005$)**

Tension = Compression

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

157* 420 = 0.85 * 24 * 520* a

a = 6.216mm.

$c = \frac{a}{\beta_1} = \frac{6.216}{0.85} = 7.313$ mm. * Note: $f_c' = 24$ MPa< 28 MPa→ $\beta_1 = 0.85$

$\varepsilon_s = \frac{d-c}{c} * 0.003$

$= \frac{315-7.313}{7.313} * 0.003 = 0.126 > 0.005$ ∴ф =0.9 OK

4.6.4.3Design of shear of rib (R1-(09))

1) Vu = 13.7 KN.

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

$= 0.75 * \frac{\sqrt{24}}{6} * 0.12 * 0.315*10^3 = 23.1$ KN.

$1.1* \phi V_c = 1.1 * 23.1 = 25.6$ KN.

→**Check for items:-**

1- <u>Item 1 :</u> $V_u \le \frac{\phi V_c}{2}$.

$13.7 \le \frac{25.6}{2} = 12.8$…....Not satisfy

2- <u>Item 2 :</u> $\frac{\phi V_c}{2} < V_u \le \phi V_c$

12.8 ≤ 13.7 ≤ ≤ 600 mm.

∴ **Item (2) is satisfy → minimum shear reinforcement is required.**

$(\frac{A_v}{s})_{min} \ge \frac{1}{16} * \frac{\sqrt{f_c'}}{f_{yt}} * b_w = \frac{1}{16} * \frac{\sqrt{24}}{420} * 0.12 = 8.75*10^{-5}$.

$\ge \frac{1}{3} * \frac{b_w}{f_{yt}} = \frac{1}{3} * \frac{0.12}{420} = 9.52*10^{-5}$……………Control.

<u>Try Φ8 (2 Legs):</u>

$\frac{2*50*10^{-6}}{S} = 9.52*10^{-5}$ → S = 1.05 m

$S \le \frac{d}{2} = \frac{315}{2} = 157.5$ mm.

≤ 600 mm.

∴ Use Φ8 @ 10 Cm

2) Vu = 23 KN.

$\phi V_c = \phi * \frac{\sqrt{f'_c}}{6} * b_w * d$

$= 0.75 * \frac{\sqrt{24}}{6} * 0.12 * 0.315 * 10^3 = 23.15$ KN.

$1.1 * \phi V_c = 1.1 * 23.15 = 25.6$ KN.

→Check for items:-

1- Item 1 : $V_u \le \frac{\phi V_c}{2}$.

$23 \le \frac{25.6}{2} = 12.8$…....Not satisfy

2- Item 2 : $\frac{\phi V_c}{2} < V_u \le \phi V_c$

$12.8 \le 23 \le 25.6$ … …. Satisfy.

∴ Item (2) is satisfy → minimum shear reinforcement is required.

$(\frac{A_v}{s})_{min} \ge \frac{1}{16} * \frac{\sqrt{f'_c}}{f_{yt}} * b_w = \frac{1}{16} * \frac{\sqrt{24}}{420} * 0.12 = 8.75*10^{-5}$.

$\ge \frac{1}{3} * \frac{b_w}{f_{yt}} = \frac{1}{3} * \frac{0.12}{420} = 9.52*10^{-5}$…………….Control.

Try Φ8 (2 Legs):

$\frac{2*50*10^{-6}}{S} = 9.52*10^{-5} \rightarrow S = 1.05$ m

$S \le \frac{d}{2} = \frac{315}{2} = 157.5$ mm.

≤ 600 mm

.Use Φ8 @ 10 Cm

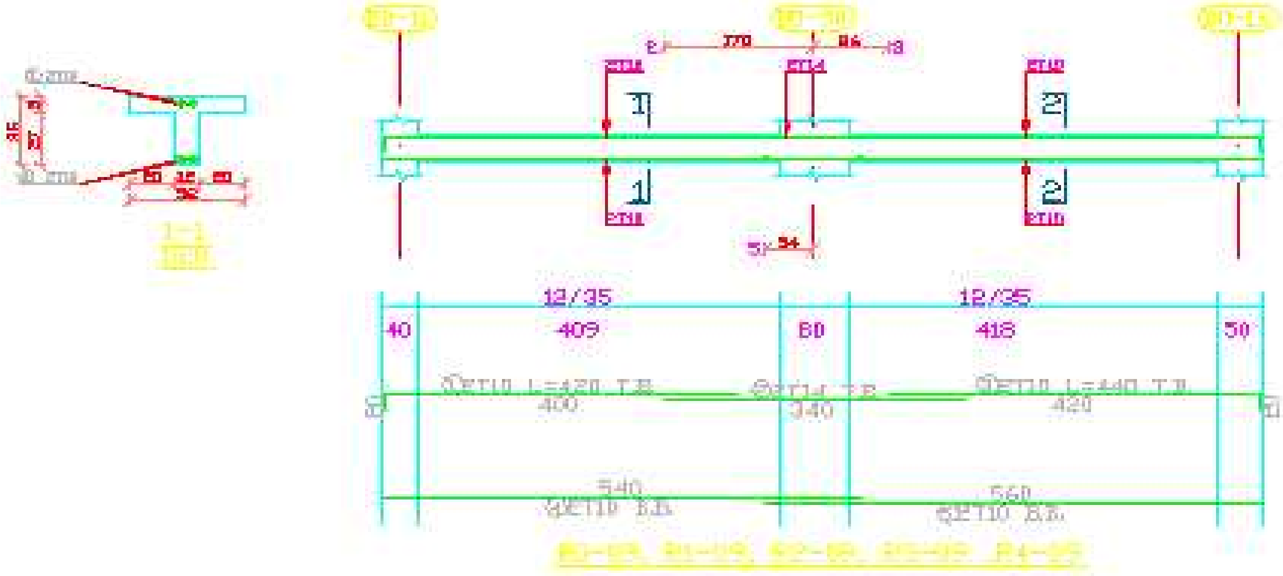


Fig (4-10): Reinforcement of Rib(R1-(09))

4.7 Design of Beam (B1-(17)):

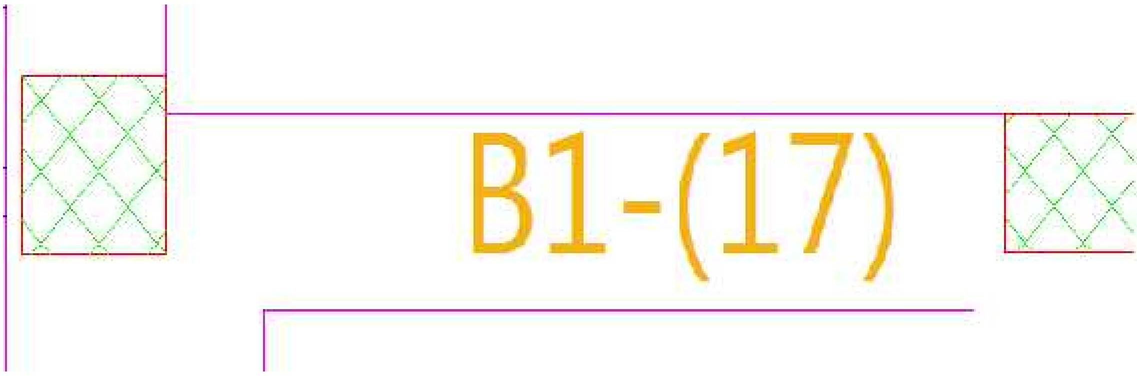


Fig (4-11) Location of beam (B1-(17))

Material :-

concrete B300 Fc' = 24N/mm^2

Reinforcement Steel fy = 420 N/mm^2

Section :-

B =50cm

h =35cm

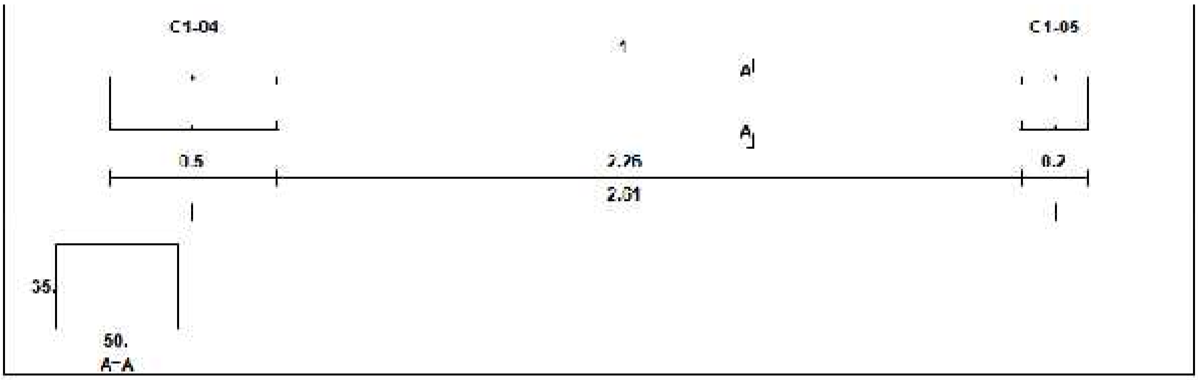


Fig (4-12) : Beam Geometry (B1-(17)).

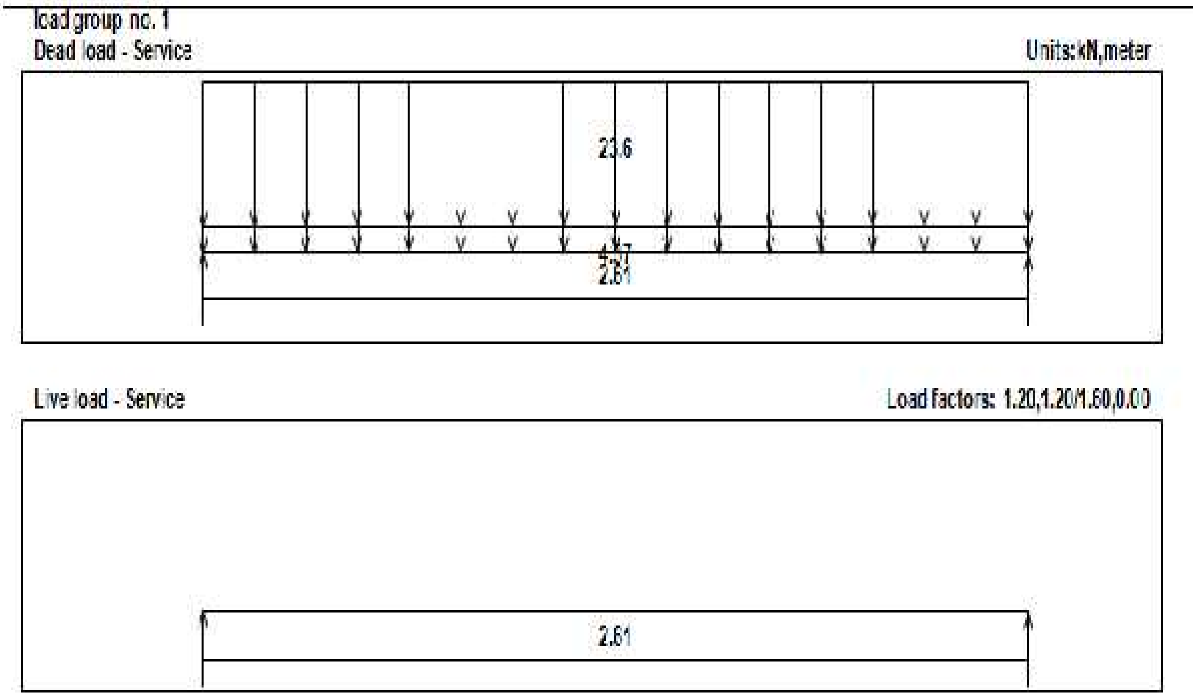


Fig (4-13) : Service Load of Beam (B1-(17))

