UNIT 8 RETAINING WALL

Structure

8.1 INTRODUCTION

Walls which retain earth or non-cohesive materials and prevent them from spreading at their angle of repose are called retaining walls. Such walls are also erected where earth is retained at different levels at a section on the ground. Retaining walls may be of brick masonry or of reinforced concrete. Brick masonry is economical for very low height.

A reinforced concrete retaining wall consisting of

(a) a wall which sustains the pressure due to retained materials, and

(b) a base which transfers the forces due to selfweight and retained materials to the ground is called a cantilever retaining wall (Figure 8.1) as all its components behave as cantilevers.

Figure 8.1 : Bending of Components of a Cantilever Type Retaining Wall due to Earth Pressure

When a retaining wall is comparatively very high, the cantilever type of retaining wall becomes uneconomical. To reduce cross sectional dimensions, counterforts (Figure 8.2) are provided for high retaining wall to support both the wall as well as the base so that they may behave as continuous slabs.

Here design and detailing of RC cantilever type of retaining wall only has been described in this unit.

Figure 8.2 : Counterfort Retaining Wall

Objectives

After studying this unit, you should be able to

- define retaining wall and describe its components, and
- design and detail a cantilever type retaining wall under different earth pressures.

8.2 EARTH PRESSURE

When Retained Earth is Horizontal

The intensity of earth pressure at any depth h_i below the retained earth surface (Figure 8.3) is given by

$$p_i = k_a \gamma h_i$$

where k_a = coefficient of active earth pressure,

$$= \frac{1 - \sin \phi}{1 + \sin \phi},$$

ϕ = angle of repose, and

γ = Density of retained earth.

Figure 8.3 : Pressure Diagram when Earth is Horizontal

When Retained Earth is of Different Densities at Different Levels

The pressure diagram under this condition shown in Figure 8.4*, is self explanatory.

* The pressure diagrams are shown in Figure 8.4. They have been drawn on the assumption that $\phi_1 > \phi_2$ and $\phi_2 < \phi_3$.

where $k_{a1} = \dfrac{1 - \sin \phi_1}{1 + \sin \phi_1}$, $k_{a2} = \dfrac{1 - \sin \phi_2}{1 + \sin \phi_2}$,

and $k_{a3} = \dfrac{1 - \sin \phi_3}{1 + \sin \phi_3}$.

Figure 8.4 : Pressure Diagram with Retained Earth having Different Densities at Different Levels

In this case, pressure on the wall is also inclined at the same angle as the inclination of retained earth.

When Water Level is above the Base of the Wall

In this case (Figure 8.5), density of submerged soil becomes $(\gamma - \gamma_w)$, where γ = unit weight of soil, and γ_w = unit weight of water.

K_{as} is coefficient of active earth pressure for submerged soil.

Figure 8.5 : Pressure Diagram when Water Table is above Base of Retaining Wall

The angle of repose for submerged soil is generally 5 to 10 less than that of soil.

When Retained Earth has Additional Surcharge

The additional pressure on retaining wall due to the surcharge load of intensity q per unit area will be uniformly distributed on the wall (Figure 8.6) and its intensity = $k_a\, q$.

Figure 8.6 : Pressure Diagram when Retained Earth has Additional Surcharge

When Retained Earth is Inclined at an Angle

When angle of inclination α is equal to or smaller than ϕ, the coefficient of active earth pressure $k_a = \cos\alpha \dfrac{\cos\alpha - \sqrt{\cos^2\alpha - \cos^2\phi}}{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\phi}}$. The pressure diagram is as shown in Figure 8.7.

Figure 8.7 : Pressure Diagram when Retained Earth is Inclined

SAQ 1

(a) Define a retaining wall.

(b) Why a retaining wall with stem and base slab is called a cantilever type retaining wall?

(c) Draw pressure diagram for

  (i) When retained earth is of different densities at different levels.

  (ii) When retained earth has additional surcharge.

8.3 DESIGN CONSIDERATIONS

A cantilever type retaining wall consists of *three* cantilever components (Figure 8.1) namely :

(a) a stem bending as cantilever under the pressure of retained earth,

(b) a toe slab bending *upward* due to reactive force from ground, and

(c) a heel slab bending *downward* due to resultant of weight of retained earth and ground pressure.

The required reinforcement on tension face of each component has been shown in Figure 8.8.

