

# Retaining Structures

## 13-1 INTRODUCTION

Retaining structures are built for the purpose of retaining, or holding back, a soil mass (or other material). Probably a majority of retaining structures are concrete walls, which are covered in Sections 13-2 through 13-6. A relatively new type of retaining structure known as *Reinforced Earth* is presented in Section 13-7. Slurry trench walls, specially constructed concrete walls built entirely below ground level, are described in Section 13-8. Anchored bulkheads, covered in Section 13-9, are useful when certain waterfront retaining structures are needed.

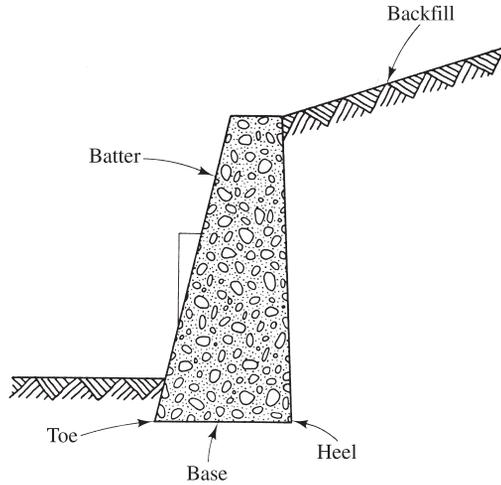
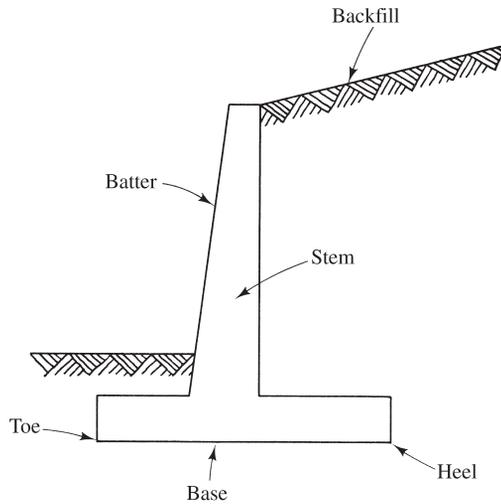
## 13-2 RETAINING WALLS

A simple retaining wall is illustrated in Figure 13-1. This type of wall depends on its weight to achieve stability; hence, it is called a *gravity wall*. In the case of taller walls, large lateral pressure tends to overturn the wall, and for economic reasons *cantilever walls* may be more desirable. As illustrated in Figure 13-2, a cantilever wall has part of its base extending underneath the backfill, and (as is shown subsequently) the weight of the soil above this part of the base helps prevent overturning.

Gravity walls are often built of plain concrete and are bulky. Concrete cantilever walls are generally more slender and must be adequately reinforced with steel. Although there are other types of retaining walls, these two are most common.

Although retaining walls may give the appearance of being unyielding, some wall movement is to be expected. In order that walls may undergo some forward yielding without appearing to tip over, they are often built with an inward slope on the outer face of the wall, as shown in Figures 13-1 and 13-2. This inward slope is called *batter*.

Material placed behind a retaining wall is commonly referred to as *backfill*. It is highly desirable that backfill be a select, free-draining, granular material, such as

**FIGURE 13-1** Gravity wall.**FIGURE 13-2** Cantilever wall.

clean sand, gravel, or broken stones. If necessary, appropriate material should be hauled in from an area outside the construction site. Clayey soils make extremely objectionable backfill material because of the excessive lateral pressure they create. The designer of a retaining wall should either (1) write specifications for the backfill and base the design of the wall on the specified backfill or (2) be given information on the material to be used as backfill and base the design of the wall on the indicated backfill. If it is possible that the water table will rise in the backfill, special designing, construction, and monitoring must go into effect.

In Chapter 12, several methods were presented for analyzing both the magnitude and the location of the lateral earth pressure acting on retaining walls. For

economic reasons, retaining walls are commonly designed for active earth pressure, developed by a free-draining, granular backfill acting on the wall. A retaining wall must (1) be able to resist sliding along the base, (2) be able to resist overturning, and (3) not introduce a contact pressure on the foundation soil beneath the wall's base that exceeds the allowable bearing pressure of the foundation soil. (Walls must also meet structural requirements, such as shear and bending moment; however, such considerations are not covered in this book.) Chapter 13 deals in more detail with retaining-wall design.

### 13-3 DESIGN CONSIDERATIONS FOR RETAINING WALLS

In designing retaining walls, the first step is to determine the magnitude and location of the active earth pressures that will be acting on the wall. These determinations can be made by utilizing any of the methods presented previously in Chapter 12. Active earth pressure is normally used to design free-standing retaining walls.

In practice, earth pressures for walls less than 20 ft (6 m) high are often obtained from graphs or tables. Almost all such graphs and tables are developed from Rankine theory. One graphic relationship is given in Figure 13-3. Use of this approach to obtain earth pressure should be self-explanatory.

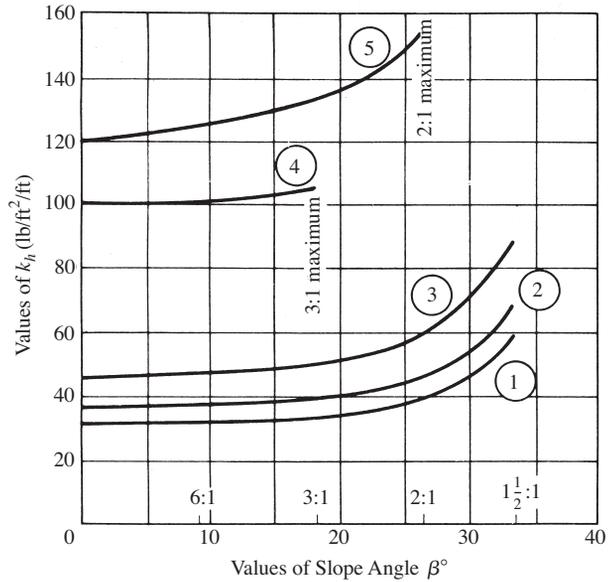
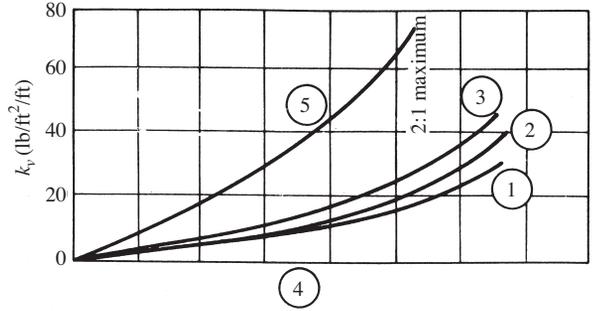
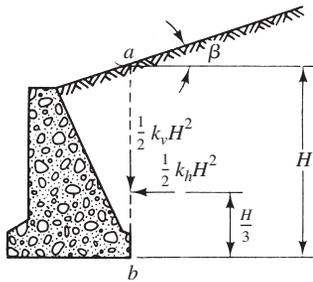
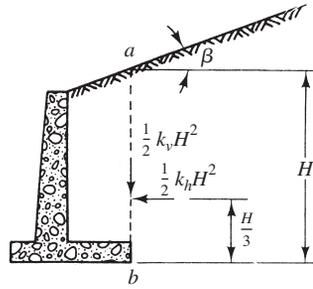
As can be noted by both the analytic methods of Chapter 12 and the graphic method of Figure 13-3, the magnitude of earth pressure on a retaining wall depends in part upon the type of soil backfill.

The next step in designing retaining walls is to assume a retaining-wall size. Normally, the required wall height will be known; thus, a wall thickness and base width must be estimated. The assumed wall is then checked for three conditions. First, the wall must be safe against sliding horizontally. Second, the wall must be safe against overturning. Third, the wall must not introduce a contact pressure on the foundation soil beneath the wall's base that exceeds the allowable bearing pressure of the foundation soil. If any of these conditions is not safe, the assumed wall size must be modified and conditions checked again. If, however, the three conditions are met, the assumed size is used for design. If (when) the three conditions are met with plenty to spare, the size might be reduced somewhat and conditions checked again. Obviously, this is more or less a trial-and-error procedure.

Another important design factor for retaining-wall design concerns the possibility of water permeating the soil behind a wall, in which case large additional pressures will be applied to the wall. Since this is undesirable, steps must usually be taken to prevent water that infiltrates the backfill soil from accumulating behind the wall. This topic is covered in more detail in Section 13-5.

### 13-4 STABILITY ANALYSIS

Common procedure in retaining-wall design is to assume a trial wall shape and size and then to check the trial wall for stability. If it does not prove to be stable by conventional standards, the wall's shape and/or size must be revised and the new wall checked for stability. This procedure is repeated until a satisfactory wall is found.



**FIGURE 13-3** Earth pressure charts for retaining walls less than 20 ft (6 m) high (1 lb/ft<sup>2</sup>/ft = 0.1571 kN/m<sup>2</sup>/m). Notes: Numerals on curves indicate soil types as described here. For material of Type 5, computations should be based on value of H 4 ft less than actual value. Types of backfill for retaining walls: ① Coarse-grained soil without admixture of fine soil particles, very free-draining (clean sand, gravel, or broken stone); ② coarse-grained soil of low permeability, owing to admixture of particles of silt size; ③ fine silty sand; granular materials with conspicuous clay content, or residual soil with stones; ④ soft or very soft clay, organic silt, or soft silty clay; ⑤ medium or stiff clay that may be placed in such a way that a negligible amount of water will enter the spaces between the chunks during floods or heavy rains.  
 Source: *Manual of Recommended Practice*, Construction and Maintenance Section, Engineering Division, Association of American Railroads, Chicago, 1958. Reprinted by permission.