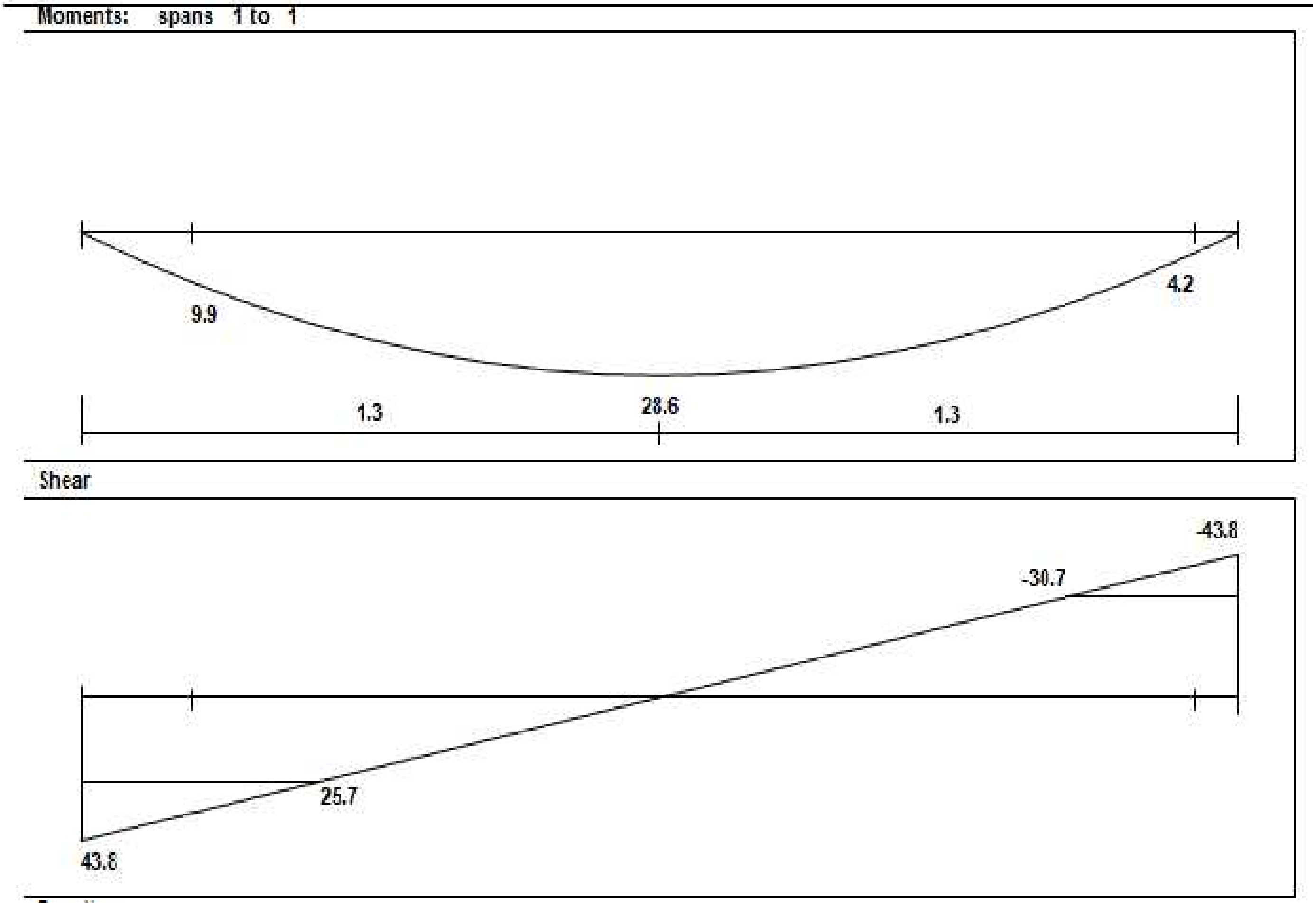


Fig (4-14) : Beam Envelop (B1-(17)).

4.7.1 Check whether the section will be act as singly or doubly reinforcement section:

→ Mu_{max} = 28.6KN.m .

b_w = 50Cm. , h =35 Cm.

d = depth - cover – diameter of stirrups – (diameter of bar/ 2)

$= 350 - 40 - 10 - \frac{12}{2} = 294$ mm.

$C_{max} = \frac{3}{7} * d = \frac{3}{7} * 294 = 126$ mm.

$a_{max} = \beta_1 * C_{max} = 0.85 * 126 = 107.1$ mm. *Note: $f_c' = 24MPa < 28\ MPa \rightarrow \beta_1 = 0.85$

$Mn_{max} = 0.85 * f_c' * b * a * (d - \frac{a}{2})$

$= 0.85 * 24 * 0.5 * 0.1071 * (0.294 - \frac{0.1071}{2}) * 10^3$

= 262.67KN.m .

$\phi = 0.65 + \frac{250}{3} * (0.004 - 0.002) = 0.816$

→ ϕMn_{max} = 0.82 * 262.67 = 215.39KN.m . * Note: $\epsilon_s = 0.004 \rightarrow \phi = 0.82$

→ ϕMn_{max} = 215.39KN.m> Mu =28.6KN.m .

∴ Singly reinforced concrete section.

4.7.2 Flexure design:

4.7.2.1 Design of Positive moment:-

1) **Maximum negative moment $Mu^{(-)}$ = 28.6KN.m .**

ϕMn_{max} = 215.39KN.m> Mu = 28.6KN.m → Singly reinforced concrete section

Mn = Mu /ɸ= 28.9 / 0.9 = 32.11KN.m .

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

$K_n = \frac{M_n}{b*d^2} = \frac{32.1*10^{-3}}{0.5*(0.294)^2} = 0.74$MPa.

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

$= \frac{1}{20.6}\ 1 - \sqrt{1 - \frac{2*0.74*20.6}{420}}$ = 0.0017

→ $A_s = \rho * b_w * d$ = 0.0017 * 500 *294 =249.9 mm^2.

$As_{min} = \frac{\sqrt{f'_c}}{4\,(f_y)} * b_w * d \ \geq \frac{1.4}{f_y} * b_w * d$ …………(ACI-10.5.1)

$= \frac{\sqrt{24}}{4*420} * 500 * 294 \ \geq \ \frac{1.4}{420} * 500 * 294$

= 428.66 mm^2<490 mm^2 …………. Larger value is control.

→ As_{min} = 490 mm^2>As_{req} =249.9 mm^2.

∴ As = 490.9 mm^2.

0f Φ12 = $\frac{As_{req}}{A_{bar}} = \frac{490.9}{113.09}$ =4.3→ # of bars = 5 bars.

∴ Use 4Φ12 → As =5* 113.09 = 565.45 mm^2>As_{req} = 490.9 mm^2 .

→ Check for strain:- ($\varepsilon_s \geq 0.005$)

Tension = Compression

A_s * fy = 0.85 * f'_c * b * a

565.45* 420 = 0.85 * 24 * 500* a

a = 23.3mm.

$c = \frac{a}{\beta_1} = \frac{23.3}{0.85}$ = 27.4 mm. * Note: f'_c = 24 MPa< 28 MPa→ β_1 = 0.85

$\varepsilon_s = \frac{d-c}{c}$ * 0.003

$= \frac{294-27.4}{27.4}$ * 0.003 = 0.029> 0.005 ∴ф =0.9 OK

∴ Use 4Φ12.

4.7.2.3Design of shear:-

1) Vu = 30.7KN .

фVc = ф * $\frac{\sqrt{f'_c}}{6}$ * b_w * d

= 0.75 * $\frac{\sqrt{24}}{6}$ * 0.5 * 0.294 * 10^3 = 90 KN.

Check for section dimensions:

$\phi Vc + (\frac{2}{3} * \phi * \sqrt{f_c'} * b_w * d) = 119.4 + (\frac{2}{3} * 0.75 * \sqrt{24} * 0.5 * 0.294 * 10^3)$

$= 119.4 + 360.1 = 479.47KN >>> Vu = 30.7$ KN.

∴ Dimension is big enough.

4.7.2.4 Check for the case of shear:

1- Item 1 : $V_u \le \frac{\phi V_c}{2}$.

$30.7 \le \frac{90}{2} = 45$….... satisfy.

∴ Item (1) is satisfy → minimum shear reinforcement is required.

$(\frac{A_v}{s})_{min} \ge \frac{1}{16} * \frac{\sqrt{f_c'}}{f_{yt}} * b_w = \frac{1}{16} * \frac{\sqrt{24}}{420} * 0.12 = 8.75*10^{-5}$.

$\ge \frac{1}{3} * \frac{b_w}{f_{yt}} = \frac{1}{3} * \frac{0.12}{420} = 9.52*10^{-5}$……………Control.

Try Φ8 (2 Legs):

$\frac{2*50*10^{-6}}{S} = 9.52*10^{-5} \rightarrow S = 1.05$ m

$S \le \frac{d}{2} = \frac{294}{2} = 147$ mm. ≤ 600 mm.

∴ Use Φ8 @ 10 Cm

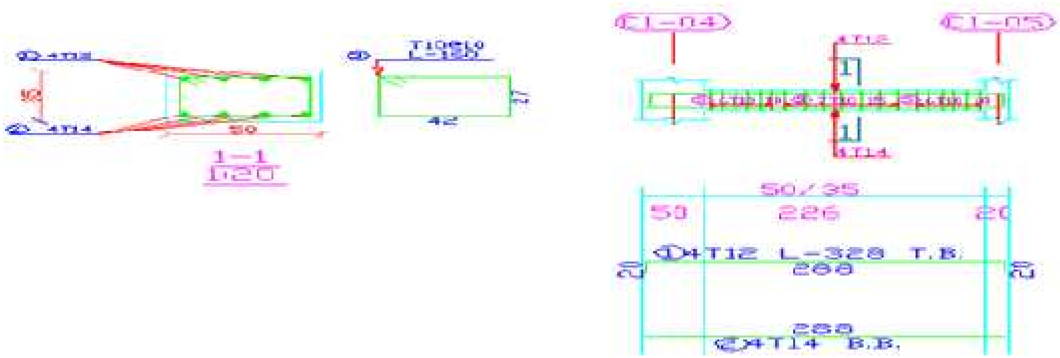


Fig. (4-15) Detail of Beamandsection(B1-(17)).

4.8 Design of Column(C32):-

4.8.1 Load calculation:

DL= 2933.68 KN LL= 993.66 KN

P_u = 5110.275 KN $P_{n,req}$=5110.275/0.65 =7862 KN

Assume rectangular section with **= 2.38%**

P_n = 0.8× Ag× (0.85× fc'+ $_g$×(fy - 0.85 fc'))

7862 = 0.8× Ag×(0.85× 24 + 0.0238× (420 - 0.85× 24))

A_g = 3285.6 cm^2

Use 60*55 cm with Ag = 3300cm^2 >$A_{g,req}$ = 3285.6 cm^2

4.8.2 Check slenderness effect:

Lu: Actual unsupported (unbraced) length.

K: effective length factor (K= 1 for braced frame).

R: radius of gyration =$\sqrt{(I/A)} = 0.3\,h$

Lu = 2.76 m

M1/M2 =1

In 60cm -Direction

Klu/r< 34- 12 ($M1/M2$) < 40

$(1 \times 2.76)/(0.3 \times 0.6)$ = 15.33<22 =>Short

In 55cm -Direction

Klu/r< 34- 12 ($M1/M2$)

$(1 \times 3.78)/(0.3 \times 0.55)$ = 16.73 > 22 => Short

Short in Both Direction

→**Here we can solve this column as short tied column**

$P_n = 0.8\times Ag\times (0.85\times fc' + \ _g\times(fy - 0.85\ fc'))$

$P_n = 0.8\times 600\times 550\times (0.85\times 24 + 0.0238 \times (420 - 0.85\times 24))$

$=7896.4$ KN $>P_{n,req}=7862$ KN …….OK

4.8.4 Design of the tie reinforcement :

S ≤16 db (longitudinal bar diameter)

S≤ 48dt (tie bar diameter).

S ≤ Least dimension.

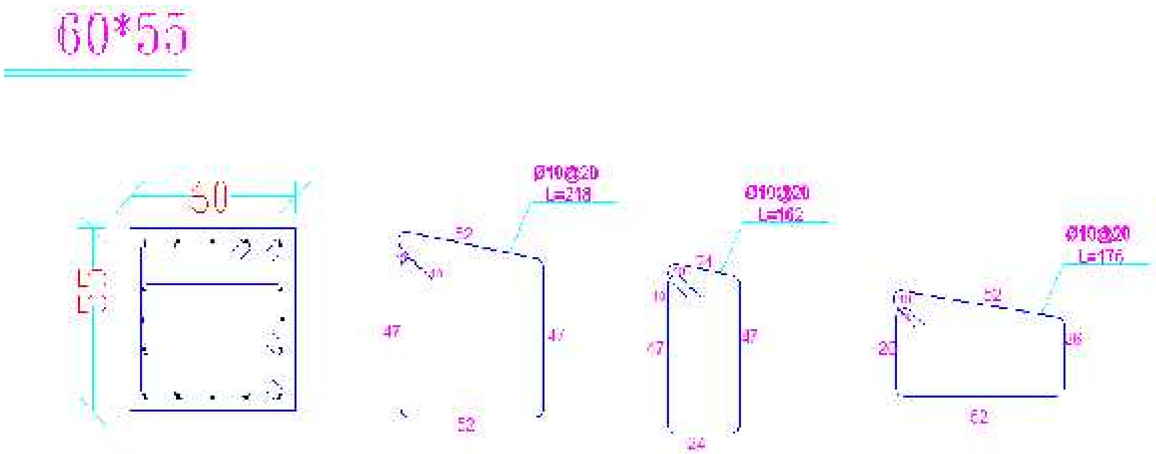
spacing $\leq 16\times d_b=16\times 2.5=40$ cm …. control

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

spacing ≤ least.dim=55 cm

Use¤10@20 cm

For UingSbCoulmn We have using **16¬25 .**



16¬25 .

