Subject code-PCI7D002

ADVANCED DESIGN OF REINFORCED CONCRETE STRUCTURES

Module-II

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SYLLABUS

Module-I - Earthquake Engineering

- Introduction to Earthquake Engineering
- Cyclic behaviour of concrete and reinforcement
- Computation of earthquake forces on building frame using Seismic Coefficient Method as per IS 1893-2002
- Base shear and storey shear calculation for multi-storeyed building frames
- Significance of ductility
- Ductility of beam
- Design and detailing for ductility
- Simple problems based on above concept as per is 13920

Module-II - Retaining walls

- Retaining walls
- Forces acting on retaining wall
- Stability requirement
- Design of Cantilever and Counterfort Retaining walls

Module-III - Bridges

- Introduction to bridges:
    - Classification and components of a standard bridge
    - Economical span
    - Location of piers and abutments
    - Vertical clearance above HFL
    - Scour depth
    - Choice of bridge type
- Standard Loadings for Road Bridges, Impact effect and impact factor calculation for RCC and steel bridges
- Design of single vent rectangular slab culvert

Module-IV- Foundations

- Design of Foundations:
    - Design of Rectangular and Trapezoidal Combined footing

References for Module-II

1. Neelam Sharma, “Reinforced Cement Concrete Design”, S. K. Kataria and Sons, 2015

Retaining Walls

INSIDE THIS CHAPTER

16.1. INTRODUCTION

Retaining walls are used to retain earth or other loose materials. These walls are commonly constructed in the following cases:

(1) In the construction of building basements.

(2) As wing wall or abutment in the bridge construction.

(3) In the construction of embankments.

The material which is retained by the retaining wall is called as Backfill. The sloping backfill is called as "Inclined Surcharge". The term surcharge means the backfill above the level of the top of the wall. The backfill exerts a push or lateral pressure on the retaining wall which tries to overturn, bend and slide the retaining wall.

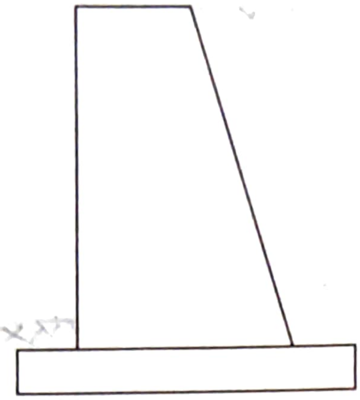
16.2. TYPES OF RETAINING WALLS

Following are the common types of retaining walls:

1. Gravity retaining wall.
2. Cantilever retaining wall
3. Counterfort retaining wall
4. Buttress retaining wall.

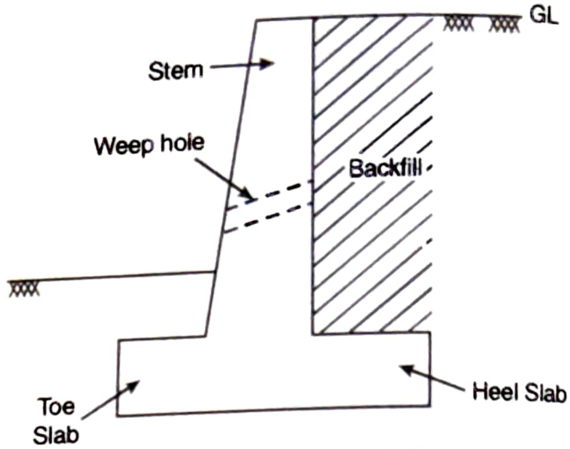
(1) Gravity Retaining Wall

A gravity retaining wall is that retaining wall in which the weight of the retaining wall provides stability against the pressure exerted by the backfill. Gravity retaining walls are made up of massive stone masonry or plain concrete. The principle of design of gravity retaining wall is that tension is not developed anywhere in the section. Therefore, the wall is designed on the basis of "Middle Third Rule".



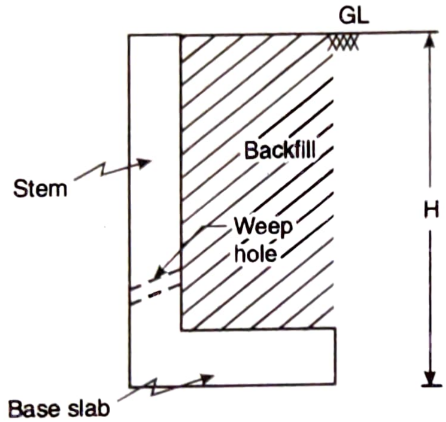
Gravity Retaining Wall

(*a*)

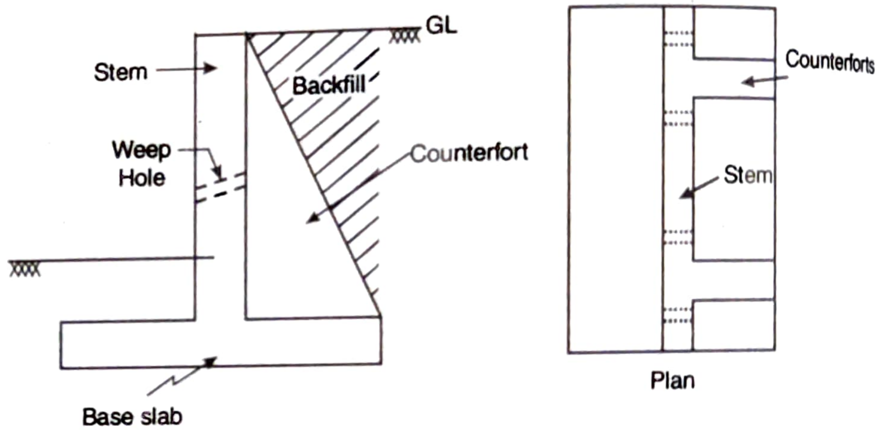


Cantilever Retaining Wall (T-Shape)

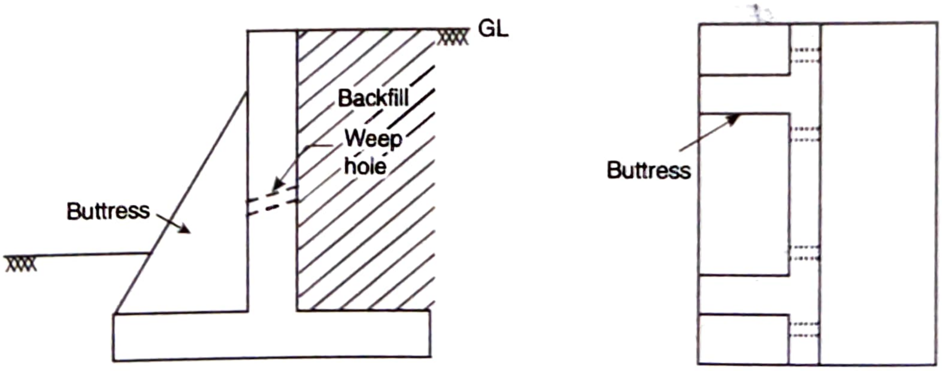
(*b*)



(*c*) Cantilever retaining wall (L-shape)



(*d*) Counterfort retaining wall



(*e*) Buttress retaining wall

Fig. 16.1. Types of retaining walls.

(2) Cantilever Retaining Wall

It is the most common type of retaining wall which consists of a vertical wall called as stem, heel slab and the toe slab as shown in Fig. 16.1(*b*) and (*c*). As all the three components of this wall act as cantilevers, the wall is called as cantilever retaining wall. The stem, heel and toe, all resists the earth pressure by bending. These walls can be L or inverted T shaped. The cantilever retaining walls are used up to a height of 6 m. The weight of the earth on the heel slab and the weight of the retaining wall together provide stability to the wall.

(3) Counterfort Retaining Wall

When the backfill of greater height is to be retained and the required height of cantilever retaining wall exceeds 6 m, then it becomes uneconomical to provide cantilever retaining wall. In such cases, counterfort retaining wall is to be provided. In these walls, counterforts are provided at some suitable interval along the length of the wall, on the backfill side, as shown in Fig. 16.1(*d*). These counterforts are concealed in the backfill and tie the vertical stem and heel slab together. In a counterfort retaining wall, the stem and the heel do not act as a cantilever slab but as a continuous slab because of the counterfort supports. This results in reduction in the maximum bending moment and shear force. The weight of the retaining wall and the weight of the earth retaining on the heel slab together impart stability to the wall.

(4) Buttress Retaining Wall

A buttress retaining wall is similar to the counterfort retaining wall as shown in Fig. 16.1(*e*), but with the difference that in buttress retaining wall, the counterforts, called as 'Buttresses', are provided on the opposite side of the backfill. These buttresses tie the stem and the toe slab together. These buttresses are designed as compression members and hence economical but still not preferred. It is because counterforts are concealed but buttresses are visible and they occupy the space in front of the wall which could have been used for some other purpose.

In addition to the retaining walls described above, there are other retaining walls such as bridge abutments and box culverts. In bridge abutments, the bridge deck provides an additional horizontal restraint to the vertical stem at the top. Therefore, the stem is designed as a beam fixed at the base and simply supported or partially supported at the top. In the case of box culverts, the side walls support the backfill as a retaining wall.

Accumulation of rain water in the backfill results in its saturation and thus a considerable increase in the earth pressure acting on the wall. This may result in unstable conditions. There are two ways of draining this rain water:

(1) By providing weep holes in the wall.

(2) By providing preforated drain pipe with filter.

Weep holes should have at least 10 cm diameter and should be adequality spaced depending on the backfill. Some filter or special geotextile material layer height is to be used on the backface of the wall for full height to avoid the entering of backfill material into weep holes.

This chapter describes the design of a cantilever retaining wall and a counterfort retaining wall.

16.3. EARTH PRESSURE ON RETAINING WALLS

The main force that acts on a retaining wall is the lateral force developed due to the earth pressure or pressure due to retained material. This force tends to destablize the retaining wall by overturning, bending and sliding the wall. The determination of this earth pressure is done by using principles of soil mechanics which you must have read in earlier classes.

The magnitude of the lateral earth pressure varies linearly with depth as shown in Fig. 16.2 and is given by the following equation

$$p = K\gamma H$$

where p is the lateral earth pressure which can be active or passive.

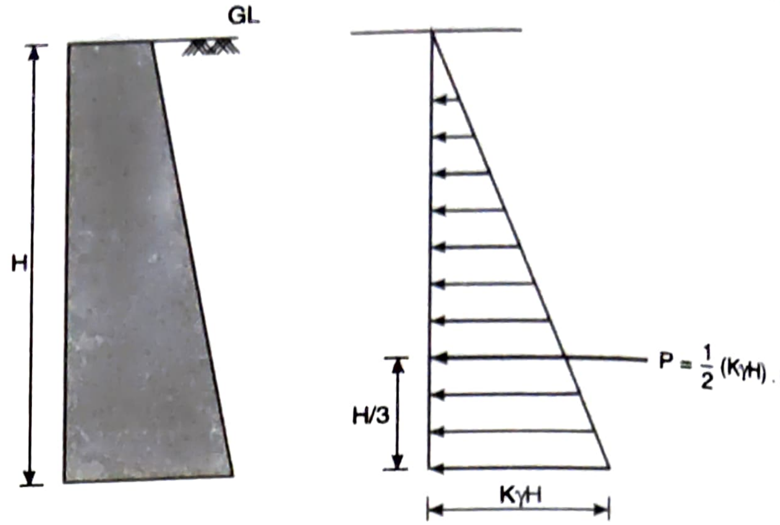


Fig. 16.2. Lateral earth pressure distribution

[Active earth pressure (p_a) is exerted on the wall when the wall has a tendency to move away from the backfill while passive earth pressure (p_b) is exerted on the wall when the wall has a tendency to move towards the backfill.

γ is the unit weight of the retained material

H is the depth of the retained material below the earth surface.

K is the coefficient of earth pressure which is determined by using either Coulomb's theory or Rankine's theory of Earth Pressure. It is written as K_a for coefficient of active earth pressure and K_p for coefficient of passive earth pressure. On the basis of Rankine's theory, following values of coefficient of earth pressure are obtained and these may be used for design of retaining walls.

(a) For horizontal backfill of cohensionless soil

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

where ϕ is the angle of internal friction or angle of repose.

(b) For sloping backfill

$$K_a = \left[\frac{\cos\theta - \sqrt{\cos^2\theta - \cos^2\phi}}{\cos\theta + \sqrt{\cos^2\theta - \cos^2\phi}}\right] . \cos\theta$$

where θ is the angle of inclination of backfill with respect to the horizontal.

Similarly, Rankine's theory also gives the coefficient of passive earth pressure.

(a) For horizontal backfill

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

(b) For sloping backfill

$$K_p = \left[\frac{\cos\theta + \sqrt{\cos^2\theta - \cos^2\phi}}{\cos\theta - \sqrt{\cos^2\theta - \cos^2\phi}}\right] . \cos\theta$$

where θ is the angle of inclination of the backfill with horizontal and

ϕ is the angle of internal friction or angle of repose.

