4

Chapter Four

Structural Analysis and Design

4-1 | **Introduction**

Reinforced concrete (RC) is a versatile composite and one of the most widely used materials in modern construction. Concrete is a relatively brittle material that is strong under compression but less so in tension. Plain, unreinforced concrete is unsuitable for many structures as it is relatively poor at with standing stresses induced by vibrations, wind loading and so on.

To increase its overall strength, steel rods, wires, mesh or cables can be embedded in concrete before it sets. This reinforcement, often known as rebar, resists tensile forces. By forming a strong bond together, the two materials are able to resist a variety of applied forces, effectively acting as a single structural element.

Reinforced concrete can be precast or cast-in-place (in situ) concrete, and is used in a wide range of applications such as; slab, wall, beam, column, foundation, and frame construction.

4-1-1 Concrete and its Classifications:

Plain concrete is made by mixing cement, fine aggregate, coarse aggregate, water, and frequently admixtures**,** **Structural concrete can be classified into:**

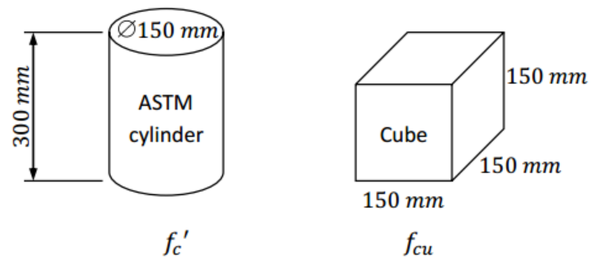
- Lightweight concrete with a unit weight from about $1350 \text{ to } 1850\ (\frac{kg}{m^3})$ produced from aggregates of expanded shale, clay, slate, and slag.
- Normal-weight concrete with a unit weight from about $1800 \text{ to } 2400\ (\frac{kg}{m^3})$ produced from the most commonly used aggregates— sand, gravel, crushed stone.
- Heavyweight concrete with a unit weight from about $3200 \text{ to } 5600\ (\frac{kg}{m^3})$ produced from such materials such as barite, limonite, magnetite, ilmenite, hematite, iron, and steel punching or shot. It is used for shielding against radiations in nuclear reactor containers and other structures.

4-1-2 Compressive strength of concrete:

The strength of concrete is controlled by the proportioning of cement, coarse and fine aggregates, water, and various admixtures. The most important variable is (w/c) ratio.

Concrete strength ($f_c{}'$) – uniaxial compressive strength measured by a compression test of a standard test cylinder ($150\ mm\ diameter\ \ by\ 300\ mm\ high$) on the 28th day–ASTM C31, C39. In many countries, the standard test unit is the cube ($200\ x\ 200\ x\ 200\ mm$).

The concrete strength depends on the size and shape of the test specimen and the manner of testing. For this reason the cylinder ($\emptyset\ 150mm\ by\ 300\ mm\ high$) strength is 80% of the $150\ mm$ cube strength and 83% of the $150\ mm$ cube strength, **figure (4-1)** demonstrate relation between cylinder and cube concrete test.

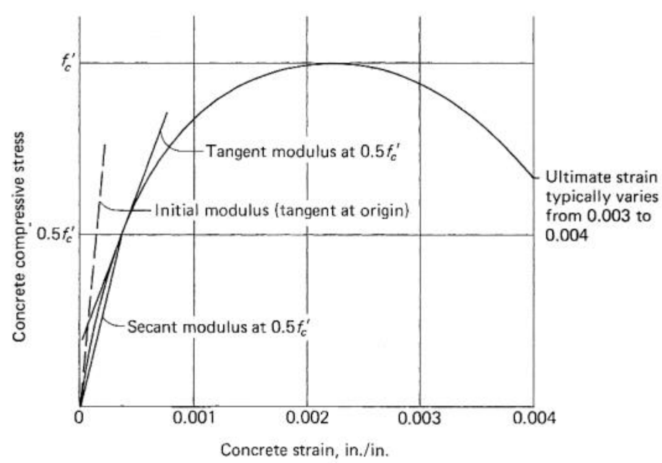


$$f_c{}' \approx 0.80\ f_{cu}$$

<u>Figure (4-1)</u> relation between cylinder and cube concrete test.

<u>4-1-3 Modulus of Elasticity of concrete:</u>

The modulus of elasticity of concrete varies, unlike that of steel, with strength. A typical stress-strain curve for concrete in compression is shown. The initial modulus (tangent at origin), the tangent modulus (at $0.5\ f_c{}'$), and the secant modulus are noted. Usually the secant modulus at from $25\ to\ 50\%$ of the compressive strength $f_c{}'$ is considered to be the modulus of elasticity. For normal weight concrete, shall be permitted to be taken as $E_c = 4700\sqrt{f_c{}'}$ (Map), **figure (4-2)** demonstrate stress-strain curve of concrete.



<u>Figure (4-2)</u> stress-strain curve of concrete.

4-1-4 Strength Design method (Ultimate strength method):

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

The strength design method may be expressed by the following:

Strength provided ≥ [strength required to carry factored loads]

Where the "strength provided" (such as moment strength) is computed in accordance with the provisions of a building code, and the "strength required" is that obtained by performing a structural analysis using factored loads.

4-1-5 Load Factors U and strength reduction Factor ∅ :

According to (ACI 318-11 9.2.1) the factor U for overload is given:

$$\boldsymbol{U = 1.4D}$$
$$\boldsymbol{U = 1.2D + 1.6L + 0.5\,(L_r\ or\ S\ or\ R)}$$
$$\boldsymbol{U = 1.2D + 1.6L + 0.5\,(L_r\ or\ S\ or\ R)}$$
$$\boldsymbol{U = 1.2D + 1.0W + 1.0\,L + 0.5(L_r\ or\ S\ or\ R)}$$
$$\boldsymbol{U = 1.2D + 1.0E + 1.0\,L + 0.2\,S}$$
$$\boldsymbol{U = 0.9D + 1.0W}$$
$$\boldsymbol{U = 0.9D + 1.0E}$$

Where:

$\boldsymbol{D}$: ***dead load .***
$\boldsymbol{L}$: ***live load.***
$\boldsymbol{L_r}$: ***roof live load.***
$\boldsymbol{S}$: ***snow load.***
$\boldsymbol{R}$: ***rain load.***
$\boldsymbol{W}$: ***Wind load.***
$\boldsymbol{E}$: *Earthquake load.*

The factor ∅ (under strength factor) **according to ACI demonstrated in figure (4-3).**

Strength Condition	ϕ Factors
1. Flexure (with or without axial force)	
Tension-controlled sections	0.90
Compression-controlled sections	
Spirally reinforced	0.75
Others	0.65
2. Shear and torsion	0.75
3. Bearing on concrete	0.65
4. Post-tensioned anchorage zones	0.85
5. Struts, ties, nodal zones, and bearing areas in strut-and-tie models	0.75

Figure (4-3) values of understrength factors related to strength condition.

4-1-6 General considerations:

1- **ACI 318-11 Building code will be used in this project.**
2- **UBC-97 code will be used for lateral loads.**
3- **Ultimate strength design method will be used during the analysis and design of this project.**
4- **The compressive strength of concrete for all structural elements is B300 which equals to** $f_c{}' = 24\ Mpa$.
5- **Yield strength of reinforcing rebar's** $f_y = 420\ Mpa$.

4-2 | Check of Minimum Thickness of Structural Member:

It will be determined according to (ACI 318-11) to achieve deflection requirements, Figure (4-4) provided minimum thickness from code.

TABLE 9.5(a) — MINIMUM THICKNESS OF NONPRESTRESSED BEAMS OR ONE-WAY SLABS UNLESS DEFLECTIONS ARE CALCULATED

	Minimum thickness, *h*			
	Simply supported	One end continuous	Both ends continuous	Cantilever
Member	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections			
Solid one-way slabs	**ℓ/20**	**ℓ/24**	**ℓ/28**	**ℓ/10**
Beams or ribbed one-way slabs	**ℓ/16**	**ℓ/18.5**	**ℓ/21**	**ℓ/8**

Notes:
Values given shall be used directly for members with normalweight concrete and Grade 420 reinforcement. For other conditions, the values shall be modified as follows:
a) For lightweight concrete having equilibrium density, w_c, in the range of 1440 to 1840 kg/m^3, the values shall be multiplied by $(1.65 - 0.0003w_c)$ but not less than 1.09.
b) For f_y other than 420 MPa, the values shall be multiplied by $(0.4 + f_y/700)$.

We take the longest beam and rib, then we compare between them in table (4-1).

Supporting type	min. thickness equation	Rib &Beam	No. of span	min. thickness
One end continuous	$\frac{L}{18.5}$	$B-R1$	6	$\frac{5.05}{18.5}=27\ cm$
Both end continuous	$\frac{L}{21}$	$B,B7$	6	$\frac{7.53}{21}=35\ cm$

Table (4-1) Determination of thickness for ribs and beams from maximum values of cases.

The thickness of slab provided from (ACI 318-11) to achieve requirements of deflection, depends on the Flexural stiff ness of slab, by manual calculation comes about

$\boldsymbol{h_{min}=35\ cm}$.

So, select **<u>Slab thickness</u>** $\boldsymbol{h=35\ cm\ (27\ cm\ Hollow\ Block+8\ cm\ \ Topping)}$.

4-3 | Design of Topping:

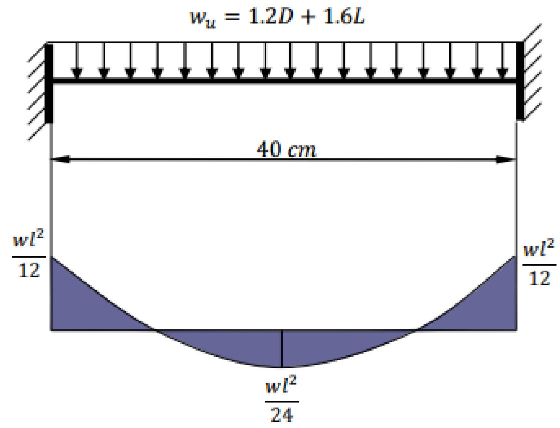
<u>4-3-1 Load calculations:</u>

Topping in One way ribbed slab can be considered as a strip of **1 meter width** and span of hollow block length with both end fixed in the ribs**, Table (4-2)** shows Load calculations on topping.

Dead Load Form	Thickness $\boldsymbol{\delta}$ $(\boldsymbol{m})$	Unit weight $\boldsymbol{\gamma}$ $(\frac{kN}{m^3})$	$\boldsymbol{\gamma\times\delta\times1}$ $(\frac{kN}{m})$
Tiles	0.03	23	$\mathbf{0.03\times23=0.69}$
Mortar	0.03	22	$\mathbf{0.03\times22=0.66}$
Coarse Sand	0.07	17	$\mathbf{0.07\times17=1.19}$
Topping	0.08	25	$\mathbf{0.08\times25=2}$
$\sum \boldsymbol{Dead\ loads}$			$\mathbf{4.54}$ $(\frac{kN}{m})$

Table (4-2) Dead Load calculations on topping.

Live load calculations $= 4 \times 1 = 4 \ (\frac{kN}{m})$



4-3-2 Factored Load:

Total Factored Load:

$$w_u = 1.2\,(4.54) + 1.6(4) = 11.85 \ (\frac{kN}{m})$$

Figure (4-5) Topping statically system.

$$M_u = \frac{w_u L^2}{12} = \frac{11.85 \ \times 0.4^2}{12} = 0.158 \ kN.m\,..for\ (1m) of\ strip\ width$$

Strength condition for plain concrete:

$\emptyset M_n \geq M_u$ (ACI 22.5.1) .... where $\emptyset = 0.55$

$M_n = 0.42\,\lambda \sqrt{f_c{}' \ S_m}$ Where S_m for rectangular section of the slab : $S_m = \frac{bh^2}{6} = \frac{1000 \times 80^2}{6}$

$$M_n = 0.42 \times \sqrt{24} \times \frac{1000 \times 80^2}{6} = 2.19 \ kN.m$$

$$0.55 \times 2.19 = 1.2 \ kN.m \gg M_u = 0.158 \ kN.m$$

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

$\rho_{\text{shrinkage}} = 0.0018$ According to (ACI 7.12.2.1)

$$A_s = \rho bt = 0.0018 \ \times 1000 \times 80 = 144 \ mm^2 \quad for\ 1m\ strip$$

Step (s,max) is the smallest of:

1. 3h = 3×80 =240 mm **control**
2. 450mm.
3. S $= 380\left(\frac{280}{f_s}\right) - 2.5C_c = 380\left(\frac{280}{\frac{2}{3}420}\right) - 2.5\,.20 = 330\text{mm}$

Take $\emptyset$ 8 @ 200 mm in both direction, $S = 200$ mm $< S_{max} = 240\ mm\,..OK$

4-4 | Design of One Way Rib Slab

4-4-1 Determination of geometry:

Requirements for Ribbed Slab (T-Beam Consideration According to ACI- 318-11).

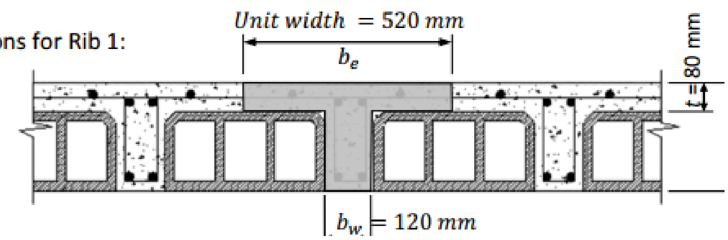
$\mathbf{b}_w \geq 10\ cm \ldots .select\ \ \mathbf{b}_w = 12\ cm$

$\mathbf{h} \leq 3.5\mathbf{b}_w = 3.5 \times 12 = 42cm \ldots select\ \ h = 35\ cm$

$\mathbf{t}_f \geq \frac{L_n}{12} \geq 50\ mm \ldots .select\ \mathbf{t}_f = 8cm$

Figure (4-6) shows typical section of rib with overall slab thickness $h = 35cm$.

➢ Load Calculations for Rib 1:



4-4-2 Load calculations for Rib:

$f_c{}' = 24\ Mpa\ \ ,\ f_y = 420\ Mpa\ \ ,L_l = 2\ (\frac{kN}{m^2})$

Dead Load Form	Thickness δ (m)	Unit weight γ $(\frac{kN}{m^3})$	$\gamma \times \delta \times 1\ \ (\frac{kN}{m})$
Tiles	0.03	23	$0.03 \times 23 \times 0.52 = 0.359$
Mortar	0.03	22	$0.03 \times 22 \times 0.52 = 0.343$
Coarse Sand	0.07	17	$0.07 \times 17 \times 0.52 = 0.619$
Topping	0.08	25	$0.08 \times 25 \times 0.52 = 1.04$
RC Rib	0.27	25	$0.27 \times 25 \times 0.12 = 0.81$
Hollow Block	0.27	10	$0.27 \times 10 \times 0.40 = 1.08$
Plaster	0.03	22	$0.03 \times 22 \times 0.52 = 0.343$
$\sum$ ***Dead loads*** **/rib**			$4.59\ \ (\frac{kN}{m})$

Table (4-3) Dead Load calculations of rib.

Live load calculations $= \mathbf{4 \times 0.52 = 2.08}$ $\left(\frac{kN}{m}\right)$ **/ rib**

Figure (4-7) shows the location of rib in slab drawing plan**.**

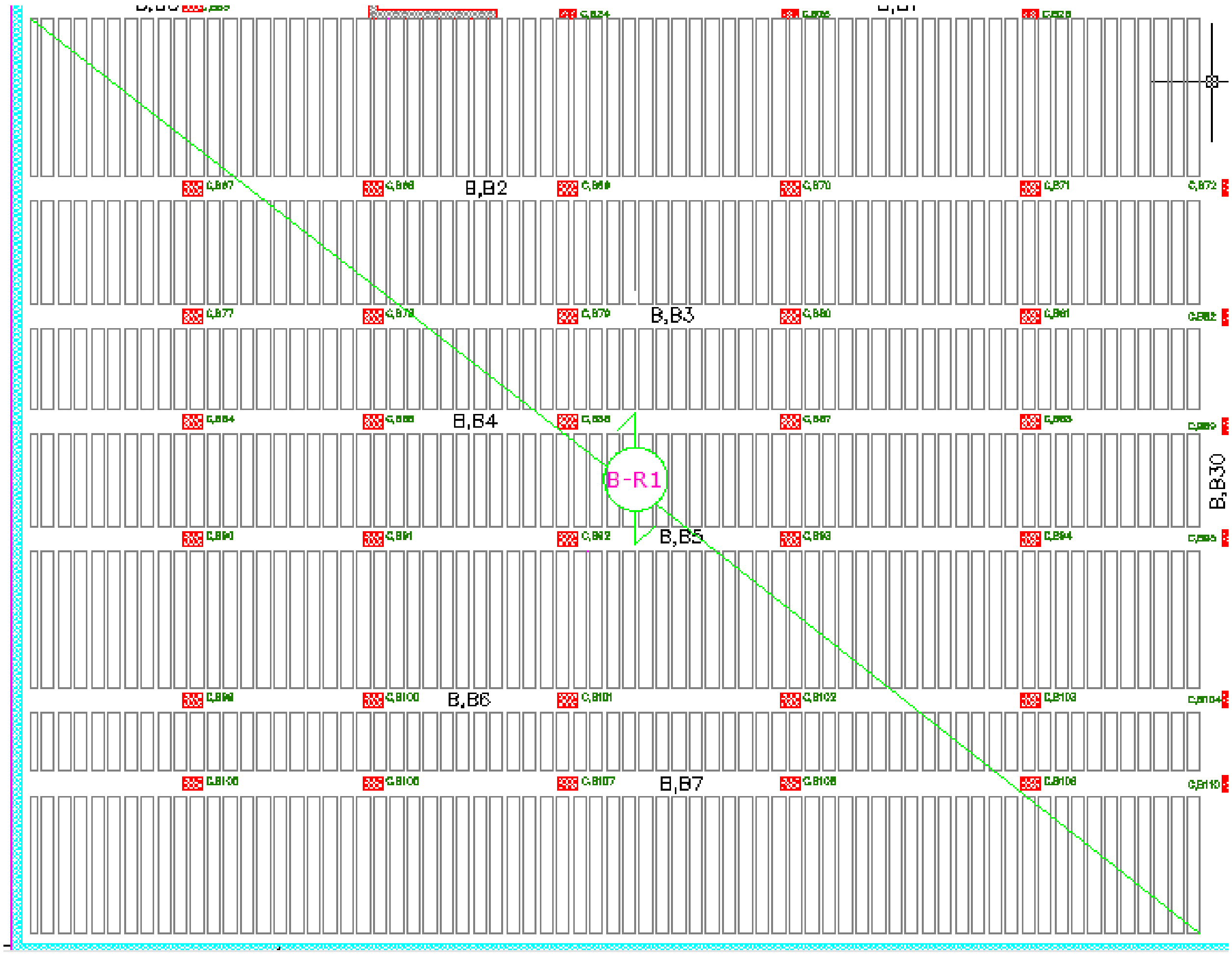


Figure (4-7) location of Rib (B-R1) in slab drawing plan.

Check for chosen effective flange width (b_e) According to (ACI 318-11)

$\boldsymbol{b_e}$ ***is the smallest of*** :

(1) $b_e \le \frac{L}{4} = \frac{1820}{4} = \mathbf{455\ mm}\ ..\boldsymbol{OK}$ ....*where* (L)*is the smallest clear span of the rib.*

(2) $b_e \le b_w + 16h_f = 120 + 16(80) = 1400\ mm.$

(3) $b_e \le$ *center to center between adjacent beams* $= 400 + 120 = 520$ mm.

