CHAPTER 14

RETAINING WALLS

Apartment building, Miami, Florida.

14.1 INTRODUCTION

Retaining walls are structural members used to provide stability for soil or other materials and to prevent them from assuming their natural slope. In this sense, the retaining wall maintains unequal levels of earth on its two faces. The retained material on the higher level exerts a force on the retaining wall that may cause its overturning or failure. Retaining walls are used in bridges as abutments, in buildings as basement walls, and in embankments. They are also used to retain liquids, as in water tanks and sewage treatment tanks.

14.2 TYPES OF RETAINING WALLS

Retaining walls may be classified as follows (refer to Fig. 14.1):

1. *Gravity walls* usually consist of plain concrete or masonry and depend entirely on their own weight to provide stability against the thrust of the retained material. These walls are proportioned so that tensile stresses do not develop in the concrete or masonry due to the exerted forces on the wall. The practical height of a gravity wall does not exceed 10 ft.
2. *Semigravity walls* are gravity walls that have a wider base to improve the stability of the wall and to prevent the development of tensile stresses in the base. Light reinforcement is sometimes used in the base or stem to reduce the large section of the wall.
3. The *cantilever retaining wall* is a reinforced concrete wall that is generally used for heights from 8 to 20 ft. It is the most common type of retaining structure because of economy and simplicity of construction. Various types of cantilever retaining walls are shown in Fig. 14.1.
4. *Counterfort retaining walls* higher than 20 ft develop a relatively large bending moment at the base of the stem, which makes the design of such walls uneconomical. One solution in this case is to introduce transverse walls (or counterforts) that tie the stem and the base together at intervals. The counterforts act as tension ties supporting the vertical walls. Economy is

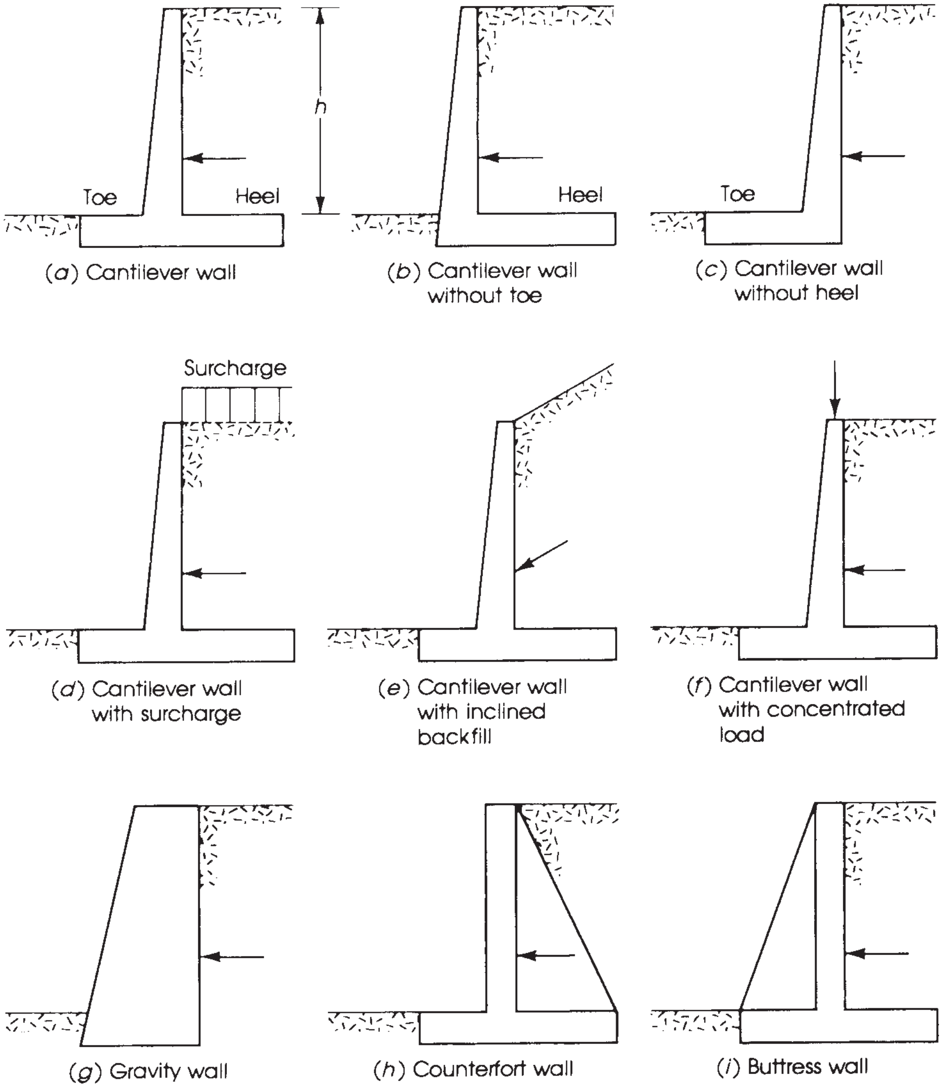


Figure 14.1 Types of retaining walls.

achieved because the stem is designed as a continuous slab spanning horizontally between counterforts, whereas the heel is designed as a slab supported on three sides (Fig. 14.1*h*).

5. The *buttressed retaining wall* is similar to the counterfort wall, but in this case the transverse walls are located on the opposite, visible side of the stem and act in compression (Fig. 14.1*i*). The design of such walls becomes economical for heights greater than 20 ft. They are not popular because of the exposed buttresses.
6. *Bridge abutments* are retaining walls that are supported at the top by the bridge deck. The wall may be assumed fixed at the base and simply supported at the top.
7. *Basement walls* resist earth pressure from one side of the wall and span vertically from the basement-floor slab to the first-floor slab. The wall may be assumed fixed at the base and simply supported or partially restrained at the top.

14.3 FORCES ON RETAINING WALLS

Retaining walls are generally subjected to gravity loads and to earth pressure due to the retained material on the wall. Gravity loads due to the weights of the materials are well defined and can be calculated easily and directly. The magnitude and direction of the earth pressure on a retaining wall depends on the type and condition of soil retained and on other factors and cannot be determined as accurately as gravity loads. Several references on soil mechanics[1,2] explain the theories and procedure for determining the soil pressure on retaining walls. The stability of retaining walls and the effect of dynamic reaction on walls are discussed in Refs. 3 and 4.

Granular materials, such as sand, behave differently from cohesive materials, such as clay, or from any combination of both types of soils. Although the pressure intensity of soil on a retaining wall is complex, it is common to assume a linear pressure distribution on the wall. The pressure intensity increases with depth linearly, and its value is a function of the height of the wall and the weight and type of soil. The pressure intensity, p, at a depth h below the earth's surface may be calculated as follows:

$$p = Cwh \tag{14.1}$$

where w is the unit weight of soil and C is a coefficient that depends on the physical properties of soil. The value of the coefficient C varies from 0.3 for loose granular soil, such as sand, to about 1.0 for cohesive soil, such as wet clay. If the retaining wall is assumed absolutely rigid, a case of earth pressure at rest develops. Under soil pressure, the wall may deflect or move a small amount from the earth, and active soil pressure develops, as shown in Fig. 14.2. If the wall moves toward the soil, a passive soil pressure develops. Both the active and passive soil pressures are assumed to vary linearly with the depth of the wall (Fig. 14.2). For dry, granular, noncohesive materials, the assumed linear pressure diagram is fairly satisfactory; cohesive soils or saturated sands behave in a different, nonlinear manner. Therefore, it is very common to use granular materials as backfill to provide an approximately linear pressure diagram and also to provide for the release or drainage of water from behind the wall.

For a linear pressure, the active and passive pressure intensities are determined as follows:

$$P_a = C_a wh \tag{14.2}$$

$$P_p = C_p wh \tag{14.3}$$

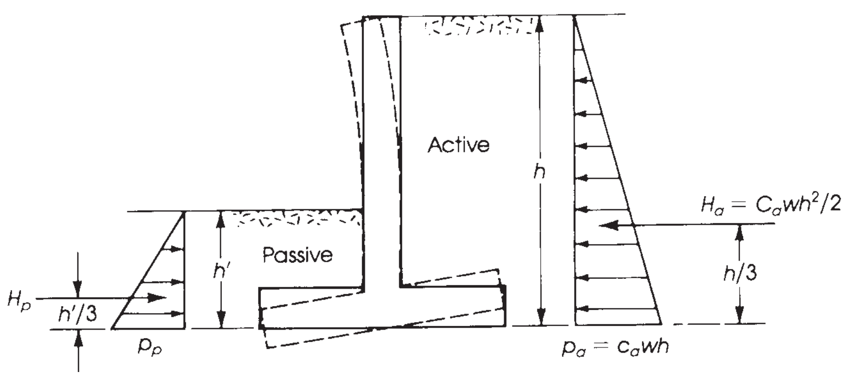


Figure 14.2 Active and passive earth pressure.

where C_a and C_p are the approximate coefficients of the active and passive pressures, respectively.

14.4 ACTIVE AND PASSIVE SOIL PRESSURES

The two theories most commonly used in the calculation of earth pressure are those of Rankine and Coulomb [1,6].

1. In Rankine's approach, the retaining wall is assumed to yield a sufficient amount to develop a state of plastic equilibrium in the soil mass at the wall surface. The rest of the soil remains in the state of elastic equilibrium. The theory applies mainly to a homogeneous, incompressible, cohesionless soil and neglects the friction between soil and wall. The active soil pressure at a depth h on a retaining wall with a horizontal backfill based on Rankine's theory is determined as follows:

$$P_a = C_a wh = wh\left(\frac{1-\sin\phi}{1+\sin\phi}\right) \tag{14.4}$$

where

$$C_a = \left(\frac{1-\sin\phi}{1+\sin\phi}\right)$$

ϕ = angle of internal friction of the soil (Table 14.1)

and

$$\text{Total active pressure,}\ H_a = \frac{wh^2}{2}\left(\frac{1-\sin\phi}{1+\sin\phi}\right) \tag{14.5}$$

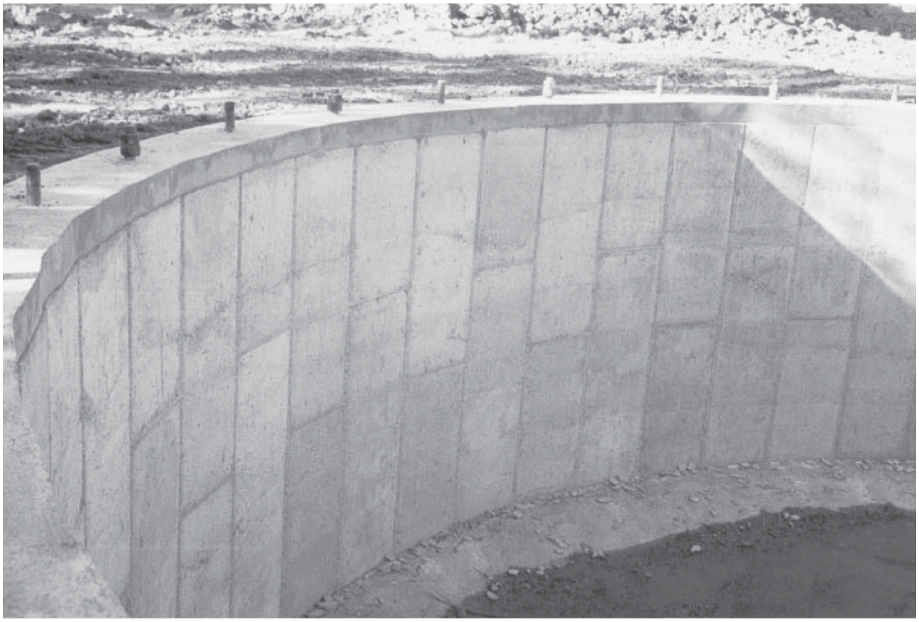
The resultant, H_a, acts at $h/3$ from the base (Fig. 14.2). When the earth is surcharged at an angle δ to the horizontal, then

$$C_a = \cos\delta\left(\frac{\cos\delta - \sqrt{\cos^2\delta - \cos^2\phi}}{\cos\delta + \sqrt{\cos^2\delta - \cos^2\phi}}\right)$$

$$P_a = C_a wh \quad \text{and} \quad H_a = C_a\frac{wh^2}{2} \tag{14.6}$$

Table 14.1 Values of w and ϕ

Type of Backfill	Unit Weight, W pcf	Unit Weight, W kg/m^3	Angle of Internal Friction, ϕ
Soft clay	90–120	1440–1920	0°–15°
Medium clay	100–120	1600–1920	15°–30°
Dry loose silt	100–120	1600–1920	27°–30°
Dry dense silt	110–120	1760–1920	30°–35°
Loose sand and gravel	100–130	1600–2100	30°–40°
Dense sand and gravel	120–130	1920–2100	25°–35°
Dry loose sand, graded	115–130	1840–2100	33°–35°
Dry dense sand, graded	120–130	1920–2100	42°–46°



Reinforced concrete retaining wall.