Based on the above formulae, the lateral earth pressure distribution can be plotted along the depth which gives maximum value as $K_a\gamma H$ as the bottom of the retaining wall *i.e.*, at the heel as shown in Fig. 16.2. The lateral force (P_h) exerted on the wall due to the earth pressure is given by the area of the triangular portion.

$$P_h = \frac{1}{2}(K_a\gamma H).H \quad \text{or} \quad \frac{1}{2}(K_p\gamma H).H$$

$$P_h = \frac{1}{2}K_a\gamma H^2 \quad \text{or} \quad \frac{1}{2}K_p\gamma H^2$$

This lateral force acts at a height $\frac{H}{3}$ from the base of the wall. The direction of this force is always parallel to the surface of the backfill. In the case of horizontal backfill, it is horizontal *i.e.*, normal to the stem of the retaining wall.

In the case of T shape cantilever retaining wall (Fig. 16.3), passive earth pressure also develops on the toe side of the retaining wall. The force developed due to passive earth pressure is generally neglected as it is very small (due to low height of the earth retained on toe slab) as compared to the force developed due to active earth pressure.

16.4. FORCES ON A CANTILEVER RETAINING WALL

Consider a cantilever retaining wall as shown in Fig. 16.3. The various forces acting on the wall are as follows:

(1) The lateral force (P_{ah}) due to active earth pressure acting at a height $\frac{H}{3}$ from the base.

(2) Weight of the earth supported on heel slab (W_1)

(3) Weight of the stem (W_2)

(4) Weight of the base slab (W_3)

In addition to the above forces, there is weight of the earth retained above the toe slab (W_4) and the passive earth pressure P_{ph} exerted on the wall. These two are neglected in the design of retaining wall as these are small in size and the design is on the conservative side by neglecting them.

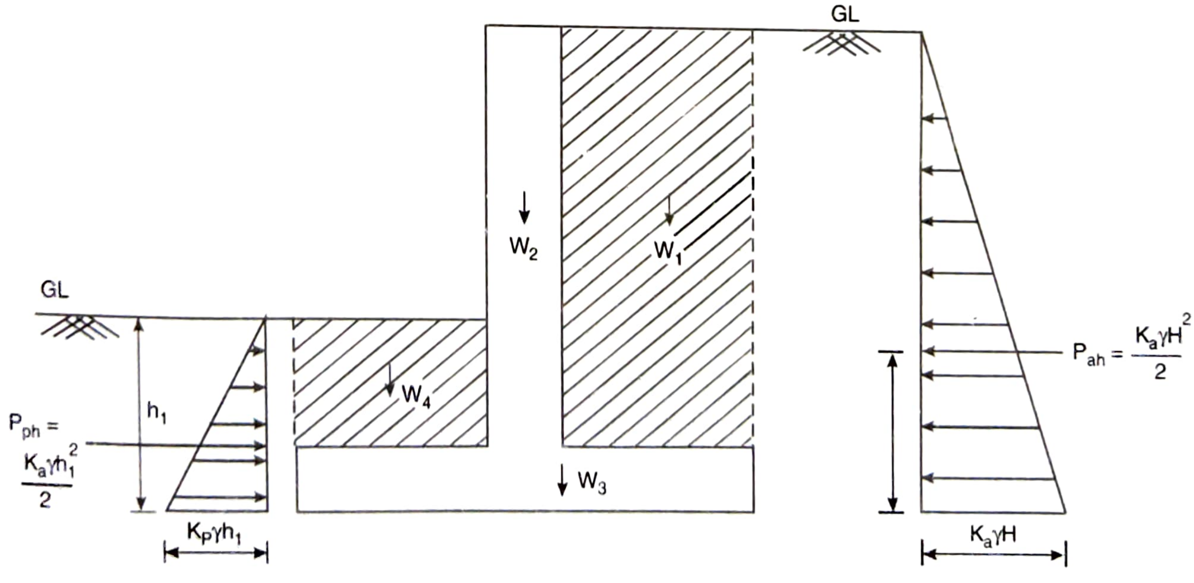


Fig. 16.3. Forces on cantilever retaining wall

16.5. STABILITY OF A CANTILEVER RETAINING WALL

A cantilever retaining wall may fail in the following ways:

(1) Overturning

(2) Sliding

(3) Failure of the undersoil

(1) Overturning

A retaining wall is subjected to overturning moments under the action of lateral force developed due to lateral earth pressure, which tries to overturn the wall about the toe end. The overturning moment (M_0) is given as:

$$M_0 = P_{ah} \times \frac{H}{3}$$

$$= \frac{1}{2}(K_a \gamma H).H.\frac{H}{3}$$

$$\therefore \quad M_0 = \frac{K_a \gamma H^3}{6}$$

The resisting moment (M_R) is provided by the weight of backfill, surcharge and self weight of the retaining wall. If ΣW is the resultant vertical load made up of self weight of retaining wall and the weight of backfill on the base slab, then resisting moment is given as:

$$M_R = \Sigma W.\bar{x}$$

where $\bar{x}$ is the position of the resultant vertical load (ΣW) from toe end.

As per code IS456:2000 Clause 20.1, the stability of the retaining wall against overturning should be ensured so that the resisting moment is not less than 1.4 times the maximum overturning moment due to characteristic imposed loads (the lateral earth pressure in the case of retaining walls). If the dead load provides the restoning moment, then as per code, only 0.9 times the characteristic dead load should be taken into consideration.

Therefore,
$$f_{s1} = \frac{0.9 M_R}{M_0}$$

$$f_{s1} \geq 1.4$$

or
$$\frac{0.9(\Sigma W.\bar{x})}{\dfrac{K_a \gamma H^3}{6}} \geq 1.4$$

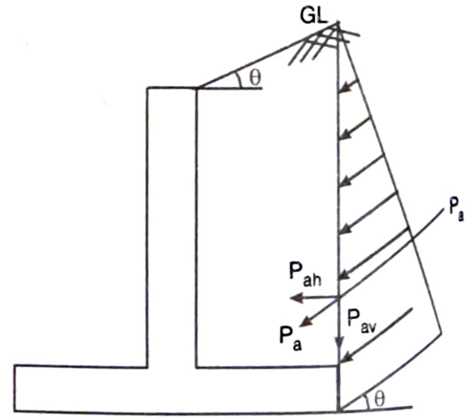


Fig. 16.4. Inclined backfill

For inclined backfill as shown in Fig. 16.4, the vertical component of the active earth pressure (P_{av}) also contributes to the restoring moment but it is neglected to make the calculations simpler and to be on the conservative side.

(2) Sliding

The lateral earth pressure tries to slide the retaining wall away from the backfill. This is opposed by the frictional force developed between the base slab and the soil. If μ is the coefficient of friction between the concrete and soil, then the frictional force resisting the sliding is given as:

$$F_R = \mu \Sigma W$$

The lateral force causing the sliding is P_{ah}.

$$F_S = P_{ah} = \frac{K_a \gamma H^2}{2}$$

Then factor of safety against sliding (f_{s2}) is given as

$$f_{s2} = \frac{F_R}{F_S}$$

$$= \frac{\mu \Sigma W}{P_{ah}}$$

As per IS 456:2000, a minimum factor of safety of 1.4 is to be ensured against sliding and only 0.9 times the characteristic dead load is to be considered for restoring force.

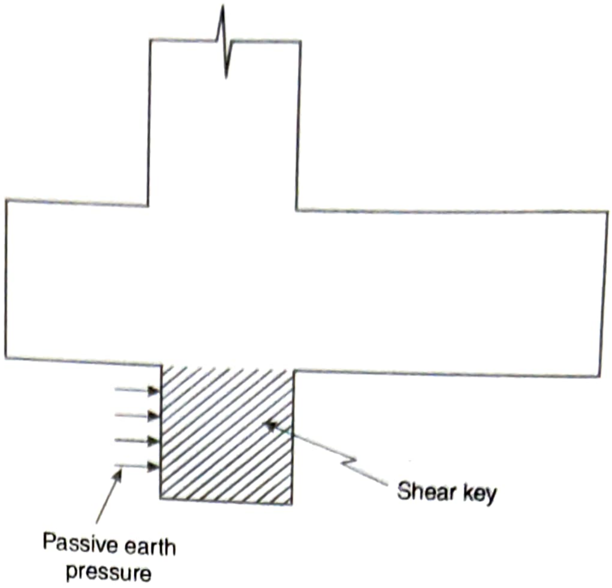


Fig. 16.5

$$\therefore \quad \frac{0.9(\mu \Sigma W)}{P_{ah}} \geq 1.4$$

If the factor of safety against sliding comes out be less than 1.4 then a shear key may be provided as shown in Fig. 16.5. This shear key increases the resistance against sliding as the passive earth pressure developed on the shear key provides additional resistance against sliding.

(3) Failure of the Under Soil

The base width of the retaining wall is designed in such a way that the maximum pressure on the under soil caused due to load distribution must not exceed the safe bearing capacity of the soil. In addition to that it is to be ensured that, no tension is developed anywhere on the section *i.e.*, the resultant load must fall in the middle third zone [as per the middle third rule] so that negative pressure (tension) is not developed any where. As you have already studied the middle third rule, it is not explained here. The resultant pressure distribution under the base slab is shown in Fig. 16.6.

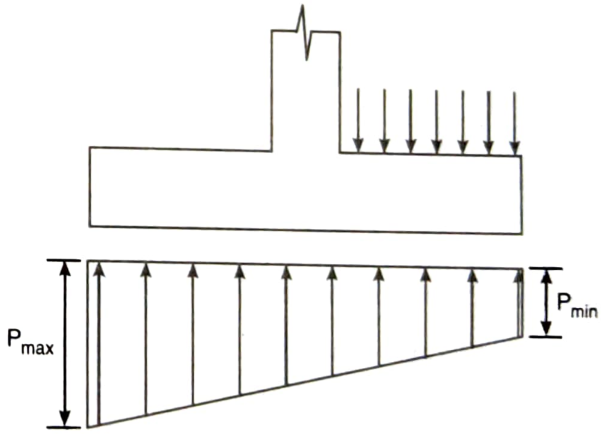


Fig. 16.6. Base pressure distribution

$$p_{max} = \frac{\Sigma W}{b}\left[1 + \frac{6e}{b}\right]$$

$$p_{min} = \frac{\Sigma W}{b}\left[1 - \frac{6e}{b}\right]$$

The maximum pressure at the base *i.e.*, p_{max} should be less than the safe bearing capacity of soil. The minimum pressure *i.e.*, p_{min} should not be negative.

Here, e is the eccentricity of the resultant load and can be obtained as below:

Total moment at toe end A

= Resisting moment about A – Overturning moment at A

$= M_R - M_0$

Total vertical load $= \Sigma W$

$$\bar{x} = \frac{M_R - M_0}{\Sigma W}$$

Eccentricity, $e = \frac{b}{2} - \bar{x}$

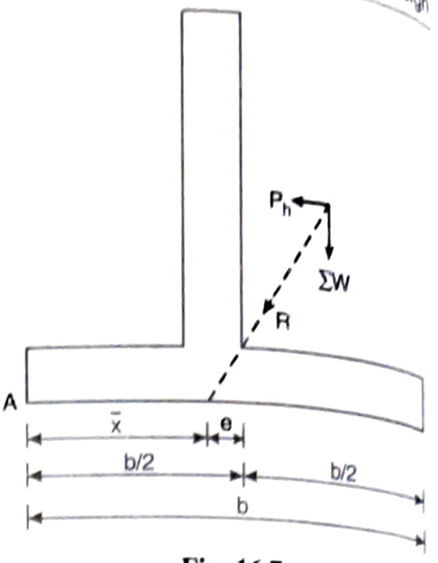


Fig. 16.7

16.6. PROPORTIONING OF THE CANTILEVER RETAINING WALL

Design of a retaining wall involves the determination of its dimensions and the amount of steel required. Before starting the actual analysis of a retaining wall, some preliminary dimensions are to be assumed. The preliminary dimensions of a retaining wall are assumed on the basis of some thumb rules which are explained below:

(1) Depth of Foundation

The minimum depth of the foundation is determined on the basis of Rankine's formula

$$h_{\min} = \left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)\frac{q_0}{\gamma}$$

where $h_{\min}$ is the depth of foundation below the earth surface

q_0 = safe bearing capacity of the soil

γ = unit weight of the soil

ϕ = angle of internal friction or angle of repose

(2) Height of the Retaining Wall (H)

The height of the material to be retained (h) is given. The depth of foundation is added to the height of the material to be retained, to get the total height of the retaining wall (H)

$$H = h + h_{\min}$$

(3) Base Width (b)

The width of the base slab can be determined by considering the equilibrium of various forces at the base. Based on exact analysis and experience, it is found that the base width (b) varies from $0.4H$ to $0.6H$.

(4) Thickness of Base Slab

For preliminary analysis, the thickness of base slab is assumed to be $\frac{H}{10}$ to $\frac{H}{15}$, where H is the total height of the retaining wall. The minimum thickness of base slab should not be less than 300 mm. The thickness assumed should be checked from bending moment and shear force requirements.