4-4-3 Structural analysis of Rib (B-R1):

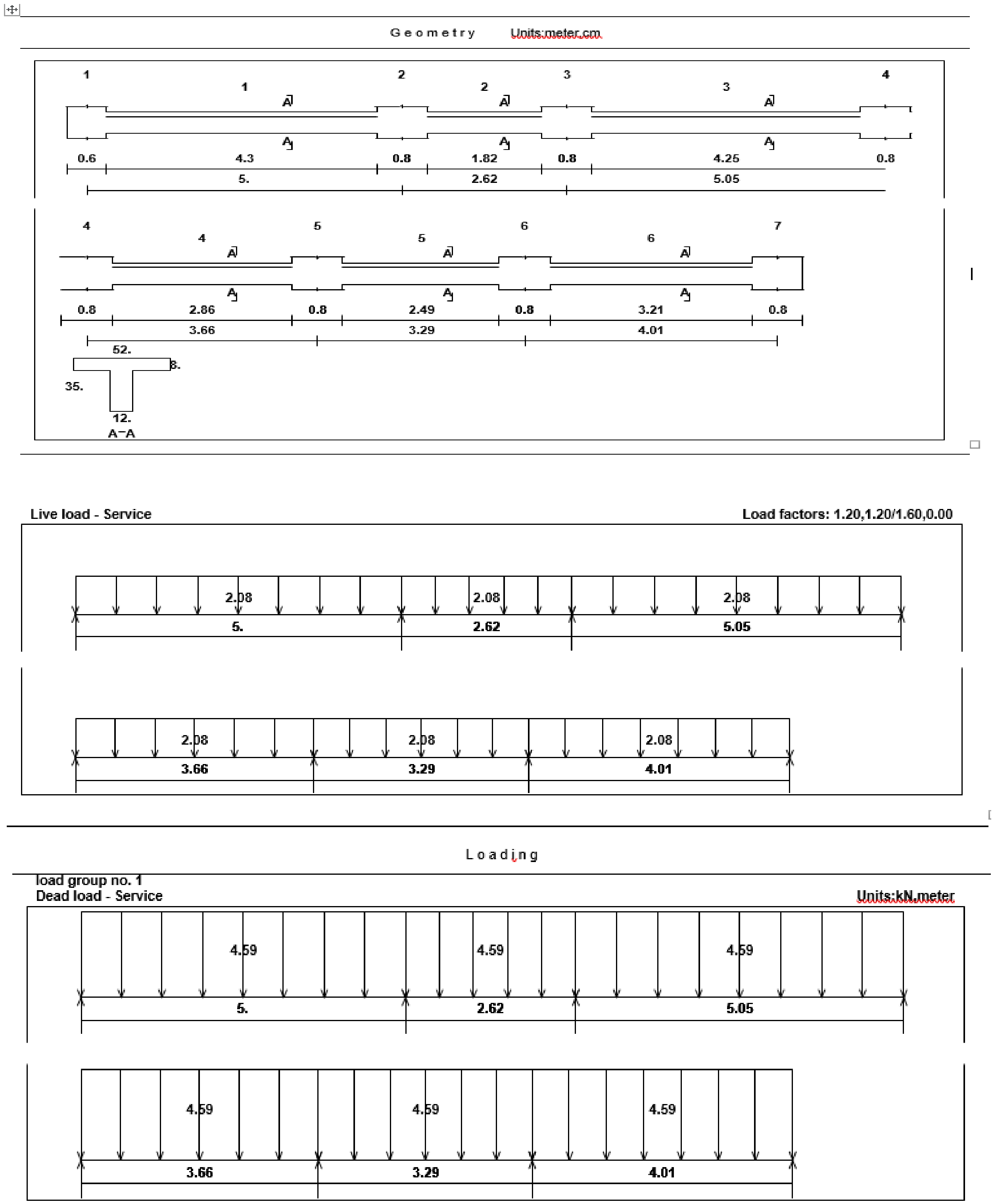


Figure (4-8) shows rib geometry and loads.

The envelope shear and moment diagrams (for all load combinations). Using the structural analysis and design program (Atir 12) we obtain the Envelope Moment diagram for Rib(B-R1).

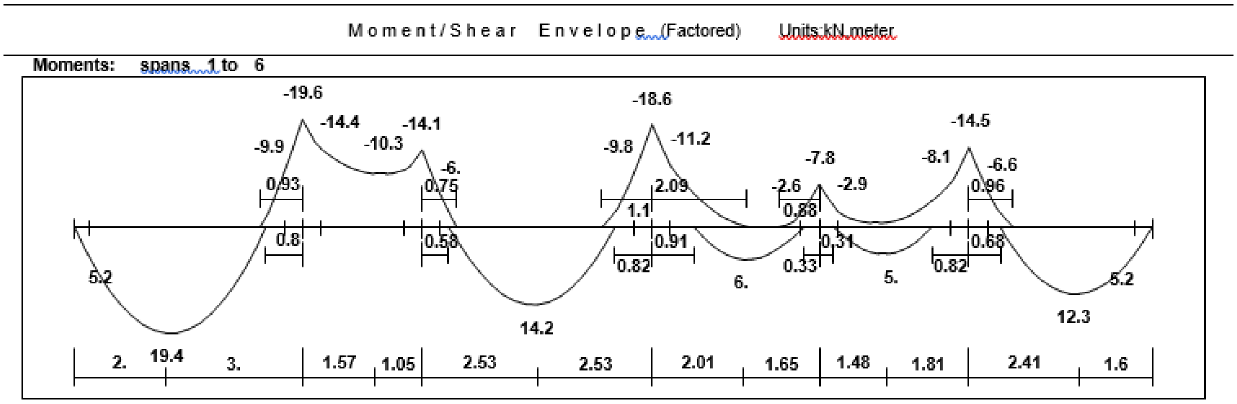


Figure (4-9) Moment envelop diagram.

4-4-4 Design Rib for Flexure (B-R1):

Design of (B-R1) for positive moments

Assume bar diameter ∅ 12 for main positive reinforcement.

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

The maximum positive moment in all spans of Rib21 $M_u = +19.4\ kN.m$

Check if $(a > h_f)$:

$$M_{nf} = 0.85\, f_c{}' b h_f \left(d - \frac{h_f}{2}\right) = 0.85(24).(520).(80).\left(314 - \frac{80}{2}\right) = 232.53\ kN.m$$

$$M_{nf} = 232.53 kN.m \gg \frac{M_u}{\emptyset} = \frac{19.4}{0.9} = 21.6\ kN.m\ \ldots..a < h_f$$

The section will be designed as rectangular section with $b = 520\ mm$.

$$R_n = \frac{M_u}{\emptyset bd} = \frac{19.4 \times 10^6}{0.9 \cdot 520 \cdot 314^2} = 0.420\ MPa\ , \quad m = \frac{f_y}{0.85\, f_c{}'} = \frac{420}{0.85 \cdot 24} = 20.58$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2R_n m}{f_y}}\right) = \frac{1}{20.58}\left(1-\sqrt{1-\frac{2\cdot 0.420\cdot 20.58}{420}}\right) = 0.001010$$

$$A_s = \rho b d = 0.001010\cdot 520\cdot 314 = 164.91\ mm^2$$

Check for $A_{s,min}$:

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

$$A_{s,min} = 0.25\,\frac{\sqrt{24}}{420}\cdot 120\cdot 314 = 109.88\ mm^2$$

$$A_{s,min} = \frac{1.4}{420}\cdot 120\cdot 314 = 125.6\ mm^2 \ - control$$

$$A_s = 164.91\ mm^2 > A_{s,min} = 125.6\ mm^2 \ldots\ldots select\ A_s = A_{req} = 164.91\ mm^2$$

Use 2Ø12 $with\ A_s = 2.26\ cm^2 \ > A_{s,req} = 1.6491\ cm^2$

Check for strain:

$$a = \frac{A_s f_y}{0.85\,f_c{}'b} = \frac{226\cdot 420}{0.85\cdot 24\cdot 520} = 8.95\ mm$$

$$c = \frac{a}{\beta_1} \quad , \quad \beta_1 = 0.85$$

$$c = \frac{8.95}{0.85} = 10.53\ mm$$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{314-10.53}{10.53}\right) = 0.086 > 0.005 \ \ - OK$$

<u>Note that:</u>

Moments equals or less than $M_u = +19.4\ kN.m$, use 2Ø12 for each rib span. So that other positive moments $M_u = +14.2\ ,+12.3\ kN.m < M_u = +19.4\ kN.m$ have reinforcement of 2Ø12 for each rib span.

Design of (Rib B-R1) for negative moments:

The maximum positive moment in all spans $M_u = -14.4 \;\; kN.m$

Assume bar diameter ∅ 12 for main negative reinforcement.

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

$$R_n = \frac{M_u}{\emptyset bd} = \frac{14.4 \times 10^6}{0.9 \cdot 520 \cdot 314^2} = 0.312\, MPa$$

$$m = \frac{f_y}{0.85\, f_c{}'} = \frac{420}{0.85 \cdot 24} = 20.58$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.58}\left(1 - \sqrt{1 - \frac{2 \cdot 0.312 \cdot 20.58}{420}}\right) = 0.000749$$

$$A_s = \rho bd = 0.000749 \cdot 520 \cdot 314 = 122.26\, mm^2$$

Check for $A_{s,min}$ **:**

$$A_{s,req} = 122.26\, mm^2 < A_{s,min} = \;125.6 \;\; mm^2 \ldots.\;\; select \;\; A_s = A_{s,min} = 125.6 \;\; mm^2$$

Use 2∅12 $with \;\; A_s = 2.262\, cm^2 \; > \; A_{s,min} = 1.256\, cm^2$

Check for strain:

$$a = \frac{A_s f_y}{0.85\, f_c{}' b} = \frac{226.2 \cdot 420}{0.85 \cdot 24 \cdot 120} = 38.8\, mm$$

$$c = \frac{a}{\beta_1} \quad , \quad \beta_1 = 0.85$$

$$c = \frac{38.8}{0.85} = 45.64\, mm$$

$$\varepsilon_s = 0.003\left(\frac{d - c}{c}\right) = 0.003\left(\frac{314 - 45.64}{45.64}\right) = 0.0176 > 0.005 \quad - OK$$

Note that:

Moments equals or less than $M_u = -14.4\ kN.m$, use $2\emptyset12$ for each rib span.
So that other negative moments:
$M_u = -10.3\ , -11.2\ , -2.9, -8.1\ \ kN.m\ < M_u = -14.4\ \ kN.m$ Have reinforcement of $2\emptyset12$ for each rib span.

4-4-5 Design Rib for Shear:

4-4-5-1 Design procedure for shear:

Design of cross section subjected to shear and Flexure:

$$\emptyset V_n \geq V_u$$

Where:

V_u : *factored shear force at the section .*
V_n : *the nominal shear strength ;*

$$V_n = V_c + V_s$$

V_c : *the nominal shear strenght provided by concrete.*

$$V_c = \frac{1}{6}\lambda\sqrt{f_c{}'}\ b_w d \qquad , \lambda = 1.0\ \ for\ normal\ weight\ concrete.$$

V_s : *the nominal shear strenght provided by shear reinforcement (stirrups).*

$$V_s = \frac{A_v \cdot f_y \cdot d}{s}$$

- **Shear conditions and cases (Items):**

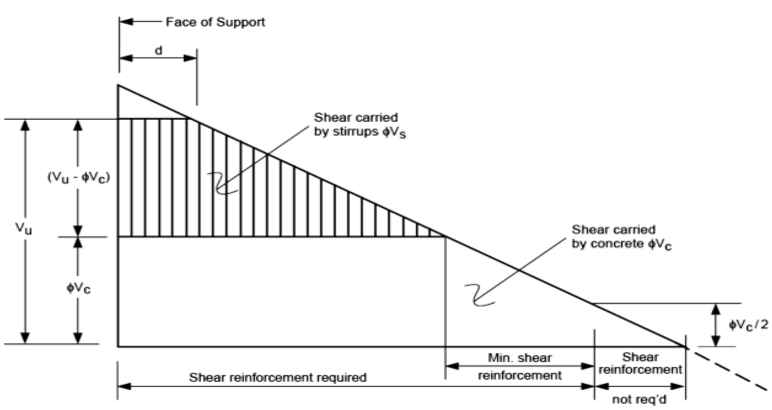


Figure (4-10) Shear diagonal.

Check for dimensions:

According to (ACI), V_s **shall not be taken grater than** $V_{s,max} = \frac{2}{3}\sqrt{f_c{}'}\ b_w d$ **, So :**

If $V_s > V_{s,max}$ **the section must be enlarged (Dimensions are not enough) where:**

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

Case I:

$$V_u \leq \frac{1}{2}\emptyset V_c \qquad - NO\ shear\ reinforcement\ is\ requered.$$

Case II:

$$\frac{1}{2}\emptyset V_c < V_u \leq \emptyset V_c \qquad - Minimum\ shear\ reinforcement\ is\ requered\ (A_{v,min})$$

$A_{v,min} = \frac{1}{16} \cdot \sqrt{f_c{}'} \cdot \frac{b_w\, s}{f_{yt}} \geq \frac{1}{3} \cdot \frac{b_w\, s}{f_{yt}}$ **, Or in the form:**

$\left(\frac{A_{v,min}}{S}\right)$ **the maximum of** $\geq \frac{1}{3} \cdot \frac{b_w}{f_{yt}}$

$\geq \frac{1}{16} \cdot \sqrt{f_c{}'} \cdot \frac{b_w}{f_{yt}}$

Here:

$$S_{max} \leq \frac{d}{2} \quad or \quad S_{max} \leq 600\ mm$$

Where:

S : *step of stirrups (spacing between stirrups).*
f_{yt} : *yield stress of stirrups .*

Case III:

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

$$\left(\frac{A_{v,min}}{S}\right) = \frac{V_{s,min}}{f_{yt}\, d} \rightarrow \quad V_{s,min} = \left(\frac{A_{v,min}}{S}\right) f_{yt}\, d$$

Then, $V_{s,min}$ is the maximum of:

$$V_{s,min} = \frac{1}{16} \cdot \sqrt{f_c{}'}\, b_w \cdot d \quad , \quad V_{s,min} = \frac{1}{3}\, b_w\, d$$

Minimum shear reinforcement is provided ($A_{v,min}$) with:

$$S_{max} \le \frac{d}{2} \quad or \quad S_{max} \le 600\ mm$$

Case IV:

$$\emptyset\left(V_c + V_{s,min}\right) < V_u \le \emptyset\left(V_c + V_{s'}\right) \qquad -\ stirrups\ are\ required$$

Where: $V_{s,min} < V_s \le V_{s'}$, $V_s = V_n - V_c = \frac{V_u}{\emptyset} - V_c$, $V_{s'} = \frac{1}{3}\sqrt{f_c{}'}\, b_w\, d$

$$\left(\frac{A_v}{S}\right) = \frac{V_s}{f_{yt}\, d}$$

Here:

$$S_{max} \le \frac{d}{2} \quad or \quad S_{max} \le 600\ mm$$

Case V:

$$\emptyset\left(V_c + V_{s'}\right) < V_u \le \emptyset\left(V_c + V_{s,max}\right) \qquad -\ stirrups\ are\ required$$

Where: $V_{s'} < V_s \le V_{s,min}$, $V_s = V_n - V_c = \frac{V_u}{\emptyset} - V_c$, $V_{s'} = \frac{1}{3}\sqrt{f_c{}'}\, b_w\, d$

$V_{s,max} = \frac{2}{3}\sqrt{f_c{}'}\ b_w d$ **And** $\left(\frac{A_v}{S}\right) = \frac{V_s}{f_{yt}\, d}$

Here:

$$S_{max} \le \frac{d}{2} \quad or \quad S_{max} \le 300\ mm$$

The shear envelops of Rib that we consider to design it (B-R1) is:

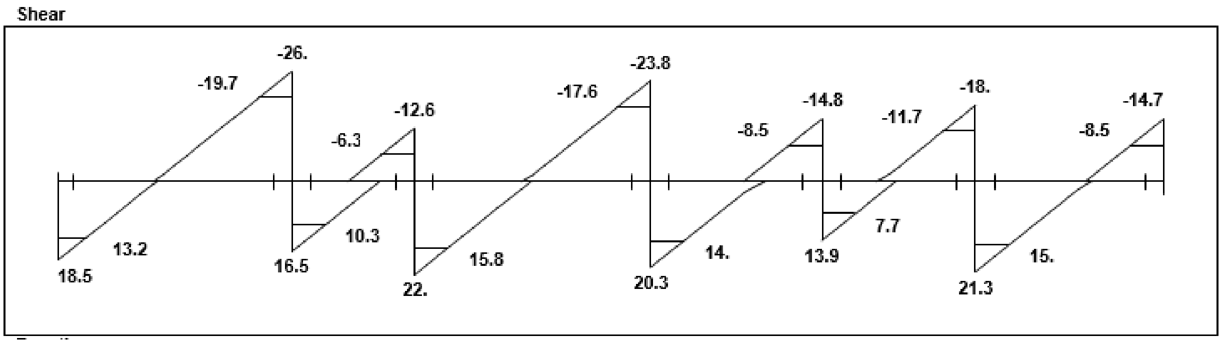


Figure (4-11) Shear envelop diagram.

The maximum shear force at the distance d from the face of support $V_u = 19.7\ kN$.

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

$$V_n = \frac{V_u}{\emptyset}$$

$$V_C = 1.1 \times \frac{1}{6}\sqrt{f_c{}'}\ b_w d = 1.1 \times \frac{1}{6}\sqrt{24} \cdot 120 \cdot 314 = 33.84\ kN$$

$$\emptyset V_C = 0.75 \cdot 33.4 = 25.38\ kN$$

$\frac{1}{2}\emptyset V_C = \frac{25.38}{2} = 12.69\ kN < V_u = 19.7 kN < \emptyset V_C = 25.38\ kN$ **- Case II**

So, Minimum shear reinforcement is required.

$\left(\frac{A_{v,min}}{S}\right)$ **the maximum of** $\geq \frac{1}{3} \cdot \frac{b_w}{f_{yt}} = \frac{1}{3} \times \frac{120}{420} = 95.23 \times 10^{-3}$ $- controled$

$\geq \frac{1}{16} \cdot \sqrt{f_c{}'} \cdot \frac{b_w}{f_{yt}} = \frac{1}{16} \times \sqrt{24} \times \frac{120}{420} = 87.48 \times 10^{-3}$

Try $\emptyset 8\ stirrup\ (2\ legs)\ with\ A_{v,\emptyset 8} = 100.53\ mm^2$

$$\frac{100.55}{S} = 95.23 \times 10^{-3} \quad \rightarrow \quad \boldsymbol{S = 1055.86\ mm}$$

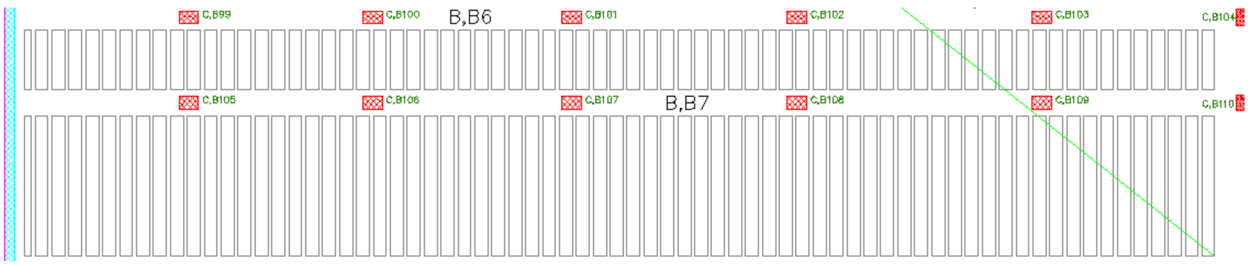
But:

$$S_{max} \le \frac{d}{2} = \frac{314}{2} = 157\ mm \quad or \quad S_{max} \le 600\ mm$$

Use $\emptyset 8\ @\ 15\ cm < S_{max} = 15.7\ cm$

4-5 | Design of Beam (B,B7)

Figure (4-7) shows the location of rib in slab drawing plan.



`Figure (4-12) location of Beam (B, B7) in slab drawing plan.

4-5-1 Beam (B,B7) geometry:

Figure (4-13) shows the geometry of beam that considered to design and it's statically system with section of $(80 \times 35\ cm)$ **Hidden beam:**

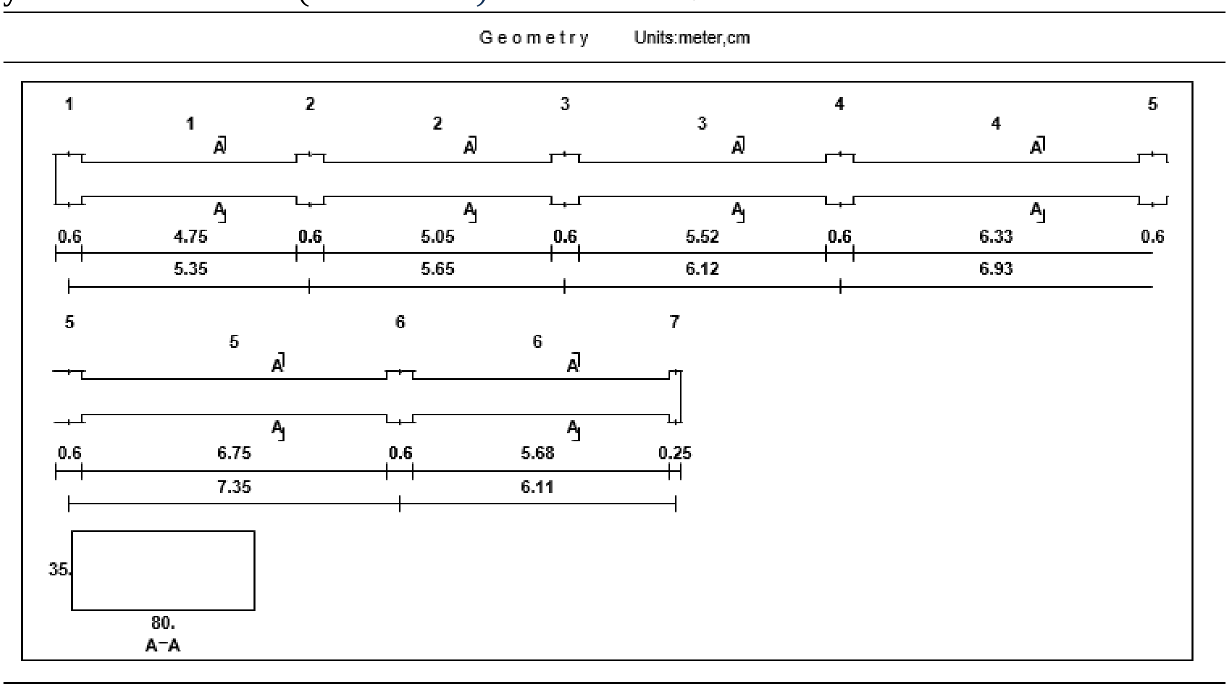


Figure (4-13) Beam (B,B7) geometry.