Figure 8.8 : Cantilever Retaining Wall Showing Reinforcements

Depth of Foundation (*D*)

Determination of depth of foundation is done according to Rankine's formula $D = \frac{p_B}{\gamma}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$ where p_B = Bearing capacity of soil.

Width of Base (*B*)

Width of footing is determined in the following ways :

(a) The width is generally taken from 0.5 *h* to 0.6 *h* where *h* is the total height of retaining wall above base.

(b) If *W* = resultant of all vertical forces,

P = resultant of all horizontal forces,

R = resultant of *W* and *P*, and

e = eccentricity of point from the centre of the base where the resultant strikes the base.

The maximum and minimum intensities of soil pressures (Figure 8.9) may be evaluated as follows :

$$p = \frac{W}{A} \pm \frac{W \times e}{I} \times \frac{B}{2} = \frac{W}{1 \times B} \pm \frac{W \times e}{\frac{1}{12} \times 1 \times B^3} \times \frac{B}{2}$$

(Here, the length of retaining wall is taken as unity.)

Figure 8.9 : Fixing Size of Components

$$p = \frac{W}{B}\left(1 \pm \frac{6e}{B}\right)$$

where p = maximum or minimum pressure on the base.

For full contact of base on the soil the minimum value of p = 0.

Substituting this value of p in the above equation, $e = \frac{B}{6}$.

Assuming same density for soil and concrete for simplicity of calculation of vertical weight, it can be easily shown that

$B = h\sqrt{\frac{k_a}{(1-n)\,(1+3n)}}$ for horizontally retained earth surface.

where h = total height of retaining wall, and $n \approx \frac{1}{3}$

It can also be shown that for the most economical design, proportioning of the base slab is done so as to align the vertical reaction R' at the base with the front face of the stem.

Stability Requirements

Overturning

$$\text{The restoring moment} \geq 1.2 \times \begin{array}{l}\text{Maximum}\\ \text{overturning}\\ \text{moment due to}\\ \text{dead load (DL)}\end{array} + 1.4 \times \begin{array}{l}\text{Maximum}\\ \text{overturning moment}\\ \text{due to imposed}\\ \text{dead load (IL)}\end{array}$$

In cases where *DL* provides the restoring moment, only 0.9 times the *DL* shall be considered. Restoring moment due to *IL* shall be ignored.

For example the retaining wall shown in Figure 8.9, overturning about point *A* on the toe must comply the following equation :

$$0.9 \times \begin{array}{l}\text{Restoring moments due}\\ \text{to DL components of}\\ \text{retaining wall and that}\\ \text{due to soil EFIJ about A}\end{array} \quad 1.2 \times \begin{array}{l}\text{Overturning moment}\\ \text{due to P + 1.4}\\ \text{overturning moment}\\ \text{due to IL if any}\end{array} \quad \geq 1$$

The restoring moment due to soil above the toe about *A* is ignored.

Sliding

The structure shall have a factor against sliding of not less than 1.4 under the most adverse combination of the applied forces. In this case, only 0.9 times the dead load (W) shall be taken into account.

i.e. $$\frac{\mu\,(0.9\,W)}{P} \geq 1.4$$

In case, if $\frac{\mu\,(0.9\,W)}{P} < 1.4,$ a key is provided to develop passive pressure (P_p) opposite to P. The depth h_2 is determined to comply the above mentioned codal provision (Figure 8.10).

Figure 8.10 : Provision of Shear Key to Prevent Sliding

A conservative estimate of P_p may be made by the formula

$$P_p = \frac{1}{2} \times k_p\ \gamma\ (h_2^2 - h_1^2)$$

where h_1 and h_2 are as shown in Figure 8.10. The positioning of shear key is done in such a way that the development length of main reinforcement of stem may extend into it.

Other Design Considerations

The thickness of the stem is about 8% of the sum of height of the wall and the equivalent height due to surcharge; but in no case less than 300 mm. The thickness of base is taken slightly more than that of stem.

The other design parameters are same as applicable for any other structure or structural components.

Example 8.1

Design a reinforced concrete cantilever retaining wall of height 5 m above base to retain a level earth fill. The water table may rise up to 2 m below the top surface of the retained earth. Other design parameters are as follows :

ϕ = 32 for soil; ϕ = 25 for submerged soil; Bearing capacity of soil p_B = 150 kN/m^2; Unit weight of soil = 19.2 kN/m^3; $f_{ck} = 25$ N/mm^2 and $f_y = 415$ N/mm^2. Coefficient of friction between soil and concrete μ = 0.35*.