(5) Thickness of the Stem

The thickness of vertical stem or wall is governed by the bending moment criteria. As the stem behaves like a cantilever, subjected to lateral pressure which is increasing with depth, it is economical to have a trapezoidal section of the stem with minimum thickness of 150 mm at top. The thickness at the base of stem should not be less than 300 mm. Initially, the thickness of stem may be assumed to be about 8 to 10% of the total height of the retaining wall or can be found from the bending moment criteria.

The preliminary dimensions of the various components of the cantilever retaining wall are used for checking the various stability criteria like overturning, sliding, safe bearing pressure and the depth requirement for maximum bending moment. If these criteria/requirements are satisfied then the dimensions are adopted and design calculations regarding the area of steel etc. are done otherwise the dimensions are revised.

16.7. STRUCTURAL BEHAVIOUR AND DESIGN OF A CANTILEVER RETAINING WALL

Consider the Fig. 16.8 showing a cantilever retaining wall subjected to a lateral force P_{ah}.

(1) Stem: The vertical wall or stem acts like a cantilever subjected to a triangular loading as shown in Fig. 16.8 with maximum pressure developed at the base. The base of the stem is subjected to maximum bending moment (M_B)

$$M_B = \frac{1}{2}(K_a \gamma h) h \times \frac{h}{3}$$

$$M_B = \frac{K_a \gamma h^3}{6}$$

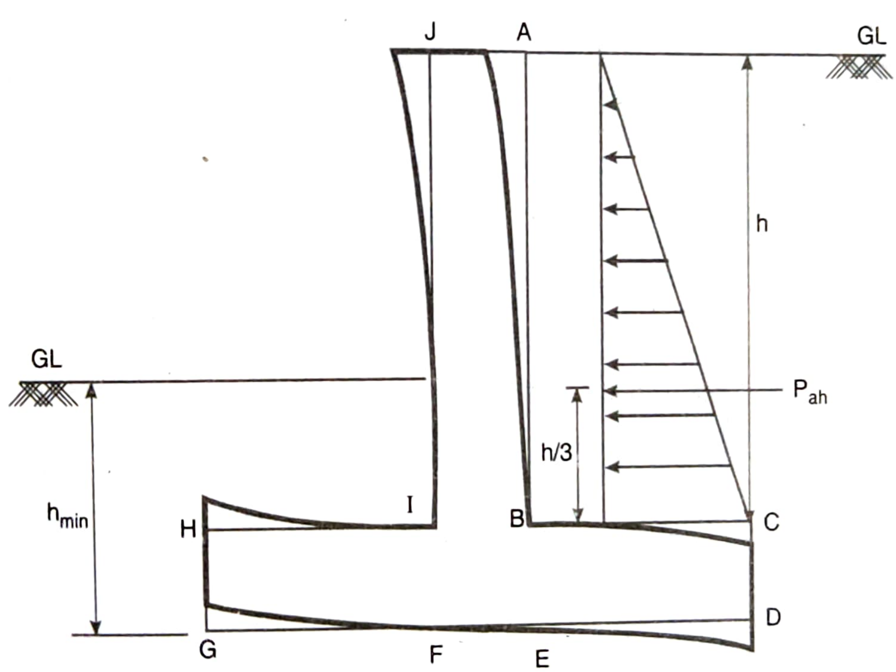


Fig. 16.8. Diagram showing the deflected shape of various components of retaining wall under loading

The stem of the retaining wall deflects as shown in the Fig. 16.8, developing tension on the face AB, retaining the earth. Therefore, main reinforcement is put along the face AB.

The depth assumed for stem must satisfy the bending moment criteria and subsequently the area of steel is calculated.

(2) Heel Slab: The heel slab is subjected to an upward soil pressure and a downward pressure due to the weight of the backfill supported on heel as shown in Fig. 16.9. The resultant pressure is calculated by subtracting these two and is downward as the pressure due to weight of backfill is more than the upward soil pressure. This causes tension on the top face *i.e.*, *BC* and hence main reinforcement is provided along this face.

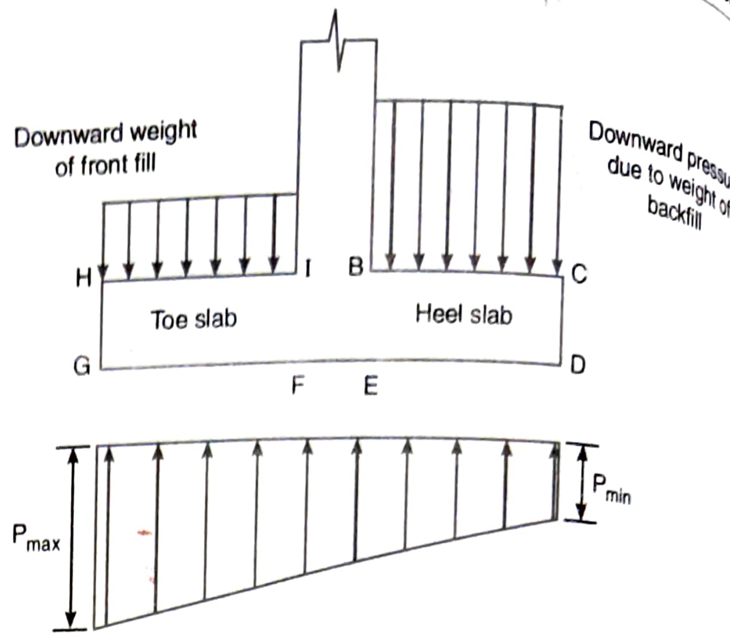


Fig. 16.9. Pressure distribution on base slab

(3) Toe Slab: The toe slab is also subjected to an upward soil pressure and a downward pressure due to the weight of the frontfill supported on toe slab as shown in Fig. 16.9. The weight of the frontfill is very small and hence neglected so the resultant pressure on the toe slab is upward which causes tension on the bottom face of the toe slab *i.e.*, GF and hence main reinforcement is put along this face.

Following Examples will explain the method of designing a cantilever retaining wall.

SOLVED EXAMPLES

Example 16.1. Design a cantilever retaining wall to retain horizontal earthen embankment of height 4 m above the ground level. The earthen backfill is having a density of 18 kN/m^3 and angle of internal friction as 30°. The safe bearing capacity of the soil is 180 kN/m^3. The coefficient of friction between soil and concrete is assumed to be 0.45. Use M20 concrete and Fe 415 steel.

Solution. Given:

$$f_{ck} = 20 \text{ N/mm}^2$$
$$f_y = 415 \text{ N/mm}^2$$
$$\phi = 30°$$
$$\mu = 0.45$$
$$\gamma = 18 \text{ kN/m}^3$$
$$\text{Safe bearing capacity of soil} = q_0 = 180 \text{ kN/m}^2$$
$$\text{Height of earthen embankment} = 4.0 \text{ m}$$

■ **Coefficient of active earth pressure (K_a)**

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \frac{1 - \sin 30°}{1 + \sin 30°}$$

$$K_a = \frac{1}{3}$$

Minimum depth of foundation (h_{min})

$$h_{min} = \frac{q_0}{\gamma}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$$

$$= \frac{180}{18}\left(\frac{1}{3}\right)^2$$

$$h_{min} = 1.11 \text{ m say } 1.2 \text{ m}$$

∴ Providing the depth of foundation as 1.2 m

Total height of retaining wall = Depth of foundation + Height of embankment

= 1.2 + 4.0

Total height of retaining wall (H) = 5.2 m

Preliminary dimensions of the retaining wall

(1) Base Width (b): It varies from 0.4 H to 0.6 H

Assuming b = 2.8 m

Length of toe slab = 0.3b to 0.4b

= 850 mm (say)

(2) Thickness of Base Slab

Thickness of base slab is assumed to be $\frac{H}{10} \simeq 500$ mm.

(3) Thickness of vertical wall or Stem (Refer Fig. 16.10)

Thickness of stem may be assumed as $\frac{H}{12}$ at base but here depth required from BM consideration is calculated.

Pressure at the base of the stem $= K_a \gamma h$ [$h = 5.2 - 0.5 = 4.7$ m]

$$= \frac{1}{3} \times 18 \times 4.7$$

$$= 28.2 \text{ kN/m}^2$$

Moment at the base of the stem $= \frac{1}{2}(K_a \gamma h) \cdot h \cdot \frac{h}{3}$

$$= \frac{1}{2} \times (28.2) \times 4.7 \times \frac{4.7}{3}$$

$$= 103.83 \text{ kNm}$$

Ultimate moment at the base of the stem

$$= 1.5 \times 103.83$$

$$= 155.74 \text{ kNm}$$

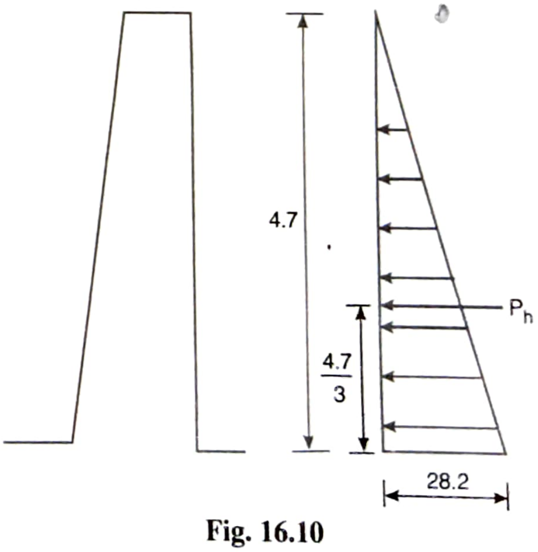


Fig. 16.10

- Minimum depth required for a balance section is

$$d_{reqd} = \sqrt{\frac{M_u}{R_u \cdot b}}$$

$R_u = 2.76$, for M20 concrete and Fe 415 steel

$$d_{reqd} = \sqrt{\frac{155.74 \times 10^6}{2.76 \times 1000}}$$

$$= \sqrt{\frac{155.74 \times 10^6}{2.76 \times 1000}}$$

$$= 238 \text{ mm}$$

Assuming 60 mm cover,

$$\text{Total depth required} = 238 + 60$$
$$= 298 \text{ mm}$$

Hence taking $D = 350$ mm at base of stem and reducing it to 150 mm at top. Figure 16.11 shows the trial section.