Retaining wall in a parking area.

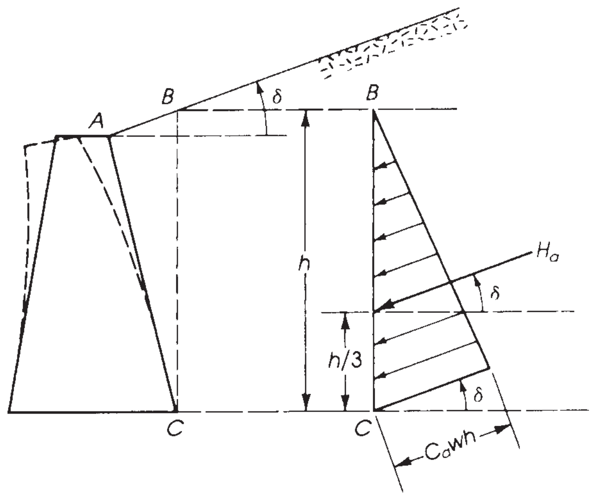


Figure 14.3 Active soil pressure with surcharge.

Table 14.2 Values of C_a

	$\phi = C_a$						
δ	$\phi = 28°$	$\phi = 30°$	$\phi = 32°$	$\phi = 34°$	$\phi = 36°$	$\phi = 38°$	$\phi = 40°$
0°	0.361	0.333	0.307	0.283	0.260	0.238	0.217
10°	0.380	0.350	0.321	0.294	0.270	0.246	0.225
20°	0.461	0.414	0.374	0.338	0.306	0.277	0.250
25°	0.573	0.494	0.434	0.385	0.343	0.307	0.275
30°	0	0.866	0.574	0.478	0.411	0.358	0.315

The resultant, H_a, acts at $h/3$ and is inclined at an angle δ to the horizontal (Fig. 14.3). The values of C_a expressed by Eq. 14.6 for different values of δ and angle of internal friction ϕ are shown in Table 14.2.

Passive soil pressure develops when the retaining wall moves against and compresses the soil. The passive soil pressure at a depth h on a retaining wall with horizontal backfill is determined as follows:

$$P_p = C_p wh = wh\left(\frac{1+\sin\phi}{1-\sin\phi}\right) \tag{14.7}$$

where

$$C_p = \left(\frac{1+\sin\phi}{1-\sin\phi}\right) = \frac{1}{C_a}$$

Total passive pressure is

$$H_p = \frac{wh^2}{2}\left(\frac{1+\sin\phi}{1-\sin\phi}\right) \tag{14.8}$$

The resultant, H_p, acts at $h'/3$ from the base (Fig. 14.2). When the earth is surcharged at an angle δ to the horizontal, then

$$C_p = \cos\delta \left(\frac{\cos\delta + \sqrt{\cos^2\delta - \cos^2\phi}}{\cos\delta - \sqrt{\cos^2\delta - \cos^2\phi}} \right)$$

$$P_p = C_p wh \qquad \text{and} \qquad H_p = C_p \frac{wh^2}{2} \tag{14.9}$$

In this case H_p acts at $h'/3$ and is inclined at an angle δ to the horizontal (Fig. 14.4). The values of C_p expressed by Eq. 14.9 for different values of δ and ϕ are shown in Table 14.3.

The values of ϕ and w vary with the type of backfill used. As a guide, common values of ϕ and w are given in Table 14.1.

2. In Coulomb's theory, the active soil pressure is assumed to be the result of the tendency of a wedge of soil to slide against the surface of a retaining wall. Hence, Coulomb's theory is referred to as the wedge theory. While it takes into consideration the friction of the soil on the retaining wall, it assumes that the surface of sliding is a plane, whereas in reality it is slightly curved. The error in this assumption is negligible in calculating the active soil pressure. Coulomb's equations to calculate the active and passive soil pressure are as follows:

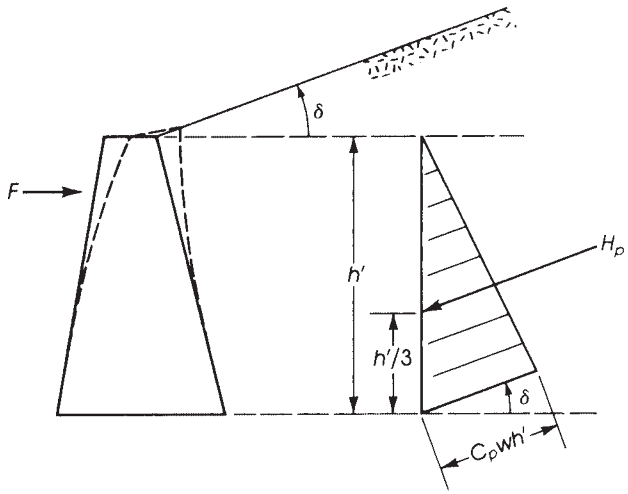


Figure 14.4 Passive soil pressure with surcharge.

Table 14.3 Values of C_p

	$\phi = C_p$						
δ	$\phi = 28°$	$\phi = 30°$	$\phi = 32°$	$\phi = 34°$	$\phi = 36°$	$\phi = 38°$	$\phi = 40°$
0°	2.77	3.00	3.25	3.54	3.85	4.20	4.60
10°	2.55	2.78	3.02	3.30	3.60	3.94	4.32
20°	1.92	2.13	2.36	2.61	2.89	3.19	3.53
25°	1.43	1.66	1.90	2.14	2.40	2.68	3.00
30°	0	0.87	1.31	1.57	1.83	2.10	2.38

The active soil pressure is

$$P_a = C_a wh$$

where

$$C_a = \frac{\cos^2(\phi - \theta)}{\cos^2\theta \cos(\theta + \beta)\left[1 + \sqrt{\dfrac{\sin(\phi + \beta)\ \sin(\phi - \delta)}{\cos(\theta + \beta)\cos(\theta - \delta)}}\right]^2} \tag{14.10a}$$

where

ϕ = angle of internal friction of soil
θ = angle of soil pressure surface from vertical
β = angle of friction along wall surface (angle between soil and concrete)
δ = angle of surcharge to horizontal

The total active soil pressure is

$$H_a = C_a \frac{wh^2}{2} = p_a \frac{h}{2}$$

When the wall surface is vertical, $\theta = 0°$, and if $\beta = \delta$, then C_a in Eq. 14.10a reduces to Eq. 14.6 of Rankine.

Passive soil pressure is

$$P_p = C_p wh' \quad \text{and} \quad H_p = \left(\frac{wh'^2}{2}\right) C_p = P_p \frac{h'}{2}$$

where

$$C_p = \frac{\cos^2(\phi + \theta)}{\cos^2\theta \cos(\theta - \beta)\left[1 - \sqrt{\dfrac{\sin(\phi + \beta)\sin(\phi + \delta)}{\cos(\theta - \beta)\cos(\phi - \delta)}}\right]^2} \tag{14.10b}$$

The values of ϕ and w vary with the type of backfill used. As a guide, common values of ϕ and w are given in Table 14.1.

3. When the soil is saturated, the pores of the permeable soil are filled with water, which exerts hydrostatic pressure. In this case the buoyed unit weight of soil must be used. The buoyed unit weight (or submerged unit weight) is a reduced unit weight of soil and equals w minus the weight of water displaced by the soil. The effect of the hydrostatic water pressure must be included in the design of retaining walls subjected to a high water table and submerged soil. The value of the angle of internal friction may be used, as shown in Table 14.1.

14.5 EFFECT OF SURCHARGE

Different types of loads are often imposed on the surface of the backfill behind a retaining wall. If the load is uniform, an equivalent height of soil, h_s, may be assumed acting on the wall to account for the increased pressure. For the wall shown in Fig. 14.5, the horizontal pressure due to the surcharge is constant throughout the depth of the retaining wall.

$$h_s = \frac{w_s}{w} \tag{14.11}$$

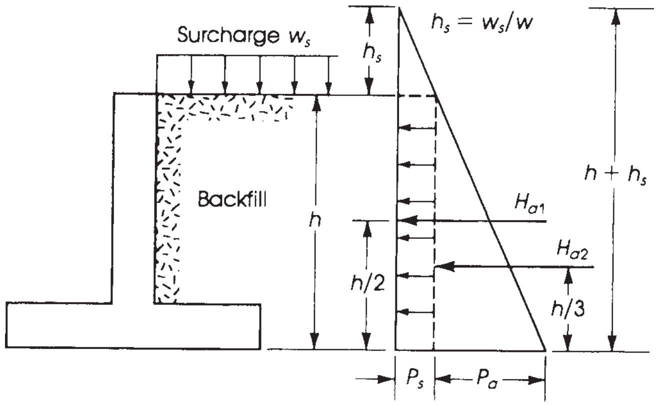


Figure 14.5 Surcharge effect under a uniform load.

where

h_s = equivalent height of soil (ft)
w_s = pressure of the surcharge (psf)
w = unit weight of soil (pcf)

The total pressure is

$$H_a = H_{a1} + H_{a2} = C_a w \left(\frac{h^2}{2} + hh_s \right) \tag{14.12}$$

In the case of partial uniform load acting at a distance from the wall, only a portion of the total surcharge pressure affects the wall (Fig. 14.6).

It is common practice to assume that the effective height of pressure due to partial surcharge is h', measured from point B to the base of the retaining wall [1]. The line AB forms an angle of 45° with the horizontal.

In the case of wheel load acting at a distance from the wall, the load is to be distributed over a specific area, which is usually defined by known specifications such as AASHTO and AREA [4] specifications.

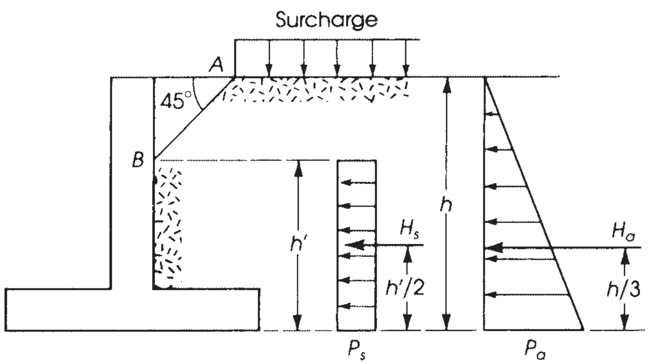


Figure 14.6 Surcharge effect under a partial uniform load at a distance from the wall.