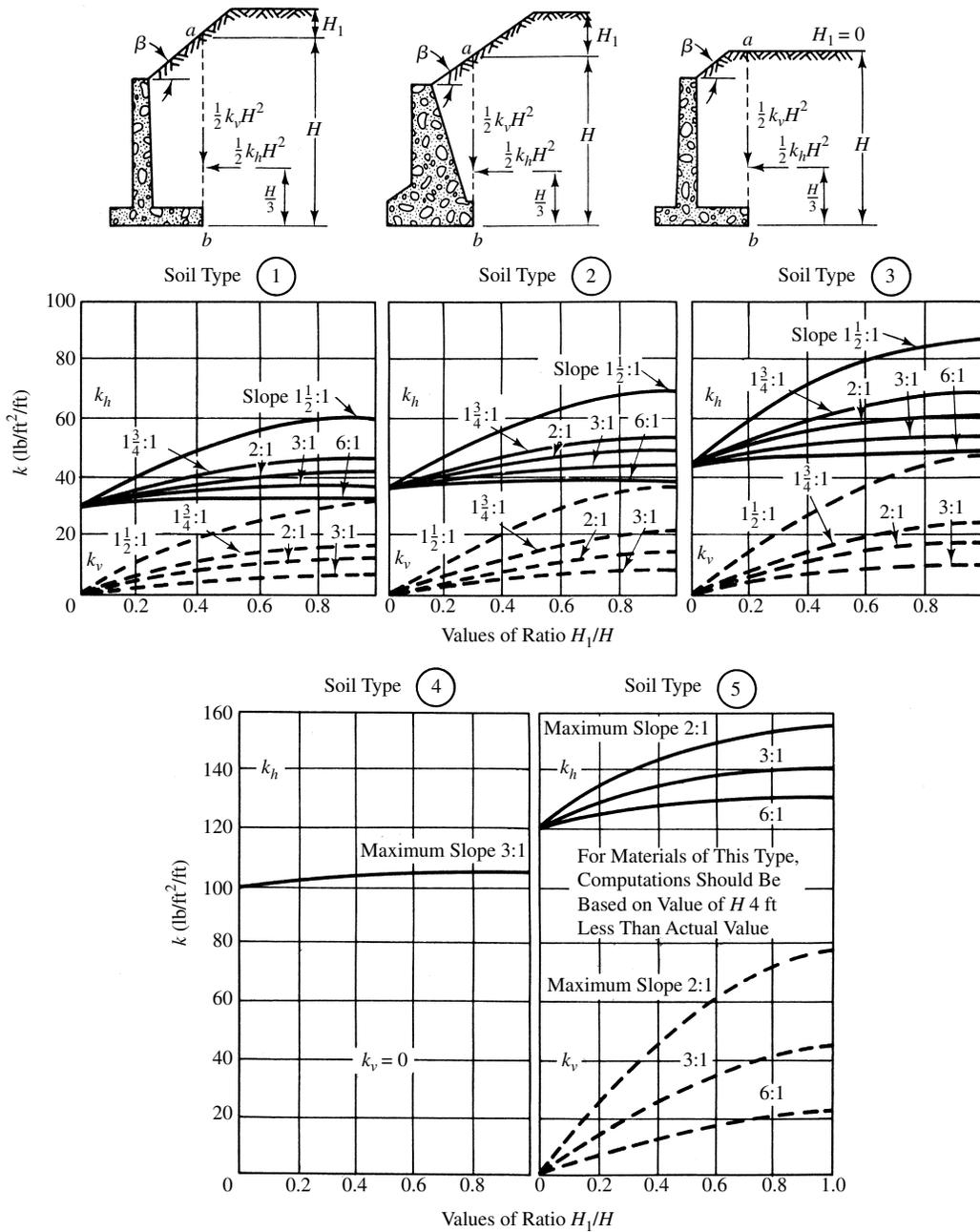


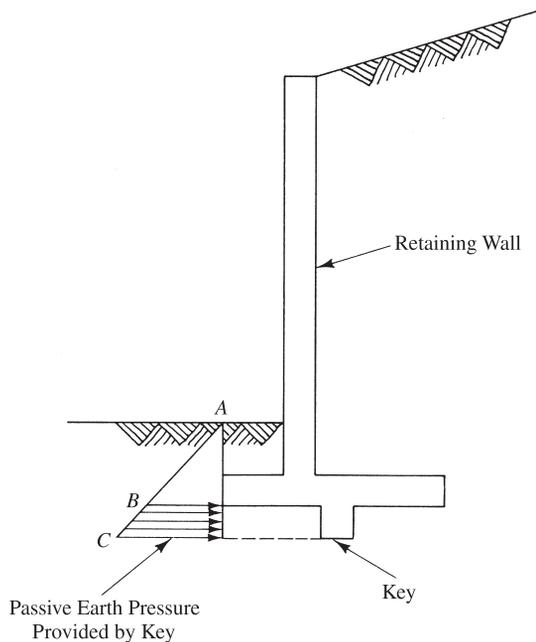
FIGURE 13-3 (continued)

If a wall is *stable*, it means, of course, that the wall does not move. Essentially, there are three means by which a retaining wall can move—horizontally (by sliding), vertically (by excessive settlement and/or bearing capacity failure of the foundation soil), and by rotation (by overturning). Standard procedure is to check for stability with respect to each of the three means of movement to ensure that an adequate factor of safety (F.S.) is present in each case. Checks for sliding and overturning hark back to the basic laws of statics. Checks for settlement and bearing capacity of foundation soil are done by settlement and bearing capacity analyses, which were presented in Chapters 7 and 9, respectively.

The factor of safety against sliding is found by dividing sliding resistance force by sliding force. The sliding resistance force is the product of the total downward force on the base of the wall and the coefficient of friction ( $\mu$ ) between the base of the retaining wall and the underlying soil. The sliding force is typically the horizontal component of lateral earth pressure exerted against the wall by backfill material.

If an adequate factor of safety against sliding is not obtained with an ordinary flat-bottomed wall, some additional sliding resistance may be achieved by constructing a “key” into the wall’s base. As shown in Figure 13–4, soil in front of the key’s vertical face provides additional resistance to sliding in the form of passive resistance (i.e., zone BC of the earth pressure diagram). Of course, soil in front of the wall and its base furnishes some passive resistance (zone AB of the earth pressure diagram of Figure 13–4); however, because this soil may be subsequently removed

**FIGURE 13–4** Sketch showing additional resistance to sliding in the form of passive resistance in front of key.



by erosion, this passive resistance is often ignored in retaining-wall design. Keys are most effective in hard soil or rock.

The factor of safety against overturning is determined by dividing total righting moment by total overturning moment. Because overturning tends to occur about the front base of a wall (at the toe), righting moments and overturning moments are computed about the wall's toe.

The factor of safety against bearing capacity failure is determined by dividing ultimate bearing capacity by actual maximum contact (base) pressure. Contact pressure is computed by the methods presented in Chapter 9.

In summary, the three factors of safety with regard to stability analysis are as follows:

$$(F.S.)_{\text{sliding}} = \frac{\text{Sliding resistance force}}{\text{Sliding force}} \quad (13-1)$$

$$(F.S.)_{\text{overturning}} = \frac{\text{Total righting moment about toe}}{\text{Total overturning moment about toe}} \quad (13-2)$$

$$(F.S.)_{\text{bearing capacity failure}} = \frac{\text{Soil's ultimate bearing capacity}}{\text{Actual maximum contact (base) pressure}} \quad (13-3)$$

Some common minimum factors of safety for sufficient stability are as follows:

$$(F.S.)_{\text{sliding}} = 1.5 \text{ (if the passive earth pressure of the soil at the toe in front of the wall is neglected)} \\ \text{(Goodman and Karol, 1968)*} \\ = 2.0 \text{ (if the passive earth pressure of the soil at the toe in front of the wall is included)} \\ \text{(Goodman and Karol, 1968)*}$$

$$(F.S.)_{\text{overturning}} = 1.5 \text{ (granular backfill soil)} \\ = 2.0 \text{ (cohesive backfill soil)(Teng, 1962)}$$

$$(F.S.)_{\text{bearing capacity failure}} = 3.0$$

The two examples that follow illustrate the investigation of stability analysis for retaining walls. Example 13-1 refers to a gravity wall and Example 13-2 to a cantilever wall.

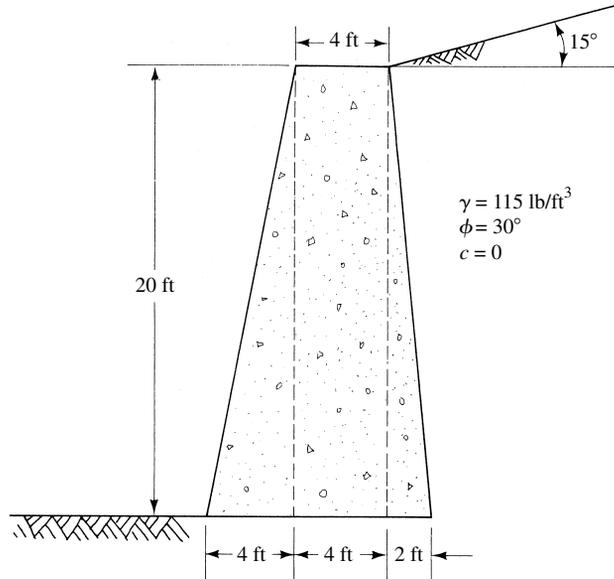
### EXAMPLE 13-1

*Given*

1. The retaining wall shown in Figure 13-5 is to be constructed of concrete having a unit weight of 150 lb/ft<sup>3</sup>.

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FIGURE 13-5



2. The retaining wall is to support a deposit of granular soil that has the following properties:

$$\begin{aligned}\gamma &= 115 \text{ lb/ft}^3 \\ \phi &= 30^\circ \\ c &= 0\end{aligned}$$

3. The coefficient of base friction is 0.55.  
4. The foundation soil's ultimate bearing capacity is 6.5 tons/ft<sup>2</sup>.

