

### **Chapter 4**

## **Structural Analysis & Design**

**4.1\* Introduction.**

**4.2\* Design method and requirements.**

**4.3\* Slabs thicknesses calculations.**

**4.4\* Design of topping slab.**

**4.5\* Design of one way ribbed slab (R2).**

**4.6\* Design of Beam (B3).**

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**4.9\* Design of long column (C6).**

**4.10\* Design of Isolated Footing (F22).**

**4.11\* Design of Stairs.**

**4.12\* Design of Shear wall.**

**4.13\* Design of footing and shear design (s1).**

### 4.1\* Introduction:

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

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

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

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

Structural concrete can be classified into:

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

### 4.2\* Design method and requirements:

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

#### ✓ **Strength design method:**

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

This load is 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  $\geq$  strength required to carry factored loads.**

**NOTE:**

The statically calculations and the key plans depends on the architectural plans. The design of the structural elements is based on the followings:

✓ Code : ACI 2008 (American Concrete Institute)  
          UBC (Universal Building Code)

✓ Materials :  
          Concrete: B300....  $f_c' = 30 \text{ N/mm}^2 \text{ (MPa)}$  For cubical specimens  
          But for cylindrical specimens (  $f_c' = 35 * 0.8 = 28 \text{ MPa}$  ).

Reinforcement steel: The specified yield strength of the reinforcement is  $FY = 420 \text{ N/mm}^2 \text{ (MPa)}$ .

✓ **Factored loads:**

The factored loads for members in this project are determined according to **ACI-code-318-08(9.2.1)** as:

$$W_u = 1.2 D_L + 1.6 L_L$$

### **4.3 \*Slabs Thicknesses calculations:-**

According to ACI-Code-318-08 table 9.5(a), the minimum thickness of non- prestressed beams or one way ribbed slabs unless deflections are computed are calculated as follow:

$L_n/18.5$       For one-end continuous.

$L_n/21$         For both ends continuous.

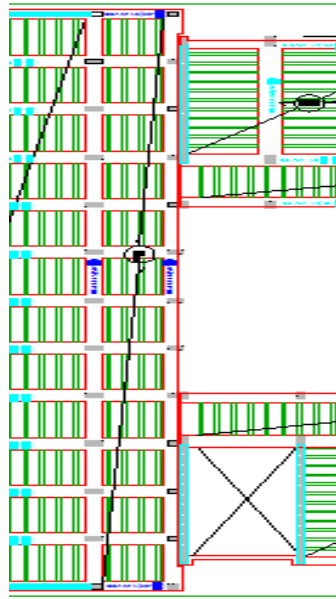


Fig 4.1: One way Ribbed slab (R2).

✓ **Statically system for (R 2) :**

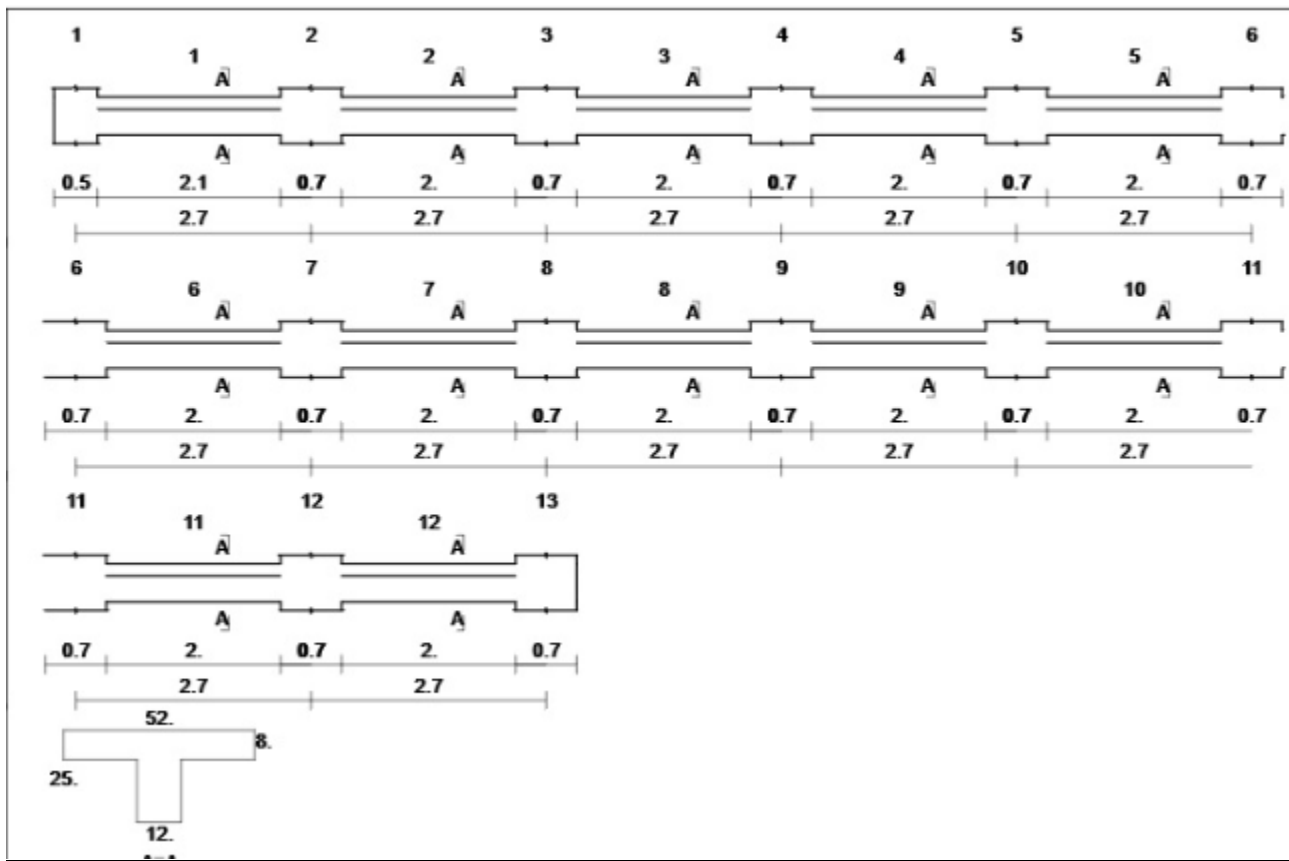
The minimum required thickness is:

$$\frac{L}{18.5} = \frac{2700}{18.5} = 146 \text{ mm} \quad \text{for end continuous supported.}$$

Select  $h_{\min} = 250\text{mm}$

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Geometry    Units: meter, cm



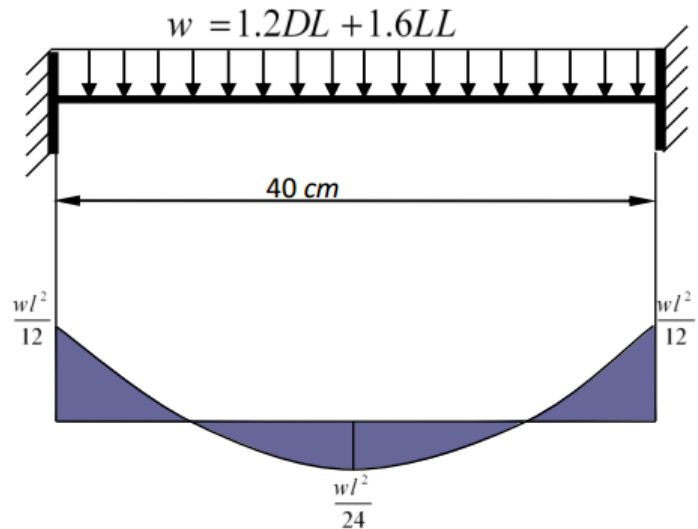
### 4.4 Design of topping slab:

#### ✓ Statically system for topping slab:

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

✓ **Load calculations:**

**Dead load calculations:**



Dead load from:	$\delta \times \gamma \times 1$	KN/m
Tiles	$0.03 \times 23 \times 1$	0.69
Mortar	$0.02 \times 22 \times 1$	0.66
Coarse sand	$0.07 \times 16 \times 1$	1.12
Topping	$0.08 \times 25 \times 1$	2
Interior partitions	$1 \times 1$	1
	$\Sigma$	5.25

**Table (4-1) calculations of dead load of topping slab**

**Live load:-**

$$L_L = 4.5 \text{ KN/m}^2 \longrightarrow L_L = 4.5 \text{ KN/m}^2 \times 1 \text{ m} = 4.5 \text{ KN/m}.$$

**Factored load:**

$$W_U = 1.2 \times 5.25 + 1.6 \times 4.5 = 13.5 \text{ KN/m}.$$

Check the strength condition for plain concrete,  $\phi M_n \geq M_u$ , where  $\phi = 0.55$ .

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$$M_n = 0.42 \lambda \sqrt{f'_c} S_m \quad (\text{ACI 22.5.1, equation 22-2}).$$

$$S_m = \frac{b \cdot h^2}{6} = \frac{1000 \times 80^2}{6} = 1066666.67 \text{ mm}^2.$$

$$\phi M_n = 0.55 \times 0.42 \times \sqrt{24} \times 1066666.67 \times 10^{-6} = 1.207 \text{ KN.m.}$$

$$M_u = \frac{W_u L^2}{12} = 0.18 \text{ KN.m} \quad (\text{negative moment}).$$

$$M_u = \frac{W_u L^2}{24} = 0.09 \text{ KN.m} \quad (\text{positive moment}).$$

$$\phi M_n >> M_u = 0.18 \text{ KN.m}$$

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

$$\rho_{\text{shrinkage}} = 0.0018 \quad \text{ACI 7.12.2.1}$$

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

Spacing(s) is the smallest of:

$$- 3h = 3 \times 80 = 240 \text{ mm. controls (ACI 10.5.4).}$$

$$- 450 \text{ mm.}$$

$$- S = 380 \left( \frac{280}{f_s} \right) - 2.5 C_c = 380 \left( \frac{280}{\frac{2}{3} \times 420} \right) - 2.5 \cdot 20 = 330 \text{ mm} \quad \text{but}$$

$$S \leq 300 \left( \frac{280}{f_s} \right) = 300 \left( \frac{280}{\frac{2}{3} \times 420} \right) = 300 \text{ mm} \quad \text{ACI 10.6.4}$$

Use  $\phi 8 @ 200 \text{ mm}$  in both directions,  $S = 200 \text{ mm} < S_{\max} = 240 \text{ mm}$  ... OK.

### 4.5 Design of one way Rib slab(R 2):

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

$$b_w \geq 10 \text{ cm} \dots \dots \dots$$

$$\dots \dots \dots \text{ACI(8.13.2)}$$

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Select  $b_w = 12\text{cm}$

$$h \leq 3.5 \cdot b_w \dots\dots\dots \text{ACI (8.13.2)}$$

Select  $h = 25\text{cm} < 3.5 \cdot 12 = 42\text{cm}$

$$t_f \geq L_n / 12 \geq 50\text{mm} \dots\dots\dots \text{ACI(8.13.6.1)} \quad \text{Select } t_f = 8\text{cm}$$

**The effective flange width ( $b_e$ ), according to ACI 8.12.2 is the smallest of:**

$$-b_e \leq \frac{L}{4} = \frac{2200}{4} = 550 \text{ mm} \quad L, \text{ is the clear span of the rib.}$$

$$-b_e \leq b_w + 16h_f = 120 + 16 \times 80 = 1400 \text{ mm.}$$

$$-b_e \leq \text{center to center spacing between adjacent beams} = 520 \text{ mm.} \quad \underline{\text{Controls}}$$

### ✓ Statically system and Dimensions.

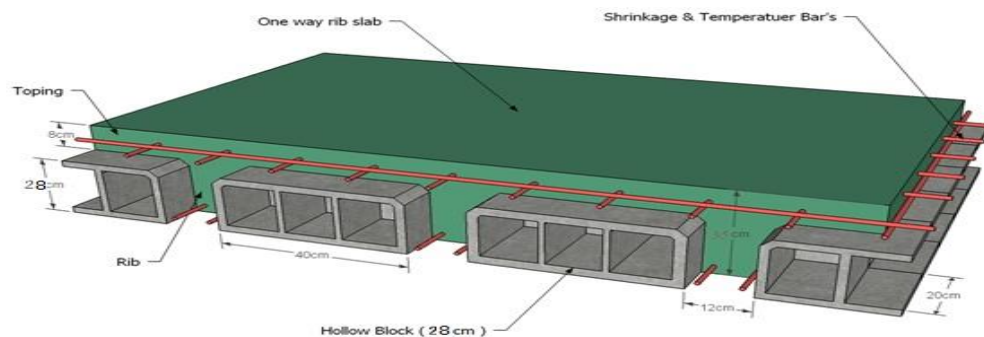


Fig 4.2: One way Ribbed slab (R2).



### ✓ Load calculations:

#### Dead load calculations:

Dead load from:	$\delta \times \gamma \times b_e$	KN/m
Tiles	$0.03 \times 23 \times 0.52$	0.3588
Mortar	$0.02 \times 22 \times 0.52$	.2288
Coarse sand	$0.07 \times 16 \times 0.52$	0.5824
Topping	$0.08 \times 25 \times 0.52$	1.04
Interior partitions	$1 \times 0.52$	.52
RC rib	$0.17 \times 25 \times 0.12$	.51
Hollow Block	$0.17 \times 9 \times 0.4$	.612
Plaster	$0.02 \times 22 \times 0.52$	.2288
	$\Sigma$	4.081

**Table (4-2) calculation of the Dead load for (R2)**

#### **Live load:**

Live load /rib =  $4.5 \text{ KN/m}^2 \times 0.52 \text{ m} = 2.34 \text{ KN/m}$ .

#### **Factored load:**

$D_u = 1.2 \times 4.081 = 4.9 \text{ KN/m}$ .