Fig. (4-16) Detail of Reinforcement of Coulmn (C32)

4.9 Design of Stair.

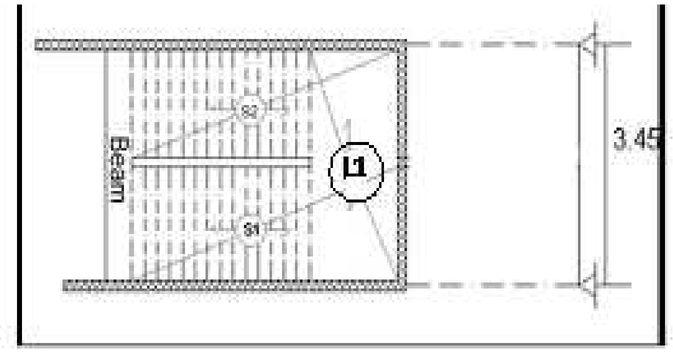


Figure 4-17: Details of stair .

4-9-1 Minimum slab:

$h_{min}= \frac{L}{20} = \frac{410}{20} = 20.5cm$ thickness for deflection (for simply supported one way

solid

$Take\ h_{min} = 250mm.$

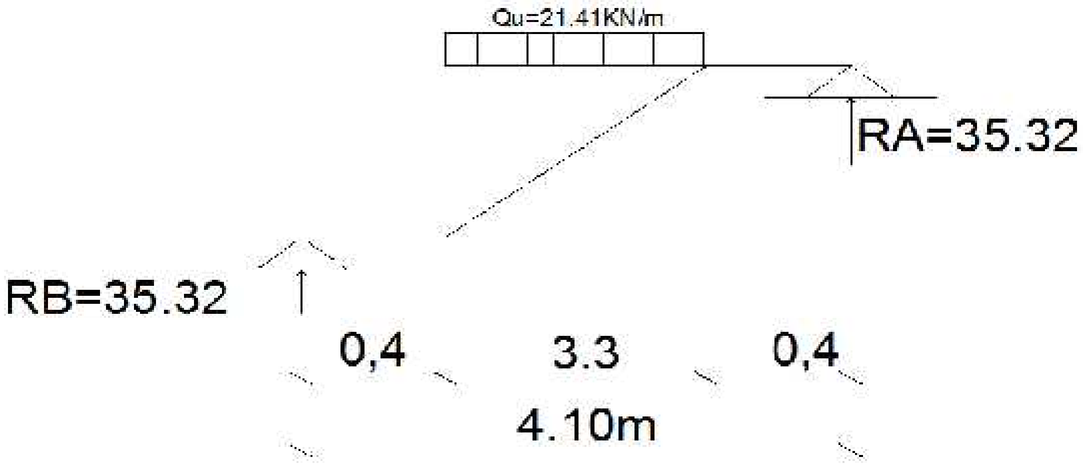


Figure 4-18: loads of the flight .

4-9-2Loads Calculation of stair case (1):

Flight Dead Load computations:

r $= tan - 1\,(\frac{rise}{run})$:

material	Quality Density KN/m^3	$W\ kN/m$
Tiles	23	$27(\frac{0.17+0.35}{0.3}) * 0.03 * 1 = 1.403$
Mortar	22	$22*(\frac{0.17+0.3}{0.3}) * 0.02 * 1 = 1.034$
Stair steps	25	$\frac{25}{0.3}*(\frac{0.17+0.3}{2}) * 1 = 2.125$
R.C solid slab	25	$\frac{25*0.25*1}{cos29.54^{\circ}} = 7.184$
Plaster	22	$\frac{22*0.03*1}{cos29.54} =0.76$
Total Dead Load	Σ	12.506KN

Table 4-4: Dead load calculation for flight of stair .

Landing Dead load computation:

Material	Quality Density KN/m^3	$W \quad KN/m$
Tiles	23	$23 * 0.03 * 1 = 0.69$
Mortar	22	$22 * 0.02 * 1 = 0.44$
R.C solid slab	25	$25 * 0.25 * 1 = 6.25$
Plaster	22	$22 * 0.03 * 1 = 0.66$
Total Dead load	Σ	8.04

Table 4-5: Dead load calculation for landing of stair.

$$* \, live\ load = \ LL = 4KN/m^2$$

Total factored load: $w = 1.2D + 1.6L$

$for\ flight \quad w = 1.2 * 12.506 + 1.6 * 4 = 21.41\ KN/m$

$for\ landing \quad w = 1.2 * 8.04 + 1.6 * 4 = 16.05N/m$

4-9-3Design of flight (Slab S1 is supported at the centerline of beam and L1).

The reaction at point A:

$$R_B = \ R_A = \frac{21.41 * 3.3}{2} = 35.32KN$$

- Check for shear strength:

Assume bar diameter Ø14 for main reinforcement.

$$d = h - 20 - \frac{d_b}{2} = 250 - 20 - \frac{14}{2} = 223mm$$

Take the maximum shear as the support reaction

$$V_u = 35.32 * cos29.54 = 28.27KN$$

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

$= \frac{\sqrt{24}}{6} * 1000 * 223 * 10^{-3} = 182.1 \, KN.$

$$ф * V_c = 0.75 * 182.1 = 136.55 \, KN/1m \, strip$$

$V_{u,max} = 28.27 < \frac{1}{2} ф * V_c = 68.27 \, KN$ ….. $\boldsymbol{The\ thickness\ of\ the\ slab\ is\ enough}$.

Calculate the maximum bending moment and steel reinforcement:

$$M_u = 35.32 * 2.05 - 21.41 * 1.65 * \frac{1.65}{2} = 43.26KN.m$$

$Mn = Mu / ф = 43.26 / 0.9 = 48.067 \, KN.m$

$$d = depth - cover - diameter\ of\ stirrups - diameter\ of \frac{bar}{\ } 2$$

$300 - 20 - \frac{14}{2} = 223 \, mm.$

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

$R_n = \frac{M_n}{b*d^2} = \frac{48.067*10^6}{1000*223^2} = 0.97MPa$

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

$= \frac{1}{20.6} \; 1 - \sqrt{1 - \frac{2*0.97*20.6}{420}} = 0.0024$

$As = \rho * b * d = 0.0024 * 1000 * 223 = 535.2 \, mm^2.$

$A_{s,min} = \rho * b * h = 0.0018 * 1000 * 250 = 450 \, mm^2$

$A_s = 535.2 \, mm^2 > A_{s,min} = 450mm^2$

use Ø14@20 $then$

$n = \frac{A_s}{A_{sØ14}} = \frac{535.2}{153.93} = 3.47 = 4$, $s = \frac{1}{n} = 4 = 0.250 \, m.$

Step (S) is smallest of:

1- $3h = 3 * 300 = 900mm$
2- $450mm$
3- $s = 380\ \frac{280}{f_s}\ - 2.5C_c = 380\ \frac{280}{\frac{2}{3}420}\ - 2.5 * 20 = 330mm$

$$s \leq 300\ \frac{280}{f_s}\ = 300\ \frac{280}{\frac{2}{3}420}\ = 300mm - control$$

$s = 200\ mm\ < s_{max} = 300\ mm - OK$

Select s=300 mm

Temperature and shrinkage reinforcement.

A_S Temperature and shrinkage $= 0.0018 * 1000 * 250 = 450mm^2$
$use\ Ø10@15\ then$

$$n = \frac{A_S}{A_{sØ10}} = \frac{450}{78.5} = 5.7 = 6 \qquad s = \frac{1}{n} = \frac{1}{6} = 0.16\ m$$

Take 150 mm

Step $(S - for$ Temperature and shrinkage reinforcement$)$ is the smallest of:

1. $5h = 5 * 250 = 1250\ mm$
2. $450mm\ - control$

$s = 150\ mm\ < s_{max} = 450\ mm - OK$

Select s=450 mm

4-9-4 Design of slab L1 (landing):

$$w_R = q_u + support\ of\ flight = 16.05 + 23.32 = 51.37\ KN/m$$

The reaction at each end

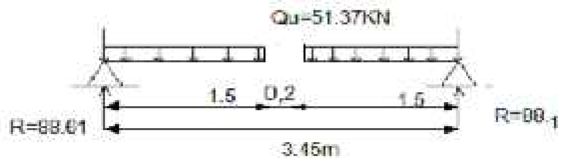


Figure 4-19: loads of landing

$$R = \frac{51.37 * 3.45}{2} = 88.61KN$$

Check for shear strength:

Assume bar diameter Ø14 for main reinforcement.

$$d = h - 20 - \frac{d_b}{2} = 25 - 20 - \frac{14}{2} = 223mm$$

Take the maximum shear as the support reaction $V_u = 88.61 - 51.37 * .323 = 72.07KN$

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

$$= \frac{\sqrt{24}}{6} * 1000 * 223 * 10^{-3} = 182.1 \, KN.$$

$$\phi * V_c = 0.75 * 182.1 = 136.56 \, KN/1m \; strip$$

$$\phi * V_c = 136.56KN > V_{u,max} = 72.07$$

..... ***The thickness of the slab is enough*** .

$use \; h = 25 \; cm$

Calculate the maximum bending moment and steel reinforcement:

$$M_u = \frac{51.37 * 3.45^2}{8} = 76.43 \; KN.m$$

$Mn = Mu/\phi = 76.43/0.9 = 84.92 \; KN.m$

$$d = depth - cover - diameter \; of \; stirrups - (diameter \; of \; bar/2$$

$$= 250 - 20 - \frac{14}{2} = 223 \; mm.$$

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

$R_n = \frac{M_n}{b*d^2} = \frac{84.92*10^6}{1000*223^2} = 1.71\ MPa$

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

$= \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2*1.71*20.6}{420}}\right) = 0.00426$

$As = \rho * b * d = 0.00426 * 1000 * 223 = 950\ mm^2.$

$A_{s,min} = \rho * b * h = 0.0018 * 1000 * 250 = 450\ mm^2$

$A_s = 950\ mm^2 > A_{s,min} = 450\ mm^2$

$use\ \emptyset 14\ then$

$n = \frac{A_s}{A_{s\emptyset 14}} = \frac{950}{153.93} = 6.2 = 7$, $s = \frac{1}{n} = 0.16$

Step (S) is smallest of:

1- $3h = 3 * 300 = 900mm$

2- $450mm$

3- $s = 380\left(\frac{280}{f_s}\right) - 2.5C_c = 380\left(\frac{280}{\frac{2}{3}420}\right) - 2.5 * 20 = 330mm$

$s \le 300\left(\frac{280}{f_s}\right) = 300\left(\frac{280}{\frac{2}{3}420}\right) = 300mm - control$

$s = 150\ mm < s_{max} = 300\ mm - OK$

- Temperature and shrinkage reinforcement.

A_S Temperature and shrinkage $= 0.0018 * 1000 * 250 = 450mm^2$

$n = \frac{A_s}{A_{s\emptyset 14}} = \frac{450}{153.93} = 2.9$, $s = \frac{1}{n} = \frac{1}{3} = 0.333\ m = .300$

Step $(S - for$ Temperature and shrinkage reinforcement$)$ is the smallest of:

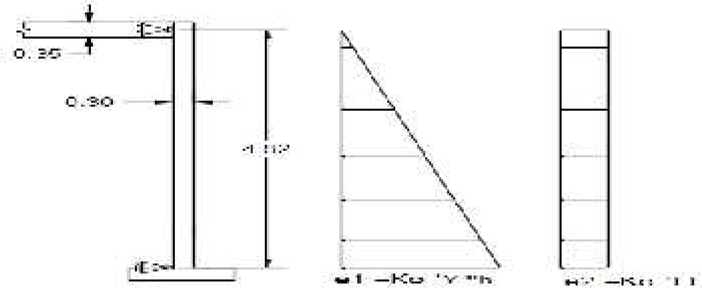
1- $5h = 5 * 250 = 1250\ mm$

2- $450mm\ - control$

$$s = 300\ mm < s_{max} = 450\ mm - OK$$

Select s=450 mm

4.10 Design of basement wall :-



4.10.1 Load Calculation:-

$f_c' = 24\ MPa$

$f_y = 420\ MPa$

$\gamma = 18\ KN/m^3$ Figure 4-20: Basement wall

$\emptyset = 30°$

$LL = 4\ KN/m^2$

$Thickness = h = 20\ cm,\ \ cover = 4\ cm$

The design will be for 1m width

- Analysis:
- Loads

Neglect the axial load, since its low value.