14.6 FRICTION ON THE RETAINING WALL BASE

The horizontal component of all forces acting on a retaining wall tends to push the wall in a horizontal direction. The retaining wall base must be wide enough to resist the sliding of the wall. The coefficient of friction to be used is that of soil on concrete for coarse granular soils and the shear strength of cohesive soils [4]. The coefficients of friction μ that may be adopted for different types of soil are as follows:

- Coarse-grained soils without silt, $\mu = 0.55$
- Coarse-grained soils with silt, $\mu = 0.45$
- Silt, $\mu = 0.35$
- Sound rock, $\mu = 0.60$

The total frictional force, F, on the base to resist the sliding effect is

$$F = \mu R + H_p \tag{14.13}$$

where

μ = coefficient of friction
R = vertical force acting on base
H_p = passive resisting force

The factor of safety against sliding is

$$\text{Factor of safety} = \frac{F}{H_{\text{ah}}} = \frac{\mu R + H_p}{H_{\text{ah}}} \geq 1.5 \tag{14.14}$$

where H_{ah} is the horizontal component of the active pressure, H_a. The factor of safety against sliding should not be less than 1.5 if the passive resistance H_p is neglected and should not be less than 2.0 if H_p is taken into consideration.

14.7 STABILITY AGAINST OVERTURNING

The horizontal component of the active pressure, H_a, tends to overturn the retaining wall about the point zero on the toe (Fig. 14.7). The overturning moment is equal to $M_0 = H_a h/3$. The weight of the concrete and soil tends to develop a balancing moment, or righting moment, to resist the overturning moment. The balancing moment for the case of the wall shown in Fig. 14.7 is equal to

$$M_b = w_1 x_1 + w_2 x_2 + w_3 x_3 = \sum wx$$

The factor of safety against overturning is

$$\frac{M_b}{M_0} = \frac{\sum wx}{H_a h/3} \geq 2.0 \tag{14.15}$$

This factor of safety should not be less than 2.0.

The resultant of all forces acting on the retaining wall, R_A, intersects the base at point C (Fig. 14.7). In general, point C does not coincide with the center of the base, L, thus causing eccentric loading on the footing. It is desirable to keep point C within the middle third to get the whole footing under soil pressure. (The case of a footing under eccentric load was discussed in Chapter 13.)

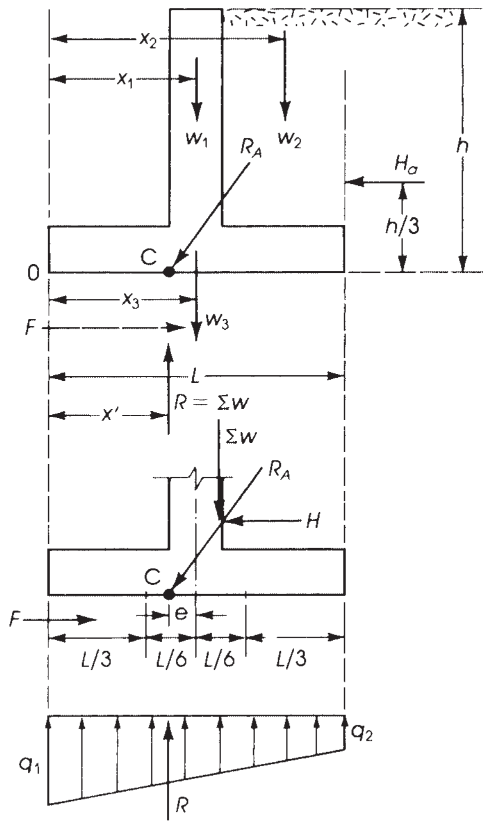


Figure 14.7 Overturning of a cantilever retaining wall.

14.8 PROPORTIONS OF RETAINING WALLS

The design of a retaining wall begins with a trial section and approximate dimensions. The assumed section is then checked for stability and structural adequacy. The following rules may be used to determine the approximate sizes of the different parts of a cantilever retaining wall.

1. *Height of Wall*. The overall height of the wall is equal to the difference in elevation required plus 3 to 4 ft, which is the estimated frost penetration depth in northern states.
2. *Thickness of Stem*. The intensity of the pressure increases with the depth of the stem and reaches its maximum value at the base level. Consequently the maximum bending moment and shear in the stem occur at its base. The stem base thickness may be estimated as $\frac{1}{12}$ to $\frac{1}{10}$ of the height h. The thickness at the top of the stem may be assumed to be 8 to 12 in. Because retaining walls are designed for active earth pressure, causing a small deflection of the wall, it is advisable to provide the face of the wall with a batter (taper) of $\frac{1}{4}$ in. per foot of height, h, to compensate for the forward deflection. For short walls up to 10 ft high, a constant thickness may be adopted.
3. *Length of Base*. An initial estimate for the length of the base of $\frac{2}{5}$ to $\frac{2}{3}$ of the wall height, h, may be adopted.

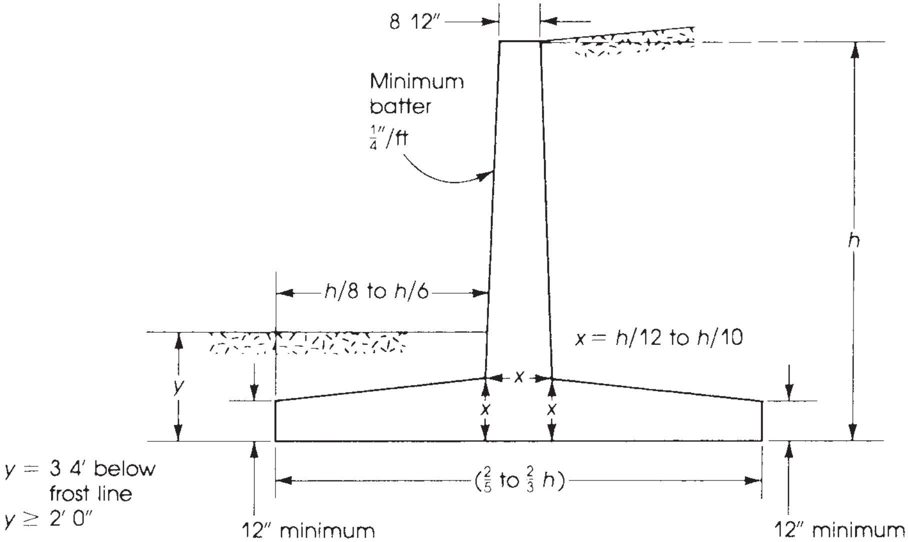


Figure 14.8 Trial proportions of a cantilever retaining wall.

4. *Thickness of Base*. The base thickness below the stem is estimated as the same thickness of the stem at its base, that is, $\frac{1}{12}$ to $\frac{1}{10}$ of the wall height. A minimum thickness of about 12 in. is recommended. The wall base may be of uniform thickness or tapered to the ends of the toe and heel, where the bending moment is 0.

   The approximate initial proportions of a cantilever retaining wall are shown in Fig. 14.8.

14.9 DESIGN REQUIREMENTS

The ACI Code, Chapter 11, provides methods for bearing wall design. The main requirements are as follows:

1. The minimum thickness of bearing walls is $\frac{1}{25}$ the supported height or length, whichever is shorter, but not less than 4 in.
2. The minimum area of the horizontal reinforcement in the wall is $0.0025bh$, where bh is the gross concrete wall area. This value may be reduced to $0.0020bh$ if no. 5 or smaller deformed bars with $f_y \geq 60$ ksi are used. For welded wire fabric (plain or deformed), the minimum steel area is $0.0020bh$.
3. The minimum area of the vertical reinforcement is $0.0015bh$, but it may be reduced to $0.0012bh$ if no. 5 or smaller deformed bars with $f_y \geq 60$ ksi are used. For welded wire fabric (plain or deformed), the minimum steel area is $0.0012bh$.
4. The maximum spacing of the vertical or the horizontal reinforcing bars is the smaller of 18 in. or three times the wall thickness.
5. If the wall thickness exceeds 10 in., the vertical and horizontal reinforcement should be placed in two layers parallel to the exterior and interior wall surfaces, as follows:

For exterior wall surfaces, at least $\frac{1}{2}$ of the reinforcement A_s (but not more than $\frac{2}{3}A_s$) should have a minimum concrete cover of 2 in. but not more than $\frac{1}{3}$ of the wall thickness. This is because the exterior surface of the wall is normally exposed to different weather conditions and temperature changes.

For interior wall surfaces, the balance of the required reinforcement in each direction should have a minimum concrete cover of $\frac{3}{4}$ in. but not more than $\frac{1}{3}$ of the wall thickness.

The minimum steel area in the wall footing (heel or toe), according to the ACI Code, Section 7.6.1, is that required for shrinkage and temperature reinforcement, which is $0.0018bh$ when $f_y = 60$ ksi and $0.0020bh$ when $f_y = 40$ ksi or 50 ksi. Because this minimum steel area is relatively small, it is a common practice to increase it to that minimum A_s required for flexure:

$$A_{s,\min} = \left(\frac{3\sqrt{f'_c}}{f_y}\right) bd \geq \left(\frac{200}{f_y}\right) bd \tag{14.16}$$

14.10 DRAINAGE

The earth pressure discussed in the previous sections does not include any hydrostatic pressure. If water accumulates behind the retaining wall, the water pressure must be included in the design. Surface or underground water may seep into the backfill and develop the case of submerged soil. To avoid hydrostatic pressure, drainage should be provided behind the wall. If well-drained cohesionless soil is used as a backfill, the wall can be designed for earth pressure only. The drainage system may consist of one or a combination of the following:

1. Weep holes in the retaining wall of 4 in. or more in diameter and spaced about 5 ft on centers horizontally and vertically (Fig. 14.9*a*).
2. Perforated pipe 8 in. in diameter laid along the base of the wall and surrounded by gravel (Fig. 14.9*b*).
3. Blanketing or paving the surface of the backfill with asphalt to prevent seepage of water from the surface.
4. Any other method to drain surface water.

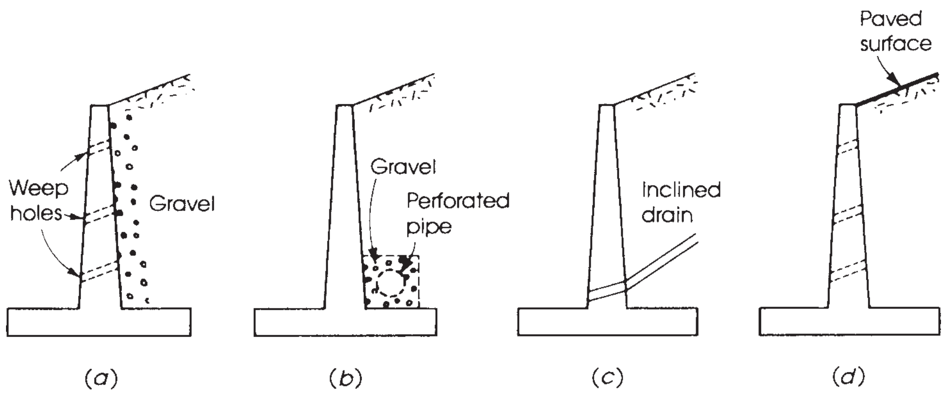


Figure 14.9 Drainage systems.

Example 14.1

The trial section of a semigravity plain concrete retaining wall is shown in Fig. 14.10. It is required to check the safety of the wall against overturning, sliding, and bearing pressure under the footing.

Given: Weight of backfill is 110 pcf, angle of internal friction is $\phi = 35°$, coefficient of friction between concrete and soil is $\mu = 0.5$, allowable soil pressure is 2.5 ksf, and $f'_c = 3$ ksi.

