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AN INTRODUCTION TO THE DESIGN OF RETAINING WALLS

by

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I. INTRODUCTION

1. Statement of the Study

Retaining walls are encountered and constructed in various fields of engineering such as civil, hydraulic structure, highway, railway, tunnel, mining, and military fortification. One of the more important steps in the design of such walls consists of analysis of earth pressure. Initial theories of retaining walls were formulated by Coulomb (1) in 1776, Poncelet (2) in 1840, and Rankine (3) in 1857, and these theories have been extended by other investigators. Developments since 1920, largely due to the influence of Dr. Karl Terzaghi (4), have led to a better understanding of the applications of the classical earth pressure theories to soil mechanics. The modern applications of these theories are given in current texts on soil mechanics (5, 6, 7, 8).

The backfill behind the retaining wall produces a lateral pressure on the wall. The determination of this force is the chief problem in the analysis and design of a retaining wall. The value of the thrust acting on a retaining wall is variable and depends mainly on the properties and moisture content of the backfill. Thus, the backfill and drainage system plays a very important role in the design of retaining walls.

2. Purpose and Scope of the Study

The purpose of this report is to review all the pertinent literature and to systematically derive some known formulae for the calculation of earth pressure with different soils under various loading conditions.

The heart of this report consists of careful and adequate review of important literature on earth pressure theories, stability, the backfill and drainage systems of retaining wall, and typical examples of designing retaining walls with given soil data.

II. REVIEW OF LITERATURE

An engineering structure designed to maintain a difference in the elevations of the ground surfaces is called a retaining wall. The earth whose ground surface is at the higher elevation is commonly called the backfill, and the wall is said to retain this backfill.

A. Principal Types of Retaining Walls and Forces Acting on the Retaining Wall

1. Principal Types of Retaining Walls

Retaining walls may be classified, based on the method of achieving stability, into six principal types: (5, 6, 7)

1). Gravity wall

The stability of a gravity wall depends entirely upon the weight of the stone or concrete masonry and any soil resting on the masonry. No reinforcement is provided except in concrete walls, where a nominal amount of steel may be placed near the exposed face to prevent surface cracking due to temperature changes. It is considered suitable for low walls which are less than about 20 ft.

2). Semigravity walls

A modification of the gravity wall is made by a very wide heel in the back of the wall, it is some what more slender than a gravity wall and the soil resting on the heel contributes a significent weight to the mass of the wall.

3). Cantilever wall

This structure consists of a concrete stem and base, both relatively thin and heavily reinforced to resist the moments and shears to which they are subjected. It is usually more economical for small to moderate height (about 20-25 ft) and is more widely used than other types.

4). Counterfort wall

This type of wall consists of a thin concrete face slab, usually vertical, supported at intervals on the backfill side by vertical ribs or counterforts that meet the face slab and base at right angles. The face slab, counterforts and base slab between the counterforts is backfilled with soil. All the members are fully reinforced. It is suitable for high retaining walls.

5). Buttressed wall

A buttressed wall resembles a counterfort wall. but the vertical cantilever members supporting the face slab are placed on the front of the wall instead of the back, and are called buttresses rather than counterforts. Because of the smaller heel projection, the backfill contributes less to the stability of butressed walls than it does to the stability of counterfort walls. The buttresses are exposed, and reduce the clearence in front of the wall. For these reasons, such walls are rarely used except where space does not permit the contribution of counterforts.

6). Crib wall

A crib type wall consists of individual structural units assembled at the site into a series of hollow bottomless cells known as cribs. The cribs are usually filled with noncohesive broken stone or coarse sand and gravel soils since proper drainage of the backfill is essential for their stability.

All retaining walls are expected to withstand the pressure of the earth that they support, but they are not usually designed to resist water pressure in addition to the earth pressure. Therefore retaining walls are provided with means for draining the water that would otherwise accumulate in the backfill.

2. Forces Acting on Retaining Wall

Some of the most common forces acting on the earth retaining structure encountered in practice are:

Weight of the wall; lateral earth pressure; vertical loads on the wall; horizontal loads on the wall; wind pressure; pore water pressure; hydrostatic pressure of water; hydrostatic uplift; hydrodynamic water pressure (wave action upon marine and other hydraulic structure); seepage forces; ice pressure by frozen water in front of wall, or by frozen soil moisture behind the wall; seismic forces such as earthquakes; dynamic impact (impacted by ships on a quay, for example); vibrations induced by machinery, pile driving operation, blasting, mining operations, or military operations; reactions of soil underneath the base of the wall; thermal forces induced by temperature.

In this discussing of the lateral earth pressure theory, only weight, surcharge and soil pressure will be dicussed.

B. Earth Pressure Theory

1. Coulomb's Earth Pressure Theory (1)

The basic assumption for the earth pressure proposed by C. A. Coulomb in 1776 are as following:

1). The soil behind the retaining wall is considered to be dry, homogeneous, isotropic, elastic, undeformable but breakable, granular material, endowed with internal friction but possessing no cohesion and capable of resisting compressive and shear stresses.

2). For the sake of convience in analysis, the rupture surface is assumed to be a plane. The rupture surface actually is not a plane, but is curved. This fact was known to Coulomb.

3). The friction forces are distributed along the plane rupture surface and the coefficient of friction is $f = \tan\phi = \frac{T}{N}$

ϕ : the proper angle of friction

T : the force tangential to the shearing surface

N : the force normal to the shearing surface

4). The failure wedge is a rigid body.

5). There is a wall friction; ie., the failure wedge moves along the back of the wall, [redacted] developing friction forces along the wall boundary.

6). Failure is two-dimensional problem; consider a unit length of an infinitely long body.

A). Active pressure

a. If the wall is a smooth vertical wall with horizontal backfill as shown in Figure 1,

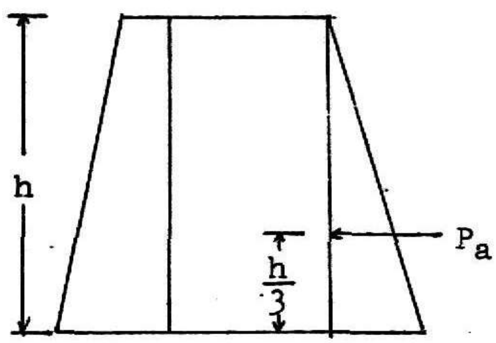


Fig. 1. Retaining wall with vertical back and horizontal backfill

the unit active pressure (p_a) is

$$p_a = \gamma h K_a$$

and the total pressure (P_a) is

$$P_a = \frac{1}{2}\gamma h^2 K_a$$

where γ is the unit weight of soil, h is the height of the wall, and K_a is the coefficient of active earth pressure as defined below:

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \tan^2(45 - \frac{\phi}{2})$$

b. Consider a wall as shown in Figure 2, the backfill is inclined at angle β and back of the wall is inclined at angle α. The rupture surface is assumed to be inclined at angle ρ.

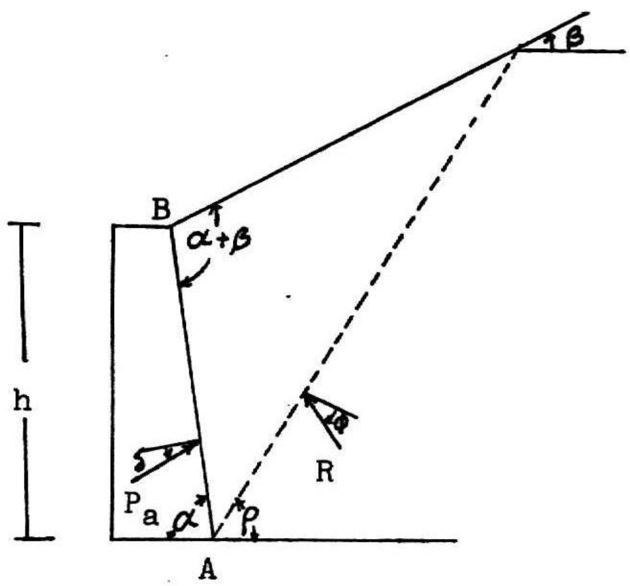


Fig. 2. Retaining wall with inclined wall and sloping surcharge

The unit active pressure is again

$$p_a = \gamma h K_a$$

and the total pressure against the surface AB is

$$P_a = \frac{1}{2}\gamma h^2 K_a$$

but the coefficient K_a in this case is defined as

$$K_a = \frac{\sin^2(\alpha - \beta)}{\sin\alpha \sin(\alpha - \delta)\left[1 + \sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\alpha-\delta)\sin(\alpha+\beta)}}\right]^2}$$

c. Backfill with uniformly distributed surcharge

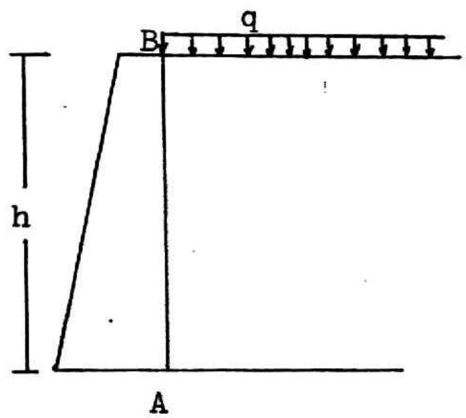


Fig. 3. Retaining wall with vertical back and uniformly distributed surcharge

The surcharge (q) on a horizontal surface of the backfill, as shown in Figure 3, can be converted into units of soil with the same unit weight as the backfill material, and thus the height (h_s) of the equivalent surcharge is $h_s = \frac{q}{\gamma}$, where γ = unit weight of backfill material. Then the unit active pressure at the top of the wall is $p_1 = q \cdot K_a = \gamma h_s K_a$ and the unit pressure at a depth h below the ground surface is

$$p_a = \gamma(h_s + h)K_a$$

The total pressure against the surface AB is seen to be

$$P_a = \gamma(h_s h + \frac{1}{2} h^2)K_a$$

d. Backfill with non-uniform surcharge

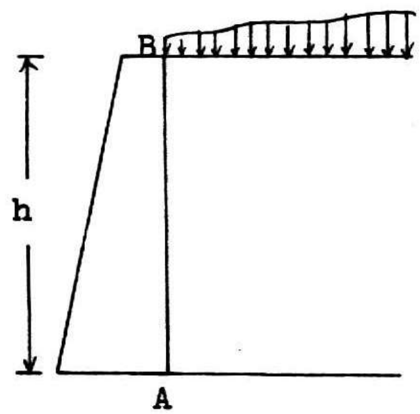


Fig. 4. Retaining wall with vertical back and non-uniform surcharge

If the surcharge on the ground surface is non-uniform as shown in Figure 4, the unit vertical pressure exerted by the surcharge load is found by averaging the surcharge and converting it to the height of the equivalent surcharge as before:

$$h_s = \frac{q}{\gamma}$$

We can then calculate the earth pressure exactly the same as in part c.