4-5-1 Load calculations for Beam:

There are 3 sources of load that accts on beam.

1. **The own weight of the beam and the weights of floor layers within the beam width as a uniform (dead load).**
2. **The load that comes from rib (B-R1) as a uniform (dead and live loads).**
3. **The load that comes from two way ribbed slab as a uniform (dead and live loads).**

The following is the description for each one:

4-5-1-1 | the own weight of beam:

Table (4-4) shows the own weight of the beam and the weights of floor layers within the beam width:

Dead Load Form	Thickness δ (m)	Unit weight γ $(\frac{kN}{m^3})$	$\gamma \times \delta \times 1$ $(\frac{kN}{m})$
Tiles	0.03	23	$0.03 \times 23 \times 0.8 = 0.552$
Mortar	0.03	22	$0.03 \times 22 \times 0.8 = 0.528$
Coarse Sand	0.07	17	$0.07 \times 17 \times 0.8 = 0.592$
RC Beam	0.35	25	$0.35 \times 25 \times 0.8 = 8.75$
Plaster	0.03	22	$0.03 \times 22 \times 0.8 = 0.528$
$\sum$ *service* D_L			10.95 $(\frac{kN}{m})$

Table (4-4) Dead Load calculations of Beam.

4-5-1-2 | the load that comes from rib (B-R1):

Its equals the reactions of (B-R1) divided on reputational unit (0.52 m), Figure (4-14) shows the reactions of rib (B-R1) on beam (B,B7):

Reactions

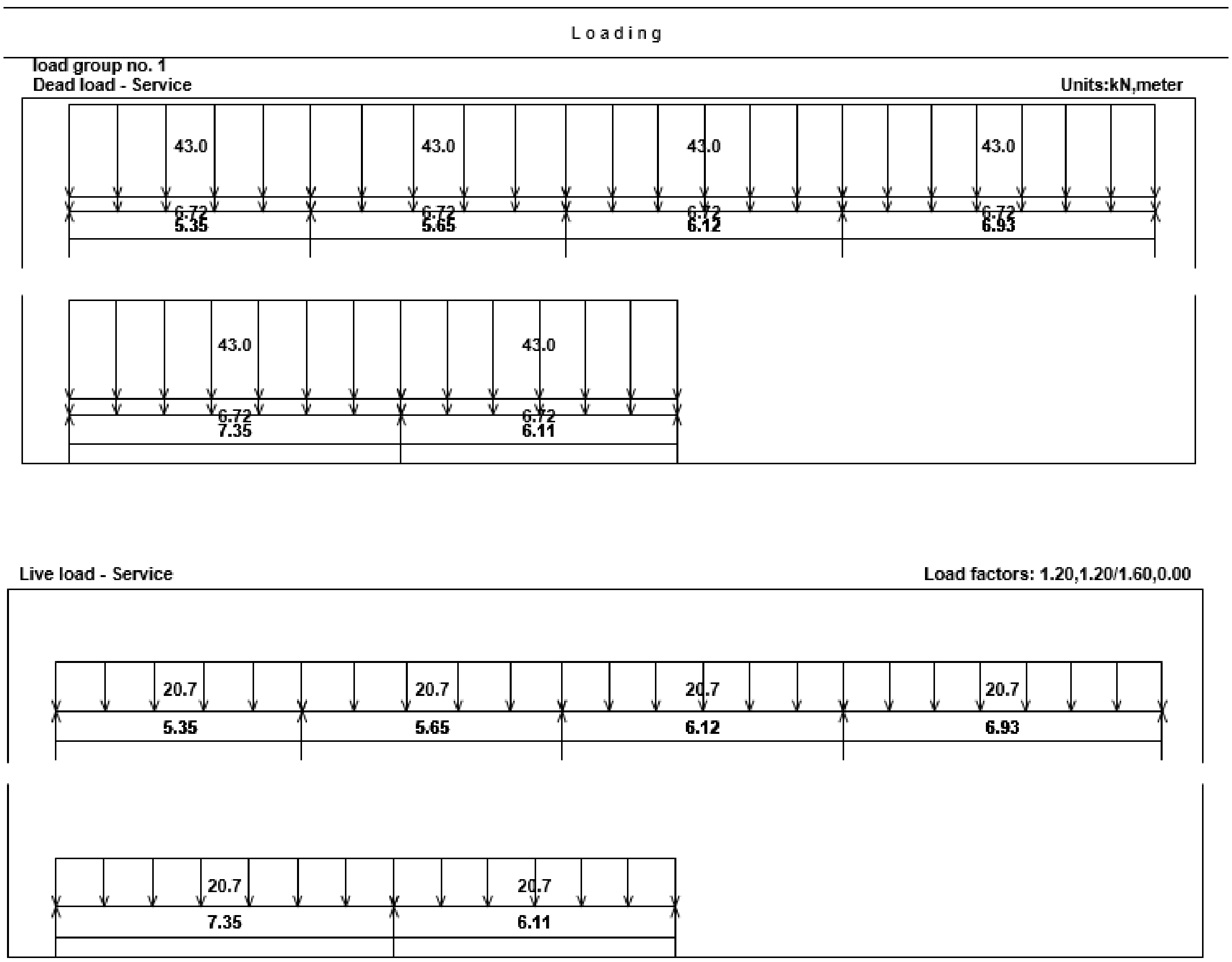
Factored							
DeadR	11.44	24.88	18.83	26.83	15.41	23.87	8.88
LiveR	7.11	17.67	15.76	17.3	13.31	15.47	5.85
Max R	18.55	42.56	34.59	44.13	28.72	39.34	14.73
Min R	11.25	26.65	19.26	32.72	17.72	28.6	8.4
Service							
DeadR	9.54	20.73	15.69	22.36	12.84	19.9	7.4
LiveR	4.44	11.05	9.85	10.81	8.32	9.67	3.66
Max R	13.98	31.78	25.54	33.17	21.16	29.56	11.06
Min R	9.42	21.84	15.96	26.04	14.29	22.85	7.1

Figure (4-14) Reactions of rib (B-R1).

So that:

$$service\ D_L = \frac{22.36}{0.52} = 43\ \left(\frac{kN}{m}\right) \quad , \quad service\ \ L_L = \frac{10.81}{0.52} = 20.76\ \left(\frac{kN}{m}\right)$$

So, the load input to analysis seems like in **Figure (4-15):**



4-5-2 Design for Flexure of Beam:

The envelope moment diagrams (for all load combinations). Using the structural analysis and design program (Atir 12), in **Figure (4-16):**

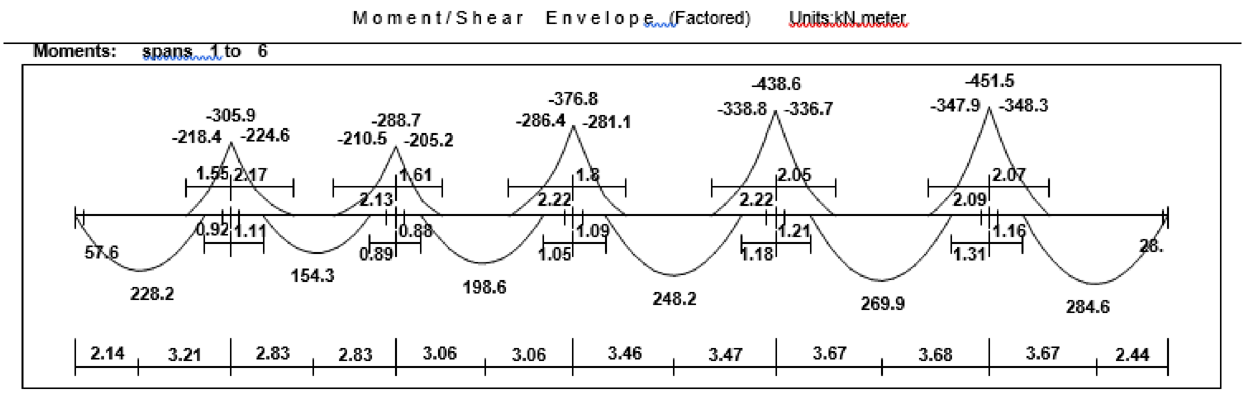


Figure (4-16) Envelope moment diagram for Beam (B,B7).

Design of positive moments:

Assume bar diameter $\emptyset\ 22$ for main positive reinforcement.

$$d = h - cover - d_{stirrups} - \frac{d_b}{2} = 350 - 40 - 10 - \frac{22}{2} = 289\ mm$$

The maximum positive moment in all spans of (B,B7) $M_u = +284.6\ \ kN.m$

$$R_n = \frac{M_u}{\emptyset bd} = \frac{284.6 \times 10^6}{0.9 \cdot 800 \cdot 289^2} = 4.733\ MPa$$

$$m = \frac{f_y}{0.85\ f_c{}'} = \frac{420}{0.85 \cdot 24} = 20.58$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.58}\left(1 - \sqrt{1 - \frac{2 \cdot 4.733 \cdot 20.58}{420}}\right) = 0.01301$$

$$A_s = \rho bd = 0.01301 \cdot 800 \cdot 289 = 3008.14\ mm^2$$

Check for $A_{s,min}$:

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

$$A_{s,min} = 0.25\,\frac{\sqrt{24}}{420}\cdot 800\cdot 289 = 674.2\ mm^2$$

$$A_{s,min} = \frac{1.4}{420}\cdot 800\cdot 289 = 770.67\ \ mm^2\ \ - control$$

$$A_{s,req} = 3008.14\ mm^2 > A_{s,min} =\ 770.67\ \ mm^2 \qquad - OK$$

Use 8∅22 $with\ \ A_s = 30.41\ cm^2\ >\ A_{s,req} = 30.0814\ cm^2$

Check for strain:

$$a = \frac{A_s f_y}{0.85\,f_c{}'b} = \frac{3041\cdot 420}{0.85\cdot 24\cdot 800} = 78.26\ mm$$

$$c = \frac{a}{\beta_1} \qquad ,\ \ \beta_1 = 0.85$$

$$c = \frac{78.26}{0.85} = 92.07\ mm$$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{289-92.07}{92.07}\right) = 0.00642 > 0.005 \quad -\ OK$$

Check for bar placement:

$$S_b = \frac{800-40\times 2-10\times 2-8\times 22}{7} = 74.86\ \ mm > 25\ mm \qquad - OK$$

Design of positive moment $M_u = +269.9\ kN.m$

Also assume bar diameter $\emptyset\ 22$

$$d = h - cover - d_{stirrups} - \frac{d_b}{2} = 350 - 40 - 10 - \frac{22}{2} = 289\ mm$$

$$R_n = \frac{M_u}{\emptyset bd} = \frac{269.9 \times 10^6}{0.9 \cdot 800 \cdot 289^2} = 4.49\ MPa$$

$$m = 20.58$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.58}\left(1 - \sqrt{1 - \frac{2 \cdot 4.68 \cdot 20.58}{420}}\right) = 0.01223$$

$$A_s = \rho bd = 0.012839 \cdot 800 \cdot 289 = 2827.4\ mm^2$$

Check for $A_{s,min}$ **:**

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

$$A_{s,min} = 0.25\ \frac{\sqrt{24}}{420} \cdot 800 \cdot 289 = 674.2\ mm^2$$

$$A_{s,min} = \frac{1.4}{420} \cdot 800 \cdot 289 = 770.67\ \ mm^2\ \ - control$$

$$A_{s,req} = 2827.4\ mm^2 > A_{s,min} = \ 770.67\ \ mm^2 \quad - OK$$

Use $8\emptyset 22\ \ with\ \ A_s = 30.41\ cm^2\ > A_{s,req} = 28.274\ cm^2$

Check for strain:

$$a = \frac{A_s f_y}{0.85\ f_c{}' b} = \frac{3041 \cdot 420}{0.85 \cdot 24 \cdot 800} = 78.26\ mm$$

$$c = \frac{a}{\beta_1} \quad , \quad \beta_1 = 0.85$$

$$c = \frac{78.26}{0.85} = 92.07\ mm$$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{289-92.07}{92.07}\right) = 0.00642 > 0.005 \quad -OK$$

Check for bar placement:

$$S_b = \frac{800 - 40\times 2 - 10\times 2 - 8\times 22}{7} = 74.86\ mm > 25\ mm \quad -OK$$

Design of positive moment $M_u = +248.2\ kN.m$

Also assume bar diameter ∅ 22

$$d = h - cover - d_{stirrups} - \frac{d_b}{2} = 350 - 40 - 10 - \frac{22}{2} = 289\ mm$$

$$R_n = \frac{M_u}{\emptyset bd} = \frac{248.2\times 10^6}{0.9\cdot 800\cdot 289^2} = 4.13\ MPa$$

$$m = 20.58$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2R_n m}{f_y}}\right) = \frac{1}{20.58}\left(1-\sqrt{1-\frac{2\cdot 4.13\cdot 20.58}{420}}\right) = 0.0111$$

$$A_s = \rho bd = 0.0111\cdot 800\cdot 289 = 2566.7\ mm^2$$

Check for $A_{s,min}$ **:**

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

$$A_{s,min} = 0.25\,\frac{\sqrt{24}}{420}\cdot 800\cdot 289 = 674.2\ mm^2$$

$$A_{s,min} = \frac{1.4}{420}\cdot 800\cdot 289 = 770.67\ mm^2 \ -control$$

$A_{s,req} = 2566.7\ mm^2 > A_{s,min} = 770.67\ mm^2 \quad - OK$

Use $7\emptyset 22$ $with\ A_s = 26.609\ cm^2 > A_{s,req} = 25.667\ cm^2$

Check for strain:

$$a = \frac{A_s f_y}{0.85\ f_c{}'b} = \frac{2660.9 \cdot 420}{0.85 \cdot 24 \cdot 800} = 68.48\ mm$$

$$c = \frac{a}{\beta_1} \quad , \quad \beta_1 = 0.85$$

$$c = \frac{68.48}{0.85} = 80.56\ mm$$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{289-80.56}{80.56}\right) = 0.00776 > 0.005 \quad - OK$$

Check for bar placement:

$$S_b = \frac{800 - 40 \times 2 - 10 \times 2 - 7 \times 22}{6} = 91\ mm > 25\ mm \quad - OK$$

Design of positive moment $M_u = +228.2\ kN.m$

Also assume bar diameter $\emptyset\ 22$

$$d = h - cover - d_{stirrups} - \frac{d_b}{2} = 350 - 40 - 10 - \frac{22}{2} = 289\ mm$$

$$R_n = \frac{M_u}{\emptyset bd} = \frac{228.2 \times 10^6}{0.9 \cdot 800 \cdot 289^2} = 3.795\ MPa$$

$$m = 20.58$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.58}\left(1 - \sqrt{1 - \frac{2 \cdot 3.795 \cdot 20.58}{420}}\right) = 0.010082$$

$$A_s = \rho bd = 0.010082 \cdot 800 \cdot 289 = 2330.96\ mm^2$$

Check for $A_{s,min}$:

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

$$A_{s,min} = 0.25\ \frac{\sqrt{24}}{420} \cdot 800 \cdot 289 = 674.2\ mm^2$$

$$A_{s,min} = \frac{1.4}{420} \cdot 800 \cdot 289 = 770.67\ \ mm^2\ \ - control$$

$$A_{s,req} = 2330.96\ mm^2 > A_{s,min} = \ 770.67\ \ mm^2$$

$$take\ A_{s,req} = \ 2330.96\ mm^2$$

Use 7∅22 $with\ \ A_s = 26.6093\ cm^2\ \ >\ A_{s,req} = 23.3096\ cm^2$

Check for strain:

$$a = \frac{A_s f_y}{0.85\ f_c{}' b} = \frac{2660.93 \cdot 420}{0.85 \cdot 24 \cdot 800} = 68.48\ mm$$

$$c = \frac{a}{\beta_1} \quad ,\quad \beta_1 = 0.85$$

$$c = \frac{68.48}{0.85} = 80.56\ mm$$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{289-80.56}{80.56}\right) = 0.00776 > 0.005 \quad - OK$$

Check for bar placement:

$$S_b = \frac{800 - 40 \times 2 - 10 \times 2 - 7 \times 22}{6} = 91\ mm\ > 25\ mm \quad - OK$$

Design of positive moment $M_u = +198.6\ kN.m$

Also assume bar diameter $\emptyset\ 22$

$$d = h - cover - d_{stirrups} - \frac{d_b}{2} = 350 - 40 - 10 - \frac{22}{2} = 289\ mm$$

$$R_n = \frac{M_u}{\emptyset bd} = \frac{198.6 \times 10^6}{0.9 \cdot 800 \cdot 289^2} = 3.30\ MPa$$

$$m = 20.58$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.58}\left(1 - \sqrt{1 - \frac{2 \cdot 3.30 \cdot 20.58}{420}}\right) = 0.00862$$

$$A_s = \rho bd = 0.00862 \cdot 800 \cdot 289 = 1992.9\ mm^2$$

Check for $A_{s,min}$ **:**

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

$$A_{s,min} = 0.25\ \frac{\sqrt{24}}{420} \cdot 800 \cdot 289 = 674.2\ mm^2$$

$$A_{s,min} = \frac{1.4}{420} \cdot 800 \cdot 289 = 770.67\ \ mm^2\ \ - control$$

$$A_{s,req} = 1992.9\ mm^2 > A_{s,min} = \ 770.67\ \ mm^2$$

$$take\ A_{s,req} = \ 1992.9\ mm^2$$

Use $6\emptyset 22$ $\ with\ \ A_s = 22.808\ cm^2\ > A_{s,req} = 20.1075\ cm^2$

Check for strain:

$$a = \frac{A_s f_y}{0.85\, f_c{}' b} = \frac{2280.8 \cdot 420}{0.85 \cdot 24 \cdot 800} = 58.7\, mm$$

$$c = \frac{a}{\beta_1} \quad , \quad \beta_1 = 0.85$$

$$c = \frac{58.7}{0.85} = 69.06\, mm$$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{289 - 69.06}{69.06}\right) = 0.00955 > 0.005 \quad - OK$$

Check for bar placement:

$$S_b = \frac{800 - 40 \times 2 - 10 \times 2 - 6 \times 22}{5} = 113.6\, mm > 25\, mm \quad - OK$$

Design of positive moment $M_u = +154.3\ kN.m$

Also assume bar diameter ∅ 22

$$d = h - cover - d_{stirrups} - \frac{d_b}{2} = 350 - 40 - 10 - \frac{22}{2} = 289\, mm$$

$$R_n = \frac{M_u}{\emptyset b d} = \frac{154.3 \times 10^6}{0.9 \cdot 800 \cdot 289^2} = 2.566\, MPa$$

$$m = 20.58$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.58}\left(1 - \sqrt{1 - \frac{2 \cdot 2.566 \cdot 20.58}{420}}\right) = 0.00655$$

$$A_s = \rho b d = 0.006550 \cdot 800 \cdot 289 = 1514.4\, mm^2$$

Check for $A_{s,min}$:

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

$$A_{s,min} = 0.25\,\frac{\sqrt{24}}{420}\cdot 800\cdot 289 = 674.2\;mm^2$$

$$A_{s,min} = \frac{1.4}{420}\cdot 800\cdot 289 = 770.67\;\;mm^2\;\;-control$$

$$A_{s,req} = 1514.4\;mm^2 > A_{s,min} = \;770.67\;\;mm^2$$

$$take\;A_{s,req} = 1514.4\;mm^2$$

Use $4\emptyset 22\;\;with\;\;A_s = 15.205\;cm^2\;>\;A_{s,req} = 15.1182\;cm^2$

Check for strain:

$$a = \frac{A_s f_y}{0.85\,f_c{}'b} = \frac{1520.5\cdot 420}{0.85\cdot 24\cdot 800} = 39.13\;mm$$

$$c = \frac{a}{\beta_1}\quad,\quad \beta_1 = 0.85$$

$$c = \frac{39.13}{0.85} = 46.04\;mm$$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{289-46.04}{46.04}\right) = 0.0158 > 0.005\qquad - OK$$

Check for bar placement:

$$S_b = \frac{800-40\times 2-10\times 2-4\times 22}{3} = 204\;mm\;>25\;mm\quad -OK$$

Design of Negative moment $M_u = -338.8\ kN.m$

Assume bar diameter ∅ 20 for main negative reinforcement.

$$d = h - cover - d_{stirrups} - \frac{d_b}{2} = 350 - 40 - 10 - \frac{20}{2} = 290\ mm$$

$$R_n = \frac{M_u}{\emptyset bd} = \frac{388.8 \times 10^6}{0.9 \cdot 800 \cdot 290^2} = 5.59\ MPa$$

$$m = 20.58$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.58}\left(1 - \sqrt{1 - \frac{2 \cdot 5.59 \cdot 20.58}{420}}\right) = 0.01591$$

$$A_s = \rho bd = 0.01591 \cdot 800 \cdot 290 = 3692.6\ mm^2$$

Check for $A_{s,min}$ **:**

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

$$A_{s,min} = 0.25\ \frac{\sqrt{24}}{420} \cdot 800 \cdot 290 = 676.5\ mm^2$$

$$A_{s,min} = \frac{1.4}{420} \cdot 800 \cdot 290 = 773.3\ mm^2\ - control$$

$$A_{s,req} = 3692.6\ mm^2 > A_{s,min} = 773.3\ mm^2 \qquad - OK$$

Use 12∅20 $with\ A_s = 37.69\ cm^2\ >\ A_{s,req} = 36.926\ cm^2$

Check for strain:

$$a = \frac{A_s f_y}{0.85\ f_c{}' b} = \frac{3769 \cdot 420}{0.85 \cdot 24 \cdot 800} = 97\ mm$$

$$c = \frac{a}{\beta_1} \quad , \quad \beta_1 = 0.85$$

$$c = \frac{97}{0.85} = 114.1\ mm$$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{290-114.1}{114.1}\right) = 0.00462 > 0.004 \quad - OK$$