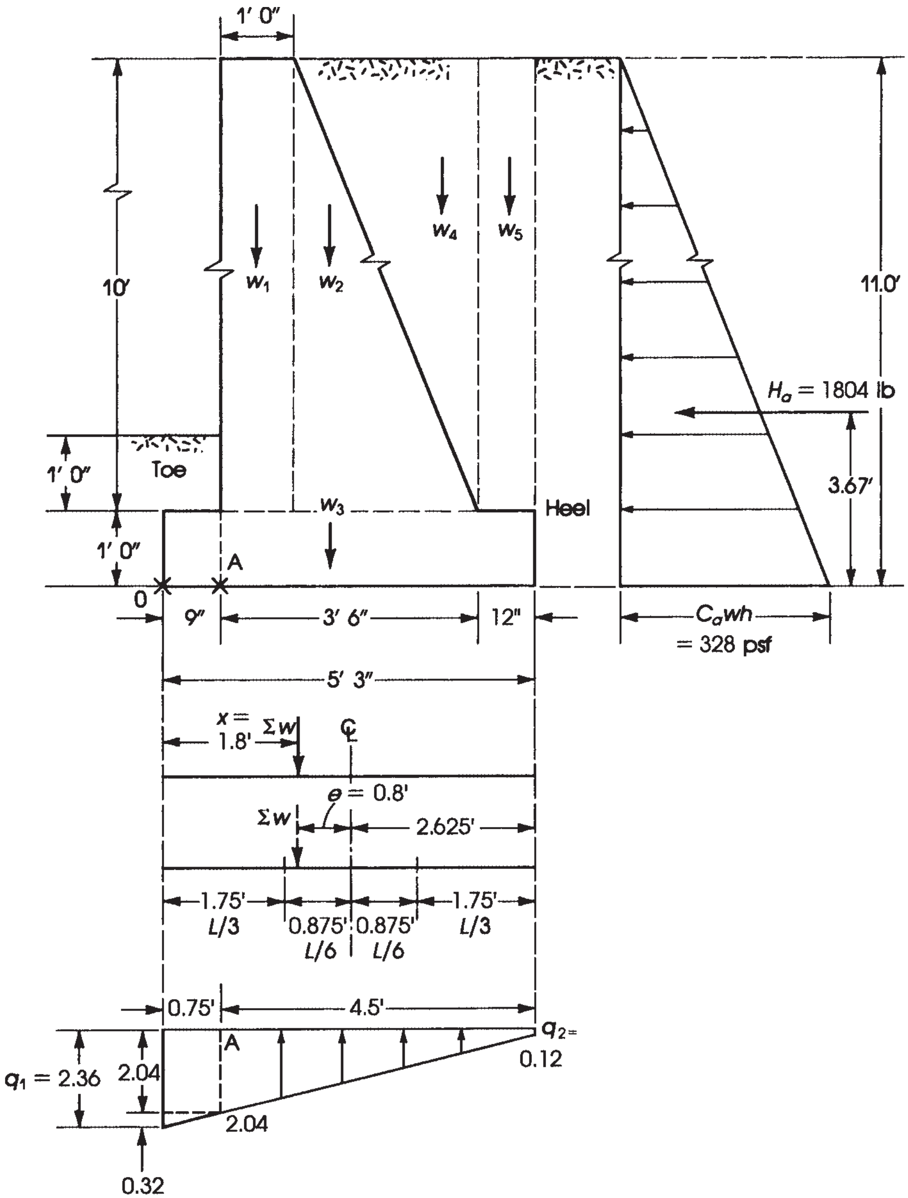


Figure 14.10 Example 14.1.

Solution

1. Using the Rankine equation,

$$C_a = \frac{1 - \sin\phi \sin\phi}{1 + \sin\phi} = \frac{1 - 0.574}{1 + 0.574} = 0.271$$

The passive pressure on the toe is that for a height of 1 ft, which is small and can be neglected.

$$H_a = \frac{C_a w h^2}{2} = \frac{0.271}{2}(110)(11)^2 = 1804 \text{ lb}$$

and H_a acts at a distance $h/3 = \frac{11}{3} = 3.67$ ft from the bottom of the base.

2. The overturning moment is $M_0 = 1.804 \times 3.67 = 6.62$ K·ft.

3. Calculate the balancing moment, M_b, taken about the toe end 0 (Fig. 14.10):

Weight (lb)	Arm (ft)	Moment (K·ft)
$w_1 = 1 \times 10 \times 145 = 1450$	1.25	1.81
$w_2 = \frac{1}{2} \times 2.5 \times 10 \times 145 = 1812$	2.60	4.71
$w_3 = 5.25 \times 1 \times 145 = 725$	2.625	2.00
$w_4 = \frac{1}{2} \times 2.5 \times 10 \times 110 = 1375$	3.42	4.70
$w_5 = \frac{12}{12} \times 10 \times 110 = 1100$	4.75	5.22

$$\sum w = R = 6.50 \text{ K} \quad M_b = \sum M = 18.44 \text{ K} \cdot \text{ft}$$

4. The factor of safety against overturning is $18.44/6.62 = 2.78 > 2.0$.

5. The force resisting sliding, $F = \mu R$, is $F = 0.5(6.50) = 3.25$ K. The factor of safety against sliding is $F/H_a = 3.25/1.804 = 1.8 > 1.5$.

6. Calculate the soil pressure under the base:

a. The distance of the resultant from toe end 0 is

$$x = \frac{M_b - M_0}{R} = \frac{18.44 - 6.62}{6.50} = 1.82 \text{ ft}$$

The eccentricity is $e = 2.62 - 1.82 = 0.80$ ft. The resultant R acts just inside the middle third of the base and has an eccentricity of $e = 0.8$ ft from the center of the base (Fig. 14.10). For a 1-ft length of the footing, the effective width of footing is 5.25 ft.

b. The moment of inertia is $I = 1.0(5.25)^3/12 = 12.1 \text{ ft}^4$. Area $= 5.25 \text{ ft}^2$.

c. The soil pressures at the two extreme ends of the footing are $q_1, q_2 = R/A \pm Mc/I$. The moment M is $Re = 6.50(0.8) = 5.2$ K·ft; $c = 2.62$ ft.

$$q_1 = \frac{6.50}{5.25} + \frac{5.2(2.62)}{12.1} = 1.24 + 1.12 = 2.36 \text{ ksf}$$

$$q_2 = 1.24 - 1.12 = 0.12 \text{ ksf}$$

7. Check the bending stress in concrete at point A of the toe.

a. Soil pressure at A (from geometry) is

$$q_A = 0.12 + \left(\frac{4.5}{5.25}\right)(2.36 - 0.12) = 2.04 \text{ ksf}$$

b. Let M_A be is calculated at A due to a rectangular stress and a triangular stress.

$$M_A = \frac{2.04(0.75)^2}{2} + (0.32 \times 0.75 \times 0.5)\left(0.75 \times \frac{2}{3}\right)$$
$$= 0.63 \text{ K} \cdot \text{ft}$$

c. The flexural stress in concrete is

$$\frac{Mc}{I} = \frac{0.63(12,000)(6)}{1728} = 26 \text{ psi}$$

where $c = h/2 = \frac{12}{2} = 6$ in. and $I = 12(12)^3/12 = 1728$ in.^{4}

d. The modulus of rupture of concrete is $7.5\lambda\sqrt{f_c'} = 410$ psi > 26 psi. The factor of safety against cracking is $410/26 = 16$. Therefore, the section is adequate. No other sections need to be checked.

Example 14.2

Design a cantilever retaining wall to support a bank of earth 16.5 ft high. The top of the earth is to be level with a surcharge of 330 psf. Given: The weight of the backfill is 110 pcf, the angle of internal friction is $\phi = 35°$, the coefficient of friction between concrete and soil is $\mu = 0.5$, the coefficient of friction between soil layers is $\mu = 0.7$, allowable soil bearing capacity is 4 ksf, $f_c' = 3$ ksi, and $f_y = 60$ ksi.

Solution

1. Determine the dimensions of the retaining wall using the approximate relationships shown in Fig. 14.8.

a. Height of wall: Allowing 3 ft for frost penetration to the bottom of the footing in front of the wall, the height of the wall becomes $h = 16.5 + 3 = 19.5$ ft.

b. Base thickness: Assume base thickness is $0.08h = 0.08 \times 19.5 = 1.56$ ft, or 1.5 ft. The height of the stem is $19.5 - 1.5 = 18$ ft.

c. Base length: The base length varies between $0.4h$ and $0.67h$. Assuming an average value of $0.53h$, the base length equals $0.53 \times 19.5 = 10.3$ ft, say, 10.5 ft. The projection of the base in front of the stem varies between $0.17h$ and $0.125h$. Assume a projection of $0.17h = 0.17 \times 19.5 = 3.3$ ft, say, 3.5 ft.

d. Stem thickness: The maximum stem thickness is at the bottom of the wall and varies between $0.08h$ and $0.1h$. Choose a maximum stem thickness equal to that of the base, or 1.5 ft. Select a practical minimum thickness of the stem at the top of the wall of 1.0 ft. The minimum batter of the face of the wall is $\frac{1}{4}$ in./ft. For an 18-ft-high wall, the minimum batter is $\frac{1}{4} \times 18 = 4.5$ in., which is less than the $1.5 - 1.0 = 0.5$ ft (6 in.) provided. The trial dimensions of the wall are shown in Fig. 14.11.

2. Using the Rankine equation:

$$C_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \frac{1 - 0.574}{1 + 0.574} = 0.271$$

3. The factor of safety against overturning can be determined as follows:

a. Calculate the actual unfactored forces acting on the retaining wall. First, find those acting to overturn the wall:

$$h_s(\text{due to surcharge}) = \frac{w_s}{w} = \frac{330}{110} = 3 \text{ ft}$$

$$p_1 = C_a w h_s = 0.271 \times (110 \times 3) = 90 \text{ psf}$$

$$p_2 = C_a w h = 0.271 \times (110 \times 19.5) = 581 \text{ psf}$$

$$H_{a_1} = 90 \times 19.5 = 1755 \text{ lb} \qquad \text{Arm} = \frac{19.5}{2} = 9.75 \text{ ft}$$

$$H_{a_2} = \tfrac{1}{2} \times 581 \times 19.5 = 5665 \text{ lb} \qquad \text{Arm} = \frac{19.5}{3} = 6.5 \text{ ft}$$

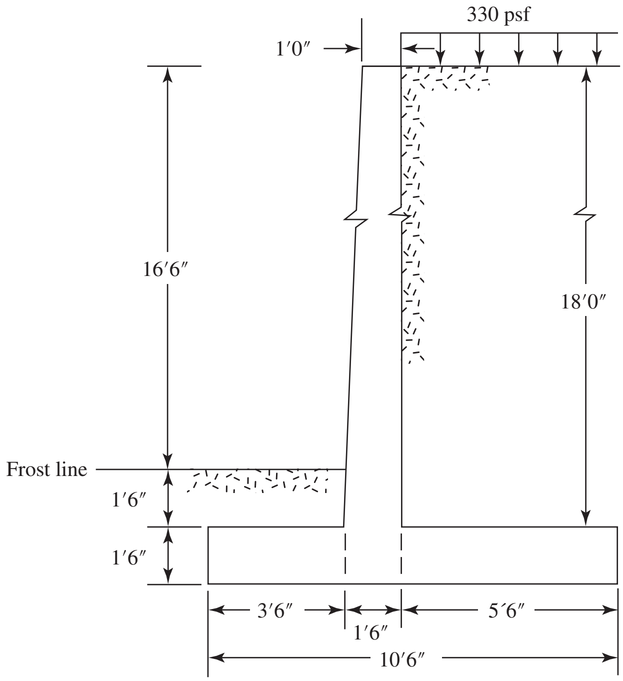


Figure 14.11 Example 14.2: Trial configuration of retaining wall.

b. The overturning moment is $1.755 \times 9.75 + 5.665 \times 6.5 = 53.93$ K·ft.

c. Calculate the balancing moment against overturning (see Fig. 14.12):

Force (lb)	Arm (ft)	Moment (K·ft)
$w_1 = 1 \times 18 \times 150 = 2{,}700$	4.50	12.15
$w_2 = \frac{1}{2} \times 18 \times \frac{1}{2} \times 150 = 675$	3.83	2.59
$w_3 = 10.5 \times 1.5 \times 150 = 2{,}363$	5.25	12.41
$w_4 = 5.5 \times 21 \times 110 = 12{,}705$	7.75	98.46

$$\sum w = R = 18.44 \text{ K} \qquad \sum M = 125.61 \text{ K} \cdot \text{ft}$$

$$\text{Factor of safety against overturning} = \frac{125.61}{53.93} = 2.33 > 2.0$$