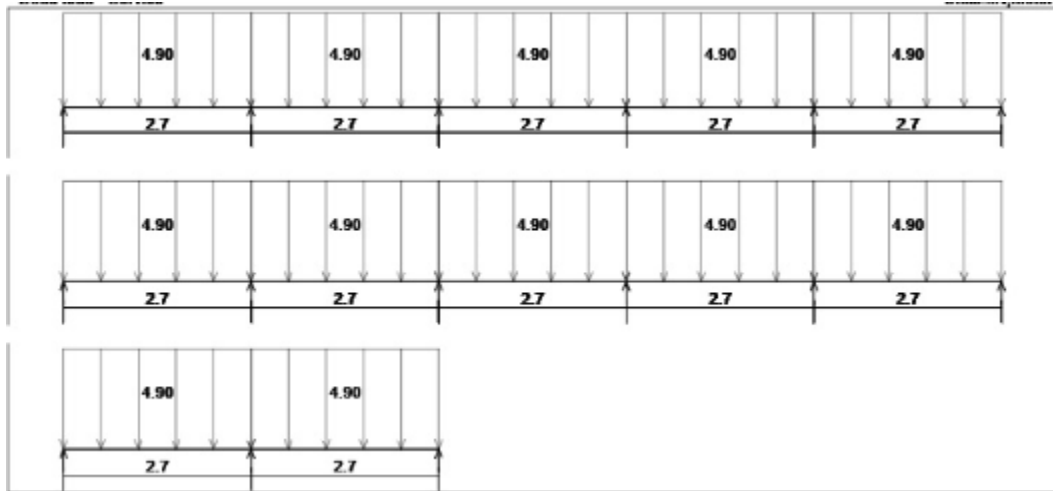
$L_u = 1.6 \times 2.34 = 3.74 \text{ KN/m}$ .

$W_u = 8.644 \text{ KN/m}$ .

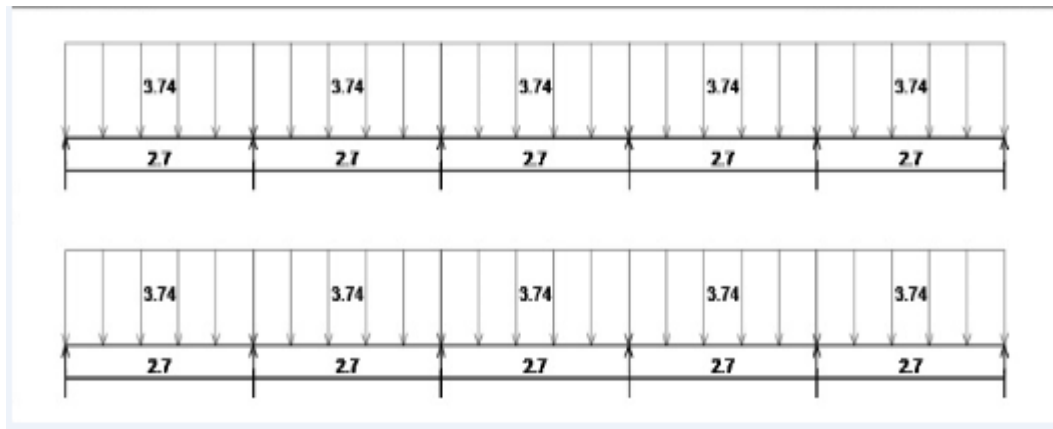
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### Factored Dead Load.



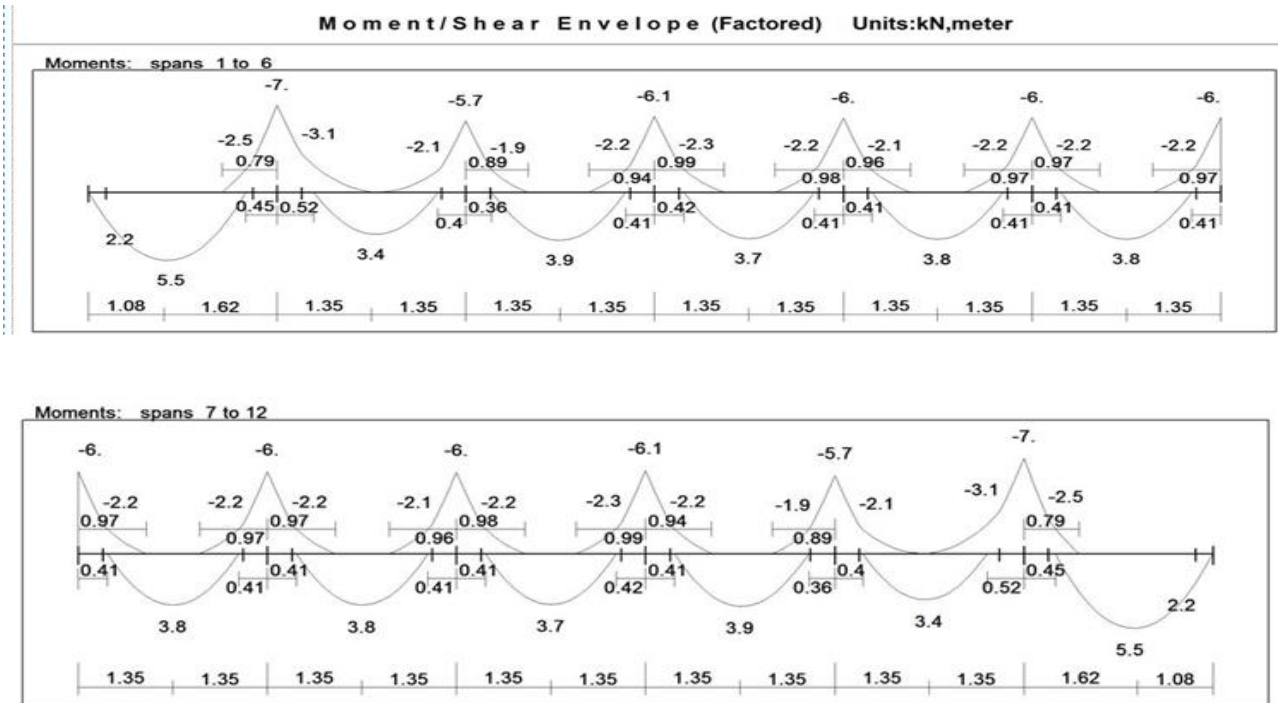
### Factored Live Load.



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### ✓ Flexural Design for Rib (R 2):

#### Moment for rib R2:



#### Design of span (2-11):

##### Design for positive moment:

$$M_u = 3.6 \text{ kN.m.}$$

Assume bar diameter 12 mm for main positive reinforcement.

$$d = h - \text{cover} - d_{\text{stirrups}} - \frac{d_b}{2} = 250 - 20 - 10 - \frac{12}{2} = 214 \text{ mm.}$$

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

$$M_{nf} = 0.85 \cdot f'_c \cdot b_e \cdot h_f \cdot \left(d - \frac{h_f}{2}\right) = 0.85 \times 24 \times 520 \times 80 \times \left(214 - \frac{80}{2}\right) \times 10^{-6} = 147.66 \text{ kN.m}$$

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$M_{nf} \gg \frac{M_u}{\phi} = 4.33 \text{ KN.m}$ , the section will be designed as **rectangular section** with  $b_e = b = 520 \text{ mm}$ .

$$R_n = \frac{M_u}{\phi b d^2} = \frac{4.33 \times 10^6}{520 \times 214^2} = 0.1818 \text{ Mpa.}$$

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

$$\rho = \frac{1}{m} \left( 1 - \sqrt{1 - \frac{2mR_n}{420}} \right) = \frac{1}{20.6} \left( 1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.1818}{420}} \right) = 0.000435$$

$$A_{s, \text{req}} = \rho \cdot b \cdot d = 0.000435 \times 520 \times 214 = 48.407 \text{ mm}^2$$

\*Check for  $A_{s, \text{min}}$ .

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

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

$$*A_{s, \text{min}} = 0.25 \frac{\sqrt{24}}{420} 120 \times 214 = 74.9 \text{ mm}^2$$

$$*A_{s, \text{min}} = \frac{1.4}{420} 120 \times 214 = 85.6 \text{ mm}^2 \text{ **Controls**}$$

$$A_{s, \text{required}} = 85.6 \text{ mm}^2.$$

Use 2Ø10,  $A_{s, \text{provided}} = 157.08 \text{ mm}^2 > A_{s, \text{required}} = 85.6 \text{ mm}^2$ ..... **Ok**

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

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### Design for span (1&12):

#### Design for positive moment:

$$M_u = 5.5 \text{ KN.m.}$$

Assume bar diameter  $\phi 12$  for main positive reinforcement.

$$d = h - \text{cover} - d_{\text{stirrups}} - \frac{d_b}{2} = 250 - 20 - 10 - \frac{12}{2} = 214 \text{ mm.}$$

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

$$\begin{aligned} M_{nf} &= 0.85 \cdot f'_c \cdot b_e \cdot h_f \cdot \left(d - \frac{h_f}{2}\right) \\ &= 0.85 \times 24 \times 520 \times 80 \times \left(214 - \frac{80}{2}\right) \times 10^{-6} = 147.66 \text{ KN.m} \end{aligned}$$

$M_{nf} \gg \frac{M_u}{\phi} = 6.11 \text{ KN.m}$ , the section will be designed as **rectangular section** with  $b_e = b = 520 \text{ mm}$ .

$$R_n = \frac{M_u}{\phi b d^2} = \frac{6.11 \times 10^6}{520 \times 214^2} = 0.256 \text{ Mpa.}$$

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

$$\rho = \frac{1}{m} \left( 1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{420}} \right) = \frac{1}{20.6} \left( 1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.256}{420}} \right) = 0.000615$$

$$A_{s, \text{req}} = \rho \cdot b \cdot d = 0.000615 \times 520 \times 214 = 68.4 \text{ mm}^2$$

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\*Check for  $A_{s,min}$ .

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

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

$$A_{s,min} = 0.25 \frac{\sqrt{24}}{420} 120 \times 214 = 74.9 \text{ mm}^2$$

$$A_{s,min} = \frac{1.4}{420} 120 \times 214 = 85.6 \text{ mm}^2 \text{ **Control**}$$

$$A_{s,required} = 85.6 \text{ mm}^2.$$

Use 2Ø10,  $A_{s,provided} = 157.08 \text{ mm}^2 > A_{s,required} = 85.6 \text{ mm}^2$ . **Ok**

$$S = \frac{120 - 40 - 20 - (2 \times 10)}{1} = 36 \text{ mm} > d_b = 10 > 25 \text{ mm} \quad \textbf{OK}$$

Check for strain:

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{157.08 \times 420}{0.85 \times 520 \times 24} = 6.22 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{6.22}{0.85} = 7.317 \text{ mm}$$

$$\varepsilon_s = 0.003 \left( \frac{d - c}{c} \right) = 0.003 \left( \frac{214 - 7.317}{7.317} \right) = 0.084741 > 0.005 \quad \textbf{Ok}$$

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### Design for moment over support (1&11):

#### Design for negative moment:

$$M_u = -3.1 \text{ KN.m.}$$

Assume bar diameter 12 mm for main positive reinforcement.

$$d = h - \text{cover} - d_{\text{stirrups}} - \frac{d_b}{2} = 250 - 20 - 10 - \frac{12}{2} = 214 \text{ mm.}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{3.1 \times 10^6}{0.9 \times 120 \times 214^2} = 0.63 \text{ Mpa.}$$

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

$$\rho = \frac{1}{m} \left( 1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{420}} \right) = \frac{1}{20.6} \left( 1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.63}{420}} \right) = 0.00152$$

$$A_{s,\text{req}} = \rho \cdot b \cdot d = 0.00152 \times 120 \times 214 = 39 \text{ mm}^2$$

\*Check for  $A_{s,\text{min}}$ .

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

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

$$1- A_{s,\text{min}} = 0.25 \frac{\sqrt{24}}{420} 120 \times 214 = 74.9 \text{ mm}^2$$

$$2- A_{s,\text{min}} = \frac{1.4}{420} 120 \times 214 = 85.6 \text{ mm}^2 \text{ **Control**}$$

$$A_{s,\text{required}} = 85.6 > 39 \text{ mm}^2.$$

Use 2 $\phi$ 10.  $A_{s,\text{provided}} = 157.08 \text{ mm}^2 > A_{s,\text{required}} = 85.6 \text{ mm}^2$ . **Ok**

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

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Check for strain:

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{157.08 \times 420}{0.85 \times 120 \times 24} = 26.95 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{26.95}{0.85} = 31.706 \text{ mm}$$

$$\varepsilon_s = 0.003 \left( \frac{d - c}{c} \right) = 0.003 \left( \frac{214 - 31.706}{31.706} \right) = 0.0172 > 0.005 \quad \mathbf{Ok}$$

### Design for negative moment over support (2-10):

$$M_u = -2.3 \text{ KN.m.}$$

Assume bar diameter  $\phi 12$  for main positive reinforcement.

$$d = h - \text{cover} - d_{\text{stirrups}} - \frac{d_b}{2} = 250 - 20 - 10 - \frac{12}{2} = 214 \text{ mm.}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{2.3 \times 10^6}{0.9 \times 120 \times 214^2} = 0.47 \text{ Mpa.}$$

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

$$\rho = \frac{1}{m} \left( 1 - \sqrt{1 - \frac{2 m R_n}{420}} \right) = \frac{1}{20.6} \left( 1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.47}{420}} \right) = 0.00112$$

$$A_{s, \text{req}} = \rho \cdot b \cdot d = 0.00112 \times 120 \times 214 = 28.8 \text{ mm}^2$$



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\*Check for  $A_{s,min}$ .