Check for bar placement:

$$S_b = \frac{800 - 40\times 2 - 10\times 2 - 12\times 20}{11} = 41.82\ mm\ > 25\ mm \quad - OK$$

Design of Negative moment $M_u = -348.3\ kN.m$

Assume bar diameter ∅ 20 for main negative reinforcement.

$$d = h - cover - d_{stirrups} - \frac{d_b}{2} = 350 - 40 - 10 - \frac{20}{2} = 290\ mm$$

$$R_n = \frac{M_u}{\emptyset bd} = \frac{348.3\times 10^6}{0.9\cdot 800\cdot 290^2} = 5.75\ MPa$$

$$m = 20.58$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.58}\left(1 - \sqrt{1 - \frac{2\cdot 5.75\cdot 20.58}{420}}\right) = 0.01648$$

$$A_s = \rho bd = 0.01648\cdot 800\cdot 290 = 3825.2\ mm^2$$

Check for $A_{s,min}$:

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

$$A_{s,min} = 0.25\ \frac{\sqrt{24}}{420}\cdot 800\cdot 290 = 676.5\ mm^2$$

$$A_{s,min} = \frac{1.4}{420}\cdot 800\cdot 290 = 773.3\ \ mm^2\ \ - control$$

$A_{s,req} = 3825.2\ mm^2 > A_{s,min} = 773.3\ mm^2 \quad - OK$

Use 13∅20 $with\ A_s = 40.8407\ cm^2 > A_{s,req} = 38.252\ cm^2$

Check for strain:

$$a = \frac{A_s f_y}{0.85\, f_c{}'b} = \frac{4084.07 \cdot 420}{0.85 \cdot 24 \cdot 800} = 105.1\ mm$$

$$c = \frac{a}{\beta_1} \quad , \quad \beta_1 = 0.85$$

$$c = \frac{105.1}{0.85} = 123.65\ mm$$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{290-123.65}{123.65}\right) = 0.00404 > 0.004 \quad - OK$$

Check for bar placement:

$$S_b = \frac{800 - 40\times 2 - 10\times 2 - 13\times 20}{12} = 36.67\ mm > 25\ mm \quad - OK$$

Design of Negative moment $M_u = -286.4\ kN.m$

Assume bar diameter ∅ 20 for main negative reinforcement.

$$d = h - cover - d_{stirrups} - \frac{d_b}{2} = 350 - 40 - 10 - \frac{20}{2} = 290\ mm$$

$$R_n = \frac{M_u}{\emptyset bd} = \frac{286.4\times 10^6}{0.9 \cdot 800 \cdot 290^2} = 4.73\ MPa$$

$$m = 20.58$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.58}\left(1 - \sqrt{1 - \frac{2 \cdot 4.73 \cdot 20.58}{420}}\right) = 0.01300$$

$$A_s = \rho bd = 0.01300 \cdot 800 \cdot 290 = 3016.3\ mm^2$$

Check for $A_{s,min}$:

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

$$A_{s,min} = 0.25\,\frac{\sqrt{24}}{420}\cdot 800\cdot 290 = 676.5\,mm^2$$

$$A_{s,min} = \frac{1.4}{420}\cdot 800\cdot 290 = 773.3\ mm^2\ -control$$

$$A_{s,req} = 3016.3\,mm^2 > A_{s,min} = 773.3\ mm^2 \quad -OK$$

Use $10\emptyset 20$ with $A_s = 31.416\,cm^2 > A_{s,req} = 30.163\,cm^2$

Check for strain:

$$a = \frac{A_s f_y}{0.85\,f_c{}'b} = \frac{3141.6\cdot 420}{0.85\cdot 24\cdot 800} = 80.85\,mm$$

$$c = \frac{a}{\beta_1}\quad ,\ \beta_1 = 0.85$$

$$c = \frac{80.85}{0.85} = 95.12\,mm$$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{290-95.12}{95.12}\right) = 0.0052 > 0.004 \quad -OK$$

Check for bar placement:

$$S_b = \frac{800-40\times 2-10\times 2-10\times 20}{9} = 55.56\,mm > 25\,mm \quad -OK$$

Design of Negative moment $M_u = -210.5\ kN.m$

Assume bar diameter $\emptyset$ 20 for main negative reinforcement.

$$d = h - cover - d_{stirrups} - \frac{d_b}{2} = 350 - 40 - 10 - \frac{20}{2} = 290\ mm$$

$$R_n = \frac{M_u}{\emptyset bd} = \frac{210.5 \times 10^6}{0.9 \cdot 800 \cdot 290^2} = 3.48\ MPa$$

$$m = 20.58$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.58}\left(1 - \sqrt{1 - \frac{2 \cdot 3.49 \cdot 20.58}{420}}\right) = 0.00915$$

$$A_s = \rho bd = 0.00915 \cdot 800 \cdot 290 = 2122.8\ mm^2$$

Check for $A_{s,min}$:

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

$$A_{s,min} = 0.25\ \frac{\sqrt{24}}{420} \cdot 800 \cdot 290 = 676.5\ mm^2$$

$$A_{s,min} = \frac{1.4}{420} \cdot 800 \cdot 290 = 773.3\ mm^2\ - control$$

$$A_{s,req} = 2122.8\ mm^2 > A_{s,min} = 773.3\ mm^2 \qquad - OK$$

Use 7∅20 $with\ A_s = 21.991\ cm^2\ >\ A_{s,req} = 21.228\ cm^2$

Check for strain:

$$a = \frac{A_s f_y}{0.85\ f_c{}' b} = \frac{2199.1 \cdot 420}{0.85 \cdot 24 \cdot 800} = 56.6\ mm$$

$$c = \frac{a}{\beta_1} \quad , \quad \beta_1 = 0.85$$

$$c = \frac{56.6}{0.85} = 66.59\ mm$$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{290-66.59}{66.59}\right) = 0.01 > 0.004 \quad -OK$$

Check for bar placement:

$$S_b = \frac{800-40\times 2-10\times 2-7\times 20}{6} = 93.3\,mm > 25\,mm \quad -OK$$

Design of Negative moment $M_u = -224.6\ kN.m$

Assume bar diameter $\emptyset\,20$ for main negative reinforcement.

$$d = h - cover - d_{stirrups} - \frac{d_b}{2} = 350-40-10-\frac{20}{2} = 290\,mm$$

$$R_n = \frac{M_u}{\emptyset bd} = \frac{224.6\times 10^6}{0.9\cdot 800\cdot 290^2} = 3.71\,MPa$$

$$m = 20.58$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2R_n m}{f_y}}\right) = \frac{1}{20.58}\left(1-\sqrt{1-\frac{2\cdot 3.71\cdot 20.58}{420}}\right) = 0.00982$$

$$A_s = \rho bd = 0.00982\cdot 800\cdot 290 = 2279.9\,mm^2$$

Check for $A_{s,min}$:

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

$$A_{s,min} = 0.25\,\frac{\sqrt{24}}{420}\cdot 800\cdot 290 = 676.5\,mm^2$$

$$A_{s,min} = \frac{1.4}{420}\cdot 800\cdot 290 = 773.3\ mm^2\ -control$$

$$A_{s,req} = 2279.9\,mm^2 > A_{s,min} = 773.3\ mm^2 \quad -OK$$

Use 8∅20 $\boldsymbol{with\ A_s = 25.133\ cm^2\ >\ A_{s,req} = 22.799\ cm^2}$

Check for strain:

$$a = \frac{A_s f_y}{0.85\, f_c{}' b} = \frac{2513.3 \cdot 420}{0.85 \cdot 24 \cdot 800} = 64.7\ mm$$

$$c = \frac{a}{\beta_1} \quad , \quad \beta_1 = 0.85$$

$$c = \frac{64.7}{0.85} = 76.1\ mm$$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{290-76.1}{76.1}\right) = 0.00843 > 0.004 \quad -OK$$

Check for bar placement:

$$S_b = \frac{800 - 40\times 2 - 10\times 2 - 8\times 20}{7} = 77.14\ mm\ > 25\ mm \quad -OK$$

4-4-5 Design Beam for Shear:

The shear envelops of beam (B,B7) as shown in Figure (4-17):

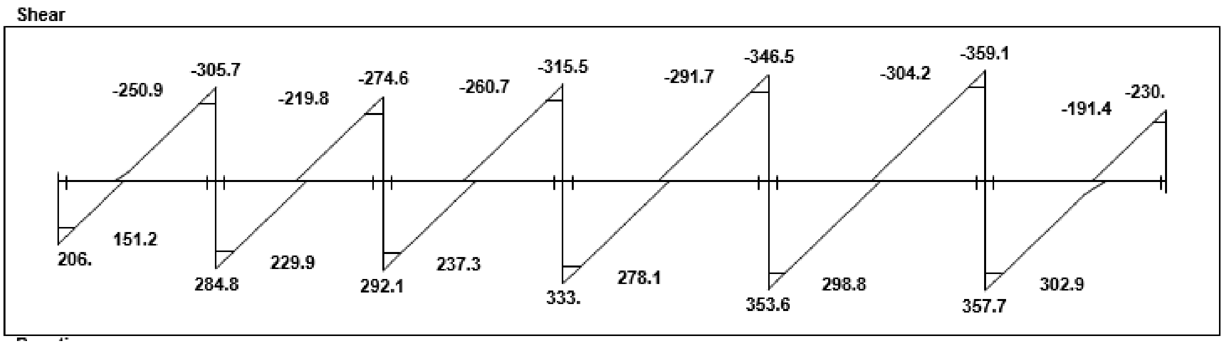


Figure (4-17) Shear envelop diagram of Beam.

The maximum shear force at the distance d from the face of support $V_u = 304.2\ kN$

For V_u = 304.2 kN

$$V_C = \frac{1}{6}\sqrt{f_c{}'}\ b_w d = \frac{1}{6}\sqrt{24}\ \cdot 800 \cdot 290 = 189.43\ kN$$

$$\emptyset V_C = 0.75 \cdot 189.43 = 142.07\ kN$$

Check cases:

Check Case III:

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

$V_{s,min}$ **is the maximum of:**

$$V_{s,min} = \frac{1}{16} \cdot \sqrt{24}\ \cdot 800 \cdot 290 = 71.04\ kN$$

$$V_{s,min} = \frac{1}{3} \cdot 800\ \cdot 290 = 77.33\ kN \qquad - controled$$

$$V_u = 304.2\ kN > \emptyset\left(V_c + V_{s,min}\right) = 0.75\ (189.43 + 77.33) = 200.07\ kN$$

Case III not satisfied.

Check Case IV:

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

$V_C = 189.43\ kN$

$$V_{s'} = \frac{1}{3}\sqrt{f_c{}'}\, b_w\, d = \frac{1}{3} \times \sqrt{24}\ \times 800 \times\ 290 = 378.9\ kN$$

$\emptyset\left(V_c + V_{s,min}\right) =\ 200.07\ kN < V_u = 304.2\ kN \leq\ \emptyset\left(V_c + V_{s'}\right) = 426.25 \quad - OK$

Shear reinforcement are required

$$V_{s'} = \frac{V_u -\ \emptyset\, V_c}{\emptyset} = \frac{304.2 - 142.07}{0.75} = 216.2\ kN$$

Use $\emptyset 10\ \ 2\ legs\ with\ A_v = 157.1\ mm^2$

$$S_{req} = \frac{420 \times 157.1 \times 290}{216.2 \times 10^3} = 88.5\ mm$$

But:

$$S_{max} \leq \frac{d}{2} \quad or \quad S_{max} \leq\ 600\ mm$$

Select $4\emptyset 10\ @10\ cm$ stirups

4.6 Design of Column :

4-6-1 Design Data:

The following table and figures gives the design parameters of column group G:

$Dead\ load\ (service)$	$4000\ kN$
$Live\ load\ (service)$	$1600\ kN$
$Length$	$4.2\ m$
k	$1\ (Braced)$
$b * h$	$80 * 65\ cm$
f_y	$420\ Mpa$
f'_c	$24\ Mpa$
Concrete cover	$40\ mm$
$Bar\ size$	$\emptyset 25\ mm$
$Type\ of\ load$	$Concentrically\ Loaded$

Table (4-5): Design Data of column group G .

4-6-2 Factored Loads:

$P_u = 1.2\ D + 1.6L$

$P_u = 1.2\ (4000) + 1.6(1600) = 7360\ kN$

4-7-3 Selecting Column Dimenssion:

Assum Ast = 0.02Ag

$\emptyset Pn, max = \emptyset 0.80[0.85. fc(Ag - Ast) + Ast. Fy]$

$7360 * 10^3$ = **0.65*0.80[0.85*24*(Ag-0.02Ag)+(0.02Ag*420)]**

$Ag = 498515.3\ \mathrm{mm}^2$

take h = 800

b = 498515.3/800 = 623 **take b = 650 mm**

Ag= 800*650 =520000 mm^2

$$\emptyset Pn, max = \emptyset 0.80[0.85.24(240000 - Ast) + Ast.420]$$

Ast= 8873.48 mm^2

Use (10Ǿ25 with As= 9817.47 mm^2 >**Ast= 8873.48** mm^2)

$$\rho = Ast/Ag$$

9817.47/520000 = 0.0188

Design of ties :

use ties Ǿ10 with spacing of ties shall not exceed the smallest of :

1- 48 times tie diameter , 48d_s = 48*10 = 480 mm.

2- 16 times the longitudinal bar diameter , 16d_b = 16*25 = 400 ***- control***

3- the least dimension of the column = 650 mm

use Ǿ10 @ 200 mm

Check for code req. :

1- Clear spacing between longitudinal bar :

Clear space = $\frac{400-40*2-10*2-25*5}{4}$ **= 106.25 mm > 40mm**

> 1.5d_b = 1.5*25 = 37.5

2- 0.01 < $\rho g = 0.0188$ **< 0.08 -ok**

3- Number of bars 20 > 4- for rectangular section -ok

4- Minimum tie diameter Ǿ10 for Ǿ25 - ok

5- Spacing of tie s = 200mm -ok

6- Arrangement of ties 106.25 < 150mm -ok

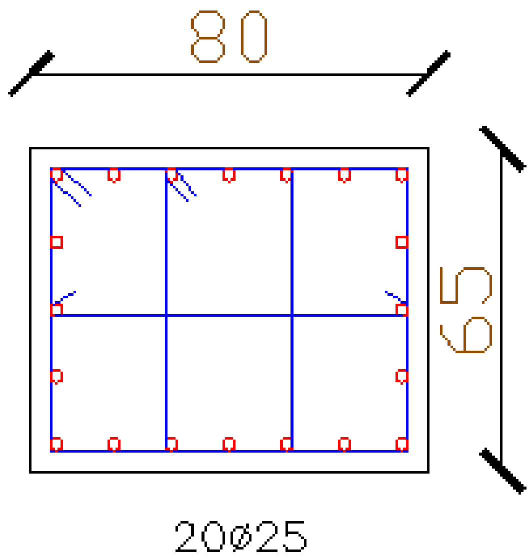


Figure (4-18): Column detailing.

4-7 | **Design of Staircase:**

live load of $L_l = 4 \left(\frac{kN}{m^2}\right)$, *assuming rise of* 150 *mm* ,

and run of 300 *mm* , $f_{c'} = 24\ Mpa$, $f_y = 420\ Mpa$.

4-7-1 plan and materials of stair:

The following figure demonstrate the plan of stair that we consider to design it figure (4-19) which is carries a uniform

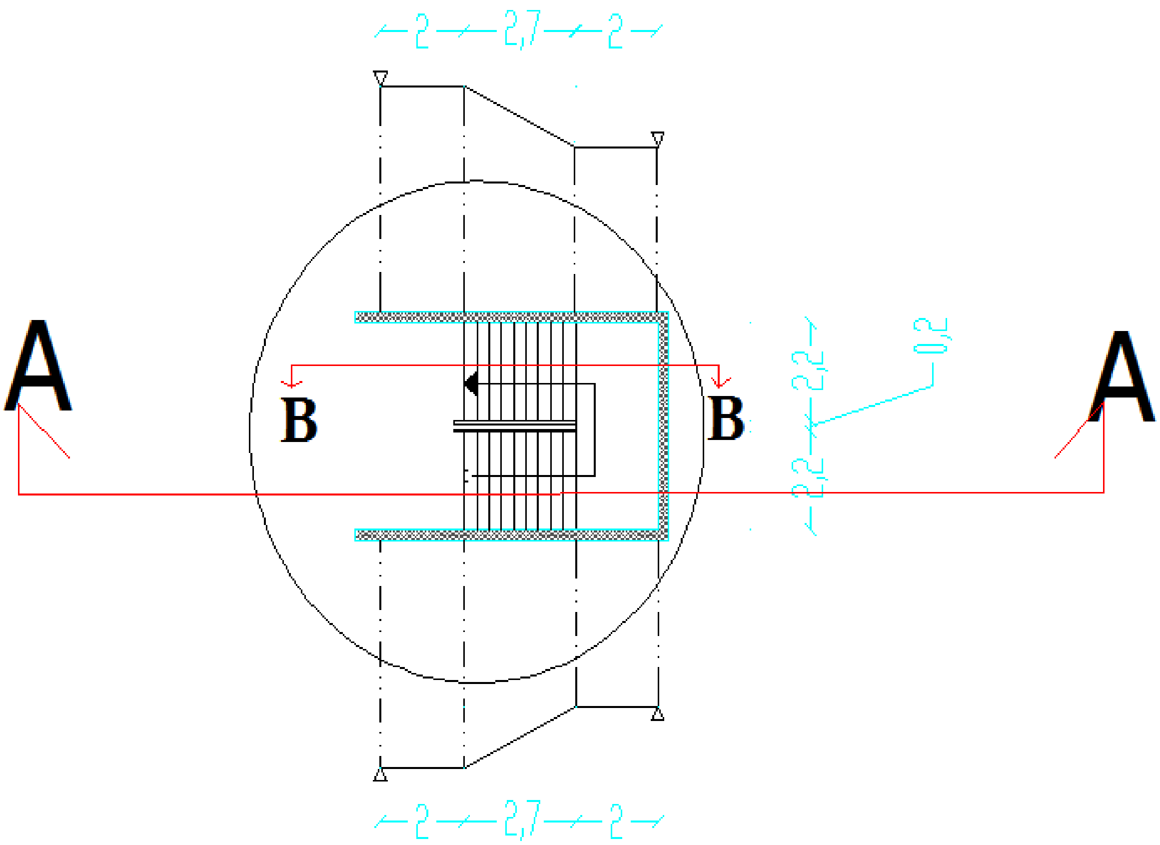


Figure (4-19): Stair Plan and structural system.

4-7-2 Structural system and minimum thickness:

1. **The structural system of this stair was taken as a simply supported (one-way solid slab) since that the flight of stair will be supported at the ends of upper and lower landings.**
2. **Minimum Slab thickness for deflection is (for simply supported one-way solid slab) is** $h_{min} = \frac{L}{20} = \frac{2.7+2}{20} = 23.5\ cm$ **, but in this case presented here where the slab ends are cast white the supporting beams and additional negative reinforcement is provided , minimum thickness can be assumed to be** $h_{min} = \frac{L}{28} = \frac{2.7+2}{28} = 16.8\ cm.$

$$Take\ h_{min} = 25\ cm$$

4-7-3 Loads and Reactions calculations:

The applied live loads are based on the plan area (horizontal projection), while the dead load is based on the sloped length. To transform the dead load into horizontal projection the figure below explains how figure (4-20).

$$\theta = tan^{-1}\left(\frac{rise}{run}\right) = tan^{-1}\left(\frac{150}{300}\right) = 26.6^{\circ}$$

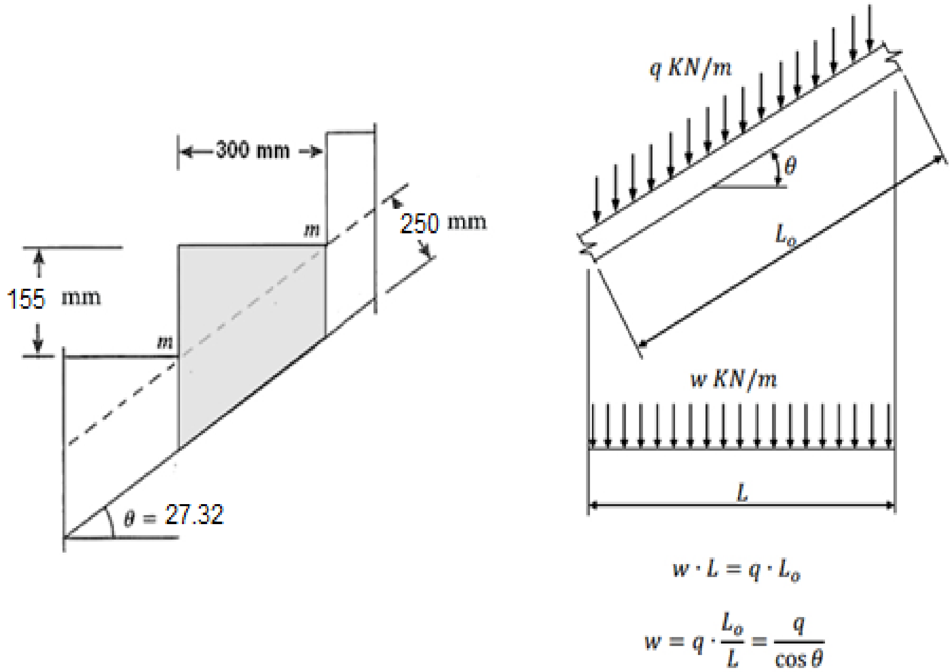


Figure (4-20): Transformation of dead load into horizontal projection.