4. Calculate the base soil pressure. Take moments about the toe end 0 (Fig. 14.12) to determine the location of the resultant R of the vertical forces.

$$x = \frac{\sum M - \sum Hy}{R} = \frac{\text{balancing } M - \text{overturning } M}{R}$$

$$= \frac{125.61 - 53.93}{18.44} = 3.89 \text{ ft} > \frac{10.5}{3} \quad \text{or} \quad 3.5 \text{ ft}$$

The eccentricity is $e = 10.5/2 - 3.89 = 1.36$ ft. The resultant R acts within the middle third of the base and has an eccentricity of $e = 1.36$ ft from the center of the base. For a 1-ft length of the

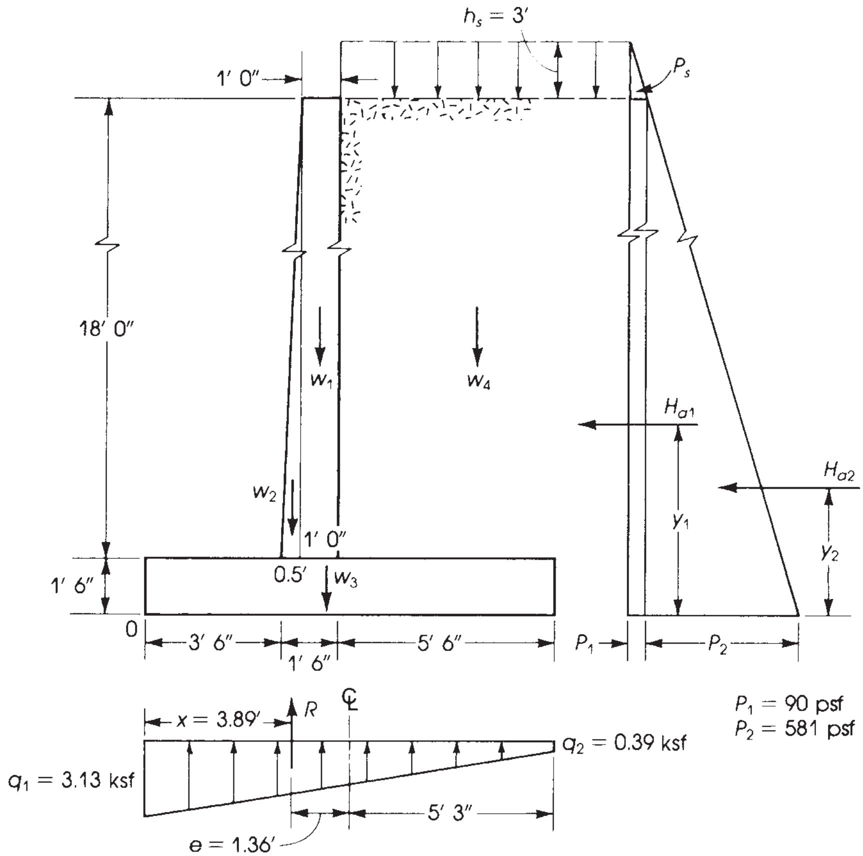


Figure 14.12 Example 14.2: Forces acting on retaining wall.

footing, area = 10.5 × 1 = 10.5 ft^2.

$$I = 1 \times \frac{(10.5)^3}{12} = 96.47\,\text{ft}^4$$

$$q_1 = \frac{R}{A} + \frac{(Re)C}{I} = \frac{18.44}{10.5} + \frac{(18.44 \times 1.36) \times 5.25}{96.47}$$

$$= 1.76 + 1.37 = 3.13\,\text{ksf} < 4\,\text{ksf}$$

$$q_2 = 1.76 - 1.37 = 0.39\,\text{ksf}$$

Soil pressure is adequate.

5. Calculate the factor of safety against sliding. A minimum factor of safety of 1.5 must be maintained.

$$\text{Force causing sliding} = H_{a1} + H_{a2} = 1.76 + 5.67 = 7.43\,\text{K}$$

$$\text{Resisting force} = \mu R = 0.5 \times 18.44 = 9.22\,\text{K}$$

$$\text{Factor of safety against sliding} = \frac{9.22}{7.43} = 1.24 < 1.5$$

The resistance provided does not give an adequate safety against sliding. In this case, a key should be provided to develop a passive pressure large enough to resist the excess force that causes

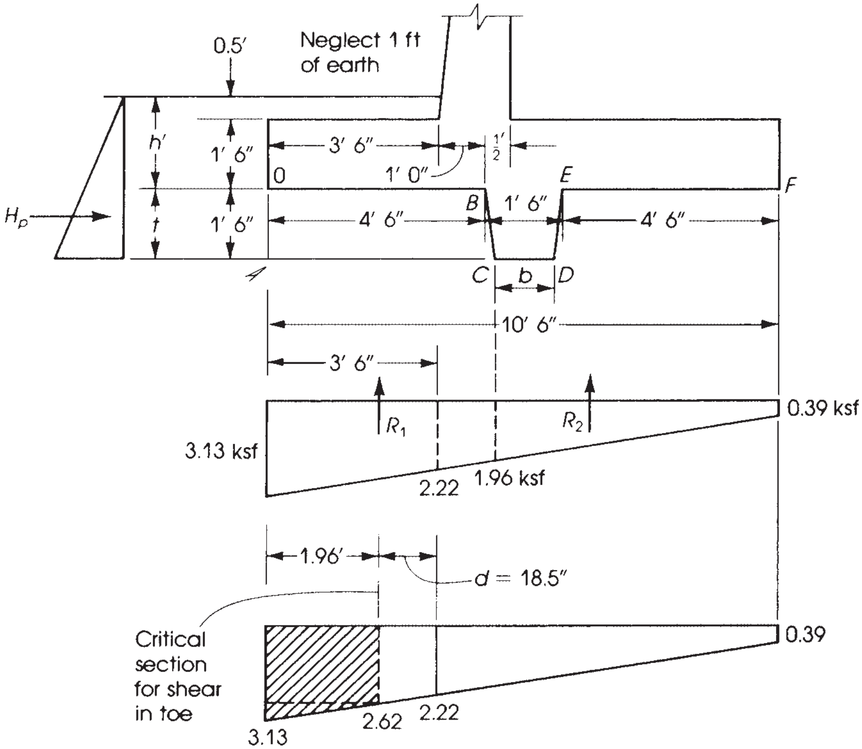


Figure 14.13 Example 14.3: Footing details.

sliding. Another function of the key is to provide sufficient development length for the dowels of the stem. The key is therefore placed such that its face is about 6 in. from the back face of the stem (Fig. 14.13). In the calculation of the passive pressure, the top foot of the earth at the toe side is usually neglected, leaving a height of 2 ft in this example. Assume a key depth of $t = 1.5$ ft and a width of $b = 1.5$ ft.

$$C_p = \frac{1+\sin\phi}{1-\sin\phi} = \frac{1}{C_a} = \frac{1}{0.271} = 3.69$$

$$H_p = \frac{1}{2}C_p w(h'+t)^2 = \frac{1}{2}\times 3.69\times 110(2+1.5)^2 = 2486\ \text{lb}$$

The sliding may occur now on the surfaces *AC, CD*, and *EF* (Fig. 14.13). The sliding surface *AC* lies within the soil layers with a coefficient of internal friction = tan ϕ = tan 35° = 0.7, whereas the surfaces *CD* and *EF* are those between concrete and soil with a coefficient of internal friction of 0.5, as given in this example. The frictional resistance is $F = \mu_1 R_1 + \mu_2 R_2$.

$$R_1 = \text{reaction of } AC = \left(\frac{3.13+1.96}{2}\right)\times 4.5 = 11.44\ \text{K}$$

$$R_2 = R - R_1 = 18.44 - 11.44 = 7.0\ \text{K}$$

$$= \text{reaction of } CDF = \left(\frac{1.96+0.39}{2}\right)\times 6 = 7.05\ \text{K}$$

$$F = 0.7(11.44) + 0.5(7.00) = 11.50\ \text{K}$$

The total resisting force is

$$F + H_p = 11.50 + 2.49 = 13.99\ \text{K}$$

The factor of safety against sliding is

$$\frac{13.99}{7.43} = 1.9 \quad \text{or} \quad \frac{11.5}{7.43} = 1.55 > 1.5$$

The factor is greater than 1.5, which is recommended when passive resistance against sliding is not included.

6. Design the wall (stem).

a. Main reinforcement: The lateral forces applied to the wall are calculated using a load factor of 1.6. The critical section for bending moment is at the bottom of the wall, height = 18 ft. Calculate the applied maximum forces:

$$P_1 = 1.6(C_a w h_s) = 1.6(0.271 \times 110 \times 3) = 143\ \text{lb}$$

$$P_2 = 1.6(C_a w h) = 1.6(0.271 \times 110 \times 18) = 858.3\ \text{lb}$$

$$H_{a1} = 0.143 \times 18 = 2.57\ \text{K} \qquad \text{Arm} = \frac{18}{2} = 9\ \text{ft}$$

$$H_{a2} = \frac{1}{2} \times 0.858 \times 18 = 7.72\ \text{K} \quad \text{Arm} = \frac{18}{3} = 6\ \text{ft}$$

$$M_u(\text{at bottom of wall}) = 2.57 \times 9 + 7.72 \times 6 = 69.45\ \text{K} \cdot \text{ft}$$

The total depth used is 18 in., $b = 12$ in., and $d = 18 - 2$ (concrete cover) -0.5 (half the bar diameter) = 15.5 in.

$$R_u = \frac{M_u}{bd^2} = \frac{69.45 \times 12,000}{12(15.5)^2} = 289\ \text{psi}$$

The steel ratio, ρ, can be obtained from Table 1 in Appendix A or from

$$\rho = \frac{0.85 f'_c}{f_y}\left[1 - \sqrt{\frac{2R_u}{\phi 0.85 f'_c}}\right] = 0.007$$

$$A_s = 0.007(12)(15.5) = 1.3\ \text{in.}^2$$

Use no. 8 bars spaced at 7 in. (1.35 in.^{2}). The minimum vertical A_s according to the ACI Code, Section 11.6, is

$$A_{s,\min} = 0.0015(12)(18) = 0.32\ \text{in.}^2 < 1.35\ \text{in.}^2$$

Because the moment decreases along the height of the wall, A_s may be reduced according to the moment requirements. It is practical to use one A_s or spacing, for the lower half and a second A_s, or spacing, for the upper half of the wall. To calculate the moment at midheight of the wall, 9 ft from the top:

$$P_1 = 1.6(0.271 \times 110 \times 3) = 143\ \text{lb}$$

$$P_2 = 1.6(0.271 \times 110 \times 9) = 429\ \text{lb}$$

$$H_{a_1} = 0.143 \times 9 = 1.29\ \text{K} \qquad \text{Arm} = \frac{9}{2} = 4.5\ \text{ft}$$

$$H_{a_2} = \frac{1}{2} \times 0.429 \times 9 = 1.9\ \text{K} \quad \text{Arm} = \frac{9}{3} = 3\ \text{ft}$$

$$M_u = 1.29 \times 4.5 + 1.9 \times 3 = 11.5\ \text{K} \cdot \text{ft}$$

The total depth at midheight of wall is

$$\frac{12+18}{2} = 15 \text{ in.}$$

$$d = 15 - 2 - 0.5 = 12.5 \text{ in.}$$

$$R_u = \frac{M_u}{bd^2} = \frac{11.5 \times 12,000}{12 \times (12.5)^2} = 73.6 \text{ psi}$$

$$\rho = 0.0017 \quad \text{and} \quad A_s = 0.0017(12)(12.5) = 0.25 \text{ in.}^2$$

$$A_{s,\min} = 0.0015 \times 12 \times 15 = 0.27 \text{ in.}^2 > 0.25 \text{ in.}^2$$