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

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

$$1- A_{s,min} = 0.25 \frac{\sqrt{24}}{420} 120 \times 214 = 74.9 \text{ mm}^2$$

$$2- A_{s,min} = \frac{1.4}{420} 120 \times 214 = 85.6 \text{ mm}^2 \text{ **Control**}$$

$$A_{s,required} = 85.6 > 28.8 \text{ mm}^2.$$

Use 2Ø10,  $A_{s,provided} = 157.08 \text{ mm}^2 > A_{s,required} = 85.6 \text{ mm}^2$ . ..... **Ok**

$$S = \frac{120 - 40 - 20 - (2 \times 12)}{1} = 36 \text{ mm} > d_b = 10 > 25 \text{ mm} \quad \textbf{OK}$$

Check for strain:

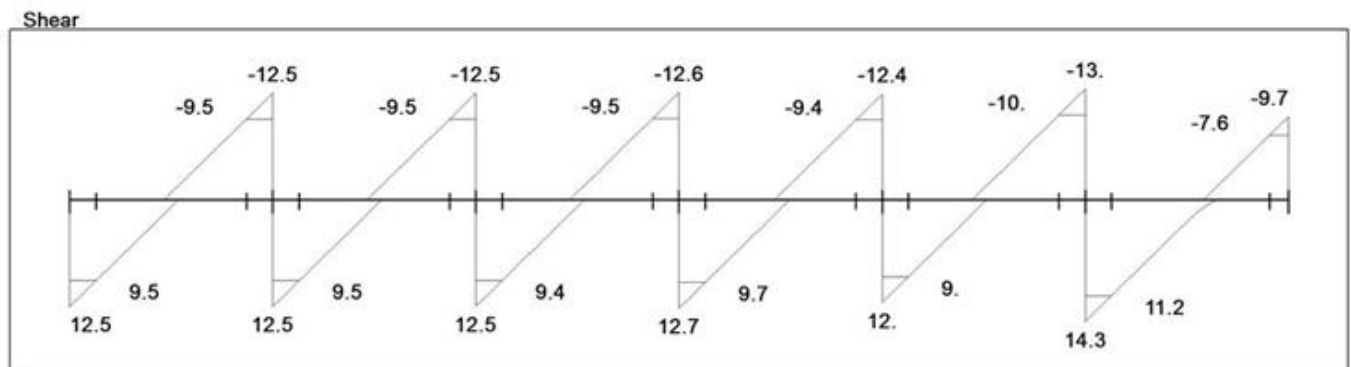
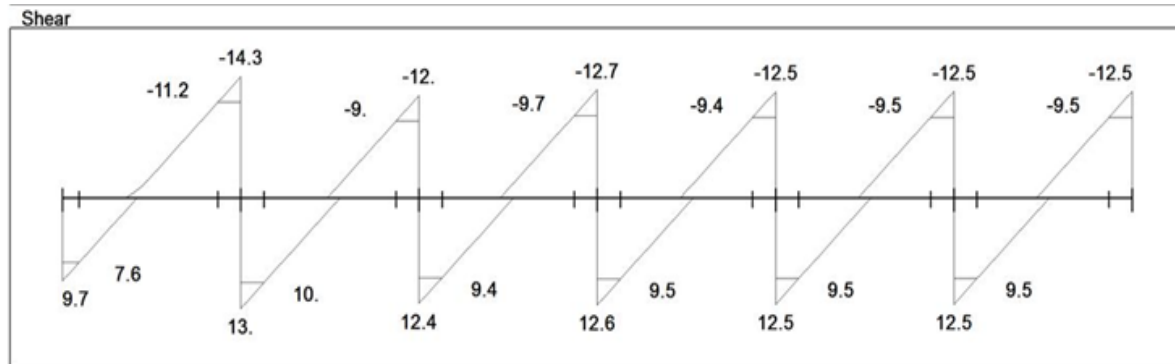
$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{157.08 \times 420}{0.85 \times 120 \times 24} = 26.95 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{26.95}{0.85} = 31.706 \text{ mm}$$

$$\epsilon_s = 0.003 \left( \frac{d - c}{c} \right) = 0.003 \left( \frac{214 - 31.706}{31.706} \right) = 0.0172 > 0.005 \quad \textbf{Ok}$$

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### Shear Design for rib (R 2):



$$V_{u \max} = 11.2 \text{ KN}$$

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

$$V_c = \frac{1.1}{6} \lambda \sqrt{f'_c} b_w d = \frac{1.1}{6} \sqrt{24} \times 120 \times 214 \times 10^{-3} = 23.06 \text{ KN}$$

$$\phi V_c = 0.75 \times 23.06 = 17.3 \text{ KN} > V_u = 11.2 \text{ KN}.$$

No shear reinforcement is required.

### 4.6 Design of Beam (B 3):

✓ Dead load calculations:

Total dead load on beam B3

Dead load from:	$\delta \times \gamma \times b_e$	KN/m
Tiles	$0.03 \times 23 \times 0.7$	0.483
Mortar	$0.02 \times 22 \times 0.7$	0.31
Coarse sand	$0.07 \times 16 \times 0.7$	0.784
Plaster	$0.02 \times 22 \times 0.7$	0.31
Interior partitions	$1 \times 0.7$	0.7
RC BEAM	$0.5 \times 25 \times 0.7$	8.75
	$\Sigma$	11.34

Table (4-3) calculation of the Dead load for (B3).

#### Reaction from rib R2

Reactions Factored							
Dead R	5.22	15.	12.76	13.36	13.2	13.24	13.23
Live R	4.52	12.23	11.66	11.91	11.77	11.78	11.78
Max R	9.73	27.23	24.42	25.27	24.97	25.02	25.
Min R	4.68	19.66	16.48	17.6	17.37	17.45	17.43

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Reaction from rib R1

Reactions Factored							
DeadR	5.22	15.	12.76	13.36	13.2	13.24	13.23
LiveR	2.51	6.8	6.48	6.62	6.55	6.55	6.55
Max R	7.73	21.81	19.24	19.98	19.74	19.79	19.77
Min R	4.92	17.59	14.83	15.72	15.52	15.58	15.56

Dead load from self-weight of beam =  $11.34 \times 1.2 = 13.6 \text{ KN/m}$ .

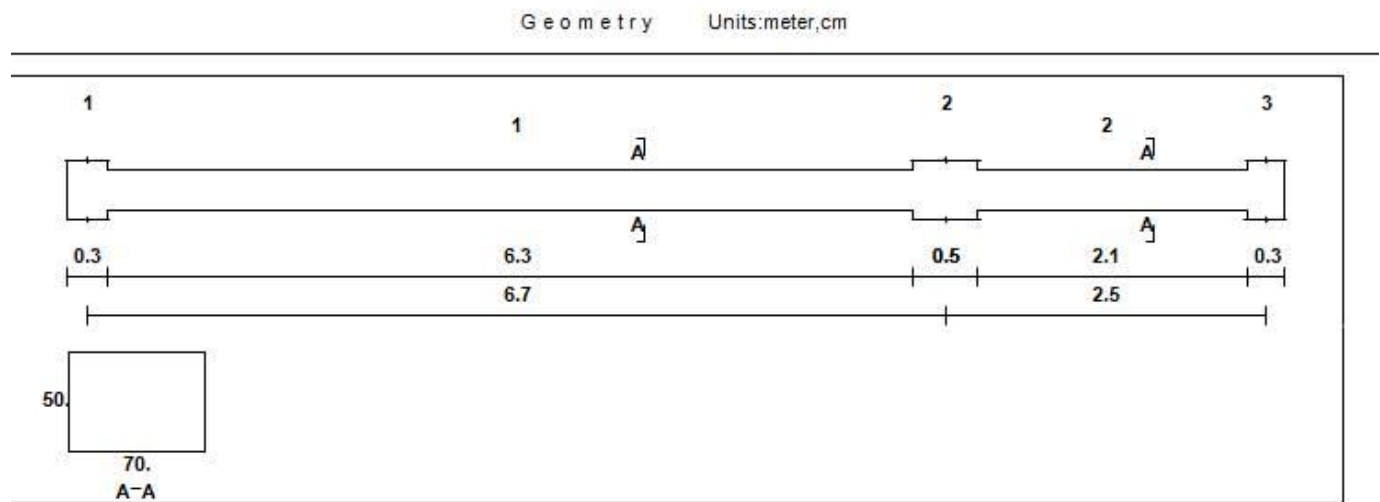
Total dead load =  $\frac{12.76}{0.52} + 12.456 = 36.686 \text{ KN/m}$ .

Live Load on corridor (span2) =  $4.5 \times 1.6 + 11.66 = 18.86 \text{ KN/m}^2$ .

Live Load on class rooms (span1) =  $2.5 \times 1.6 + 6.48 = 10.48$ .

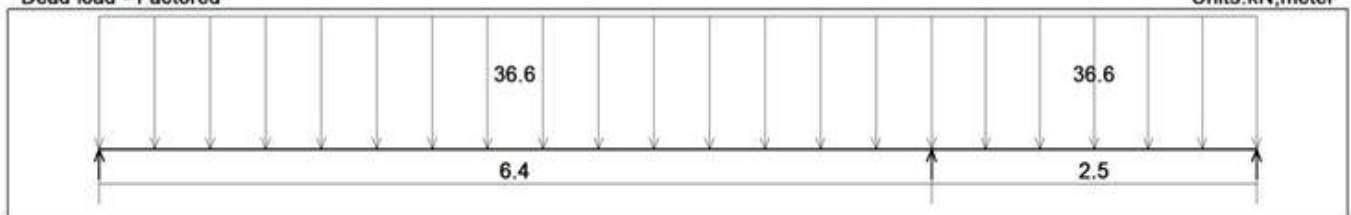
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### Statically system and Dimensions for Beam B3:

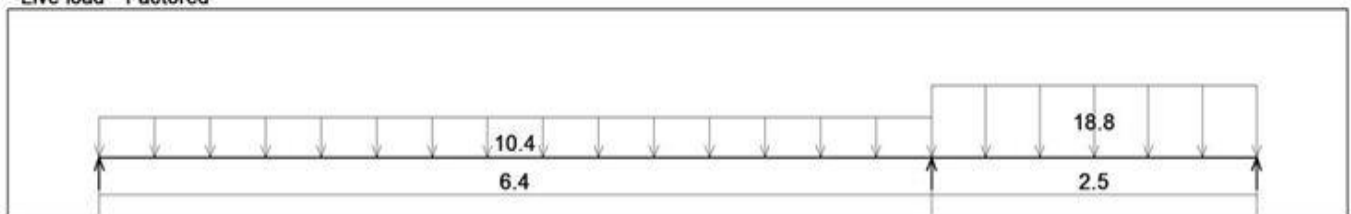


load group no. 1  
Dead load - Factored

Units: kN, meter

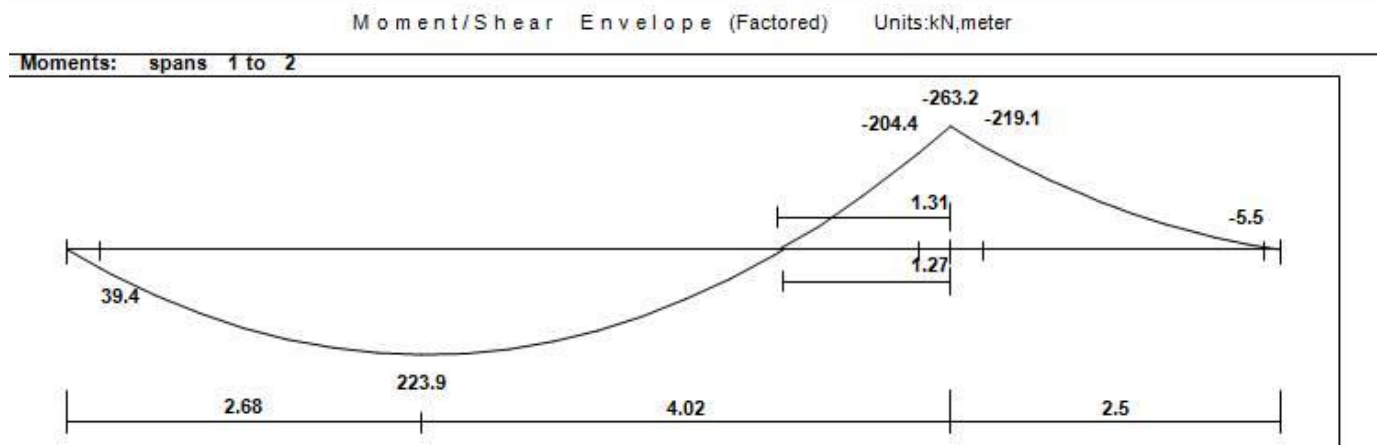


Live load - Factored



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### ✓ Flexural Design for Beam (B3):



### Design for positive moment for beam B3 :

#### Span 1:

Max positive moment = 223.9 kN/m.

Assume bar diameter 20 mm for main positive reinforcement.

$$d = h - \text{cover} - d_{\text{stirrups}} - \frac{d_b}{2} = 500 - 40 - 10 - \frac{20}{2} = 440 \text{ mm.}$$

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

$$M_{nf} = 0.85 \cdot f'_c \cdot b \cdot h_f \cdot \left(d - \frac{h_f}{2}\right)$$

$$= 0.85 \times 24 \times 700 \times 500 \times \left(440 - \frac{500}{2}\right) \times 10^{-6} = 1356.6 \text{ kN.m}$$

$$M_{nf} \gg \frac{M_u}{\phi} = 248.78 \text{ kN.m, the section will be designed as rectangular section}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{223.9 \times 10^6}{0.9 \times 700 \times 440^2} = 1.835 \text{ Mpa.}$$

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

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{420}}\right) = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2 \times 1.835 \times 20.6}{420}}\right) = 0.0046 > \rho_{\min} = 0.00033$$

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$$A_{s,req} = p.b.d = 0.0046 \times 440 \times 700 = 1416.8 \text{ mm}^2. \text{ **Control**}$$

\*Check for  $A_{s,min}$ .

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

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

$$4 \quad A_{s,min} = 0.25 \frac{\sqrt{24}}{420} 700 \times 440 = 899.15 \text{ mm}^2$$

$$5 \quad A_{s,min} = \frac{1.4}{420} 700 \times 440 = 1026.7 \text{ mm}^2$$

$$A_{s,required} = 1416.8 \text{ mm}^2.$$

Use 5Ø18,  $A_{s,provided} = 1527 \text{ mm}^2 > A_{s,required} = 1416.8 \text{ mm}^2$ . **Ok**

$$S = \frac{700 - 80 - 20 - (5 \times 18)}{4} = 127.5 \text{ mm} > d_b = 18$$

$> 25 \text{ mm}$       **OK**

Check for strain:

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{1527 \times 420}{0.85 \times 700 \times 24} = 44.9 \text{ mm}.$$

$$c = \frac{a}{\beta_1} = \frac{44.9}{0.85} = 52.83 \text{ mm}.$$

$$\epsilon_s = 0.003 \left( \frac{d - c}{c} \right) = 0.003 \left( \frac{440 - 52.83}{52.83} \right) = 0.022 > 0.005 \quad \textbf{Ok}$$

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### Design for Negative moments:

#### Over Support 2:

$$M_u = 219.1 \text{ KN.m.}$$

Assume bar diameter 20 mm for main negative reinforcement.

$$d = h - \text{cover} - d_{\text{stirrups}} - \frac{d_b}{2} = 500 - 40 - 10 - \frac{20}{2} = 440 \text{ mm.}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{219.1 \times 10^6}{0.9 \times 700 \times 440^2} = 1.80 \text{ Mpa.}$$

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

$$\rho = \frac{1}{m} \left( 1 - \sqrt{1 - \frac{2mR_n}{420}} \right) = \frac{1}{20.6} \left( 1 - \sqrt{1 - \frac{2 \times 20.6 \times 1.8}{420}} \right) = 0.00448$$

$$A_{s,\text{req}} = \rho \cdot b \cdot d = 0.00393 \times 700 \times 440 = 1381.11 \text{ mm}^2. \text{ **Control**}$$

\*Check for  $A_{s,\text{min}}$ .