- ## Flight Dead Load computation:

Table (4-6) shows Dead Load calculations on Flight of stair:

Dead Load Form	Unit weight γ $(\frac{kN}{m^3})$	w $(\frac{kN}{m})$
Tiles	27	$23 \times \left(\frac{0.150 + 0.35}{0.3}\right) \times 0.03 \times 1 = 1.15$
Mortar	22	$22 \times \left(\frac{0.150 + 0.35}{0.3}\right) \times 0.03 \times 1 = 1.1$
Stair steps	25	$\frac{25}{0.3} \times \left(\frac{0.150 \times 0.3}{2}\right) \times 1 = 1.875$
Reinforced concrete (solid slab)	25	$\frac{25 \times 0.25 \times 1}{cos\ 26.6} = 7$
plaster	22	$\frac{22 \times 0.03 \times 1}{cos\ 26.6} = 0.74$
$\sum$ ***Tota Dead loads*** kN/m		11.9

Table (4-6) Dead Load calculations on flight.

- **Landing Dead Load computation:**

Table (4-7) shows Dead Load calculations on Landing of stair:

Dead Load Form	Unit weight γ $(\frac{kN}{m^3})$	$\gamma \times \delta \times 1$ $(\frac{kN}{m})$
Tiles	23	$23 \times 0.03 \times 1 = 0.69$
Mortar	22	$22 \times 0.03 \times 1 = 0.66$
Reinforced concrete (solid slab)	25	$25 \times 0.25 \times 1 = 6.25$
plaster	22	$22 \times 0.03 \times 1 = 0.66$
$\sum$ *Tota Dead loads kN/m*		7.6

Table (4-7) Dead Load calculations on Landing.

- **Live Load:** $L_l = 4 \left(\frac{kN}{m^2}\right)$.
- **Total Factored Load:** $w = 1.2\ D_L + 1.6\ L_l$

For flight : $w = 1.2\ (11.9) + 1.6(4) = 20.7\ \left(\frac{kN}{m}\right)$.

For Landing : $w = 1.2\ (7.6) + 1.6(4) = 15.5\ \left(\frac{kN}{m}\right)$.

4-7-4 Design of flight 1:

The support reaction of flighting is:

$$\frac{[(7.75*4)+(20.7*2.7)]}{2} = 43.4 \left(\frac{kN}{m}\right)$$

. as shown in figure (4-31).

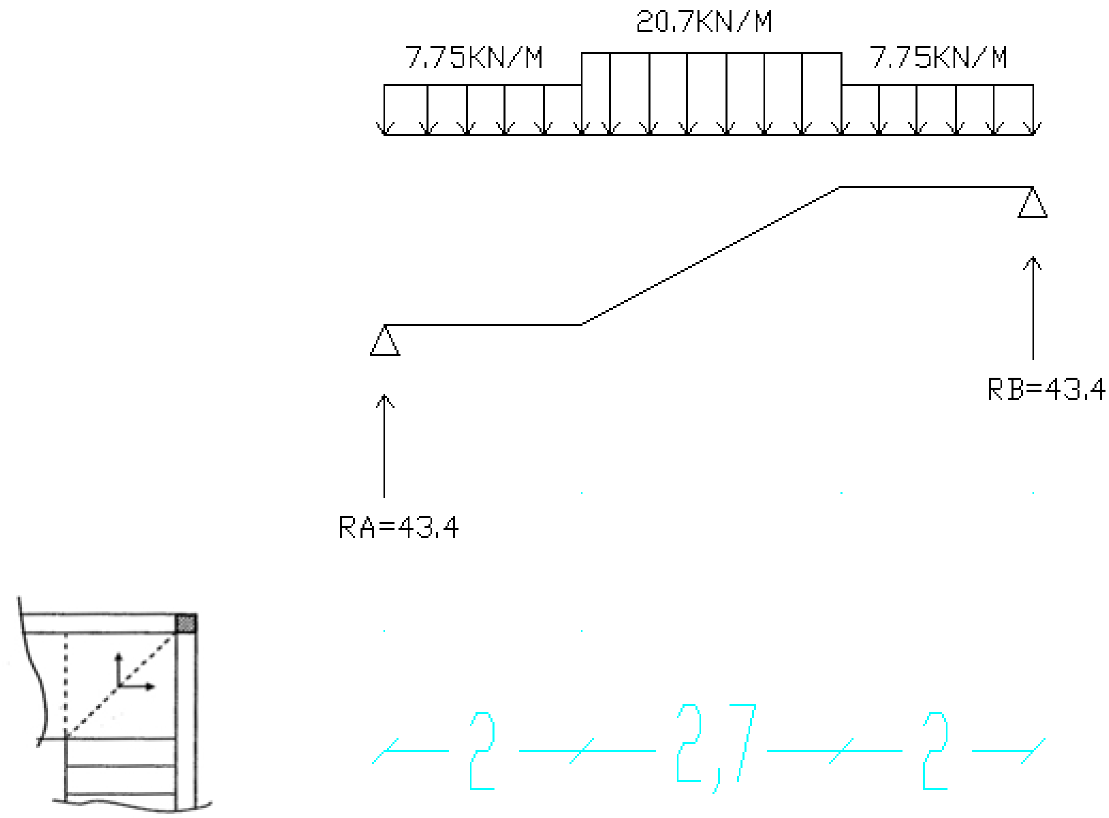


Figure (4-21): Loads and reactions on statically system of flight.

Shear and moment calculations:

- **Check for shear strength:**

Assume bar diameter $\emptyset 14$ $for\ main\ rinforcemnt.$

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

Assume wall width 25 cm

$$V_u = 43.4 - 7.75 \times (0.150 + 0.223) = 40.4\ \ kN$$

$$V_c = \frac{1}{6}\sqrt{f_{c'}}b_w d = \frac{1}{6} \times \sqrt{24} \times 1000 \times 223 = 182.7\ kN\ ..for\ 1\ m\ strip$$

$$\phi = 0.75 - for\ shear$$

$$\phi V_c = 0.75\ \times 182.7 = 136.55\ kN ..\ for\ 1m\ strip$$

$$V_{u,max} = 40.4\ kN\ < \frac{1}{2}\phi V_c = 68.27kN$$

$\therefore$ *The thickness of the slab is adequate enough*

- **Calculation of maximum moment and steel reinforcement:**

$$M_{u,max} = 43.4 \times \left(\frac{6.7}{2}\right) - 7.75 \cdot (2) \cdot \left(2 + \frac{2.7}{2}\right) - 20.7.(1.35).\left(\frac{1.35}{2}\right)$$
$$= 74.6\ kN.m\ /\ \mathrm{m}$$

$$assume\ bar\ diameter\ \emptyset 14\ for\ main\ rinforcemnt\ with\ \ , d = 223\ mm$$

$$R_n = \frac{M_u}{\emptyset bd} = \frac{74.6 \times 10^6}{0.9 \times 1000 \times 223^2} = 1.67\ Mpa\ \ , m = \frac{f_y}{0.85\ f_c{}'} = \frac{420}{0.85 \times (24)} = 20.6$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \cdot 1.67 \cdot 20.6}{420}}\right) = 0.00415$$

$$A_s = \rho bd = 0.00415 \times 1000 \times 223 = 925.5\ \ mm^2$$

$$A_{s,min} = 0.0018bh = 0.0018 \times 1000 \times 250 = 450mm^2$$

$$A_s = 925.5\ mm^2\ > A_{s,min} = 450\ mm^2\ \ \ , use\ \emptyset 14$$

$$Use\ \ 7\emptyset 14@15\ cm\ \ with\ \ A_{s,prov} = 1077\ mm^2 > A_s = 925.5\ mm^2 for\ (1m)\ strip$$

Check maximum step for main reinforcement (the smallest of):

1. $3h = 3 \times 250 = 750\ mm$
2. $450mm.$
3. $S = 380\ \left(\frac{280}{f_s}\right) - 2.5C_c = 380\left(\frac{280}{\frac{2}{3}\times 420}\right) - 2.5 \times 20 = 330mm$

$$S_{max} = 300\left(\frac{280}{f_s}\right) = 300\left(\frac{280}{\frac{2}{3}\times 420}\right) = 300mm \;-controled$$

$$S = 15\ cm \;<\; S_{max} = 30\ cm - OK$$

- **Temperature and shrinkage reinforcement:**

$A_s(temperature\ and\ shrinkagr) = 0.0018bh = 0.0018(1000)(250) = 450\ mm^2$

$Use\ \ 4\emptyset 12@20$

$cm\ \ with\ \ A_{s,prov} = 452.4\ mm^2 > A_s = 450\ mm^2 for\ (1m)\ strip$

Check maximum step for temperature and shrinkage (the smallest of):

1. $5h = 5 \times 250 = 1250\ mm$
2. $450mm. \;\; - controled$

$$S = 20\ cm \;<\; S_{max} = 45\ cm - OK$$

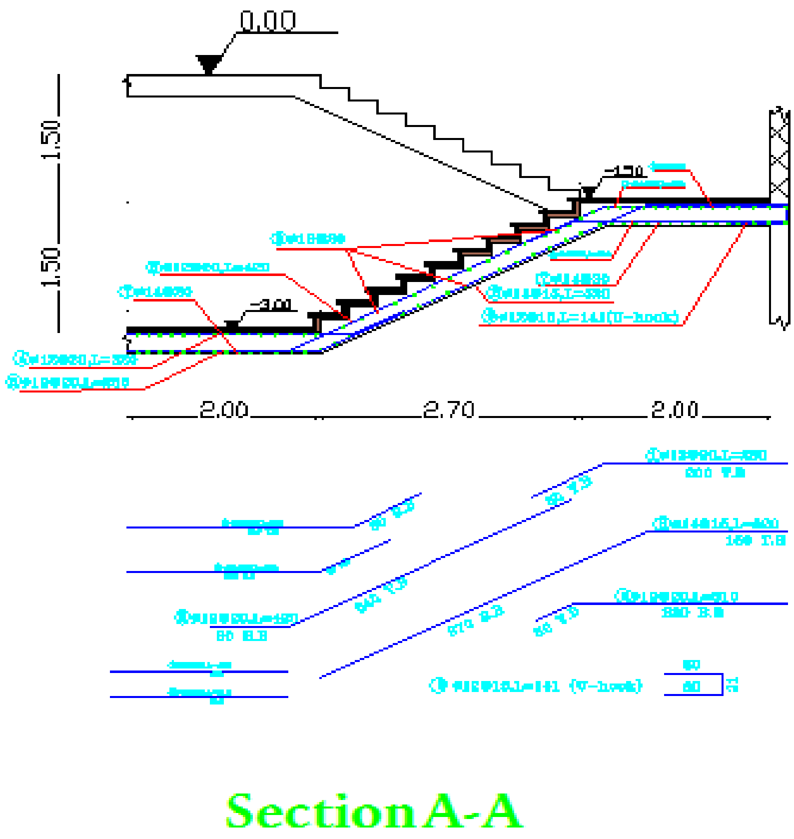


Figure (4-22): Detailing of flight 1.

4-7-5 Design of flight 2:

The support reaction of flighting is:

$$\frac{[(7.75*4)+(20.7*2.7)]}{2} = 43.4\ \left(\frac{kN}{m}\right)$$ **. as shown in figure (4-23).**

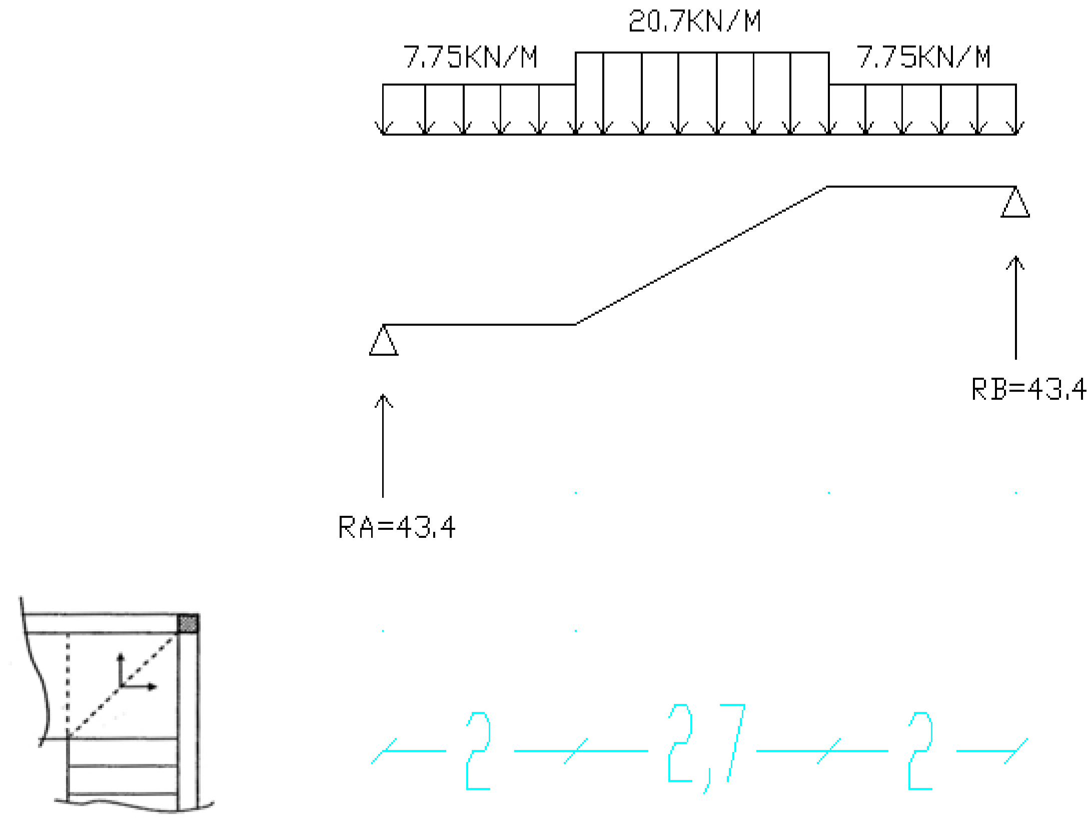


Figure (4-23): Loads and reactions on statically system of flight.

Shear and moment calculations:

- **Check for shear strength:**

  Assume bar diameter $\emptyset 14\ for\ main\ rinforcemnt.$

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

Assume wall width 30 cm

$$V_u = 43.4 - 7.75 \times (0.150 + 0.223) = 40.4\ \ kN$$

$$V_c = \frac{1}{6}\sqrt{f_{c'}}b_w d = \frac{1}{6} \times \sqrt{24} \times 1000 \times 223 = 182.7\ kN \,..\, for\ 1\ m\ strip$$

$$\phi = 0.75 - for\ shear$$

$$\phi V_c = 0.75\ \times 182.7 = 136.55\ kN \,..\ for\ 1m\ strip$$

$$V_{u,max} = 40.4\ kN\ < \frac{1}{2}\phi V_c = 68.27kN$$

$\therefore$ ***The thickness of the slab is adequate enough***

- **Calculation of maximum moment and steel reinforcement:**

$M_{u,max} = 74.6\ kN.m$ / m

$$assume\ bar\ diameter\ \emptyset 14\ for\ main\ rinforcemnt\ with\ \ , d = 223\ mm$$

$$R_n = \frac{M_u}{\emptyset bd} = \frac{74.6 \times 10^6}{0.9 \times 1000 \times 223^2} = 1.67\ Mpa\ \ , m = \frac{f_y}{0.85\ f_c{}'} = \frac{420}{0.85 \times (24)} = 20.6$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \cdot 1.67 \cdot 20.6}{420}}\right) = 0.00415$$

$$A_s = \rho bd = 0.00415 \times 1000 \times 223 = 925.5\ \ mm^2$$

$$A_{s,min} = 0.0018bh = 0.0018 \times 1000 \times 250 = 450mm^2$$

$$A_s = 925.5\ mm^2 > \ A_{s,min} = 450\ mm^2\ \ \ , use\ \emptyset 14$$

$$Use\ 7\emptyset 14@15\ cm\ \ with\ \ A_{s,prov} = 1077\ mm^2 > A_{s,min} = 450\ \ mm^2 for\ (1m)\ strip$$

Check maximum step for main reinforcement (the smallest of):

4. $3h = 3 \times 250 = 750\ mm$

5. $\mathbf{450}\boldsymbol{mm}.$

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

$$\boldsymbol{S_{max} = 300\left(\frac{280}{f_s}\right) = 300\left(\frac{280}{\frac{2}{3}\times 420}\right) = 300mm} \ - \boldsymbol{controled}$$

$\boldsymbol{S = 15\ cm < S_{max} = 30\ cm - OK}$

- **Temperature and shrinkage reinforcement:**

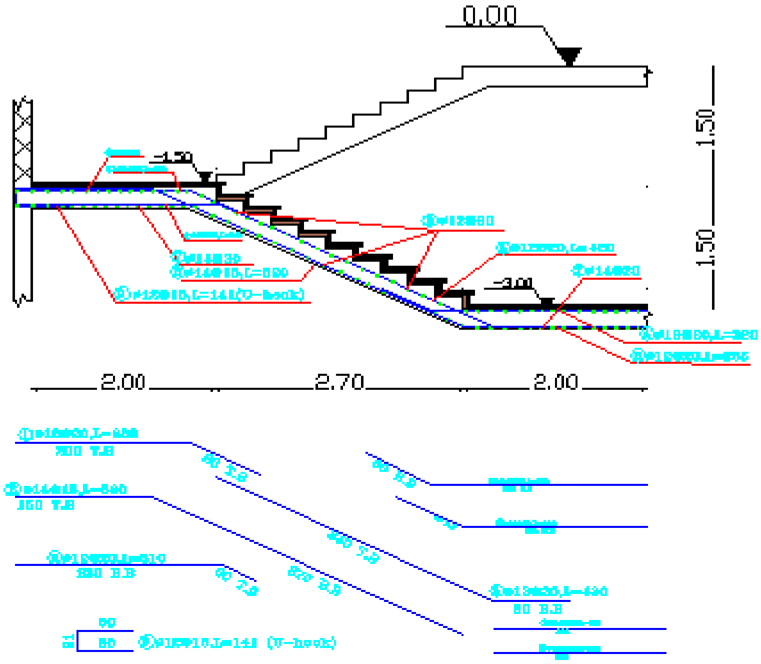
$\boldsymbol{A_s(temperature\ and\ shrinkagr) = 0.0018bh = 0.0018.(1000)(250) = 450\ mm^2}$

$\boldsymbol{Use\ \ 3\emptyset 14@30\ cm\ \ with\ \ A_{s,prov} = 461.7\ mm^2 > A_s = 450\ mm^2 for\ (1m)\ strip}$

Check maximum step for temperature and shrinkage (the smallest of):

3. $\boldsymbol{5h = 5\times 250 = 1250\ mm}$
4. $\boldsymbol{450mm.}$ $\ - \boldsymbol{controled}$

$\boldsymbol{S = 30\ cm \leq S_{max} = 45\ cm - OK}$



Section B-B

Figure (4-24): Detailing of flight 2.

4-7-6 Design of Landing:

The support reaction of flighting is:

$$\frac{[(7.75*2.2)+(7.75*2.2)+(15.5*0.2)]}{2} = \mathbf{18.6}\ \left(\frac{kN}{m}\right)$$. **as shown in figure (4-25).**

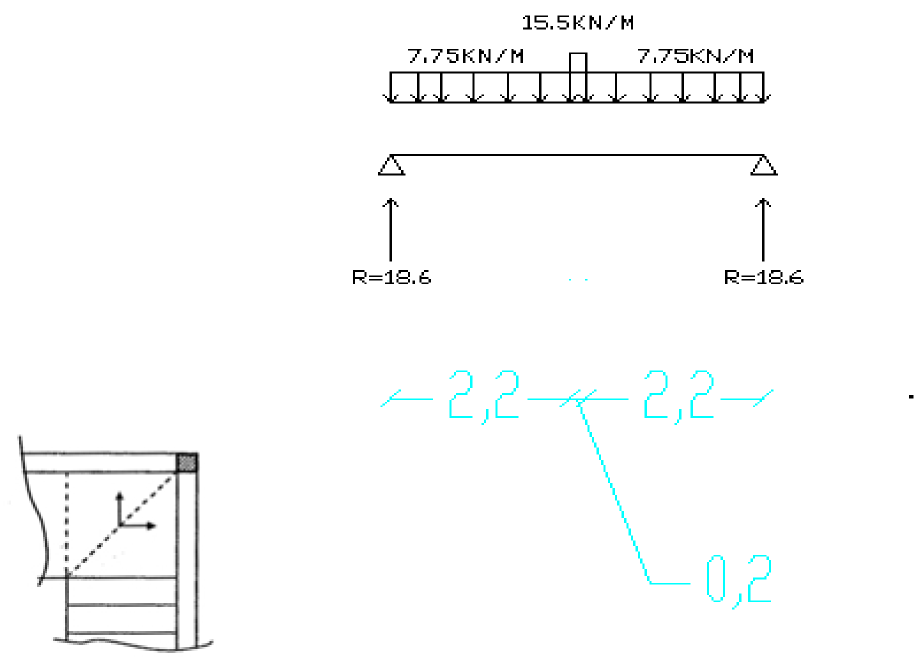


Figure (4-25): Loads and reactions on statically system of landing.