*A curtain wall is provided at *D* (Figure 8.11) below the base so that the soil on LHS of curtain wall is not effected by the water on the other side.

Solution

Depth of Foundation (*D*)

Depth of footing as per Rankine formula,

$$D = \frac{p_B}{q}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$$

or, $$D = \frac{150\times 1}{19.20}\left(\frac{(1-\sin 32)}{(1+\sin 32)}\right)^2 = 0.74 \text{ m}$$

Provided depth of footing below *GL* =1.0 m.

Width of Base (*B*)

$$B = h\sqrt{\frac{k_a}{(1-n)\,(1+3n)}}$$

where $k_a = \frac{(1-\sin 32)}{(1+\sin 32)} = 0.31$

and taking n = 0.4, $B = 5\sqrt{\frac{(0.31)}{(1-0.4)\,(1+3\times 0.4)}} = 2.423$ m

* In Figures 8.11 and 8.12 the *final* width of base slab B = 3 m has been shown.

Taking first trial width B = 2.5 m and, therefore, width of toe slab nB = 0.4 × 2.5 = l m.

The designed section of retaining wall is shown in Figure 8.11*.

Figure 8.11 : Section of Retaining Wall

Stability Requirements

Determination of Loads

Taking 1 m length of retaining wall (Figure 8.12).

Assuming thickness of stem = 0.45 m and that of base slab = 0.5 m.

(a) Wt. of stem W_1 = 0.45 × 4.5 × 1 × 25 = 50.625 kN at 1.225 m from A.

(b) Wt. of base slab W_2 = 0.5 × 2.5 × 1 × 25 = 31.25 kN at 1.25 m from A.

(c) Wt. of earth over heel slab W_3 = (2.5 – 1.45) × (5 – 0.5) × 1 × 19.2 = 90.72 kN at 1.45 + $\frac{(2.5-1.45)}{2}$ from A =1.975 m from A.

Figure 8.12 : Evaluation of Design Loads for Stem

(d) Lateral pressure due to earth above ground water level

$P_1 = \frac{1}{2} k_a \gamma h_1^2$.

$$k_a = \frac{1 - \sin 32}{1 + \sin 32} = 0.31.$$

Substituting $k_a = 0.31$ in the above equation

$$P_1 = \frac{1}{2} \times 0.31 \times 19.2 \times 2^2 = 11.90 \text{ kN at } 3.67 \text{ m from } A$$

(e) Lateral pressure due to earth above ground water level on height h_2

$$P_2 = k_{as}\ \gamma\ h_1\, h_2$$

$$k_{as} = \frac{1 - \sin\ 25^o}{1 + \sin\ 25^o} = 0.41$$

Substituting $k_{as} = 0.41$ in the above equation

$$P_2 = 0.41 \times 19.2 \times 2 \times 3 = 47.23 \text{ kN at } 1.5 \text{ m from } A.$$

(f) Earth pressure due to saturated soil

$$P_3 = \frac{1}{2} k_{as}\ (\gamma - \gamma_w)\ h_2^2 = \frac{1}{2} \times 0.41 \times (19.2 - 10) \times 3^2 = 16.97 \text{ kN}$$

at 1 m from A.

(g) Lateral pressure due to water

$$= \frac{1}{2} \times \gamma_w \times h_2^2 = \frac{1}{2} \times 10 \times 3^2 = 45 \text{ kN} \text{ at 1 m from } A.$$

Total moment of all forces about A

$$\sum M_A = (50.625 \times 1.225) + (31.25 \times 1.25) + (90.72 \times 1.975)$$
$$- (11.9 \times 3.67) - (47.23 \times 1.5) - (16.97 \times 1) - (45 \times 1)$$
$$= 103.762 \text{ kNm}.$$

Total weight, $\sum V = 50.625 + 31.25 + 90.72 = 172.595$ kN

$\therefore$ Distance of resultant from $A = \frac{\sum M_A}{\sum V} = \frac{103.762}{172.595} = 0.6$ m

The resultant must lie within the middle third of base width (i.e. within 2.5/3 = 0.833 m) for full contact of base slab with soil underneath.

In this case $\frac{B}{3} = \frac{2.5}{3} = 0.833 \text{ m} > 0.6 \text{ m}$.