B). Passive pressure

a. For a smooth vertical wall with horizontal backfill as shown in Figure 5, using the same symbols as before except for the coefficient of passive earth pressure which is defined below:

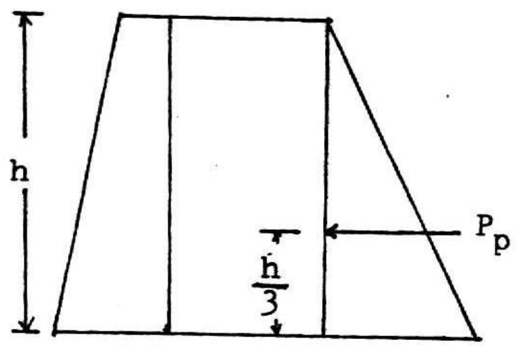


Fig. 5. Retaining wall with vertical wall and horizontal backfill

The unit passive pressure is

$$p_p = \gamma h K_p$$

and the total pressure is

$$P_p = \frac{1}{2}\gamma h^2 K_p$$

where

$$K_p = \frac{1 + \sin\phi}{1 - \sin\phi} = \tan^2(45 + \frac{\phi}{2})$$

b. For the typical wall as shown in Figure 6, the unit passive pressure is again

$$p_p = \gamma h K_p$$

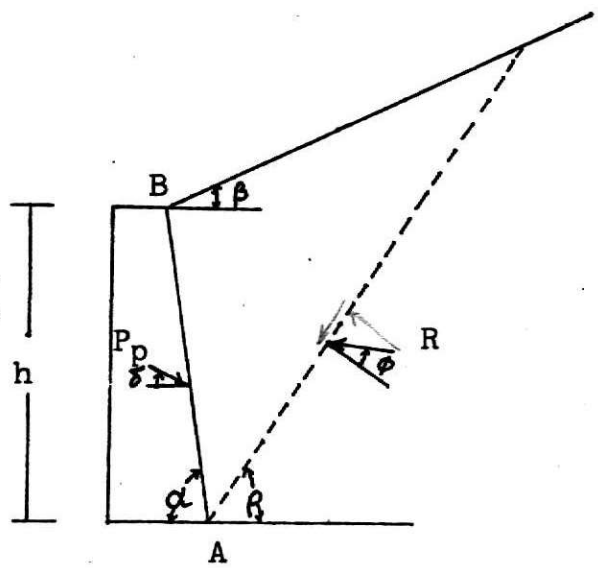


Fig. 6. Retaining wall with inclined back and slpping surcharge

The total pressure against the surface AB is

$$P_p = \frac{1}{2}\gamma h^2 K_p$$

and K_p is defined as

$$K_p = \frac{\sin^2(\alpha - \phi)}{\sin\alpha \sin(\alpha+\delta)\left[1 - \sqrt{\frac{\sin(\phi+\delta)\sin(\phi+\beta)}{\sin(\alpha+\delta)\sin(\alpha+\beta)}}\right]^2}$$

c. Backfill with surcharge

If there is a uniformly or nonuniformly distributed surcharge load on the ground surface, the procedure explained for computing active earth pressure can be adapted to computing passive pressure by using the formulae with K_p replacing K_a.

2. <u>Rankine's Earth Pressure Theory</u>

The basic assumptions underlying Rankine's earth pressure (3) are as following:

1). The soil mass is semiinfinite, homogeneous, dry, and cohesionless.

2). The ground surface is plane which may be horizontal or inclined.

3). The back of the wall is vertical and smooth (no wall friction).

4). The soil mass is in a state of plastic equilibrium.

A). Earth pressure in cohesionless soil

a. Dry backfill with no surcharge

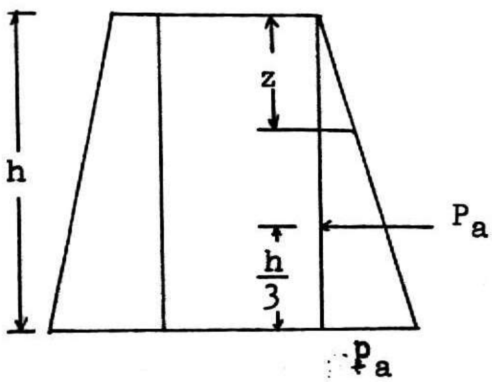


Fig. 7. Retaining wall with vertical wall and horizontal backfill

The intensity of active pressure at the depth of z shown as Figure 7 is

$$p_a = \gamma z K_a$$

where

$$K_a = \tan^2(45 - \frac{\phi}{2})$$

The total earth pressure P_a or the resultant pressure per unit length of wall can be found:

$$P_a = \frac{1}{2}\gamma h^2 K_a$$

The resultant pressure is horizontal and acts at a height of $\frac{H}{3}$ above the base of the wall.

This is the same as the Coulomb theory.

b. Backfill with sloping surcharge

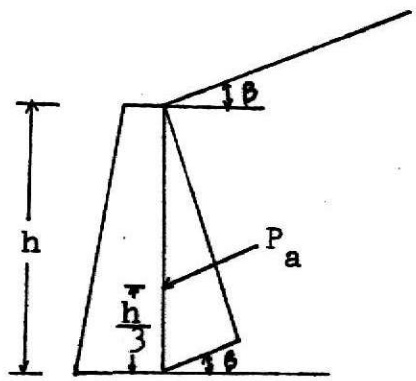


Fig. 8. Retaining wall with vertical back and sloping surcharge

When the top surface of the backfill is inclined to the horizontal as shown in Figure 8. The Rankine coefficient of active pressure is

$$K_a = \cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

The unit active earth pressure at any depth z is given by

$$p_a = \gamma z K_a$$

and total active pressure for a wall of height h is given by

$$P_a = \frac{1}{2}\gamma h^2 K_a$$

The resultant pressure is always parallel to the surcharge line.

c. Submerged backfill

For a retaining wall with the backfill submerged up to an elevation h_1 below the surface as shown in Figure 9.

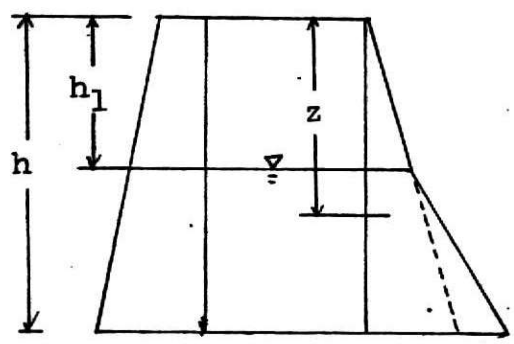


Fig. 9. Retaining wall with vertical back and partly submerged backfill

The active pressure at any depth z, where $z < h_1$, is

$$p_a = \gamma z K_a$$

When $z > h_1$, the total pressure is made up of two components: (1). that due to the submerged weight of soil and (2). that due to the water pressure. Thus

$$p_a = [\gamma h_1 + \gamma'(z - h_1)] K_a + \gamma_w (z - h_1)$$

where $\gamma' = \frac{G - 1}{1 + e} \gamma_w$

When free water stands both on the back and the face of the wall up to the same level, the water pressure need not be considered.