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

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

$$1- A_{s,\text{min}} = 0.25 \frac{\sqrt{24}}{420} 700 \times 440 = 899.15 \text{ mm}^2$$

$$2- A_{s,\text{min}} = \frac{1.4}{420} 700 \times 440 = 1026.7 \text{ mm}^2$$

$$A_{s,\text{required}} = 1381.11 \text{ mm}^2.$$

$$\text{Use } 6\phi 14, A_{s,\text{provided}} = 1526.8 \text{ mm}^2 > A_{s,\text{required}} = 1381.11 \text{ mm}^2. \text{ **Ok**}$$

$$S = \frac{700 - 80 - 20 - (6 \times 14)}{5} = 127.5 \text{ mm} > d_b = 18$$

$$> 25 \text{ mm} \quad \text{OK}$$



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Check for strain:

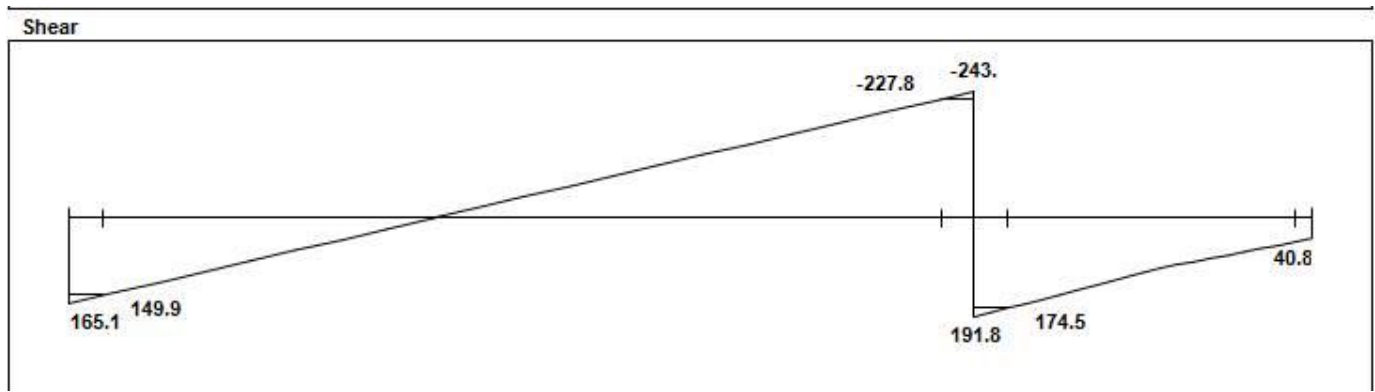
Check for strain:

$$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{1527 \times 420}{0.85 \times 700 \times 24} = 44.9 \text{ mm.}$$

$$c = \frac{a}{\beta_1} = \frac{44.9}{0.85} = 52.83 \text{ mm.}$$

$$\varepsilon_s = 0.003 \left( \frac{d - c}{c} \right) = 0.003 \left( \frac{440 - 52.83}{52.83} \right) = 0.022 > 0.005 \quad \mathbf{Ok}$$

### Shear Design for beam (B3):



$$V_{u \max} = 227.8 \text{ kN}$$

$$V_c = \frac{1}{6} \lambda \sqrt{f'_c} b_w d = \frac{1}{6} \sqrt{24} \times 700 \times 440 \times 10^{-3} = 251.5 \text{ kN}$$

$$\phi V_c = 0.75 \times 251.5 = 188.61 \text{ kN.}$$

Shear reinforcement is required.

$$\phi V_c < V_c$$

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Case III :  $\phi V_c < V_u \leq \phi(V_c + V_{s,min})$

Provide minimum shear reinforcement

$$V_{s,min} \geq \frac{1}{16} * \sqrt{f'_c} * b_w * d = \frac{1}{16} * \sqrt{24} * 7000 * 440 * 10^{-3} = 94.3 \text{ KN.}$$

$$\phi V_{s,min} = 70.73 \text{ KN}$$

$$\geq \frac{1}{3} * b_w * d = \frac{1}{3} * 700 * 440 * 10^{-3} = 102.7 \text{ KN}$$

$$\phi V_{s,min} = 70.73 \dots \dots \dots \text{control}$$

$$\phi V_c = 188.61 \text{ KN} < V_u = 227.8 \text{ KN} \leq \phi(V_c + V_{s,min}) = 259.34 \text{ KN}$$

For use  $\phi 10 - 2\text{legs}$

$$s = \frac{A_v * f_y * d}{V_s = \frac{V_u}{\phi} - V_c} = \frac{157.07 * 420 * 440}{52.2} * 10^{-3} = 307 \text{ mm}$$

$$S \leq \frac{d}{2} = \frac{440}{2} = 220 \text{ mm.}$$

$$\leq 600 \text{ mm.}$$

**$\therefore$  Use 2 Leg  $\Phi 10 @ 20 \text{ Cm C/C}$**

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### 4.7Pos: Design of one way solid slab.

NOTE:

- ✓ \*B300....  $f_c' = 25 \text{ N/mm}^2 \text{ (MPa)}$
- ✓ The specified yield strength of the reinforcement  $\{f_y = 420 \text{ N/mm}^2 \text{ (MPa)}\}$
- ✓ Live Load (LL) =  $5 \text{ KN/m}^2$

#### ✓ Load calculations:

##### Dead load calculations:

Dead load from:	$\delta \times \gamma$	KN/m
Slab	$0.15 \times 25 \times 1$	3.75
Plaster	$0.02 \times 22 \times 1$	0.44
	$\Sigma$	4.19

Table (4-4) calculation of the one way solid slab Dead load for (S1)

#### Dead load:

Dead load =  $4.19 \text{ KN/m}^2$ .

#### Live load:

Live load =  $5 \text{ KN/m}^2$ .

#### Factored load:

$D_u = 1.2 \times 4.19 = 5.028 \text{ KN/m}$ .

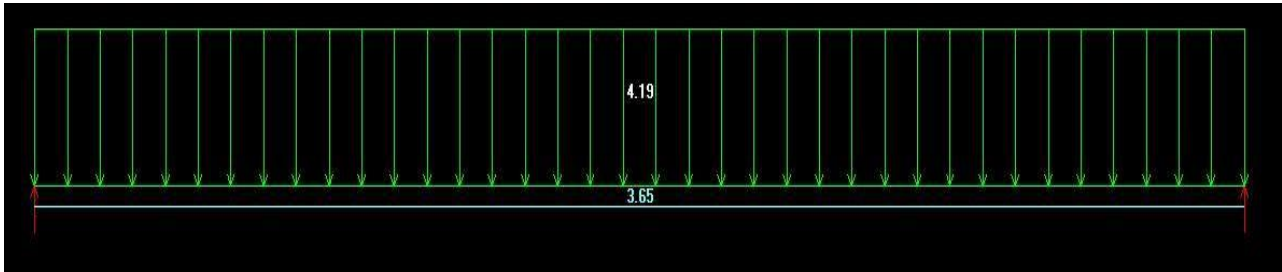
$L_u = 1.6 \times 5 = 8 \text{ KN/m}$ .

$W_u = 13.028 \text{ KN/m}$ .

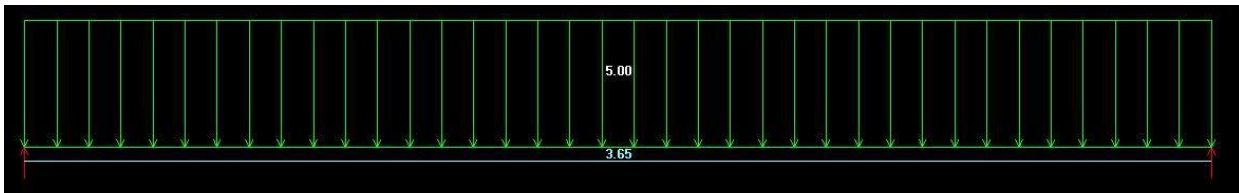
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Dead load service

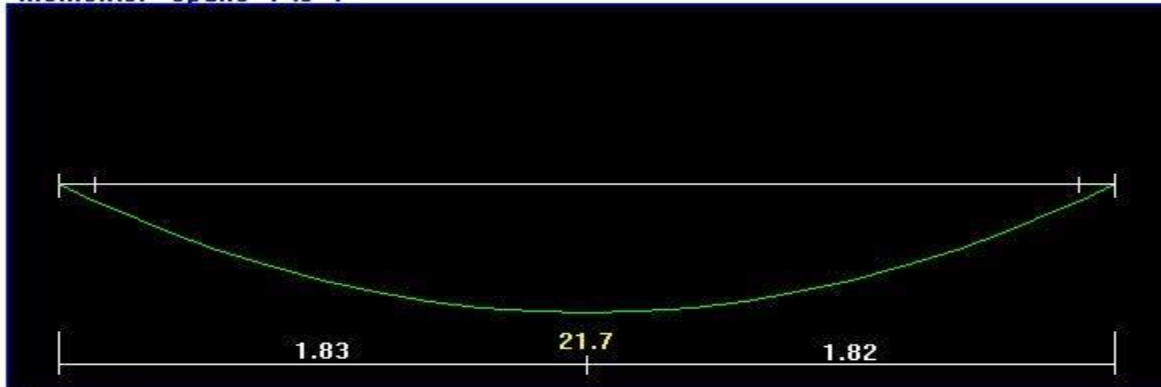


Live load service

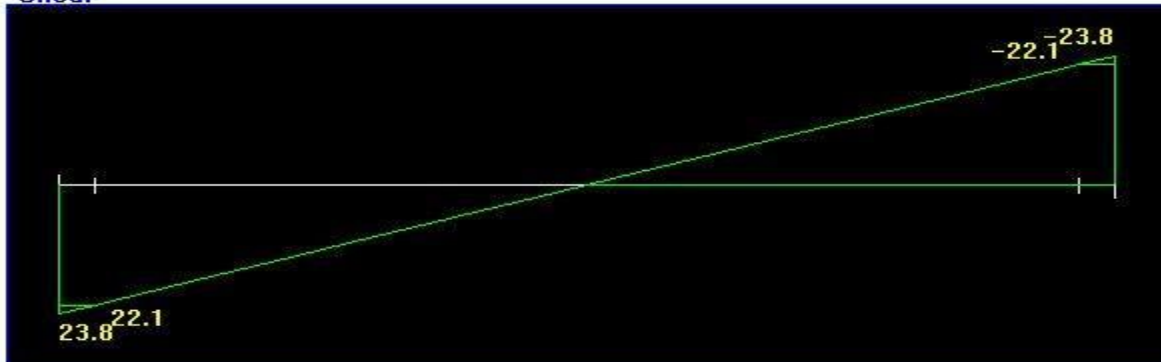


✓ Flexural Design for (S1):

Moments: spans 1 to 1



Shear



**For shear:**

**check whether thickness is adequate for shear:**

$$V_{u,max} = 22.1 \text{ KN/ 1m strip}$$

$$d = h - \text{cover} - d_{\text{stirrups}} - \frac{d_b}{2} = 150 - 20 - \frac{12}{2} = 124 \text{ mm.}$$

$$\Phi V_c = \frac{1}{6} * \Phi * \sqrt{f_c'} * b_w * d$$

$$= \frac{1}{6} * 0.75 * \sqrt{25} * 1000 * 124 = 77.5 \text{ KN / 1 m strip}$$

$$\frac{1}{2} \Phi V_c = \frac{77.5}{2} = 38.75 \text{ KN/1m strip}$$

$$V_{u,max} = 22.1 \text{ KN/ 1m strip} < \Phi V_c$$

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

**Design for positive moment.**

$$M_u = 21.7 \text{ KN.m.}$$

Assume bar diameter  $\phi$  12 for main positive reinforcement.

$$d = h - \text{cover} - \frac{d_b}{2} = 150 - 20 - \frac{12}{2} = 124 \text{ mm.}$$

the section will be designed as **rectangular section**  $b = 1000 \text{ mm}$ .

$$R_n = \frac{M_u}{\phi b d^2} = \frac{21.7 \times 10^6}{0.9 \times 1000 \times 124^2} = 1.57 \text{ Mpa.}$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 \times 25} = 19.76$$

$$\rho = \frac{1}{m} \left( 1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{420}} \right) = \frac{1}{19.76} \left( 1 - \sqrt{1 - \frac{2 \times 19.76 \times 1.57}{420}} \right) = 0.00389$$

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$$A_{s,req} = \rho \cdot b \cdot d = 0.00389 \times 1000 \times 124 = 482 \text{ mm}^2/\text{m} \quad \text{Control}$$

\*Check for  $A_{s,min}$ .

$$A_{s,min} = 0.0018 \times 1000 \times 150 = 270 \text{ mm}^2/\text{m}$$

$$A_{s,required} = 482 \text{ mm}^2/\text{m}.$$

Use  $\phi 12/20 \text{ cm}$  ,  $A_{s,provided} = 565 \text{ mm}^2/\text{m} > A_{s,required} = 482 \text{ mm}^2/\text{m}$  . **Ok**

**step ( s ) is the smallest of :-**

$$\leq 380 \left( \frac{280}{f_s} \right) - 2.5 * C_c$$

$$\leq 380 * \left( \frac{280}{\frac{2}{3}f_y} \right) - 2.5 * 20 = 380 * \left( \frac{280}{\frac{2}{3} * 420} \right) - 2.5 * 20 = 330 \text{ mm}$$

$$\leq 300 \left( \frac{280}{f_s} \right) = 300 * \left( \frac{280}{\frac{2}{3}f_y} \right) = 300 * \left( \frac{280}{\frac{2}{3} * 420} \right) = 300 \text{ mm (control)}$$

$$\leq 3 * h = 3 * 150 = 450 \text{ m}$$

$$\leq 450 \text{ mm}.$$

**Use  $\Phi 12 @ 20 \text{ cm C/C}$  in main directions.**

**( Temperature and Shrinkage ) :**

$$\rightarrow \rho = 0.0018$$

$$A_{s \text{ min}} = \rho_{min} * b * h = 0.0018 * 1000 * 150 = 270 \text{ mm}^2 \quad (\text{control})$$

**Use  $\Phi 10 @ 20 \text{ mm}$**

**step ( s ) is the smallest of :-**

$$\leq 5 * h = 5 * 150 = 750 \text{ m}$$

$$\leq 450 \text{ mm. (Control)}$$

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### - Check for strain:

Tension = Compression

$$A_s * F_y = 0.85 * f_c' * b * a$$

$$565 * 420 = 0.85 * 25 * 1000 * a$$

$$a = 11.167m$$

$$c = \frac{a}{\beta_1} = \frac{11.167}{0.85} = 13.138mm$$

$$\epsilon_s = \frac{124 - 13.138}{13.138} * 0.003$$

$$\epsilon_s = 0.03 > 0.005 \longrightarrow ok$$

### 4.8Pos: Design of two way solid slab:

#### Dead load calculations:

##### For 1m strip

Dead load from:	$\delta \times \gamma$	KN/m
Slab	0.25×25×1	6.25
Plaster	0.02×22×1	0.44
	$\Sigma$	6.69

Table (4-4) calculation of the Dead load two solid slab (S2)

#### **Dead load:**

Dead load = 6.69 KN/m<sup>2</sup>.