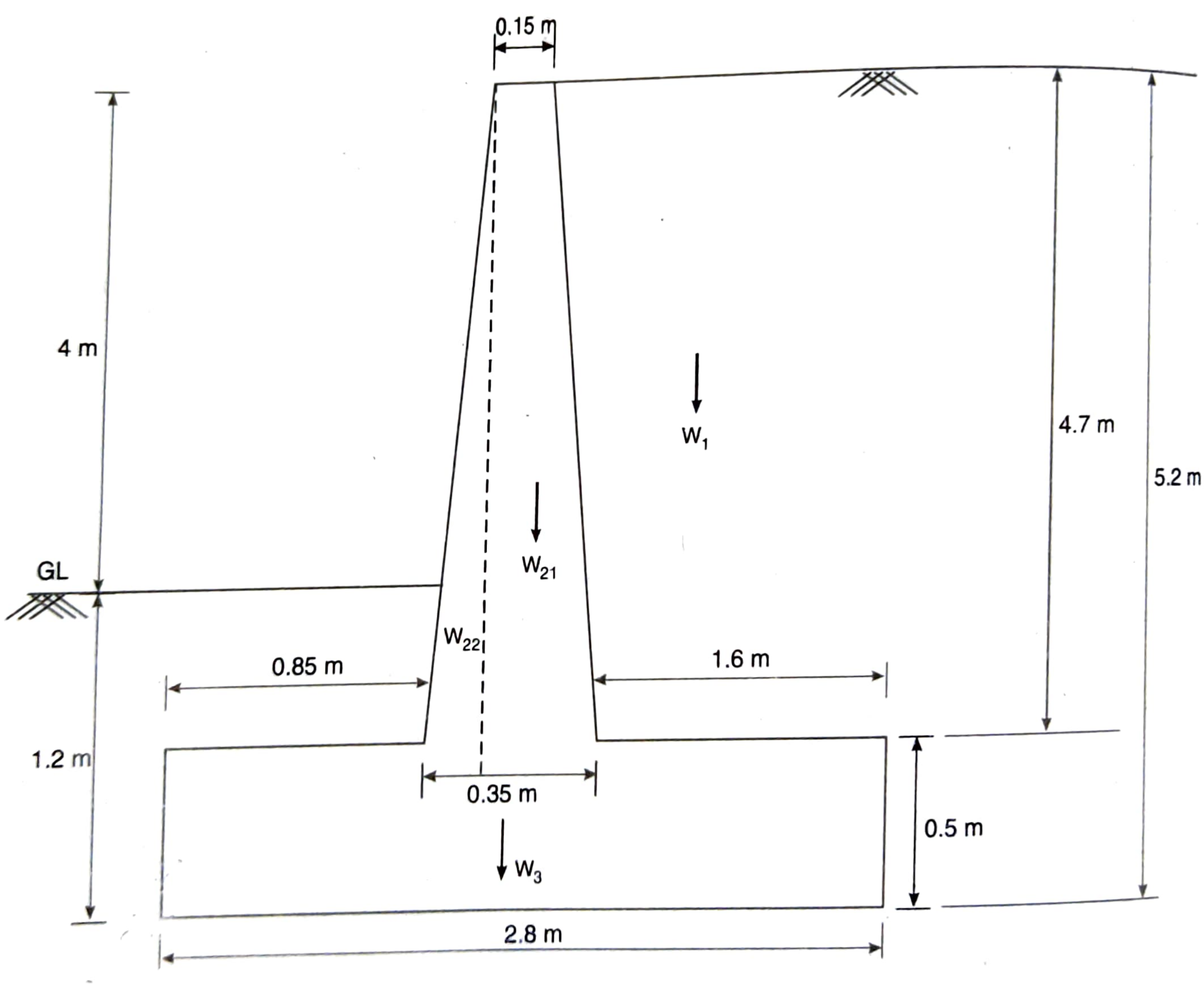


Fig. 16.11

■ **Forces Acting on the Retaining Wall:** Refer Fig. 16.11

Type of Force	Magnitude of Force (kN)	Position of force from toe end O (m)	Bending moment at toe end O (kNm)
(1) **Overturning force** $P_{ah} = \frac{1}{2}(K_a \gamma H).H$	$\frac{1}{2} \times \left(\frac{1}{3} \times 18 \times 5.2\right) \times 5.2$ = 81.12	$\frac{H}{3} = \frac{5.2}{3}$ = 1.733	81.12 × 1.733 = 140.61 ΣM_0 = **140.61**
(2) **Restoring forces**			
(*a*) Weight of backfill (W_1)	1.6 × 4.7 × 18 = 135.36	$2.8 - \frac{1.6}{2} = 2.0$	270.72
(*b*) Weight of stem			
(*i*) Weight of rectangular portion (W_{21})	0.15 × 4.7 × 25 = 17.625	$0.85 + 0.35 - \frac{0.15}{2}$ = 1.125	19.828
(*ii*) Weight of triangular portion (W_{22})	$\frac{1}{2} \times 0.2 \times 4.7 \times 25$ = 11.75	$0.85 + \frac{2}{3} \times 0.2 =$ 0.983	11.554
(*c*) Weight of base slab (W_3)	0.5 × 2.8 × 25 = 35	$\frac{2.8}{2} = 1.4$	49
	ΣW = **199.735**		ΣM_R = **351.1**

■ **Stability Checks**

(1) **Overturning**

$$\frac{0.9 M_R}{M_0} = \frac{0.9 \times 351.10}{140.61}$$

$$= 2.2 > 1.4 \text{ hence o.k.}$$

(2) **Sliding**

$$\frac{0.9 F_R}{F_S} \geq 1.4$$

$$F_R = \mu \Sigma W = 0.45 \times 199.735 = 89.88 \text{ kN}$$

$$F_S = P_{ah} = 81.12 \text{ kN}$$

$$\frac{0.9 F_R}{F_S} = \frac{0.9 \times 89.46}{81.12} = 0.99 < 1.4$$

Hence, shear key is to be provided to increase the resistance against sliding.

(3) **Base Pressure**

Resultant moment at toe end O $= M_R - M_0$

$= 351 - 140.61$

$= 210.49$ kNm

The resultant vertical load $= \Sigma W = 199.73$ kN

It acts at a distance $\bar{x}$ from the toe end O (refer Fig. 16.12)

$$\bar{x} = \frac{210.49}{199.73} = 1.05 \text{ m}$$

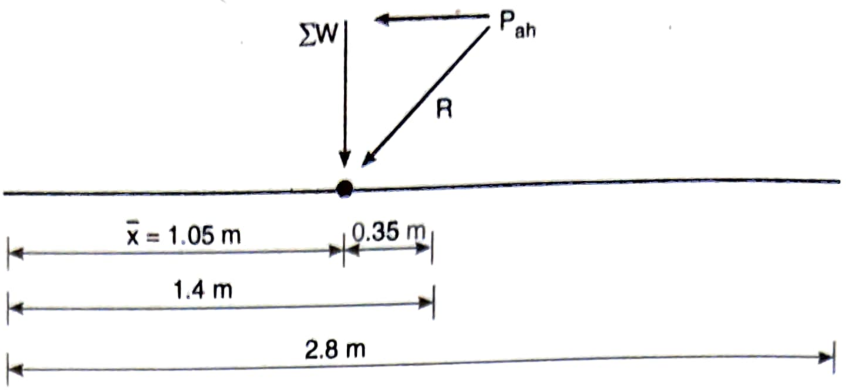


Fig. 16.12.

$$e = \frac{b}{2} - \bar{x} = 1.4 - 1.05$$

$$e = 0.35 \text{ m}$$

which lies in the middle third zone *i.e.*, $\frac{b}{6}$ from centre (0.466 m). Hence OK

■ **Maximum pressure at toe end O**

$$p_{max} = \frac{\Sigma W}{b}\left[1 + \frac{6e}{b}\right]$$

$$= \frac{199.73}{2.8}\left[1 + \frac{6 \times 0.35}{2.8}\right]$$

$$p_{max} = 124.83 \text{ kN/m}^2 < 180 \text{ kN/m}^2 \text{ (safe } BC \text{ of soil).}$$ Hence OK

■ **Minimum pressure at heel end $= p_{min}$**

$$p_{min} = \frac{\Sigma W}{b}\left[1 - \frac{6e}{b}\right]$$

$$= \frac{199.73}{2.8}\left[1 - \frac{6 \times 0.35}{2.8}\right]$$

$= 17.83$ kN/m^2, which is positive.

Hence OK, as no tension develops anywhere on the base slab.

1. Design of Stem

The depth required for stem is already checked while assuming the preliminary dimensions

$\therefore$

$$D = 350$$

$$d = 350 - 60$$

$$= 290 \text{ mm}$$

Maximum moment at base of stem = 155.73 kNm

Area of steel (A_{st}) in stem

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

$$155.73 \times 10^6 = 0.87 \times 415 \times A_{st} \times 290 \left[1 - \frac{415 A_{st}}{20 \times 1000 \times 290}\right]$$

$$A_{st}^2 - 13979.23 A_{st} + 20794392.5 = 0$$

On solving the equation

$$A_{st\ reqd} = 1693\ \text{mm}^2$$

Using 16 mm diameter bars, $A_\phi = 201\ \text{mm}^2$

$$\text{Spacing required} = \frac{201 \times 1000}{1693} = 118\ \text{mm}$$

Hence, provide 16 mm diameter, Fe 415 bars @ 100 mm c/c.

Distribution steel

Distribution steel is provided @ 0.12% of total x-sectional area

$$A_{st} = \frac{0.12}{100} \times 1000 \times \left(\frac{150 + 350}{2}\right) \qquad \left[\left(\frac{150+350}{2}\right) \text{is the average thickness of the stem}\right]$$

$$A_{st} = 300\ \text{mm}^2$$

Using 8 mm diameter bars, $A_\phi = 50.3\ \text{mm}^2$

$$\text{Spacing required} = \frac{50.3 \times 1000}{300} = 167.5\ \text{mm}$$

Hence, provide 8 mm diameter Fe 415 bars @ 150 mm c/c, on the inner face of the stem as distribution steel.

Similarly provide 8 mm diameter Fe 415 bars @ 150 mm c/c in both directions at the outer face (front face) of the stem as temperature and shrinkage reinforcement since this face is exposed to weather.

Check for shear

The critical section for shear is at a distance d from base of stem *i.e.*, $h = 4.7 - 0.29 = 4.41$

$$\text{Shear force at this section of the stem} = \frac{1}{2}\left(\frac{1}{3} \times 18 \times 4.41\right) \times 4.41$$

$$= 58.3\ \text{kN}$$

$$V_u = 1.5 \times 58.3$$

$$V_u = 87.52\ \text{KN}$$

$$\text{Nominal shear stress} = \frac{V_u}{bd}$$

$$\tau_v = \frac{87.52 \times 1000}{1000 \times 290} = 0.30\ \text{N/mm}^2$$

For

$$p_t = \frac{201 \times 1000}{1000 \times 290} = 0.69\%$$

$$\tau_c = 0.54\ \text{N/mm}^2 \qquad \text{(from Table 5.1)}$$

$$\tau_c = 0.54\ \text{N/mm}^2 > \tau_v \quad \text{hence OK.}$$

Curtailment of tension reinforcement

- As the stem of retaining wall behaves like a cantilever, the bending moment goes on reducing towards the top of the wall and becomes zero at the top. Therefore, tension reinforcement can be curtailed along the height of the stem.
- Development length, L_d, for 16 mm diameter bars

$$L_d = \frac{0.87 f_y \phi}{4\tau_{bd.}}$$

$$L_d = \frac{0.87 \times 415 \times 16}{4 \times 1.6 \times 1.2}$$

$$= 752 \text{ mm}$$

Therefore, no bar can be curtailed up to a distance of 752 mm from base of the stem. Curtailing bars at a distance 1000 mm from base of the stem *i.e.*,

$$4700 - 1000 = 3700 \text{ mm from top of the stem}$$

$$\text{Total depth at this section} = 150 + \frac{200 \times 3700}{4700}$$

$$= 307 \text{ mm}$$

$$\text{Effective depth at this section} = 307 - 60 = 247 \text{ mm}$$

Moment due to earth pressure at 3.7 m from top

$$= \frac{K_a \gamma h^3}{6}$$

$$= \frac{1}{6}\left[\frac{1}{3} \times 18 \times 3.7^3\right]$$

$$= 50.7 \text{ kNm}$$

$$M_u = 1.5 \times 50.7$$

$$M_u = 76 \text{ kNm}$$

Area of steel required for an ultimate bending moment of 76 kNm

$$76 \times 10^6 = 0.87 \times 415 \times A_{st} \times 247\left[1 - \frac{415 A_{st}}{20 \times 1000 \times 247}\right]$$

On solving, we get $A_{st\ reqd} = 924 \text{ mm}^2$

Using 16 mm diameter bars,

$$\text{Spacing required} = \frac{201 \times 1000}{924} = 217 \text{ mm}$$

Hence half of the bars can be curtailed but as per IS code, 12ϕ or d distance, whichever is more, is to be provided beyond the point of curtailment. Hence curtailment the bars at 1.3 m from base or 3.4 m from top of stem. Thus providing 16 mm diameter bars @ 200 mm c/c after a distance of 1.3 m from base of stream.

- Similarly, one more curtailment can be done at 1.5 m from top of stem.

$$\text{Moment at this section} = \left(\frac{18 \times 1.5^3}{3 \times 6}\right)$$

$$= 3.375 \text{ kNm}$$

$$M_u = 1.5 \times 3.375$$
$$= 5.1 \text{ kNm}$$

$$\text{Depth at this section} = 150 + \frac{200}{4700} \times 3200$$
$$= 286 \text{ mm}$$
$$d = 286 - 60 = 226 \text{ mm}$$

$$5.1 \times 10^6 = 0.87 \times 415 \times A_{st} \times 226 \left[1 - \frac{415 A_{st}}{20 \times 1000 \times 226}\right]$$

$$A_{st\ reqd} = 65 \text{ mm}^2 < A_{st\ min} \text{ i.e., } 300 \text{ mm}^2$$

Hence curtailing another half of the bars at 1.5 m from top and providing 16 mm diameter bars @ 400 mm c/c.

2. **Design of Heel Slab**

The pressure distribution on heel slab is shown in Fig. 16.13.

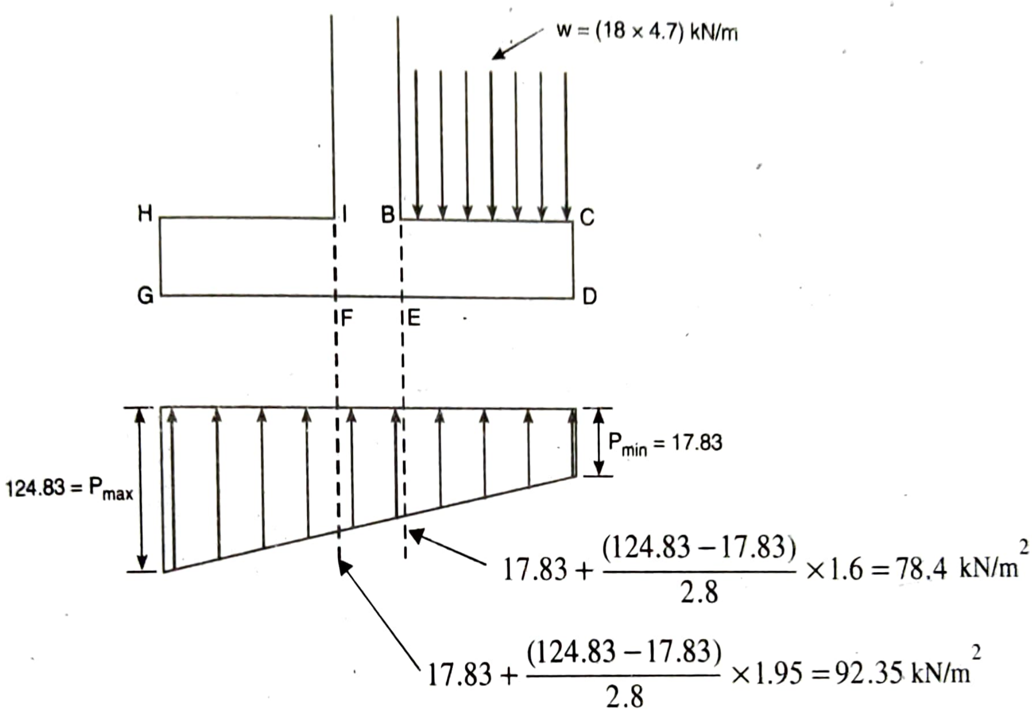


Fig. 16.13.