d. Backfill with uniform surcharge

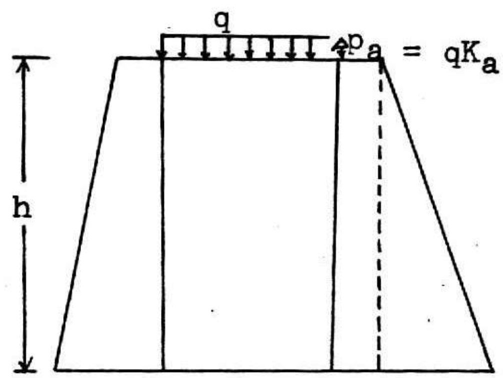


Fig.10. Retaining wall with vertical back and uniformly distributed surcharge

When a backfill carries a uniform distributed load of intensity q per unit area, as shown in Figure 10, the effective vertical pressure at any depth with the backfill increased by q, and therefore, the increase in active earth pressure Δp_a at every point of the back of the wall is given by:

$$\Delta p_a = qK_a$$

(2). Passive earth pressure

a. Dry backfill with no surcharge

The intensity of passive earth pressure at depth z is

$$p_p = \gamma z K_p$$

where

$$K_p = \tan^2(45 + \frac{\phi}{2}) = \frac{1}{K_a}$$

Then the total pressure P_p for a depth h is

$$P_p = \frac{1}{2} \gamma h^2 K_p$$

b. Backfill with sloping surface

When the ground surface is sloping at an angle β to the horizontal, the following expression for the passive earth pressure can be obtained:

$$p_p = \gamma z K_p$$

where

$$K_p = \cos\beta \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}$$

The resultant pressure is always parallel to the surcharge line and given by

$$P_p = \frac{1}{2}\gamma h^2 K_p$$

c. Submerged backfill

When water pressure exists only on the back of the wall up to a depth h_1 below the top, the lateral pressure at depth z, where $z > h_1$, is given by

$$p_p = [\gamma h_1 + \gamma'(z - h_1)] K_p + \gamma_w(z - h_1)$$

d. Backfill with uniform surcharge

When a uniform surcharge of intensity q acts on the ground surface the passive earth pressure is increased at every depth by Δp_p

$$\Delta p_p = qK_p$$

B). Earth pressure in cohesio~~nless~~ve soil

(1). Active earth pressure

a. Dry backfill with no surcharge

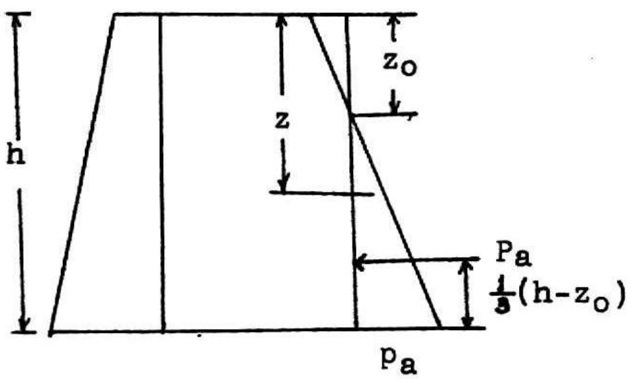


Fig. 11. Retaining wall with vertical back and horizontal backfill

Figure 11 represents a wall of height h with a horizontal backfill. The active earth pressure at any depth z is defined by equation

$$p_a = \gamma z K_a - 2c\sqrt{K_a}$$

where c is cohesion of soil. The resultant force is

$$P_a = \frac{1}{2}\gamma h^2 K_a - 2ch\sqrt{K_a} + \frac{2c^2}{\gamma}$$

The depth z_o at which $p_a = 0$ is given by

$$z_o = \frac{2c}{\gamma}\,\frac{1}{\sqrt{K_a}}$$

b. Backfill with sloping surcharge

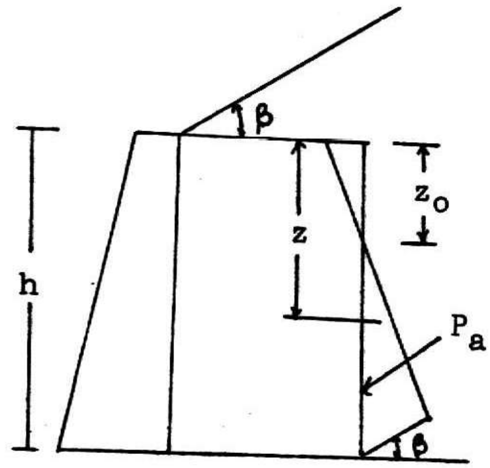


Fig. 12. Retaining wall with vertical back and sloping surcharge

As in Figure 12, the intensity of active pressure at any depth z is

$$p_a = \gamma z K_a - 2c\sqrt{K_a}$$

where

$$K_a = \cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

c. Submerged backfill

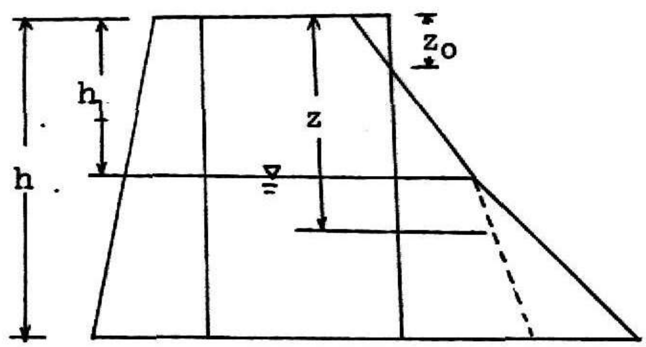


Fig. 13. Retaining wall with vertical back and partly submerged backfill

When water pressure exists only on the back of the wall to a depth h_1 below the top, the lateral pressure at depth z, where $z > h_1$, as shown in Figure 13 is given by

$$p_a = \left[\gamma h_1 + \gamma'(z - h_1)\right] K_a - 2c\sqrt{K_a} + \gamma_w(z - h_1)$$

d. Backfill with uniform surcharge

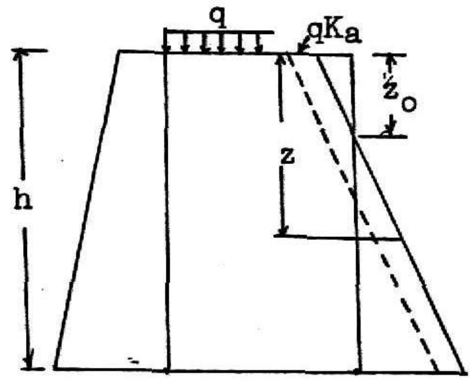


Fig. 14. Retaining wall with vertical back and uniformly distributed surcharge

When the backfill carries a uniform surcharge of intensity q as shown in Figure 14, the active pressure at every elevation on the back increased by $q \cdot K_a$:

$$p_a = \gamma z K_a - 2c\sqrt{K_a} + qK_a$$

The depth z_o at which $p_a = 0$ is given by

$$z_o = \frac{2c}{\gamma} \cdot \frac{1}{\sqrt{K_a}} - \frac{q}{\gamma}$$

(2). Passive earth pressure

a. Dry backfill with no surcharge

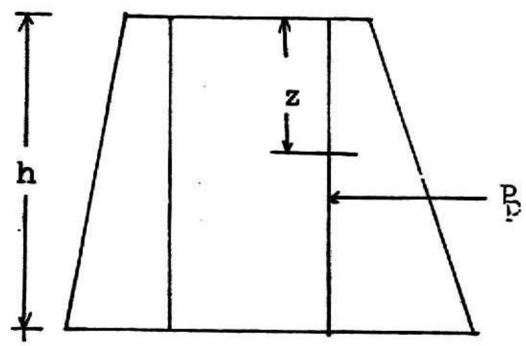


Fig. 15. Retaining wall with vertical back and horizontal backfill.

For a vertical wall with horizontal ground surface as shown in Figure 15. The passive pressure of a cohesive soil at a depth of z is given by

$$p_p = \gamma z K_p + 2c\sqrt{K_p}$$

and the total pressure P_p for a depth h is

$$P_p = \frac{1}{2}\gamma h^2 K_p + 2ch\sqrt{K_p}$$

b. Backfill with sloping surcharge

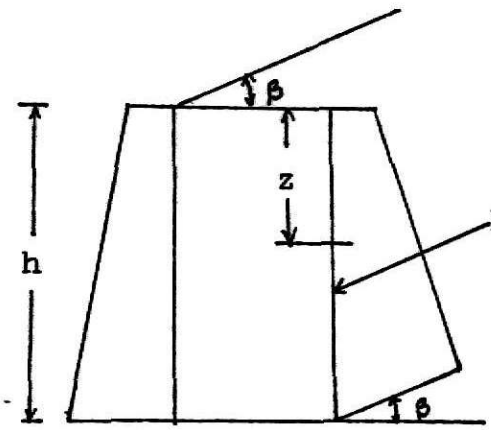


Fig. 16. Retaining wall with vertiacl back and sloping backfill

When the ground surface is sloping to the horizontal surface as shown in Figure 16, the intensity of passive pressure at any depth z is

$$p_p = \gamma z K_p + 2c\sqrt{K_p}$$

where

$$K_p = \cos\beta \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}$$

c. Submerged backfill

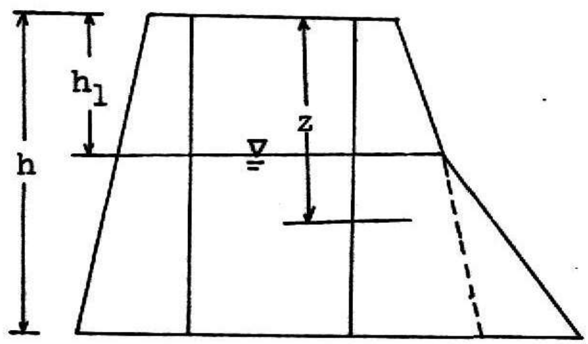


Fig. 17. Retaining wall with vertical back and partly submerged backfill

When water pressure exists only on the back of the wall up to a depth h_1 below the top as Figure 17, the lateral pressure at depth z, where $z > h_1$, is given

$$p_p = [\gamma h_1 + \gamma'(z - h_1)] K_p + 2c\sqrt{K_p} + \gamma_w(z - h_1)$$

d. Backfill with uniform surcharge

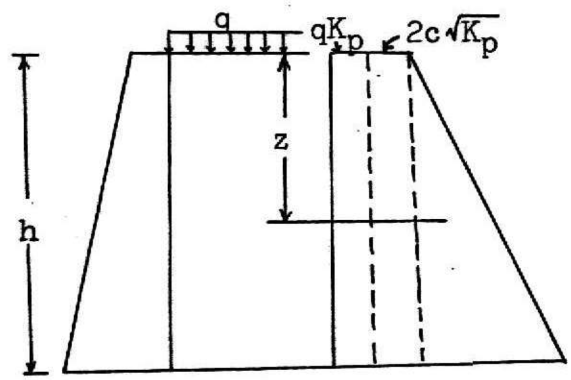


Fig. 18. Retaining wall with vertical back and uniformly distributed surcharge

When a uniform surcharge of intensity q acts on the ground surface as Figure 18, the passive pressure at every elevation on the back increased by $q \cdot K_p$:

$$p_p = \gamma z K_p + 2c\sqrt{K_p} + qK_p$$

C. Stability of Retaining Walls

1. Structural Stability (7, 8, 9)

The stability of a retaining wall should be proportioned to have the following minimum safety factors:

Factor of safety against overturning

= 1.5 (for granular backfill)

= 2.0 (for cohesive backfill)

Factor of safety against sliding

= 1.5 (for all effects of the soil in the front of the wall are neglected)

= 2.0 (for all effects of the soil in the front of the wall are considered)

The lateral pressure due to the backfill and surcharge tends to tip the retaining wall over about its toe. This overturning moment is stabilized by the weight of the wall and the weight of the soil above the base of the wall. In other words, the factor of safety against overturning:

$$SF\ (\text{overturning}) = \frac{M_r}{M_o}$$

where M_r = moments to resist overturning about toe

M_o = moments tending to overturn member about toe

The passive resistance of the soil in front of the wall is commonly neglected in the stability analysis.