#### **Live load:**

Live load = 5 KN/m<sup>2</sup>.



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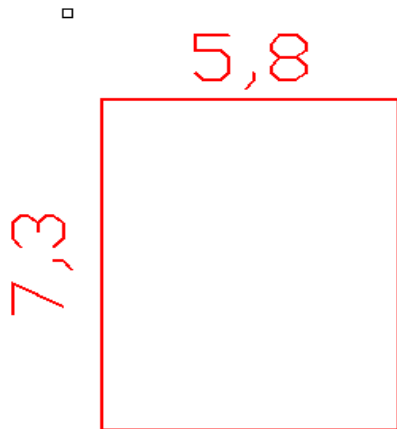
### Determination of factored dead & live load

$$\text{Factored dead load} = 1.2 * \text{Dead load} = 1.2 * 6.69 = 8.028 \text{ KN/m}^2.$$

$$\text{Factored Live load} = 1.6 * \text{live load} = 1.6 * 5 = 8 \text{ KN/m}^2.$$

$$W = 8.025 + 8 = 16.025 \text{ KN/m}^2$$

### Statically system for S19



### ✓ Flexural Design for (S19) :

#### Moment's calculations:-

$$M_a = C_a w l_a^2 b \quad \text{and} \quad M_b = C_b w l_b^2 b$$

$$L_a/L_b = 5.8/7.3 = 0.79 \dots\dots\dots \text{Case 1}$$

**\*Negative moments:**

#### Short direction

$$C_{a,neg}(l_a/l_b=0.79) = 0$$

$$M_{a-neg} = C_a * W * L_a^2 * b = 0.00 \text{ KN.m}$$

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### Long direction

$$C_{b,neg}(I_a/I_b=0.79) = 0$$

$$M_{b+ve} = C_b * W * L_b^2 * b = 0.00 \text{ kN.m}$$

**Positive moments :**

### Short direction

$$C_{a,D}(I_a/I_b=0.95) = 0.056$$

$$M_{a+ve,D} = C_a * W * L_a^2 * b = 0.056 * 8.025 * 5.8^2 * 1 = 15.11 \text{ KN.m}$$

$$C_{a,L}(I_a/I_b=0.79) = 0.056$$

$$M_{a+ve,L} = C_a * W * L_a^2 * b = 0.056 * 8 * 5.8^2 * 1 = 15.07 \text{ KN.m}$$

$$M_{a+ve} = M_{a+ve,L} + M_{a+ve,D} = 30.18 \text{ KN.m}$$

### Long direction

$$C_{b,D}(I_a/I_b=0.79) = 0.023$$

$$M_{b+ve,D} = C_b * W * L_b^2 * b = 0.023 * 8.025 * 7.3^2 * 1 = 9.84 \text{ KN.m}$$

$$C_{b,L}(I_a/I_b=0.79) = 0.023$$

$$M_{b+ve,L} = C_b * W * L_b^2 * b = 0.023 * 8 * 7.3^2 * 1 = 9.8 \text{ KN.m}$$

$$M_{b+ve} = M_{b+ve,L} + M_{b+ve,D} = 19.64 \text{ KN.m}$$

**\*Negative moments at Discontinuous edge (1/3 \* positive moments):**  $M_{c,neg} = \frac{30.18}{3} = 10.06 \text{ KN.m}$

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**Design for Negative and Positive moments:**

\* *Short direction l = 5.8*

$$d = h - \text{cover} - d_{\text{stirrups}} - \frac{d_b}{2} = 250 - 20 - 10 - \frac{12}{2} = 214 \text{ mm.}$$

**Positive Moment:**

Midspan: ( $M_u = +30.18 \text{ KN.m}$ )

$$R_n = \frac{M_u}{\phi b d^2} = \frac{30.18 \times 10^6}{0.9 \times 1000 \times 214^2} = 0.732 \text{ Mpa.}$$

$$m = \frac{f_y}{0.85 f'_c} = \frac{420}{0.85 \times 25} = 19.76$$

$$\rho = \frac{1}{m} \left( 1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{420}} \right) = \frac{1}{19.76} \left( 1 - \sqrt{1 - \frac{2 \times 19.76 \times 0.732}{420}} \right) = 0.001774$$

$$A_s = \rho \cdot b \cdot d = 0.001774 \times 1000 \times 214 = 379.64 \text{ mm}^2 / \text{m} .$$

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

$$A_{s,\min} = 0.0018 * 1000 * 250 = 450 \text{ mm}^2 / \text{m}$$

$$A_{s,\text{required}} = 450 \text{ mm}^2 .$$

Use  $\phi 12 / 20 \text{ cm}$  **Bottom**,  $A_{s,\text{provided}} = 565 \text{ mm}^2 / \text{m} > A_{s,\text{required}} = 450 \text{ mm}^2 / \text{m} . \quad \text{Ok}$

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### - Check for strain:

Tension = Compression

$$A_s * F_y = 0.85 * f_c' * b * a$$

$$565 * 420 = 0.85 * 25 * 1000 * a$$

$$a = 11.17 \text{ m}$$

$$c = \frac{a}{\beta_1} = \frac{11.17}{0.85} = 13.14 \text{ mm}$$

$$\varepsilon_s = \frac{214 - 13.14}{13.14} * 0.003$$

$$\varepsilon_s = 0.046 > 0.005 \longrightarrow \text{ok}$$

\* long direction  $l = 7.3$

$$d = h - \text{cover} - d_{\text{stirrups}} - \frac{d_b}{2} = 250 - 20 - 10 - \frac{12}{2} = 214 \text{ mm.}$$

### Positive Moment:

Midspan: ( $M_u = +19.64 \text{ KN.m}$ )

$$R_n = \frac{M_u}{\phi b d^2} = \frac{19.64 \times 10^6}{0.9 \times 1000 \times 214^2} = 0.467 \text{ Mpa.}$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 \times 25} = 19.76$$

$$\rho = \frac{1}{m} \left( 1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{420}} \right) = \frac{1}{19.76} \left( 1 - \sqrt{1 - \frac{2 \times 19.76 \times 0.467}{420}} \right) = 0.001124$$

$$A_s = \rho \cdot b \cdot d = 0.001124 \times 1000 \times 214 = 240.1 \text{ mm}^2 / \text{m.}$$

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

$$A_{s,\min} = 0.0018 * 1000 * 250 = 450 \text{ mm}^2 / \text{m}$$

$$A_{s,\text{required}} = 450 \text{ mm}^2. \text{ Use } \phi 12 / 20 \text{ cm}$$

$$\text{Bottom, } A_{s,\text{provided}} = 565 \text{ mm}^2 > A_{s,\text{required}} = 450 \text{ mm}^2. \quad \text{Ok}$$

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### - Check for strain:

Tension = Compression

$$A_s * F_y = 0.85 * f_c' * b * a$$

$$565 * 420 = 0.85 * 25 * 1000 * a$$

$$a = 11.17m$$

$$c = \frac{a}{\beta_1} = \frac{11.17}{0.85} = 13.14mm$$

$$\varepsilon_s = \frac{214 - 13.14}{13.14} * 0.003$$

$$\varepsilon_s = 0.046 > 0.005 \longrightarrow ok$$

### ✓ Shear Design for (S2):

$$W_a(l_a/l_b=0.79) = 0.71$$

- The total load on the panel being  $(16.025 \times 7.3 \times 5.8 = 678.5 \text{ KN})$
- The load at face of the long beam is  $(0.20 \times 678.5 \times 1 / (5.8 \times 2) = 11.7 \text{ KN})$

$$V_{ud} = 11.7 - 0.285 \times 16.025 = 7.13 \text{ KN}$$

$$V_c = \frac{1.1}{6} \lambda \sqrt{f_c'} b_w d = \frac{1.1}{6} \sqrt{25} \times 1000 \times 214 \times 10^{-3} = 196.17 \text{ KN}$$

$$\phi V_c = 0.75 \times 196.17 = 147.13 \text{ KN}$$

$$V_{ud} < \phi V_c.$$

No shear reinforcement is required

### 4.9 Design of long column (C 6):

The total live and dead load at on column at 4th story

**LL=110    DL=612**

$P_{uTotal} = 722 \text{ KN}$  is factor

#### 4.9.1 Check the slenderness effect:

(Non-sway system braced  $K=1$ )

$$\left(\frac{M_1}{M_2}\right) = 1 \quad \text{braced frame with } M \text{ min}$$

$$\frac{kL_u}{r} < 34 - 12 \left(\frac{M_1}{M_2}\right) \leq 40 \quad \text{ACI(10.12.2)}$$

$$r = \sqrt{\frac{I}{A}} \approx 0.3h = 0.3 \times 0.25 = 0.075 \quad \text{because is rectangel}$$

$$r = \sqrt{\frac{I}{A}} \approx 0.3h = 0.3 \times 0.5 = 0.15.$$

$$L_u = 3.12 \text{ m}$$

$$\frac{kL_u}{r_x} = \frac{1 \times 3.38}{0.075} = 45 > (34 - 12) = 22$$

So the column is long at y axis

$$\frac{kL_u}{r_y} = \frac{1 \times 3.38}{0.15} = 22.5 < (34 - 12) = 22$$

So the column is long at x axis

### 4.9.2 Calculate $e_{\min}$ , $M_{\min}$ :

$$e_{\min} = 15 + 0.03h = 15 + 0.03 \times 250 = 22.5 \text{ mm.}$$

$$M_{\min} = P_u \times e_{\min} = 722 \times 0.0225 = 16.25 \text{ KN.m}$$

$$f'_c = 24 \text{ Mpa} \quad , \quad F_y = 420 \text{ Mpa}$$

$$E_c = 4700 \sqrt{f'_c} = 4700 \sqrt{24} = 23025.2 \text{ Mpa.}$$

$$I_g = \frac{b \cdot h^3}{12} = \frac{500 \cdot 250^3}{12} = 0.65 \times 10^9 \text{ mm}^4.$$

$$\beta_{\text{dns}} = \frac{D_u}{P_u} = \frac{612}{720} = 0.85 < 1.$$

$$E.I = \frac{0.4 E_c I_g}{1 + \beta_{\text{dns}}} = \frac{0.4 \times 23025.2 \times 0.65}{1.85} = 3236 \text{ KN.m}^2$$

### 4.9.3 Determine of Euler buckling load:

$$P_c = \frac{\pi^2 EI}{(K l_u)^2} = \frac{\pi^2 \times 3236}{(3.38)^2} = 2795.5 \text{ KN}$$

### 4.9.4 Calculate the moment magnifier factor:

$$C_m = 0.6 + 0.4 \left( \frac{M_1}{M_2} \right) = 1 > 0.4$$

$$\delta_{ns} = \frac{C_m}{1 - \frac{P_u}{0.75 P_c}} = \frac{1}{1 - \frac{722}{0.75 \times 2795.5}} = 1.52 > 1 \quad ok$$

The magnified (e) and (M):

$$e = \delta_{ns} e_{min} = 1.52 \times 22.5 = 34.2 \text{ mm}$$

$$M = \delta_{ns} M_{min} = 1.52 \times 16.25 = 24.7 \text{ KN.m.}$$

From interaction diagram selecting the 25 mm bars & 10 mm stirrups

$$e/h = 34.2/250 = 0.14$$

$$(d-d')/h = (250 - 80 - 10 \times 2 - 25)/250 = 0.5$$

$$\phi P_n / A_g = 722 \times 1000 \times 0.145 / (250 \times 500) = 0.838 \text{ Ksi}$$

From the interaction diagram constructed in PCA \_ COLUMN program at four side:

$$\rho = 0.01.$$

$$A_s = \rho \times A_g = 0.01 \times (250 \times 500) = 1250 \text{ mm}^2$$

$$n_{\phi 14} = \frac{1250}{153.9} = 8\phi 14$$

Use 8 $\phi$ 14



### 4.9.5 Calculate $e_{min}$ , $M_{min}$ :

$$e_{min} = 15 + 0.03h = 15 + 0.03 \times 500 = 30 \text{ mm.}$$

$$M_{min} = P_u \times e_{min} = 722 \times 0.03 = 21.66 \text{ KN.m}$$

$$f'_c = 24 \text{ Mpa} \quad , \quad F_y = 420 \text{ Mpa}$$

$$E_c = 4700 \sqrt{f'_c} = 4700 \sqrt{24} = 23025.2 \text{ Mpa.}$$

$$I_g = \frac{b.h^3}{12} = \frac{250.500^3}{12} = 2.6 \times 10^9 \text{ mm}^4.$$

$$\beta_{dns} = \frac{D_u}{P_u} = \frac{612}{720} = 0.85 < 1.$$

$$E.I = \frac{0.4 E_c I_g}{1 + \beta_{dns}} = \frac{0.4 \times 23025.2 \times 2.6}{1.85} = 12944 \text{ KN.m}^2$$

### 4.9.6 Determine of Euler buckling load:

$$P_c = \frac{\pi^2 EI}{(Kl_u)^2} = \frac{\pi^2 \times 12944}{(3.38)^2} = 11182 \text{ KN}$$

### 4.9.7 Calculate the moment magnifier factor:

$$C_m = 0.6 + 0.4 \left( \frac{M_1}{M_2} \right) = 1 > 0.4$$

$$\delta_{ns} = \frac{C_m}{1 - \frac{P_u}{0.75P_c}} = \frac{1}{1 - \frac{722}{0.75 \times 11182}} = 1.09 > 1 \quad ok$$

The magnified (e) and (M):

$$e = \delta_{ns} e_{min} = 1.09 \times 30 = 32.8 \text{ mm}$$

$$M = \delta_{ns} M_{min} = 1.09 \times 21.66 = 23.61 \text{ KN.m.}$$

From interaction diagram & select the  $\phi 25$  bars &  $\phi 10$  stirrup

$$e/h = 32.8/500 = 0.06$$

$$(d-d')/h = (500 - 80 - 10 \times 2 - 25)/500 = 0.75$$

$$\phi P_n / A_g = 722 \times 1000 \times 0.145 / (250 \times 500) = 0.838 \text{ Ksi}$$

From the interaction diagram constructed in PCA \_ COLUMN program at four side:

$$\rho = 0.01.$$

$$A_s = \rho \times A_g = 0.01 \times (250 \times 500) = 1250 \text{ mm}^2$$

$$n_{\phi 14} = \frac{1250}{153.9} = 8\phi 14$$

Use  $8\phi 14$

### 4.9.8 Design of stirrups:

The spacing of ties shall not exceed the smallest of:

- $16 \times d_b = 16 \times 14 = 224 \text{ mm}$                       control.
- $48 \times d_s = 48 \times 10 = 480 \text{ mm}$
- Least diminution of the column = 300 mm

Use  $\phi 10 @ 200 \text{ mm}$ .