In other words, the resultant is falling outside the middle third of *B*; hence width of slab shall be increased to 3 m for next trial. All calculations shall be as above but for the following :

(i) Wt. of base slab $W_2 = 0.5 \times 3 \times 1 \times 25$

$= 37.5$ kN at 1.5 m from *A*.

(ii) Wt. of earth over heel slab

$W_3 = (3 - 1.45) \times (5 - 0.5) \times 1 \times 19.2$

$= 133.92$ kN at $1.45 + \frac{(3-1.45)}{2} = 2.225$ from *A*.

Total moment of all forces about *A*

$$\sum M_A = (50.625 \times 1.225) + (37.5 \times 1.5) + (133.92 \times 2.225) - (11.9 \times 3.67) - (47.23 \times 1.5) - (16.97 \times 1) - (45 \times 1) = 239.75 \text{ kN-m}$$

$$\sum V = W = 50.625 + 37.5 + 133.92 = 222.045 \text{ kN}$$

$\therefore$ Distance of resultant from *A*

$$\frac{\sum M_A}{\sum V} = \frac{239.75}{222.045} = 1.08 \text{ m from } A.$$

Hence, the resultant is within the middle third (i.e. lying between 1 m and 2 m) of the base width and, therefore, the whole base area will be in contact with the soil.

Eccentricity, $e = \frac{3.0}{2} - 1.08 = 0.42$ m on LHS

$\therefore$ Maximum and minimum pressures

$$p_{A,D} = \frac{W}{B}\left(1 \pm \frac{6e}{B}\right) = \frac{222.045}{3} \times \left(1 \pm \frac{6 \times 0.42}{3}\right)$$

or, $p_A = 136.19 \text{ kN/m}^2 < 150 \text{ kN/m}^2$

and $p_d = 11.84 \text{ kN/m}^2 < 150 \text{ kN/m}^2$ (Refer Figure 8.13.)

Check for Overturning

$$= \frac{0.9 \times \text{Restoring Moment due to } DL}{\text{Overturning Moment}}$$

$$= \frac{0.9 \times (50.625 \times 1.225 + 37.5 \times 1.5 + 133.92 \times 2.225)}{1.2 \times (11.9 \times 3.67 + 47.23 \times 1.5 + 16.97 \times 1 + 45 \times 1)}$$

$$= \frac{0.9 \times 416.238}{1.2 \times 176.488} = 1.77 > 1$$

Hence, O.K.

Check for Sliding

$$\frac{\mu \times 0.9\ W}{P}$$

(Substituting values of μ, W and P in the above expression)

$$= \frac{0.35 \times 0.9 \times 222.045}{(11.9 + 47.23 + 16.97 + 45)} = \frac{0.35 \times 0.9 \times 222.045}{121.1} = 0.578 < 1.4$$

Hence, a shear key is required to develop a passive pressure P_p such that

$$\frac{\mu \times 0.9\ W}{P - P_p} \geq 1.4$$

or, $$\frac{0.35 \times 0.9 \times 222.045}{121.1 - P_p} \geq 1.4$$

or, $49.96 \geq 121.1 - P_p$

or, $P_p = 71.14$ kN

From equation (Refer Figure 8.10)

$$P_p = \frac{1}{2} \times k_p\, \gamma\, (h_2^2 - h_1^2)$$

Length of shear key is determined as follows :

$$P_p = 71.14 \text{ kN};\ k_p = \frac{1 + \sin 25^o}{1 - \sin 25^o} = 2.464;\ h_1 = 1 - 0.3 = 0.7 \text{ m}$$

Substituting above values in the equation for P_p

$$71.14 = \frac{2.464 \times 19.2}{2}\,(h_2^2 - 0.7^2)$$

or, $h_2 = 1.87$ m

$$h_2 = h_1 + d_{sk} + x_{sk} \tan 25^o$$

or, $1.87 = 0.7 + d_{sk} + 1.2 \tan 25^o$ (Taking $x_{sk} = 1.2$ m)

or, $d_{sk} = 0.61$ m

Therefore, a shear key of $d_{sk} = 0.65$ m is provided

Providing width of shear key = 250.

Shear resistance of the key for minimum shear strength for M 25

= 250 × 1000 × 0.29 = 72.5 kN > 71.14 kN.