The horizontal component of all lateral pressures tends to cause the retaining wall to slide along the base of the wall. It is common practice to omit the soil in the front of the wall in sliding-stability analysis. The sliding force along the bottom of the wall is resisted by a horizontal force which consists of friction, adhesion, or a combination of both.

$$\text{SF (sliding)} = \frac{\text{resisting forces}}{\text{driving forces}} = 1.5 \text{ or } 2.0$$

When this factor of safety is difficult to attain, a key may be constructed under the base slab. The key is generally located under the stem as shown in Figure 19. So that the vertical reinforcing bars may be extended into the key. The addition of a key, in fact, increases passive resistance. The benefit is generally small unless the key is embedded in rock or hard soil.

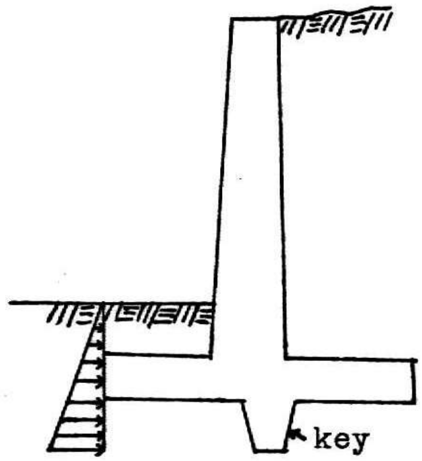


Fig. 19. Stability against sliding using a base key

The usual requirements which insure stability are as follows:

1). The point of application of the resultant force of the weight of the wall, earth pressure, and surcharge (if any) on a earth foundation must fall within the middle third of the base, because soil will not take tensile stresses.

2). The angle of inclination, θ, of the resultant force on the base must not exceed the angle of the friction between the soil and the base of the wall as shown in Figure 20.

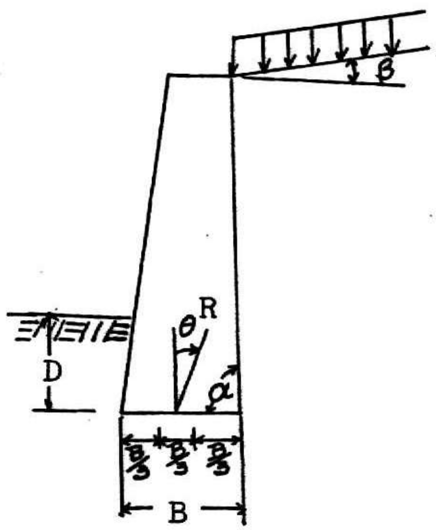


Fig. 20. The point of application of the resultant force

3). The maximum contact pressure exerted by the wall on the soil should not exceed the safe bearing capacity.

4). The driving or active moment about the toe of the wall should be less than the resisting or passive moment.

2. Foundation Stability

A retaining wall must be also proportioned to have sufficient factor of safety against failure of the foundation soil. A satisfactory design should be one in which the shear strength of all foundation soils is sufficient to withstand the shear stresses introduced by the retaining wall and the backfill with an adequate margin of safety.

Hansen and J. Brinch (10) extended Terzaghi bearing pressure formulae to include the effect of the shape and depth of footing as well as the inclination of loads. The ultimate bearing capacity of granular soil is

$$Q_{ult} = \frac{1}{2}\gamma B N_r(1-1.5\frac{H}{V})^2 + \gamma D N_q(1+0.1\frac{D}{B})(1-1.5\frac{H}{V})$$

and the ultimate bearing capacity of cohesive soil is

$$Q_{ult} = 5c(1 + 0.2\frac{D}{B})(1 - 1.3\frac{H}{V}) + \gamma D$$

where Q_{ult} = ultimate bearing capacity

γ = unit weight of soil

B = width of the base as shown in Figure 20

H, V = horizontal and vertical components of the resultant force R

D = depth of the base as shown in Figure 20

N_r, N_q = bearing capacity factors of soil see table 3-1

c = cohesive of soil

ϕ	N_c	N_q	N_r
0	5.7	1.0	0.0
5	7.3	1.6	0.5
10	9.6	2.7	1.2
15	12.9	4.4	2.5
20	17.7	7.4	5.0
25	25.1	12.7	9.7
30	37.2	22.5	19.7
34	52.6	36.5	35.0
35	57.8	41.4	42.4
40	95.7	81.3	100.4
45	172.3	173.3	297.5
48	258.3	287.9	780.1
50	347.5	415.1	1,153.2

Table 3-1 Bearing-capacity factor of soil

The actural bearing pressure is

$$q = \frac{V}{B'}$$

where B' is the useful width of footing, equal to 2ab or 2bc whichever is smaller as shown in Figure 21. Then the factor of safety against bearing failure is

$$SF = \frac{Q_{ult}}{q}$$

must be greater than 2.0 for granular soils and greater than 3.0 for cohesive soils.

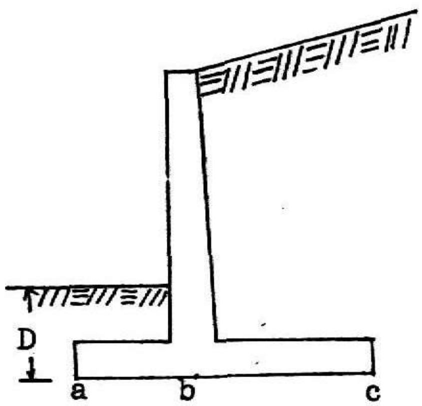


Fig. 21. Stability against bearing failure

The bearing capacity of the foundation may be more accurately determined by first assuming the load V is applied at the center of the base, then multiplying by reduction factor shown in Figure 22 and Figure 23. In Figure 22, e is eccentricity and is given by

$$e = \frac{M_c}{V}$$

where M_c = moments about the centroid of the footing

V = Vertical component of the resultant force R

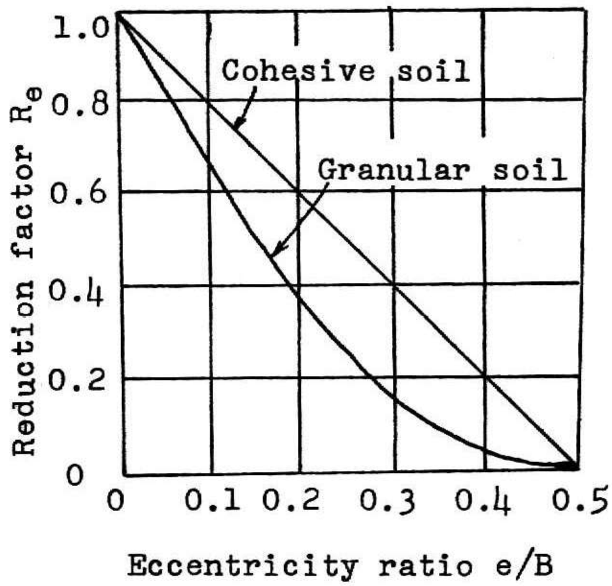


Fig. 22. Bearing capacity of eccentrically loaded footing

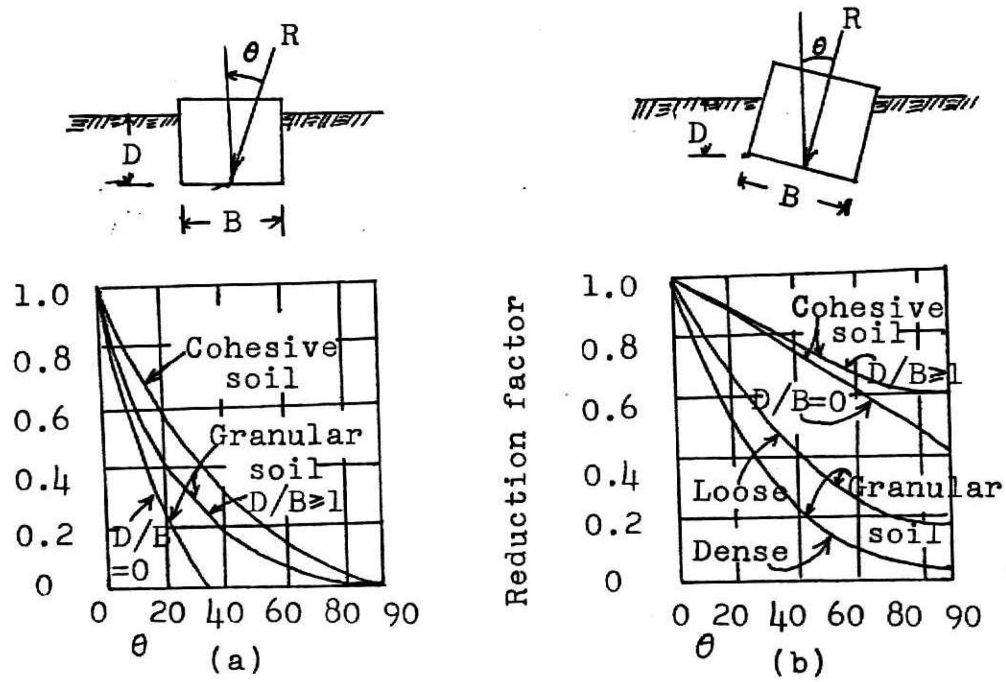


Fig. 23. Bearing capacity of footing subjected to inclined load: (a) horizontal foundation (b) inclined foundation

D. The Backfill and Drainage Systems of Retaining Walls

1. Backfill

According to the AREA Manual (9) "Backfill is defined as all material behind a wall, whether undisturbed ground or fill, that contributes to the pressure against the wall."