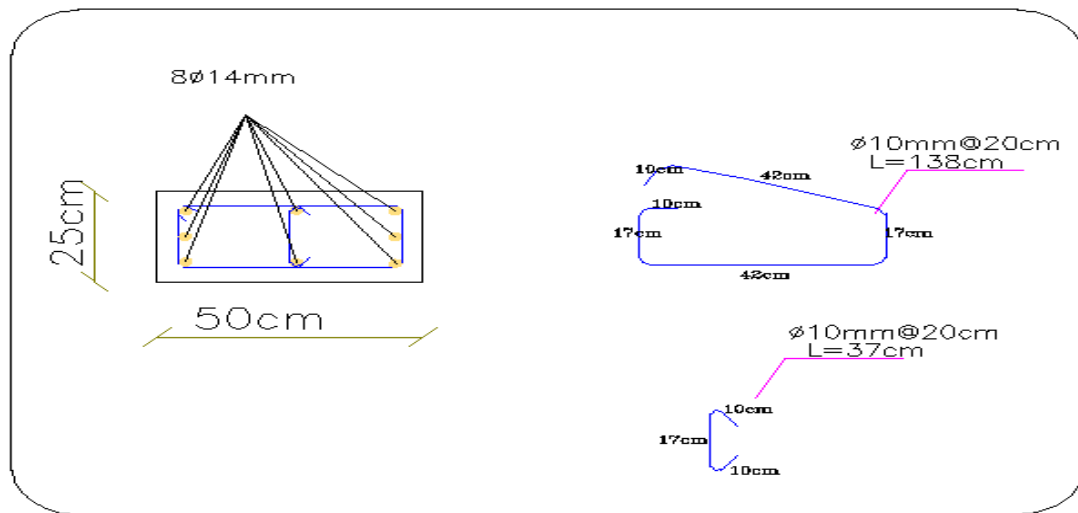


Fig. (4-3) section of column (6).

### 4.9.9 Check for code requirements:

$$\text{clear spacing between longitudinal bars} = \frac{300 - 40 \times 2 - 10 \times 2 - 3 \times 16}{2} = 164 \text{ mm}$$

$$101 \text{ mm} > 40 \text{ mm}$$

$$> 1.5 d_b = 24 \text{ mm .ok}$$

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- gross reinforcement ratio = 0.0169 ,       $0.01 \leq 0.01 < 0.08$       ok
- NO of bars = 12 > 4 bars for square columns.
- Min ties diameter :  $\phi 10$  for  $\phi 32$  longitudinal bars and smaller.

### 4.10 Design of Isolated Footing (F22):-

#### 4.10.1 Determination of Loads:

Total factored load = 1440 KN.

Total services load = 1160 KN

Column Dimensions = 50\*25 cm.

Soil density = 18 KN/m<sup>3</sup>.

Service surcharge = 5KN/m<sup>2</sup>.

Allowable soil Pressure = 400 KN/m<sup>2</sup>.

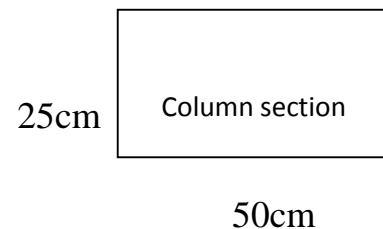
Assume footing to be about (60 cm) thick.

Footing weight =  $25 \times 0.6 = 15$  KN/m<sup>2</sup>.

Soil weight above the footing =  $0.4 \times 18 = 7.2$  KN/m<sup>2</sup>.

$q_{\text{allow}} = 400 - 7.2 - 15 = 377.8$  KN/m<sup>2</sup>

$f_c = 25$  Mpa       $F_y = 420$  Mpa



#### 4.10.2 Determination of Footing Area:

$$A = \frac{1160}{377.8} = 3.07 \text{ m}^2$$

Try 2.0\*1.7 m with area =  $3.4 \text{ m}^2 \geq A_{\text{req}} = 3.07 \text{ m}^2$

Determinate  $q_u = 1440 / 3.4 = 423.53$  KN/m<sup>2</sup>

### 4.10.3 Check for one-way shear strength:-

$$V_u = 423.53 * 2.0 \left( \frac{2.0}{2} - 0.25/2 - d \right)$$

$$\phi V_c = \phi \left( \frac{1}{6} * \sqrt{f_c'} * b_w * d \right)$$

$$\phi V_c = 0.75 * \frac{1}{6} * \sqrt{25} * 2000 * d$$

$$\phi V_c = V_u$$

$$d = 0.353m$$

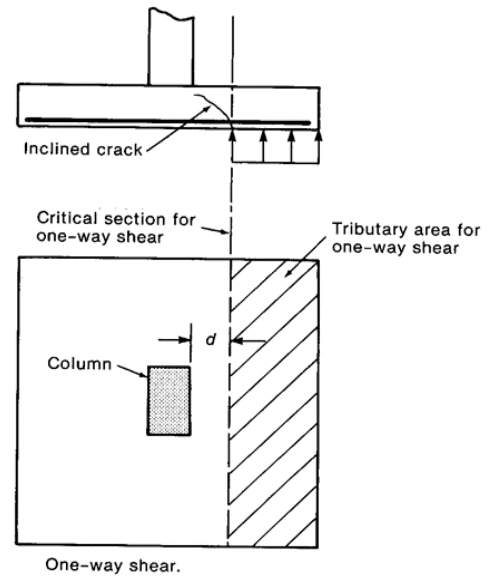


Fig (4-4): One way shear.

### 4.10.4 Determination of the depth of footing based on shear strength:-

Assume,  $\Phi=20\text{mm}$  , cover=75mm

$$H=353+75+20=448\text{mm}$$

Take  $H = 500\text{mm}$

$$d= 500-75-20=405\text{mm}$$

## 4.10.5 Check for two-way shear action (punching):-

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

$$\phi V_c = \phi \cdot \frac{1}{6} \left( 1 + \frac{2}{\beta_c} \right) \sqrt{f'_c} b_o d$$

$$\phi V_c = \phi \cdot \frac{1}{12} \left( \frac{\alpha_s}{b_o/d} + 2 \right) \sqrt{f'_c} b_o d$$

$$\phi V_c = \phi \cdot \frac{1}{3} \sqrt{f'_c} b_o d$$

$$\beta_c = \frac{\text{Column Length (a)}}{\text{Column Width (b)}} = \frac{50}{25} = 2.0$$

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

$$b_o = 2(d + a_1) + 2(d + a_2) = 2(0.405 + 0.25) + 2(0.405 + 0.5) = 3.12m$$

$$\alpha_s = 40 \quad \text{for interior column}$$

$$\phi V_c = \phi \cdot \frac{1}{6} \left( 1 + \frac{2}{\beta_c} \right) = \frac{0.75}{6} * (1 + 2/2.0) = 0.25$$

$$\phi V_c = \phi \cdot \frac{1}{12} \left( \frac{\alpha_s * d}{b_o} + 2 \right) = \frac{0.75}{12} * \left( \frac{40 * 0.405}{3.12} + 2 \right) = 0.45$$

$$\phi V_c = \phi \cdot \frac{1}{3} = \frac{0.75}{3} = 0.25 \dots \dots \dots \text{control}$$

$$\phi V_c = \phi \cdot \frac{1}{3} \sqrt{f'_c} b_o d = 0.25 * \sqrt{25} * 3120 * 405 * 10^{-3} = 1579.5KN$$

$$Vu = 423.53 * \{ (1.7 * 2.0) - (0.25 + 0.405) * (0.5 + 0.405) \} = 1439.4kN$$

$$\phi V_c > Vu_c \dots \dots \dots \text{satisfied}$$

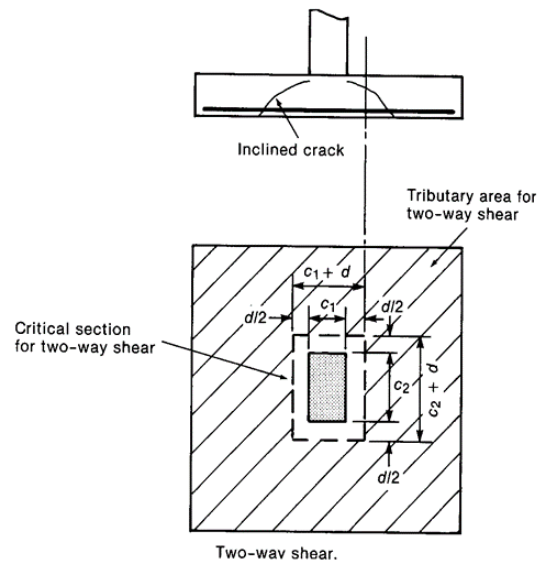


Fig (4-5): Two-way shear

### 4.10.6 Design for Bending Moment:

1) At short direction column "25cm"

Assume,  $\Phi=20\text{mm}$  , cover=75mm

Take  $H = 500\text{mm}$

$B=1.7\text{m}$

$$\begin{aligned} M_u &= \left( q_{ult} \times B \times \left( \frac{L}{2} - \frac{a}{2} \right) \right) \times 0.5 \left( \frac{L}{2} - \frac{a}{2} \right) \\ &= \left( 423.53 \times 1.7 \times \left( \frac{2.0}{2} - \frac{0.25}{2} \right) \right) \times 0.5 \left( \frac{2.0}{2} - \frac{0.25}{2} \right) = 275.6 \text{ KN.m} \end{aligned}$$

$$M_n = 275.6 / 0.9 = 306.2 \text{ KN.m}$$

$$d = 500 - 75 - 20 / 2 = 415 \text{ mm}$$

$$R_n = \frac{M_n}{b * d^2} = \frac{306.2 \times 10^6}{1700 \times 415^2} = 1.046 \text{ Mpa}$$

$$m = \frac{F_y}{0.85 f_c'} = \frac{420}{0.85 \times 25} = 19.76$$

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

$$\rho = \frac{1}{19.76} \left( 1 - \sqrt{1 - \frac{2 \times 19.76 \times 1.046}{420}} \right) = 0.002533$$

$$A_{s_{req}} = 0.002533 \times 1700 \times 415 = 1787.03 \text{ mm}^2$$

$$A_{s_{min}} = 0.0018 * 1700 * 500 = 1530 \text{ mm}^2$$

$$A_{s_{min}} = 1530 \text{ mm}^2 \leq A_{s_{req}} = 1787.03 \text{ mm}^2$$

$$\# \text{ of bar in on meter} = \frac{1787.03}{153.94} = 12$$

Use 13Ø14 with  $A_s = 2001.22 \text{ mm}^2 \geq A_{s_{req}} = 1787.03$

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check for spacing

$$s = \frac{1700 - 2 * 75 - 13 * 14}{12} = 114mm$$

Spacing "s" is the smallest of:

1. 450mm      control
  2.  $3h = 3 * 500 = 1500mm$
- $S = 114 < 450$       o k

2) At long direction column "50cm"

Assume,  $\Phi = 20mm$  , cover = 75mm

Take  $H = 550mm$

$L = 2.0m$

$$\begin{aligned} M_u &= \left( q_{ult} \times B \times \left( \frac{B}{2} - \frac{b}{2} \right) \right) \times 0.5 \left( \frac{B}{2} - \frac{b}{2} \right) \\ &= \left( 423.53 \times 2.0 \times \left( \frac{1.7}{2} - \frac{0.5}{2} \right) \right) \times 0.5 \left( \frac{1.7}{2} - \frac{0.5}{2} \right) = 152.5 \text{ KN.m} \end{aligned}$$

$$M_n = 152.5 / 0.9 = 170 \text{ KN.m}$$

$$d = 500 - 75 - 20 / 2 = 415 \text{ mm}$$

$$R_n = \frac{M_n}{b * d^2} = \frac{152.5 \times 10^6}{2000 \times 415^2} = 0.4427 \text{ Mpa}$$

$$m = \frac{F_y}{0.85 f_c'} = \frac{420}{0.85 \times 25} = 19.76$$

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

$$\rho = \frac{1}{19.76} \left( 1 - \sqrt{1 - \frac{2 \times 19.76 \times 0.4427}{420}} \right) = 0.001065$$

$$A_{s_{req}} = 0.001065 \times 2000 \times 415 = 884.16 \text{ mm}^2$$

$$A_{s_{min}} = 0.0018 * 2000 * 500 = 1800 \text{ mm}^2$$

$$A_{s_{min}} = 1800 \text{ mm}^2 \geq A_{s_{req}} = 884.16 \text{ mm}^2$$

$$\# \text{ of bar in on meter} = \frac{1800}{153.94} = 13$$

Use 13Ø14 with  $A_s = 2001.2 \text{ mm}^2 \geq A_{s_{req}} = 1800 \text{ mm}^2$



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check for spacing :

$$s = \frac{2000 - 2 * 75 - 13 * 14}{12} = 139mm$$

Spacing "s" is the smallest of:

1. 450mm      control
  2.  $3h = 3 * 500 = 1500mm$
- $S = 139 < 450$       o k

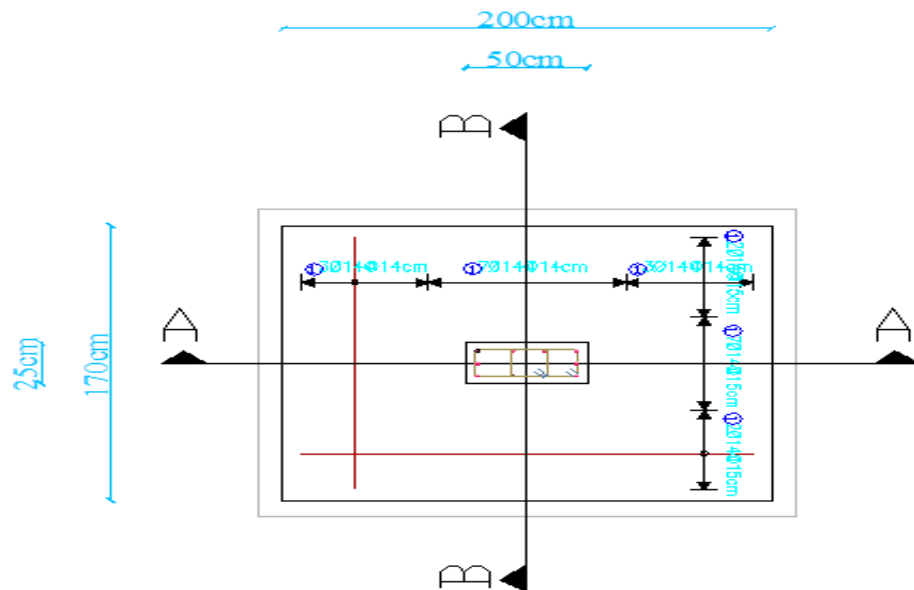


Fig (4-6): Reinforcement of F22.