$e_1 = K_o * \gamma * h$

$e_2 = K_o * LL$

$K_o = 1 - \sin \emptyset$

So,

$K_o = 1 - \sin 30 = 1 - 0.5 = 0.5$

$e_o = 0.5 * 18 * 2.9$

$$E_o = 26.415 * \frac{2.935}{2} = 38.76\ KN/m^2$$

$e_L = 0.5 * 4 = 2\ KN/m^2$

$E_L = 2 * 2.935 = 5.87\ KN/m^2$

Support reactions:

$B_X = 21.025\ KN$

$A_X = 38.625\ KN$

$V = 0 \quad at \quad y = ?$

$21.025 - P\ y\ * \frac{y}{2} - 2 * y = 0$

$\frac{P_y}{y} = \frac{26.415}{2.935} = 9$

$21.025 - 9 * y * \frac{y}{2} - 2 * y = 0$

$4.5y^2 + 2y - 21.025 = 0$

$y = 2\ m$

$$M_{u,max} = 21.025 * 2 - 9 * 2 * \frac{2}{3} * \frac{2}{2} - 2 * 2 * \frac{2}{2} = 26.05\ KN.m$$

Factored internal forces

$V_u = 1.6 * V_{max} = 1.6 * 38.625 = 61.8\ KN$

$M_u = 1.6 * M_{max} = 1.6 * 26.05 = 41.68\ KN$

- Design

Design of shear

$d = 200 - 40 - 8 = 152\ mm$

$V_u = 61.8\ KN$

$$\phi V_c = 0.75 * \frac{\sqrt{f_c'}}{6} b_w d = \phi V_c = 0.75 * \frac{\sqrt{24}}{6} * 1000 * 152 = 93\ KN > V_u = 61.8\ KN$$

The thickness of Wall is Adequate Enough

Design of flexure

Vertical reinforcement of Tension face

$$M_u = 41.68\ KN.m$$

$$Mn = Mu/\phi = 41.68/0.9 = 46.31\ KN.m$$

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

$R_n = \frac{Mn}{b*d^2} = \frac{46.31*10^6}{1000*(152)^2} = 2.0\ MPa$

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

$= \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2*2*20.6}{420}}\right) = 0.005$

$A_{s,req} = \rho bd = 0.005 * 1000 * 152 = 760\ mm^2$

$A_{s,min} = 0.0012 * 1000 * 200 = 240\ mm$

$$A_{s,req} = 760mm^2 > A_{s,min} = 240\ mm^2 \ldots.\ OK$$

$$\therefore A_{s,req} = 760\ mm^2$$

$$Select\ \mathbf{7Ø12}\ with\ A_{s,pro} = 791.68mm^2 > A_{s,req} = 760mm^2 \ldots.OK$$

Vertical reinforcement of Compression face

$A_{s,min}\ for\ flexture = 0.25 * \frac{\sqrt{fc'}}{fy} * bw * d = 0.25 * \frac{\sqrt{24}}{420} * 1000 * 152 = 443mm^2/m$

$A_{s,min}\ for\ flexture = \frac{1.4}{fy} * bw * d = \frac{1.4}{420} * 1000 * 152 = 506.67\ mm^2/m$

$$Select\ \mathbf{5Ø12}\ with\ A_{s,pro} = 565.5\ mm^2 > A_{s,min} = 506.67\ mm^2/m$$

For inside wall Ø12@15 cm =7.91cm^2> 7.60 cm^2

For outside wall Ø12@20 cm =5.65cm^2> 5.1 cm^2

Horizontal Reinforcement due to Cracking:

$Asreq\,h = 0.002 * b * h = 0.002 * 100 * 20 = 4\ cm^2/m$

For one side $As = 2\ cm^2/m$

Select for one side horizontal reinforcement $\emptyset 10@20cm = 3.93\ cm^2 > 2\ cm^2$

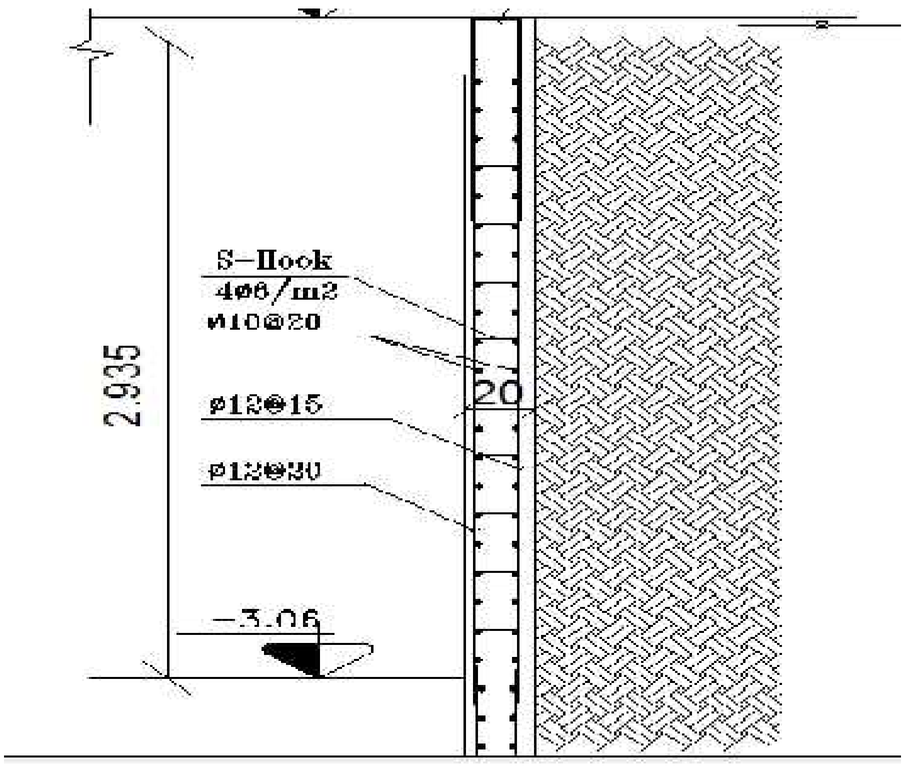


Figure 4-21: reinforcement of Basement wall

4.11 Design of Isolated Footing (F5 C50).

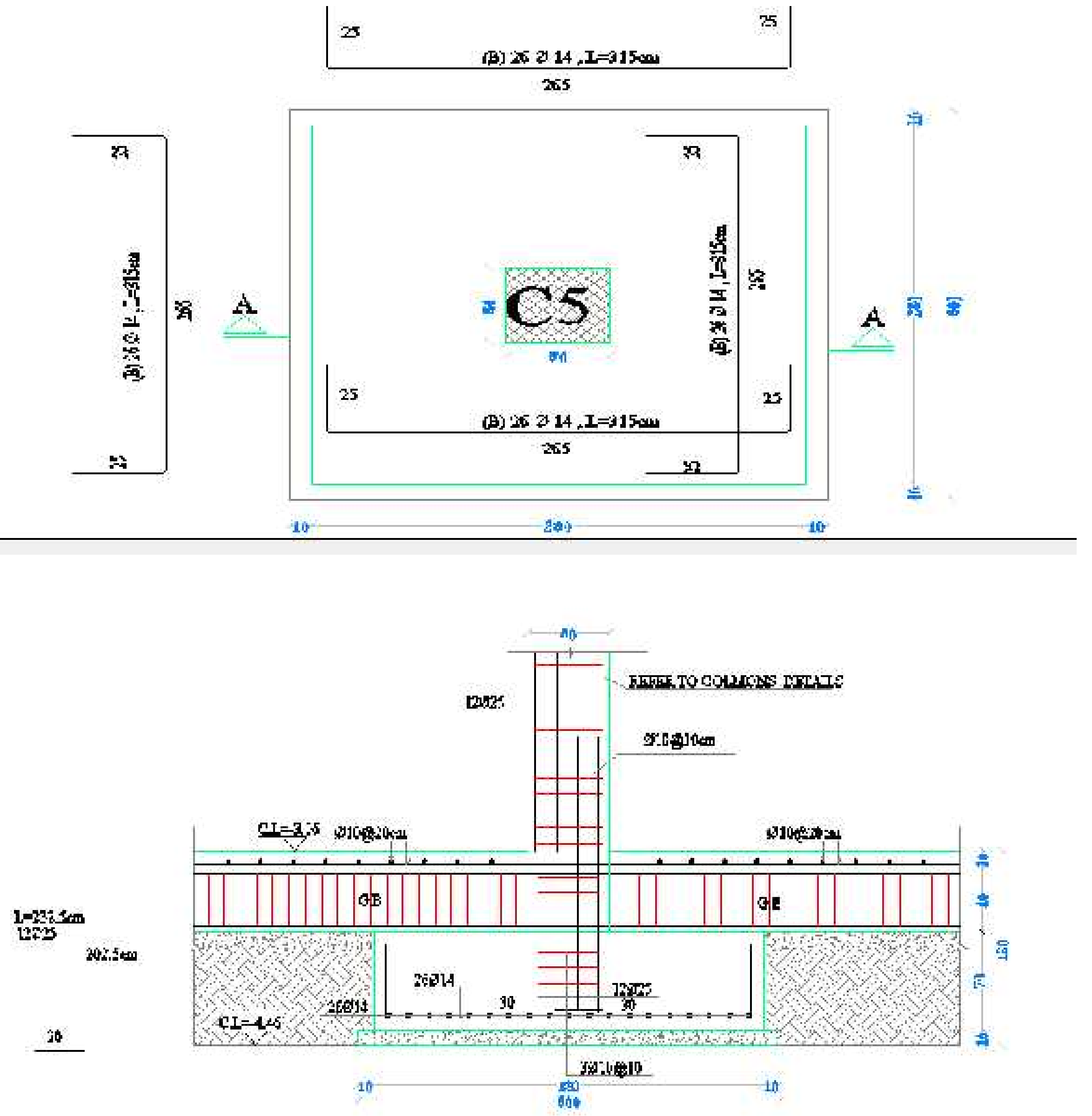


Fig. (4-22) : Footing geometry

From column group5:-

DL= 1823.96 KN

LL= 818.37KN

Factored load = 3498.14kN .

Soil weight = 18 kN/m3.

Allowable soil pressure = 400kN/m2.

Fc' = 24 Mpa

Fy = 420 Mpa

Cover = 7.5 cm

4.11.1Determine the net soil pressure:

use steel bar □14

Assume h = 70 cm ………d = 700-75-14 = 611 mm

Weight of footing= 0.7*25= 17.5 KN/m^2

Weight of soil= 1*18 = 18 KN/m^2

Total surcharge load foundation:

W= 17.5+18 = 35.5KN/m^2

qall.net = 400 –35.5= 364.5 KN/m^2

4.11.2 : Design of the footing area:

A= Pn/(qall. net) = (2642.33)/(364.5) = 7.25m^2

A= b*l

Take b= 2.80 m

l= 7.25/2.80= 2.6, take l= 2.80m

qu= 3498/(2.80 * 2.80)= 446.2 KN/m

4.11.3 Check for one way shear:

For X- direction:

Vu = ((2.80– 0.50)*0.5 – 0.611)× 446.2×2.80

Vu = 673.4 KN

For Y- direction:

Vu = ((L – a)*0.5 – d)× qu×b

Vu = ((2.80– 0.5)*0.5 – 0.611)× 446.2×2.80

Vu = 673.4 KN

ϕVc,x= ϕ (√(fc ′) *bw*d) / 6

= 0.75 * √24 * 2800*611*10^{-3} / 6

= 1047.6 KN >Vux = 673.4 KN => OK

ϕVc,y= ϕ (√(fc ′) *bw*d) / 6

= 0.75 * √24 * 2800*611*10^-3/ 6

= 1047.6 Kn>Vuy = 673.4 KN => OK

4.11.4 Check for two way shear:

Vu,x = qu*(b*l - (a+d) (c + d))

=446.2 (2.80*2.80 - (0.5+0.611) (0.5 + 0.611))

= 2506.7 KN.

s = 40 for interior column

β= 50/(50) = 1.0

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

bo= 2* (a+d+c+d)

= 2 *(0.50+0.611*2+0.5)

= 4.444 m

Vc the smallest of:

Vc $=1/6*(1+2/B)\sqrt{fc'}*b*d$ ..where $1/6*(1+2/B)=1/6*(1+2/1.0)=0.50$

Vc $= 1/12((\alpha sd)/b+2)\sqrt{fc'}*b*d$ ..where

$$1/12((\alpha sd)/b+2)=1/12((40*0.611)/4.444+2)=0.625$$

$V_c=1/3*\sqrt{fc'}*b*d$ where $1/3=0.333$ … … … … control

Take $V_c=1/3*\sqrt{fc'}*b*d= 1/3*\sqrt{24}*4444*611*[10]^{(-3)}=4434.04$ KN

$\emptyset V_c=0.75*7057.8=3325.5$ KN

$\emptyset V_c=3325.5>V_u=2506.7$KN … … … ok

4.11.5Design for bending moment:

4.11.5.1 Design flexure for long And Short direction:

use steel bar □14

b =2.8m , h =700mm , d= 611mm

$M_u=446.2*2.80*(0.5)^2/2=156.17$ KN. m

$m = f_y/(0.85\ fc') = 420/(0.85*24) = 20.59.$

$R_n = M_u/(\emptyset\ b*d^2) = (156.17*[10]^6)/(0.9*2800*[(611)]^2) = 0.17$MPa.