Weight of earth supported on heel = 18 × 4.7 = 84.6 kN/m

Self weight of heel slab = 0.5 × 1.0 × 25 = 12.5 kN/m

Total load = 97.1 kN/m

$$\text{Maximum bending moment at } B = \frac{97.1 \times 1.6^2}{2} - \frac{17.83 \times 1.6^2}{2} - \frac{1}{2}(78.4 - 17.83) \times 1.6 \times \frac{1.6}{3}$$
$$= 101.5 - 25.8$$
$$= 75.7 \text{ kNm}$$
$$M_u = 1.5 \times 75.7 = 113.6 \text{ kNm}$$

$$d_{reqd} = \sqrt{\frac{113.6 \times 10^6}{2.76 \times 1000}} = 202 \text{ mm} < 440 \text{ mm. Hence OK.}$$

Area of steel for heel slab

$$113.5 \times 10^6 = 0.87 \times 415 \times 440\left(1 - \frac{415 A_{st}}{20 \times 1000 \times 440}\right)$$

$$A_{st} = 741 \text{ mm}^2$$

$\therefore$ Spacing of 12 mm bars $= \dfrac{113 \times 1000}{4741} = 152$ mm

$\therefore$ Provide 12 mm diameter bars @ 150 mm c/c at the top face of the heel slab *i.e.*, *BC*.

Distribution steel is provided @ 0.12% of sectional area in the other direction

$$\frac{0.12}{100} \times 1000 \times 500 = 600 \text{ mm}^2$$

Using 10 mm diameter bars, $A_\phi = 78.5 \text{ mm}^2$, spacing required = 100 mm

Hence, provide same 10 mm dia bars @ 100 mm c/c in the other direction.

3. Design of Toe Slab

The weight of frontfill above the toe slab is neglected and maximum moment is calculated at the face of the stem.

$$\text{Maximum moment} = \frac{92.35 \times 0.85^2}{2} + \frac{1}{2}(124.83 - 92.35) \times 0.85 \times \frac{2}{3} \times 0.85$$

$$= 33.36 + 7.82 = 41.2 \text{ kNm}$$

$$M_u = 1.5 \times 41.2 = 61.8 \text{ kNm}$$

Area of steel for toe slab

$$61.80 \times 10^6 = 0.87 \times 415 \times A_{st} \times 440\left[1 - \frac{415 A_{st}}{20 \times 1000 \times 440}\right]$$

$$A_{st}^2 - 21209.88 A_{st} + 8251001.3 = 0$$

$$A_{st} = 396 \text{ mm}^2 < A_{st\,min}\ (600 \text{ mm}^2)$$

Hence providing minimum area of steel of 600 mm^2. Therefore provide 10 mm diameter bars @ 100 mm c/c in both directions.

4. Design of Shear key

As the wall is not safe in sliding, shear key is to be provided below the stem as shown in Fig. 16.14.

Pressure at face of shear key = 92.35 kN/m

Coefficient of passive earth pressure $= \dfrac{1 + \sin\phi}{1 - \sin\phi}$

$$K_P = \frac{1.5}{0.5} = 3$$

Let the depth of key $= a$

Resistance offered by shear key $= 3 \times 92.35 \times a$

$= 277.05a$

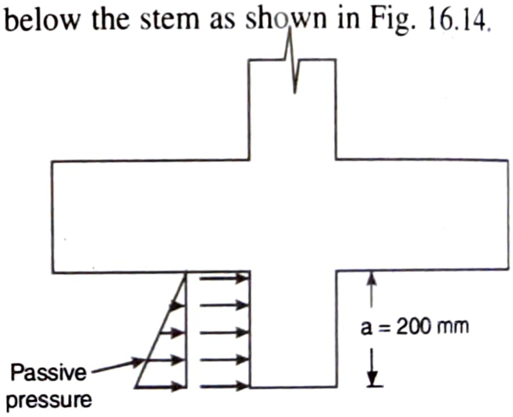


Fig. 16.14

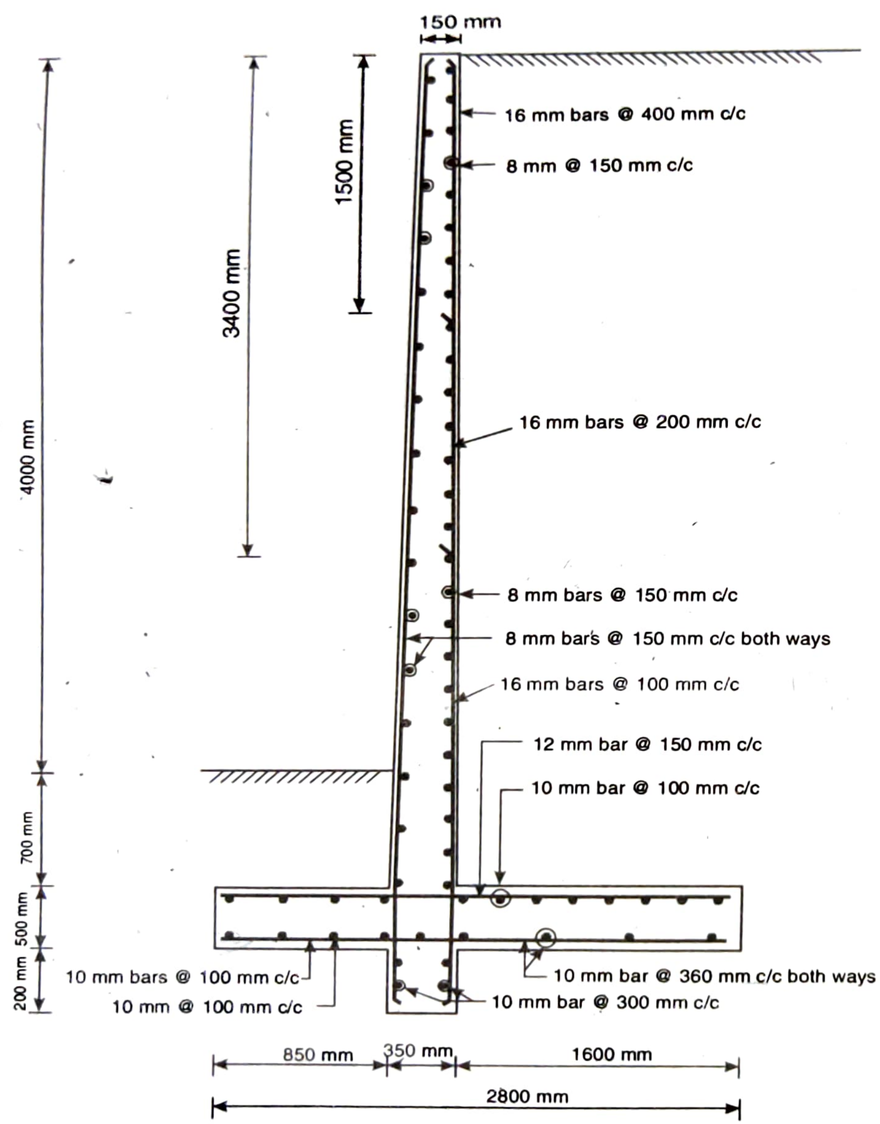
Factor of safety against sliding alongwith shear key

$$= \frac{0.9\mu \sum W + 277.05a}{P_{ah}} = \frac{0.9 \times 89.88 + 277.05a}{81.12} = 1.4$$

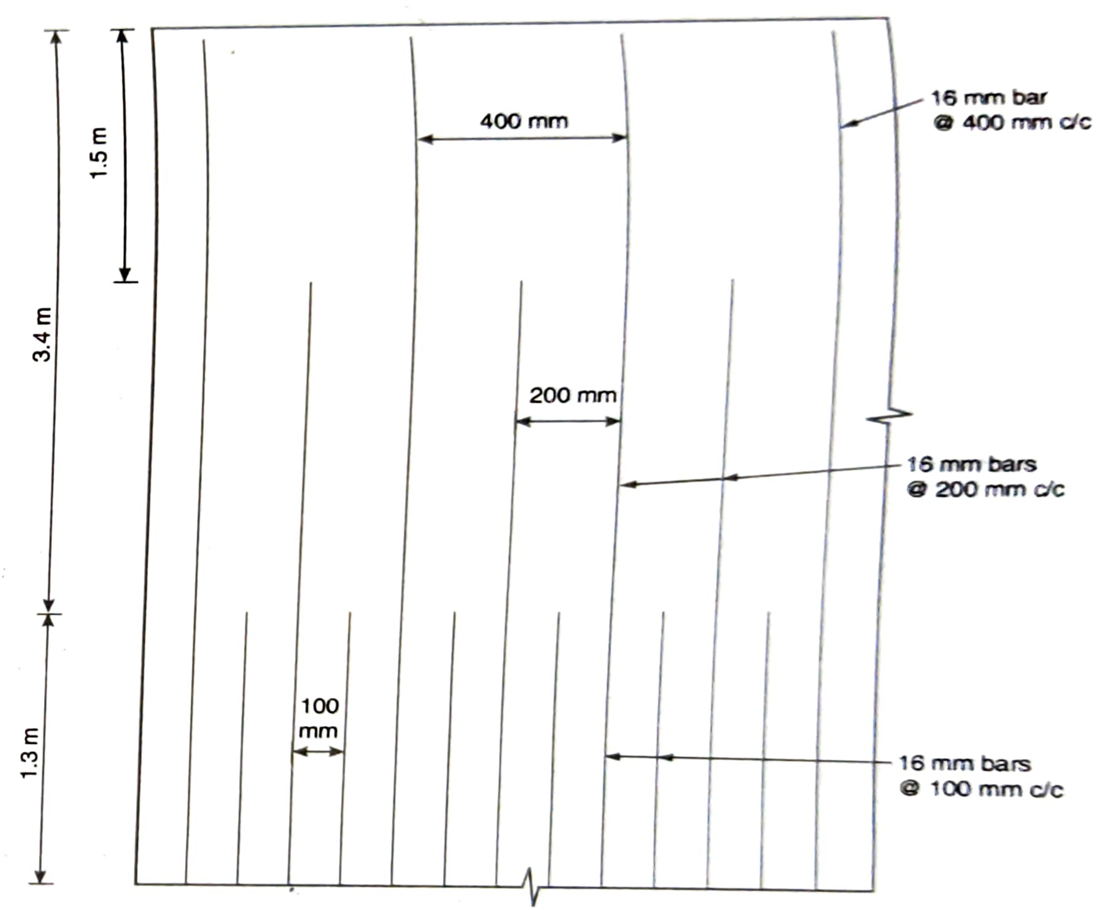
$\Rightarrow$ $a = 0.118$ m

However, provide a 200 mm × 200 mm shear key.

The details of the reinforcement are shown in Fig. 16.15.