Use no. 4 vertical bars spaced at 8 in. (0.29 in.^{2}) with similar spacing to the lower vertical steel bars in the wall.

b. Temperature and shrinkage reinforcement: The minimum horizontal reinforcement at the base of the wall according to ACI Code, Section 11.6, is

$$A_{s,\min} = 0.0020 \times 12 \times 18 = 0.432 \text{ in.}^2$$

(for the bottom third), assuming no. 5 bars or smaller.

$$A_{s,\min} = 0.0020 \times 12 \times 15 = 0.36 \text{ in.}^2$$

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

$$0.5A_s = 0.5 \times 0.432 = 0.22 \text{ in.}^2$$

Use no. 4 horizontal bars spaced at 8 in. ($A_s = 0.29$ in.^{2}) at both the internal and external surfaces of the wall. Use no. 4 vertical bars spaced at 12 in. at the front face of the wall to support the horizontal temperature and shrinkage reinforcement.

c. Dowels for the wall vertical bars: The anchorage length of no. 8 bars into the footing must be at least 22 in. Use an embedment length of 2 ft into the footing and the key below the stem.

d. Design for shear: The critical section for shear is at a distance $d = 15.5$ in. from the bottom of the stem. At this section, the distance from the top equals $18 - 15.5/12 = 16.7$ ft.

$$P_1 = 1.6(0.271 \times 110 \times 3) = 143 \text{ lb}$$

$$P_2 = 1.6(0.271 \times 110 \times 16.7) = 796 \text{ lb}$$

$$H_{a_1} = 0.143 \times 16.7 = 2.39 \text{ K}$$

$$H_{a_2} = \frac{1}{2} \times 0.796 \times 16.7 = 6.6 \text{ K}$$

$$\text{Total } H = 2.39 + 6.6 = 9.0 \text{ K}$$

$$\phi V_c = \phi(2\lambda\sqrt{f'_c})bd = \frac{0.75 \times 2 \times 1}{1000} \times \sqrt{3000} \times 12 \times 15.5$$

$$= 15.28 \text{ K} > 9.0\text{K}$$

7. Design of the heel: A load factor of 1.2 is used to calculate the factored bending moment and shearing force due to the backfill and concrete, whereas a load factor of 1.6 is used for the surcharge. The upward soil pressure is neglected because it will reduce the effect of the backfill and concrete on the heel. Referring to Fig. 14.12, the total load on the heel is

$$V_u = \frac{1.2[(18 \times 5.5 \times 110) + (1.5 \times 5.5 \times 150)] + 1.6(3 \times 5.5 \times 100)}{1000}$$

$$= 17.5\ \mathrm{K}$$

$$M_u(\text{at face of wall}) = V_u \times \frac{5.5}{2} = 48.1\ \mathrm{K \cdot ft}$$

The critical section for shear is usually at a distance d from the face of the wall when the reaction introduces compression into the end region of the member. In this case, the critical section will be considered at the face of the wall because tension and not compression develops in the concrete.

$$V_u = 17.2\ \mathrm{K}$$

$$\phi V_c = \phi(2\lambda\sqrt{f_c'})bd = \frac{0.75 \times 2 \times 1}{1000} \times \sqrt{3000} \times 12 \times 14.5$$

$$= 14.3\ \mathrm{K}$$

where ϕV_c is less than V_u of 17.2 K, and the section must be increased by the ratio 17.5/14.3 or shear reinforcement must be provided.

$$\text{Required } d = \frac{17.2}{14.3} \times 14.5 = 17.4\ \text{in.}$$

$$\text{Total thickness required} = 17.4 + 3.5 = 20.9\ \text{in.}$$

Use a base thickness of 22 in. and $d = 18.5$ in.

$$R_u = \frac{M_u}{bd^2} = \frac{48.1 \times 12{,}000}{12 \times (18.5)^2} = 140.5\ \text{psi} \quad \rho = 0.0027$$

$$A_s = \rho bd = 0.60\ \text{in}^2$$

$$\text{Min. shrinkage } A_s = 0.0018(12)(22) = 0.475\ \text{in.}^2$$

$$\text{Min. flexural } A_s = 0.0033(12)(18.5) = 0.733\ \text{in.}^2$$

Use no. 6 bars spaced at 7 in. ($A_s = 0.76$ in.^{2}). The development length for the no. 6 top bars equals 1.4 l_d = 35 in. Therefore, the bars must be extended 3 ft into the toe of the base.

Temperature and shrinkage reinforcement in the longitudinal direction is not needed in the heel or toe, but it may be preferable to use minimal amounts of reinforcement in that direction, say, no. 4 bars spaced at 12 in.

8. Design of the toe: The toe of the base acts as a cantilever beam subjected to upward pressures, as calculated in step 4. The factored soil pressure is obtained by multiplying the service load soil pressure by a load factor of 1.6 because it is primarily caused by the lateral forces. The critical section for the bending moment is at the front face of the stem. The critical section for shear is at a distance d from the front face of the stem because the reaction in the direction of shear introduces compression into the toe.

Referring to Fig. 14.13, the toe is subjected to an upward pressure from the soil and downward pressure due to self-weight of the toe slab:

$$V_u = 1.6\left(\frac{3.13 + 2.62}{2}\right) \times 1.96 - 1.2\left(\frac{22}{12} \times 0.150\right) \times 1.96$$

$$= 837\ \mathrm{K}$$

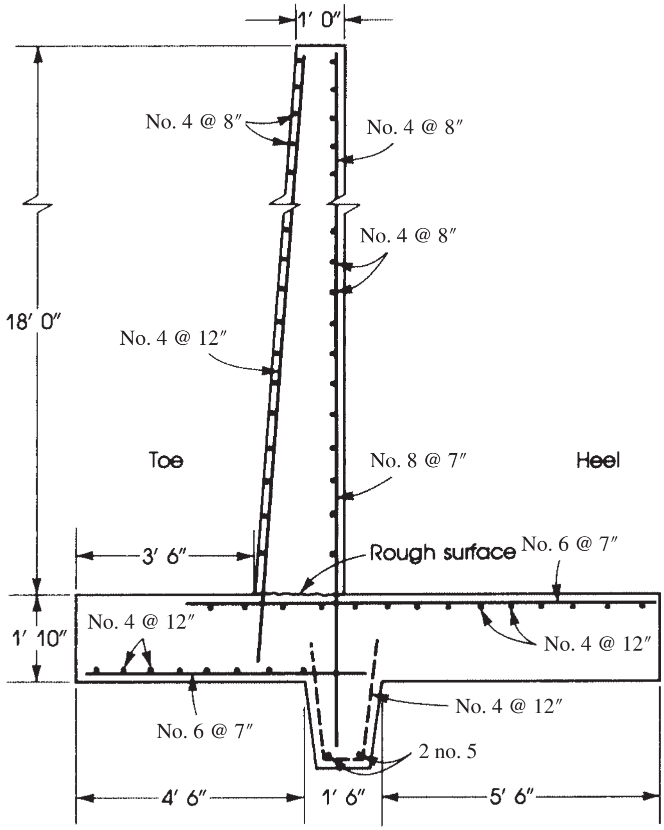


Figure 14.14 Example 14.2: Reinforcement details.

This is less than ϕV_c of 14.3 K calculated for the heel in step 7.

$$M_u = 1.6\left[\frac{2.22}{2} \times (3.5)^2 + (3.13 - 2.22) \times 3.5 \times 0.5\left(\frac{2}{3} \times 3.5\right)\right]$$

$$- 1.2\left[\left(\frac{22}{12} \times 0.150\right) \times \frac{(3.5)^2}{2}\right] = 25.7\ \text{K} \cdot \text{ft}$$

$$R_u = \frac{M_u}{bd^2} = \frac{25.7 \times 12,000}{12 \times (18.5)^2} = 75\ \text{psi} \quad \rho = 0.0017$$

$$A_s = 0.0017(12)(18.5) = 0.377\ \text{in.}^2$$

$$\text{Min. shrinkage } A_s = 0.0018(12)(22) = 0.475\ \text{in.}^2$$

$$\text{Min. flexural } A_s = 0.0033(12)(18.5) = 0.733\ \text{in.}^2$$

Use no. 6 bars spaced at 7 in., similar to the heel reinforcement. Development length of no. 6 bars equals 25 in. Extend the bars into the heel 25 in. The final reinforcement details are shown in Fig. 14.14.

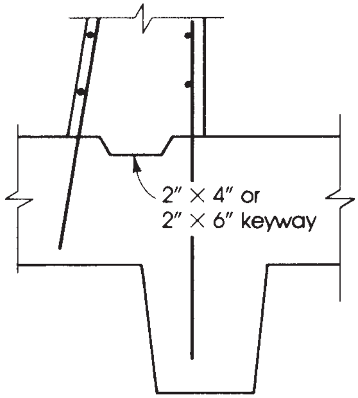


Figure 14.15 Example 14.2: Keyway details.

9. Shear keyway between wall and footing: In the construction of retaining walls, the footing is cast first and then the wall is cast on top of the footing at a later date. A construction joint is used at the base of the wall. The joint surface takes the form of a keyway, as shown in Fig. 14.15, or is left in a very rough condition (Fig. 14.14). The joint must be capable of transmitting the stem shear into the footing.
10. Proper drainage of the backfill is essential in this design because the earth pressure used is for drained backfill. Weep holes should be provided in the wall, 4 in. in diameter and spaced at 5 ft in the horizontal and vertical directions.

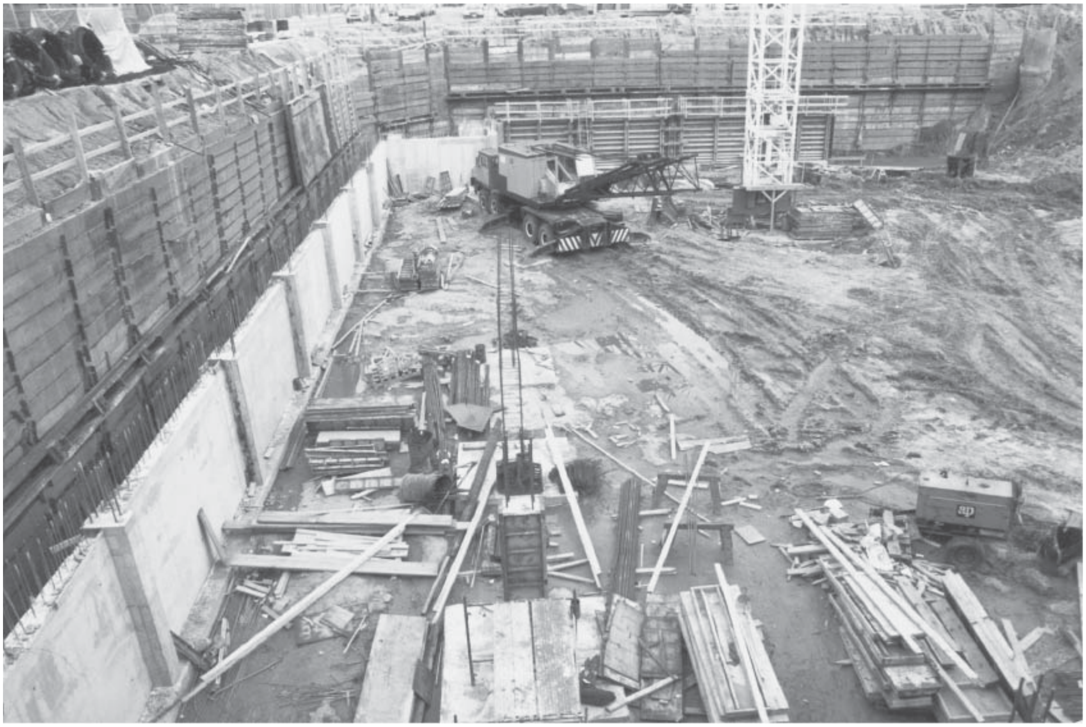
14.11 BASEMENT WALLS

It is a common practice to assume that basement walls span vertically between the basement-floor slab and the first-floor slab. Two possible cases of design should be investigated for a basement wall.