Shear and moment calculations:

- **Check for shear strength:**

Assume bar diameter $\boldsymbol{\emptyset 14}$ $\boldsymbol{for\ main\ rinforcemnt.}$

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

Assume wall width 30 $\boldsymbol{cm}$

$$\boldsymbol{V_u = 18.6 - 7.75 \times (0.150 + 0.223) = 15.7\ \ kN}$$

$$\boldsymbol{V_c = \frac{1}{6}\sqrt{f_{c'}}b_w d = \frac{1}{6} \times \sqrt{24} \times 1000 \times 223 = 182.7\ kN\ ..for\ 1\ m\ strip}$$

$$\boldsymbol{\phi = 0.75 - for\ shear}$$

$$\boldsymbol{\phi V_c = 0.75\ \times 182.7 = 136.55\ kN\ ..\ for\ 1m\ strip}$$

$$V_{u,max} = 15.7\ kN < \frac{1}{2}\phi V_c = 68.27kN$$

$\therefore$ *The thickness of the slab is adequate enough*

- **Calculation of maximum moment and steel reinforcement:**

$M_{u,max} = 3.5\ kN.m$ / m

assume bar diameter $\emptyset 14$ *for main rinforcemnt with* $, d = 223\ mm$

$$R_n = \frac{M_u}{\emptyset bd} = \frac{3.5 \times 10^6}{0.9 \times 1000 \times 223^2} = 0.08\ Mpa\ \ , m = \frac{f_y}{0.85\ f_c{}'} = \frac{420}{0.85 \times (24)} = 20.6$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \cdot 0.08 \cdot 20.6}{420}}\right) = 0.00019$$

$$A_s = \rho bd = 0.00019 \times 1000 \times 223 = 41.6\ \ mm^2$$

$$A_{s,min} = 0.0018bh = 0.0018 \times 1000 \times 250 = 450mm^2$$

$$A_s = 41.6\ mm^2 < A_{s,min} = 450\ mm^2\ \ \ , use\ \emptyset 14$$

$$Use\ 3\emptyset 14@20\ cm\ \ with\ \ A_{s,prov} = 461.7\ mm^2 > A_{s,min} = 450\ \ mm^2 for\ (1m)\ strip$$

Check maximum step for main reinforcement (the smallest of):

7. $3h = 3 \times 250 = 750\ mm$
8. $450mm.$
9. $S = 380\ \left(\frac{280}{f_s}\right) - 2.5C_c = 380\left(\frac{280}{\frac{2}{3}\times 420}\right) - 2.5 \times 20 = 330mm$

$$S_{max} = 300\ \left(\frac{280}{f_s}\right) = 300\left(\frac{280}{\frac{2}{3} \times 420}\right) = 300mm\ \ - controled$$

$$S = 20\ cm < \ S_{max} = 30\ cm - OK$$

- **Temperature and shrinkage reinforcement:**

$$A_s(temperature\ and\ shrinkage) = 0.0018bh = 0.0018(1000)(250) = 450\ mm^2$$

$$Use\ 4\emptyset12@20\ cm\ \ with\ \ A_{s,prov} = 452.4\ mm^2 > A_s = 450\ mm^2 for\ (1m)\ strip$$

Check maximum step for temperature and shrinkage (the smallest of):

5. $5h = 5 \times 250 = 1250\ mm$
6. $450mm.\ \ - controled$

$$S = 30\ cm \leq\ S_{max} = 45\ cm - OK$$

4-8 | **Design isolated Footing(F1):**

4-8-1 Materials and Loads:

Isolated footing that we consider to design with materials of:

$$f_{c'} = 24\ Mpa\ , f_y = 420\ Mpa\ .$$

Dead Load (service) $= 1000\ kN.$
Live Load (service) $= 200\ kN.$
Total services load $=\ 1000 + 200 = 1200\ kN.$
Total Factored load $=\ 1.2(1000) + 1.6(200) = 1520\ kN.$
Column dimension$(\ a \times b) = 40\ cm \times 60\ cm.$
Soil density $=\ 18\ (\frac{KN}{m^3}).$
Allowable bearing capacity $q_{all} = 400\ (\frac{kN}{m^2})$

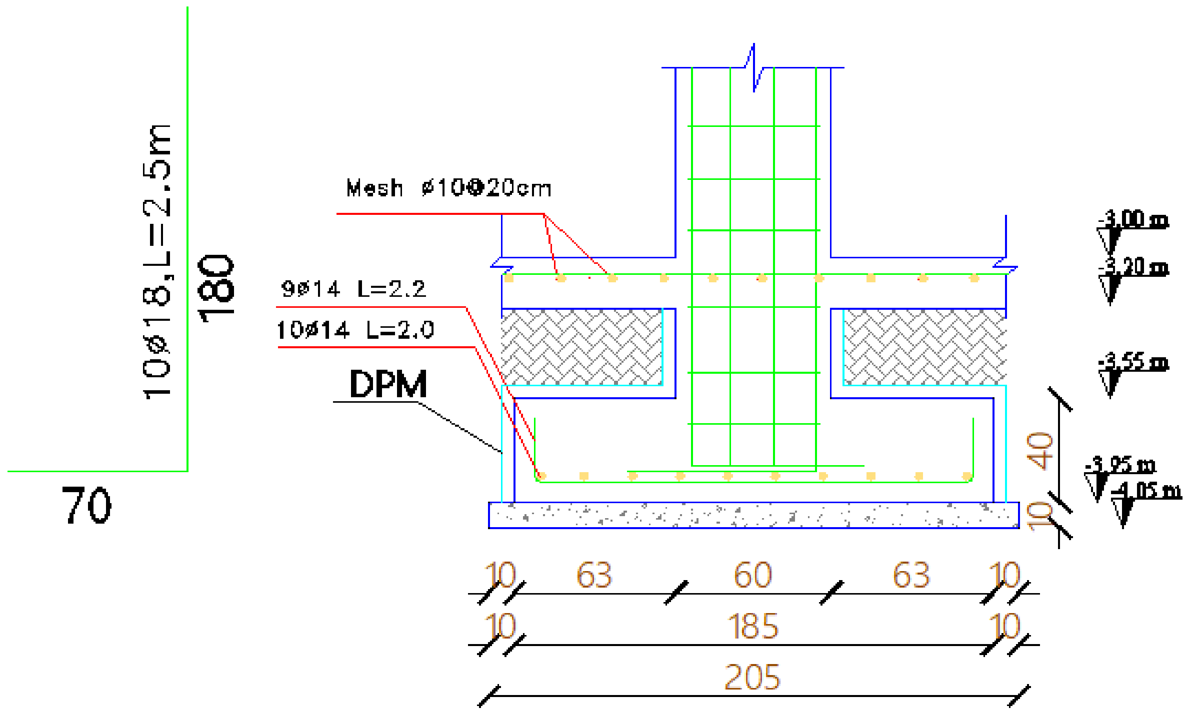


Figure (4-26): Footing Section detailing.

Assume $h = 40\ cm.$

- **Area of footing:**

$$A = \frac{p_t}{q_{all-net}} = \frac{1200}{400} = 3.00\ m^2$$

Assume rect. Footing

Select $B = 1.65\ m$

Select $L = 1.85\ m$

- **Bearing pressure:**

$$q_u = \frac{1520}{1.65 \times 1.85} = 498\ \left(\frac{kN}{m^2}\right)$$

4-8-2 Design:

- **Design of one-way shear strength:**

Critical Section at Distance d From The Face of Column Assume $= 40\ cm$.
Bar diameter $\emptyset 18$ for main reinforcement and 7.5 cm Cover.

$$d = 550 - 75 - 20 = 307\ mm$$

$$V_u = q_u \times \left(\frac{B-a}{2} - d\right) \times L = 498 \times \left(\frac{1.65 - 0.4}{2} - 0.307\right) \times 1.85 = 293\ kN$$

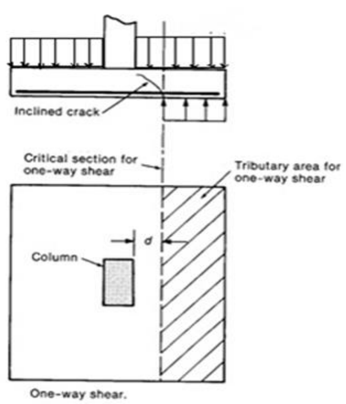


Figure (4-27): one-way shear calculation.

$$\emptyset V_C = \emptyset \times \frac{1}{6} \times \sqrt{f_{c'}} \times b \times d = 0.75 \times \frac{1}{6} \times \sqrt{24} \times 1650 \times 307 = 310.19\ kN$$

$$\emptyset V_C = 310.19 kN > V_u = 293\ kN - Safe$$

- **Design of Tow-way shear strength:**

$$V_u = p_u - FR_b$$

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

$$V_u = 498 \times [(1.65 * 1.85) - (0.6 + 0.307)(0.4 + 0.307)] = 1200.8\ kN$$

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

1. $\emptyset V_C = \emptyset \times \frac{1}{6}\left(1 + \frac{2}{\beta_c}\right) \times \sqrt{f_{c'}} \times b_\circ \times d$
2. $\emptyset V_C = \emptyset \times \frac{1}{12}\left(\frac{\alpha_s}{\frac{b_\circ}{d}} + 2\right) \times \sqrt{f_{c'}} \times b_\circ \times d$
3. $\emptyset V_C = \emptyset \times \frac{1}{3} \times \sqrt{f_{c'}} \times b_\circ \times d$

Where:

$$\beta_c = \frac{column\ Length\ (a)}{column\ width\ (b)} = \frac{60}{40} = 1.5$$

$$b_\circ = Perimeter\ of\ critical\ section\ taken\ at\ (d/2)\ from\ the\ loaded\ area.$$

$$= 2 \times (0.6 + 0.307) + 2 \times (0.4 + 0.307) = 3.228\ m$$

$$\alpha_s = 40\ for\ interior\ coulmn$$

Substituting values in equations:

$$\emptyset V_C = 0.75 \times \frac{1}{6}\left(1 + \frac{2}{1.5}\right) \times \sqrt{24} \times 3228 \times 307 = 1416\ kN$$

$$\emptyset V_C = 0.75 \times \frac{1}{12}\left(\frac{40 * 0.307}{3.228} + 2\right) \times \sqrt{24} \times 3228 \times 307 = 1761.17\ kN$$

$$\emptyset V_C = 0.75 \times \frac{1}{3} \times \sqrt{24} \times 3228 \times 307 = 1213.72\ kN - CONTROL$$

$$\emptyset V_C = 1213.72\ kN > V_u = 1200\ kN$$

- **Design Bending moment for long dirction:**

  Critical Section at the Face of Column

$$select\ \emptyset 18$$

d = 400-75-18/2 = 316mm

$$M_u = 498 \times 1.85 \times 0.73 \times \frac{0.73}{2} = 245.5\ kN.m$$

$$R_n = \frac{M_u}{\emptyset bd^2} = \frac{245.5 \times 10^6}{0.9 \times 1850 \times 316^2} = 1.48\ MPa$$

$$m = \frac{420}{0.85 \times 24} = 20.58$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.58}\left(1 - \sqrt{1 - \frac{2 \times 20.58 \times 1.48}{420}}\right) = 0.00266$$

$$A_{S,req} = \rho \times b \times d = 0.00266 \times 1850 \times 316 = 1519.96\ mm^2$$

$$A_{Smin} = 0.0018 \times 1850 \times 400 = 1332\ mm^2$$

$$A_{S,req} = 1519.96\ \ mm^2 > A_{Smin} = 1332\ \ mm^2 - OK$$

Check maximum step (S) is the smallest of:

1. $3h = 3 \times 400 = 1200\ mm$
2. $450\ mm - control$

$$Use\ 10\emptyset 14\ with\ A_{s,prov} = 1539.38\ mm^2 > A_{S,req}\ 1519.96\ \ mm^2$$

S=1850-75*2-10*14 / 9 = 173.33 mm

S=173.33 < Smax =450 mm, select S=150mm

Check for strain:

$$a = \frac{A_{s.}f_y}{0.85b\ f'_c} = \frac{1539.38 \times 420}{0.85 \times 1850 \times 24} = 17.13\ mm$$

$$c = \frac{a}{\mathcal{B}_1} = \frac{17.13}{0.85} = 20.15\ mm$$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{316-20.15}{20.15}\right) = 0.044 > 0.005\ \ldots\ldots 0k$$

- **Design Bending moment for short dirction:**

Critical Section at the Face of Column

$select\ \emptyset 14$

d = 400 -75 - 14 -14/2 = 304mm

$$M_u = 498 \times 1.65 \times 0.73 \times \frac{0.73}{2} = 218.94\ kN.m$$

$$R_n = \frac{M_u}{\emptyset bd^2} = \frac{218.94 \times 10^6}{0.9 \times 1650 \times 304^2} = 1.59\ MPa$$

$$m = \frac{420}{0.85 \times 24} = 20.58$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.58}\left(1 - \sqrt{1 - \frac{2 \times 20.58 \times 1.59}{420}}\right) = 0.0025$$

$$A_{S,req} = \rho \times b \times d = 0.003 \times 1650 \times 304 = 1254\ mm^2$$

$$A_{Smin} = 0.0018 \times 1650 \times 400 = 1188\ mm^2$$

$$A_{S,req} = 1254\ \ mm^2 > A_{Smin} = 1188\ mm^2 - OK$$

Check maximum step (S) is the smallest of:

3. $3h = 3 \times 400 = 1200\ mm$
4. $450\ mm - control$

$$Use\ 9\emptyset 14\ with\ A_{s,prov} = 1385.44\ mm^2 > A_{S,req} = 1254\ \ mm^2$$

S=1650-75*2-9*14 / 8 = 188.625 mm

S=188.625 < Smax =450 mm, select S=150mm

Check for strain:

$$a = \frac{A_{s.f_y}}{0.85b\, f_c'} = \frac{1385.44 \times 420}{0.85 \times 1650 \times 24} = 17.29\, mm$$

$$c = \frac{a}{\mathcal{B}_1} = \frac{17.29}{0.85} = 20.34\, mm$$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{304-20.34}{20.34}\right) = 0.042 > 0.005\ \ldots\ldots 0k$$

- **<u>Development Length In Footing :-</u>**

Tension Development Length In Footing :-

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

$Assume\ Ktr = 0\ (No\ stripes)$

$$cb = 50 + \frac{14}{2} = 57mm\ \ Or\ cb = \frac{150}{2} = 75\, mm$$

$$\frac{ktr+cb}{db} = \frac{0+57}{14} = 4.07 > 2.5$$

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

$Ld_{T\,req} = \frac{9}{10} * \frac{420}{1*\sqrt{24}} * \frac{1*1*0.8}{2.5} * 14 = 345.67\, mm >$ **300mm**

$Ld_{T\ available} = \frac{1850-600}{2}$ -75= 600 mm

$Ld_{T\ available}$ = 600 mm >$ld_{req} = 345.67\ mm$…….. OK

Compression Development Length In Footing :-

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

$Ld_{Creq} = \frac{0.24*420*14}{\sqrt{24}}$ = 288.06> 0.043*420*14 = 252.84>200mm

Ld_{Creq}= 288.06 mm

$Ldc_{available}$ = 600 – 75 – 14 – 14 = 497mm >Ld_{Creq}= 288.06 mm …….. Ok

Lap Splice of Dowels in Column:-

Lsc = $0.071 \times fy \times db = 0.071 \times 420 \times 14$ = 417.48 mm > 300 mm

Select Lsc = 500 mm

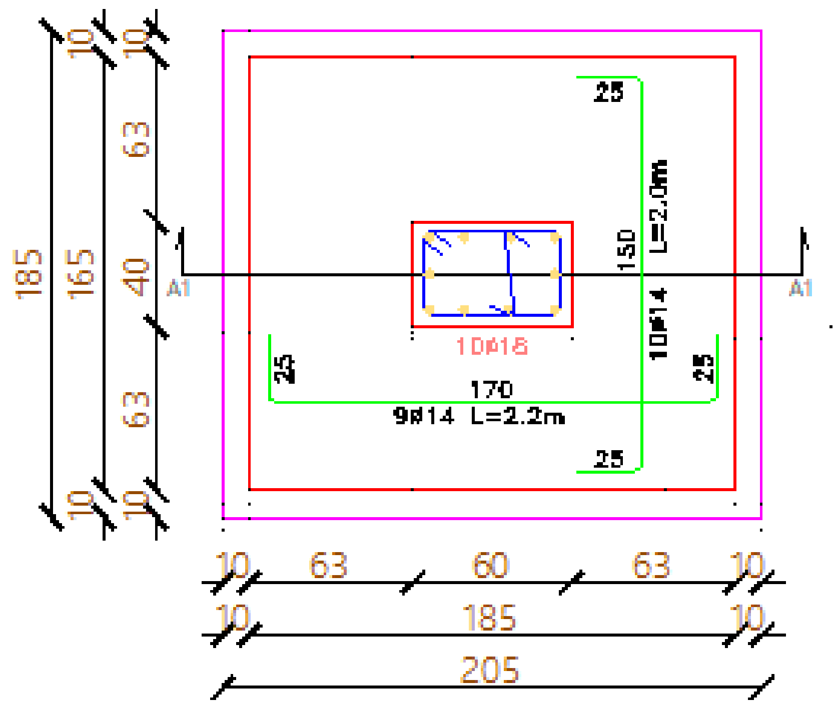


Figure (4-28): Detailing of footing.

4.9 Design of Basement Wall:

❖ **Material :**

⇒ **concrete B300 Fc' = 24N/mm^2**
⇒ **Reinforcement Steel Fy = 420 N/mm^2**

$\emptyset = 35°$ $\quad\quad$ $\gamma = 18.00 KN/m^3$

$$Ko = 1 - \sin \emptyset$$
$$= 1 - \sin 35$$
$$= 0.426$$

✓ **Load on basement wall:**

For 1m length of wall:

*** Weight of backfill:**

$q1 = Ko * \gamma * h$

= 0.426*18.0*3.0 =23.0 KN/m

*** Load from live load:**

LL=5 KN/m2

$q2 = Ko * LL$

= 0.426 * 5 =2.13 KN/m

q_1 due to triangular loading, q_2 is due to uniform loading.

By using structural analysis software (Atir) we obtain the shear and moment diagrams.

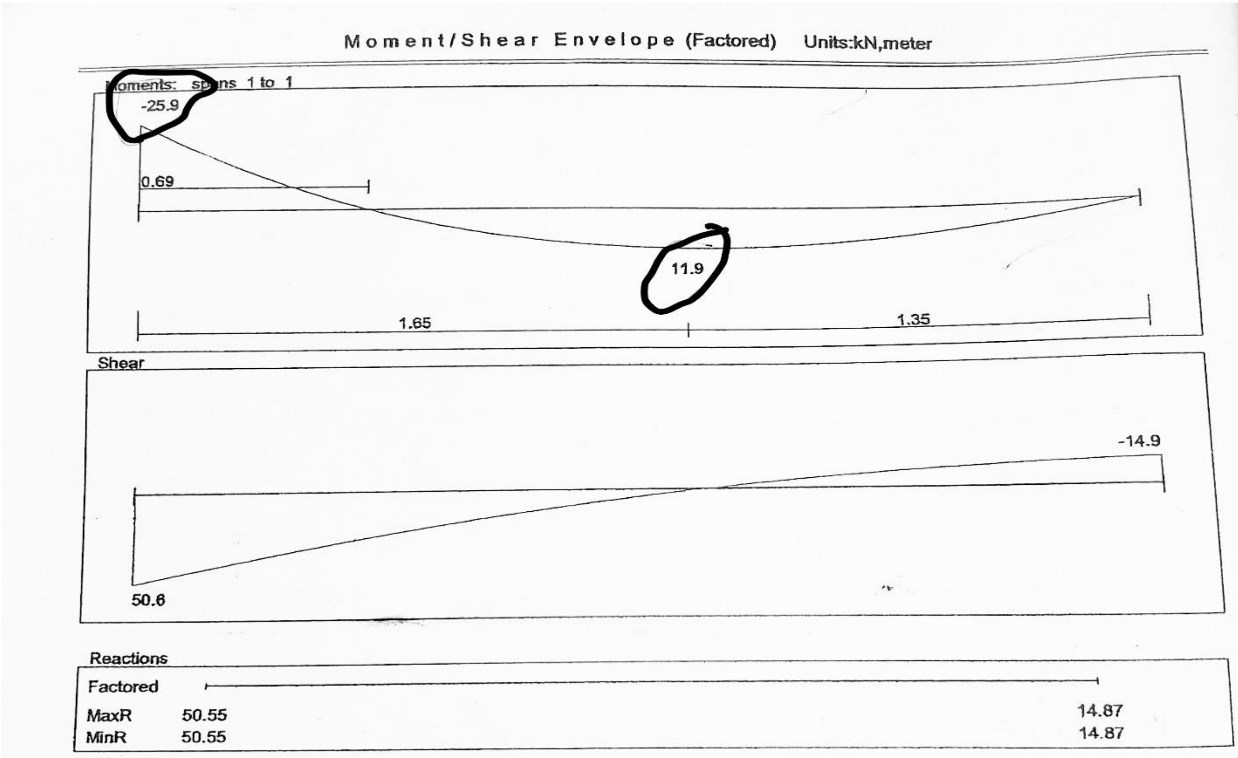


Figure (4-29): Moment and Shear envelops of basement.