*Required*

Check the stability of the proposed retaining wall; that is, check the factor of safety against

1. Sliding.
2. Overturning.
3. Bearing capacity failure.

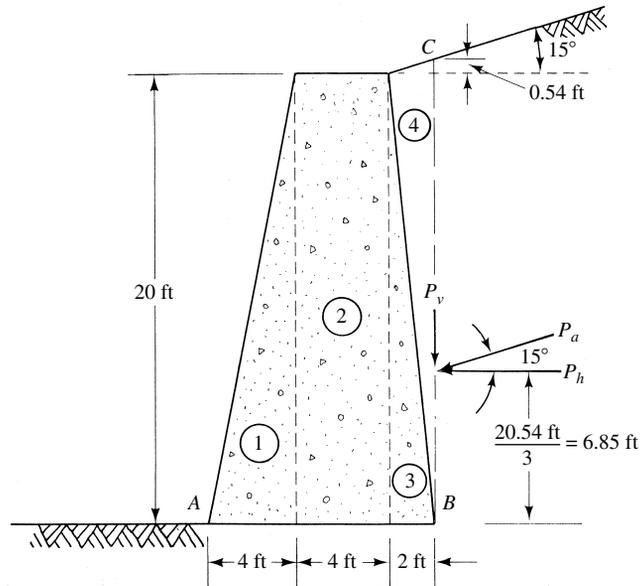
**Solution**

*Calculation of Active Earth Pressure on the Back of the Wall by Rankine Theory*

From Eqs. (12-10) and (12-11),

$$P_a = \frac{1}{2} \gamma H^2 \cos \beta \frac{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \phi}}{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi}}$$

FIGURE 13-6



Referring to Figure 13-6, one finds that

$$H = \overline{BC} = 20 \text{ ft} + (2 \text{ ft})(\tan 15^\circ) = 20.54 \text{ ft}$$

$$P_a = \left(\frac{1}{2}\right)(0.115 \text{ kip/ft}^3)(20.54 \text{ ft})^2(\cos 15^\circ) \frac{\cos 15^\circ - \sqrt{\cos^2 15^\circ - \cos^2 30^\circ}}{\cos 15^\circ + \sqrt{\cos^2 15^\circ - \cos^2 30^\circ}}$$

$$P_a = 9.05 \text{ kips/ft}$$

$P_a$  acts parallel to the surface of the backfill; therefore,

$$\begin{aligned} \text{Horizontal component } (P_h) &= P_a \cos 15^\circ = (9.05 \text{ kips/ft}) \cos 15^\circ \\ &= 8.74 \text{ kips/ft} \end{aligned}$$

$$\text{Vertical component } (P_v) = P_a \sin 15^\circ = (9.05 \text{ kips/ft}) \sin 15^\circ = 2.34 \text{ kips/ft}$$

Calculation of Righting Moment (See Figure 13-6)

Component	Weight of Component (kips/ft)	Moment Arm from A (ft)	Righting Moment about A (ft-kips/ft)
1	$(0.15)(\frac{1}{2})(4)(20) = 6.00$	$(\frac{2}{3})(4) = \frac{8}{3}$	16.0
2	$(0.15)(4)(20) = 12.00$	$4 + \frac{1}{2} = 6$	72.0
3	$(0.15)(\frac{1}{2})(2)(20) = 3.00$	$4 + 4 + (\frac{1}{3})(2) = \frac{26}{3}$	26.0
4	$(0.115)(\frac{1}{2})(20.54)(2) = 2.36$	$4 + 4 + (\frac{2}{3})(2) = \frac{28}{3}$	22.0
$P_v$	<u>2.34</u>	$4 + 4 + 2 = 10$	<u>23.4</u>
	$\Sigma V = 25.70$		$\Sigma M_r = 159.4$

*Calculation of Overturning Moment*

Overturning moment ( $M_0$ ) = (8.74 kips/ft)(6.85 ft) = 59.9 ft-kips/ft

1. From Eq. (13-1),

$$(F.S.)_{\text{sliding}} = \frac{\text{Sliding resistance force}}{\text{Sliding force}} \quad (13-1)$$

$$\begin{aligned} (F.S.)_{\text{sliding}} &= \frac{(\mu)(\Sigma V)}{P_h} = \frac{(0.55)(25.70 \text{ kips/ft})}{8.74 \text{ kips/ft}} \\ &= 1.62 > 1.5 \quad \therefore \text{O.K.} \end{aligned}$$

2. From Eq. (13-2),

$$(F.S.)_{\text{overturning}} = \frac{\text{Total righting moment about toe}}{\text{Total overturning moment about toe}} \quad (13-2)$$

$$\begin{aligned} (F.S.)_{\text{overturning}} &= \frac{\Sigma M_r}{\Sigma M_0} = \frac{159.4 \text{ ft-kips/ft}}{59.9 \text{ ft-kips/ft}} \\ &= 2.66 > 1.5 \text{ (for granular backfill)} \quad \therefore \text{O.K.} \end{aligned}$$

*Base Pressure Calculations*

Location of resultant  $R$  ( $= \Sigma V$ ) if  $R$  acts at  $\bar{x}$  from the toe (point A):

$$\begin{aligned} \bar{x} &= \frac{\Sigma M_A}{\Sigma V} = \frac{\Sigma M_r - \Sigma M_0}{\Sigma V} = \frac{159.4 \text{ ft-kips/ft} - 59.9 \text{ ft-kips/ft}}{25.70 \text{ kips/ft}} = 3.87 \text{ ft} \\ e &= \frac{4 \text{ ft} + 4 \text{ ft} + 2 \text{ ft}}{2} - 3.87 \text{ ft} = 1.13 \text{ ft} < \frac{L}{6} \text{ (i.e., } \frac{10}{6} \text{ ft, or } 1.67 \text{ ft)} \quad \therefore \text{O.K.} \end{aligned}$$

(i.e.,  $R$  acts within the middle third of the base)

Using the flexural formula, from Eq. (9-11) (see Chapter 9), one gets

$$q = \frac{Q}{A} \pm \frac{M_x y}{I_x} \pm \frac{M_y x}{I_y} \quad (9-11)$$

Here,

$$Q = \text{Resultant } (R) = \Sigma V = 25.70 \text{ kips}$$

$$A = (1 \text{ ft})(10 \text{ ft}) = 10 \text{ ft}^2$$

$$M_x = 0 \quad (\text{one-way bending})$$

$$M_y = Q \times e = (25.70 \text{ kips})(1.13 \text{ ft}) = 29.04 \text{ ft-kips}$$

$$x = \frac{10 \text{ ft}}{2} = 5 \text{ ft}$$

$$I_y = \frac{bh^3}{12} = \frac{(1 \text{ ft})(10 \text{ ft})^3}{12} = 83.33 \text{ ft}^4$$

$$q = \frac{25.70 \text{ kips}}{10 \text{ ft}^2} \pm \frac{(29.04 \text{ ft-kips})(5 \text{ ft})}{83.33 \text{ ft}^4}$$

$$q_L = 2.57 \text{ kips/ft}^2 + 1.74 \text{ kips/ft}^2 = 4.31 \text{ kips/ft}^2 = 2.16 \text{ tons/ft}^2$$

$$q_R = 2.57 \text{ kips/ft}^2 - 1.74 \text{ kips/ft}^2 = 0.83 \text{ kip/ft}^2 = 0.42 \text{ ton/ft}^2$$

The pressure distribution is shown in Figure 13-7.

3. From Eq. (13-3),

$$(\text{F.S.})_{\text{bearing capacity failure}} = \frac{\text{Soil's ultimate bearing capacity}}{\text{Actual maximum contact (base) pressure}} \quad (13-3)$$

$$(\text{F.S.})_{\text{bearing capacity failure}} = \frac{6.5 \text{ tons/ft}^2}{2.16 \text{ tons/ft}^2} = 3.01 > 3 \quad \therefore \text{O.K.}$$

### EXAMPLE 13-2

*Given*

1. The retaining wall shown in Figure 13-8.
2. The backfill material is Type 1 soil (Figure 13-3).
3. The unit weight and  $\phi$  angle of the backfill material are  $120 \text{ lb/ft}^3$  and  $37^\circ$ , respectively.
4. The coefficient of base friction is 0.45.
5. Allowable soil pressure is  $3 \text{ kips/ft}^2$ .
6. The unit weight of the concrete is  $150 \text{ lb/ft}^3$ .

*Required*

1. The factor of safety against sliding. Analyze both without and with passive earth pressure at the toe.
2. The factor of safety against overturning.
3. The factor of safety against failure of the foundation soil.

FIGURE 13-7

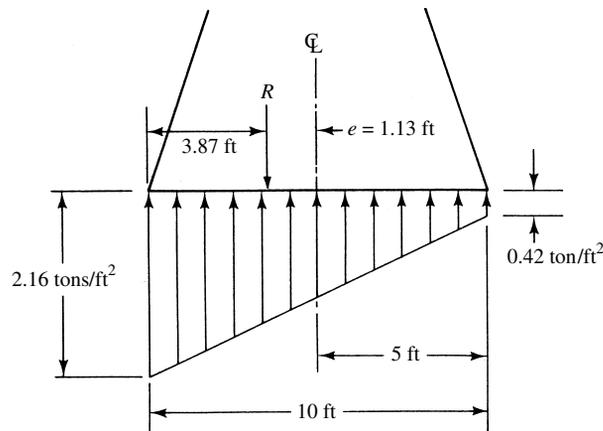
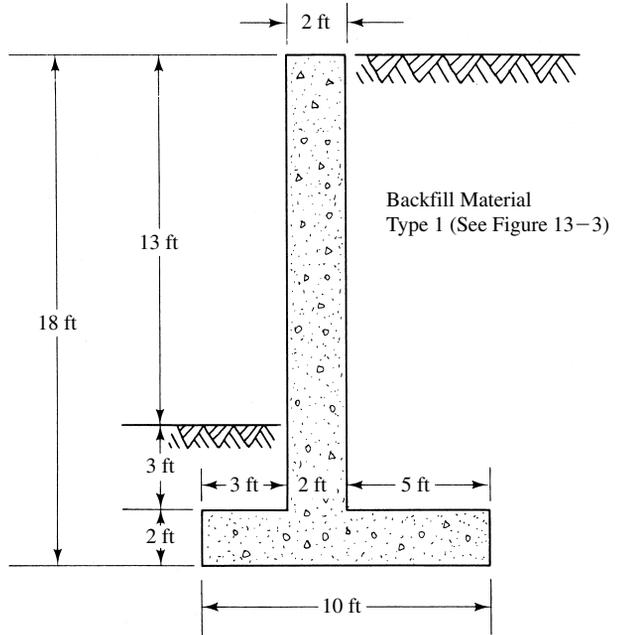


FIGURE 13-8



**Solution**

Calculation of the Active Earth Pressure by Figure 13-3

From Figure 13-3,

$$P_h = \frac{1}{2}k_h H^2 \text{ with } \beta = 0^\circ \text{ and Type 1 backfill material, } k_h = 30 \text{ lb/ft}^2/\text{ft}$$

$$P_h = (\frac{1}{2})(30 \text{ lb/ft}^2/\text{ft})(18 \text{ ft})^2 = 4860 \text{ lb/ft} = 4.86 \text{ kips/ft}$$