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

$= 1/20.59(1- \sqrt{1-(2*20.59*0.17)/420})= 0.00041$

$A_s = \rho * b *d = 0.00041 * 2800 * 611 = 701.428mm^2.$

$$As_{min} = 0.0018*b*h= 0.0018*2800*700=3528mm2$$

$As_{min} = 3528\ mm^2 > As_{req} = 701.428\ mm^2.$

∴ $As = As_{min} = 3528\ mm^2.$

$n= As_{req}/(A_{bar}\emptyset 14) = (3528)/153.94 = 25.2$

∴ Use 26Φ14

$S=(2800-75*2-26*14)/25=91.44$mm

Step S is the smallest of

3h = 3*700=2100mm

450………control

S = 91.44 $< S_max = $ 450 $\ldots\ldots\ldots.ok$

CITY CENTER (B) DESIGN

4.12 Design of Flat slab :

The design done by using SAFE program.

4.12.1 Load calculation:

Assume slab thickness 30cm.

No.	Material	Thickness cm	Quality Density KN/m^3	Calculation
١	Slab	30	25	0.30×25 = 7.5
٢	Sand	7	17	0.07×17 = 1.19
٣	Mortar	2	22	0.02×22 =0.44
٤	Tile	3	23	0.03×23 =0.69
٥	Plaster	2	22	0.02×22 =0.44
6	Partitions			2.38

	∑ =	12.64	KN/m^2

Table (4 – 6) Calculation of the total dead load for flat slab.

4.12.2 Check for punching shear:-

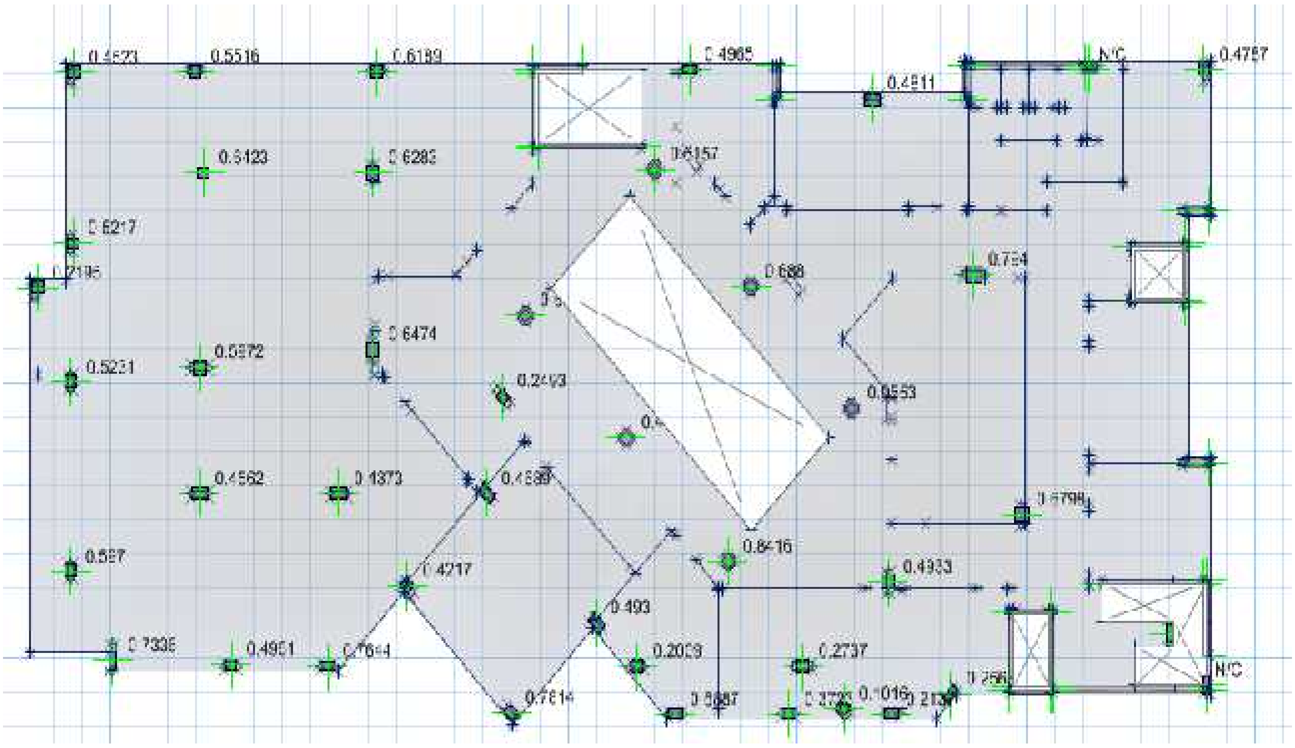


Figure (4-23): Punching Shear Capacity Ratios / Shear Reinforcement for flat slab

As shown all ratios less than 1 , so we don't have punching reinforcement .

4.12.3 Design for bending moment:

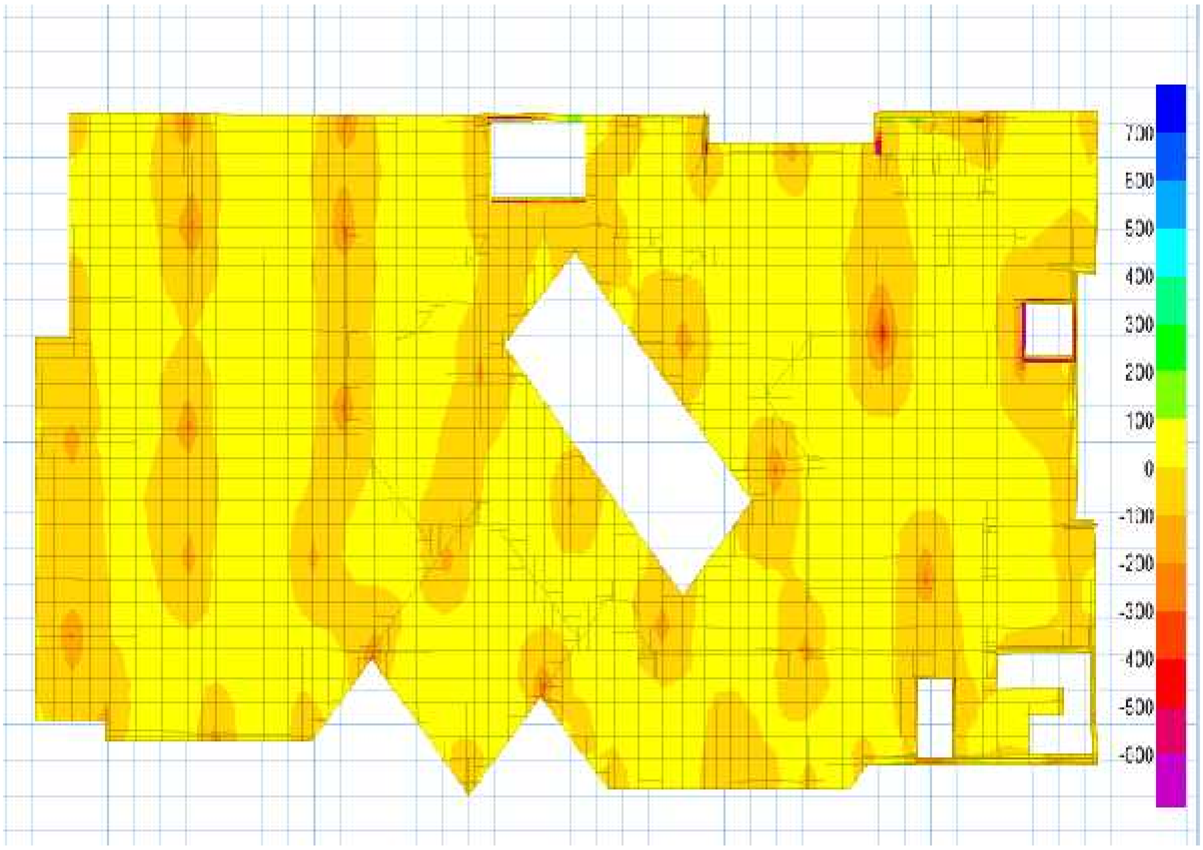


Figure (4-24): moment distribution in x-direction

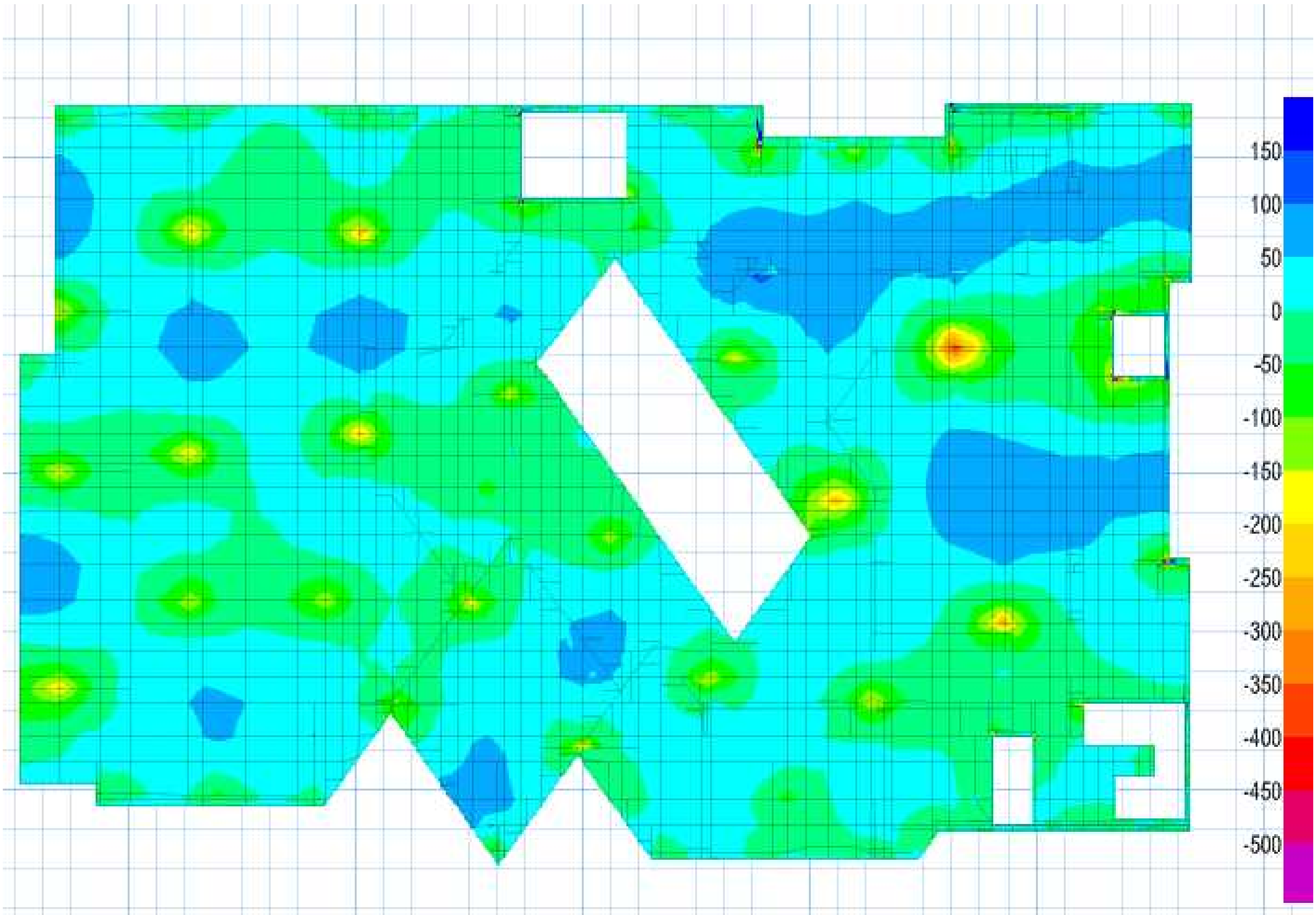


Figure (4-25): moment distribution in y-direction

The design of flat slab done by using Finite Element method.