✓ Design of the shear force:

Assume h = 300 mm,

$\mathbf{d = 300 - 20 - 14 = 266\ mm}$

Vmax =50.6 KN

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

= 162.89 KN , Vu $< \phi Vc$

No shear Reinforcement is required.

✓ Design of bending moment:

For Mu max = 25.91 KN.m :

$$\mathbf{Mn = \frac{Mu}{0.9} = \frac{25.91}{0.9} = 28.788\ KN.m}$$

$$\mathbf{Rn} = \frac{\mathbf{Mn} * \mathbf{10^6}}{\mathbf{b} * \mathbf{d^2}} = \frac{\mathbf{28.788} * \mathbf{10^6}}{\mathbf{1000} * \mathbf{266^2}} = \mathbf{0.407\ Mpa}$$

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

$$\boldsymbol{\rho} = \frac{\mathbf{1}}{\mathbf{m}} * \left(\mathbf{1} - \sqrt{\mathbf{1} - \frac{\mathbf{2} * \boldsymbol{R}\mathbf{n} * \mathbf{m}}{\mathbf{Fy}}}\right)$$

$$= \frac{\mathbf{1}}{\mathbf{20.59}} * \left(\mathbf{1} - \sqrt{\mathbf{1} - \frac{\mathbf{2} * \mathbf{0.407} * \mathbf{20.59}}{\mathbf{420}}}\right)$$

$$= \mathbf{0.979} * \mathbf{10^{-3}}$$

$$\mathbf{Asreq} = \boldsymbol{\rho} * \mathbf{b} * \mathbf{d} = \mathbf{0.979} * \mathbf{10^{-3}} * \mathbf{1000} * \mathbf{266} = \mathbf{2.6\ cm^2/m}$$

$\mathbf{Amin(for\ flexture)} = \mathbf{0.25}\frac{\sqrt{fc}}{fy}\boldsymbol{bw\ d}$ **= 7.75** $\mathbf{cm^2/m} \geq \frac{\mathbf{1.4}}{fy}\ \boldsymbol{bw\ d}$ **= 8.86** $\mathbf{cm^2/m}$

$$\therefore\ \mathbf{Amin}\ \geq \boldsymbol{Areq}$$

n = $\frac{\mathbf{886}}{\mathbf{113.1}}$ **= 7.8 , take 8Ø12/m or Ø12@125 mm**

→ Select Ø12@12. 5cm/m

Vertical reinforcement at compression face:

As req =As min = 3. 60 $\mathbf{cm^2/m}$

Ø12@10cm/m

✓ **<u>Design of the horizontal reinforcement:</u>**

$$\mathbf{Asmin}\ = \mathbf{0.0012} * \mathbf{b} * \mathbf{h} = \mathbf{0.002} * \mathbf{1000} * \mathbf{300} = \mathbf{6\ cm^2/m}$$

SelectØ12@30cm/m, in two layer.

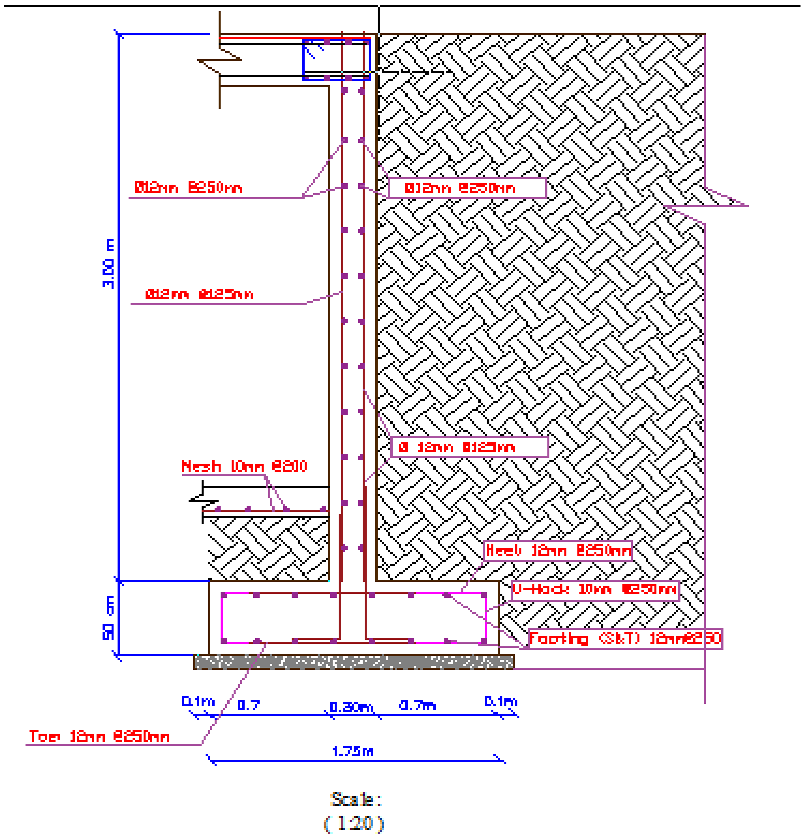


Figure (4-30): Detailing of basement.

4.10 Design of Shear Wall (SW,4)

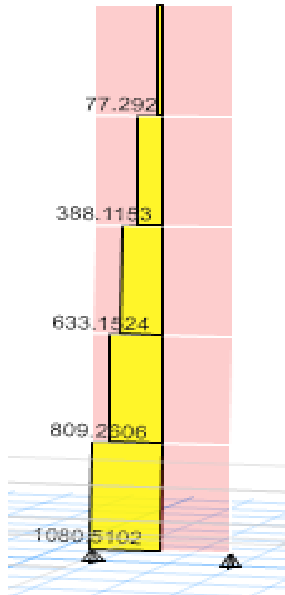


Fig 4.31: Shear Diagram of Shear Wall.

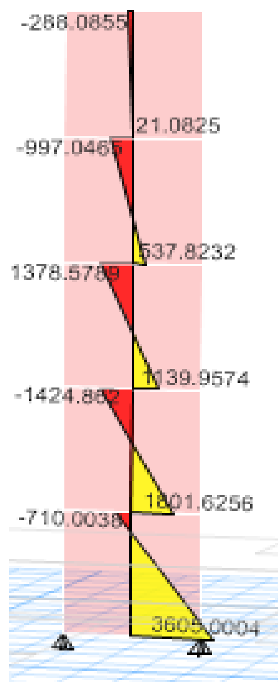


Fig 4.32: Moment Diagram of Shear Wall.

✓ **Material and Sections:- (From Shear Wall 4)**

⇒ **concrete B300** **Fc' = 24 N/mm^2**

⇒ **Reinforcement Steel** **Fy = 420 N/mm^2**

⇒ **Shear Wall Thickness** **h = 25 cm**

⇒ **Shear Wall Width** **Lw = 4.0 m**

⇒ **Shear Wall Height** **Hw = 4.2 m**

✓ **Design of Horizontal Reinforcement:-**

$$\sum Fx = Vu = 1080.5 \ \ KN$$

The critical Section is the smaller of:

$$\frac{lw}{2} = \frac{4.00}{2} = 2.00m....Control$$

$$\frac{hw}{2} = \frac{21}{2} = 10.5m$$

$$storyheight(Hw) = 4.20m.$$

$$d = 0.8 \times Lw = 0.8 \times 4.00 = 3.20m$$

$$\emptyset \boldsymbol{V_{nmax}} = \emptyset \frac{\mathbf{5}}{\mathbf{6}} \sqrt{\boldsymbol{f_c}'} \boldsymbol{hd}$$
$$= \mathbf{0.75 * 0.833 * \sqrt{24} * 250 * 3200 = 2448.5}\ \boldsymbol{KN > V_u} = \mathbf{1080.5}\boldsymbol{KN}$$

V_c **is the smallest of :**

$$1-\ V_c=\frac{1}{6}\sqrt{f_c{}'}hd=\frac{1}{6}\sqrt{24}*250*3200=653.2KN$$ **……..Control**

$$2-\ V_c=0.27\sqrt{f_c{}'}hd+\frac{N_u d}{4l_w}=0.27\sqrt{24}*250*3200+0=1058.2KN$$

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

$$=\left[0.05\sqrt{24}+\frac{4.00\left(0.1\sqrt{24}+0\right)}{1.34}\right]250*3200=1365.9\ KN$$

$\Rightarrow M_u=3605\ KN.m$

$$\frac{M_u}{V_u}-\frac{l_w}{2}=\frac{3605}{1080.5}-\frac{4.00}{2}=1.34$$

Vc =$653.2KN$

Vu = 1080.5 KN > $\frac{1}{2}$ *** 0.75*653.2 = 244.95 KN   Needs reinforcement**

$\emptyset*vc+\emptyset vs=vu$
$\emptyset*vs$**=vu-**$\emptyset*vc$
Vs=vu/$\emptyset-vc$
Vs=1080.5/0.75-653.2= 787.5 KN

$\frac{A_{vh}}{s_h}=\frac{vs}{fyd}=\frac{787.5}{420*3200}=$ **0.000586** $mm^2/\ m$

- Maximum spacing is the least of:

$\frac{Lw}{5}=\frac{4000}{5}$ **= 800 mm**
3*h = 3*250 = 750 mm

450 mm ……. Control

Take ρ **= 0.0025**

Try $\emptyset$**10 (**$A_S=78.5\ mm^2$**) two layers**

$\rho = \frac{A_{vh}}{hS_h} = \frac{2*78.5}{250S_h} =$**0.0025**

S_h**= 251.2mm**

→ use ∅10@200 mm in tow layer

✓ Design of Vertical Reinforcement:-

$\frac{A_{vv}}{S_v} = \frac{A_{vv}}{S_v} = \left[0.0025 + 0.5\left(2.5 - \frac{h_w}{Lw}\right)\left(\frac{A_{vh}}{S_h*h} - 0.0025\right)\right]$***300**

$\left[0.0025 + 0.5\left(2.5 - \frac{21}{4.00}\right)\left(\frac{157}{200*250} - 0.0025\right)\right]$***300**

$\frac{A_{vv}}{S_v} = 0.486$

Try ∅14 ($A_S = 154\ mm^2$) two layers

$\frac{2*154}{S_v} = 0.486$

S_v**= 634 mm**

- Maximum spacing is the least of :

$\frac{Lw}{3}$ **=** $\frac{4000}{3}$ **= 1333.3 mm**

3*h = 3*250 = 750mm

450 mm ……. Control

→ use ∅14@200 mm in tow layer

✓ **Design of Bending Moment:-**

$$A_{st} = \left(\frac{4000}{200}\right) * 2 * 154 = 6160\ mm^2$$

$$w = \left(\frac{A_{st}}{L_w h}\right)\frac{f_y}{f_c{}'} = \left(\frac{6160}{4000 * 250}\right)\frac{420}{24} = 0.1078$$

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

$$\frac{C}{l_w} = \frac{w + \alpha}{2w + 0.85\beta_1} = \frac{0.1078 + 0}{2 * 0.1078 + 0.85 * 0.85} = 0.115$$

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

$$= 0.9[0.5 * 6160 * 420 * 4000(1 + 0)(1 - 0.115)] = 4121.4\ KN \geqq 3605\ KN.m$$

Mub = Mu-∅Mn = 3605 - 4121.4 = -516.4 KN.m

X≥ $\frac{lw}{600*\frac{\Delta h}{hw}}$=$\frac{4000}{600*.1}$=6.67 mm

Lb≥ $\frac{X}{2} = 3.34\ mm$

Since Smallest value of Lb & Mub not requires Boundary.

✓ **Detaling of shear wall (SW4):-**

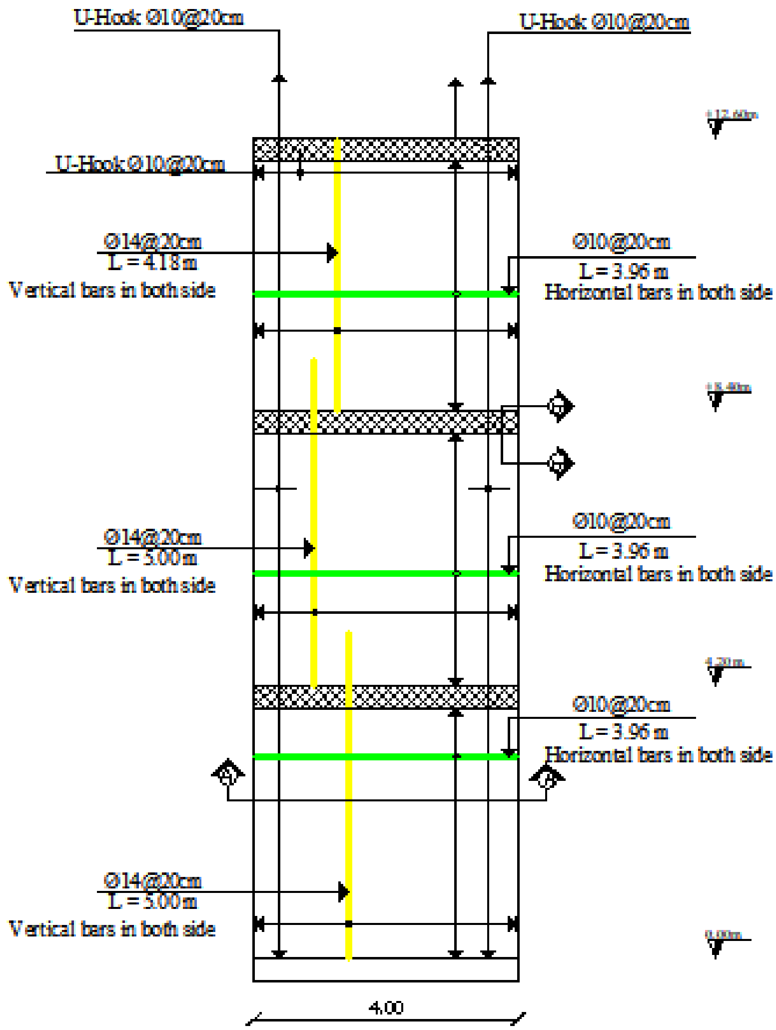


Fig 4.33: Detailing of Shear Wall.

4.11 Design of Retaining Wall:

❖ **Material :**

⇒ **concrete B300** **Fc' = 24N/mm^2**

⇒ **Reinforcement Steel** **Fy = 420 N/mm^2**

$\boldsymbol{\emptyset = 35°}$ $\boldsymbol{\gamma\ \text{soil} = 18.00\text{KN}/\text{m}^3}$ $\boldsymbol{\gamma\ \text{conc} = 25\text{KN}/\text{m}^3}$

$\boldsymbol{\text{coefficient of friction between concrete and soil is } \mu = 0.5}$
$\boldsymbol{\text{allowable soil presssure} = 400\ \text{KN}/\text{m}^3}$
$\boldsymbol{\text{surcharge} = 5\ \text{KN}/\text{m}^2}$

✓ **<u>Size of retaining wall:</u>**

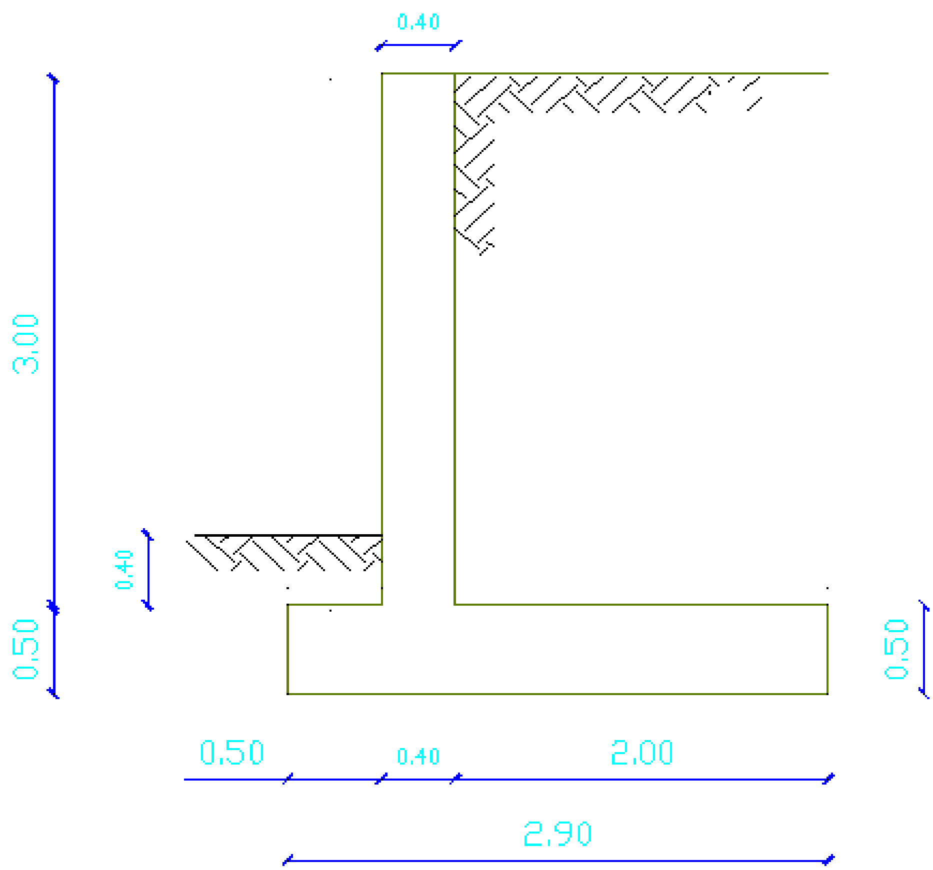


Fig 4.34: Section plan of retaining wall.

$$Ca = \frac{1 - \sin \emptyset}{1 + \sin \emptyset} = \frac{1 - \sin 35}{1 + \sin 35} = 0.271$$

$$Pa1 = Ca\omega h = 0.271 \times 18 \times 3.5 = 17.07 \text{ KN/m}^2$$
$$Pa2 = Cap = 0.271 \times 5 = 1.36 \text{ KN/m}^2$$

$$Ha1 = Ca\frac{wh^2}{2} = \frac{Pa1h}{2} = \frac{17.07 * 3.5}{2} = 30$$

$$Ha2 = Caph = Pa2h = 1.36 \times 3.5 = 4.76 \text{ KN}$$

$$Ha = Ha1 + Ha2 = 30 + 4.76 = 34.76 \text{ KN}$$

$$Cp = \frac{1}{Ca} = 3.7$$

$$Hp = Cp\frac{wh^2}{2} = 3.7\frac{18 * 0.6^2}{2} = 12 \text{ KN/m2}$$

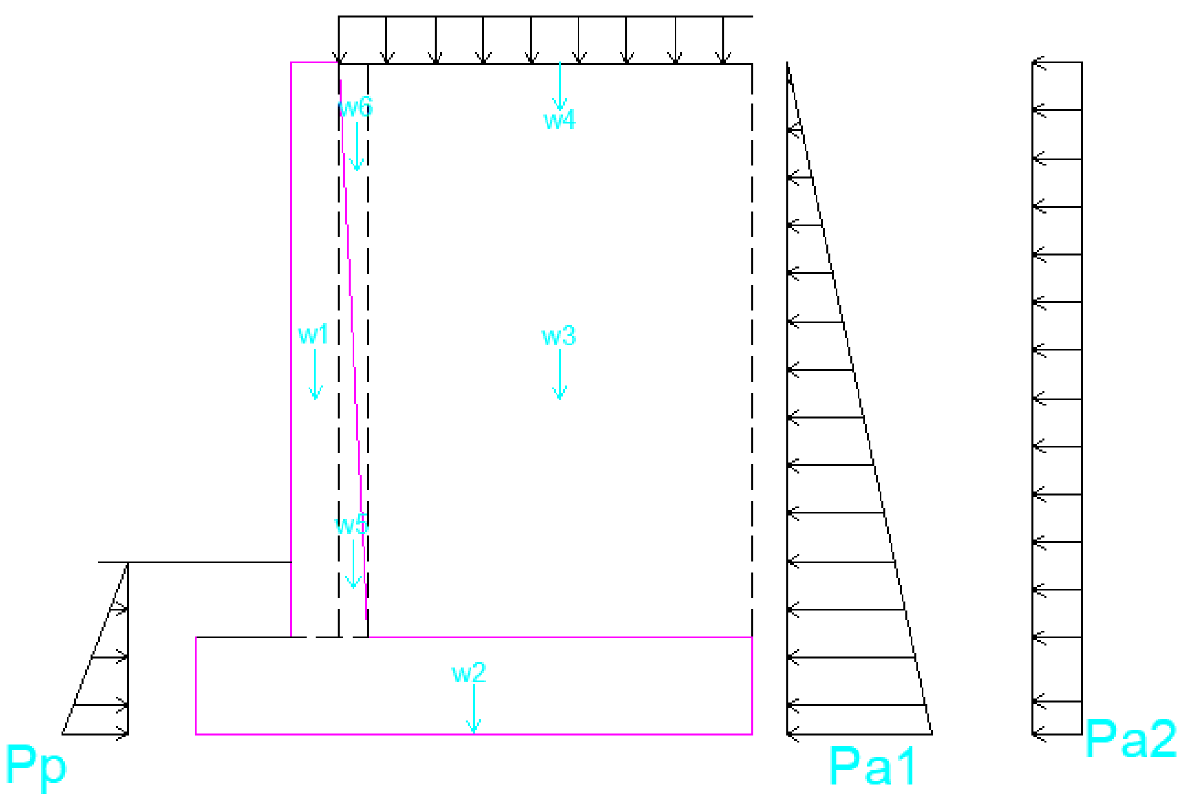


Fig 4.35: loads of Retaining Wall on stem .