Design of Stem

BM at the Top of Base Slab

$$M = 11.90 \times (3.67 - 0.5) + 0.41 \times 19.2 \times 2 \times \frac{(3-0.5)^2}{2} + \frac{1}{2} \times 0.41$$

$$\times (19.2 - 10) \times 2.5^2 \times \frac{2.5}{3} + \frac{1}{2} \times 10 \times 2.5^2 \times \frac{2.5}{3} = 98.186 \text{ kNm.}$$

$$\therefore \quad M_u = 1.5 \times 98.186 = 147.28 \text{ kN-m}$$

At distance d = 450 – 56 = 394 from top of base = (5.00 – 0.5 – 0.394) = 4.106 m from top of stem

$$V_u = 1.5 \times \left(-11.90 + 0.41 \times 19.2 \times 2 \times (4.106 - 2) + \frac{1}{2} \times 0.41 \times (19.2 - 10) \times 2.106^2 + \frac{1}{2} \times 10 \times 2.106^2 \right)$$

or, $V_u = 113.397$ kN

Thickness of Stem (D) *from BM Consideration*

$$M_u = 0.36 \frac{x_{u,\max}}{d} \left(1 - 0.42 \frac{x_{u,\max}}{d}\right) bd^2 f_{ck}$$

or $184.181 \times 10^6 = 0.36 \times 0.48 \times (1 - 0.42 \times 0.48) \times 1000 \times d^2 \times 25$

or, $d = 206.643$ mm

or, $D = 206.643 + 50 + 8 = 264.643$ mm (Taking $\phi = 16$).

'*D*' *from Other Design Considerations*

(a) 8% of the height of wall = $\dfrac{8 \times 4.5 \times 1000}{100} = 360$ mm

(b) ≥ 300.

Hence, provided $D = 360$ mm.

$\therefore$ $d = 360 - 50 - 8 = 302$ mm

$$M_u = 0.87\ f_y\ A_{st}\ d \left(1 - \frac{A_{st}\ f_y}{bd\ f_{ck}}\right)$$

or, $147.28 \times 10^6 = 0.87 \times 415 \times A_{st} \times 302 \times \left(1 - \dfrac{A_{st} \times 415}{1000 \times 302 \times 25}\right)$

or, $147.28 \times 10^6 = 109037.1\ A_{st} - 5.993\ A_{st}^2$

or, $A_{st}^2 - 18194.07\ A_{st} - 24575337.89 = 0$

After solving the above equation, we get

$A_{st} = 1469.41$ mm^2

Provided ϕ 16 @ 105 c/c.

D from shear consideration

$$p_t\% = 100 \times \frac{\left(\dfrac{1000 \times 201}{105}\right)}{1000 \times 302} = 0.634\%$$

$$\tau_c = 0.49 + \frac{(0.57 - 0.49)}{(0.75 - 0.5)} \times (0.634 - 0.5) = 0.533 \text{ N/mm}^2$$

$$\tau_v = \frac{V_u}{bd} = \frac{113.397 \times 1000}{1000 \times 302} = 0.375 < 0.533 \text{ N/mm}^2$$

Hence, O.K.

Temperature and shrinkage reinforcement (Figure 8.13).

Figure 8.13 : Evaluation of Design Loads for Heel Slab

$$A_{st,\min} = \frac{0.12}{100} \times 1000 \times 360 = 432 \text{ mm}^2$$

Hence, provided ϕ 10 @ 180 mm c/c.

Design of Heel Slab

$$p_F = 11.84 + \frac{(136.19 - 11.84) \times 1.64}{3} = 79.818 \text{ kN/m}^2$$

Shear at the face of stem (FC)

= wt of back fill + wt of slab – pressure from soil

$$= (5 - 0.5) \times 1 \times 1.64 \times 19.2 + 0.5 \times 1.64 \times 1 \times 25$$

$$- \frac{(79.818 + 11.84)}{2} \times 1 \times 1.64$$

= (141.696 + 20.5 – 75.16) = 87.04 kN

$$\therefore \quad V_u = 1.5 \times 87.04 = 130.56 \text{ kN}$$

BM at the face of stem (FC)

$$M_u = 1.5 \times \left\{ (141.696 + 20.5) \times \frac{1.64}{2} - 75.16 \times \frac{(2 \times 11.84 + 79.818)}{(11.84 + 79.818)} \times \frac{1.64}{3} \right\}$$

= 129.909 kN-m.