Selected Ø14/15cm in both direction for top reinforcement

Selected Ø14/15cm in both direction for bottom reinforcement

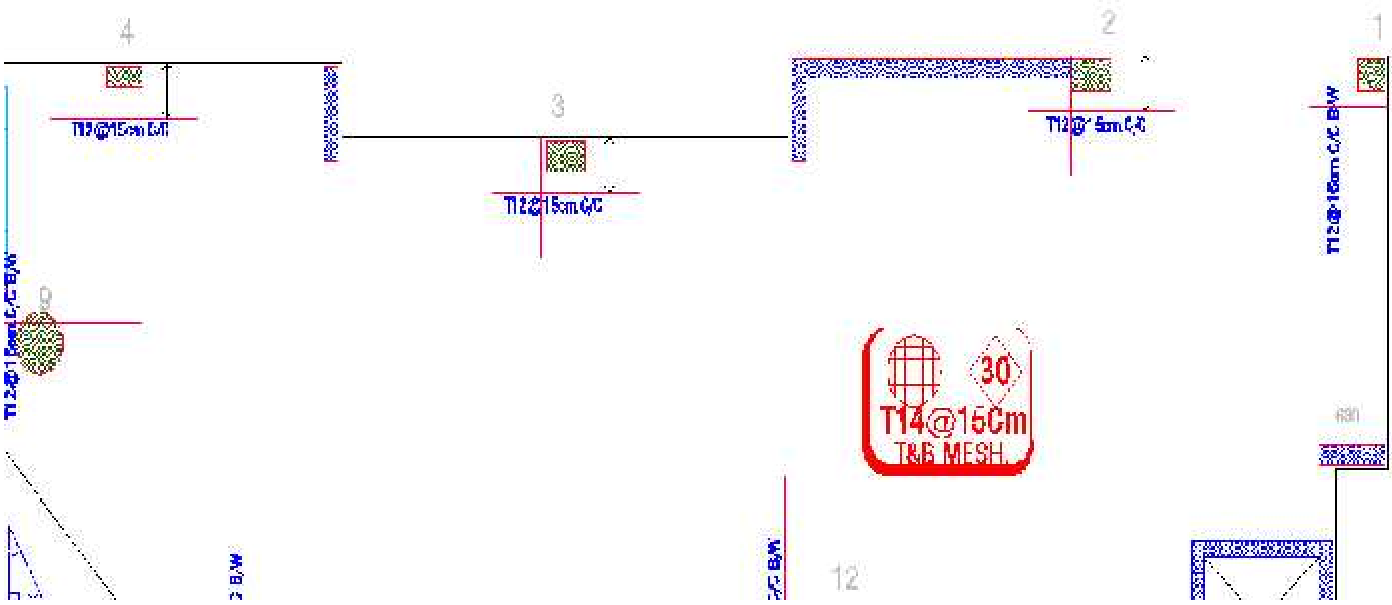


Figure (4-26): Reinforcement for flat**.**

4.13 Design of column (C44):-

4.13.1 Load calculation:

DL= 845 KN LL= 200 KN

P_u = 1333 KN $P_{n,req}$=1333/0.65 =2050.76 KN

Assume rectangular section with ρ= 1.67% > 1%

$P_n = 0.8\times Ag\times (0.85\times fc'+\rho_g\times(fy - 0.85 fc'))$

$2050.76 = 0.8\times Ag\times(0.85\times 28 + 0.0167 \times (420 - 0.85\times 28))$

$A_g = 842.78\ cm^2$

Use 25×60 cm with Ag = 1500 cm² >$A_{g,req}$ = 842.78 cm^2

4.13.2 Check slenderness effect:

Lu: Actual unsupported (unbraced) length.

K: effective length factor (K= 1 for braced frame).

R: radius of gyration =$\sqrt{(I/A)} = 0.3\,h$

Lu = 4.3 m

M1/M2 =1

In 25 cm -Direction

$Klu/r < 34 - 12\ (M1/M2) < 40$

$(1 \times 4.3)/(0.3 \times 0.25)$ =57.33>22 =>long

In 60cm -Direction

$Klu/r < 34 - 12\ (M1/M2)$

$(1 \times 4.3)/(0.3 \times 0.6)$ = 23.89> 22 =>Long

Long in x direction

Long in y direction

4.13.3 Calculation for reinforcement:

In 25 cm -Direction

E_c= 4700 × √28 =24870.1MPa

$\&_{dns}$= $(1.2\,D\,(sustained))/Pu$ = (1.2 ∗ 845)/1333 = 0.76

I_g= $b \times (h〗^3/12$= 60 × (25〗^3/12 = 0.00781 m^4

EI =$(0.4 \times Ec \times I)/(1+\ \beta dns)$ = (0.4 × 24870.1 × 0.00781)/(1 + 0.76) = 44.14 MN.m^2

P_c = $(\pi$^2 × $EI)$/ 〖(Klu)〗^2

= $(\pi$^2 × 44.1)/ 〖(1.0 × 4.3)〗^2

= 23.6 MN

Cm = 0.6 + 0.4×($M$1/$M$2) = 1

$\&_{ns}$= $Cm/(1-(Pu)/(0.75\,Pc))$ = 1/(1 − (1333)/(0.75 × 23.6 × 1000)) = 1.1< 1.4

e_{min} = 15 + 0.03 h = 15+ 0.03 × 250 = 22.5mm

e = e_{min} × $\&_{ns}$=22.5× 1.15 =25.875 mm

$e/h =$ 25.875/250 = 0.099< 0.1…….(e = 0.082h < 0.1h)

In 60 cm -Direction

E_c= 4700 × √28 =24870.1MPa

$\&_{dns}$= $(1.2\,D\,(sustained))/Pu$ = (1.2 ∗ 845)/1333 = 0.76

I_g= $b \times (h〗^3/12$= 25 × (60〗^3/12 = 0.0045 m^4

EI =$(0.4 \times Ec \times I)/(1+\ \beta dns)$ = (0.4 × 24870.1 × 0.0045)/(1 + 0.76) = 25.44 MN.m^2

P_c = $(\pi$^2 × $EI)$/ 〖(Klu)〗^2

= $(\pi$^2 × 25.44)/ 〖(1.0 × 4.3)〗^2

= 13.57 MN

Cm = 0.6 + 0.4×($M1/M2$) = 1

$\&_{ns}$= $Cm/(1-(Pu)/(0.75\,Pc))$ = $1/(1-(1333)/(0.75\times13.57\times1000))$ = 1.15< 1.4

e_{min} = 15 + 0.03 h = 15+ 0.03 × 600 = 33 mm

e = e_{min} × $\&_{ns}$ = 33× 1.15 =37.95 mm

$e/h=$ 37.95/600 = 0.063< 0.1…….(e = 0.082h < 0.1h)

→**Here we can solve this column as short tied column**

P_n = 0.8× Ag× (0.85× fc' + $_g$×(fy - 0.85 fc'))

P_n = 0.8× 250×600× (0.85× 28 + 0.0167 × (420 - 0.85× 28))

=3649.9 KN >$P_{n,req}$=2050.76 KN …….OK

4.13.4 Design of the tie reinforcement :

S ≤16 db (longitudinal bar diameter)

S≤ 48dt (tie bar diameter).

S ≤ Least dimension.

spacing ≤16×d_b=16×2.0=32 cm

spacing ≤48×dt=48×1.0 = 48 cm

spacing ≤least.dim=25 cm…. control

20cm ≤ 25cm…..ok

Use¤10@20 cm

For column 5(c44)

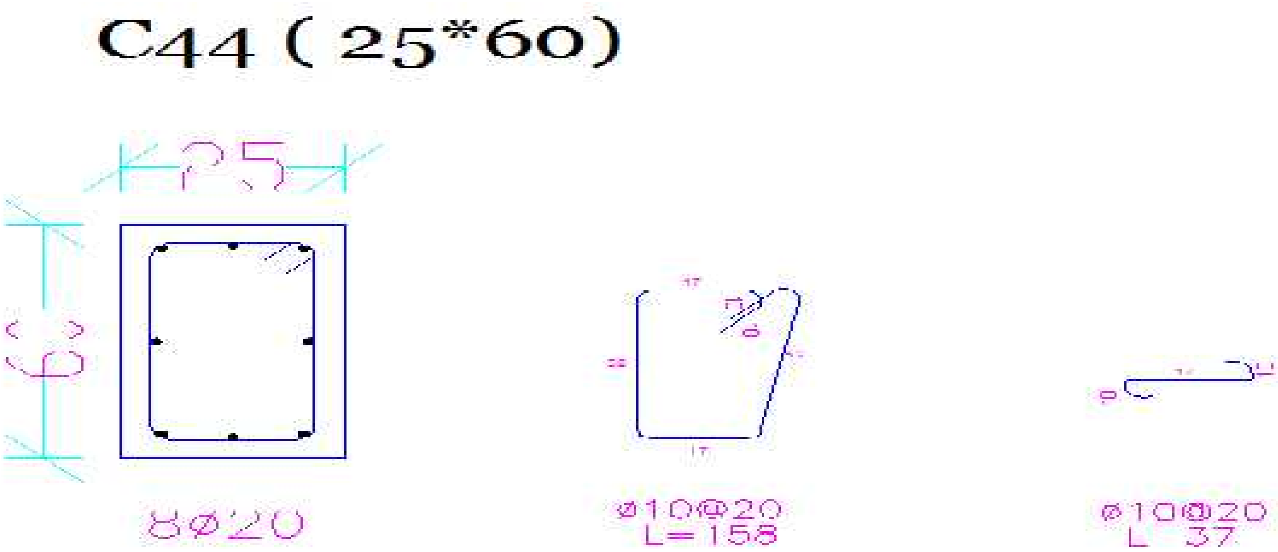
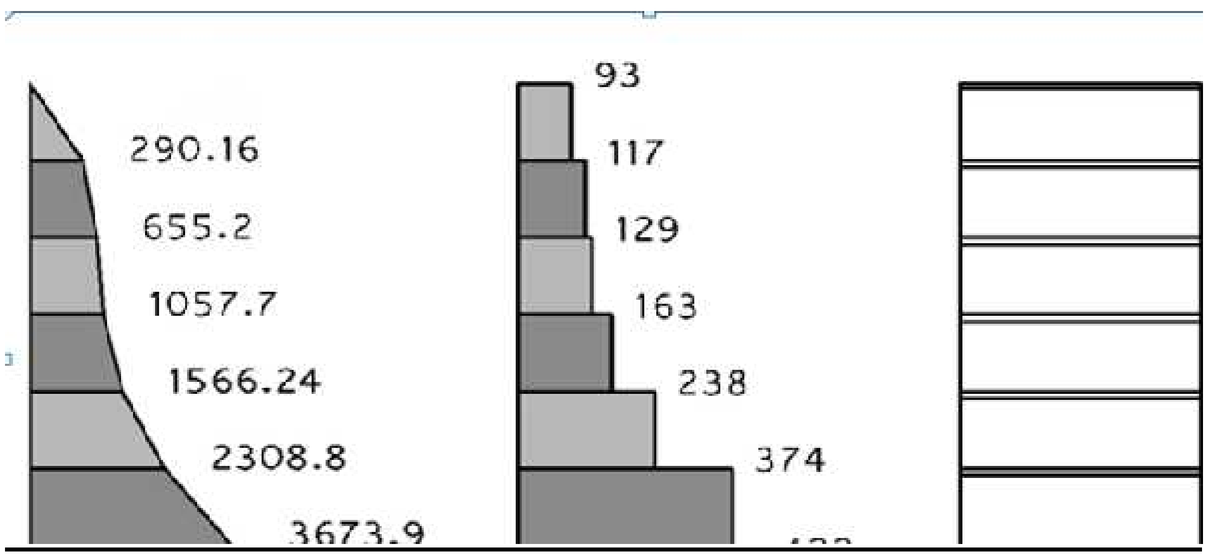


Fig. 4–27 :Reinforcement of column 44

4.14:-Design of Shear wall.(sw15)



(Figure 4-28:Moment and shear diagram)