Weight [KN]	Arm [m]	Moment [KN · m]
$\omega 1 = 0.25 \times 3 \times 25 = 18.75$	$\frac{0.25}{2} + 0.5 = 0.625$	11.72
$\omega 2 = 0.6 \times 2.9 \times 25 = 43.5$	1.45	63.1
$\omega 3 = 2 \times 3 \times 18 = 108$	1.9	205.2
$\omega 4 = 2.15 \times 5 = 10.75$	1.83	19.6
$\omega 5 = \frac{0.15 \times 3}{2} \times 25 = 5.625$	$\frac{1}{3} \times 0.15 + 0.25 + 0.5 = 0.8$	4.5
$\omega 6 = \frac{0.15 \times 3}{2} \times 18 = 4.05$	$\frac{2}{3} \times 0.15 + 0.25 + 0.5 = 0.85$	3.44

Table (4-8) Moment and Loads Calculation.

$\sum \omega = 190.7$ KN
Mb $= \sum M = 307.6$ KN $\cdot$ m .

$F = \mu R + Hp, = F = 0.5 \times 190.7 + 12 = 107.35$ KN.

factor of safety against sliding is $\frac{F}{Ha} = \frac{107.35}{34.76} = 3.09 > 1.5$ ..ok

The overturning moment is $Mo = 30 \frac{3.5}{3} + 4.76 \frac{3.5}{2} = 43.33$ KN. m

The balancing moment, Mb, taken about the toe end 0 is Mb $= 307.6$ KN $\cdot$ m.

The factor of safety against overturning is $\frac{Mb}{Mo} = \frac{307.6}{43.33} = 7.1 > 2$ ..ok

$$x = \frac{Mb - Mo}{\sum \omega} = \frac{307.6 - 43.33}{190.7} = 1.386$$

The eccentricity is $e = 1.45 - 1.386 = 0.064$ m.

$$\frac{L}{6}=\frac{2.9}{6}=0.483\ m>e=0.064\ m$$

$$I=\frac{1*2.9^3}{12}=2.03\ \text{m}^4 \quad , \quad Area=2.9\ \text{m}^2$$

$$q\ max=\frac{190.7}{2.9}+\frac{190.7*0.064}{2.03}\times 1.45=74.5\ \text{KN/m}^2$$

$$q\ min=\frac{190.7}{2.9}-\frac{190.7*0.064}{2.03}\times 1.45=57\ \text{KN/m}^2$$

- ✓ **Design of stem:**

Shear design

$$Ha1=Ca\frac{wh^2}{2}=0.271\times\frac{18\times 3^2}{2}=22\ \text{KN}$$

$$Ha2 = Caph = 0.271\times 5\times 3 = 4.07\ \text{KN}$$
$$Ha = Ha1 + Ha2 = 22 + 4.07 = 26.07\ \text{KN}$$

$$Vu = 1.6\times 26.07 = 41.7\ \text{KN}$$

$$d=400-75-\frac{20}{2}=315\ \text{mm}$$

$$\phi Vc=\frac{\phi\sqrt{f_c'}*b_w*d}{6}=0.75/6*\sqrt{24}\times 1000\times 315\times 10^{-3}=192.9\ \text{KN}$$

$$Vu = 41.7\ \text{KN} <\frac{1}{2}\phi V=96.5\ \text{KN}$$

The thickness of 40 cm at the stem end is adequate enough.

flexural design

$$Mu = 1.6\times[22\times\frac{3}{3}+4.07\times\frac{3}{2}] = 45\ KN\cdot m$$
Take $\phi = 0.9$ for flexure

$$\mathbf{Rn} = \frac{\mathbf{Mu} * \mathbf{10^6}}{\mathbf{\phi b} * \mathbf{d^2}} = \frac{\mathbf{45} * \mathbf{10^6}}{\mathbf{0.9} * \mathbf{1000} * \mathbf{315^2}} = \mathbf{0.504\ Mpa}$$

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

$$\boldsymbol{\rho} = \frac{\mathbf{1}}{\mathbf{m}} * \left(\mathbf{1} - \sqrt{\mathbf{1} - \frac{\mathbf{2} * \boldsymbol{R}\mathbf{n} * \mathbf{m}}{\mathbf{Fy}}}\right)$$

$$= \frac{\mathbf{1}}{\mathbf{20.59}} * \left(\mathbf{1} - \sqrt{\mathbf{1} - \frac{\mathbf{2} * \mathbf{0.504} * \mathbf{20.59}}{\mathbf{420}}}\right)$$

$$= \mathbf{1.22} * \mathbf{10^{-3}}$$

$$\mathbf{Asreq} = \boldsymbol{\rho} * \mathbf{b} * \mathbf{d} = \mathbf{1.22} * \mathbf{10^{-3}} * \mathbf{1000} * \mathbf{315} = \mathbf{383\ mm^2/m}$$

Try 12 @ 200 mm with As = 565.5 mm2/m

$$\mathbf{Vertical\quad As, min = 0.0012bh = 0.0012 \times 1000 \times 400 = 480\ mm2/m}$$

$$\mathbf{As = 565.5\ mm2/m > As, min, Vertical = 480\ mm2/m \quad OK}$$

→ Select ∅12@20cm/m

Temperature and shrinkage reinforcement: The minimum horizontal reinforcement at the base of the wall according to ACI Code, Section 14.3, is Horizontal

$$\mathbf{Horizontal\quad As, min = 0.002bh = 0.002 \times 1000 \times 400 = 800\ mm2/m}$$

assuming a ∅ 16 bars or smaller.

Because the front face of the wall is mostly exposed to temperature changes, use one − half to two − thirds of the horizontal bars at the external face of the wall and place the balance at the internal face.

$$\mathbf{0.5As = 0.5 \times 800 = 400\ mm2/m}$$

Use ∅ 12 horizontal bars spaced at 200 mm , with As = 565.5 mm2/m

→ Select ∅12@20cm/m

✓ Design of toe:

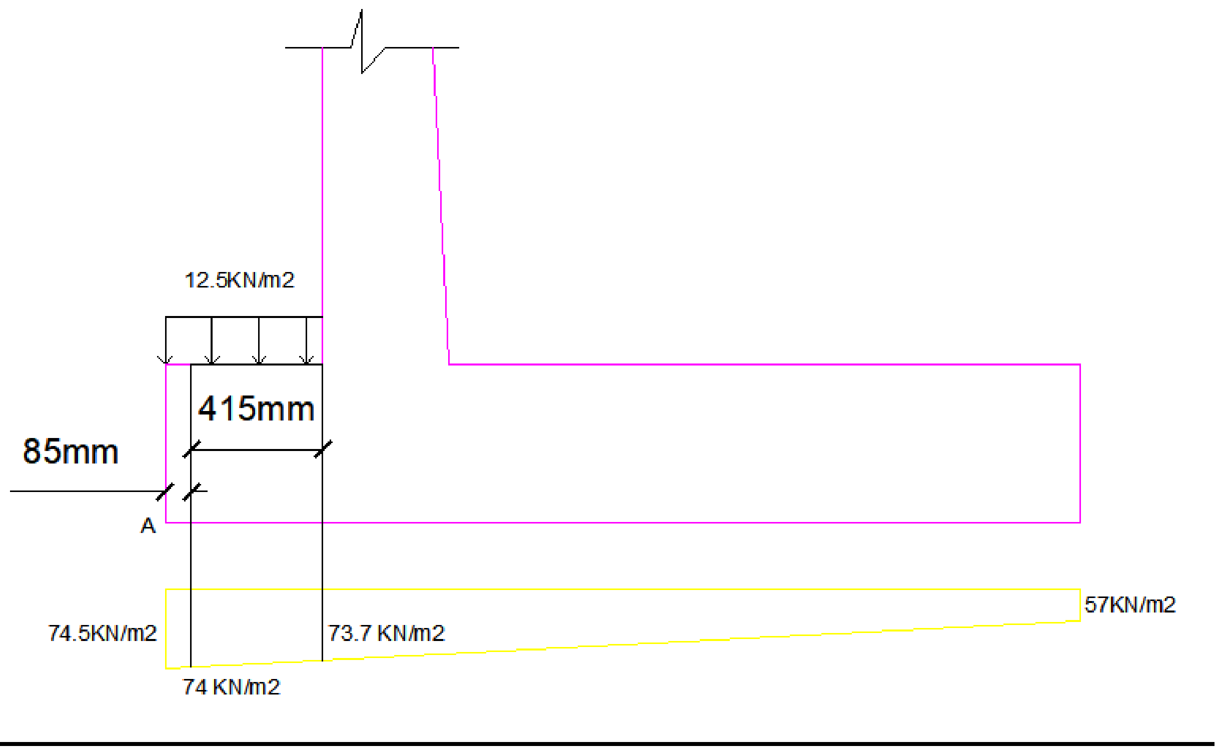


Fig 4.36: Loads of Retaining Wall on toe.

The downward pressure due to self-weight of the toe slab = 0.5 × 25 = 12.5 $KN/m2$

$$The\ slope = \frac{74.5-57}{2.9} = 6.03$$

The pressure at the face (section A)

$$qA = 76.7 - 0.5 \times 6.03 = 73.7\frac{KN}{m2}$$

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

Shear design

The pressure at distance d from the face

$$qud = 74.5 - 0.085 \times 6.03 = 74\ KN/m2$$

$$\mathbf{Vud = 1.6 \times \left(\frac{74.5 + 74}{2} \times 0.085\right) - 0.9 \times 12.5 \times 0.085 = 9.14\ KN}$$

$$\phi Vc = \frac{\phi\sqrt{f_c'} * b_w * d}{6} = 0.75/6 * \sqrt{24} \times 1000 \times 415 \times 10^{-3} = 254.13 \text{ KN}$$

$$\mathbf{Vud = 9.14\ KN < \frac{1}{2}\phi V = 127.07\ KN}$$

The thickness of 50 cm at the toe slab end is adequate enough.

flexural design

$$\boldsymbol{Mu} = \mathbf{1.6 \times \left[\frac{74.5 - 73.7}{2} \times 0.5 \times \frac{2}{3} \times 0.5 + 73.7 \times \frac{0.5^2}{2}\right] - 0.9 \times 12.5 \times \frac{0.5^2}{2}}$$
$$= \mathbf{13.44}\ \boldsymbol{KN \cdot m}$$

Take $\phi = 0.9$ for flexure

$$\mathbf{Rn = \frac{Mu * 10^6}{\phi b * d^2} = \frac{13.44 * 10^6}{0.9 * 1000 * 415^2} = 0.088\ Mpa}$$
$$\mathbf{m = \frac{Fy}{0.85 * fc'} = \frac{420}{0.85 * 24} = 20.59}$$

$$\boldsymbol{\rho} = \mathbf{\frac{1}{m} * \left(1 - \sqrt{1 - \frac{2 * \mathit{R}n * m}{Fy}}\right)}$$
$$= \mathbf{\frac{1}{20.59} * \left(1 - \sqrt{1 - \frac{2 * 0.088 * 20.59}{420}}\right)}$$
$$= \mathbf{2.1 * 10^{-4}}$$

$$\mathbf{Asreq = \rho * b * d = 2.1 * 10^{-4} * 1000 * 415 = 87.14\ mm^2/m}$$

Check for $A_{s,min}$:

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

$$A_{s,min} = 0.25\,\frac{\sqrt{24}}{420}\cdot 1000\cdot 415 = 1182\ mm^2/m$$

$$A_{s,min} = \frac{1.4}{420}\cdot 1000\cdot 415 = 1383.3\ mm^2/m\ -control$$

$$n = \frac{1383.3}{314.2} = 4.4, \qquad take\ 5\ \emptyset\ 20/m \qquad or\ \ \emptyset\ 20\ @\ 200\ mm$$

→ Select Ø20@20cm/m

$$\text{Minimum shrinkage Ash} = 0.0018\text{bh} = 0.0018 \times 1000 \times 500 = 900\ \text{mm2/m}$$

→ Select Ø14@30cm/m

✓ **Design of heel:**

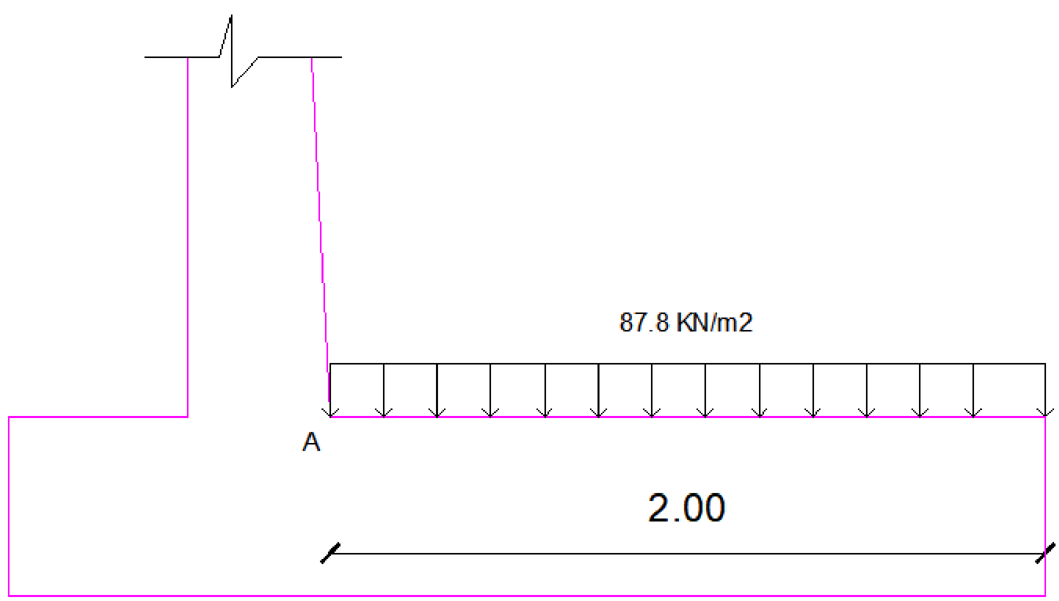


Fig 4.37: Loads of Retaining Wall on Heel.

$$\mathbf{wu = 1.2 \times (3 \times 18 + 0.5 \times 25) + 1.6 \times 5.0 = 87.8\ KN/m2}$$
$$\mathbf{Vu = 87.8 \times 1\ m \times 2 = 175.6\ KN}$$

$$\mathbf{d = 500 - 75 - \frac{20}{2} = 415\ mm}$$

$$\phi Vc = \frac{\phi \sqrt{f_c'} * b_w * d}{6} = 0.75/6 * \sqrt{24} \times 1000 \times 415 \times 10^{-3} = 254.13\ \text{KN}$$

$$\mathbf{\frac{1}{2}\phi V = 127.07\ KN < Vu = 175.6\ KN < \phi V = 254.13\ KN}$$

The thickness of 50 cm at the toe slab end is adequate enough.

flexural design

$$\boldsymbol{Mu = 87.8 \times \left[\frac{2^2}{2}\right] = 175.6\ KN \cdot m}$$

Take $\phi = 0.9$ for flexure

$$\mathbf{Rn = \frac{Mu * 10^6}{\phi b * d^2} = \frac{175.6 * 10^6}{0.9 * 1000 * 415^2} = 1.133\ Mpa}$$
$$\mathbf{m = \frac{Fy}{0.85 * fc'} = \frac{420}{0.85 * 24} = 20.59}$$

$$\mathbf{\rho = \frac{1}{m} * \left(1 - \sqrt{1 - \frac{2 * \boldsymbol{R}n * m}{Fy}}\right)}$$
$$\mathbf{= \frac{1}{20.59} * \left(1 - \sqrt{1 - \frac{2 * 1.133 * 20.59}{420}}\right)}$$
$$\mathbf{= 2.78 * 10^{-3}}$$

$$\mathbf{Asreq = \rho * b * d = 2.78 * 10^{-3} * 1000 * 415 = 1153.7\ mm^2/m}$$

Check for $A_{s,min}$:

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

$$A_{s,min} = 0.25\ \frac{\sqrt{24}}{420} \cdot 1000 \cdot 415 = 1182\ mm^2/m$$

$$A_{s,min} = \frac{1.4}{420} \cdot 1000 \cdot 415 = 1383.3\ \ mm^2/m\ - control$$

$$A_{s,req} = 1153.7 < A_{s,min} = 1383.3 \qquad ,Take\ \ A_{s,min} = 1383.3$$

$$n\ = \frac{1383.3}{314.2} = \ 4.4, \qquad take\ 5\ \emptyset\ 20/m \qquad or\ \ \emptyset\ 20\ @\ 200\ mm$$

→ **Select Ø20@20cm/m**

Minimum shrinkage Ash $= 0.0018\text{bh} = 0.0018 \times 1000 \times 500 = 900\ \text{mm2/m}$

→ **Select Ø14@30cm/m**

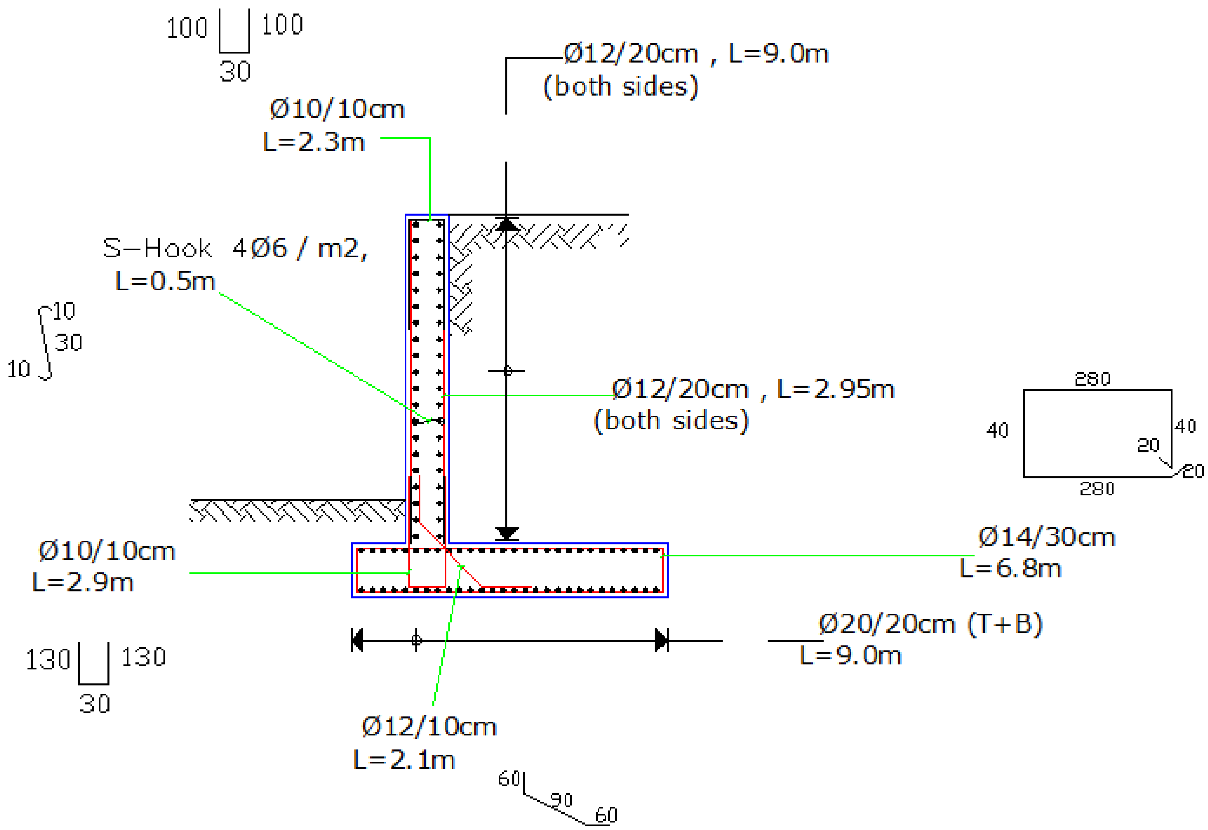


Fig 4.38: Detailing of Retaining Wall.