D from Moment Consideration

$$M_u = 0.36 \frac{x_{u,\max}}{d} \left(1 - 0.42 \times \frac{x_{u,\max}}{d} \right) b d^2 f_{ck}$$

or $\quad 129.909 \times 10^6 = 0.36 \times 0.48 \times (1 - 0.42 \times 0.48) \times 1000 \times d^2 \times 25$

or, $\quad d$ = 194.07 mm

and $\quad D$ = 194.07 + 50 + 8 = 252.07 mm (Taking ϕ = 16)

Keeping heel thickness throughout same as

D = 500 mm

$\therefore \quad d = 500 - 50 - 6 = 444$ mm

$$\therefore \quad M_u = 0.87 f_y A_{st} d \left(1 - \frac{A_{st} f_y}{bd f_{ck}}\right)$$

or, $$129.909 \times 10^6 = 0.87 \times 415 \times A_{st} \times 444 \times \left(1 - \frac{A_{st} \times 415}{1000 \times 444 \times 25}\right)$$

or, $$129.909 \times 10^6 = 160306.2\, A_{st} - 5.993\, A_{st}^2$$

or, $$A_{st}^2 - 26748.91\, A_{st} + 21676789.59 = 0$$

After solving the above equation, we get

$$A_{st} = 836.54 \text{ mm}^2/\text{m}$$

Provided ϕ 12 @ 135 mm c/c (A_{st} = 837.03 mm^2/m)

Check for shear

$$\tau_v = \frac{V_u}{bd} = \frac{130.56 \times 10^3}{1000 \times 444} = 0.294 \text{ N/mm}^2$$

$$p_t\,\% = 100 \times \frac{\dfrac{1000 \times 113}{135}}{1000 \times 444} = 0.188\%$$

$$\tau_c = 0.29 + \frac{(0.36 - 0.29)}{(0.25 - 0.15)} \times (0.188 - 0.15) = 0.317 \text{ N/mm}^2 > 0.294 \text{ N/mm}^2$$

Hence, O.K.

Design of Toe Slab

Pressure at face of stem (BG)

$$P_{BG} = 11.84 + \frac{(136.19 - 11.84)}{3} \times 2 = 94.74 \text{ kN/m}^2$$

The pressure diagram on the base from soil is shown in Figure 8.14.

Figure 8.14 : Evaluation of Design Loads for Toe Slab

Total force on toe, i.e. on length *AB*

$$= \frac{(136.19 + 94.74)}{2} \times 1 = 115.465 \text{ kN}$$

Distance of resultant of this force from *B*

$$= \frac{(2 \times 136.19 + 94.74)}{(136.19 + 94.74)} \times \frac{1}{3} = 0.53 \text{ m from } B$$

$\therefore$ Max BM on toe slab, i.e. M_B = 115.465 × 0.53 = 61.196 kN-m

M_u =1.5 × 61.196 = 91.794 kNm

Depth from BM Consideration

$$M_{u,\lim} = 0.36 \frac{x_{u,\max}}{d} \left(1 - 0.42 \frac{x_{u,\max}}{d}\right) bd^2 \ f_{ck}$$

$$91.794 \times 10^6 = 0.36 \times 0.48 \times (1 - 0.42 \times 0.48) \times 1000 \times d^2 \times 25$$

or, $d = 163.14$ mm

Assuming nominal cover = 50 and bar diameter, ϕ = 12.

$$D = 163.14 + 50 + 6 = 219.14 \text{ mm}$$

For exact value of BM at *B*, the BM due to self wt. of the toe slab at that section shall be deducted

Self wt. of toe slab = $0.3 \times 1 \times 1 \times 25 = 7.5$ kN/m

$\therefore$ Moment about *B* = $\frac{7.5 \times 1^2}{2} = 3.75$ kN/m

This is very small in comparison to moment due to earth pressure hence, may be neglected to be on the conservative side.

Provided *D* = 300 mm

$\therefore$ d = 300 – 50 – 6 = 244 mm

$$M_u = 0.87 \ f_y \ A_{st} \left(1 - \frac{A_{st} \, f_y}{bd f_{ck}}\right)$$

or, $91.794 \times 10^6 = 0.87 \times 415 \, A_{st} \times 244 \times \left(1 - \frac{A_{st} \times 415}{1000 \times 244 \times 25}\right)$

or, $91.794 \times 10^6 = 88096.2 \ A_{st} - 5.993 \ A_{st}^2$

or, A_{st}^2 –14699.84 A_{st} + 15316869.68 = 0

After solving the above equation, we get

A_{st} = 1128.62 mm^2

Adopted ϕ 12 @ 100 mm c/c.