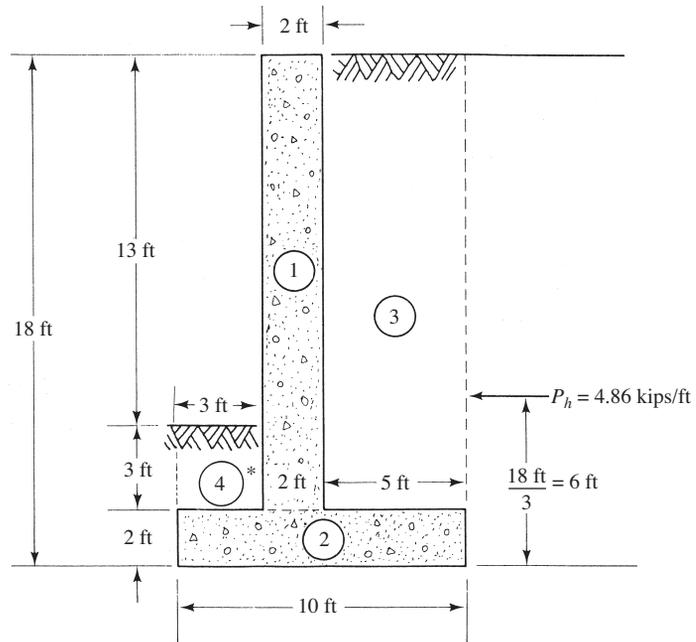
$$P_v = \frac{1}{2}k_v H^2 \text{ with } \beta = 0^\circ, k_v = 0$$

$$P_v = 0$$

Calculation of Righting Moment (See Figure 13-9)

Component	Weight of Component (kips/ft)	Moment Arm from Toe (ft)	Righting Moment about Toe (ft-kips/ft)
1	$(0.15)(2)(13 + 3) = 4.8$	$3 + \frac{1}{2} = 4.0$	19.2
2	$(0.15)(2)(10) = 3.0$	$\frac{10}{2} = 5.0$	15.0
3	$(0.12)(5)(13 + 3) = 9.6$	$3 + 2 + \frac{1}{2} = 7.5$	72.0
4	$(0.12)(3)(3) = 1.1$	$\frac{3}{2} = 1.5$	1.6
	$\Sigma V = 18.5$		$\Sigma M_r = 107.8$

FIGURE 13-9



•Assume the soil above the toe is also Type 1 backfill material.

Calculation of Overturning Moment (See Figure 13-9)

Overturning moment ( $M_o$ ) = (4.86 kips/ft)(6 ft) = 29.16 ft-kips/ft

1. Factor of safety against sliding:

a. Without passive earth pressure analysis (neglect passive earth pressure at the toe): From Eq. (13-1),

$$(\text{F.S.})_{\text{sliding}} = \frac{\text{Sliding resistance force}}{\text{Sliding force}} \quad (13-1)$$

$$(\text{F.S.})_{\text{sliding}} = \frac{(\mu)(\Sigma V)}{P_h} = \frac{(0.45)(18.5 \text{ kips/ft})}{4.86 \text{ kips/ft}} = 1.71 > 1.5 \quad \therefore \text{O.K.}$$

b. With passive earth pressure at the toe:

Sliding resistance = Passive earth pressure at toe + Friction available along base

According to Rankine theory for level backfill, from Eqs. (12-12) and (12-15),

$$P_p = \frac{1}{2} \gamma H^2 \frac{1 + \sin \phi}{1 - \sin \phi}$$

$$P_p = \left(\frac{1}{2}\right)(0.12 \text{ kip/ft}^3)(5 \text{ ft})^2 \left(\frac{1 + \sin 37^\circ}{1 - \sin 37^\circ}\right) = 6.03 \text{ kips/ft}$$

$$\begin{aligned}
 (\text{F.S.})_{\text{sliding}} &= \frac{(\mu)(\Sigma V) + P_p}{P_h} \\
 (\text{F.S.})_{\text{sliding}} &= \frac{(0.45)(18.5 \text{ kips/ft}) + 6.03 \text{ kips/ft}}{4.86 \text{ kips/ft}} = 2.95 > 2.0 \quad \therefore \text{O.K.}
 \end{aligned}$$

2. *Factor of safety against overturning:* From Eq. (13-2),

$$(\text{F.S.})_{\text{overturning}} = \frac{\text{Total righting moment}}{\text{Total overturning moment}} \quad (13-2)$$

$$(\text{F.S.})_{\text{overturning}} = \frac{\Sigma M_r}{\Sigma M_o} = \frac{107.8 \text{ ft-kips/ft}}{29.16 \text{ ft-kips/ft}} = 3.70 > 1.5 \quad \therefore \text{O.K.}$$

#### Base Pressure Calculations

Location of resultant  $R (= \Sigma V)$  if  $R$  acts at  $\bar{x}$  from the toe:

$$\bar{x} = \frac{\Sigma M_{\text{toe}}}{\Sigma V} = \frac{\Sigma M_r - \Sigma M_o}{\Sigma V} = \frac{107.8 \text{ ft-kips/ft} - 29.16 \text{ ft-kips/ft}}{18.5 \text{ kips/ft}} = 4.25 \text{ ft}$$

$$e = \frac{10 \text{ ft}}{2} - 4.25 \text{ ft} = 0.75 \text{ ft} < \frac{L}{6} \text{ (i.e., } \frac{10}{6} \text{ ft, or } 1.67 \text{ ft)} \quad \therefore \text{O.K.}$$

Using the flexural formula, from Eq. (9-11), one gets

$$q = \frac{Q}{A} \pm \frac{M_x y}{I_x} \pm \frac{M_y x}{I_y} \quad (9-11)$$

Here,

$$Q = \text{Resultant } (R) = \Sigma V = 18.5 \text{ kips}$$

$$A = (1 \text{ ft})(10 \text{ ft}) = 10 \text{ ft}^2$$

$$M_x = 0$$

$$M_y = Q \times e = (18.5 \text{ kips})(0.75 \text{ ft}) = 13.9 \text{ ft-kips}$$

$$x = \frac{10 \text{ ft}}{2} = 5 \text{ ft}$$

$$I_y = \frac{bh^3}{12} = \frac{(1 \text{ ft})(10 \text{ ft})^3}{12} = 83.33 \text{ ft}^4$$

$$q = \frac{18.5 \text{ kips}}{10 \text{ ft}^2} \pm \frac{(13.9 \text{ ft-kips})(5 \text{ ft})}{83.33 \text{ ft}^4}$$

$$q_L = 1.85 \text{ kips/ft}^2 + 0.83 \text{ kip/ft}^2 = 2.68 \text{ kips/ft}^2$$

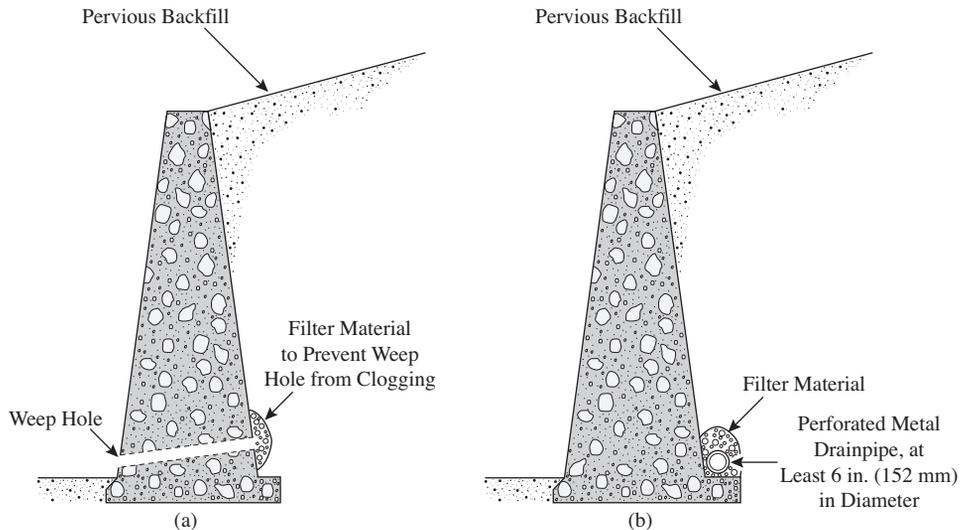
$$q_R = 1.85 \text{ kips/ft}^2 - 0.83 \text{ kip/ft}^2 = 1.02 \text{ kips/ft}^2$$

3. Because  $q_L$  is 2.68 kips/ft<sup>2</sup>, which is less than the allowable soil pressure of 3.0 kips/ft<sup>2</sup> (given), the wall is safe against failure of the foundation soil.

### 13-5 BACKFILL DRAINAGE

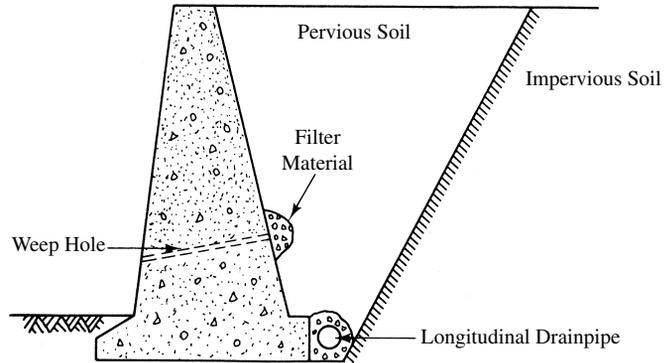
If water is allowed to permeate the soil behind a retaining wall, large additional pressure will be applied to the wall. Unless the wall is designed to withstand this large additional pressure (not the usual practice), it is imperative that steps be taken to prevent water that infiltrates the backfill soil from accumulating behind the wall.

One method of preventing water from accumulating behind a wall is to provide an effective means of draining away any water that enters the backfill soil. To accomplish this, it is highly desirable to use as backfill material a highly pervious soil such as sand, gravel, or crushed stone. To remove water from behind the wall, one can place 4- to 6-in. (102- to 152-mm) weep holes, which are pipes extending through the wall (see Figure 13-10a), every 5 to 10 ft (1.5 to 3 m) along the wall. A perforated drainpipe placed longitudinally along the back of the wall (Figure 13-10b) may also be used to remove water from behind the wall. In this case, the pipe is surrounded by filter material, and water drains through the filter material into the pipe and then through the pipe to one end of the wall. In both cases (weep holes and drainpipes), a filter material must be placed adjacent to the pipe to prevent clogging, and the pipes must be kept clear of debris.