(*a*) Section

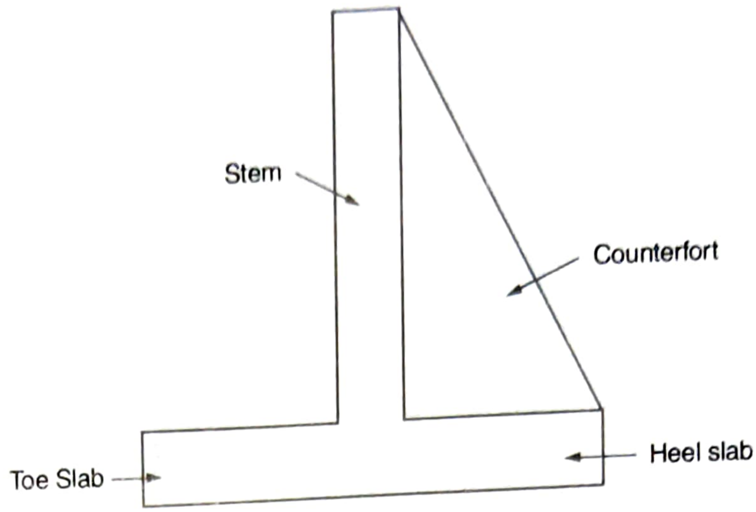


(*b*) L-Section of Stem

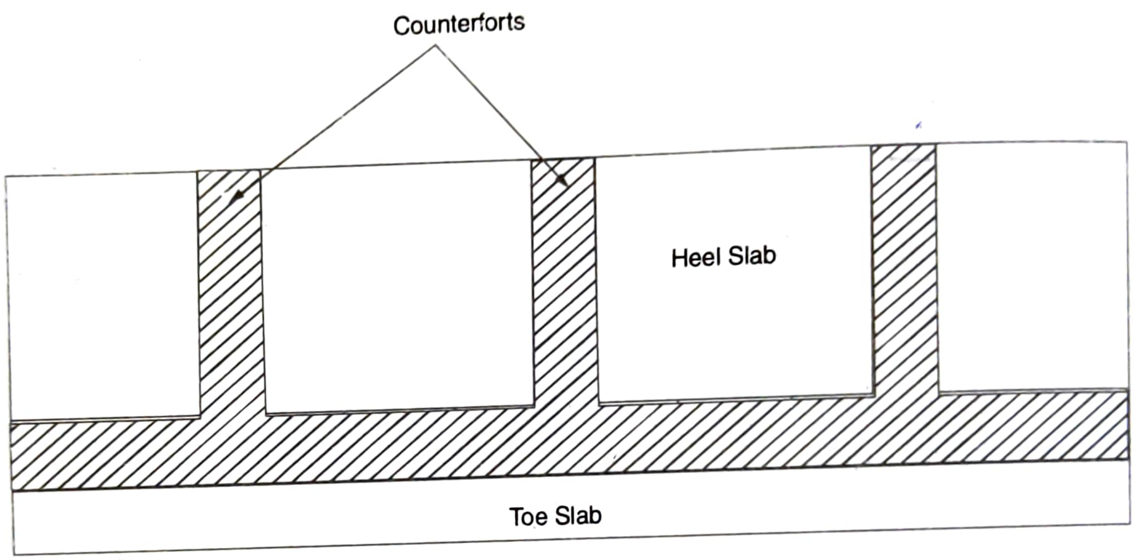
Fig. 16.15. Cantilever Retaining Wall

16.8. COUNTERFORT RETAINING WALL

When the height of the backfill is more than 6.0 m, then counterfort retaining wall is used. It is uneconomical to provide cantilever retaining wall, for larger heights, as the section becomes very thick due to larger bending moments. The structural behaviour of the stem and heel in a counterfort retaining wall is entirely different from the cantilever retaining wall. As the counterfort supports the stem and heel, they behave like a continuous slabs supported on the three edges *i.e.*, two at counterforts and one on base slab as shown in Fig. 16.16.



(*a*) Elevation



(*b*) Plan

Fig. 16.16. Counterfort retaining wall

The design of the various components of the counterfort retaining wall is done as explained below:

(1) Design of Stem: The stem of the counterfort retaining wall acts as a continuous slab supported on the counterforts which are spaced at 3 to 3.5 m along the length of the retaining wall. The stem is subjected to earth pressure which tries to deflect the wall away causing tension on the outer face and compression on the inner face. Therefore, main reinforcement is put on the outer face along the length of the retaining wall. Due to the fixity provided by the counterforts supports, some negative bending moment develops at the supports which causes tension on the inner face near the counterforts. Hence, main reinforcement is also provided at inner face near the counterforts. The maximum bending moment occurs at the base of the stem.

The load at the base of the stem, say w per m length is determined as below:

$$w = p_a \times 1 \times 1$$

$$w = K_a \gamma H \text{ per m length}$$

The maximum negative bending moment in the stem at the counterfort supports may be assumed as $\frac{wl^2}{12}$ and the positive bending moment at the mid span may be taken as $\frac{wl^2}{16}$.

(2) Design of Counterforts: Counterforts are attached to the stem and the heel slab. They act like a T-beam of varying cross-section. The earth pressure acting on the stem is transferred to the counterforts whiich tries to separate the counterforts from stem. Therefore, horizontal ties are provided which connect stem and the counterfort together firmly. Similarly the downward weight of backfill acting on the heel slab also tries to separate out the heel slab and counterfort and hence ties are also provided to connect the heel slab and the counterfort. The counterfort act like a T-beam of varying section supported on the edges AB and BC and free at AC. As the outer face AC is in tension, main reinforcement is put parallel to the edge AC. The depth of the T-beam is considered as the depth at junction of stem and base and taken as shown in Fig. 16.17.

The spacing of counterforts is kept about 3 to 3.5 m and the thickness of counterforts may be taken as same as that of base slab. Counterforts are designed for the maximum bending moment given by

$$M_{\max} = \left[K_a \frac{\gamma h^3}{6} . l \right]$$

where h is the height of retaining wall above base

l = spacing of counterforts

(3) Design of Heel Slab: The heel slab behaves like the stem. It acts as a continuous slab supporting on three edges and subjected to downward weight of backfill and upward soil pressure. The resultant load (p) acts in the downward direction, so the heel slab deflects downward, causing tension at the bottom face in between the counterforts and at the top face near the counterforts. The maximum negative moment occurs at the counterforts and may be assumed as $\frac{pl^2}{12}$. The maximum positive bending moment may be assumed as $\frac{pl^2}{16}$.

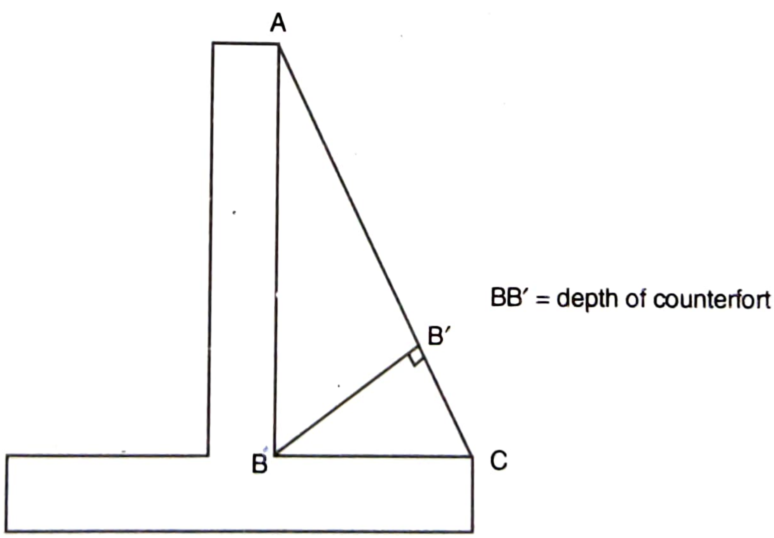


Fig. 16.17. Counterfort retaining wall

(4) Design of Toe Slab: The design of toe slab in a counterfort retaining wall is same as that in a cantilever retaining wall. It behaves like a cantilever bending upwards due to soil pressure. If counterforts are also provided on the toe slab then it also acts like a continuous slab supported on counterforts.

Note: *When the height of the retained material becomes larger such that even the counterfort retaining wall becomes uneconomical then the stem of retaining wall is anchored at the back by some anchor rod or tie rods. This back anchor takes all the lateral earth pressure and results in great reduction of bending moment and shear force values.*

The following example will explain the design of counterfort retaining wall in detail.

Example 16.2. Design a counterfort retaining wall to retain 4 m earth above ground level. The top of the earth is to be level. The density of earth is 15 kN/m^2. The angle of internal friction of soil is 30°. The safe bearing capacity of soil is 200 kN/m^2 and the coefficient of friction between soil and wall is 0.6. **[M.D.U. Haryana May 2012]**

Solution. Given:

$$\gamma = 15 \text{ kN/m}^2,$$
$$\phi = 30°$$
$$\mu = 0.6,$$
$$q_0 = 200 \text{ kN/m}^2$$
$$h = 4.0 \text{ m}$$

Using M20 concrete and Fe 415 steel

$$f_y = 415 \text{ N/mm}^2, \quad f_{ck} = 20 \text{ N/mm}^2$$

■ **Minimum Depth of Foundation**

$$h_{\min} = \frac{q_0}{\gamma}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$$

$$= \frac{200}{15}\left(\frac{1-\sin 30°}{1+\sin 30°}\right)^2$$

$$h_{\min} = 1.48 \text{ m}$$

Taking depth of foundation $= 1.5$ m

Overall depth of wall $= 4.0 + 1.5 = 5.5$ m

■ **Proportioning of retaining wall**

1. Width of base slab is kept approximately as 0.6 H

$\therefore$ $b = 3.0$ m

2. Assuming thickness of base slab $= \dfrac{H}{12}$ say 500 mm.

3. Toe projection = 0.3 b say 0.9 m

4. Spacing of counterforts = 3.0 m.

5. Width of counterforts = 0.05 H say 300 mm

6. Thickness of stem $= \dfrac{H}{20}$ say 300 mm, (equal to the thickness of counterforts).

Figure 16.18 shows the trial section of the retaining wall.

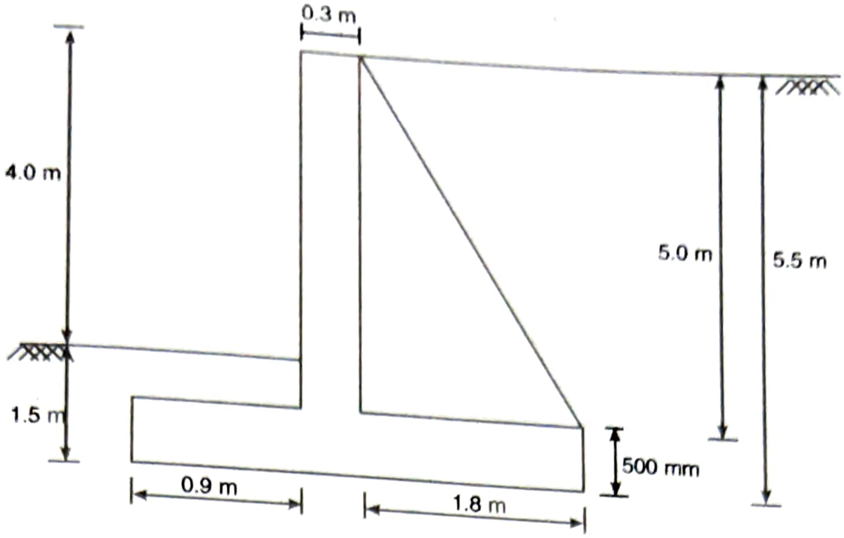


Fig. 16.18. Trial section

■ **Forces Acting on the Retaining Wall**

Force (Type)	Force (kN)	Distance from toe edge m	Moment about toe edge (kNm)
1. **Overturning force** $P_h = \frac{1}{2}(K_a\gamma H)H$	$\frac{1}{2}\times\left(\frac{1}{3}\times 15\times 5.5\right)5.5$ $= 75.625$	$\frac{H}{3}=\frac{5.5}{3}=1.833$	138.65
	$F_S = 75.625$ kN		$M_0 = 138.65$
2. **Restoring forces**			
(*i*) Weight of backfill (W_1)	15 × 5 × 1.8 = 135	$3.0-\frac{1.8}{2}=2.1$	283.5
(*ii*) Weight of stem (W_2)	0.3 × 5.0 × 25 = 37.5	$0.9+\frac{0.3}{2}=0.915$	34.31
(*iii*) Weight of base slab (W_3)	$0.5\times 3\times 25 = 37.5$	$\frac{3.0}{2}=1.5$	56.25
	$\Sigma W = 210$ kN		$M_R = 374.06$ kNm

■ **Stability Checks:**

(1) **Overturning:**

$$\text{Factor of safety against overturning} = \frac{0.9M_R}{M_0}$$

$$= \frac{0.9\times 374.06}{138.65}$$

$= 2.4 > 1.4$ hence OK

(2) **Sliding:**

$$\text{Factor of safety against sliding} = \frac{0.9\mu\sum W}{F_S}$$

$$= \frac{0.9 \times 0.6 \times 210}{75.625}$$

$$= 1.49 > 1.4 \quad \text{hence o.k.}$$

(3) **Base pressure check**

$$\text{Net moment about toe edge} = M_R - M_0$$

$$= 374.06 - 138.65$$

$$= 235.41 \text{ kNm}$$

The point of application of resultant where it cuts base;

$$\bar{x} = \frac{\text{Net moment}}{\sum W} = \frac{235.41}{210}$$

$$\bar{x} = 1.121 \text{ m}$$

$$e = \frac{b}{2} - x = \frac{3.0}{2} - 1.121$$

$$e = 0.379 \text{ m} < \frac{b}{6} \text{ i.e., } 0.5 \text{ m} \quad \text{Hence OK}$$

$$p_{max} = \frac{\sum W}{b}\left[1 + \frac{6e}{b}\right]$$

$$= \frac{210}{3.0}\left[1 + \frac{6 \times 0.379}{3.0}\right]$$

$$p_{max} = 123.06 \text{ kN/m}^2 < 200 \text{ kN/m}^2 \text{ (safe B.C. of soil)} \quad \text{Hence OK}$$

$$p_{min} = \frac{\sum W}{b}\left[1 - \frac{6e}{b}\right]$$

$$= \frac{210}{3.0}\left[1 - \frac{6 \times 0.379}{3}\right]$$

$$p_{min} = 16.94 \text{ kN/m}^2 \quad \text{which is +ve. Hence ok.}$$

- **Design of stem**

Maximum horizontal pressure at the base of stem = $\left(\frac{1}{3} \times 15 \times 5\right)$

$$P_h = 25 \text{ kN/m}^2$$

The stem acts as a horizontal slab supported on counterforts with $w = 25 \times 1 = 25$ kN/m