Depth from Shear Consideration

Check for Shear

Bearing pressure from soil at distance *d* from face of the stem, i.e. at (1000 – 244) = 756 mm from *A*.

$$= 11.84 + \frac{(136.19 - 11.84)}{3} \times 2.244 = 104.85 \text{ kN/m}^2$$

$$\therefore \quad V \text{ at } 756 \text{ from } A = \frac{(136.19 + 104.85)}{2} \times 0.756 = 91.113 \text{ kN}$$

Shear force due to self weight = 0.756 × 3.3 × 25 = 62.73 kN

This is small in comparison to shear due to earth pressure; hence neglected to be on the conservative side.

$$\tau_v = \frac{V_u}{bd} = \frac{1.5 \times 91.113 \times 1000}{1000 \times 244} = 0.56 \text{ N/mm}^2$$

$$p_t\,\% = 100\,\frac{A_s}{bd} = 100 \times \frac{\left(\frac{1000 \times 113}{100}\right)}{1000 \times 244} = 0.463\%$$

$$\tau_c = 0.36 + \frac{(0.49 - 0.36)}{(0.50 - 0.25)} \times (0.463 - 0.36) = 0.414 \text{ N/mm}^2$$

Hence $\tau_v < \tau_c$

Next Taking

Bearing pressure at distance d = 444 from B on LHS (for D = 500 mm)

$$= 11.84 + \frac{(136.19 - 11.84)}{3} \times 2.444 = 113.14 \text{ kN/mm}^2$$

$$\therefore \quad V = \frac{(136.19 + 113.14)}{2} \times (1 - 0.444) = 69.314 \text{ kN}$$

$$\therefore \quad V_u = 1.5 \times 69.314 = 103.97 \text{ kN}$$

Provided D = 500 mm

$$\tau_v = \frac{V_u}{bd} = \frac{103.971 \times 1000}{1000 \times 444} = 0.234 \text{ N/mm}^2$$

$$M_u = 0.87\, f_y\, A_{st}\, d \left(1 - \frac{A_{st}\, f_y}{bd f_{ck}}\right)$$

or, $$91.794 \times 10^6 = 0.87 \times 415 \times A_{st} \times 444 \times \left(1 - \frac{A_{st} \times 415}{1000 \times 444 \times 25}\right)$$

or, $$91.794 \times 10^6 = 160306.2\, A_{st} - 5.993\, A_{st}^2$$

or, $$A_{st}^2 - 26748.91\, A_{st} + 15316869.68 = 0$$

or, $$A_{st} = \frac{26748.91 \pm \sqrt{(26748.91^2 - 4 \times 15316869.68)}}{2}$$

$$= 585.429 \text{ mm}^2$$

$$A_{st\,\min} = \frac{0.12 \times 1000 \times 500}{100} = 600 \text{ mm}^2 > 585.429 \text{ mm}^2$$

Provided ϕ 12 @ 185 < 300 < 3 × 444 (= 1332).

For this reinforcement

$$p_t\,\% = 100 \times \frac{\frac{1000 \times 113}{190}}{1000 \times 444} = 0.134\% \text{ and}$$

Corresponding $\tau_c = 0.29 \text{ N/mm}^2 > 0.234 \text{ N/mm}^2$

Hence, O.K.

The reinforcement detailing has been shown in Figure 8.15.

Figure 8.15 : Reinforcement Detailing

SAQ 2

(a) What are the stability requirements while designing a retaining wall?

(b) If sliding resistance of the base slab is not sufficient what additional provision is made to enhance it?

8.4 SUMMARY

A cantilever type reinforced concrete retaining wall consists of three components – stem, toe slab and heel slab – each bending as cantilever under applied forces.

The height of wall is determined by the height of retained earth whereas the lengths of toe and heel slabs are determined by approximate formula and thumb rules based on experience.

The depth of foundation is determined by Rankine's formula for cohensionless soil.

The width of base slab depends on the bearing capacity of soil, over turning and sliding considerations. Sometimes a key is provided at the base of the footing to share resistance against sliding.

8.5 ANSWERS TO SAQs

SAQ 1

(a) Refer Section 8.1.

(b) Refer Section 8.1.

(c) Refer Section 8.2.

SAQ 2

(a) Refer Section 8.3.

(b) Refer Section 8.3.