**FIGURE 13-10** (a) Weep hole; (b) perforated drainpipe.

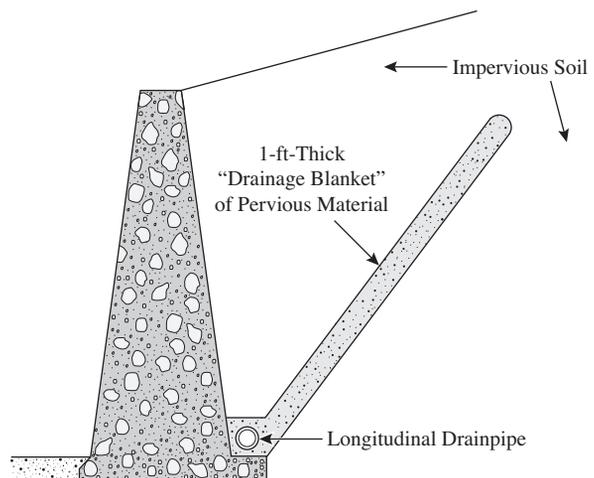
**FIGURE 13-11** Sketch showing wedge of pervious material adjacent to wall (Hough, 1969).



If a less pervious material (silt, granular soil containing clay, etc.) has to be used as backfill because a free-draining, granular material is too expensive in the locality, it is highly desirable to place a wedge of pervious material adjacent to the wall, as shown in Figure 13-11. If this is not possible, a “drainage blanket” of pervious material may be placed as shown in Figure 13-12.

A highly impervious soil (clay) is very undesirable as backfill material because, in addition to the excessive lateral earth pressure it creates, it is difficult to drain and may be subject to frost action. Also, clays are subject to swelling and shrinking. If clayey soil must be used as backfill material, it is advisable to place a wedge of pervious material adjacent to the wall between the wall and clay backfill (as shown in Figure 13-11) (Peck et al., 1974).

**FIGURE 13-12** Sketch showing drainage blanket with longitudinal drain.



## 13–6 SETTLEMENT AND TILTING

A certain amount of settlement by retaining walls is to be expected, just as by any other structures resting on footings or piles. In the case of retaining walls on granular soils, most of the expected settlement will have occurred by the time construction of the wall and placement of backfill have been completed. With retaining walls on cohesive soils, for which consolidation theory is applicable, settlement will occur slowly and for some period of time after construction has been completed.

The amount of settlement for retaining walls resting on spread footings can be determined by using the principles of settlement analysis for footings (see Chapter 7). For walls resting on piles, the amount of settlement can be estimated by using the principles of settlement analysis for pile foundations (see Chapter 10). To keep settlement relatively uniform, one must ensure that the resultant force is kept near the middle of the base.

If the soil upon which a retaining wall rests is not uniform in bearing capacity along the length of the wall, differential settlement may occur along the wall, which could cause the wall to crack vertically. If soil of poor bearing capacity occurs for only a very short distance, differential settlement may not be a problem, as the wall tends to bridge across poor material. If, however, poor bearing capacity of the soil exists for a considerable distance along the length of the wall, differential settlement will likely happen unless the designer takes this into account and implements remedies to correct the situation. Possible remedies include improving the soil (e.g., by replacement, compaction, or stabilization of the soil) and changing the footing's width. If computed settlement is excessive, pile foundations may be used.

In addition to settlement, a retaining wall is subject to tilting caused by eccentric pressure on the base of the wall. Tilting can be reduced by keeping the resultant force near the middle of the base. In many cases, walls tilt forward because the resultant force intersects the base at a point between the center and toe.

It is difficult to determine the amount of tilting to be expected, and rough estimates must suffice. If stability requirements are met in accordance with established design procedures (see Section 13–4), the amount of tilting may be expected to be in the order of magnitude of one-tenth of 1% of the height of the wall or less. However, if the subsoil consists of a compressive layer, this amount may be exceeded (Teng, 1962).

## 13–7 REINFORCED EARTH\* WALLS

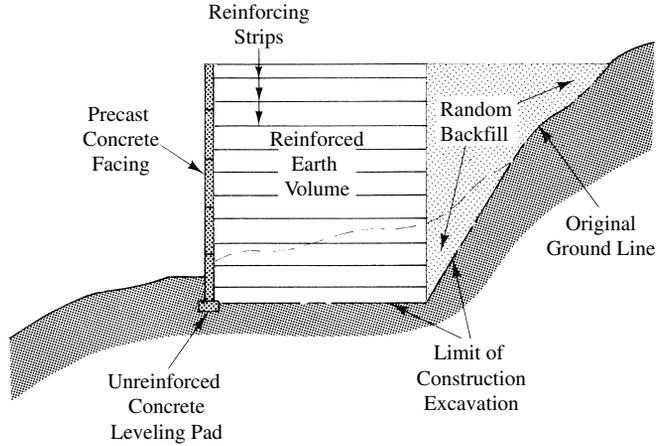
One alternative to the conventional retaining wall for holding soil embankments is known as a *Reinforced Earth wall*, a patented method first developed in France by Vidal. It is particularly useful where high or otherwise-difficult-to-construct retaining structures are needed.

A typical section of a Reinforced Earth structure is shown in Figure 13–13. The wall consists of precast concrete facings resting on an unreinforced concrete leveling

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\*Reinforced Earth is a registered trademark of The Reinforced Earth Company, Arlington, VA.

**FIGURE 13-13** Typical section of a Reinforced Earth retaining wall.  
 Source: Courtesy of The Reinforced Earth Company.



pad. However, the “wall” does not have to be particularly strong and can therefore be constructed of thinner materials. It is sometimes referred to as the *skin*. The Reinforced Earth volume is cohesionless soil, spread and compacted in layers. Reinforcing strips, commonly made of ribbed, galvanized steel, are placed atop each layer and bolted to the skin material (i.e., wall element).

Figure 13-14 gives another sketch of a Reinforced Earth wall, which depicts length, width, thickness, and horizontal and vertical spacings ( $L$ ,  $w$ ,  $t$ ,  $s$ , and  $h$ , respectively) of the reinforcing strips. Design considerations require that (1) the skin (wall element) resist soil pressure from adjacent soil layers, (2) strip length be long enough to support the skin, and (3) the strip be strong enough to resist its internal tension. Typical strip spacings are about 1 ft (0.3 m) vertically and 2 ft (0.6 m) horizontally.

To evaluate the criteria for Reinforced Earth design, consider a strip at depth  $z$  below the wall top. Here, the force against the area of wall that must be supported by a strip can be determined by the following equation (Lee et al., 1973):

$$T = \gamma z K_a s h \quad (13-4)$$

where  $T$  = tensile force per strip  
 $\gamma$  = unit weight of backfill soil  
 $z$  = depth from wall top to strip  
 $K_a$  = coefficient of active earth pressure (Rankine)  
 $s$  = horizontal spacing between strips  
 $h$  = vertical spacing between strips

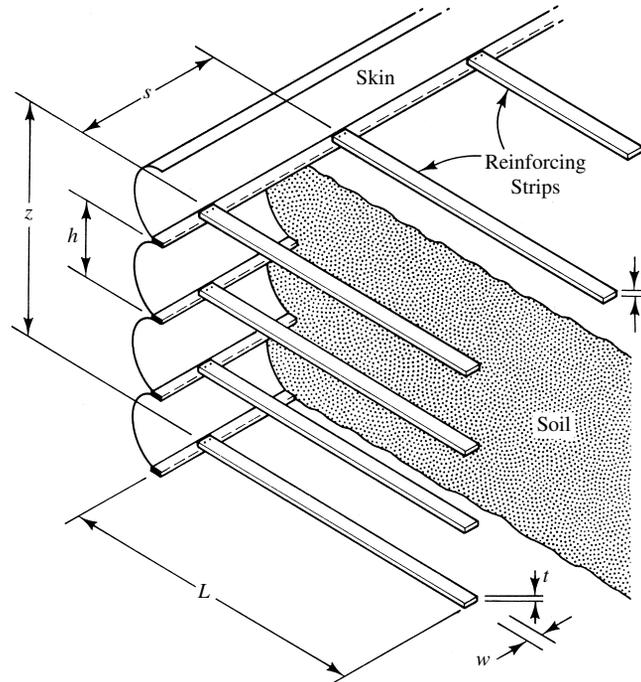
The frictional resistance of a strip at depth  $z$  ( $F$ ) developed between the strip's top and bottom faces and the backfill soil is as follows (Lee et al., 1973):

$$F = (\gamma z \tan \delta)(2Lw) \quad (13-5)$$

**FIGURE 13-14** Component parts and key dimensions of a Reinforced Earth wall.

Thickness =  $t$ .

Source: K. L. Lee, B. D. Adams, and J.-M. J. Vagneron, "Reinforced Earth Retaining Walls," *J. Soil Mech. Found. Eng. Div., Proc. ASCE*, 99 (SM10) 745-764 (1973).



where  $\delta$  is the angle of friction between the strip's surfaces and the backfill soil and other terms are as previously defined.  $\tan \delta$  can be taken as  $\tan(\phi/2)$ , where  $\phi$  is the soil's angle of internal friction.

The required minimum length of the strip ( $L_{\min}$ ) can be evaluated by equating  $T$  [Eq. (13-4)] and  $F$  [Eq. (13-5)] and including an appropriate factor of safety against pullout (F.S., normally 1.5 to 2.0). Hence,

$$(\text{F.S.})(\gamma z K_a s h) = (\gamma z \tan \delta)(2Lw) \quad (13-6)$$

Solving for  $L_{\min}$  (i.e.,  $L$ ) yields the following:

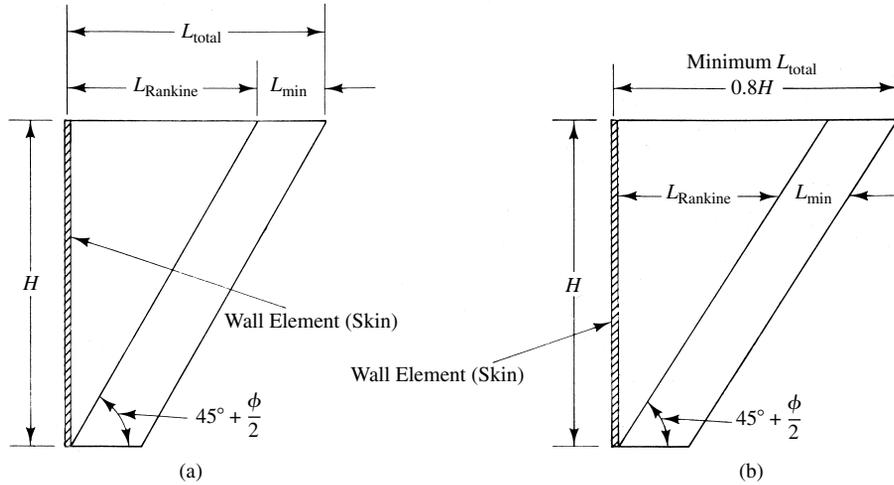
$$L_{\min} = \frac{(\text{F.S.})(K_a s h)}{2w \tan \delta} \quad (13-7)$$