Ideal backfill material are purely granular soil - clean sand, gravel, or sand and gravel containing less than about 5 per cent of very fine sand, silt or clay particles. Clean sand and gravel is considered superior to all other soils because they are free draining, are not sensitive to frost action, and do not break down with the passing of time.

Sand backfill not only has the advantage of generating a lower active pressure than cohesive soil; it also results in a more predictable pressure. Furthermore, if the sand is drained by an underdrain or weep holes in the wall, the accumulation of water behind the wall, with its resultant high lateral pressure, will be prevented.

Soils containing a large amount of clay, silt, or organic matter should be avoided because such soils exert excessively large earth pressure. If only such soils are available, they should be carefully compacted by the use of sheepfoot roller so that all the chunks are brocken up and no conspicuous voids are left in place. In silty and clayey soils the design may have to be based on the assumption that the entire backfill is fully saturated because of the tendency of these types of soil to retain water for a long period of time.

Clays are undesirable as backfill because they can hardly be drained, are like to experience alternate swelling and shrinking with the seasons, and may lose much of their shearing strength if moisture accumulates. If shrinkage cracks in a clay backfill become filled with rainwater, the wall may be subjected to full hydrostatic pressure as well as the earth pressure, even if drains have been provided. Whenever possible, it is considered good practice to insert a wedge-shaped body of free draining material between the wall and a clay backfill.

The table below rates the soils of the Unified Classification (11) for selection as backfill.

GW,SW,GP	Excellent, free-draining backfill.
GM,GC,SM,SC	Good if kept dry but require good drainage. May be subjected to some frost action.
ML	Satisfactory if kept dry but requires good drainage. Subject to frost. Neglect any cohesion in design.

CL,MH,OL	Poor. Must keep dry. Subject to frost. Wall deflection likely to be large and progressive unless at-rest pressure is used.
CH,OH	Should not be used for backfill because of swelling.
Pt	Should not be used.

Artificial material such as cider and crushed slag often make good backfill. All the cohesionless backfills are best when well compacted because the higher internal friction angle and the resistance to vibration offset the higher weight.

Backfill material should be compacted to prevent large ground subsidence due to consolidation under its own weight. The amount of compaction required for each job depends on the material used, and on the nature of the job. More strict control of compaction is necessary where the backfill is cohesive soil and where the ground settlement is objectable.

2. Drainage Systems

The pressure of water in a retaining fill increase the earth thrust in an uncertain but considerable amount. For this reason means must be provided for the removal of any water that may collect in the fill behind the wall.

The primary function of a drainage system behind a retaining wall is to prevent water from accumulating in the backfill and coming in contact with the back of the wall and thereby subjecting the wall to hydrostatic pressure. The ease of drainage depends upon the permeability of the backfill materials:

a). In pervious backfill, weep holes or a line of drain pipe will suffice.

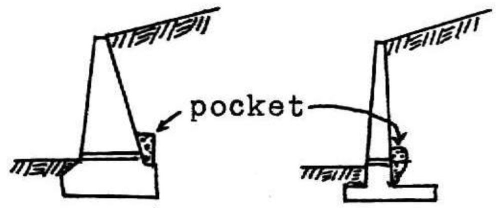
b). Semipervious backfills (soils containing a small amount of fine sand, silt, or clay particles) require strips of filter material in addition to the drain pipes or weep holes.

c). In fine-grained backfill, a drainage blanket or double blankets are necessary.

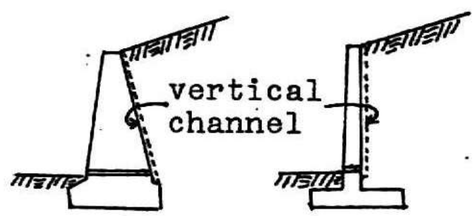
The types of drainage system are as following (5, 7, 8):

a). The simplest system consist of weep holes or

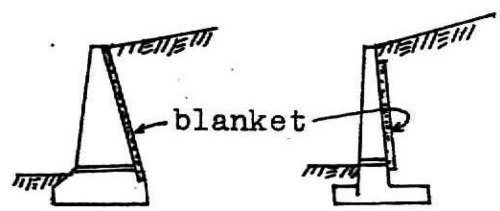
(a). Weep Holes only



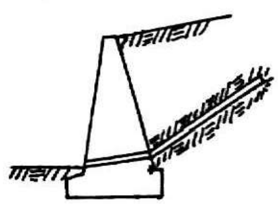
(b). Weep Holes with Pocket Drains



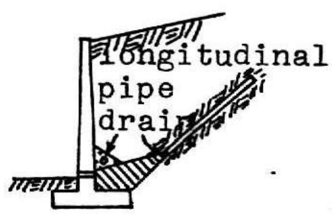
(c). Channel Drains



(d). Continuous Blanket Drains



(e). Inclined Drain



(f). Prevents Ice Lenses

(g). Expansive Backfill

Fig. 24. Backfill drain for retaining walls

weepers through the wall, located above the ground level in front of the wall as shown in (a) of Figure 24. The weep holes are usually formed by vitified clay pipes 4 in. or more in diameter. They are spaced from 5 to 15 ft laterally. Usually only one row is provided just above the ground surface in front of the wall, but occasionally one or more additional rows are installed at higher elevations. Such a system with one row of weepers is suitable when the backfill consists of a permeable material which drains readily.

b). A deposit or pocket of coarse material may be placed around the end of each weep hole, as illustrated in (b), to facilitate the drainage of a backfill which has a considerable degree of permeability. These may be called pocket drains.

c). This system consists of a continuous horizontal channel of coarse material along the inner ends of weep holes as shown in (c). These may be supplemented by vertical channels of coarse material about 1 ft square in horizontal cross section placed midway between the weep holes and with the lower ends joining the horizontal channel. The upper ends of the channel should be covered with relatively impervious material to prevent surface water from entering the drainage system directly. These may be called channel drains.

d). As in (d), the entire back of a retaining wall may be covered with a blanket drain about 1 ft thick with its lower joining the inner ends of weep holes to increase the effectiveness of a drainage system. The upper edge of the blanket should be covered with relatively impervious materials.

e). As in (e), the inclined drain is the most effective type that can be installed because it not only deposes of the water which seeps into the backfill, but reduces or eliminates increases in earth pressures which are due to seepage.

f). The drain shown in (f) avoids pressures due to ice layers which may form in the backfill adjecent to the back of a wall.

g). The system as shown in (g) minimizes the change in moisture content in the backfill and thereby prevents failures from expending due to the increase of their moisture content in cohesive backfills.

E. Numerical Examples

Example 1

Given: A wall system as shown in Figure 25

Find: Lateral forces by using Coulomb and Rankine equations

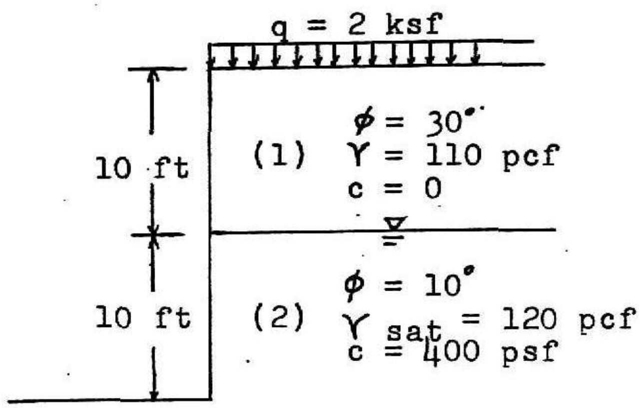


Fig. 25. Ref. Example 1

1. Coulomb analysis

A). Active pressure

$$K_{a1} = \tan^2(45 - \frac{30}{2})$$

$$= 0.333$$

$$K_{a2} = \tan^2(45 - \frac{10}{2})$$

$$= 0.705$$

$$\sqrt{K_{a2}} = 0.84$$

The equivalent surcharge (γ = 110 pcf) is

$$h_s = \frac{q}{\gamma} = \frac{2}{0.11} = 18.2 \text{ ft}$$

(a). at h = 0

$$p_1 = \gamma h_s K_{a1}$$
$$= 0.11 \times 18.2 \times 0.333$$
$$= 0.666 \text{ ksf}$$