First, when the wall only has been built on top of the basement floor slab, the wall will be subjected to lateral earth pressure with no vertical loads except its own weight. The wall in this case acts as a cantilever, and adequate reinforcement should be provided for a cantilever wall design. This case can be avoided by installing the basement and the first-floor slabs before backfilling against the wall.

Second, when the first-floor and the other floor slabs have been constructed and the building is fully loaded, the wall in this case will be designed as a propped cantilever wall subjected to earth pressure and to vertical load.

In addition to drainage, a waterproofing or damp-proofing membrane must be laid or applied to the external face of the wall. The ACI Code, Section 11.3.1.1, specifies that the minimum thickness of an exterior basement wall and its foundation is 7.5 in. In general, the minimum thickness of bearing walls is $\frac{1}{25}$ of the supported height or length, whichever is shorter, or 4 in.



Basement wall.

Example 14.3

Determine the thickness and necessary reinforcement for the basement retaining wall shown in Fig. 14.16. Given: Weight of backfill = 110 pcf, angle of internal friction = 35°, $f_c' = 3$ ksi, and fy = 60 ksi.

Solution

1. The wall spans vertically and will be considered as fixed at the bottom end and propped at the top. Consider a span of L = 9.75 ft, (10 – 3/12 = 9.75 ft) as shown in Fig. 14.16.
2. For these data, the different lateral pressures on a 1-ft length of the wall are as follows:

   For an angle of internal friction of 35°, the coefficient of active pressure is $C_a = 0.271$. The horizontal earth pressure at the base is $P_a = C_a wh$. For $w = 110$ pcf and an basement height of $h = 10$ ft, then

$$P_a = 0.271 \times 0.110 \times 10 = 0.3 \text{ ksf}$$

$$H_a = 0.271 \times 0.110 \times \frac{100}{2} = 1.49 \text{ K/ft of wall}$$

   H_a acts at $h/3 = \underline{10/3} = 3.33$ ft from the base. An additional pressure must be added to allow for a surcharge of about 200 psf on the ground behind the wall. The equivalent height to the surcharge is

$$h_s = \frac{200}{110} = 1.82 \text{ ft}$$

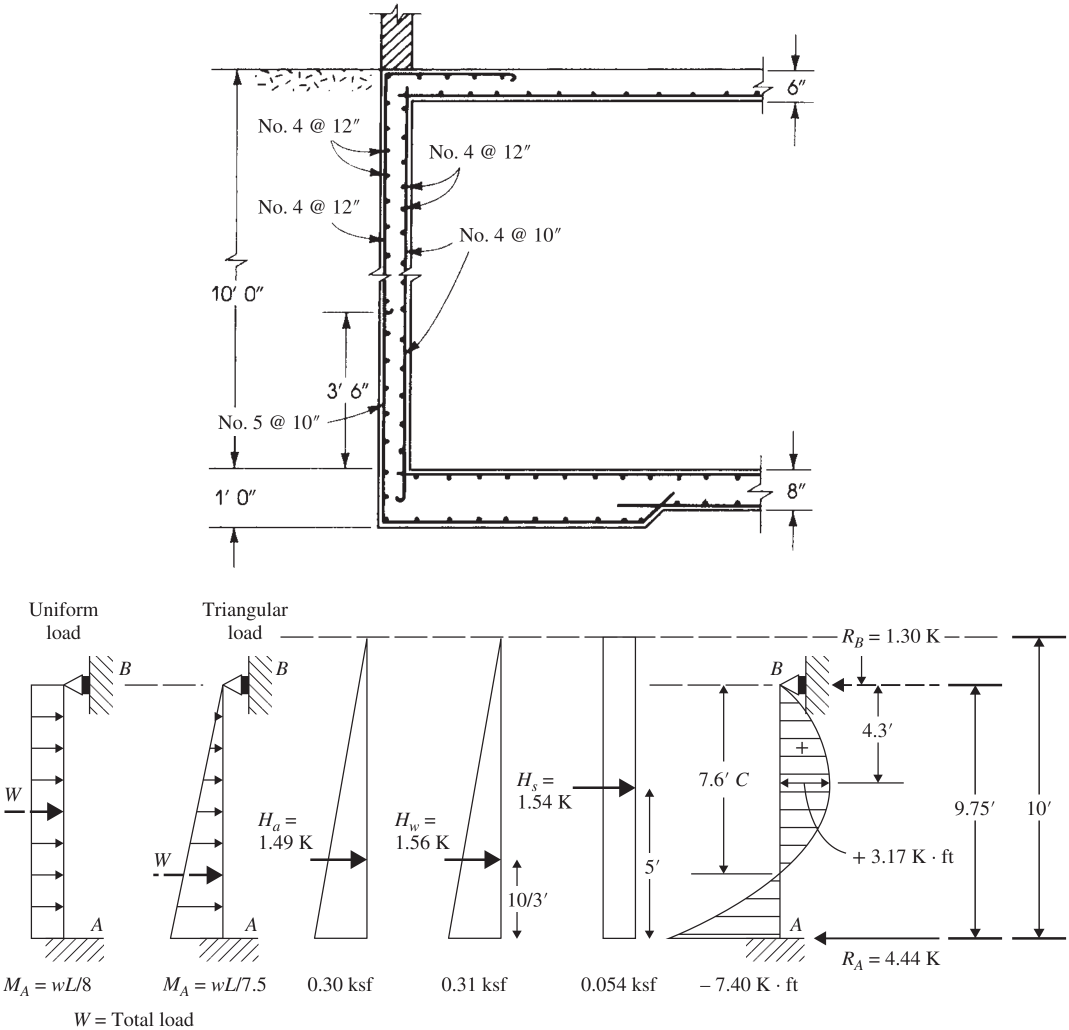


Figure 14.16 Example 14.3: Basement wall.

$$P_s = C_a w h_s = 0.271 \times 0.110 \times 1.82 = 0.054\ \text{ksf}$$

$$H_s = C_a w h_s \times h = 0.054 \times 10 = 0.54\ \text{K/ft of wall}$$

H_s of the surcharge acts at $h/2 = 5$ ft from the base

In the preceding calculations, it is assumed that the backfill is dry, but it is necessary to investigate the presence of water pressure behind the wall. The maximum water pressure occurs when the whole height of the basement wall is subjected to water pressure, and

$$P_w = wh = 62.5 \times 10 = 625\ \text{psf}$$

$$H_w = \frac{wh^2}{2} = \frac{0.625 \times 10^2}{2} = 3.125\ \text{K/ft of wall}$$

The maximum pressure may not be present continuously behind the wall. Therefore, if the ground is intermittently wet, a percentage of the preceding pressure may be adopted, say, 50%:

In summary: Due to active soil pressure, $P_a = 0.30$ ksf, $H_a = 1.49$ K. Due to water pressure, $P_w = 0.31$ ksf, $H_w = 1.56$ K. Due to surcharge, $P_s = 0.054$ ksf, $H_s = 0.54$ K. H_a and H_w are due to triangular loadings, whereas H_S is due to uniform loading. 10 ft are used to calculate H_a, H_w and H_S as the applied pressure over entire wall without the base. Pressure calculation is shown above.

3. Referring to Fig. 14.16 and using moment coefficients of a propped beam subjected to triangular and uniform loads, and a load factor = 1.6, the maximum moment M_u at A can be calculated from:

$$M_u = 1.6(H_a + H_w)\frac{L}{7.5} + 1.6H_s\frac{L}{8}$$

$$= 1.6\left(\frac{(1.49 + 1.56)}{7.5} \times 9.75 + 0.54 \times \frac{9.75}{8}\right) = 7.40\ \text{K} \cdot \text{ft}$$

$$R_B(9.75) = 1.6\left(\frac{(1.49 + 1.56)(9.75)}{3} + \frac{0.54(9.75)}{2}\right) - 7.40$$

$$R_B = 1.30\ \text{K}$$

$$R_A = 1.6(1.49 + 1.56 + 0.54) - 1.30 = 4.44\ \text{K}$$

Maximum positive bending moment within the span occurs at the section of 0 shear. Assume x from the top roof of the basement wall:

$$V_u = 1.3 - 1.6(0.054x) - 1.6\left((0.30 + 0.31)\frac{x^2}{2}\right) = 0$$

$$x = 4.3\ \text{ft}$$

$$M_c = 1.3 \times 4.3 - 1.6\left[\frac{0.054}{2}(10 - 9.75 + 4.3)^2 + \frac{[(0.31 + 0.30)(10 - 9.75 + 4.3)/10]}{2}\frac{(10 - 9.75 + 4.3)^2}{3}\right]$$

$$= +3.17\ \text{K} \cdot \text{ft}$$

4. Assuming 0.01 steel ratio and one feet strip, Ru = 332 psi,

$$d = \sqrt{\frac{M_u}{R_u b}} = \sqrt{\frac{7.40 \times 12}{0.332 \times 12}} = 4.72\ \text{in.}$$

Total depth = 4.72 + 1.5 (concrete cover) + 0.25 = 6.47 in. Use a $7\frac{1}{2}$ in. slab; $d = 5.75$ in.

$$R_u = \frac{M_u}{bd^2} = \frac{7.40 \times 12{,}000}{12 \times (5.75)^2} = 224\ \text{psi}$$

The steel ratio is $\rho = 0.005$ and $A_s = 0.005 \times 12 \times 5.75 = 0.345$ in.^{2}

$$\text{Minimum } A_s = 0.0015bh = 0.0015(12)(7.5) = 0.135\ \text{in.}^2 \quad \text{(vertical bars)}$$

$$\text{Minimum } A_s(\text{flexure}) = 0.0033(12)(5.75) = 0.23\ \text{in.}^2$$

Use no. 5 bars spaced at 10 in. (As = 0.37 in.^{2}).

5. For the positive moment, Mc = +3.17 K·ft:

$$R_u = \frac{3.17 \times 12{,}000}{12 \times (5.75)^2} = 96\ \text{psi} \quad \rho = 0.0018$$

$$A_s = 0.0018 \times 12 \times 5.75 = 0.125\ \text{in.}^2 < 0.23\ \text{in.}^2$$

Use no. 4 bars spaced at 10 in. (As = 0.24 in^2).

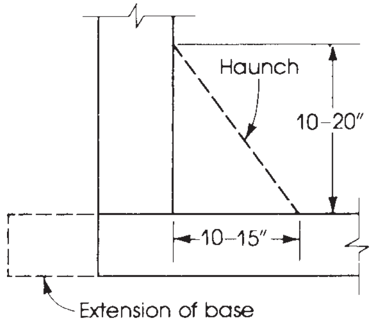


Figure 14.17 Example 14.3: Adjustment of wall base.

6. Zero moment occurs at a distance of 7.6 ft from the top and 2.15 ft from the base. The development length of no. 5 bars is 14 in. Therefore, extend the main no. 5 bars to a distance of 2.15 + 1.2 = 3.35 ft, or 3.5 ft, above the base; then use no. 4 bars spaced at 12 in. at the exterior face. For the interior face, use no. 4 bars spaced at 10 in. throughout.
7. Longitudinal reinforcement: Use a minimum steel ratio of 0.0020 (ACI Code, Section 11.6), or As $= 0.0020 \times 7 \times 12 = 0.17\ \text{in}^2$. Use no. 4 bars spaced at 12 in. on each side of the wall.
8. If the bending moment at the base of the wall is quite high, it may require a thick wall slab, for example, 12 in. or more. In this case a haunch may be adopted, as shown in Fig. 14.17. This solution will reduce the thickness of the wall because it will be designed for the moment at the section exactly above the haunch.
9. The basement slab may have a thickness greater than the wall thickness and may be extended outside the wall by about 10 in. or more, as required.