$L_{\min}$  is measured beyond the zone of Rankine failure, as shown in Figure 13-15a. That is,

$$L_{\text{total}} = L_{\text{Rankine}} + L_{\min} \quad (13-8)$$

where

$$L_{\text{Rankine}} = H \tan\left(45^\circ - \frac{\phi}{2}\right) \quad (13-9)$$



**FIGURE 13-15** Minimum length of strip.

For overall stability, a minimum length ( $L_{total}$ ) of 80% of the wall height,  $H$ , is suggested (See Figure 13-15b). That is,

$$(L_{total})_{\text{minimum}} = 0.80H \quad (13-10)$$

Strip thickness can be determined from the basic stress equation

$$t = \frac{T}{wf_s} \quad (13-11)$$

where  $t$  = strip thickness

$T$  = tensile force (per strip) [from Eq. (13-4)]

$w$  = strip width

$f_s$  = allowable stress for strip material

It may be noted from Eq. (13-4) that the tensile force per strip is greatest at the bottom of the wall (i.e., where  $z$  is greatest). At lesser values of  $z$ , the tensile force is less, but the friction on each strip is also reduced. Accordingly, the total strip area should be constant at all depths to provide the same resistance to strip pullout. Also, from Eq. (13-7) it is clear that the minimum length of the strip is independent of depth. For these reasons as well as simplicity in construction, usually the same size, length (Figure 13-13), and spacing of strips are used throughout a Reinforced Earth structure.

It should be emphasized that Reinforced Earth structures must use cohesionless soils as backfill material because of their needed high friction.

### EXAMPLE 13-3

*Given*

A 6-m-high Reinforced Earth wall is to be constructed with level backfill (Figure 13-14) and will have no surcharge on the backfill. A granular soil with a unit weight

of  $17.12 \text{ kN/m}^3$  and an angle of internal friction of  $34^\circ$  will be used for backfill material. The steel strips' width, vertical spacing, horizontal spacing, and allowable stress are 75 mm, 0.3 m, 1.0 m, and  $138,000 \text{ kN/m}^2$ , respectively. The factor of safety against pullout is to be 1.5.

*Required*

1. Total length of strip required.
2. Thickness of strip required.

**Solution**

1. From Eq. (13-7),

$$L_{\min} = \frac{(\text{F.S.})(K_a sh)}{2w \tan \delta} \quad (13-7)$$

$$\text{F.S.} = 1.5 \quad (\text{given})$$

From Eq. (12-14),

$$K_a = \frac{1 - \sin \phi}{1 + \sin \phi} \quad (12-14)$$

$$K_a = \frac{1 - \sin 34^\circ}{1 + \sin 34^\circ} = 0.283$$

$$s = 1.0 \text{ m} \quad (\text{given})$$

$$h = 0.3 \text{ m} \quad (\text{given})$$

$$w = 75 \text{ mm} = 0.075 \text{ m} \quad (\text{given})$$

$$\delta = \frac{\phi}{2} = \frac{34^\circ}{2} = 17^\circ$$

Therefore, substituting into Eq. (13-7) yields

$$L_{\min} = \frac{(1.5)(0.283)(1.0 \text{ m})(0.3 \text{ m})}{(2)(0.075 \text{ m})(\tan 17^\circ)} = 2.78 \text{ m}$$

From Eq. (13-9),

$$L_{\text{Rankine}} = H \tan\left(45^\circ - \frac{\phi}{2}\right) \quad (13-9)$$

$$L_{\text{Rankine}} = (6.0 \text{ m}) \tan\left(45^\circ - \frac{34^\circ}{2}\right) = 3.19 \text{ m}$$

From Eq. (13-8),

$$L_{\text{total}} = L_{\text{Rankine}} + L_{\min} \quad (13-8)$$

$$L_{\text{total}} = 3.19 \text{ m} + 2.78 \text{ m} = 5.97 \text{ m}$$

From Eq. (13-10),

$$(L_{\text{total}})_{\text{minimum}} = 0.80H \quad (13-10)$$

$$(L_{\text{total}})_{\text{minimum}} = (0.80)(6.0 \text{ m}) = 4.80 \text{ m}$$

Therefore, use a total length of strip of 5.97 m. (In practice, one would probably specify 6 m.)

2. From Eq. (13-11),

$$t = \frac{T}{wf_s} \quad (13-11)$$

From Eq. (13-4),

$$T = \gamma z K_a s h \quad (13-4)$$

$$\gamma = 17.12 \text{ kN/m}^3 \quad (\text{given})$$

$$z = 6.0 \text{ m} \quad (\text{given})$$

$$\begin{aligned} T &= (17.12 \text{ kN/m}^3)(6.0 \text{ m})(0.283)(1.0 \text{ m})(0.3 \text{ m}) \\ &= 8.72 \text{ kN} \end{aligned}$$

$$f_s = 138,000 \text{ kN/m}^2 \quad (\text{given})$$

Therefore, substituting into Eq. (13-11) yields

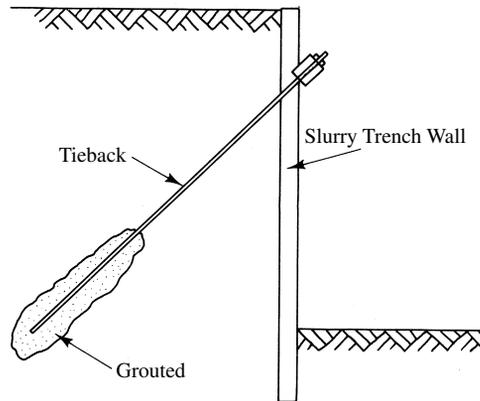
$$t = \frac{8.72 \text{ kN}}{(0.075 \text{ m})(138,000 \text{ kN/m}^2)} = 0.00084 \text{ m, or } 0.84 \text{ mm}$$

## 13-8 SLURRY TRENCH WALLS

The *slurry trench method* of constructing a wall to retain earth is applicable for retaining walls built entirely below ground level. The procedure includes excavating a trench the width of the wall while simultaneously filling the excavation with a viscous bentonite slurry, which exerts a lateral pressure and thereby helps stabilize the excavated wall. When excavation is complete, reinforcing steel is placed in the bentonite-slurry-filled trench, and, with the trench's soil walls acting as forms, concrete is poured from the bottom up. The concrete is delivered by a tremie (a long canvas tube) that is slowly raised as the excavated trench is filled with the concrete. Being displaced by the poured concrete, the lighter bentonite slurry rises to the top, where it is removed and may be saved for later use. Adjacent wall sections may be keyed together by a steel beam that becomes a part of the wall.

After the concrete has hardened, soil is removed from one side to expose the face of the wall and allow installation of a tieback system (see Figure 13-16). Depending on the height of wall, two or more levels of tiebacks may be used.

**FIGURE 13-16** Tieback for slurry trench wall.



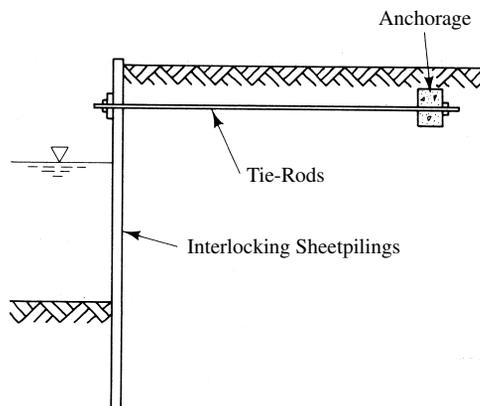
### 13-9 ANCHORED BULKHEADS

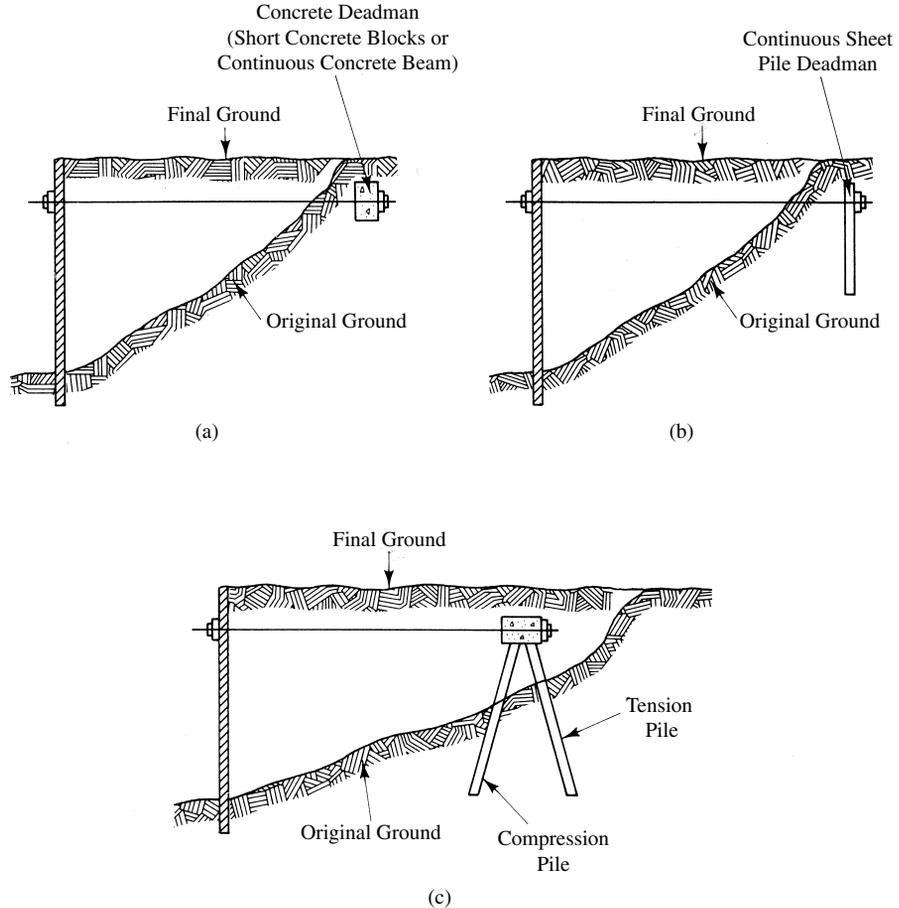
*Anchored bulkheads* are often used for various waterfront structures (e.g., wharves and waterfront retaining walls). As illustrated in Figure 13-17, they consist of interlocking sheetpilings driven into the soil and anchored by steel tie-rods or cables attached near the pilings' tops, extended through the ground, and anchored securely somewhere away from the sheet pile wall in firm soil. The distant anchors may be "deadmen" or braced piles.