(b). at h = 10 ft

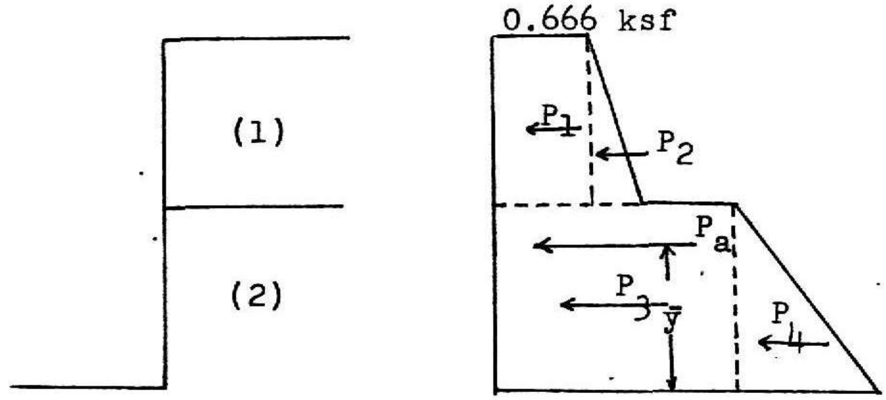
$$p_2 = \gamma h K_{a1} + 0.666$$
$$= 0.11 \times 10 \times 0.333 + 0.666$$
$$= 0.366 + 0.666$$
$$= 1.032 \text{ ksf}$$

(c). at h = 10 ft + dh

$$p_2' = \gamma(h_s + h)K_{a2} - 2c\sqrt{K_{a2}}$$
$$= 0.11 \times (18.2 + 10) \times 0.705 - 2 \times 0.4 \times 0.84$$
$$= 2.18 - 0.672$$
$$= 1.508 \text{ ksf}$$

(d). at h = 20 ft

$$p_3 = 1.508 + \gamma' h K_{a2} + \gamma_w h$$
$$= 1.508 + (0.12 - 0.0624) \times 10 \times 0.705$$
$$+ 0.624 \times 10$$
$$= 1.508 + 0.42 + 0.624$$
$$= 2.538 \text{ ksf}$$



$P_1 = 0.666 \times 10 = 6.66$ kips

$P_2 = 0.5 \times 0.366 \times 10 = 1.83$ kips

$P_3 = 1.508 \times 10 = 15.08$ kips

$P_4 = 0.5 \times 1.03 \times 10 = 5.15$ kips

$P_a = 6.66 + 1.83 + 15.08 + 5.15$

$= 28.72$ kips

$\bar{y}$ above base

$$28.72 \times \bar{y} = 6.66 \times 15 + 1.83 \times \left(10 + \frac{10}{3}\right) + 15.08 \times 5 + 5.15 \times \frac{10}{3}$$

$$= 100 + 24.4 + 75.4 + 17.2$$

$$= 217$$

$$\bar{y} = \frac{217}{28.67} = 7.56 \text{ ft}$$

B). Passive pressure

$$K_{p1} = \tan^2(45 + \frac{30}{2})$$
$$= 3$$
$$K_{p2} = \tan^2(45 + \frac{10}{2})$$
$$= 1.42$$
$$\sqrt{K_{p2}} = 1.19$$
$$h_s = 18.2 \text{ ft } (\gamma = 110 \text{ pcf})$$

(a). at $h = 0$

$$p_1 = \gamma h_s K_{p1}$$
$$= 0.11 \times 18.2 \times 3$$
$$= 6 \text{ ksf}$$

(b). at $h = 10$ ft

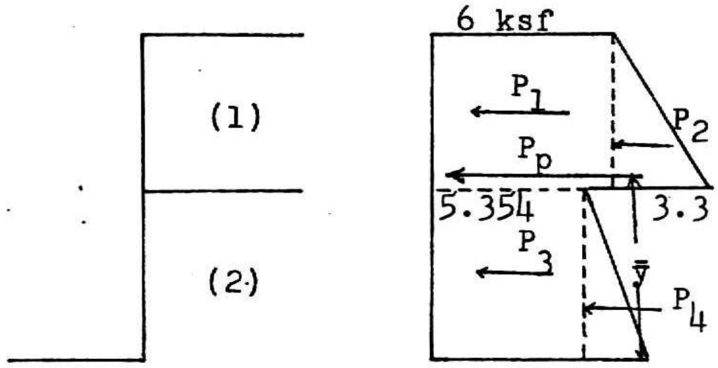
$$p_2 = (h_s + h)\gamma K_{p1}$$
$$= (18.2 + 10) \times 0.11 \times 3$$
$$= 9.3 \text{ ksf}$$

(c). at $h = 10$ ft + dh

$$p_2' = (h_s + h)\gamma K_{p2} + 2c\sqrt{K_{p2}}$$
$$= (18.2 + 10) \times 0.11 \times 1.42$$
$$+ 2 \times 0.4 \times 1.19$$
$$= 4.4 + 0.952$$
$$= 5.352 \text{ ksf}$$

(d). at h = 20 ft

$$p_3 = 5.352 + \gamma' h K_{p2} + \gamma_w h$$
$$= 5.352 + (0.12 - 0.0624) \times 10 \times 1.42$$
$$+ 0.624 \times 10$$
$$= 5.352 + 0.82 + 0.624$$
$$= 6.796 \text{ ksf}$$



$$P_1 = 6 \times 10 = 60 \text{ kips}$$
$$P_2 = 0.5 \times 3.3 \times 10 = 16.5 \text{ kips}$$
$$P_3 = 5.352 \times 10 = 53.52 \text{ kips}$$
$$P_4 = 0.5 \times 1.444 \times 10 = 7.22 \text{ kips}$$
$$P_p = 60 + 16.5 + 53.52 + 7.22$$
$$= 137.24 \text{ kips}$$

$\bar{y}$ above base

$$137.24 \times \bar{y} = 60 \times 15 + 16.5 \times (10 + \frac{10}{3})$$
$$+ 53.52 \times 5 + 0.5 \times 7.22 \times \frac{10}{3}$$

$= 900 + 220 + 267.6 + 12.05$

$= 1399.65$

$\bar{y} = 10.2$ ft

2. Rankine analysis

A). Active pressure

$K_{a1} = 0.333$

$K_{a2} = 0.705, \sqrt{K_{a2}} = 0.84$

(a). at $h = 0$

$$p_1 = q \times K_{a1}$$
$$= 2 \times 0.333$$
$$= 0.666 \text{ ksf}$$

(b). at $h = 10$ ft

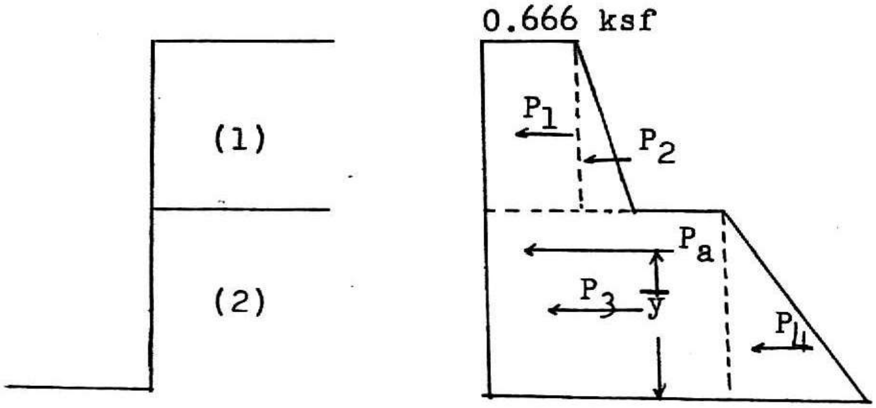
$$p_2 = \gamma h K_{a1} + 0.666$$
$$= 0.11 \times 10 \times 0.333 + 0.666$$
$$= 1.032 \text{ ksf}$$

(c). at $h = 10$ ft + dh

$$p_2' = (q + \gamma h) \times K_{a2} - 2c\sqrt{K_{a2}}$$
$$= (2 + 0.11 \times 10) \times 0.705 - 2 \times 0.4 \times 0.84$$
$$= 2.18 - 0.672$$
$$= 1.508 \text{ ksf}$$

(d). at h = 20 ft

$$p_3 = 1.508 + \gamma' h K_{a2} + \gamma_w h$$
$$= 1.508 + (0.12 - 0.624) \times 10 \times 0.705$$
$$+ 0.0624 \times 10$$
$$= 1.508 + 0.406 + 0.624$$
$$= 2.538 \text{ ksf}$$



$P_1 = 0.666 \times 10 = 6.66$ kips

$P_2 = 0.5 \times 0.366 \times 10 = 1.83$ kips

$P_3 = 1.508 \times 10 = 15.08$ kips

$P_4 = 0.5 \times 1.03 \times 10 = 5.15$ kips

$P_a = 6.66 + 1.83 + 15.08 + 5.15 = 28.72$ kips

$\bar{y}$ above base

$$28.72 \times \bar{y} = 6.66 \times 15 + 1.83 \times (10 + \frac{10}{3})$$
$$+ 15.08 \times 5 + 5.15 \times \frac{10}{3}$$
$$= 100 + 24.4 + 75.4 + 17.2$$
$$= 217$$

$$\bar{y} = 7.56 \text{ ft}$$

B). Passive pressure

$K_{p1} = 3.0$

$K_{p2} = 1.42, \sqrt{K_{p2}} = 1.19$

(a). at h = 0

$$p_1 = q \times K_{p1}$$
$$= 2 \times 3$$
$$= 6 \text{ ksf}$$

(b). at h = 10 ft

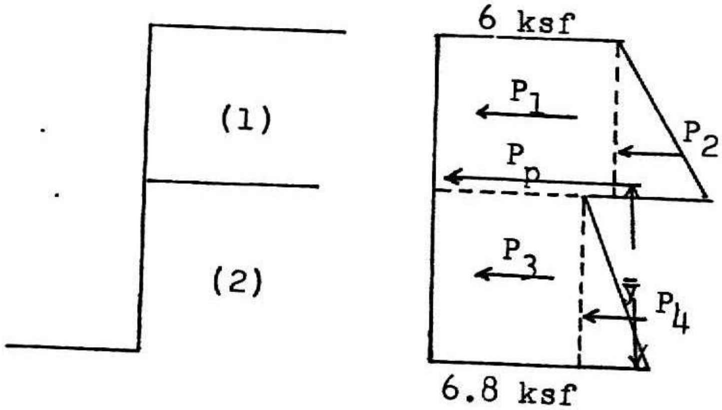
$$p_2 = qK_{p1} + \gamma hK_{p1}$$
$$= 6 + 0.11 \times 10 \times 3$$
$$= 9.3 \text{ ksf}$$