Fc = 28MPa

Fy = 420 MPa

t =25cm .shear wall thickness

Lw = 6.20m ,shear wall width

Hw_1 for one wall = 3.00 m

$Hw_2$for one wall = 5.1 m story height

$Hw_3$for one wall = 3.3 m story height

4.14.1: Design of shear

$$\sum Fx = Vu = 374KN$$

4.14.2: Design of the Horizontal reinforcement:

The critical Section is the smaller of:

$$\frac{lw}{2} = \frac{6.20}{2} = 3.10\,m$$

$$\frac{hw}{2} = \frac{21.30}{2} = 10.65\,m$$

$$storyheigh \quad (Hw) = 3.30\,m \ldots\ldots \quad control$$

$$d = 0.8 \times lw = 0.8 \times 6.20 = 4.96\,m$$

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

$$= 0.75 * 0.83 * \sqrt{28} * 250 * 4960 * 10^{-3} = 4084.5KN > V_u$$

V_c is the smallest of :

$$1 - V_c = \frac{1}{6} \sqrt{f_c'}hd = \frac{1}{6}\sqrt{28} * 250 * 4960 * 10^{-3} = 1093.58KN$$

$$2 - V_c = 0.27 \sqrt{f_c'}hd + \frac{N_u d}{4l_w} = 0.27\sqrt{28} * 250 * 4960 * 10^{-3} + 0 = 1771.6KN$$

$$3 - V_c = \left[0.05 \sqrt{f_c} + \frac{l_w \left(0.1 \sqrt{f_c'} + 0.2 \frac{N_u}{l_w h} \right)}{\frac{M_u}{V_u} - \frac{l_w}{2}} \right] hd$$

$$= \left[0.05\sqrt{28} + \frac{6.20 \left(0.1\sqrt{28} + 0 \right)}{6.70} \right] 250 * 4960 = 935.25KN \ldots \text{cont}$$

$$\frac{M_u}{V_u} - \frac{l_w}{2} = \frac{2308.8}{374} - \frac{6.20}{2} = 3.07$$

Vu = 374KN< $\frac{1}{2}$ *0.75*935.25 = 380.8 KN **No need reinforcement**

- Minimum shear reinforcementis required:

Take ρ = 0.0025

- Maximum spacing is the least of :

$\frac{Lw}{5} = \frac{6200}{5}$ = 1240mm

3*h = 3*250 = 750mm

450 mm ……. Control

Try ϕ 12 (As = 113.1 mm2) for two layers

$\rho = \frac{Avh}{h*S2} = \frac{2*113.1}{250*S2}$ = 0.0025

S2 = 455.1mm , ϕ 12@250 mm

→useϕ 12 @250mm in two layer

4.14.3: Design for Vertical reinforcement: -

$\frac{h_w}{L_w} = \frac{21.30}{6.20} = 3430$ mm

$\frac{Lw}{3} = \frac{6200}{3}$ = 2066.67mm

450 mm ……. Control

3*h = 3*250 = 750mm

A_{nv}=0.0025*S*h

Try Φ 12 (As = 113.1 mm2)

113.1*2=0.0025*S*250

S=452.4

Select Φ 12 @250mm In tow layer.

4.14.4: Design of bending moment (uniformly distribution flexural reinforcement) :

$$A_{st} = \frac{6200}{250} * 2 * 113.1 = 5609.76mm^2$$

$$w = \frac{A_{st}}{L_w h} \frac{f_y}{f_c'} = \frac{5609.7}{6200 * 250} \frac{420}{28} = 0.05$$

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

$$\frac{C}{l_w} = \frac{w + \alpha}{2w + 0.85\beta_1} = \frac{0.05 + 0}{2 * 0.05 + 0.85 * 0.85} = 0.06$$

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

$$= 0.9 \ 0.5 * 5609.7 * 420 * 6200(1 + 0)(1 - 0.0806) \ = 6043KN.m > Mu$$

Select Φ 12 @250mm for vertical reinforcement .

4.15 Design of the Mat Foudation reinforcement :-

Design done by using SAFE.

4.15.1 Load calculation:

Density of soil = 18KN/m^3
Allowable soil pressure = 400kN/m^2
Fc'= 28Mpa
Fy= 420 Mpa
Cover= 7.5 cm
Take the reaction of columns and walls from ETABS.

4.15.2 Determine the soil pressure:

Subgrade Modulus of soil = 120*400 = 48000KN/m^3

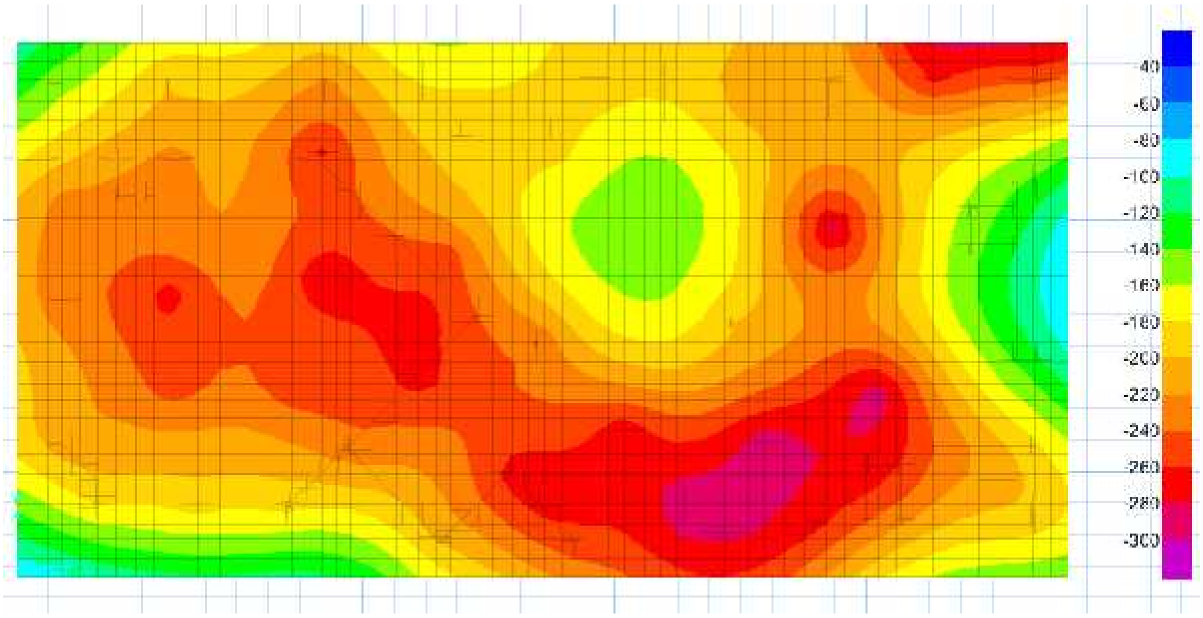


Figure (4-29): Soil pressure diagram

Max pressure =300 KN/m^2<400KN/m^2

4.15.3 Check for punching shear:-

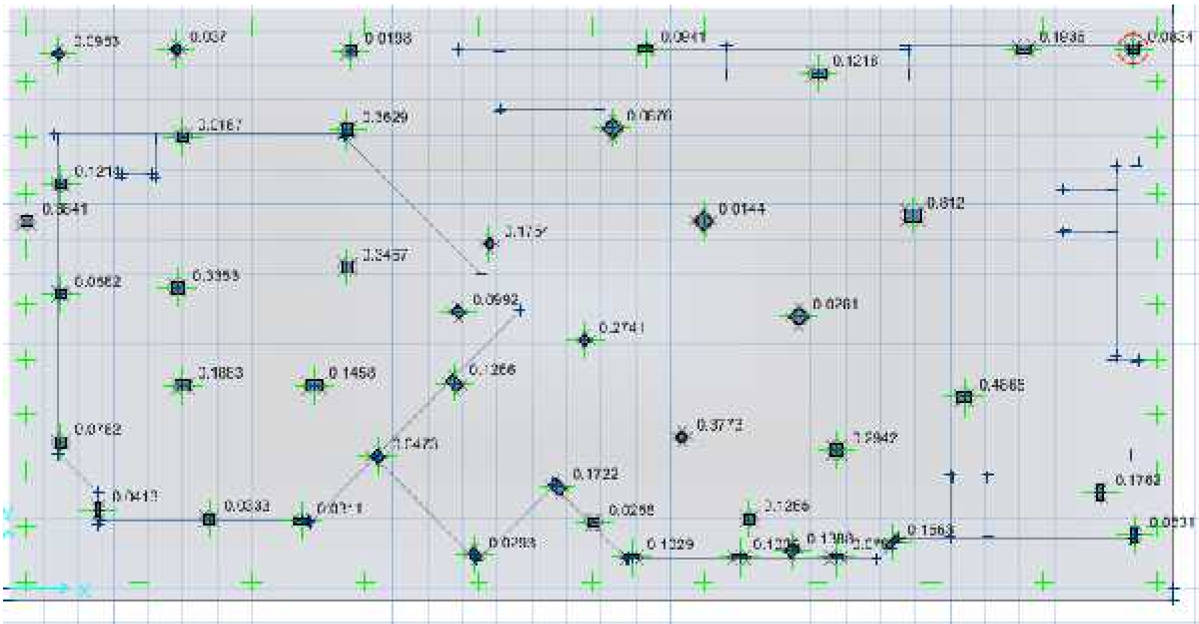


Figure (4-30): Punching Shear Capacity Ratios / Shear Reinforcement for flat

As shown all ratios less than 1 , so we don't have punching reinforcement .