In general, bulkheads can be constructed by one of two methods. Either the bulkhead can be built (driven) in open water and fill placed behind it, or the bulkhead can be constructed in the natural ground and earth removed from its face. The former type is known as a *fill bulkhead*; the latter, a *dredged bulkhead*.

As related in Figure 13-18, there are several possible anchoring systems available. Concrete and sheet pile deadmen (Figure 13-18a and 13-18b, respectively)

**FIGURE 13-17** Anchored bulkhead.



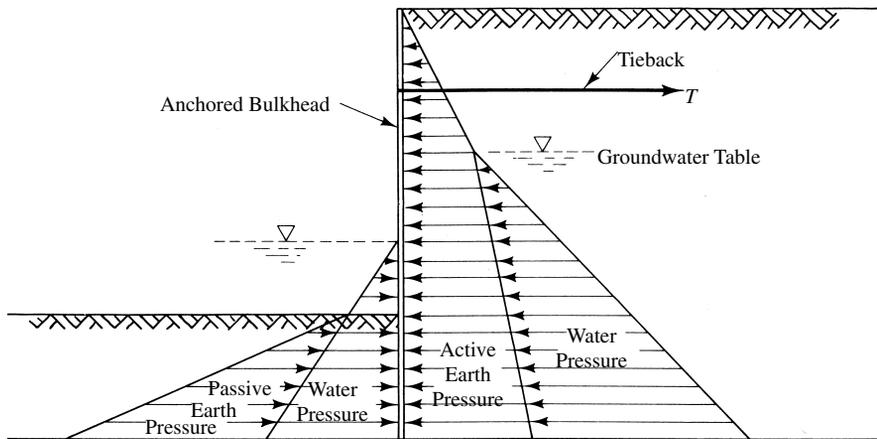


**FIGURE 13-18** Alternative anchoring systems for anchored bulkheads: (a) concrete deadman; (b) sheet pile deadman; (c) braced piles.

are usable in strong soils. They are, however, rather bulky and therefore require sufficient space. Where upper soils are weak or space is severely limited, braced piles (Figure 13-18c) may be used as anchors.

The fill behind a bulkhead applies lateral pressure to it and tends to push it forward. It is restrained at its top by the anchors and at its bottom by the soil in front of the bulkhead. It is also restrained by the standing water, but this tends to be offset by groundwater behind the bulkhead. Anchored bulkheads are usually subjected to fluctuating water levels; hence, engineers must base their design on “worst-case” conditions. (For example, tidal fluctuations produce high pressures behind the wall compared with those in front when the tide is out.)

Figure 13-19 shows the various forces acting on a typical anchored bulkhead. Actuating forces result from active pressure of the soil backfill and from water



**FIGURE 13-19** Forces acting on anchored bulkheads.

behind the wall. Resisting forces are composed of tension in the cable or tie-rod to the anchor, water pressure on the wall's front side, and passive resistance pressure of the soil within the penetrating depth of the sheetpiling. The ratio of total resisting force to total actuating force gives the bulkhead's factor of safety. The factor of safety can be increased by driving the sheetpiling deeper into the soil.

#### EXAMPLE 13-4

*Given*

An anchored bulkhead is to be constructed as shown in Figure 13-20, and a factor of safety of 1.5 is to be used.

*Required*

Analyze the bulkhead system and determine the tension in the anchor rod (tieback).

**Solution**

From Eqs. (12-10) and (12-12),

$$P_a = \frac{1}{2}\gamma H^2 K_a \quad (12-10)$$

$$P_p = \frac{1}{2}\gamma H^2 K_p \quad (12-12)$$

From Eq. (12-14),

$$K_a = \frac{1 - \sin \phi}{1 + \sin \phi} \quad (12-14)$$

$$K_a = \frac{1 - \sin 32^\circ}{1 + \sin 32^\circ} = 0.307$$

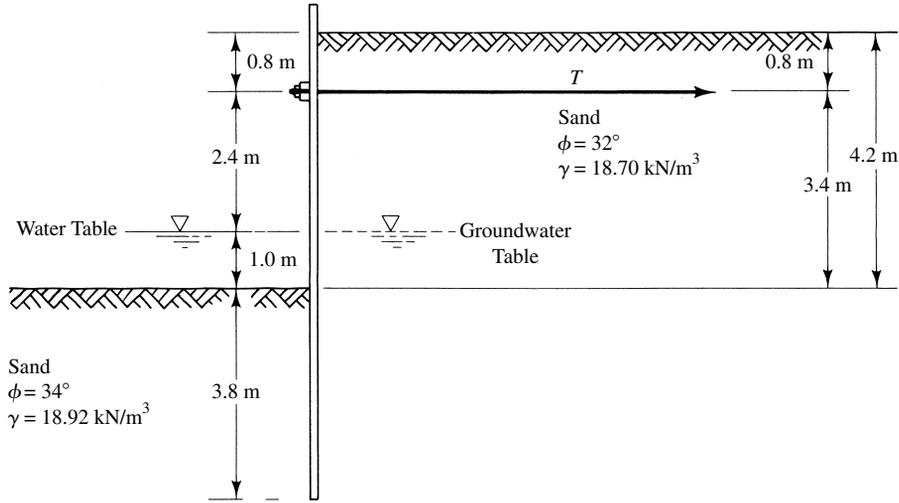


FIGURE 13-20

From Eq. (12-15),

$$K_p = \frac{1 + \sin \phi}{1 - \sin \phi} \tag{12-15}$$

$$K_p = \frac{1 + \sin 34^\circ}{1 - \sin 34^\circ} = 3.54$$

Component	Active Earth or Water Pressure (kN/m)	Moment about Tie Point (kN · m/m)
1	$(\frac{1}{2})(18.7)(3.2)^2(0.307) = 29.39$	$(29.39)[(3.2)(\frac{1}{3}) - 0.8] = 39.2$
2	$(3.2)(18.7)(4.8)(0.307) = 88.18$	$(88.18)[(4.8)(\frac{1}{2}) + 2.4] = 423.3$
3	$(\frac{1}{2})(18.7 - 9.81)(4.8)^2(0.307) = 31.44$	$(31.44)[(4.8)(\frac{1}{3}) + 2.4] = 176.1$
4	$(\frac{1}{2})(9.81)(4.8)^2 = 113.01$	$(113.01)[(4.8)(\frac{1}{3}) + 2.4] = 632.9$
	262.02	1271.5

Referring to Figure 13-21, one may determine applicable pressure and moments as follows:

To simplify the analysis that follows, let

- $R_{wp}$  = resistance from water pressure (left of sheetpiling) (i.e.,  $P_5$ )
- $(P_6)_{\min}$  = minimum required passive earth pressure in component 6
- $d_{P_6}$  = distance from tie point to  $P_6$

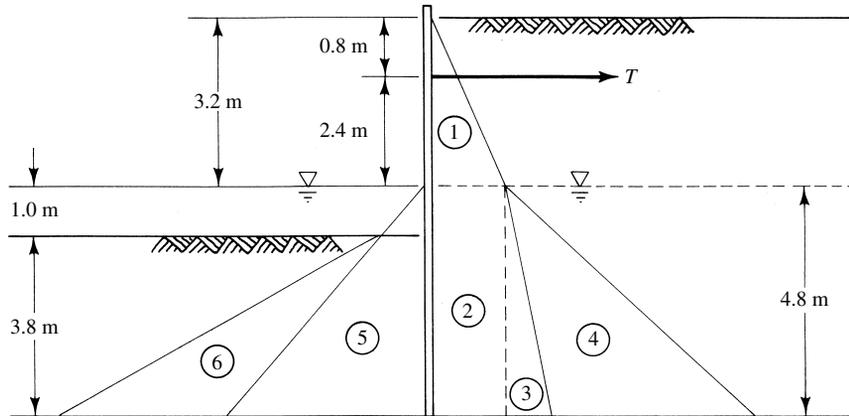


FIGURE 13-21

$d_{R_{wp}}$  = distance from tie point to  $R_{wp}$  (i.e.,  $d_{P_5}$ )

$(P_6)_{\text{mm}}$  = maximum mobilizable  $P_6$

$$\Sigma \text{ Moments}_{\text{tie point}} = 0 \curvearrow +$$

$$[(P_6)_{\text{min}}](d_{P_6}) + (R_{wp})(d_{R_{wp}}) - \text{Sum of moments of } P_1, P_2, P_3, \text{ and } P_4 = 0 \quad (\text{A})$$

$$d_{P_6} = (3.8 \text{ m})\left(\frac{2}{3}\right) + 1.0 \text{ m} + 2.4 \text{ m} = 5.933 \text{ m}$$

$$R_{wp} = \text{Force of component 5} = \frac{\frac{1}{2}\gamma H^2}{\text{F.S.}}$$

$$R_{wp} = \frac{\left(\frac{1}{2}\right)(9.81 \text{ kN/m}^3)(4.8 \text{ m})^2}{1.5} = 75.34 \text{ kN/m}$$

$$d_{R_{wp}} = (4.8 \text{ m})\left(\frac{2}{3}\right) + 2.4 \text{ m} = 5.600 \text{ m}$$

The sum of the moments of  $P_1, P_2, P_3,$  and  $P_4 = 1271.5 \text{ kN} \cdot \text{m/m}$  (from preceding tabulation). Substituting into Eq. (A) gives the following:

$$[(P_6)_{\text{min}}](5.933 \text{ m}) + (75.34 \text{ kN/m})(5.600 \text{ m}) - 1271.5 \text{ kN} \cdot \text{m/m} = 0$$

$$(P_6)_{\text{min}} = 143.20 \text{ kN/m}$$

$$(P_6)_{\text{mm}} = \frac{\frac{1}{2}(\gamma_{\text{soil}} - \gamma_{\text{water}})h^2 K_p}{\text{F.S.}}$$

$$(P_6)_{\text{mm}} = \frac{\left(\frac{1}{2}\right)(18.92 \text{ kN/m}^3 - 9.81 \text{ kN/m}^3)(3.8 \text{ m})^2(3.54)}{1.5} = 155.2 \text{ kN/m}$$

Because  $[(P_6)_{\text{mm}} = 155.2 \text{ kN/m}] > [(P_6)_{\text{min}} = 143.20 \text{ kN/m}]$ , the design is O.K. because the sheet pile penetration depth is adequate to develop sufficient passive resistance.