(c). at h = 10 ft + dh

$$p_2' = (q + \gamma h)K_{p2} + 2c\sqrt{K_{p2}}$$
$$= (2 + 0.11) \times 10 \times 1.42 + 2 \times 0.4 \times 1.19$$
$$= 2.84 + 1.56 + 0.952$$
$$= 5.352 \text{ ksf}$$

(d). at h = 20 ft

$$p_3 = 5.352 + \gamma' hK_{p2} + \gamma_w h$$
$$= 5.352 + (0.12 - 0.0624) \times 10 \times 1.42$$
$$+ 0.0624 \times 10$$
$$= 5.352 + 0.82 + 0.624$$
$$= 6.796 \text{ ksf}$$



$$P_1 = 6 \times 10 = 60 \text{ kips}$$

$$P_2 = 0.5 \times 3.3 \times 10 = 16.5 \text{ kips}$$

$$P_3 = 5.352 \times 10 = 53.52 \text{ kips}$$

$$P_4 = 0.5 \times 1.444 \times 10 = 7.22 \text{ kips}$$

$$P_p = 60 + 16.5 + 53.52 + 7.22 = 137.27 \text{ kips}$$

$\bar{y}$ above base

$$137.27 \times \bar{y} = 60 \times 15 + 16.5 \times (10 + \frac{10}{3}) + 53.52 \times 5 + 0.5 \times 7.22 \times \frac{10}{3}$$

$$= 1399.65$$

$$\bar{y} = 10.2 \text{ ft}$$

It is shown by these analyses that the values obtained are the same for the Coulomb and Rankine analysis.

Example 2

Given: A wall system as shown in Figure 26

Find: Lateral forces

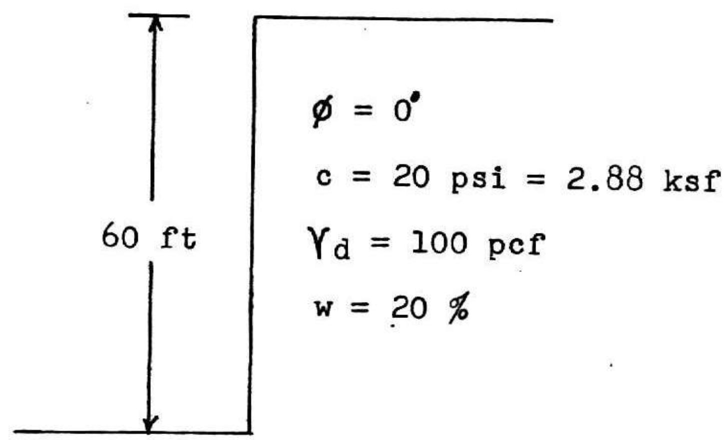


Fig. 26. Ref. Example 2

Solution:

$$\gamma = (1 + w)\gamma_d$$
$$= (1 + 0.2) \times 0.1$$
$$= 0.12 \text{ kcf}$$

1. Active pressure

$$K_a = \tan^2(45 - 0)$$
$$= 1$$
$$\sqrt{K_a} = 1$$

(a). at $h = 0$

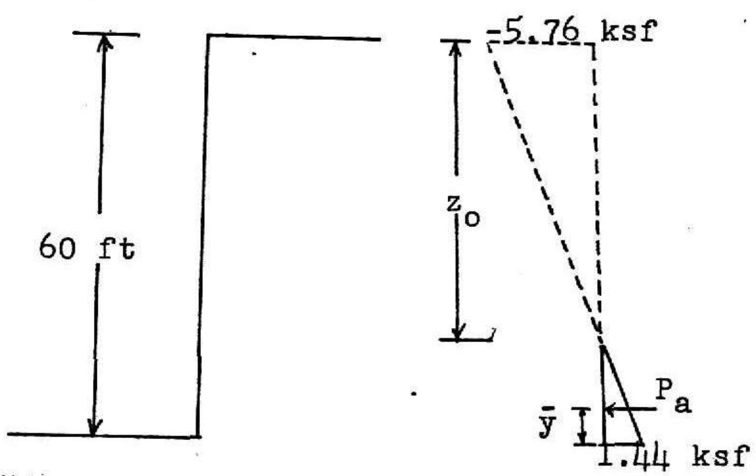
$$p_a = -2c\sqrt{K_a}$$
$$= -2 \times 2.88 \times 1$$
$$= -5.76 \text{ ksf}$$

(b). at h = 60 ft

$$p_a = \gamma h K_a - 2c\sqrt{K_a}$$
$$= 0.12 \times 60 \times 1 - 2 \times 2.88 \times 1$$
$$= 7.2 - 5.76$$
$$= 1.44 \text{ ksf}$$

The depth z_o at which $p_a = 0$ is

$$z_o = \frac{2c}{\gamma}\frac{1}{\sqrt{K_a}} = \frac{2 \times 2.88}{0.12} \times 1 = 48 \text{ ft}$$



$$P_a = 0.5 \times 1.44 \times (60 - 48)$$
$$= 8.64 \text{ kips}$$

$\bar{y}$ above base

$$\bar{y} = \frac{1}{3} \times 12 = 4 \text{ ft}$$

B. Passive pressure

$$K_p = \frac{1}{Ka} = 1$$

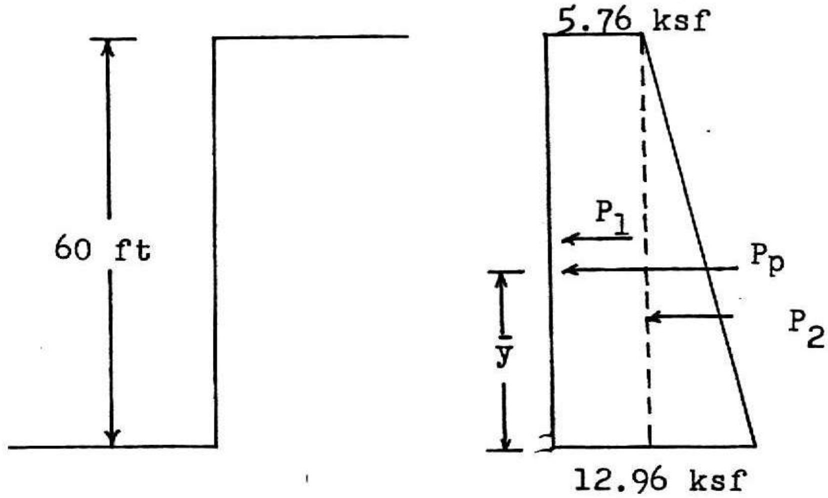
$$\sqrt{K_p} = 1$$

(a). at h = 0

$$p_p = 2c\sqrt{K_p}$$
$$= 2 \times 2.88 \times 1$$
$$= 5.76 \text{ ksf}$$

(b). at h = 60 ft

$$p_p = \gamma h K_p + 2c\sqrt{K_p}$$
$$= 0.12 \times 60 \times 1 + 2 \times 2.88 \times 1$$
$$= 7.2 + 5.66$$
$$= 12.96 \text{ ksf}$$



$P_1 = 5.76 \times 60 = 345.6$ kips

$P_2 = 0.5 \times (12.96 - 5.76) \times 60 = 216$ kips

$P_p = 345.6 + 216 = 561.6$ kips

$\bar{y}$ above base

$$561.6 \times \bar{y} = 345.6 \times 30 + 216 \times 20$$

$$= 10368 + 4320$$

$$= 14688$$

$$\bar{y} = \frac{14688}{561.6}$$

$$= 28.43 \text{ ft}$$

It is recognized that in most natural clays, K_a can vary from 1 to 5 due to the swelling forces of the clay and therefore the above analysis is sometimes unreliable.