4.15.4 Design for bending moment:

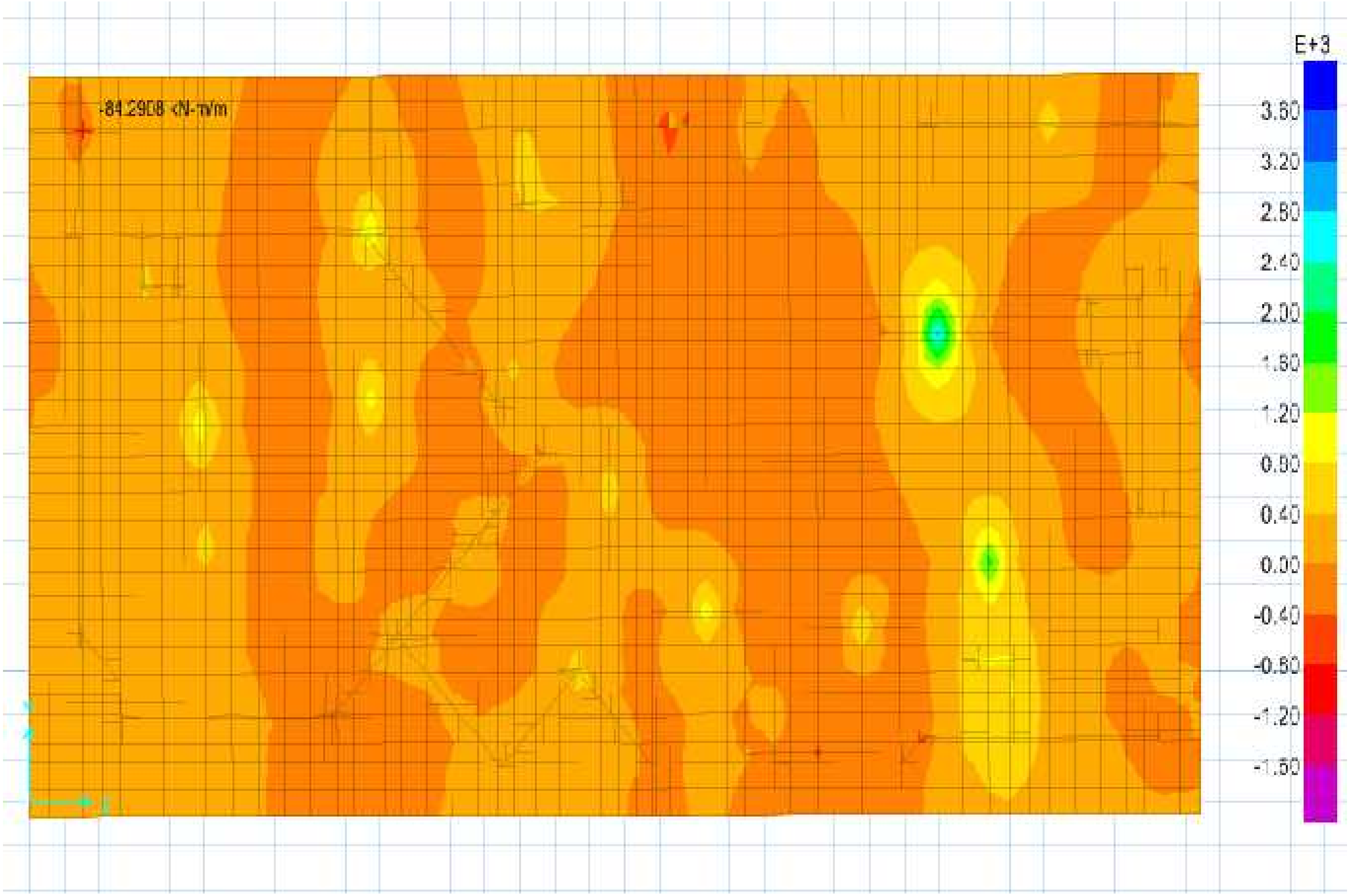


Figure (4-31): moment distribution in x-direction

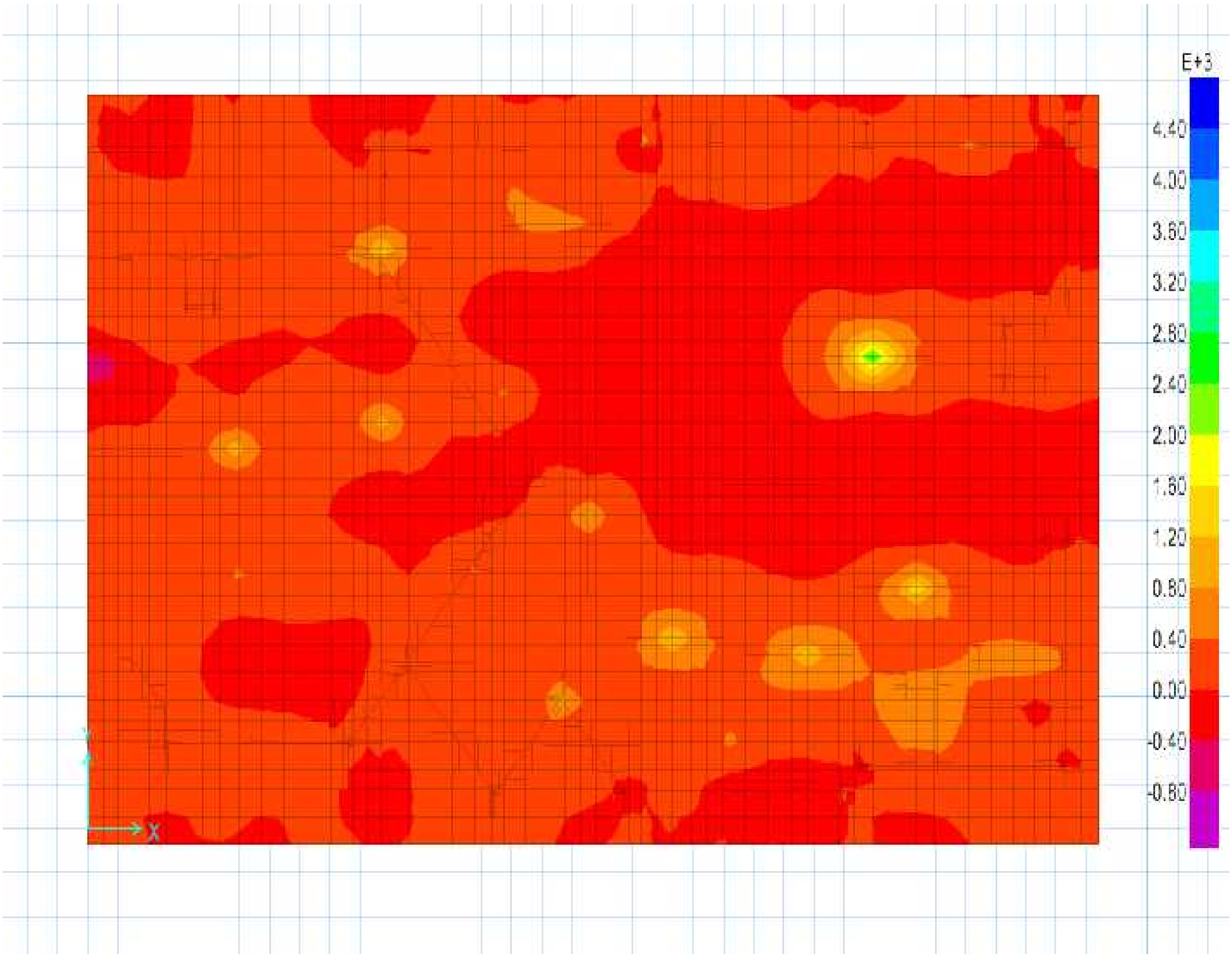


Figure (4-32): moment distribution in y-direction

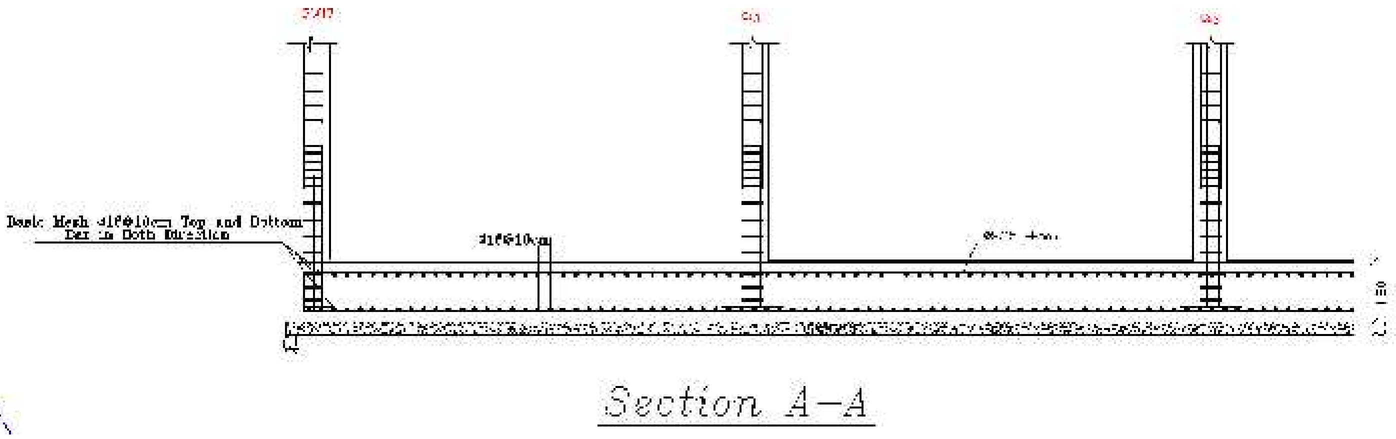


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Figure (4-33): reinforcement of mat foundation .

The design of mat foundation by using Finite Element method

Selected basic mesh 16/10cm for top reinforcement

Selected basic mesh 16/10cm for bottom reinforcement

- ## 4.16.4 :Design of weld:

The calculation of weld based on the following :

1) Fillet weld is used.
2) The plates are A36(fy=36 ksi,Fu=58 ksi)
3) The plat thickness is (t=0.5 in)
4) The electrodes having F_{Exx}=70 ksi
5) The shielded metal arc welding (SMAW) is used.

1st) Design of weld between the vertical member and the Gusset plate in the corners of the truss:

The section of the vertical member is angle (L3*3*3/8) , Ag=2.11 in^2 ,y=0.884.
The value of Max. compression in the vertical member is Vu=20.662Kips.

Max. weld size (a_{max})=$t-\frac{1}{16}=\frac{3}{8}-\frac{1}{16}=\frac{5}{16}\ in$

Min . Weld size(a_{min})=$\frac{3}{16}\ in$

Use weld size (a)=$\frac{1}{4}\ in$

(Figure 4- 40) weld between vertical member and gusset plate)

- Design strength of weld :

Ø×Rnw= Ø× te×0.6×FExx
Ø×Rnw=0.75 $\times(0.707\times 1/4)\times 0.6\times 70=5.57$ kips

- Design strength of base material :

Ø×Rn= Ø× (0.6×fy)×t=1.0×0.6×36×$\frac{3}{8}$= 8.1 kips >5.57 kips….ok

Or

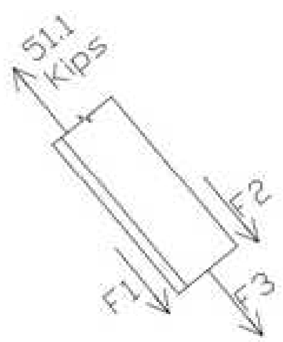
Ø×Rn= Ø× (0.6×fu)×t=0.75×0.6×58×$\frac{3}{8}$= 9.79kips >5.57 kips….ok

$f1$=5.57*3=16.71 kips
$f2$=20.662-16.71=3.952 kips

$$lw2=\frac{f2}{\text{Ø}\times Rnw}=\frac{3.952}{5.57}=0.71in\ \ldots\ldots use\ 1.0\ in$$

2nd) Design of weld between the diagonal member and the gusset plate:

- The section of the diagonal member is angel (L3*3*3/8)
- For the vertical member use the same size and dimension of weld for the previous vertical member.



(Figure 4- 41:weld between diagonal member and gusset plate)

The value if Max. Tension in the diagonal member is Tu = 51.1 kip.

Max. weld size (a_{max})=$t-\frac{1}{16}=\frac{3}{8}-\frac{1}{16}=\frac{5}{16}\ in$

Min = Weld size(a_{min})=$\frac{3}{16}\ in$

Use weld size (a)=$\frac{1}{4}\ in$

- Design strength of weld :

Ø×Rnw= Ø× te×0.6×FExx

$Ø\times Rnw = 0.75 \times 0.707 \times \frac{1}{4} \times 0.6 \times 70 = 5.57$ kips

- Design strength of base material :

$Ø\times Rn = Ø\times (0.6\times fy)\times t = 1.0\times 0.6\times 36\times \frac{3}{8} = 8.1$kip >5.57 kip….ok

Or

$Ø\times Rn = Ø\times (0.6\times fu)\times t = 0.75\times 0.6\times 58\times \frac{3}{8} = 9.79$kip >5.57 kip….ok

$$F3 = 3 * 5.57 = 16.71\ kips$$

$$\sum M\ at\ F1 = 0$$

$$= 16.71 * 1.5 + F2 * 3 - 51.1 * (3 - 0.884) = 0$$

F2=27.69 kips

F1=51.1-16.71-27.69=6.7 kips

$$lw1 = \frac{f1}{Ø \times \text{Rnw}} = \frac{6.7}{5.57} = 1.21in \ \ldots\ldots use\ 1.5\ in$$

$$lw2 = \frac{f2}{Ø \times \text{Rnw}} = \frac{27.69}{5.57} = 4.97in \ \ldots\ldots use\ 5\ in$$

Check for rupture

$$L = \frac{(5+1.5)}{2} = 3.25$$

$$U = 1 - \frac{x}{l} = 1 - \frac{0.884}{3.25} = 0.728$$

$$ØtPn = 0.75 * fu * Ae$$

$$ØtPn = 0.75 * 58 * 0.728 * 2.11 = 66.82\ kips > 51.1\ kips \ldots..ok$$

3rd) Design of weld between the bottom member and the gusset plate:

The section of the bottom member is angel (W6*12)

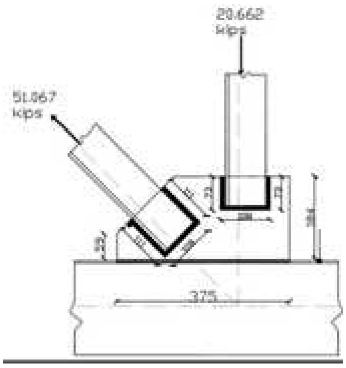
11 /2.54= 4.33 in

$$Ru = \sqrt{(Rv+Ry)^2 * (Rh+RX)^2}$$

$$Rv = \frac{Py}{L} = 0$$

$$Rh = \frac{Px}{L} = \frac{20.662}{14.76 * 2} = 0.7\ kip/in$$

$$Ip = 2 * \frac{14.76^2}{12} = 535.93\ \text{in}^3$$



(Figure 4- 42weld between gust plate and bottom member)

$$Rx = \frac{M * Y}{Ip} = 0 \ldots . y = 0$$

$$Ry = \frac{M * x}{Ip} = \frac{20.662 \quad * (\frac{4.33}{2}}{535.93} = 0.1$$

$$Ru = \sqrt{(0 + 0.1)^2 + (0.7 + 0)^2} = 0.71\ kip/in$$

$$Ø * Rnw = Ru$$

0.75*(0.707a)*0.6*70=0.71……a=0.032 in

$$\boldsymbol{Take\ a = \frac{2}{16} in}$$