### 4.10.7 Design of the column –footing joint:

$$P_u = 1440 \text{ KN}$$

$$\phi(0.85 * f_c * A_1) \sqrt{\frac{A_2}{A_1}}$$

Where  $\sqrt{\frac{A_2}{A_1}} \leq 2$  ,  $A_2$  is area lower base

$A_1$  is the area of section column

$$\phi = 0.65$$

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**The allowable bearing on the base of the column is**

$$\phi(0.85 * f_c * A_1) = 0.65 * 0.85 * 25 * 250 * 500 * 10^{-3} = 1726.7 \text{KN}$$

**$f_c$  for column**

**The allowable bearing on the footing is**

$$\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{2.0 * 1.7}{0.25 * 0.5}} = 5.21 \geq 2 \quad \text{control 2}$$

$$\phi(0.85 * f_c * A_1) \sqrt{\frac{A_2}{A_1}} = 0.65 * 0.85 * 25 * 250 * 500 * 2 * 10^{-3} = 3453 \text{KN}$$

**$f_c$  for footing**

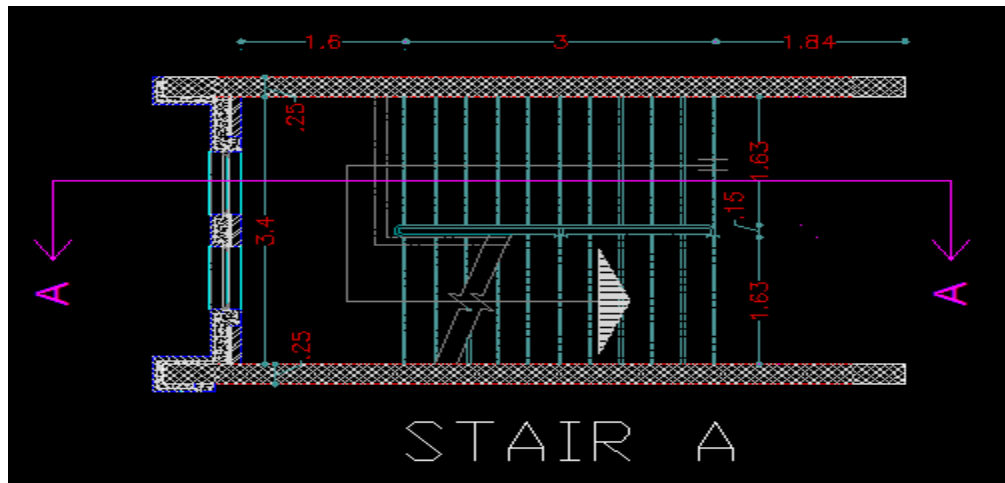
**$P_u = 1440 < 3453$  the dowels are not needed**

**The min area of dowels  $A_{smin} = 0.005 * A_g = 0.005 * 250 * 500 = 625 \text{mm}^2$**

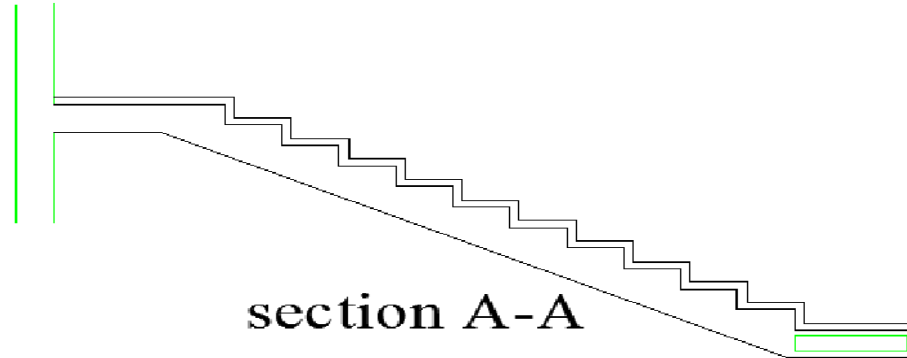
**Select 8 $\phi$ 14**

### 4.11 Design of Stairs:

1-Determination of slab thickness:



**Fig (4.7) Stair**



**Fig (4.8) Section in stair**

$$L = 0.7 + 3 + 0.7 = 4.4 \text{ m.}$$

$$h_{\text{req}} = 4.4 / 20$$

$$h_{\text{req}} = 440 / 20 = 22 \text{ cm} \dots\dots\dots \text{take } h = 25 \text{ cm.}$$

$\Rightarrow$  **Use  $h = 25\text{cm}$ .**

$$\theta = \tan^{-1}(1.56 / 3) = 28.8$$

$$\cos \theta = 0.88$$

**2-Load Calculations at section (A-A):**

**- Load on Flight:**

Dead Load:

$$\text{Tiles} = 0.03 \times 23 \times ((0.35 + 0.15) / 0.30) = 1.15 \text{ KN/m.}$$

$$\text{mortar} = 0.02 \times 22 \times ((0.15 + 0.30) / 0.3) = 0.66 \text{ KN/ m.}$$

$$\text{Plaster} = (0.03 \times 22) / (\cos 28.8) = 0.58 \text{ KN/ m.}$$

$$\text{Steps} = ((0.15 \times 0.3) / 2) \times 25 / 0.3 = 1.875 \text{ KN / m.}$$

$$\text{Slab} = 0.25 \times 25 / \cos 28.8 = 7.13 \text{ KN/ m.}$$

$$\text{Total dead load} = 11.4 \text{ KN/ m.}$$

-

## Chapter 4      Structural Analysis & Design

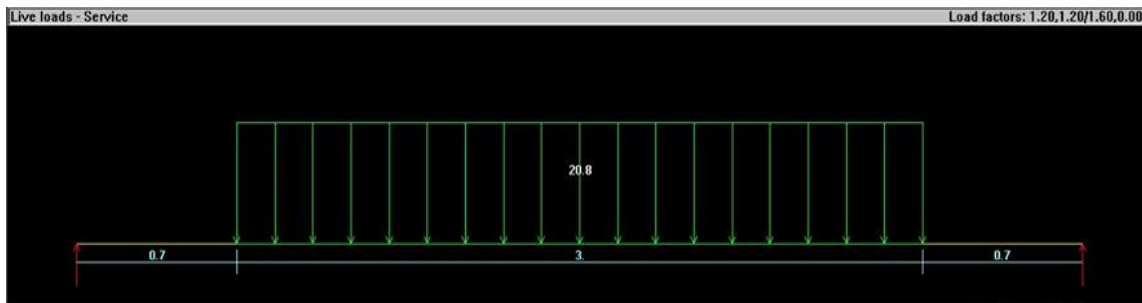
### Load on landing:

Tiles	$= 0.03 \times 23$	$= 0.69 \text{ KN/m.}$
mortar	$= 0.02 \times 22$	$= 0.44 \text{ KN/ m.}$
Plaster	$= 0.03 \times 22$	$= 0.66 \text{ KN/ m.}$
Slab	$= 0.25 \times 25$	$= 6.25 \text{ KN/ m.}$
Total dead load		$= 8.04 \text{ KN/ m.}$

### Live load:

Live load for stairs  $= 4.5 \text{ KN/ m}^2$ .

**on Flight**  $Q_u = 1.2 \times 11.4 + 1.6 \times 4.5 = 20.88 \text{ KN/m.}$



### 3 - Design for Shear :

- Assume  $\emptyset 14$  for main reinforcement:-

So,  $d = 250 - 20 - 14/2 = 223 \text{ mm}$

#### Shear

**Support reactions at B&A = 31.32 KN**

$V_u = 31.32 \text{ KN.}$

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

$$\phi V_c = \frac{0.75 * \sqrt{25} * 1000 * 223}{6} = 139.38 \text{ KN}$$

$V_u = 32.32 \text{ KN} < \phi V_c = 139.38 \text{ KN.}$

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

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### 4 -Design for Bending Moment :

The Following figure shows the Moment Envelope acting on the stair

$$M_u = (1.5 \times 0.75 \times 20.88) - (31.32 \times 2.2) = 45.4 \text{ kN.m}$$

$$M_u = 45.4 \text{ kN.m}$$

$$M_n = M_u / 0.9 = 45.4 / 0.9 = 50.4 \text{ KN.m.}$$

$$d = 223 \text{ mm.}$$

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

$$K_n = \frac{50.4 \times 10^6}{1000 \times 223^2} = 1.014 \text{ MPa .}$$

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

$$m = \frac{420}{0.85 \times 25} = 19.76$$

$$\rho = \frac{1}{m} \left( 1 - \sqrt{1 - \frac{2mK_n}{f_y}} \right) = \frac{1}{19.76} \left( 1 - \sqrt{1 - \frac{2 \times 19.76 \times 1.014}{420}} \right) = 2.475 \times 10^{-3}$$

$$A_{s_{req}} = 2.475 \times 10^{-3} \times 1000 \times 223 = 552 \text{ mm}^2$$

$$A_{s_{min}} = 0.0018 \times b \times h = 0.0018 \times 1000 \times 250 = 450 \text{ mm}^2$$

$$A_{s_{min}} = 450 \text{ mm}^2 \leq A_{s_{req}} = 552 \text{ mm}^2$$

$$\text{Use } \Phi 12 \gg 552/113 = 4.89$$

$$\text{Use } \Phi 12 @ 20 \text{ cm c/c } \dots\dots\dots$$

$$A_s \text{ provided} = 565 > A_s \text{ req.} \dots\dots\dots \text{OK.}$$

### 5 - Secondary reinforcement:

$$A_{s_{Shrinkage}} = 0.0018 \times b \times h = 0.0018 \times 1000 \times 200 = 360 \text{ mm}^2$$

$$\text{Use } \Phi 12 @ 250 \text{ mm } \dots\dots\dots$$

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### Design of landing (1):

- Load on landing :

Dead Load:

#### - Load on landing:

$$\text{Tiles} = 0.03 \times 23 = 0.69 \text{ KN/m.}$$

$$\text{mortar} = 0.02 \times 22 = 0.44 \text{ KN/ m.}$$

$$\text{Plaster} = 0.03 \times 22 = 0.66 \text{ KN/ m.}$$

$$\text{Slab} = 0.25 \times 25 = 6.25 \text{ KN/ m.}$$

$$\text{Total dead load} = 8.04 \text{ KN/ m.}$$

Live load:

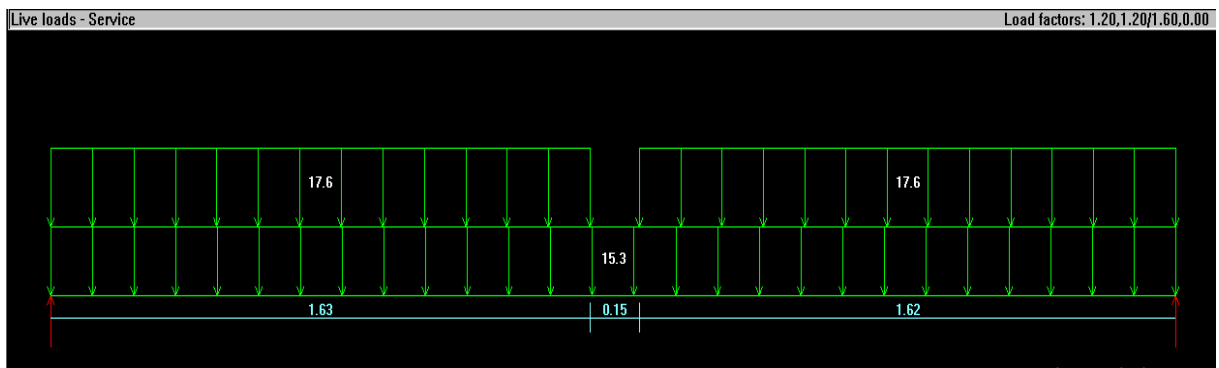
$$\text{Live load for stairs} = 4.5 \text{ KN/ m}^2.$$

$$\text{Load from flight} = (31.32) \text{ KN}$$

$$\text{on landing } Q_u = 1.2 \times 8.04 + 1.6 \times 4.5 = 16.85 \text{ KN/m.}$$

$$\& (31.32/1.63) = 19.21 \text{ KN/m}$$

$$\& (31.32/1.63) = 19.21 \text{ KN/m}$$



### -2 Design of Shear :

▪ Assume  $\varnothing 14$  for main reinforcement:-

$$\text{So, } d = 250 - 20 - 7 = 223 \text{ mm.}$$

$$\text{Support reaction at B \& A} = 60 \text{ KN}$$

$$V_u = 60 \text{ KN.}$$

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

$$\phi V_c = \frac{0.75 * \sqrt{25} * 1000 * 223}{6} = 139.38 \text{ KN}$$

$$V_u = 60 \text{ KN} < \phi V_c = 139.38 \text{ KN} .$$

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

### -3 Design for Bending Moment :

The Following figure shows the Moment Envelope acting on the stair

$$M_u = (16.85 * 1.7 * 0.85) + (19.21 * 1.63 * 0.89) - (60 * 1.7) = 49.8 \text{ kN.m}$$

$$M_u = 49.8 \text{ kN.m} .$$

$$= M_u / 0.9 = 49.8 / 0.9 = 55.3 \text{ KN.m.}$$

$$d = 223 \text{ mm.}$$

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

$$K_n = \frac{55.3 * 10^6}{1000 * 223^2} = 1.112 \text{ MPa} .$$

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

$$m = \frac{420}{0.85 \times 25} = 19.76$$

$$\rho = \frac{1}{m} \left( 1 - \sqrt{1 - \frac{2mK_n}{f_y}} \right) = \frac{1}{19.76} \left( 1 - \sqrt{1 - \frac{2 * 19.76 * 1.112}{420}} \right) = 2.72 * 10^{-3}$$

$$A_{s_{req}} = 2.72 * 10^{-3} * 1000 * 223 = 606.73 \text{ mm}^2 .$$

$$A_{s_{min}} = 0.0018 * b * h = 0.0018 * 1000 * 250 = 450 \text{ mm}^2$$

$$A_{s_{min}} = 450 \text{ mm}^2 \leq A_{s_{req}} = 606.73 \text{ mm}^2$$

Use  $\Phi 14 \setminus 20 \text{ cm}$

.As provided =  $153.9 \setminus 0.2 = 770 > A_{s_{req}}$ .....OK.