- Maximum –ve moment at counterfort supports

$$= \frac{w.l^2}{12} = \frac{25 \times 3^2}{12} = 18.75 \text{ kNm}$$

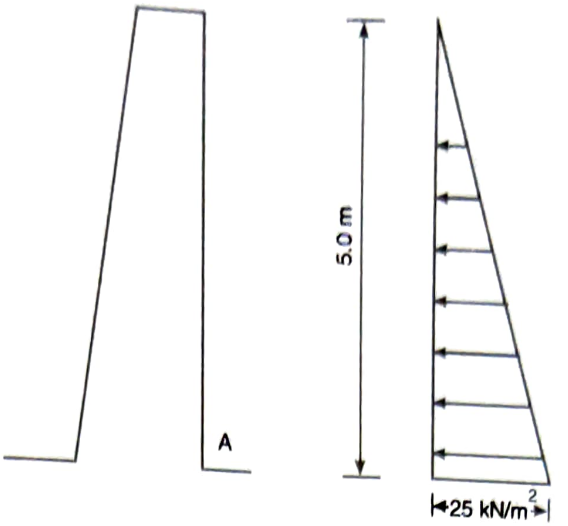


Fig. 16.19

$$M_u = 1.5 \text{ M}$$
$$= 1.5 \times 18.75$$
$$= 28.125 \text{ kNm}$$

Maximum positive moment at mid span

$$= \frac{w.l^2}{16} = \frac{25 \times 3^2}{16}$$
$$= 14.1 \text{ kNm}$$
$$M_u = 1.5 \times 14.1 = 21.1 \text{ kNm}$$

■ **Depth check**

$$d = \sqrt{\frac{M_u}{R_u .b}} = \sqrt{\frac{28.12 \times 10^6}{2.76 \times 1000}}$$
$$= 101 \text{ mm} < 250 \text{ mm} \quad \text{hence OK}$$

[assuming effective cover as 50 mm, $d = 300 - 50 = 250$ mm]

■ **Area of steel required**

$$28.125 \times 10^6 = 0.87 \times 415 \times A_{st} \times 250\left[1 - \frac{415 A_{st}}{20 \times 1000 \times 250}\right]$$
$$A_{st} = 320 \text{ mm}^2$$

Using 10 mm diameter bars

$$A_\phi = 78.5 \text{ mm}^2$$
$$\text{Spacing required} = \frac{78.5 \times 1000}{320} = 245 \text{ mm}$$
$$A_{st\ min} = 0.12\% \text{ of x-sectional area}$$
$$= 0.12 \times 1000 \times 0.3 = 360 \text{ mm}^2 > 320 \text{ mm}^2$$

Hence $A_{st\ min}$ is to be provided

$$\text{Spacing required} = \frac{78.5 \times 1000}{360} = 218 \text{ mm}$$

Hence provide 10 mm diameter bars @ 200 mm c/c in both direction, all along the height of stem. It also takes care of +ve moment *i.e.*, 21.1 kNm at mid span. The spacing can be increased to 300 mm near the top of the stem as the pressure decrease towards the top of the stem.

- **Shear check:** Maximum shear force at the face of counterfort

$$= \frac{25 \times (3 - 0.3)}{2} = 33.75 \text{ kN}$$

$$V_u = 1.5 \times 33.75$$
$$= 50.625 \text{ kN}$$

$$\tau_v = \frac{V_u}{bd} = \frac{50.625 \times 10^3}{1000 \times 250}$$

$$\tau_v = 0.2 \text{ N/mm}^2$$
$$p_t = 0.157\%$$
$$\tau_c = 0.28 \text{ N/mm}^2$$ [From table 19 IS 456 or Table 5.1 of the book]

$\tau_v < \tau_c$ Hence ok.

- **Design of toe slab:** The pressure distribution under the base slab is as shown below

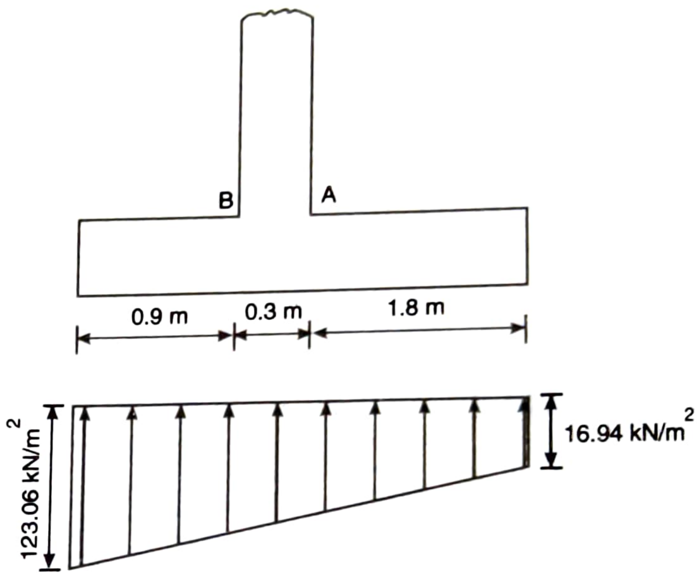


Fig. 16.20.

$$\text{Pressure below point } A = 16.94 + \left(\frac{123.06 - 16.94}{3.0}\right) \times 1.8$$
$$= 80.61 \text{ kN/m}^2$$

$$\text{Pressure below point } B = 16.94 + \frac{(123.06 - 16.94)}{3.0} \times 2.1$$
$$= 91.22 \text{ kN/m}^2$$

Neglecting the weight of earth retained on the toe slab, the cantilever moment at the section *B* is

$$= 91.22 \times \frac{0.9^2}{2} + \frac{1}{2}(123.06 - 91.22) \times 0.9 \times \frac{2}{3} \times 0.9$$
$$= 45.54 \text{ kNm}$$

$$M_u = 1.5 \times 45.54$$
$$= 68.31 \text{ kNm}$$

$$d_{reqd} = \sqrt{\frac{68.31 \times 10^6}{2.76 \times 1000}}$$

$$= 157 \text{ mm} < d \text{ provided. Hence o.k.}$$

$$\text{Total depth} = 500 \text{ mm}$$
$$\text{Effective cover} = 60 \text{ mm}$$
$$d_{provided} = 500 - 60 = 440 \text{ mm}$$

Area of steel required for toe slab:

$$68.31 \times 10^6 = 0.87 \times 415 \times A_{st} \times 440 \left[1 - \frac{415 A_{st}}{20 \times 1000 \times 440}\right]$$

$$A_{st\ reqd} = 440 \text{ mm}^2$$

$$A_{st\ min} = \frac{0.12}{100} \times 1000 \times 500 = 600 \text{ mm}^2 > 440 \text{ mm}^2$$

hence provide $A_{st} = 600 \text{ mm}^2$

Using 12 mm diameter bars, $A_\phi = 113 \text{ mm}^2$

$$\text{Spacing required} = \frac{113 \times 1000}{600} = 188 \text{ mm}$$

Hence provide 10 mm diameter bars @ 180 mm c/c in both directions in toe slab.

■ **Shear design:** The critical section for shear is at a distance 'd' from face of the stem *i.e.*, 0.44 m from stem or

$$0.9 - 0.44 = 0.46 \text{ m from toe edge}$$

$$\text{Pressure at this section} = 91.22 + \frac{1}{2}\left(\frac{123.06 - 91.22}{0.9}\right) \times 0.44$$

$$= 99.0 \text{ kN/m}^2$$

$$\text{S.F. at this section} = 99 \times 0.46 + \frac{1}{2}(123.06 - 99) \times 0.46$$

$$= 51.1 \text{ kN per m run}$$

$$V_u = 1.5 \times 51.1$$
$$V_u = 76.65 \text{ kN}$$

$$\tau_V = \frac{76.65 \times 10^3}{1000 \times 440} = 0.17 \text{ N/mm}^2$$

$$p_t = \frac{600 \times 100}{1000 \times 440} = 0.14\%$$

$$\tau_c = 0.28 \text{ N/mm}^2$$

$$\tau_v < \tau_c \quad \text{Hence OK}$$

■ **Design of heel slab**

The heel slab also acts as a continuous slab supported on counterforts like stem.

$$\text{Weight of backfill} = 1.0 \times 5.0 \times 15$$
$$= 75 \text{ kN/m}^2 \text{ per m run}$$
$$\text{Self weight of slab} = 1.0 \times 0.5 \times 25 = 12.5 \text{ kN/m}^2$$
$$\text{Total downward weight} = 75 + 12.5 = 87.5 \text{ kN/m}^2$$

Maximum downward pressure at the edge of the heel slab

$$= 87.5 - 16.94 = 70.56 \text{ kN/m}^2$$

$$M = \frac{70.56 \times 3^2}{12} = 52.92 \text{ kNm}$$

$$M_u = 1.5 \times 52.92$$
$$M_u = 79.38 \text{ kNm}$$

Area of Steel required:

$$79.38 \times 10^6 = 0.87 \times 415 \times A_{st} \times 440 \left[1 - \frac{415 A_{st}}{20 \times 1000 \times 440}\right]$$

$$A_{st} = 512 \text{ mm}^2 < 600 \text{ mm}^2 \ (A_{st\ min})$$

Hence provide 10 mm diameter @ 180 mm c/c in both directions.

■ **Design of counterforts:** Counterforts are designed as a triangular beam (beam of varying depth) supported on the stem and heel slab. It is also to be designed for the tension which tries to pull the counterfort away from stem and heel.

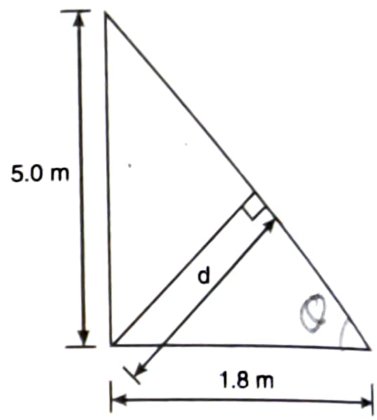


Fig. 16.21.

$$\tan \theta = \frac{5.0}{1.8} = 2.77$$

$$\theta = 43.71°$$

Depth of the triangular beam, (*d*)

$$d = 1.8 \sin \theta$$
$$= 1.8 \sin 43.71°$$
$$d = 1.243 \text{ m}$$

Maximum moment on the counterforts

$$= \left(\frac{1}{2} K_a \gamma h.h.\frac{h}{3}\right) \times L \quad \text{where } L \text{ is the spacing of counterforts}$$

$$= \left(\frac{1}{2} \times \frac{1}{3} \times 15 \times 5 \times 5 \times \frac{5}{3}\right) \times 3.0$$

$$M = 312.5 \text{ kNm}$$
$$M_u = 1.5 \times 312.5 = 468.75 \text{ kNm}$$

Area of steel required:

$$468.75 \times 10^6 = 0.87 \times 415 \times A_{st} \times 1243 \left[1 - \frac{415 A_{st}}{20 \times 300 \times 1243}\right]$$

$$A_{st} = 1114 \text{ mm}^2$$

$$A_{st\,min} = \frac{0.85bd}{f_y} = \frac{0.85 \times 300 \times 1243}{415}$$

$$= 763 \text{ mm}^2 < 1114 \text{ mm}^2 \quad \text{Hence OK}$$

Providing 4 bars bars of 20 mm diameter

$$A_{st\,provided} = 4 \times \frac{\pi}{4} \times 20^2 = 1256 \text{ mm}^2 \quad \text{[curtailing 2 bars near the top]}$$

■ Design for Horizontal Tension in Counterforts

Horizontal ties are used for taking horizontal tension, caused due to the lateral earh pressure.

Considering the bottom 1 m height of the stem.

Maximum lateral pressure at the bottom

$$= K_a \gamma h$$

$$= \frac{1}{3} \times 15 \times 5$$

$$= 25 \text{ kN/m}^2$$

Total lateral pressure to be taken by counterforts

$$= 25(3 - 0.3) \text{ per m run}$$

$$= 67.5 \text{ kN}$$

$$\text{Factored tensile force} = 1.5 \times 67.5$$

$$= 101.25 \text{ kN}$$

Area of steel required: $T = 0.87 f_y A_{st}$

$$A_{st} = \frac{101.25 \times 1000}{0.87 \times 415} = 281 \text{ mm}^2$$

Providing 10 mm bars $A_\phi = 78.5 \text{ mm}^2$

$$\text{Spacing required} = \frac{78.5 \times 1000}{281} = 279 \text{ mm}$$

Provide 10 mm diameter ties @ 260 mm c/c in the horizontal direction.

■ Design for Vertical tension in counterforts

The vertical tension in counterforts is caused due to the downward pressure which tries to separate out the counterfort and the heel.