Example 3

Given: A wall system as shown in Figure 27

Find: Lateral forces

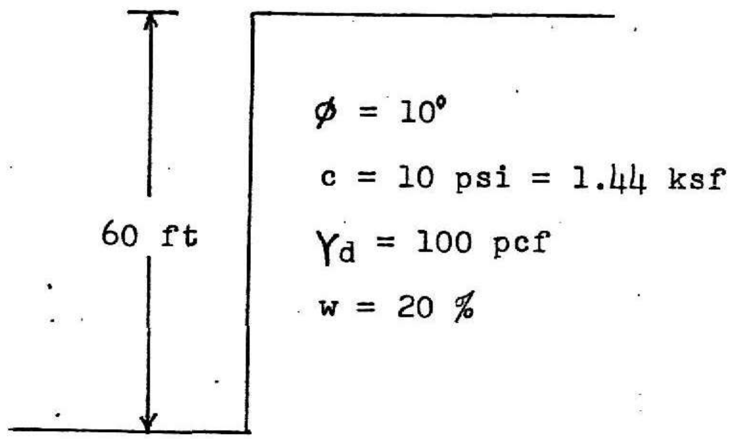


Fig. 27. Ref. Example 3

Solution:

$$\gamma = (1 + 0.2) \times 0.1$$
$$= 0.12 \text{ kcf}$$

1. Active pressure

$$K_a = \tan^2(45 - \frac{10}{2}), \qquad \sqrt{K_a} = 0.84$$
$$= 0.7$$

(a). at h = 0

$$p_a = -2c\sqrt{K_a}$$
$$= -2c \times 1.44 \times 0.84$$
$$= -2.41 \text{ ksf}$$

(b). at h = 60 ft

$$p_a = \gamma h K_a - 2c\sqrt{K_a}$$

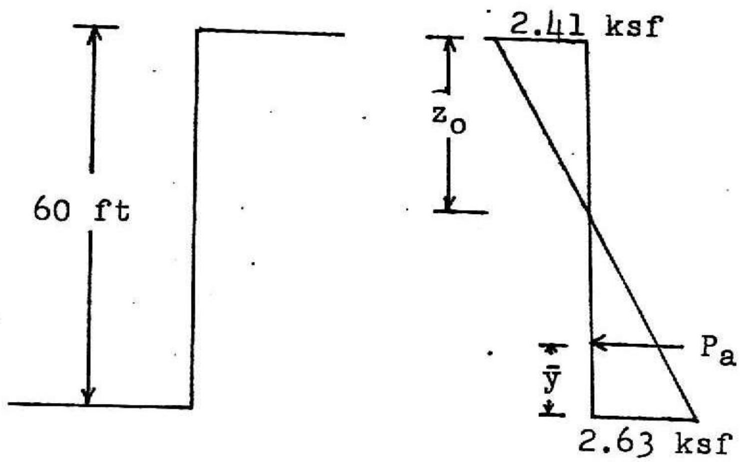
$$= 0.12 \times 60 \times 0.7 - 2.41$$

$$= 5.04 - 2.41$$

$$= 2.63 \text{ ksf}$$

The depth z_o at which $p_a = 0$ is

$$z_o = \frac{2c}{\gamma}\frac{1}{\sqrt{K_a}} = \frac{2 \times 1.44}{0.12} \times \frac{1}{0.84} = 28.6 \text{ ft}$$



$$P_a = 0.5 \times 2.63 \times (60 - 28.6)$$

$$= 412.91 \text{ kips}$$

$\bar{y}$ above base

$$\bar{y} = \frac{1}{3} \times 31.4 = 10.46 \text{ ft}$$

2. Passive pressure

$$K_p = \frac{1}{Ka} = 1.43$$

$$\sqrt{K_p} = 1.195$$

(a). at h = 0

$$p_p = 2c\sqrt{K_p}$$

$$= 2 \times 1.44 \times 1.195$$

$$= 3.44 \text{ ksf}$$

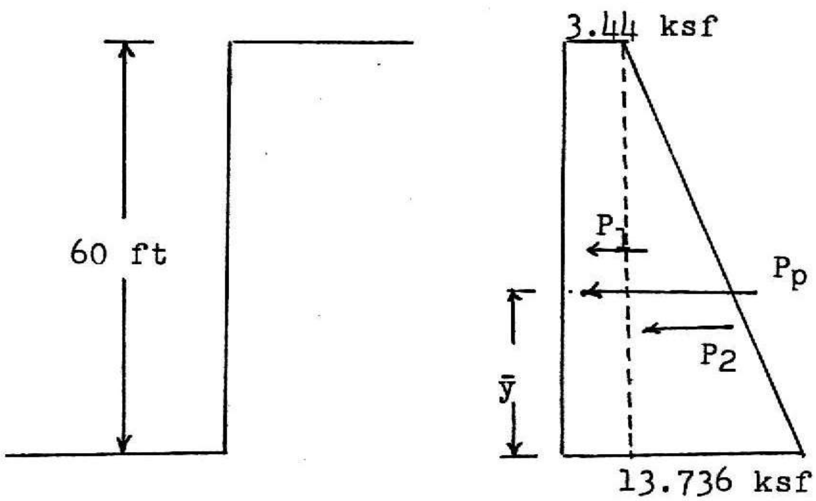
(b). at h = 60 ft

$$p_p = \gamma h K_p + 2c\sqrt{K_p}$$

$$= 0.12 \times 60 \times 1.43 + 2 \times 1.44 \times 1.195$$

$$= 10.296 + 3.44$$

$$= 13.736 \text{ ksf}$$



$P_1 = 3.44 \times 60 = 206.4$ kips

$P_2 = 0.5 \times (13.736 - 3.44) \times 60 = 308.88$ kips

$P_p = 206.4 + 308.88 = 515.28$ kips

$\bar{y}$ above base

$$515.28 = 206.4 \times 30 + 308.88 \times 20$$

$$= 6192 + 6177.6$$

$$= 12369.6$$

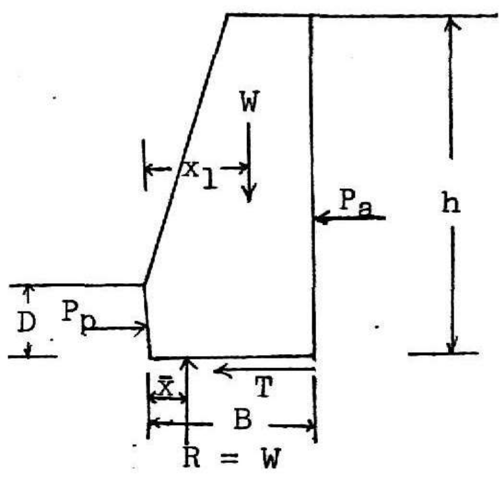
$$\bar{y} = \frac{12369.6}{515.28}$$

$$= 24 \text{ ft}$$

Example 4

Given: The forces acting on the simple mass concrete gravity wall shown in Figure 28

Find: Check safety factors



$W = 700$ kips

$P_a = 260$ kips

$P_p = 145$ kips

$D = 5$ ft

$B = 10$ ft

$\phi = 30°$

$h = 20$ ft

$T = 405$ kips

$x_1 = 6.2$ ft

Fig. 28. Ref Example 4

1. Factor of safety against sliding

$$S.F. \text{ (sliding)} = \frac{\text{resisting forces}}{\text{driving forces}}$$

$$= \frac{T + P_p}{P_a}$$

$$= \frac{405 + 145}{260}$$

$$= 2.12$$

The Code suggests that this factor of safety should be not less than 2.0.

2. Factor of safety against overturning

If the wall overturns about the toe, the reaction R may be assumed to act approximately through A. The factor of safety against overturning will be defined by

$$S.F. \text{ (overturning)} = \frac{M_r}{M_o}$$

$$= \frac{700 \times 6.2 + 145 \times \frac{5}{3}}{260 \times \frac{20}{3}}$$

$$= \frac{4340 + 241}{1730}$$

$$= 2.65$$

The Code suggests that this factor of safety should be not less than 2.0.

3. Pressure beneath the toe

The line of action R may be determined by equating moments about A.

$$R\bar{x} = 700 \times 6.2 + 145 \times \frac{5}{3} - 260 \times \frac{20}{3}$$
$$= 4340 + 242 - 1730$$
$$= 2852$$
$$\bar{x} = 4.07 \text{ ft}$$

Then the eccentricity of R from the centre of the foundation is

$$5 - 4.07 = 0.93$$

The maximum pressure under the toe of foundation is

$$p_{max.} = \frac{R}{B}\left(1 + \frac{6e}{B}\right)$$
$$= \frac{700}{100}\left(1 + \frac{10 \times 0.93}{10}\right)$$
$$= 70 \times 1.558$$
$$= 109 \text{ ksf}$$

The Code suggests that the maximum pressure should be not exceed 0.5 times ultimate bearing capacity.

III. CONCLUSIONS

1. In general, Coulomb's theory applies to walls with backs which are plane or can be assumed to be plane, such as the common types of gravity and semi-gravity walls; Rankine's theory applies to walls whose backs are not plane such as common types of cantilever and counterfort walls.

2. The Rankine solution is commonly used for the design of retaining walls up to about 20 ft in height because it neglects wall friction. For walls over 20 ft in height, or where wall friction is considered, the Coulomb solution should be performed to determine the lateral pressure.

3. The presence of cohesion in the soil seems to advantageously concerning proportioning of the wall and calculations of stability in the case of active as well as passive earth pressure; it decreases the magnitude of the active earth pressure and increases the passive resistance. This is known to be in great error in soils that are subject to swelling with increasing moisture.

4. In a c-ϕ soil the active pressure, p_a, will have a positive value as long as:

$$\gamma z K_a \geqslant 2c\sqrt{K_a}$$

$$z \geqslant \frac{2c}{\gamma} \cdot \frac{1}{\sqrt{Ka}}$$

5. A c-ϕ soil is able to support itself with a vertical bank to a certain height:

$$H_o = \frac{H_{cr}}{2} = \frac{2c}{\gamma} \cdot \frac{1}{\sqrt{K_a}}$$

6. Apart from the effects of water on the cohesive properties of a soil, there will also be a decrease in active pressure below the water table, since the submerged density of the soil is used:

$$p_a = K_a \gamma' z$$

Hower, the total pressure on the back of the wall will increase due to the water pressure.

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AN INTRODUCTION TO THE DESIGN OF RETAINING WALLS

by

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ABSTRACT

The object of this report is to present some general principals for the design of the retaining walls.

The first part indicates the types of the retaining wall according to their geometrical design and function.

Theoretical methods for computing the earth pressure against retaining walls are presented in part two. These methods are based on Coulomb's and Rankine's theories.

In part three, some formulae and specifications which insure the stability of retaining walls have been explained A retaining wall must be proportioned to have sufficient factor of safety against the failure of structure as well as foundation soil.

The fourth part presents the types of drainage system behind a retaining wall is to prevent water from accumulating in the backfill and coming in contact with the back of the wall and thereby subjecting the wall to hydrostatic pressure. The type of material used for backfill has important effects on the magnitude of the lateral earth pressure against a wall.