To determine the tension ( $T$ ) in the anchor rod, one must perform the following calculations:

$$\Sigma \text{ Forces}_{\text{horizontal}} = 0$$

$$T + R_{wp} + (P_6)_{\text{min}} - (P_1 + P_2 + P_3 + P_4) = 0$$

$$P_1 + P_2 + P_3 + P_4 = 262.02 \text{ kN/m} \quad (\text{from previous tabulation})$$

$$T + 75.34 \text{ kN/m} + 143.20 \text{ kN/m} - 262.02 \text{ kN/m} = 0$$

$$T = 43.48 \text{ kN/m}$$

### EXAMPLE 13-5

*Given*

A continuous deadman is to be designed and installed near the ground surface, as shown in Figure 13-22. Anchor rod (tieback) tension is to be 75 kN/m.

*Required*

Design the deadman, using a factor of safety against anchor resistance failure of 1.5.

**Solution**

$$\text{Capacity of deadman} = \text{Tieback tension} = \frac{\frac{1}{2}\gamma H^2(K_p - K_a)}{\text{F.S.}}$$

From Eq. (12-14),

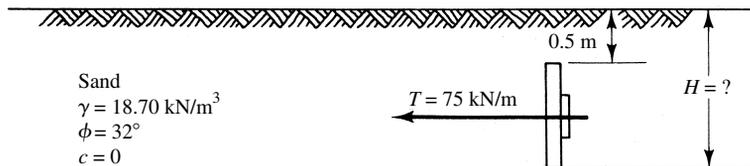
$$K_a = \frac{1 - \sin \phi}{1 + \sin \phi} \quad (12-14)$$

$$K_a = \frac{1 - \sin 32^\circ}{1 + \sin 32^\circ} = 0.307$$

From Eq. (12-15),

$$K_p = \frac{1 + \sin \phi}{1 - \sin \phi} \quad (12-15)$$

FIGURE 13-22



$$K_p = \frac{1 + \sin 32^\circ}{1 - \sin 32^\circ} = 3.255$$

$$75 \text{ kN/m} = \frac{(\frac{1}{2})(18.70 \text{ kN/m}^3)(H)^2(3.255 - 0.307)}{1.5}$$

$$H = 2.02 \text{ m}$$

The deadman should be placed with its bottom 2.02 m below the ground surface.

### 13-10 PROBLEMS

- 13-1. A proposed L-shaped reinforced-concrete retaining wall is shown in Figure 13-23. The backfill material will be Type 2 soil (Figure 13-3), and its unit weight is  $125 \text{ lb/ft}^3$ . The coefficient of base friction is estimated to be 0.48, and allowable soil pressure for the foundation soil is 4 kips/ft<sup>2</sup>. Determine the (a) factor of safety against overturning, (b) factor of safety against sliding, and (c) factor of safety against failure of the foundation.
- 13-2. Investigate the stability against overturning, sliding resistance (consider passive earth pressure at the toe), and foundation soil pressure of the retaining wall shown in Figure 13-24. The retaining wall is to support a deposit of granular soil, which has a unit weight of  $17.30 \text{ kN/m}^3$  and an angle of internal friction of  $32^\circ$ . The coefficient of base friction is 0.50. Allowable soil

FIGURE 13-23

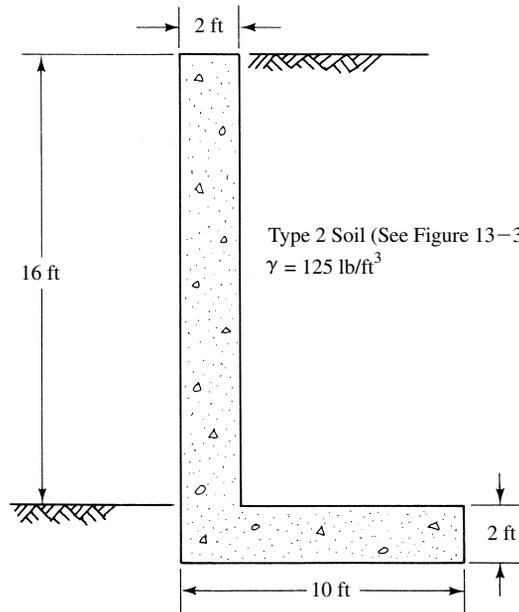
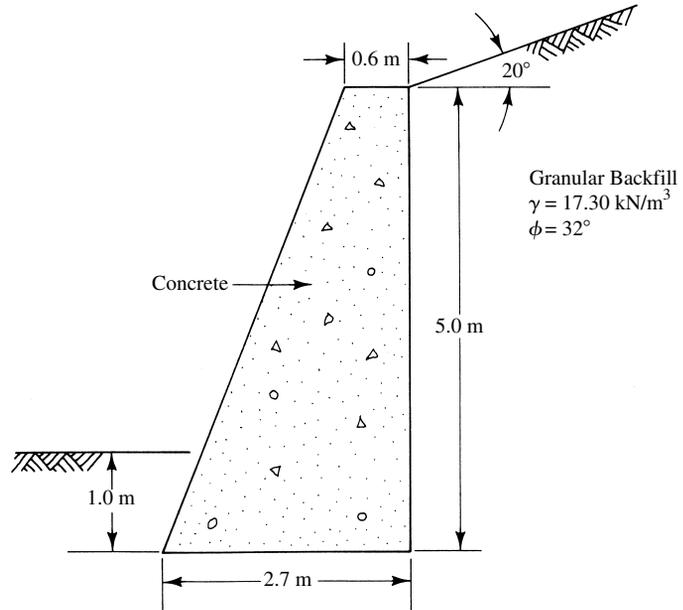


FIGURE 13-24



pressure for the foundation soil is  $144 \text{ kN/m}^2$ . Use Rankine theory to calculate both active and passive earth pressures.

- 13-3. For the retaining wall shown in Figure 13-25, compute the factors of safety against overturning and sliding (analyze the latter both without and with passive earth pressure at the toe). Also determine the soil pressure at the base of the wall. Use the Rankine equation to compute passive earth pressure.
- 13-4. An 8-m-high Reinforced Earth wall is to be built with level backfill and without a surcharge on the backfill. Sand with an angle of internal friction of  $36^\circ$  and a unit weight of  $17.0 \text{ kN/m}^3$  will be used as backfill material. The steel strips are 90 mm wide and 0.762 mm thick and have an allowable stress of  $138,000 \text{ kN/m}^2$ . For vertical spacing of 0.4 m, determine the required total length and horizontal spacing of the steel strips.
- 13-5. Analyze the anchored bulkhead system shown in Figure 13-26, using a factor of safety of 1.5. Determine the anchor rod tension per unit length of sheetpiling.
- 13-6. A continuous deadman is to be designed and constructed near ground surface, as shown in Figure 13-27. Anchor rod tension is to be  $79 \text{ kN/m}$ . Using a factor of safety of 1.5, determine how far the bottom of the deadman should be placed below the ground surface.

FIGURE 13-25

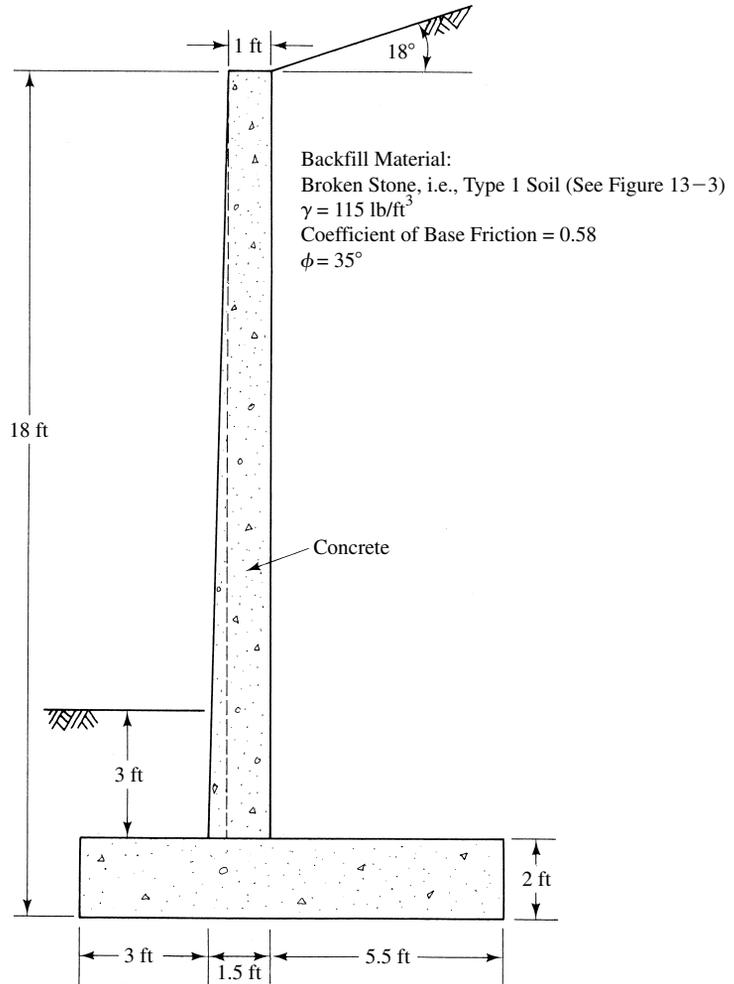


FIGURE 13–26

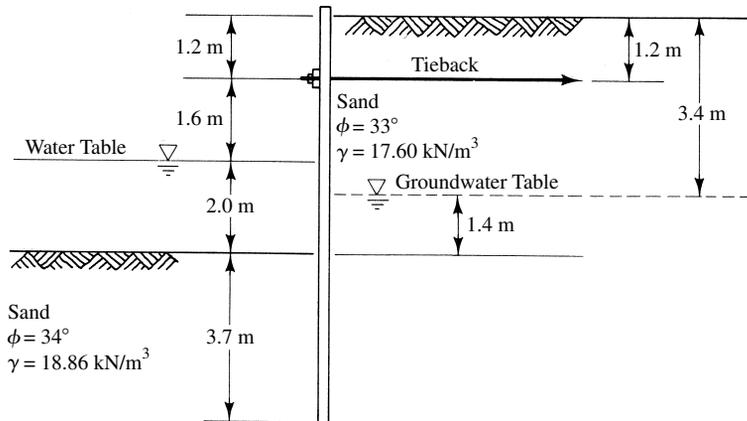


FIGURE 13–27