Maximum downward pressure on the counterfort at the edge of heel = 70.56 kN/m^2

$$\text{Factored tensile force} = 1.5 \times 70.56$$

$$= 105.84 \text{ kN}$$

$$\text{Area of steel required } A_{st} = \frac{T}{0.87 f_y}$$

$$= \frac{105.84 \times 1000}{0.87 \times 415}$$

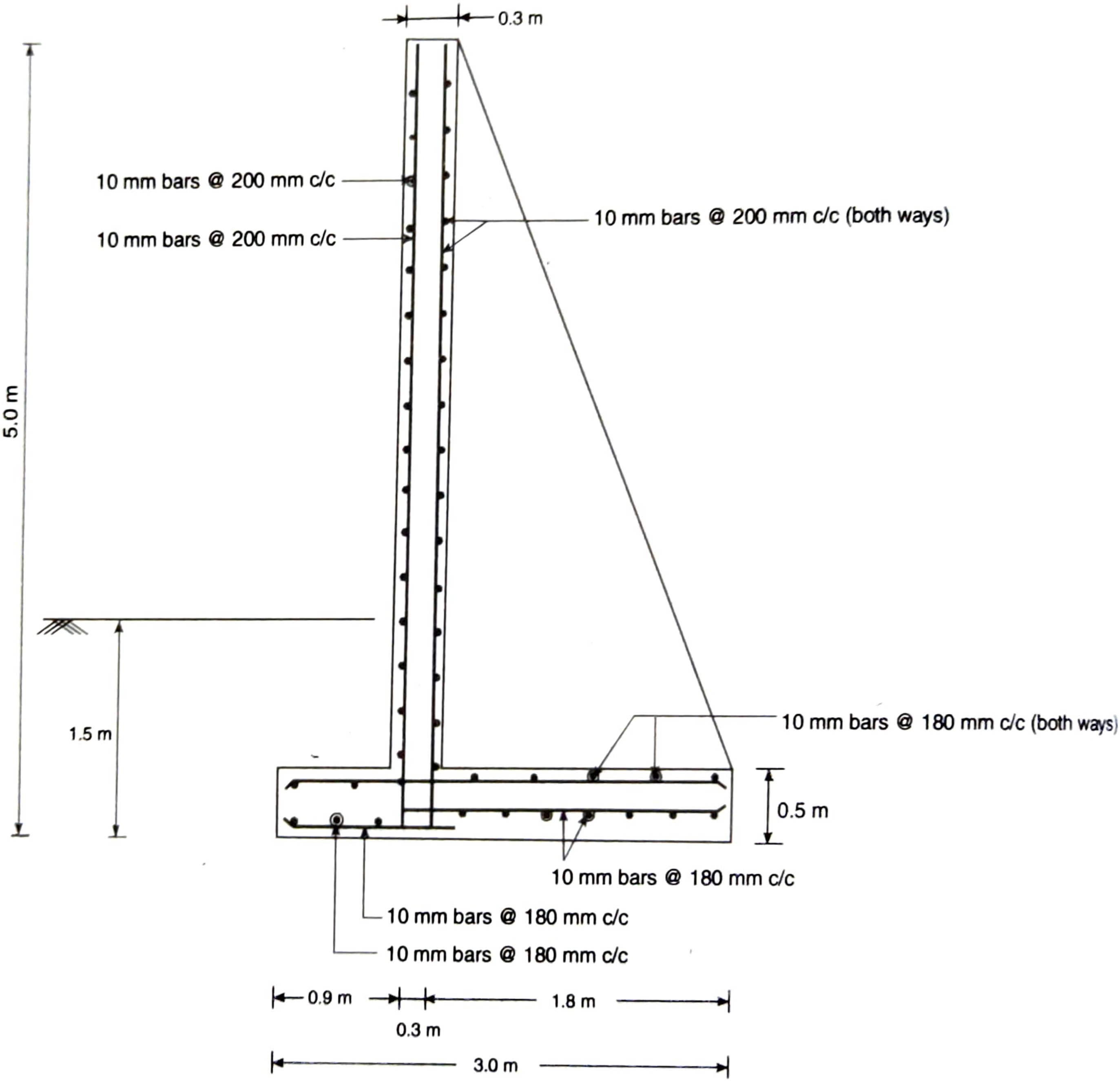
$$A_{st} = 293 \text{ mm}^2$$

Using 10 mm diameter bars,

$$S_{v\,reqd} = 267 \text{ mm}$$

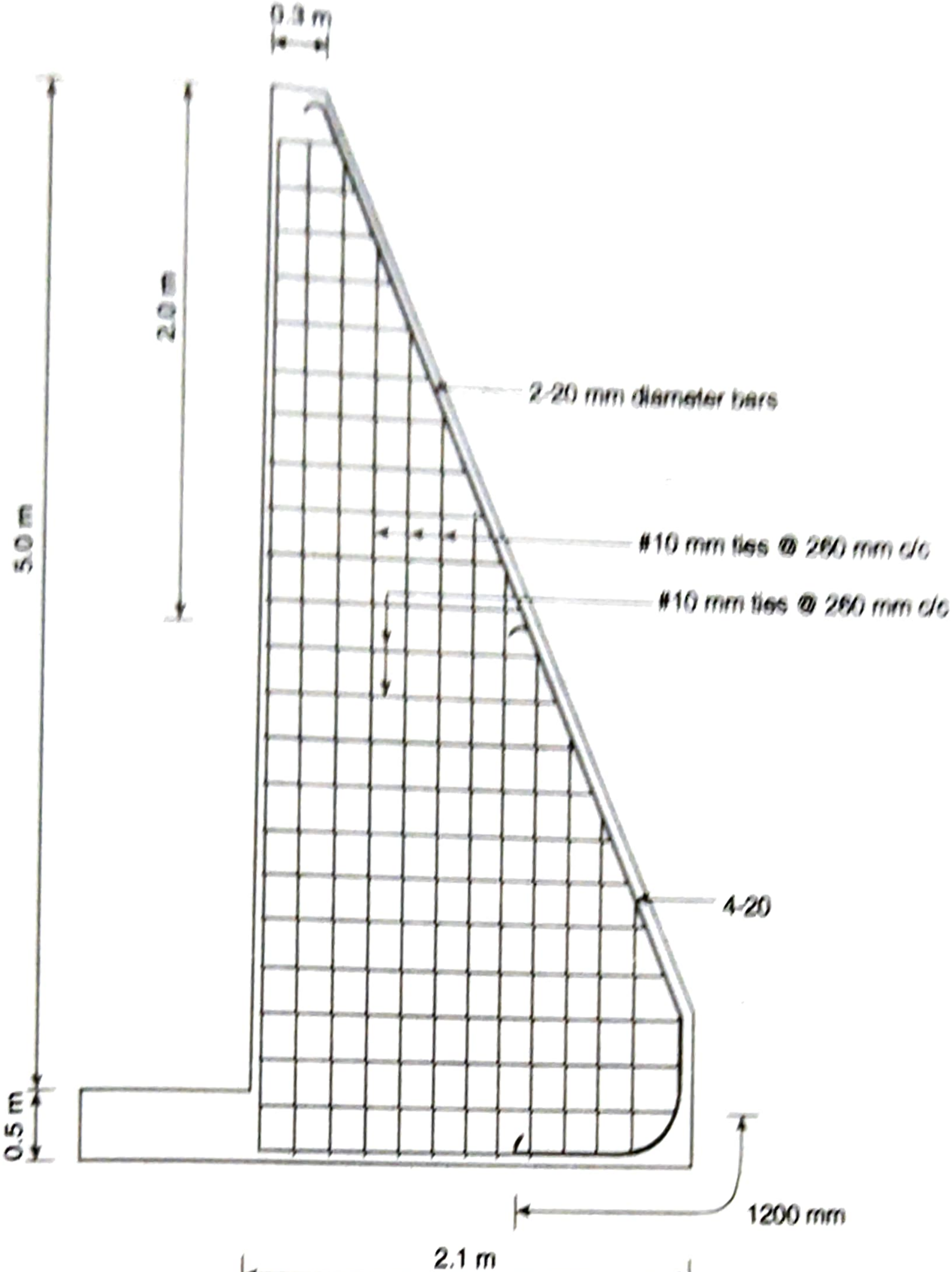
Hence provide 10 mm diameter ties @ 260 mm c/c.

The detailing of reinforcement of the counterfort retaining wall is shown in Fig. 16.22.

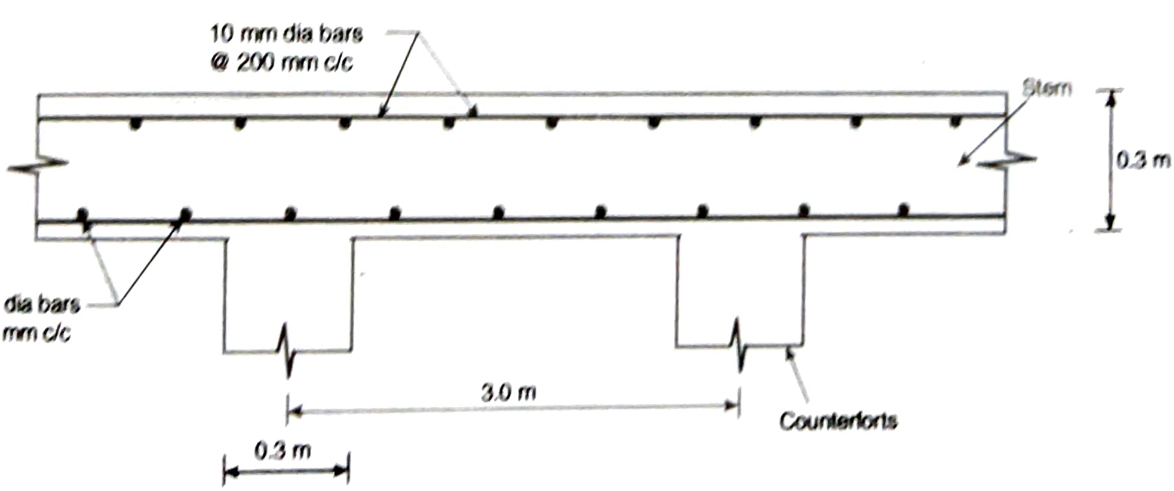


(*a*) Section at midspan of counterforts

Fig. 16.22

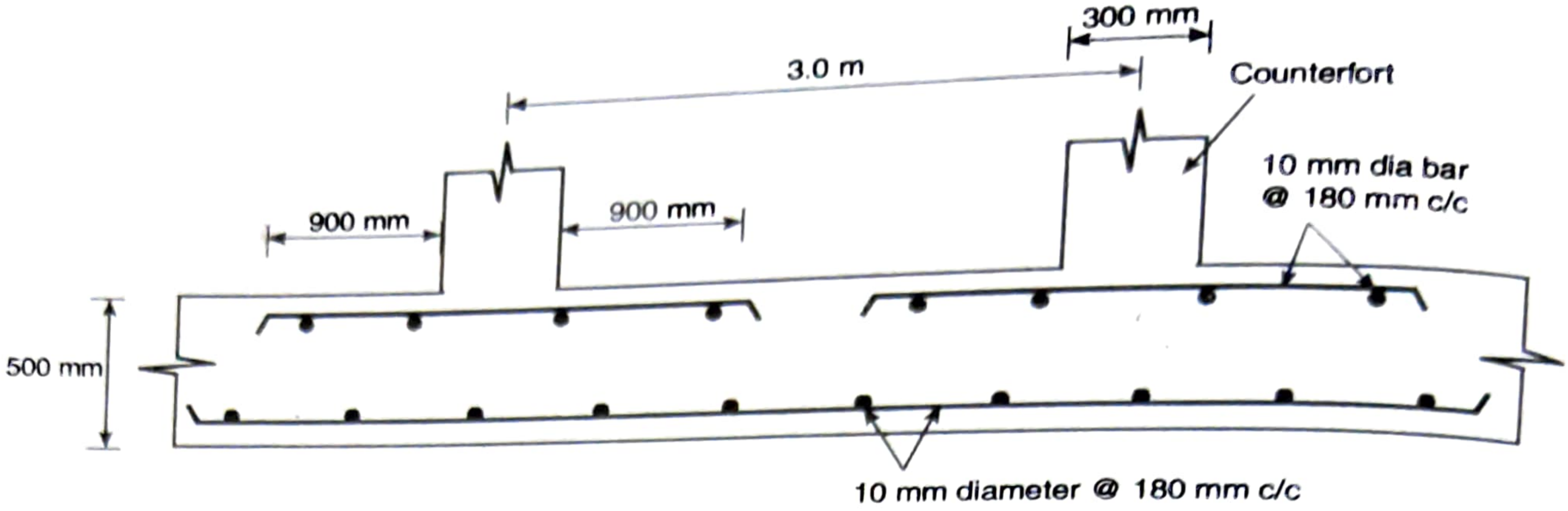


(b) Section at counterfort



(c) Details of Stress reinforcement at base

Fig. 16.22



(*d*) Details of heel slab (L-section)

Fig. 16.22