Cheek spacing :

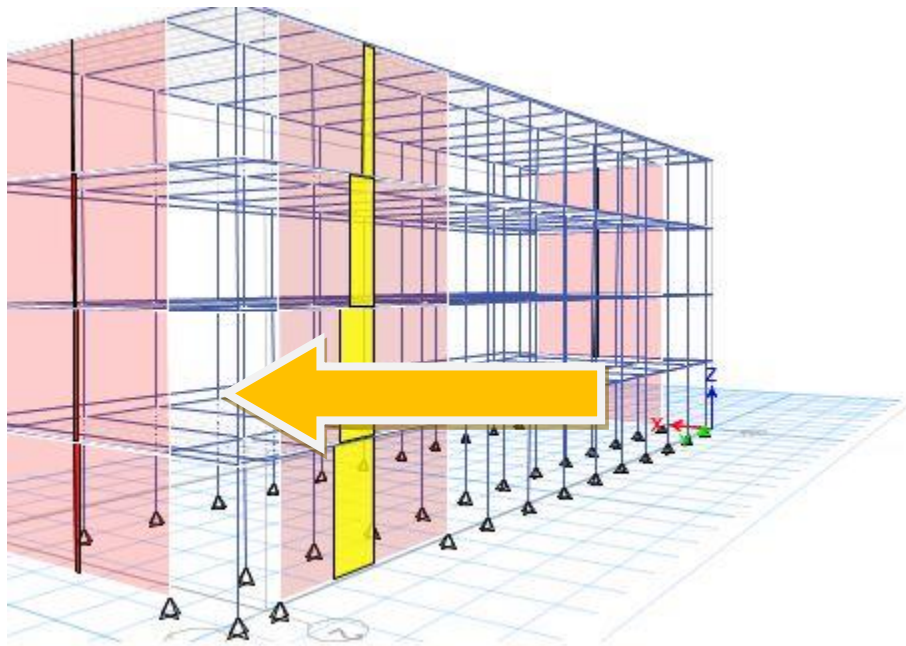
$$3h = 3 \times 250 = 750 \text{ mm}$$

$$450$$

$$= 380(280 \sqrt[3]{2} \times 420) - 2.5 \times 20 = 330$$

$$= 300(280 \sqrt[3]{2} \times 420) = 300 \dots \dots \text{control}$$

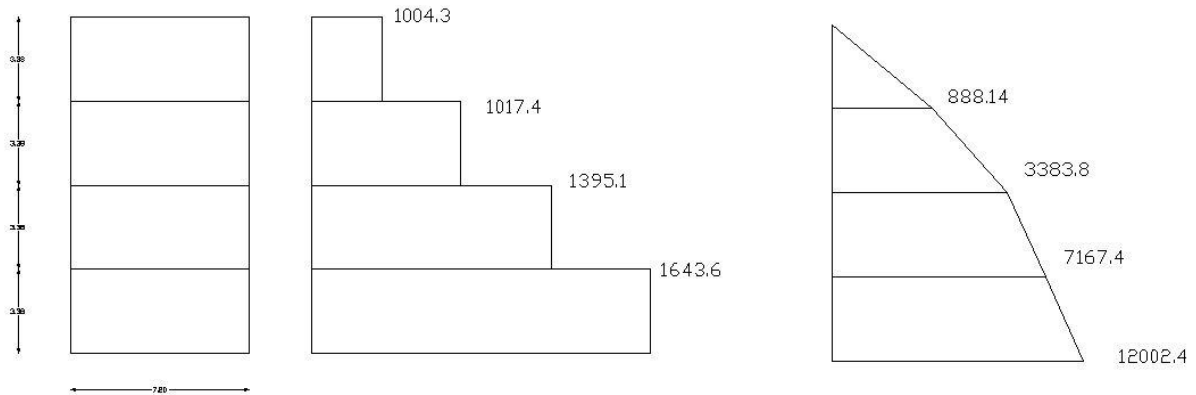
### 4.12 Design of shear wall:



**Fig (4.9) Place of shear wall**



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$$F_c = 28 \text{ MPa}$$

$$F_y = 420 \text{ MPa}$$

$t = 25 \text{ cm}$  .shear wall thickness

$L_w = 7.2 \text{ m}$  .shear wall width

$H_w$  for one wall = 3.38 m story height

### Design for shear

$$\sum F_x = V_u = 1643.6 \text{ KN}$$

### **-Design horizontal reinforcement:**

The critical Section is the smaller of:

$$\frac{l_w}{2} = \frac{7.2}{2} = 3.6 \text{ m} \dots \text{control}$$

$$\frac{h_w}{2} = \frac{13.52}{2} = 6.76 \text{ m}$$

$$\text{story height } (H_w) = 3.38 \text{ m}$$

$$d = 0.8 \times l_w = 0.8 \times 7.2 = 5.76 \text{ m}$$

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$V_c$  is the smallest of :

$$\begin{aligned}
 1 - V_c &= \frac{1}{6} \sqrt{f_c'} h d = \frac{1}{6} \sqrt{28} * 250 * 5760 = 1270 \text{ KN} \dots \text{cont} \\
 2 - V_c &= 0.25 \sqrt{f_c'} h d + \frac{N_u d}{4 l_w} = 0.25 \sqrt{28} * 250 * 5760 + 0 = 1905 \text{ KN} \\
 3 - V_c &= \left[ 0.5 \sqrt{f_c'} + \frac{l_w \left( \sqrt{f_c'} + 2 \frac{N_u}{l_w h} \right)}{\frac{M_u}{V_u} - \frac{l_w}{2}} \right] 0.1 * h d = \left[ 0.5 \sqrt{28} + \frac{7.2 (\sqrt{28} + 0)}{3.702} \right] 250 * 5760 * 0.1 \\
 &= 1863 \text{ KN}
 \end{aligned}$$

$V_u = 1643.6 \text{ KN} < 0.75 * 1270 = 952.5 \text{ KN}$       **horizontal reinforcement is required.**

$$\begin{aligned}
 \phi v_c + \phi v_s &= v_u \\
 v_s &= \frac{v_u}{\phi} - v_c = \frac{1643.6}{0.75} - 1270 = 921.4667 \text{ KN}
 \end{aligned}$$

**Will be carried for horizontal reinforcement**

$$\begin{aligned}
 \frac{A_{vh}}{s} &= \frac{v_s}{f_y * d} = \frac{921.466 * 10^3}{420 * 5760} = 0.380 \\
 \left( \frac{A_{vh}}{s} \right)_{min} &= 0.0025 * 250 = 0.625 \dots \text{cont}
 \end{aligned}$$

$$\begin{aligned}
 S_{max} &= \frac{l_w}{5} = \frac{3380}{5} = 676 \text{ mm} \\
 S_{max} &= 3 * h = 3 * 250 = 750 \text{ mm}
 \end{aligned}$$

$A_{vh}$  = for two layers of horizontal reinforcement.  
(Horizontal reinforcement with two legs).

Select  $\phi 10$ :

$$A_{vh} = 2 * \frac{\pi * 10^2}{4} = 157 \text{ mm}^2$$

$$\frac{A_{vh}}{S_{req}} = 0.625$$

$$S_{req} = \frac{157}{0.625} = 251.2 \text{ mm}$$

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$s = 250\text{mm} < s_{max} = 600\text{mm}$ .  
select  $\phi 10 @ 250\text{mm}$  at each side.

- Minimum shear reinforcements required:

Take  $\rho = 0.0025$ .

- Maximum spacing is the least of :

$$\frac{L_w}{5} = \frac{7200}{5} = 1440\text{mm}$$

$$3 \cdot h = 3 \cdot 250 = 750\text{mm}$$

**Try  $\phi 10$  ( $A_s = 78.5 \text{ mm}^2$ ) for two layers**

$$\rho = \frac{A_v h}{h \cdot S_2} = \frac{2 \cdot 78.5}{250 \cdot S_2} = 0.0025$$

$$S_2 = 251.2 \text{ mm} , \quad \phi 10 @ 250 \text{ mm}$$

**-Design for Vertical reinforcement:-**

$$\frac{h_w}{L_w} = \frac{13.52}{7.2} = 1.878$$

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

$$\rho_v = \mathbf{0.0025}$$

**Select  $\Phi 12 @ 200\text{mm}$ . In two layer**

- Maximum spacing is the least of :

$$\frac{L_w}{3} = \frac{7200}{3} = 2400\text{mm}$$

$$3 \cdot h = 3 \cdot 250 = 750\text{mm}$$

$$S_{rq} = \frac{2 \cdot 113.09}{0.764} = 296.05\text{mm} = 29.6\text{cm} .$$

$$s = 200\text{mm} < s_{max} = 600\text{mm}.$$

Select  $\Phi 12 @ 200\text{mm}$  in two layer.

**-Design for bending moment (uniformly distribution flexural reinforcement) :**

$$M_u = 12002.37 \text{ KN.m}$$

$$A_{st} = \left( \frac{7200}{200} \right) * 2 * 113.09 = 8142.5 \text{ mm}^2$$

$$w = \left( \frac{A_{st}}{L_w h} \right) \frac{f_y}{f'_c} = \left( \frac{8142.5}{7200 * 250} \right) \frac{420}{28} = 0.0678$$

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

$$\frac{C}{l_w} = \frac{w + \alpha}{2w + 0.85\beta_1} = \frac{0.0678 + 0}{2 * 0.0678 + 0.85 * 0.85} = 0.07901$$

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

$$= 0.9 [0.5 * 12002.37 * 420 * 7200 (1 + 0) (1 - 0.07901)] = 15042.37 \text{ KN.m} > M_u \dots \text{No need}$$

### 4-12: Design of Strip Footing Under Shear Wall:

#### Loads from Shear Wall

- DL = 1600 Service Dead Load.
- LL = 38.5 Service Live Load.
- DL (Factored/m) = DL/Lw = 1920/ 8= 240KN/m
- LL (Factored/m) = LL/Lw = 61.6/8 = 7.7KN/m
- DL (Service/m) = DL/Lw = 1600/8 = 200KN/m
- LL (Service/m) = LL/Lw = 38.5/8 = 4.813 KN/m

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$$\text{Total Factored / m} = 240\text{KN/m} + 7.7\text{KN/m} = 247.7\text{KN/m}$$

- Service Surcharge =  $5.5 \text{ KN/m}^2$
- Allowable soil pressure =  $400 \text{ KN/ m}^2$
- Soil density =  $18 \text{ KN/m}^3$

Try 50cm thickness:

$$q_{a,net} = 400 - 0.5 \times 25 - 5.5 = 382 \text{KN/m}^2 \quad ,$$

and:

$$A = \frac{P_u}{q_{a,net}} = \frac{200 + 4.183}{382} = 0.6 \text{ m}^2/\text{m} \quad , \text{ length of the wall}$$

$$A = b \times 1\text{m} \dots\dots b = 0.6 \text{ m take } b = 1.00 \text{ m}$$

**Take it (1 m × 1 m)**

### ✓ Design of footing and shear design (s1):

$$P_u = 1.2 \times 222.2 + 1.6 \times 5.3 = 275.12 \text{ KN/m}$$

$$q_{ult} = \frac{P_u}{A} = \frac{275.12}{1 \times 1} = 275.12 \text{ KN/m}^2.$$

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### Check for One Way Shear Strength:

$$V_u = \left( \frac{l}{2} - \frac{a}{2} - d \right) * q_u * 1m = \left( \frac{1.00}{2} - \frac{0.25}{2} - d \right) * 275.12 * 1$$

$$\phi V_c = \frac{0.75}{6} \sqrt{25} * 1000 * d$$

$$\text{Let, } \phi V_c = V_u$$

$$d = 0.115m$$

assume

$$h = 250 + 75 + 20/2 = 335mm$$

$$h = 400 \text{ mm, } d = 400 - 75 - 20/2 = 315 \text{ mm}$$

### ✓ Design for Bending Moment of long direction:

h (mm)	d (mm)	b(mm)
400	315	1000

Table (4-6)

$$M_u = 275.12 * 1 * 0.85 * 0.85 / 2 = 99.4 \text{ KN.m/m}$$

$$m = \frac{f_y}{0.85 * f_c'} = \frac{420}{0.85 * 25} = 19.76$$

$$R_n = \frac{M_u / \phi}{b * d^2} = \frac{99.4 * 10^6 / 0.9}{1000 * (315)^2} = 1.113 \text{ Mpa}$$

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

$$\rho = \frac{1}{19.76} \left( 1 - \sqrt{1 - \frac{2(19.76)(1.113)}{420}} \right) = 0.00272$$

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$$A_{s_{req}} = 0.00272 (1000) (315) = 952 \text{ mm}^2/\text{m} > A_{s_{min}} = 720 \text{ mm}^2/\text{m} \dots \text{OK}$$

$$A_{s_{min}} = 0.0018 * b * h == 0.0018 (1000) (400) = 720 \text{ mm}^2/\text{m}$$

Try  $\Phi$  16:

$$n = 952/201 = 5 \quad , \quad S = \frac{1}{n} = \frac{1}{5} = 0.2 \text{ m}$$

Take 5  $\Phi$  16 @  $A_s = 1005 \text{ cm}^2 > A_{s_{req}} = 952 \text{ cm}^2$

Use  $\Phi$  16 @ 20cm

Mu(KN.m)	m	Rn	$\rho$	$A_{s_{req}}(\text{mm}^2)$	$A_{s_{min}}(\text{mm}^2)$	S(mm)
99.4	19.76	1.113Mpa	0.00272	952	720	200

Table (4-7)

- Step (s) is the smallest of :-

$$1) 3 * h = 3 * 400 = 1200 \text{ mm}$$

$$2) 450 \text{ mm} \dots \text{Control}$$

$$S = 200 \text{ mm} < S_{,max} = 450 \text{ mm} \dots \text{OK}$$

- Select the minimum temperature reinforcement.

$$A_{s_{min}} = 0.0018 * b * h == 0.0018 (1000) (400) = 720 \text{ mm}^2/\text{m}$$

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**- The maximum spacing (s) is:-**

$$1. 5 \cdot h = 5 \cdot 400 = 2000 \text{ mm}$$

2. 450 mm.... control

$$n = 720/154 = 4.68 \quad , \quad S = \frac{1}{5} = 0.2\text{m}$$

**Use  $\Phi$  14 @ 20 cm**