SUMMARY

Sections 14.1–14.3

1. A retaining wall maintains unequal levels of earth on its two faces. The most common types of retaining walls are gravity, semigravity, cantilever, counterfort, buttressed, and basement walls.
2. For a linear pressure, the active and passive pressure intensities are

$$P_a = C_a wh \quad \text{and} \quad P_p = C_p wh$$

According to Rankine's theory,

$$C_a = \frac{1 - \sin\phi}{1 + \sin\phi} \quad \text{and} \quad C_p = \frac{1 + \sin\phi \sin\phi}{1 - \sin\phi \sin\phi}$$

Values of C_a and C_p for different values of ϕ and δ are given in Tables 14.2 and 14.3.

Sections 14.4 and 14.5

1. When soil is saturated, the submerged unit weight must be used to calculate earth pressure. The hydrostatic water pressure must also be considered.
2. A uniform surcharge on a retaining wall causes an additional pressure height, $h_s = w_s/w$.

Sections 14.6–14.8

1. A total frictional force, F, to resist sliding effect is

$$F = \mu R + H_p \qquad \text{(Eq. 14.13)}$$

$$\text{Factor of safety against sliding} = \frac{F}{H_{\text{ah}}} \geq 1.5 \qquad \text{(Eq. 14.14)}$$

2. Factor of safety against overturning is

$$\frac{M_b}{M_0} = \frac{\sum wx}{H_a h/3} \geq 2.0$$

3. Approximate dimensions of a cantilever retaining wall are shown in Fig. 14.8.

Sections 14.9 and 14.10

1. Minimum reinforcement is needed in retaining walls.
2. To avoid hydrostatic pressure on a retaining wall, a drainage system should be used that consists of weep holes, perforated pipe, or any other adequate device.
3. Basement walls in buildings may be designed as propped cantilever walls subjected to earth pressure and vertical loads. This case occurs only if the first-floor slab has been constructed. A surcharge of 200 psf may be adopted.

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PROBLEMS

14.1 Check the adequacy of the retaining wall shown in Fig. 14.18 with regard to overturning, sliding, and the allowable soil pressure. Given: Weight of backfill = 110 pcf, the angle of internal friction is $\phi = 30°$, the coefficient of friction between concrete and soil is $\mu = 0.5$, allowable soil pressure = 3.5 ksf, and $f_c' = 3$ ksi.

14.2 Repeat Problem 14.1 with Fig. 14.19.

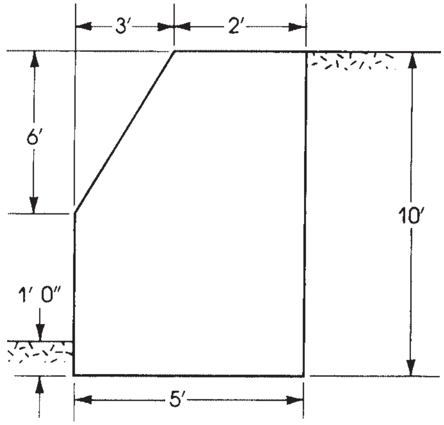

Figure 14.18 Problem 14.1: Gravity wall.

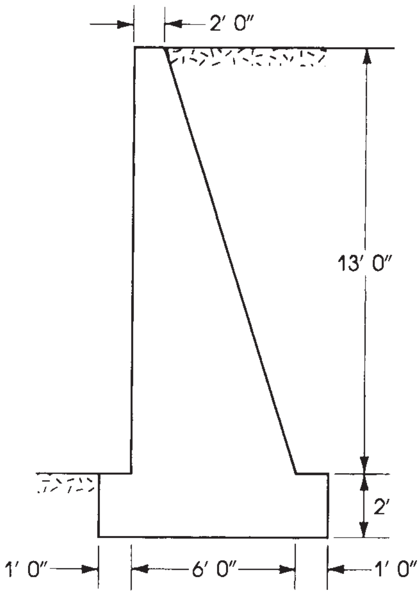

Figure 14.19 Problem 14.2: Semigravity wall.

14.3 For each problem in Table 14.4, determine the factor of safety against overturning and sliding. Also, determine the soil pressure under the wall footing and check if all calculated values are adequate (equal or below the allowable values). Given: Weight of soil = 110 pcf, weight of concrete = 150 pcf, and the coefficient of friction between concrete and soil is 0.5 and between soil layers is 0.6. Consider that the allowable soil pressure of 4 ksf and the top of the backfill is level without surcharge. Ignore the passive soil resistance. See Fig. 14.20 ($\phi = 35°$).

14.4 Repeat Problems 14.3e–h, assuming a surcharge of 300 psf.

Table 14.4 Problem

Problem No.	H	h_f	A	B	C	L
a	12	1.00	2.0	1.0	4.0	7
b	14	1.50	2.0	1.5	4.5	8
c	15	1.50	2.0	1.5	4.5	8
d	16	1.50	3.0	1.5	4.5	9
e	17	1.50	3.0	1.5	4.5	9
f	18	1.75	3.0	1.75	5.25	10
g	19	1.75	3.0	1.75	5.25	10
h	20	2.00	3.0	2.0	6.0	11
i	21	2.00	3.5	2.0	6.5	12
j	22	2.00	3.5	2.0	6.5	12

Refer to Fig. 14.20. All dimensions are in feet.

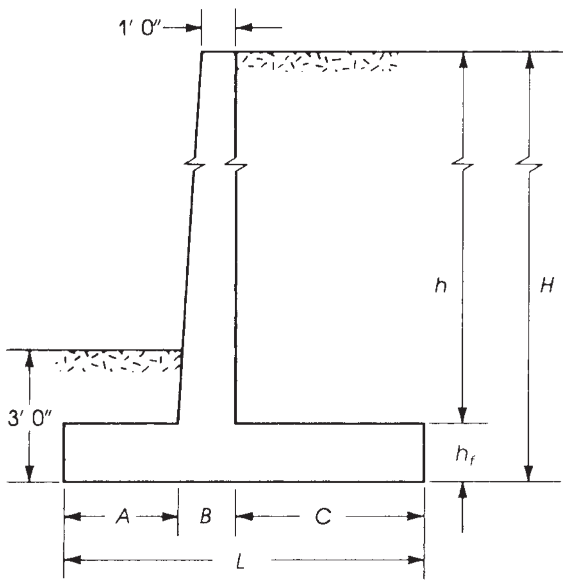

Figure 14.20 Problem 14.3.

14.5 Repeat Problems 14.3e–h, assuming that the backfill slopes at 10° to the horizontal. (Add key if needed.)

14.6 For Problems 14.3e–h, determine the reinforcement required for the stem, heel, and toe, and choose adequate bars and distribution. Use $f_c' = 3$ ksi and $f_y = 60$ ksi.

14.7 Determine the dimensions of a cantilever retaining wall to support a bank of earth 16 ft high. Assume that frost penetration depth is 4 ft. Check the safety of the retaining wall against overturning and sliding only. Given: Weight of backfill = 120 pcf, angle of internal friction = 33°, coefficient of friction between concrete and soil = 0.45, coefficient of friction between soil layers = 0.65, and allowable soil pressure = 4 ksf. Use a 1.5 × 1.5-ft key if needed.

14.8 A complete design is required for the retaining wall shown in Fig. 14.21. The top of the backfill is to be level without surcharge. Given: Weight of backfill soil = 110 pcf, angle of internal friction = 35°, the coefficient of friction between concrete and soil is 0.55 and that between soil layers is 0.6. Use $f'_c = 3$ ksi, and $f_y = 60$ ksi, and an allowable soil pressure of 4 ksf.

14.9 Check the adequacy of the cantilever retaining wall shown in Fig. 14.22 for both sliding and overturning conditions. Use a key of 1.5×1.5 ft if needed. Then determine reinforcement needed for the stem, heel, and toe, and choose adequate bars and distribution. Given: Weight of soil = 120 pcf, the angle of internal

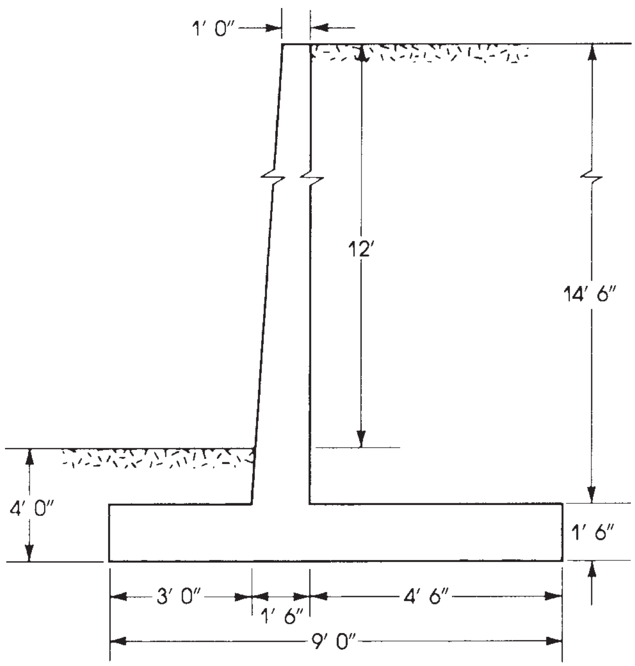

Figure 14.21 Problem 14.8: Cantilever retaining wall.

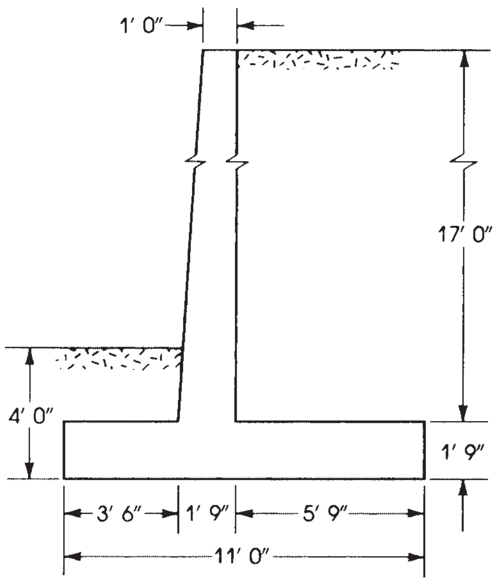

Figure 14.22 Problem 14.9: Cantilever retaining wall.

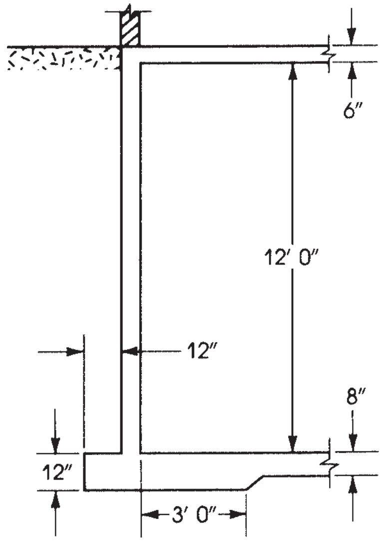

Figure 14.23 Problem 14.11: Basement wall.

friction is $\phi = 35°$, the coefficient of friction between concrete and soil is 0.52 and that between soil layers is 0.70. Use $f_c' = 3$ ksi, $f_y = 60$ ksi, an allowable soil pressure of 4 ksf, and a surcharge of 300 psf.

14.10 Repeat Problem 14.9, assuming the backfill slopes at 30° to the horizontal.

14.11 Determine the thickness and necessary reinforcement for the basement wall shown in Fig. 14.23. The weight of backfill is 120 pcf and the angle of internal friction is $\phi = 30°$. Assume a surcharge of 400 psf and use $f_c' = 3$ ksi and $f_y = 60$ ksi.

14.12 Repeat Problem 14.11 using a basement clear height of 14 ft.